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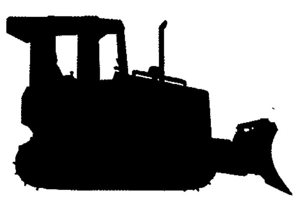
P.E. Review Series

Construction Engineering for Civil Engineering License



By

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New Civil Engr Construction Module for the Civil PE Exam

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How to Use This Book

This book was written to help you prepare for the PE-Civil. The following is a suggested strategy preparation for the exam:

1- Have all references organized before you start. Also, familiarize yourself with the comprehensive summary provided in this book.
2- Read the questions and **ALL** answers carefully and look for **KEY WORDS** in the question and the 4 possible answers.
3- Solve the easy questions first (ones that need **no** or **minimum** calculations) and record your answers on the answer sheet.
4- Work on questions that require lengthy calculations and record your answers on the answer sheet.
5- Questions that seem difficult or not familiar to you and may need considerable time in searching in your references should be left to the end
6- Never leave an answer blank
7- Remember the **D**^{3} rule:

DO NOT EXPECT THE EXAM TO BE EASY
DO NOT PANIC
DO NOT WASTE TOO MUCH TIME ON A SIGNLE EASY, DIFFICULT, OR UNFAMILAR PROBLEM

About the Author

Dr. Shahin A. Mansour, PE, has been teaching PE, Seismic, Surveying and EIT courses for the last 22 years. Dr. Mansour taught Civil Engineering Courses for 7 years at New Mexico State University, Las Cruces, NM. Also, he has been teaching Civil and Construction Engineering Courses (graduate & undergraduate) at CSU, Fresno, CA, for the last 18 years.

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Principles and Practice of Engineering

Civil BREADTH Exam Specifications
Effective Beginning with the April 2008 Examinations

Approximate Percentage of Examination

I. CONSTRUCTION **20%**

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  1. Excavation and embankment (cut and fill)
  2. Borrow pit volumes
  3. Site layout and control
- B. Estimating Quantities and Costs
  1. Quantity take-off methods
  2. Cost estimating
- C. Scheduling
  1. Construction sequencing
  2. Resource scheduling
  3. Time-cost trade-off
- D. Material Quality Control and Production
  1. Material testing (e.g., concrete, soil, asphalt)
- E. Temporary Structures
  1. Construction loads

II. GEOTECHNICAL **20%**

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  1. Soil classification
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- B. Engineering Properties of Soils and Materials
  1. Permeability
  2. Pavement design criteria
- C. Soil Mechanics Analysis
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  2. Lateral earth pressure
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- D. Earth Structures
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- E. Shallow Foundations
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1. Determinate analysis

C. Mechanics of Materials
1. Shear diagrams
2. Moment diagrams
3. Flexure
4. Shear
5. Tension
6. Compression
7. Combined stresses
8. Deflection

D. Materials
1. Concrete (plain, reinforced)
2. Structural steel (structural, light gage, reinforcing)

E. Member Design
1. Beams
2. Slabs
3. Footings

IV. TRANSPORTATION **20%**

A. Geometric Design
1. Horizontal curves
2. Vertical curves
3. Sight distance
4. Superelevation
5. Vertical and/or horizontal clearances
6. Acceleration and deceleration

V. WATER RESOURCES AND ENVIRONMENTAL **20%**

A. Hydraulics – Closed Conduit
1. Energy and/or continuity equation (e.g., Bernoulli)
2. Pressure conduit (e.g., single pipe, force mains)
3. Closed pipe flow equations including Hazen-Williams, Darcy-Weisbach Equation
4. Friction and/or minor losses
5. Pipe network analysis (e.g., pipeline design, branch networks, loop networks)
6. Pump application and analysis

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2. Storm frequency

3. Hydrographs application
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5. Time of concentration
6. Runoff analysis including Rational and SCS methods
7. Erosion
8. Detention/retention ponds

D. Wastewater Treatment
1. Collection systems (e.g., lift stations, sewer networks, infiltration, inflow)

E. Water Treatment
1. Hydraulic loading
2. Distribution systems

Total	**100%**

Notes

1. The examination is developed with questions that will require a variety of approaches and methodologies including design, analysis, and application. Some questions may require knowledge of engineering economics.
2. The knowledge areas specified under 1, 2, 3, etc., are examples of kinds of knowledge, but they are not exclusive or exhaustive categories.
3. The breadth (AM) exam contains 40 multiple-choice questions. Examinee works all questions.
4. Score results are combined with depth exam results for final score

Civil—CONSTRUCTION Depth Exam Specifications
Effective Beginning with the April 2008 Examinations

	Approximate Percentage of Examination
I. Earthwork Construction and Layout	**10%**
A. Excavation and embankment (cut and fill)	
B. Borrow pit volumes	
C. Site layout and control	
D. Earthwork mass diagrams	
II. Estimating Quantities and Costs	**17.5%**
A. Quantity take-off methods	
B. Cost estimating	
C. Engineering economics	
1. Value engineering and costing	
III. Construction Operations and Methods	**15%**
A. Lifting and rigging	
B. Crane selection, erection, and stability	
C. Dewatering and pumping	
D. Equipment production	
E. Productivity analysis and improvement	
F. Temporary erosion control	
IV. Scheduling	**17.5%**
A. Construction sequencing	
B. CPM network analysis	
C. Activity time analysis	
D. Resource scheduling	
E. Time-cost trade-off	
V. Material Quality Control and Production	**10%**
A. Material testing (e.g., concrete, soil, asphalt)	
B. Welding and bolting testing	
C. Quality control process (QA/QC)	
D. Concrete mix design	
VI. Temporary Structures	**12.5%**
A. Construction loads	
B. Formwork	
C. Falsework and scaffolding	
D. Shoring and reshoring	
E. Concrete maturity and early strength evaluation	
F. Bracing	
G. Anchorage	

H. Cofferdams (systems for temporary excavation support)
I. Codes and standards [e.g., American Society of Civil Engineers (ASCE 37), American Concrete Institute (ACI 347), American Forest and Paper Association-NDS, Masonry Wall Bracing Standard]

VII. Worker Health, Safety, and Environment **7.5%**

A. OSHA regulations
B. Safety management
C. Safety statistics (e.g., incident rate, EMR)

VIII. Other Topics **10%**

A. Groundwater and well fields
    1. Groundwater control including drainage, construction dewatering
B. Subsurface exploration and sampling
    1. Drilling and sampling procedures
C. Earth retaining structures
    1. Mechanically stabilized earth wall
    2. Soil and rock anchors
D. Deep foundations
    1. Pile load test
    2. Pile installation
E. Loadings
    1. Wind loads
    2. Snow loads
    3. Load paths
F. Mechanics of materials
    1. Progressive collapse
G. Materials
    1. Concrete (prestressed, post-tensioned)
    2. Timber
H. Traffic safety
    1. Work zone safety

Total **100%**

Notes

1. The examination is developed with problems that will require a variety of approaches and methodologies including design, analysis, and application. Some problems may require knowledge of engineering economics.

2. The knowledge areas specified under A, B, C, etc., are examples of kinds of knowledge, but they are not exclusive or exhaustive categories.

3. Each depth (PM) exam contains 40 multiple-choice questions. Examinee chooses **one** depth examination and works all questions in the depth examination chosen.

4. Score results are combined with breadth exam results for final score.

NCEES CONSTRUCTION Design Standards
Effective Beginning with the April 2008 Examinations

ABBREVIATION	DESIGN STANDARD TITLE
1- ASCE 37-02	Design Loads on Structures During Construction, 2002, American Society of Civil Engineers, Reston, VA, www.asce.org.
2- NDS	National Design Specification for Wood Construction, 2005, American Forest & Paper Association/American Wood Council, Washington, DC, www.awc.org.
3- CMWB	Standard Practice for Bracing Masonry Walls During Construction, 2001, Council for Masonry Wall Bracing, Mason Contractors Association of America, Lombard, IL, www.masoncontractors.org.
4- AISC	Steel Construction Manual, 13th ed. (or 9th edition ASD, or 3rd edition LRFD), American Institute of Steel Construction, Inc., Chicago, IL, www.aisc.org.
5- ACI 347-04	Guide to Formwork for Concrete, 2004, American Concrete Institute, Farmington Hills, MI, www.concrete.org, (in ACI SP-4, 7th edition appendix).
6- ACI SP-4	Formwork for Concrete, 7th ed., 2005, American Concrete Institute, Farmington Hills, MI, www.concrete.org.
7- OSHA	Occupational Safety and Health Standards for the Construction Industry, 29 CFR Part 1926, (US federal version), US Department of Labor, Washington, DC.
8- MUTCD-Pt 6	Manual on Uniform Traffic Control Devices – Part 6 Temporary Traffic Control, 2003, US Federal Highway Administration, www.fhwa.dot.gov.

Reference categories for **Construction** depth module

- Construction surveying
- Construction estimating
- Construction planning and scheduling
- Construction equipment and methods
- Construction materials
- Construction design standards (see above)

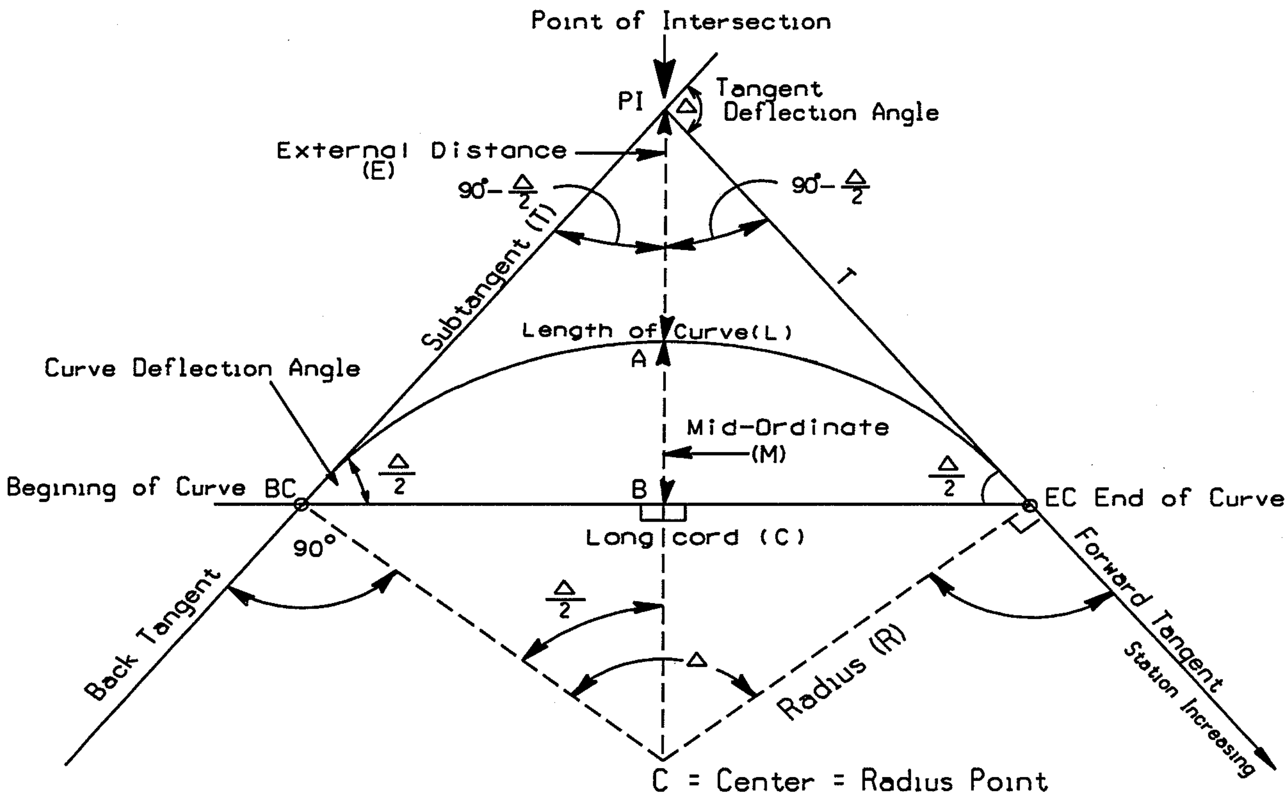
A COMPREHENSIVE SUMMARY OF EQUATIONS & TOPICS OF CONSTRUCTION ENGINEERING

Codes Equations, Non-Code Equations, Tables, Flow Charts, Figures, At a Glance Steps

Per NCEES List of References

I. Earthwork Construction and Layout 10% (4 questions)

GEOMETRY OF HORIZONTAL CIRCULAR CURVES



$$T = R \tan\frac{\Delta}{2} \tag{1}$$

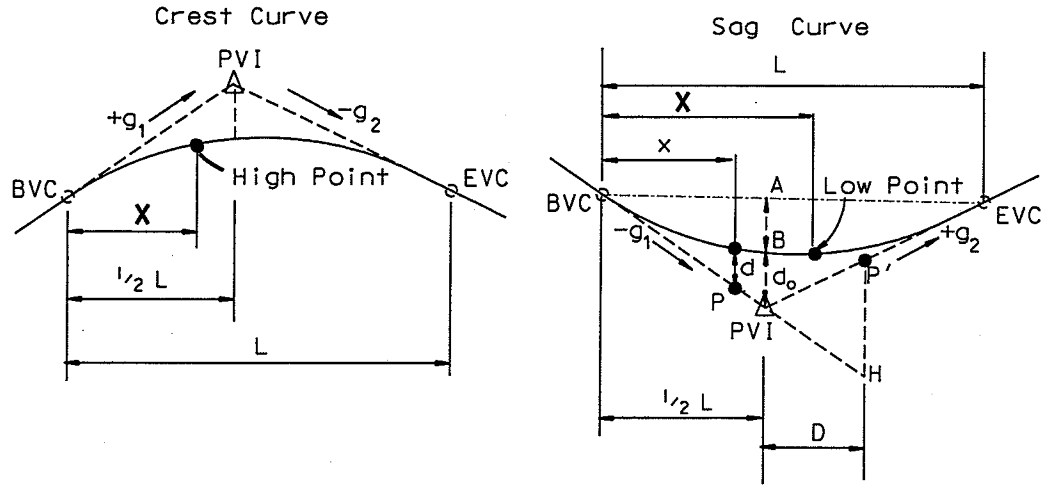
$$C = 2\ R\ Sin\frac{\Delta}{2} = 2\,T\cos(\Delta/2) \tag{2}$$

$$L = 2\pi\ R\frac{\Delta^{\circ}}{360^{\circ}} = R\,\Delta\,(radians) = (100\,ft)\left(\frac{\Delta}{D}\right) \tag{3}$$

$$M = R\,(1-\cos\frac{\Delta}{2}) = \frac{C}{2}\tan\frac{\Delta}{4} = E\,Cos\frac{\Delta}{2} \tag{4}$$

$$E = R\left[\frac{1}{Cos(\Delta/2)}-1\right] = R\,(Sec\frac{\Delta}{2}-1) = T\tan\frac{\Delta}{4} = R\tan\frac{\Delta}{2}\tan\frac{\Delta}{4} \tag{5}$$

GEOMETRY OF PARABOLIC VERTICAL CURVES



$$y_x = y_{BVC} + g_1 x + \frac{rx^2}{2} \tag{6}$$

$$r = \frac{g_2 - g_1}{L} \tag{7}$$

Where: x = distance from BVC to a point on the curve

r = rate of grade change per station

$$d_o = \frac{|g_1 - g_2|L}{8} \tag{8}$$

The slope S, in percentage, of the tangent to the curve at any point on the curve is given by the following formula:

$$S = g_1 - \frac{x(g_1 - g_2)}{L} \tag{9}$$

The distance D in feet from Vertex to P' is given as:

$$D = \frac{100(Y_H - Y_{P'})}{(g_1 - g_2)} \tag{10}$$

The distance between the curve and the grade line (tangent) "d" is given as:

$$d = offset = \frac{rx^2}{2} = \frac{x^2(g_2 - g_1)}{200L} \quad \text{(L = curve length in feet)} \tag{11}$$

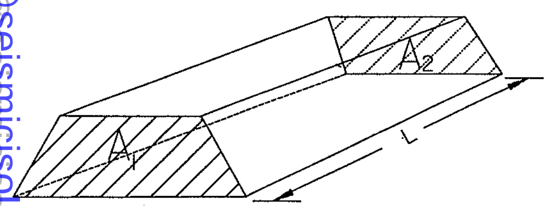
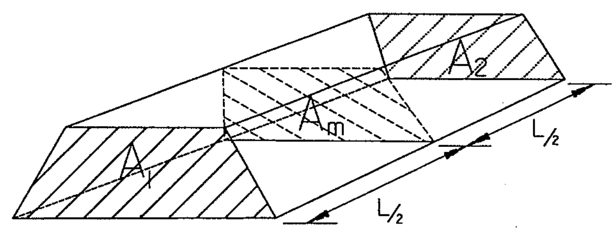
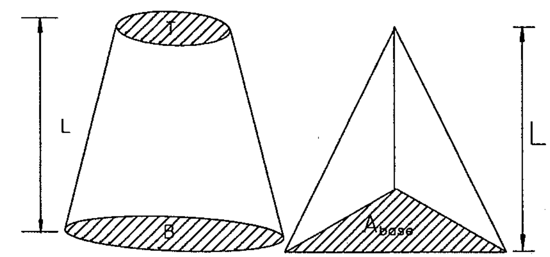
Low and high points are determined as follows:

$$X = \frac{-g_1}{r} = \frac{-g_1 L}{g_2 - g_1} = \frac{g_1 L}{g_1 - g_2} \tag{12}$$

GENERAL EQUATION FOR VOLUME COMPUTATION

$$\text{Volume} = \text{Cross Sectional Area} \times \text{Length} \tag{13}$$

Cross-Section Methods of Volume Measurement

Average end-area formula	Prismoidal formula	Pyramid formula
➢ It gives approximate and larger (overestimates) volumes. ➢ It ignores the slopes and the orientation of the ends and sides. ➢ It favors contractors. ➢ Increased accuracy is obtained by decreasing the distance "L" between cross sections.	➢ It gives more accurate and less volumes than the average-end area. ➢ It applies to all volumes of all geometrics solids than can be considered prismoids. ➢ If favors the owner. ➢ The difference between the volumes obtained by the average -end-area and the prismoidal formula is called the <u>*prismoidal correction.*</u>	➢ It is used when one end area is small (about 5 % of the other end area) or zero. ➢ When one end area is <u>not zero</u> <u>truncated pyramid</u> will produce more accurate volumes.
 $V = \frac{A_1 + A_2}{2} \times \frac{L}{27} \ (yd^3)$ (14) $A_1 \& A_2 =$ end areas in square feet $L =$ distance between end areas in feet $V = \frac{A_1 + A_2}{2} \times L \ (m^3)$ (15) $A_1 \& A_2 =$ end areas in square meters $L =$ distance between end areas in meters.	 $V = (A_1 + 4A_m + A_2)\frac{L}{6 \times 27}(yd^3)$ (16) $A_1 \& A_2 =$ end areas in square feet $A_m =$ area of a "computed" section midway (halfway) between $A_1 \& A_2$ $L =$ horizontal distance between end areas in feet.	 $V_{pyramid} = \frac{A_{base} L}{3 \times 27}(yd^3)$ (17) $A_{base} =$ area of the base $L =$ distance between the two ends. $V_{trunc.pyramid} = \frac{L}{3 \times 27}(T + B + \sqrt{TB})(yd^3)$ (18) $T =$ top area in square feet $B =$ bottom area in square feet $L =$ distance between the two ends in feet

VOLUME COMPUTATION BY BORROW PIT METHOD

Borrow Pit Method At a Glance

A- Before Excavating:

1. Construct a rectangular grid with a regular uniform intervals over the area containing the borrow pit. If a full square grid may not be achievable, other figure areas (e.g., a triangle, a rectangle) may be used.
2. A control point (horizontal and vertical control) is set up in a safe location (so it will not be disturbed) nearby the borrow pit area.
3. Determine the horizontal and vertical positions of the intersecting points of the grid with respect to the reference (control) point i.e. creating a surface of the original ground before excavation.

B- After Excavating:

1. A similar survey is required where the same interesting grid points are located horizontally and vertically.
2. This will create a surface of the sides and bottom of the borrow bit (surface after excavation).

C- Volume Computation:

1. The difference between the two surfaces is the volume in cubic yards or meters.

$$Volume = \frac{\sum(depth\ at\ the\ corners\ of\ the\ grid)}{number\ of\ the\ corners} \times area\ of\ grid \qquad (19)$$

LIMIT OF ECONOMIC OVERAHAUL (L.E.H.)

Under certain conditions, it may pay to borrow or waste. In other words, the overhaul distance is profitable only to a limit.

Let:

C_E = cost to excavate 1 unit volume (includes free haul)

C_{EW} = cost to excavate and waste 1 unit volume

C_B = cost to borrow 1 unit volume

C_{OH} = cost for overhaul per station unit

F = free-haul distance, stations

$L.E.H.$ = limit of economic haul, stations

The cost to excavate and waste plus the cost to borrow equals the cost of excavation plus the cost of overhaul:

$$C_{EW} + C_B = C_{OH}\ (L.E.H. - F) + C_E \tag{20}$$

Assume that

$$C_{EW} = C_E$$

Then,

$$L.E.H. = \frac{C_B}{C_{OH}} + F \tag{21}$$

SWELL & SHRINKAGE FACTORS, COMPACTED & BANK VOLUMES

Swell factor		Shrinkage factor	
$= \dfrac{Loose\ unit\ weight\ (Luw)}{Bank\ unit\ weight\ (Buw)}$	(22)	$= \dfrac{Bank\ unit\ weight\ (Buw)}{Compacted\ unit\ weight\ (Cuw)}$	(24)
$= \dfrac{1}{(1+Swell)}$	(23)	$= \dfrac{1}{(1-Shrinkage)}$	(25)
$Compacted\ Volume(CCY) = Bank\ Volume(BCY) \times Shrinkage\ Factor$			(26)
$Bank\ Volume\ (BCY) = Loose\ Volume\ (LCY) \times Swell\ Factor$			(27)

The following figure shows the swell and shrinkage factors in relation to the condition of the soil.

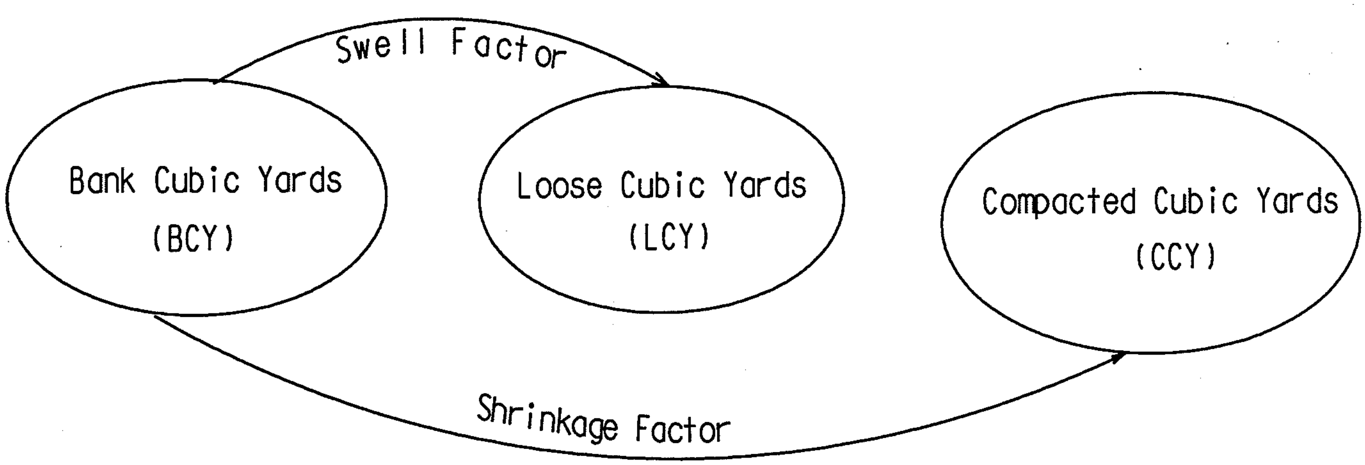


Figure 1-1 Shrinkage and Swell Factors

II. Estimating Quantities and Costs 17.5% (7 questions)

LABOR COST

Work hours = Quantity takeoff × productivity rate (28)

Adjusted work hours = Work hours × productivity factor (29)

2.3 ENGINEERING ECONOMICS

Factor Name	Converts	Symbol	Formula
Single Payment Compound Amount	to F given P	(F/P, i%, n)	$(1+i)^n$ (30)
Single Payment Present Worth	to P given F	(P/F, i%, n)	$(1+i)^{-n}$ (31)
Uniform Series Sinking Fund	to A given F	(A/F, i%, n)	$\frac{i}{(1+i)^n-1}$ (32)
Capital Recovery	to A given P	(A/P, i%, n)	$\frac{i(1+i)^n}{(1+i)^n-1}$ (33)
Uniform Series Compound Amount	to F given A	(F/A, i%, n)	$\frac{(1+i)^n-1}{i}$ (34)
Uniform Series Present Worth	to P given A	(P/A, i%, n)	$\frac{(1+i)^n-1}{i(1+i)^n}$ (35)
Uniform Gradient Present Worth	to P given G	(P/G, i%, n)	$\frac{(1+i)^n-1}{i^2(1+i)^n}-\frac{n}{i(1+i)^n}$ (36)
Uniform Gradient Future Worth*	to F given G	(F/G, i%, n)	$\frac{(1+i)^n-1}{i^2}-\frac{n}{i}$ (37)
Uniform Gradient Uniform Series	to A given G	(A/G, i%, n)	$\frac{1}{i}-\frac{n}{(1+i)^n-1}$ (38)

* $F/G = (F/A) \times (A/G)$ (39)

NOMENCLATURE AND DEFINITIONS:

A .... Uniform amount per interest period
B .... Benefit
BV........ Book value
C.......... Cost
d........... Combined interest rate per interest period
D_j .... Depreciation in year j
F.......... Future worth, value, or amount
f .... General inflation rate per interest period
G.......... Uniform gradient amount per interest period
i .... Interest rate per interest period
i_e .... Annual effective interest rate
m.......... Number of compounding periods per year
n........... Number of compounding periods; or the expected life of an asset
P.......... Present worth, value, or amount
r........... Nominal annual interest rate
S_n .... Expected salvage value in year n

Subscripts:

j .... at time j
n........... at time n

NON-ANNUAL COMPOUNDING:

$$i_e = \left(1 + \frac{r}{m}\right)^m - 1 \tag{40}$$

BREAK-EVEN ANALYSIS:

By altering the value of any one of the variables in a situation, holding all of the other values constant, it is possible to find a value for that variable that makes the two alternatives equally economical. This value is the break-even point.

Break-even analysis is used to describe the percentage of capacity of operation for a manufacturing plant at which income will just cover expenses.

The payback period is the period of time required for the profit or other benefits of an investment to equal the cost of the investment.

INFLATION:

To account for inflation, the dollars are deflated by the general inflation rate per interest period f, and then they are shifted over the time scale using the interest rate per interest period i. Use a combined interest rate per interest period d for computing present worth values P and Net P. The formula for d is:

$$d = i + f + (i \times f) \tag{41}$$

DEPRECIATION:

Depreciation is the decline in market value of the piece of equipment due wear and tear and age of the equipment. Any depreciation calculation must correspond to the IRS guideline for equipment life, which is five years. It should be noted that the cost of tires (for wheeled equipment) is subtracted from the purchase price due to different years of life between equipment and tires.

1- Straight Line:

The straight line method of depreciation provides uniform annual depreciation for each year of equipment life. The annual depreciation is calculated as follows:

$$D_j = \frac{C - S_n}{n} \tag{42}$$

2- Accelerated Cost Recovery System (ACRS):

$$D_j = (factor)\ C \tag{43}$$

A table of modified factors is provided below.

MODIFIED ACRS FACTORS				
	Recovery Period (Years)			
	3	5	7	10
Year	Recovery Rate (Percent)			
1	33.3	20.0	14.3	10.0
2	44.5	32.0	24.5	18.0
3	14.8	19.2	17.5	14.4
4	7.4	11.5	12.5	11.5
5		11.5	8.9	**9.2**
6		5.8	8.9	7.4
7			8.9	6.6
8			4.5	6.6
9				6.5
10				6.5
11				3.3

3- Sum of the Years Digits (SOYD)

The sum of the years digits method (SOYD) of depreciation provides a nonuniform depreciation, with higher amounts in the first years, decreasing in amount through the fifth year. The calculation involves multiplying the digit for the year in inverse order times the amount to depreciate, and then dividing by the sum of the years' digits (1 + 2 + 3 + 4 + 5 = 15 for a five-year life).

$$D_j = \frac{n+1-j}{\sum_{j=1}^{n} j}(C-S_n) \tag{44}$$

BOOK VALUE :

$$BV = initial\ cost - \Sigma D_j \tag{45}$$

TAXATION:

Income taxes are paid at a specific rate on taxable income. Taxable income is total income less depreciation and ordinary expenses. Expenses do not include capital items, which should be depreciated.

CAPITALIZED COSTS:

Capitalized costs are present worth values using an assumed perpetual period of time.

$$\text{Capitalized Costs} = P = \frac{A}{i} \tag{46}$$

BONDS:

Bond Value equals the present worth of the payments the purchaser (or holder of the bond) receives during the life of the bond at some interest rate i. Bond Yield equals the computed interest rate of the bond value when compared with the bond cost.

RATE-OF-RETURN:

The minimum acceptable rate-of-return (MARR) is that interest rate that one is willing to accept, or the rate one desires to earn on investments. The rate-of-return on an investment is the interest rate that makes the benefits and costs equal.

BENEFIT-COST ANALYSIS:

In a benefit-cost analysis, the benefits B of a project should exceed the estimated costs C.

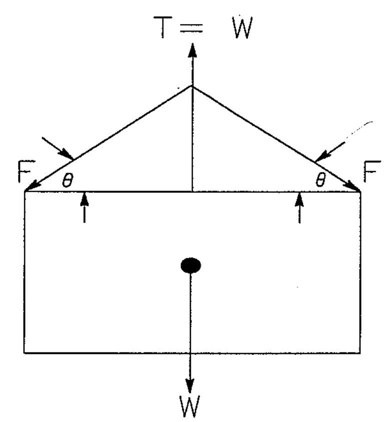
$$B - C \geq 0,\ or\ B/C \geq 1 \tag{47}$$

III. Construction Operations and Methods 15% (6 questions)

$$W_1 \times x_1 = W_2 \times x_2 \qquad (48)$$

The general formula is given by the following equation:

$$F = \frac{T}{2\ Sin\ \theta} = \frac{W}{2\ Sin\ \theta} \qquad (49)$$



Where:

W = weight of the load
T = tension in the cable
F = force in the sling
θ = horizontal angle of the sling

Productivity Analysis and Improvement

Loaders:

Production (LCY) per hour = Bucket payload (LCY) per cycle × Cycles per hour (50)

Production Cycle Time = $\text{load}_t + \text{travel}_t + \text{dump}_t + \text{return}_t$ (51)

Dozers:

The following formula calculates a dozer pushing production in loose cubic yards (LCY) per a 60 – minute

$$\text{Production (LCY/hr)} = \frac{60\,\text{min} \times blade\ \ load}{push\ time\ (\text{min}) + return\ time\ (\text{min}) + maneuver\ time\ (\text{min})} \qquad (52)$$

The production of a dozer could also be calculated using the ***rule-of-thumb formula*** proposed by equipment manufacturer International Harvester (IH)

$$\text{Production (LCY per 60-min hr)} = \frac{net\,hp\ \times 330}{D+50} \qquad (53)$$

Where:

net hp = net horsepower at the flywheel for a power-shift crawler dozer
D = one-way push distance, in feet

Excavators:

$$\text{Production} = \frac{3{,}600 \text{ sec} \times Q \times F \times (AS:D)}{t} \times \frac{E}{60-\min hr} \times \frac{1}{volume\ correction} \quad (54)$$

Where:

Q = heaped bucket capacity, lcy
F = bucket fill factor
$AS{:}D$ = angle of swing and depth (height) of cut
t = cycle time in seconds
E = efficiency (minutes worked per hour)

$$\text{Volume correction (convert LCY to BCY)} = \frac{1}{1+swell\ \ factor}$$

For backhoe, the production equation is given as:

Production (backhoe - excavation) =

$$= \frac{3{,}600 \text{ sec} \times Q \times F}{t} \times \frac{E}{60-\min hr} \times \frac{1}{volume\ correction} \quad (55)$$

Where:

Q = heaped bucket capacity, lcy
F = bucket fill factor
t = cycle time in seconds
E = efficiency (minutes worked per hour)

$$\text{Volume correction (convert LCY to BCY)} = \frac{1}{1+swell\ \ factor}$$

Hauling Equipment (Scrapers and Trucks):

I- Scrapers:

$$\text{Production Cycle Time} = \text{load}_t + \text{haul}_t + \text{dump}_t + \text{turn}_t + \text{return}_t + \text{turn}_t \quad (56)$$

II- Trucks:

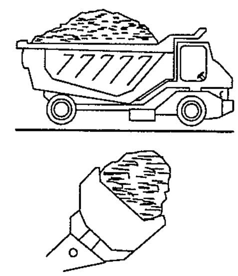
The following equation calculates the required number of trucks for hauling operation:

$$Number\ of\ Trucks = \frac{Truck\ Cycle\ Time}{Load\ Time} \quad (57)$$

Where:

Load Time = Truck Capacity/ Loader Production at 100% Efficiency, or
Load Time = Number of Bucket loads × Loader Cycle Time

$$\text{Number of bucket loads} = \frac{Truck\ \ Capacity\,(LCY)}{Bucket\ Capacity\,(LCY)} = \quad (58)$$



Finishing Equipment (Graders):

I- Spreading Productivity of Graders:

$$\text{Production (BCY) per hr} = 3.0 \times hp \times efficiency \quad (59)$$

Where:

hp = flywheel horsepower
$efficiency$ = 1.0 for 50-min working hour

II- Productivity Time for Graders: The total time required to complete a grader operation is given by the following formula:

$$Total\ Time = \frac{P \times D}{S \times E} \quad (60)$$

Where:

P = number of passes required
D = distance traveled in each pass, miles or feet
S = speed of grader, in mph or feet per minute (fpm)
E = grader efficiency factor

III- Fine Grading Productivity for Graders: Graders are used for finishing and fine grading to the surface layer on construction projects. The surface area in square foot or yard will be used to calculate the productivity as follows:

$$\text{Production (square yard per hr)} = \frac{5{,}280 \times S \times W \times E}{N \times 9\, sf\,/\,sy} \quad (61)$$

Where:

S = speed of grader, in mph or feet per minute (fpm)
W = effective width per grader pass, in feet
E = grader efficiency factor
N = number of passes over the same area

IV. Scheduling
17.5% (7 questions)

Program Evaluation Review Technique (PERT)

PERT is a system of scheduling using key events and uncertain time estimates. It is used primarily for projects never before attempted, such as the development of the first nuclear submarine or a new rocket. Duration of activities are estimated using a three-point estimate.

Let:

a = optimistic time (minimum),
m = most likely time, and
b = pessimistic time (maximum)

The expected time, $t_e = \dfrac{a + 4m + b}{6}$

OR

Estimate or mean $\mu = \dfrac{\textit{pessimistic} + 4\,\textit{most likely} + \textit{optimistic}}{6}$

Risk is measured by standard deviation (σ)

$$\sigma = \frac{\textit{Pessimistic} - \textit{Optimistic}}{6} = \frac{\textit{Maximum} - \textit{Minimum}}{6}$$

Abbreviations & Definitions for CPM:

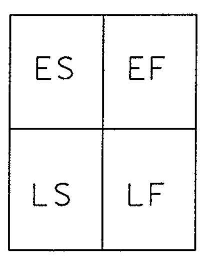
ES: Earliest Start
EF: Earliest Finish
LS: Late Start
LF: Latest Finish

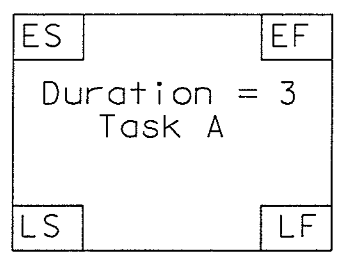
Float = LF – EF

OR

Float = LS – ES

- Float > 0 Time is available
- Float = 0 Situation is critical
- Float < 0 Project is behind schedule





V. Material Quality Control and Production 10% (4 questions)

- The yield is the volume of fresh concrete produced in a batch and is equal to:

$$yield = \frac{Total\ \ weight\ of\ material\ per\ batch}{Unit\ \ weight\ of\ concrete} \tag{5-1}$$

The units of the yield are cubic feet or cubic meters

- The absolute volume is calculated according to the following equation:

$$Ansolute\ \ volume = \frac{Weight\ of\ the\ material}{Specific\ gravity\ of\ the\ material \times Unit\ weight\ of\ water} \tag{5-2}$$

Units and Constants At a Glance

WATER

8.33 lb / gallon
62.4 lb / cubic feet
7.48 gallons / cubic feet

PORTLAND CEMENT (CONVENTIONALLY USED VALUES)

94 lb / sack
94 lb / cubic foot (bulk- loose)
1 cubic feet / sack (bulk-loose)
Specific Gravity 3.15 (unless given)

CONCRETE

Hardrock Concrete Unit Weight ....130 –155 lb / cubic foot
Lightweight Concrete Unit Weight Range .... 105 – 120 lb / cubic foot

AGGREGATE

Sand Specific Gravity.. 2.50 – 2.65
Coarse Specific Gravity.............................. .... 2.55 – 2.70

FLY ASH

Class F .... 2.40 – 2.60
Class C .... 2.20 – 2.50

STANDARD UNITS AND MEASURES

1 Cubic yard = 27 cubic feet
1 ton = 2,000 lb

VI. Temporary Structures
12.5% (5 questions)

Table 6-1 Form Pressure

$$C_C = w \times h \qquad (4\text{-}1)$$

$$C_{C\,SI} = 23.5\, h_{SI} \qquad (4\text{-}1\ SI)$$

Where:

C_C (C_{CSI}) = is lateral pressure, psf (kPa);
w = is the unit weight of fresh concrete, pcf ; and
h (h_{SI}) = is the depth of fluid or plastic concrete, ft (m).

Table 2 Classes or Working Surfaces for Combined Uniformly Distributed Loads

Operational Class	Uniform Load[a] psf (kN/m^2)
Very light duty; sparsely populated with personnel; Hand tools; *very small amounts of construction materials*	20 (0.96)
Light duty: sparsely populated with personnel; hand Operated equipment; staging of materials for lightweight *construction*	25 (1.20)
Medium duty: concentrations of personnel; staging of materials *for averag construction*	50 (2.40)
Heavy duty: material placement by motorized buggies; staging of materials for heavy construction	75 (3.59)

[a]Loads do not include dead load, D; construction dead lad, C_D; or fixed material loads, C_{FML}.

Table 6-2 Form Pressure for concrete made with Type I cement
(weight = 150 pcf, slump $\leq 4''$ & normal internal vibration to a depth of $4'$)

For columns	For walls		Slipform Pressure
	$R < 7\ ft/h$	$7\ ft/h \leq R \leq 10\ ft/h$	
$C_C = (150 + 9{,}000\,R/T)$ (4−2) $C_{CSI} = 7.2 + \frac{785\,R_{SI}}{T_{SI} + 17.8}$ (4−2SI) $600\ psf \leq C_c \leq 3000$ psf $C_c \leq 150\,h$ Where: R (R_{SI}) = rate of concrete placement, ft/h ; and T (T_{SI}) = the temperature of concrete in the forms, °F (°C).	$C_C = (150 + 9{,}000\,R/T)$ (4−3) $C_{CSI} = 7.2 + \frac{785\,R_{SI}}{T_{SI} + 17.8}$ (4−3SI) $600\ psf \leq C_c \leq 2000$ psf $C_c \leq 150\,h$ Where: R (R_{SI}) = rate of concrete placement, ft/h ; and T (T_{SI}) = the temperature of concrete in the forms, °F (°C).	$C_C = 150 + 43{,}400/T + 2{,}800\,R/T$ (4−4) $C_{CSI} = 7.2 + \frac{244\,R_{SI}}{T_{SI} + 17.8} + \frac{1156}{T_{SI} + 17.8}$ (4−4SI) Where: R (R_{SI}) = rate of concrete placement, ft/h ; and T (T_{SI}) = the temperature of concrete in the forms, °F (°C).	$C_C = c + 6{,}000\,R/T$ (4−5) $C_{CSI} = c_{SI} + \frac{524\,R_{SI}}{T_{SI} + 17.8}$ (4−5SI) where: c (c_{SI}) = 100 psf for concrete placed in 6 to 10-in. lift with slight vibration or no revibration, and = 150 psf for concrete that requires additional vibration, such as gastight or containment structures; Cc($C_{C\,SI}$) = lateral pressure, psf R(R_{SI}) = the rate of concrete placement, ft/h ; and T (T_{SI}) = the temperature of concrete in the forms, °F (°C).

VII. Worker Health, Safety, and Environment 7.5% (3 questions)

Table 7-1 OSHA Maximum Allowable Side Slope (OSHA Table B-1)

Soil Type	Maximum Allowable Side Slope (H:V Ratio) for excavation < 20 feet deep(3)	Slope Angle(1) (Repose angle), degrees
Rock	0 : 1 (Vertical)	90
Type A(2) (depth < 12 ft)	½ H : 1V	63
Type A	¾ H : 1V	53
Type B	1H:1V	45
Type C	1 ½ H : 1V	34

(1) Maximum allowable angles expressed in degrees from horizontal. Angles have been rounded off.
(2) A short term maximum allowable slope of ½ H: 1V is allowed in excavations in Type A soil that are 12 feet (3.67 m) or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet (3.67 m) in depth shall be ¾ H: 1V (53°).
(3) Shoring or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.

➤ OSHA Recordable Incident Rate

The OSHA Recordable Incident Rate (or Incident Rate) is calculated by multiplying the number of recordable cases by 200,000, and then dividing that number by the number of labor hours at the company.

$$IR = \frac{Number\ of\ OSHA\ \mathrm{Re}coradable\ Cases\ x\,200{,}000}{Number\ of\ Employee\, labor\ hours\ worked}$$

For example, a company has 17 full-time employees and 3 part-time employees that each work 20 hours per week. This equates to 28,400 labor hours each year. If the company experienced 2 recordable injuries, then the formula works like this:

$$IR = \frac{2\ x\,200{,}000}{28{,}400} = 14.08$$

What is now known is that for every 100 employees, 14.08 employees have been involved in a recordable injury or illness.

➢ Lost Time Case Rate

The Lost Time Case Rate (LTC) is a similar calculation, only it uses the number of cases that contained lost work days. The calculation is made by multiplying the number of incidents that were lost time cases by 200,000 and then dividing that by the employee labor hours at the company.

$$LTC\ Rate = \frac{Number\ of\ \ Lost\,Time\ \ Cases\ \ x\,200{,}000}{Number\ of\ \ Employee\,labor\ hours\ worked}$$

Using the previous company example, assume that one of the two recordable cases had lost work days associated with the incident. The calculations would look like this:

$$LTC\ Rate = \frac{1\ x\,200{,}000}{28{,}400} = 7.04$$

What is now known is that for every 100 employees, 7.04 employees have suffered lost time because of a work related injury or illness.

VIII. Other Topics 10% (4 Questions)

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@seismicisolation

PART 1

Civil BREADTH (A.M.) Exam Specifications

CONSTRUCTION

Construction Breadth (A.M.) Topics

Chapter A: Earthwork Construction and Layout

1. Excavation and embankment (cut and fill) ....
2. Borrow pit volumes ....
3. Site layout and control ....

Chapter B: Estimating Quantities and Costs

1. Quantity take-off methods ....
2. Cost estimating ....

Chapter C: Scheduling

1. Construction sequencing ....
2. Resource scheduling ....
3. Time-cost trade-off ....

Chapter D: Material Quality Control and Production

1. Material testing (e.g., concrete, soil, asphalt)...................................

Chapter E: Temporary Structures

1. Construction loads ....

Chapter A: Earthwork Construction and Layout

A.1 EXCAVATION AND EMBANKMENT (CUT AND FILL)

A.1.1 Introduction:

The movement of soil or rock from one location to another for construction purposes is called earthwork. A volume of earth that is excavated, that is, removed from its natural location, is called cut. Excavated material that is placed and compacted in a different location is called embankment or fill. The construction of the grade line for a new highway, street, pipeline or railway typically involves much cut and fill as shown in Figure A-1; the grading, or reshaping, of the ground for a building site also involves cut and fill.

CUT

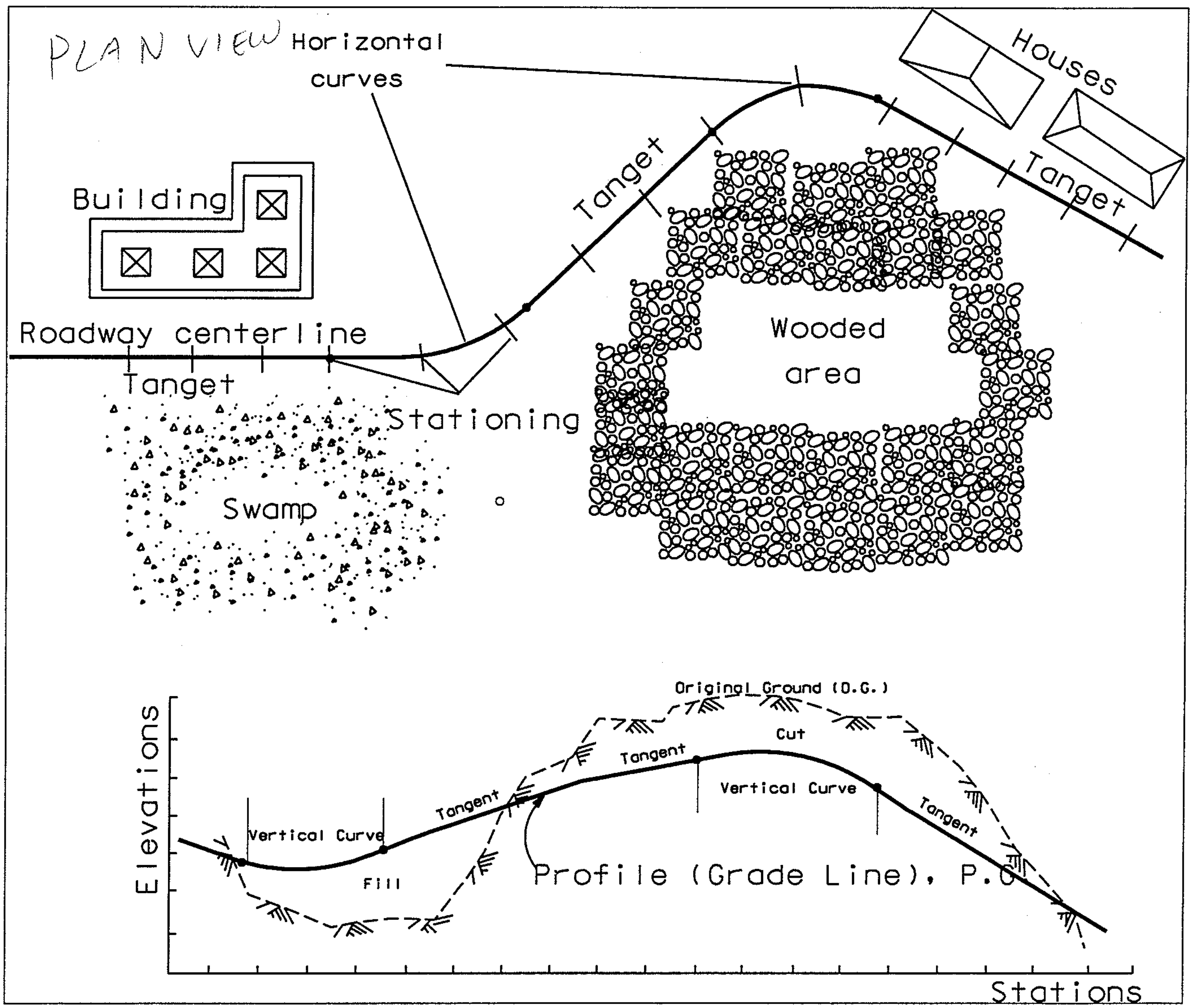


Figure A- 1 Horizontal Alignment, Vertical Alignment, Cut and Fill Relationships

Circular Curves

A.1.2 Horizontal and Vertical Alignments:

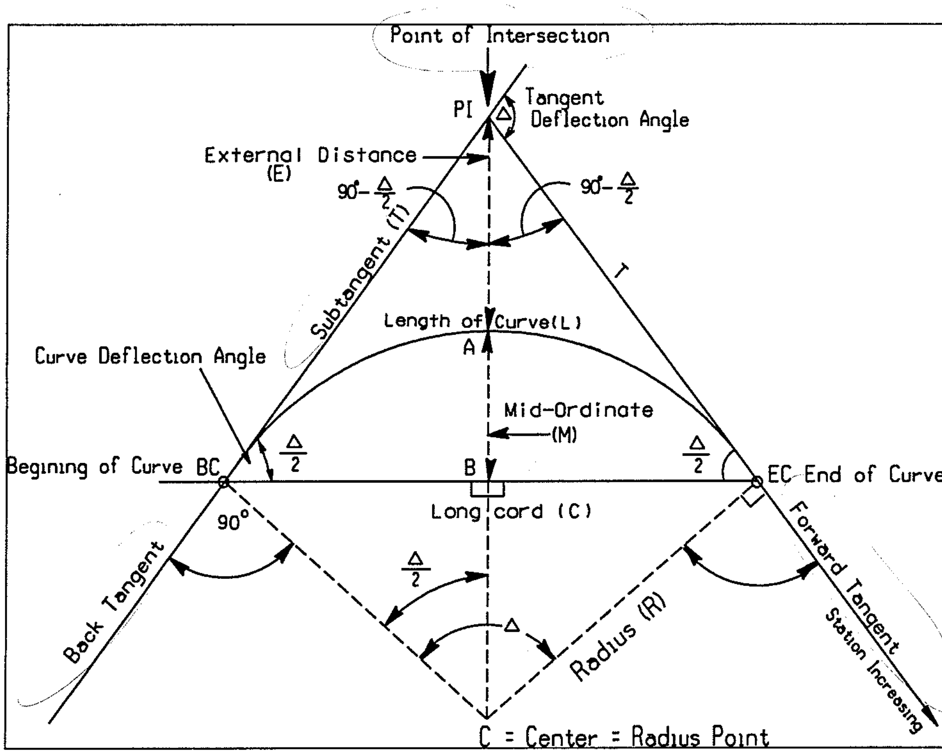


Figure A- 2 Terminology of Horizontal Curve

GEOMETRY OF HORIZONTAL CIRCULAR CURVES

$$T = R \tan \frac{\Delta}{2} \qquad (1)$$

Long cord

$$C = 2\,R\,Sin\,\frac{\Delta}{2} = 2\,T\cos(\Delta/2) \qquad (2)$$

$$L = 2\pi\,R\frac{\Delta^{\circ}}{360^{\circ}} = R\,\Delta\,(radians) = (100\,ft)\left(\frac{\Delta}{D}\right) \qquad (3)$$

$$M = R\,(1-\cos\frac{\Delta}{2}) = \frac{C}{2}\tan\frac{\Delta}{4} = E\,Cos\,\frac{\Delta}{2} \qquad (4)$$

$$E = R\left[\frac{1}{Cos(\Delta/2)} - 1\right] = R\,(Sec\frac{\Delta}{2} - 1) = T\tan\frac{\Delta}{4} = R\tan\frac{\Delta}{2}\tan\frac{\Delta}{4} \qquad (5)$$

Note: A common mistake is to determine the station of the "EC" by adding the "T" distance to the "PI". Although the "EC" is physically a distance of "T" from the "PI", the stationing (chainage) must reflect the fact that the centerline no longer goes through the "PI". The centerline now takes the shorter distance "L" from the "BC" to the "EC".

Sample Problem A-1: Horizontal Curve Chord, Middle Ordinate & External Distance

Given: $\Delta = 16°38'$, R = 1000 ft, PI Sta. @ 6 + 26.57	**Find:** BC and EC stations, length of chord (C), middle ordinate (M) and external distance (E)

LC

Solution:

station — 100' / 100 m

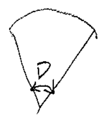
$$T = R\tan\frac{\Delta}{2} = 1000 \tan 8.3167° = 146.18 \text{ ft}$$

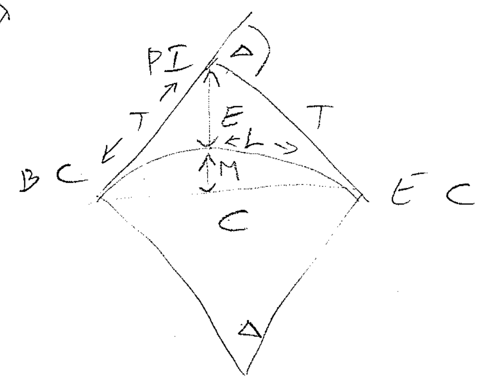
Arc definition: 100'

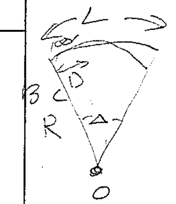
$$L = 2\pi R\frac{\Delta°}{360°} = R\Delta(\overset{\pi/180°}{radians}) = (100\,ft)\left(\frac{\Delta}{D}\right)$$

Degree of curve

$$= 2\pi \times 1000 \times \frac{16.6333}{360} = 290.31 \text{ ft}$$







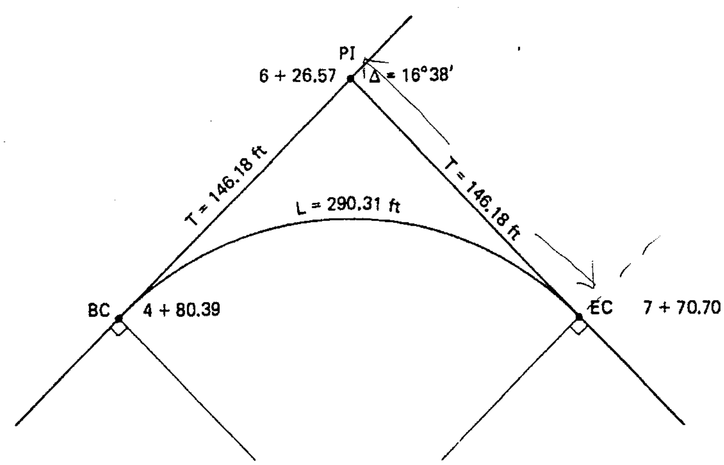
$\frac{\Delta}{D} = \frac{L}{100'}$

PI at	6 + 26.57
−T	1 + 46.18
BC =	4 + 80.39 ⇐
+ L	2 + 90.31
EC =	7 + 70.70 ⇐

$$C = 2RSin\frac{\Delta}{2} = 2T\cos(\Delta/2) = 2 \times 1000 \times \sin 8.3167° = 289.29 \text{ ft} \Leftarrow$$

$$M = R(1-\cos\frac{\Delta}{2}) = \frac{1}{2}C\tan\frac{\Delta}{2} = 1000(1 - \cos 8.3167°) = 10.52 \text{ ft} \Leftarrow$$

$$E = R(\frac{1}{\cos(\Delta/2)} - 1) = R\tan\frac{\Delta}{2}\tan\frac{\Delta}{4} = 1000(\sec 8.3167° - 1) = 10.63 \text{ ft} \Leftarrow$$



Parabolic Curves $y = a + bx + cx^2$

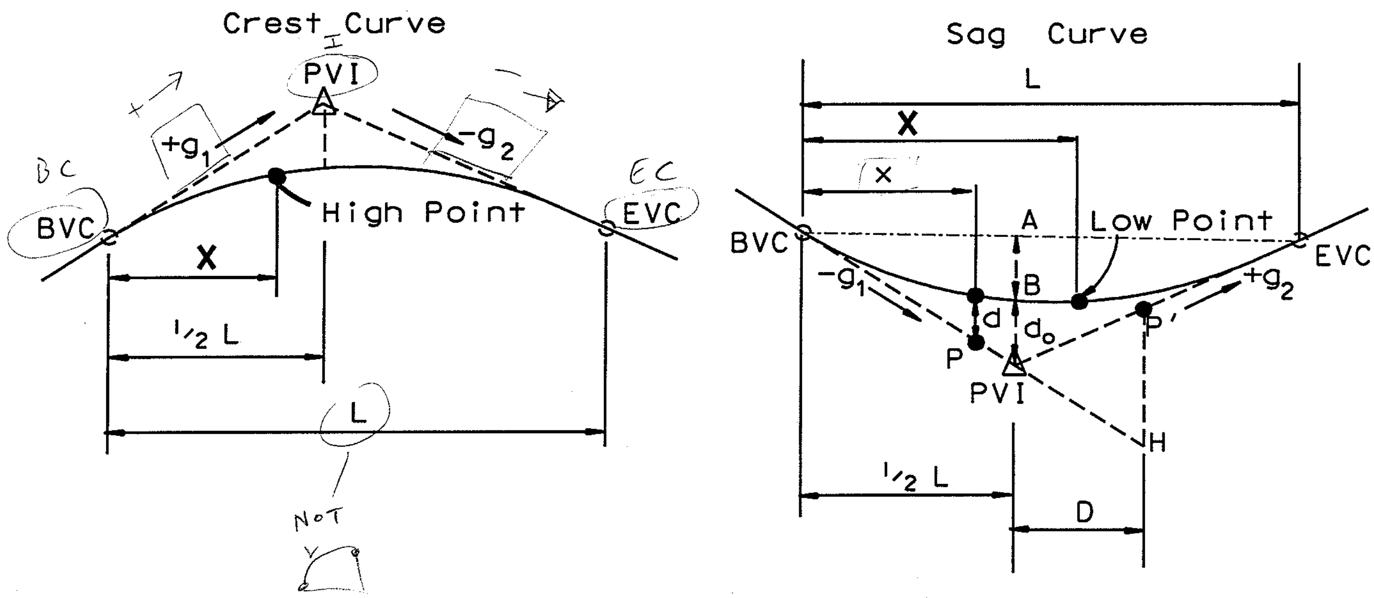


Figure A-3 Crest and Sag Vertical Curves Terminology

GEOMETRIC PROPERTIES OF THE PARABOLIC VERTICAL CURVES

Symmetric $\frac{L}{2} = \frac{L}{2}$

Any Verticle Curve (crest, Sag)

1. $$y_x = y_{BVC} + g_1 x + \frac{rx^2}{2} \qquad (6)$$

$$r = \frac{g_2 - g_1}{L} \qquad (7)$$

L stations or feet

Where: x = distance from BVC to a point on the curve

r = rate of grade change per station

2. The grade lines (g_1 and g_2) intersect midway between the BVC and the EVC ; that is, BVC to PVI = ½ L = PVI to EVC. This is only true for symmetrical vertical curves.

3. The curve lies midway between the PVI and the midpoint of the chord; that is, A – B = B – PVI = d_o which can be calculated as follows:

   Either:

   d_o = ½ (difference in elevation of PVI and mid-chord elevation)

   & mid-chord elevation = ½ (Elevation of BVC + Elevation of EVC)

   Or:

$$d_o = \frac{|g_1 - g_2| L}{8} \qquad (8)$$

4. The slope S, in percentage, of the tangent to the curve at any point on the curve is given by the following formula:

$$S = g_1 - \frac{x(g_1 - g_2)}{L} \qquad (9)$$

5. The distance D in feet from Vertex to P′ is given as:

$$D = \frac{100(Y_H - Y_{P'})}{(g_1 - g_2)} \qquad (10)$$

6. The distance between the curve and the grade line (tangent) "d" is given as:

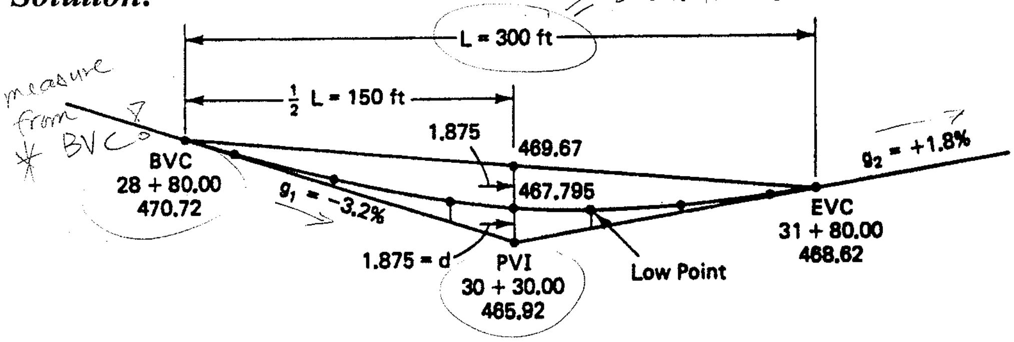
$$d = offset = \frac{rx^2}{2} = \frac{x^2\,(g_2 - g_1)}{200\,L} \quad \text{(L = curve length in feet)} \qquad (11)$$

7. Low and high points are determined as follows:

$$X = \frac{-g_1}{r} = \frac{-g_1 L}{g_2 - g_1} = \frac{g_1 L}{g_1 - g_2} \qquad (12)$$

Sample Problem A-2: Low point on a vertical curve	
Given: L = 300 ft, $g_1 = -3.2\%$, $g_2 = +1.8\%$, PVI at 30 + 30, and elevation = 465.92	***Find:*** Location of the low point and its elevation.

Solution:



$$X = \frac{-g_1}{r} = \frac{-g_1 L}{g_2 - g_1} = \frac{g_1 L}{g_1 - g_2} = \frac{-(-3.2)(3)}{(+1.8)-(-3.2)} = 1.92\ Sta. = 192.00\,ft$$

x100

This means that the low point is located at a distance of 192.00 ft from BVC i.e. at Station = (28 + 80.00) + (1 + 92.00) = 30 + 72.00

Remember: All *distances* used to located a low or a high point or used to determine an elevation of a point on a vertical curve are *measured from BVC.*

$$y_x = y_{BVC} + g_1 x + \frac{rx^2}{2}$$

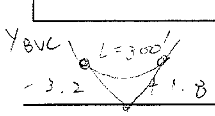
For SAG or CREST

$g_2 - g_1$ X

Y_{BVC}? (150′)(3.2%)

$$= [465.92 + (1.5)(3.2)] + (-3.2)(1.92) + (\frac{1.8-(-3.2)}{3.00})(\frac{1.92^2}{2}) = 467.65\,ft @ Sta\ 30+72.00$$

3 station / 2 station



PVI = 465.92

L/2

Unit

Earthwork quantities or volumes are measured in terms of cubic yards (yd^3) or cubic meters (m^3). Generally, the volume is computed as the product of an area and a depth or distance (length).

$$Volume = Cross\ Sectional\ Area \times Length \tag{13}$$

Area

2 : 1 English
H : V

A.1.3 Cross Sections and Areas:

A cross section is a short profile taken perpendicular to the centerline of a roadway or other facility. The cross section at a station along a road will typically show the profile of the original ground surface, the base of the roadway, and the side slopes of the cut or fill. The cross section of the facility will be viewed as we look to the east (if the facility is an E-W orientation) or to the north (if the facility is a N-S orientation). The base is the horizontal line to which the cut or fill is first constructed; its width depends primarily on the number of lanes and the width of roadway shoulders as shown in Figure A-4.

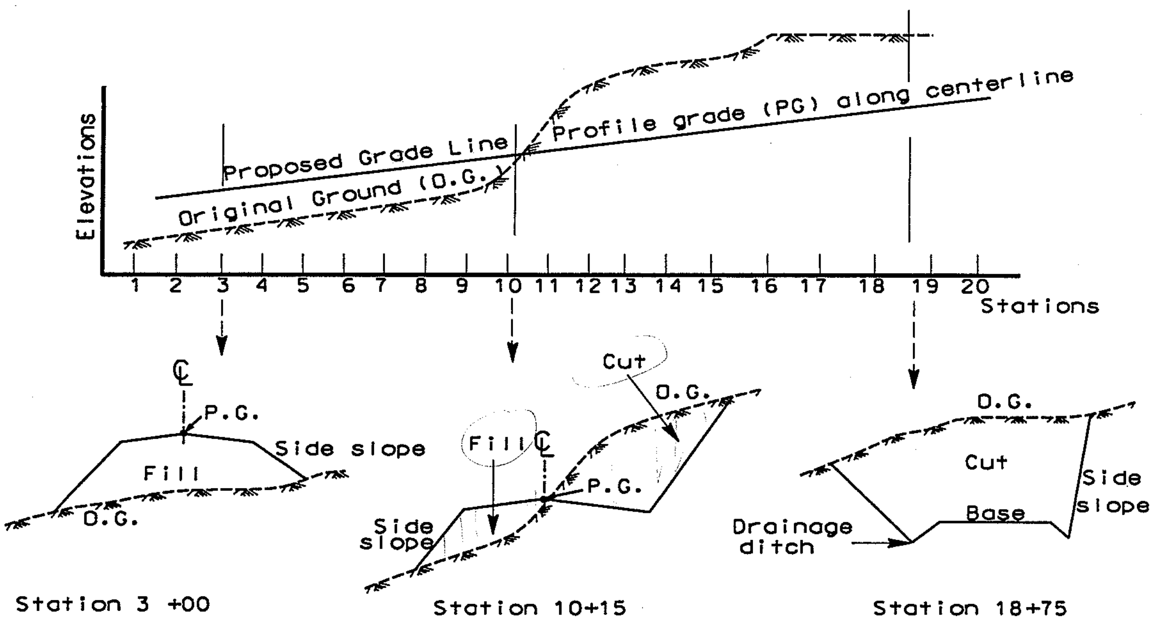


Figure A- 4 Cross Sections, Grade Line (PG) and Original Ground (OG) Relationships

A side slope is expressed as the ratio of a horizontal distance to a corresponding unit of vertical distance for the cut or fill slope as shown in Figure A-5. This ratio depends largely on the type of soil and on the natural angle of repose at which it remains stable.

$S_2 > S_1$

slope = tan θ = rise / run

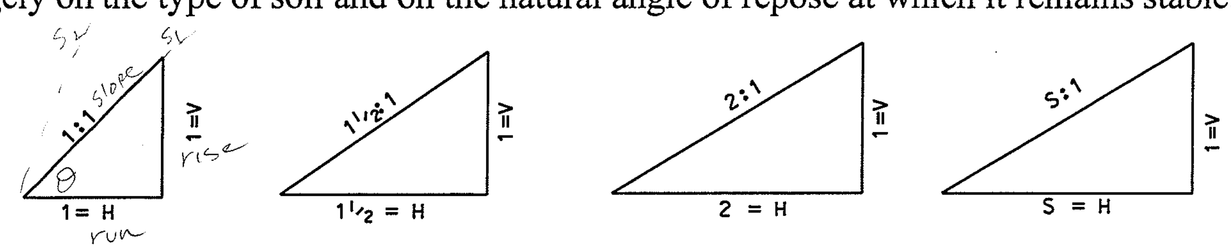
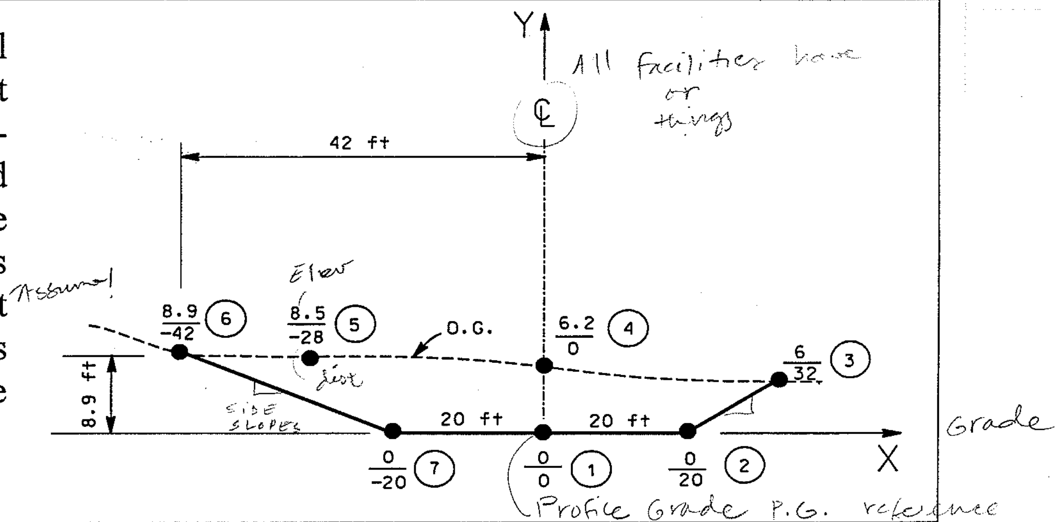


Figure A- 5 Side Slope in Relation to Horizontal and Vertical Distances

A side slope of 1:1 is possible for some compacted embankment sections, whereas a flatter ratio of 2:1(English Units) or more is typical for a side slope in a cut section. Of course, a vertical concrete retaining wall may be built to hold back the soil where very flat side slopes would require excessively wide right-of-way acquisition. Note that the definition of side slope is opposite that of gradient, slope which is "rise over run".

Sample Problem A-3: Side Slope Calculations

Given: A cross-sectional area of a proposed street is given below. The cross-section notes are provided for the critical points. The notes are given in terms of cut depth (with respect to base) and distances (with respect to centerline of the facility). Assume!



Find: The steeper side slope of the proposed facility is most nearly:

(A) 1:1
(B) 2 : 1
(C) 2.5 : 1
(D) 2.75 : 1

The side slope connecting points 2 and 3 is:

$$\frac{rise}{run} = \frac{\textit{Horizontal Difference}}{\textit{Vertical Difference}} = \frac{(32-20)}{(6-0)} = \frac{12}{6} = \frac{2}{1}$$

The side slope connecting points 6 and 7 is:

$$= \frac{(42-20)}{(8.9-0)} = \frac{22}{8.9} = \frac{2.47}{1}$$

Answer: B ⇐

NOTE: A side slope 2.5:1 is flatter than 2:1

A.1.4 Plotting Cross Sections:

Route cross sections are usually plotted to scale on a special grid or "cross-section paper"; a typical scale is 1 in = 5 ft (1: 60) for both the vertical and the horizontal axes. Sometimes the vertical scale is exaggerated if the depth of cut or fill is very shallow. For wide sections with flat side slopes, a horizontal scale as small as 1 in = 20 ft (1: 240) may be used to conserve space on the paper. A cross section is usually drawn for each half-station or quarter-station interval along the route, and the station number is recorded just below the section view as shown in Figure A- 6.

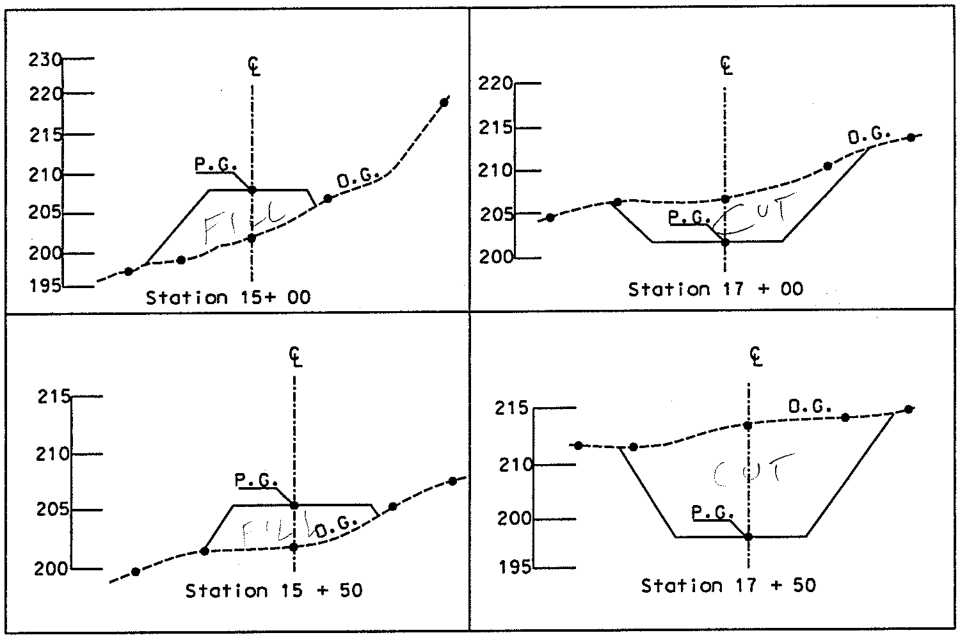


Figure A- 6 Cross Sections, Grade Line (PG) and Original Ground (OG) Relationships

A.1.5 Methods of Measuring Cross Sectional Area

The area enclosed in a section by the natural terrain, the side slopes, and the base can be determined in several ways. The following table summarizes the different methods for measuring areas.

$V = AL$

Table A-1 Methods of Measuring Areas

Field Measurements	Map Measurements
More accurate than map measurements	Accuracy depends on: a) quality of surveying data used to produce maps b) map scale, and c) precision of the drafting process
Types of field measurements: 1- division of the tract into simple figures (rectangles, triangles and trapezoids), 2- offsets from a straight line, 3- coordinates (criss-cross), and 4- double meridian distances (DMD)	Types of map measurements: 1- counting coordinates squares, 2- dividing area into triangles, rectangles, 3- digitizing coordinates, and 4- planimeter. parameter of area.

Sample Problem A-4: Plotting Cross-Sections

Given: Slope staking information for a new road with a 22 feet roadbed width is given below:

CUT

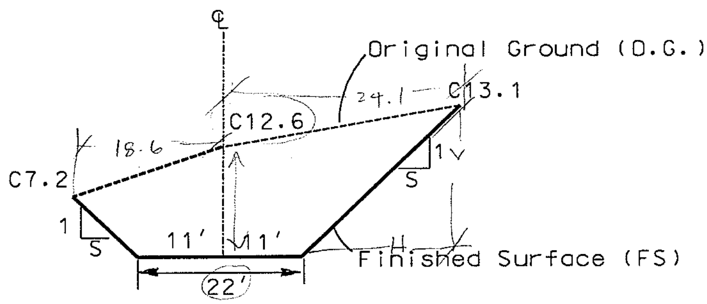
C7.2 / 18.6	C12.6 / 0.0	C13.1 / 24.1
LT	℄	RT

TIP: Draw the cross section of the proposed road. Then calculate the slope which is defined as: Run/Rise OR H/V

Find: The side slope of the proposed facility is most nearly:

(A) 1:1
(B) 1.5 : 1
(C) 1.75 : 1
(D) 2 : 1

Solution:



$$\text{Right side slope (S)} = \frac{\textit{Horizontal Difference}}{\textit{Vertical Difference}} = \frac{(24.1-11)}{(13.1-0)} = \frac{13.1}{13.1} = \frac{1}{1}$$

$$\text{Left side slope (S)} = \frac{\textit{Horizontal Difference}}{\textit{Vertical Difference}} = \frac{(18.6-11)}{(7.2-0)} = \frac{7.6}{7.2} = \frac{1.05}{1}$$

Answer: A ⇐

A.1.6 Measurement of area by planimeter:

Planimeters are excellent tools to use when you measure irregular shaped areas on plans or drawings. They eliminate the need for grids, charts, or calculations done by hand. There are two types of planimeters: mechanical and electronic. The precision obtained using a planimeter depends on operator skill, accuracy of the plotted map, and other factors. Results correct within ½ to 1 % can be obtained.

Some general requirements for the use of a planimeter are to:

- Perform all work on a smooth, horizontal surface.
- Select and mark a starting point on the perimeter of the figure. Movement of the tracer arm around the figure should begin and end exactly at that point. It is more important to start and stop at the same point than to precisely follow the perimeter.

- Trace the perimeter in a clockwise direction (so that the readings increase). If the tracer point strays slightly off the perimeter, compensated by moving off to the other side of the line to make the areas of the errors about equal.

- To avoid blunders, and for increased accuracy, trace the figure until you get at least <u>three consistent area readings</u> with the planimeter, and use an average reading to compute the enclosed area. Compute the average of only those consistent area readings.

Sample Problem A-5: Area by Planimeter

(Mechanical)

Given: An electronic planimeter is used to trace a cross section of a new highway that was drawn to a scale of 1 in = 10 ft. The measured area is 32.6 in^2.

on the map.

Find: The scaled area of the section in square yards is most nearly:

(A) 120.74 yd^2
(B) 262.22 yd^2
(C) 362.22 yd^2
(D) 392.22 yd^2

Solution:

The given scale is 1 in = 10 ft $\Rightarrow$ $(1\text{ in})^2 = (10\text{ ft})^2$ OR $1\text{ in}^2 = 100\text{ ft}^2$
The cross-section area, then, is $32.6\text{ in}^2 \times 100\text{ ft}^2/1\text{ in}^2 = 3260\text{ ft}^2$
And, $3260\text{ ft}^2 \times (1\text{ yd}^2/9\text{ ft}^2) = 362.22\text{ yd}^2$

UNITS ∇

Answer: C⇐

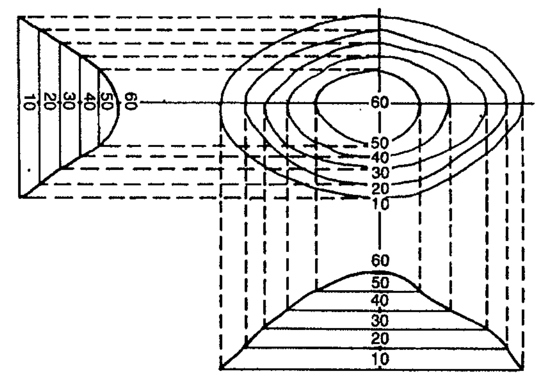
$1'' = 10' \equiv 1 : 120$ ratio $= \frac{1}{120}$

A.1.7 Methods of Volume Measurements:

AL

Table A-2 Methods of Volume Measurement

AD

Cross-Section Method	Unit Area (Borrow-Pit)	Contour–Area Method
➢ Consists of measuring ground elevations and corresponding distances left and right perpendicular to the centerline of the project. ➢ Areas (end areas) are calculated at certain locations depending on the original ground elevations. ➢ Using the end areas, volumes can be calculated by: 1- Average-end- area, 2- Prismodial formula, and 3- Pyramid formula	➢ The area to be covered is divided into 10, 25, 50 or 100 foot squares depending on the project size and accuracy. A transit or tape is used to lay out the squares on the area. A bench mark of known or assumed elevation is established outside the area in a safe place not to be disturbed. ➢ Corners of the squares are designated by letters or numbers. ➢ A level is set up, a plus sight taken on the BM and minus sight read on corners of the squares. The average reading of the four corners is multiplied by the area of each square.	➢ Volumes based on contours can be obtained from contour maps by planimetering the area enclosed by each contour and multiplying the average of areas adjacent by the contour intervals (CI)  Using the above figure and assuming the area was measured using a planimeter and given as follows: Area within 10-ft contour = 19, 400 ft^2 Area within 20-ft contour = 16, 400 ft^2 The volume = $V = \frac{10(19{,}400+16{,}400)}{2\times 27} = 7{,}458 yd^3$

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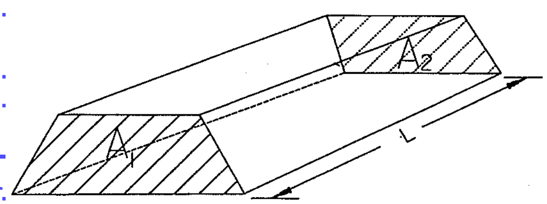
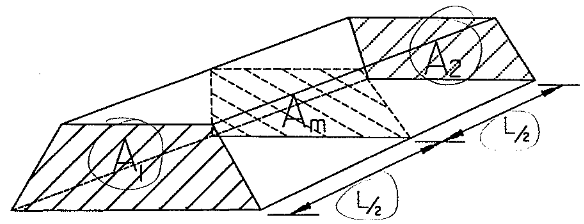
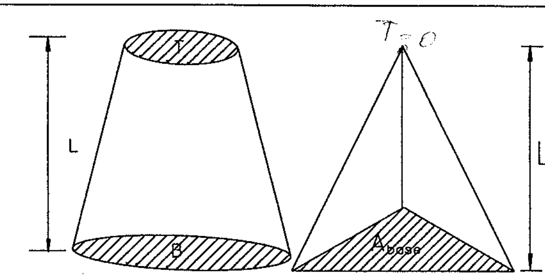
A.1.8 Comparison of the Cross-Section Formulas

Table A-3 Cross-Section Methods of Volume Measurement

Average end-area formula	**Prismoidal formula**	**Pyramid formula**
➢ It gives approximate and larger (overestimates) volumes. ➢ It ignores the slopes and the orientation of the ends and sides. ➢ It favors contractors. ➢ Increased accuracy is obtained by decreasing the distance "L" between cross sections.	➢ It gives more accurate and less volumes than the average-end area. ➢ It applies to all volumes of all geometric solids than can be considered prismoids. ➢ If favors the owner. ➢ The difference between the volumes obtained by the average - end-area and the prismoidal formula is called the *prismoidal correction.*	➢ It is used when one end area is small (about 5% of the other end area) or zero. ➢ When one end area is not zero truncated pyramid will produce more accurate volumes. 100 ft² 5 ft² 100' $V_{pyramid} = (\frac{1}{3})(100)(100) \Rightarrow 3333.33\ ft^3$ $V = (\frac{5+100}{2})100 \Rightarrow 5250\ ft^3$ Ave end Method.
		
$V = \frac{A_1 + A_2}{2} \times \frac{L}{27} (yd^3)$ (14) $ft^3 \rightarrow yd^3$ conversion $A_1 \& A_2 =$ end areas in square feet $L =$ distance between end areas in feet $V = \frac{A_1 + A_2}{2} \times L\ (m^3)$ (15) $A_1 \& A_2 =$ end areas in square meters $L =$ distance between end areas in meters.	$V = (A_1 + 4A_m + A_2)\frac{L}{6 \times 27}(yd^3)$ (16) $A_1 \& A_2 =$ end areas in square feet (Average) $A_m =$ area of a "computed" section midway (halfway) between $A_1 \& A_2$ $L =$ horizontal distance between end areas in feet.	$V_{pyramid} = \frac{A_{base} L}{3 \times 27}(yd^3)$ (17) $A_{base} =$ area of the base $L =$ distance between the two ends. $V_{Truncated\ Py.} = \frac{L}{3 \times 27}(T + B + \sqrt{TB})(yd^3)$ (18) $T =$ top area in square feet $B =$ bottom area in square feet $L =$ distance between the two ends in feet

$V_{truncated} = \frac{L}{3}(T + B + \sqrt{TB}) \Rightarrow \frac{100}{3}(100 + 5 + \sqrt{500}) \Rightarrow 4245\ ft^3$

USC

Sample Problem A-6: Volume using average-end-method

***Given*:** The following data of the cross section for a roadway is given in the following table.

Station	Cut (ft^2)	Fill (ft^2)
1+ 00	0	50
2+ 00	250	70
3+ 00	100	0

Find: The amount of imported borrow (deficit) or waste (excess) using the average end area method is most nearly:

(A) 350 yd^3 waste
(B) 760 yd^3 imported borrow
(C) 760 yd^3 waste
(D) 1100 yd^3 imported borrow

Solution:

$$V_{Cut}=\left(\frac{0+250}{2}\right)\left(\frac{100}{27}\right)+\left(\frac{250+100}{2}\right)\left(\frac{100}{27}\right)=1111.11\ Cu\,Yd$$

$$V_{Fill}=\left(\frac{50+70}{2}\right)\left(\frac{100}{27}\right)+\left(\frac{70+0}{2}\right)\left(\frac{100}{27}\right)=351.85\ Cu\,Yd$$

$$V_{Net}=V_{Cut}-V_{Fill}=1111.11-351.85=759.26\ Cu\,Yd$$

≈ 760 yd^3 Answer: C ⇐

Sample Problem A-7: Volume using average-end-area & pyramid formulas

***Given*:** The following data of the cross section for a roadway is given in the following table.

Station	Cut (ft^2)	Fill (ft^2)
1+ 00	0	50
2+ 00	250	70
3+ 00	100	0

Find: The amount of imported borrow (deficit) or waste (excess) is most nearly: (Solve the problem using the pyramid method wherever its applicable) vs. 760 yd^3

(A) 650 yd^3 waste
(B) 650 yd^3 imported borrow
(C) 760 yd^3 waste
(D) 760 yd^3 imported borrow

Solution: pyramid Ave end

$$V_{Cut}=\left(\frac{250}{3}\right)\left(\frac{100}{27}\right)+\left(\frac{250+100}{2}\right)\left(\frac{100}{27}\right)=956.79\ Cu\,Yd$$

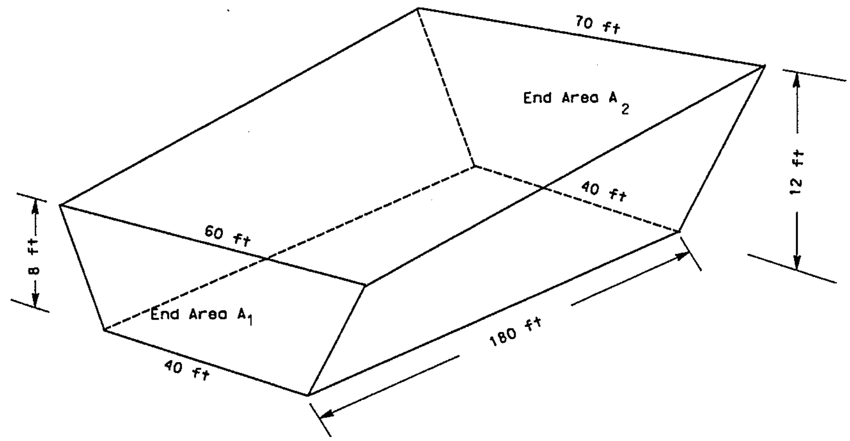
$$V_{Fill}=\left(\frac{50+70}{2}\right)\left(\frac{100}{27}\right)+\left(\frac{70}{3}\right)\left(\frac{100}{27}\right)=308.64\ Cu\,Yd$$

$$V_{Net}=V_{Cut}-V_{Fill}=956.79-308.64=648.15\ Cu\,Yd$$

Answer: A ⇐

Sample Problem A-8: Volume using Prismodial Formula

Given: The following cross-section of a proposed irrigation canal is given below.



Find: The volume is most nearly:

A) 3500 yd^3
B) 3510 yd^3
C) 3535 yd^3
D) 3565 yd^3

Solution:

Because of the variation on the two end areas and the problem statement contains enough information, the prismodial formula will be used.

The middle area (half way between the two end areas) dimensions:
Base = 40 ft
Height = (8 + 12)/2 = 10 ft
Top = (60 + 70)/2 = 65 ft

The middle area = $A_m = \left(\dfrac{40+65}{2}\right)\times 10 = 525\,ft^2$

The end area = $A_1 = \left(\dfrac{40+60}{2}\right)\times 8 = 400\,ft^2$

The end area = $A_2 = \left(\dfrac{40+70}{2}\right)\times 12 = 660\,ft^2$

Using Equation 1-17

$$V = (A_1 + 4A_m + A_2)\frac{L}{6\times 27}(yd^3)$$

$$=(400+4\times 525+660)\frac{180}{6\times 27} = 3511.11\ yd^3$$

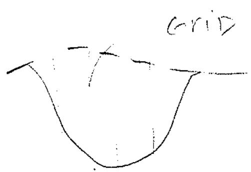
Answer: B ⇐

A.2 BORROW PIT VOLUMES

Lengthy

Volume by the Grid Method when fill material must be hauled to a jobsite from an outside source, such as for embankment construction, the source is called a **borrow pit**. Payment for borrow is generally on a unit price basis (i.e., dollars per cubic yard or cubic meter), and the surveyor is called on to measure the quantity of the material excavated from the borrow pit. This is done by the grid method (Figure A-9). A set of permanent marks or stakes are established just outside the borrow pit area to form a grid of small squares; the squares are usually 25, 50 or 100 ft in size.

Rod shots are taken at the intersections of the grid, before and after excavation, and each change in elevation is computed. For one square, the volume of borrow is approximately equal to the average of the elevation change at the corners times the area of the square.



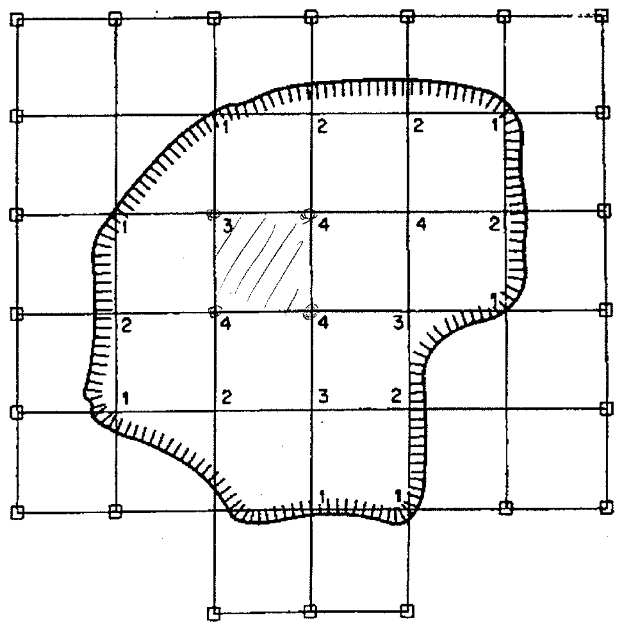


Figure A- 9 Grid Method for Computing Volumes of Excavation at a Borrow Pit

For example, if the changes in elevations at the corners of a 25-ft square are 4.2, 3.6, 4.9, and 5.1 ft, the excavated volume is simply:

Ave elev 4 corners A

$$= (4.2 + 3.6 + 4.9 + 5.1)/4 \times (25\ \text{ft})^2$$
$$= (4.45\ \text{ft})(625\ \text{ft}^2) = 2{,}781\ \text{ft}^3/(27\ \text{ft}^3/\text{yd}^3) = 103\ \text{yd}^3$$

Quantities computed in cubic feet must be divided by 27 ft^3/yd^3 to convert the volume to cubic yards.

$$Volume = \frac{\sum(\text{depth at the corners of the grid})}{\text{number of the corners}} \times \text{area of grid} \qquad (19)$$

Adjacent squares can be combined as a group, and the change in elevation at each grid point can be multiplied by the number of grid squares it touches, to avoid repetitive computations; this is illustrated by the numbers at each grid point in Figure A-9. The sum of these results is divided by four and multiplied by the area of one square. Volumes for the parts of the borrow pit not covered by a full grid square may be computed by taking the product of the figure area (e.g., a triangle) and the average depth of cut. The following two examples illustrate the use and the application of the Borrow Pit Method for volume computation.

Borrow Pit Method At a Glance

A- Before Excavating:

1. Construct a rectangular grid with regular uniform intervals over the area containing the borrow pit. If a full square grid may not be achievable, other figure areas (e.g., a triangle, a rectangle) may be used.
2. A control point (horizontal and vertical control) is set up in a safe location (so it will not be disturbed) nearby the borrow pit area.
3. Determine the horizontal and vertical positions of the intersecting points of the grid with respect to the reference (control) point i.e. creating a surface of the original ground before excavation.

B- After Excavating:

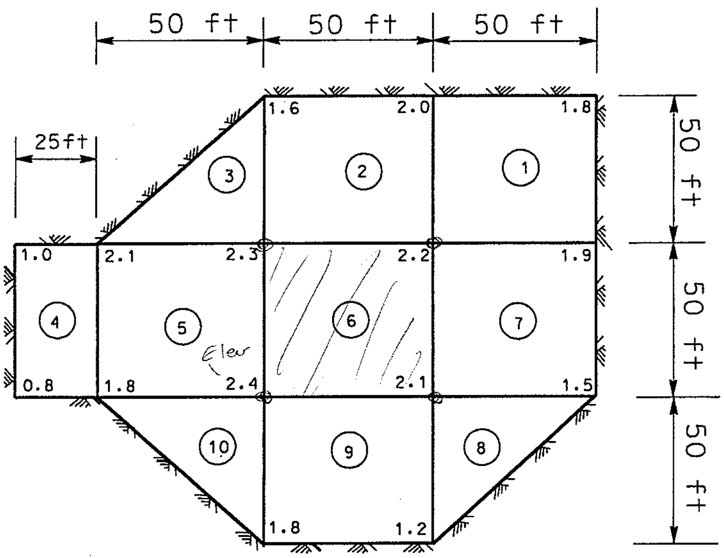
1. A similar survey is required where the same intersecting grid points are located horizontally and vertically.
2. This will create a surface of the sides and bottom of the borrow bit (surface after excavation).

C- Volume Computation:

1. The difference between the two surfaces is the volume in cubic yards or meters.

Sample Problem A-9: Volume by Borrow Pit Method

Given: A borrow pit grid is shown below. The numbers at the grid points represent the depth of cut, in feet.



Find: The excavated volume in cubic yards is most nearly:

(A) 1455 yd^3
(B) 1495 yd^3
(C) 1515 yd^3
(D) 1545 yd^3

Solution:

The grid figures can be grouped as six adjacent 50-ft squares (1, 2, ~~4~~ 7, 5, 6, and 9), three separate triangles (3, 8, and 10), and a 50 x 25 ft rectangle (~~7~~ 4).

I - Squares:

n

For the group of squares, the sum of the corners is: $(1.8) + (2 \times 2.0) + (1.6) + (2 \times 1.9) + (4 \times 2.2) + (3 \times 2.3) + (2.1) + (1.5) + (3 \times 2.1) + (3 \times 2.4) + (1.8) + (1.2) + (1.8)$
$= 48.8$ ft.

The total volume excavated within those grid squares is:
$= (48.8/6 \times 4)(50 \times 50) \times 6 = 30{,}500 \text{ ft}^3$

# of squares corners per 1 shape A

II - Triangles:

corners

For triangle 3, the average cut is: $= (1.6 + 2.3 + 2.1)/3 = 2.0$ ft. Avg cut
The area of the triangle is: $= ½ \times 50 \times 50 = 1250 \text{ ft}^2$. ½bh
The approximate volume = area x height = $(2.0 \text{ ft})(1250 \text{ ft}^2) = 2{,}500 \text{ ft}^3$ multiply together
For triangle 8, the volume is: $= ½ (1.5 + 2.1 + 1.2)/3 \times 50 \times 50 = 2{,}000 \text{ ft}^3$
For triangle 10, the volume is: $= ½ (1.8+2.4+1.8)/3 \times 50 \times 50 = 2{,}500 \text{ ft}^3$

III – Rectangle:

For the rectangle 7, the volume is: $= (2.1+1.0 + 0.8 + 1.8)/4 \times 25 \times 50 = 1{,}781 \text{ ft}^3$
Finally, the total volume $= 30{,}500 + 2{,}500 + 2{,}000 + 2{,}500 + 1{,}781 = 39{,}281 \text{ ft}^3$
$= 39{,}281 \text{ ft}^3 / (27 \text{ ft}^3/\text{yd}^3) = 1454.9 \text{ yd}^3$

Answer: A ⇐

A.3 SITE LAYOUT AND CONTROL

A.3.1 Introduction:

All structures (buildings, bridges, houses, ….etc) and all other construction operations (roads, railroads, pipeline, …etc) must be located during construction with respect to a reference. This reference most of the time is the property limits. A centerline (known as centerline construction, layout line, LOL, centerline surveying) is another reference to layout the facility before and during construction. Therefore, in the initial stage of the construction of a facility, the survey involves the careful retracing and verification of the property lines and any reference lines that will be used during construction. Once the property lines/LOL are established, the building/facility is located according to plan, with all corners or offset distances marked in the field.

Site layout is a part/phase of the site planning of any construction project. The site planning consists of different phases; depending on the type of the facility, as follows:

1- Site design
2- Site zoning
3- Site parking
4- Site analysis
5- Site section
6- Site grading

Whether the property line or other reference lines are used to layout the construction phases of the project, the first requirement for any project is to establish good horizontal and vertical control.

A.3.2 Horizontal control:

Horizontal control for a construction project is provided by two or more points on the ground, permanently or semi-permanently monumented, and precisely fixed in position horizontally by distance and direction, or coordinates. Horizontal control can be established by the traditional ground surveying methods of traversing, triangulation, trilateration, or by using GPS. For small areas, horizontal control for a small project is generally established by traversing. Until recently, triangulation and trilateration were the most economical procedures available for establishing basic control for projects extending over large areas. These techniques have been replaced by Global Positioning System (GPS), which is not only highly accurate but also very efficient.

The datum for horizontal control used for the project must be clearly indicated on the maps of that project. North American Datum of 1927 (NAD27), which is based on Clarke ellipsoid of 1986, was used until the inception of NAD83. The NAD83 employs a different set of defining parameters than NAD27, and it uses a reference surface of different dimensions, the GRS80 ellipsoid. Thus, the latitudes and longitudes of points in NAD83 are somewhat different from their values in NAD27.

A.3.3 Vertical control:

Vertical control is provided by bench marks in or near the tract to be surveyed, and it becomes the foundation for correctly portraying relief on a topographic map. Vertical control is usually established by running lines of differential levels, starting from and closing on established bench marks. Project bench marks are established throughout the mapping area in strategic locations, and their elevations determined by including them as turning points in the differential leveling lines.

Temporary benchmarks will be surveyed onto all major sites from the closest bench-mark, and then the work will be verified by closing the survey into another independent benchmark. The surveyor establishes a minimum of three temporary benchmarks at each site to ensure that, if one is destroyed, at least two will be available for all layout work.

Leveling is the procedure used when one is determining the vertical position of points and the differences in elevation between points that are remote from each other. An elevation is a vertical distance above or below a reference datum. In surveying, the reference datum that is universally employed is that of mean sea level (MSL) usually assigned an elevation of zero. In North America, 19 years of observations at tidal stations in 26 locations on the Atlantic, Pacific, and Gulf of Mexico shorelines were reduced and adjusted to provide the national geodetic vertical datum (NGVD) of 1929. This datum has been further refined to reflect gravimetric and other anomalies in the 1988 general control readjustment (North American Vertical Datum-NAVD 88). V 1988 # 1983

The following figure shows an example of site layout and control where the surveyor sets up the property lines as shown on the site plan (a licensed surveyor may be required at this stage to establish the legal lines). The building corners are then set out by measuring the setbacks from the property lines. After the corners have been staked, diagonal distances (on line or offsets) can be measured to verify the layout- that is, for the building shown (50 ft × 60 ft), the diagonals should be $\sqrt{(50)^2+(60)^2} = 78.10$ ft. At an accuracy requirement of 1/5000, the diagonal tolerance would be 0.016 ft = 0.20 inch (i.e., X/78.1 = 1/5000). Once the corners have been laid out and verified, the offsets and batter boards can be laid out. The same figure shows the location of batter boards and string lines used for control of an L-shaped building; the batter boards and string lines are usually set at the first-floor elevation.

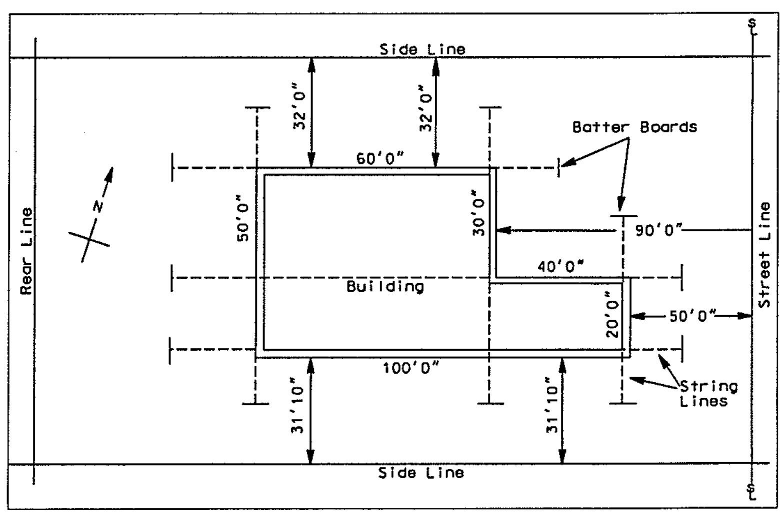


Figure A-10 Layout of a Building

A.3.4 Site Reconnaissance Checklist

General Considerations:

- I. What features are native to topology and climate?
- II. What is required for construction method selected?
- III. What features are needed to support construction force?
- IV. What features might encroach on local society or environment?

I. Features native to topology and climate

1. actual topology (excessive grades, etc.)
2. elevation
3. geology (soil characteristics, rock, etc.)
4. ground cover
5. excessive seasonal effects
6. wind direction
7. natural defenses
8. drainage
9. subsurface water conditions
10. seismic zones

II. Features required that contribute to construction method

1. accessibility to site (rail, road, water)
2. labor availability (skill, cost, attitude)
3. material availability (salvage, cost, attitude)
4. locate borrow pits (gravel, sand, base, fill)
5. locate storage areas, plant sites
6. alternate building, campsites
7. general working room about site
8. location of existing structures and utilities
9. conflicts with existing structures and utilities
10. overhead
11. disposal areas
12. land usage
13. local building practices

III. Features to support construction force

1. billeting/shelter
2. food (also on-job meals)
3. special equipment
4. clothing
5. communications
6. local hazards
7. fire/ security protection available
8. local customs/culture

9. potable water
10. sanitary facilities (also for job)
11. entertainment
12. small stores
13. medical
14. banking, currency
15. transportation
16. local maintenance available

IV. Features that might encroach on local society or environment:

1. noise
2. dust
3. blasting
4. hauling over roads
5. use of water
6. burning (smoke)
7. drainage (create problems)
8. flight operations
9. disposal areas
10. utility disruption
11. relocation problems
12. work hours
13. economy impact
14. community attitude
15. security
16. political

Sample Problem A-10: Sit Layout & Control

Given: You have staked the corners of a 60 ft × 80 ft rectangular concrete pad for a new house at 4-foot offset. To check your work, you would:

(A) subtract the offset from each side
(B) turn the angles at each corner
(C) measure the diagonals and compare them with calculated values
(D) measure each side 2 times

Solution:

- The best method to check the layout in the filed is to calculate the diagonals and compare the measured values with the calculated values.
- Turning the angles and measuring the sides will not insure that the work was laid out correctly.
- Also, subtracting the offset from each side can't insure that the house was laid out according to the plans.

Answer: C ⇐

Sample Problem A-11: Sit Layout & Control

Given: Which of the following is the least important for a site layout for a new shopping center:

(A) the horizontal and vertical control of the site
(B) the existing underground utilities
(C) accessibility to the site
(D) wind direction

Solution:

- All four items are listed in the site reconnaissance checklist. The horizontal and vertical control is extremely important because all distances (H &V) are based on the control.
- Underground utilities are important and ALL high risk utilities should be potholed and the elevations of these utilities must be shown on the layout. All conflicts with utilities either high or low risk must be resolved before construction starts.
- Accessibility to the site is important.
- Wind direction is the least important among all others.

Answer: D⇐

Chapter B: Estimating Quantities and Costs

B.1 QUANTITY TAKE-OFF (TAKEOFF) METHODS

The term quantity take-off is sometimes called material quantity take-off, or simply take-off. Take-off is the ***process of reading the plans and determining the quantities of work required to build the project.*** It is the estimator's interpretation of the designer's intent. Take-off can be performed by hand calculations or by using the computer. Area and volume calculations, such as earthwork and paving, are usually more efficiently done with the computer. Structure take-offs, such as concrete and reinforcing steel, are usually performed by hand calculations or with spreadsheets because the details are often spread over multiple drawings.

PS & E

To prepare an estimate the estimator reviews the plans and specifications and performs a quantity take-off to determine the type and amount of work required to build the project. Before starting the quantity take-off, the estimator must know how the project is to be constructed and he or she prepares a well-organized checklist of all items required to construct the project.

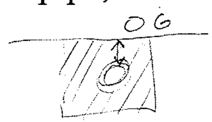
The quantity of material in a project can be accurately determined from the drawings. The estimator must review each sheet of the drawings, calculate the quantity of material, and record the amount and unit of measure on the appropriate line item in the estimate. The unit costs of different material should be obtained from material suppliers and used as the basis of estimating the costs of materials for the project. If the prices quoted for materials do not include delivery, the estimator must include appropriate costs for transporting materials to the project. The cost of taxes on materials should be added to the total cost of all materials at the end of the estimate.

Each estimator must develop a system of quantity take-off that ensures that a quantity is not omitted or calculated twice. A common error in estimating is completely omitting an item or counting an item twice. A well-organized checklist of work will help reduce the chances of omitting an item. The estimator must also add an appropriate percentage for waste for those items where waste is likely to occur during construction. For example, a 5 percent waste might be added to the volume of mortar that is calculated for bricklaying.

PS means

The material quantity takeoff is extremely important for cost estimating because it often establishes the quantity and unit of measure for the costs of labor and the contractor's equipment. For example, the quantity of concrete material for piers might be calculated as 20,000 cubic yards(cy). The labor-hours and the cost of labor required to place the concrete would also be based on 20,000 cy of material. Also the number and the cost of the contractor's equipment that would be required to install the concrete would be based on 20,000 cy of material. Therefore, the estimator must carefully and accurately calculate the quantity and unit of measure of all material in the project.

For unit-price estimates, the take-off quantity may verify the bid quantity, but not include all of the work for which a take-off is required. For example, an item to install 18-in. PVC pipe will state the length of pipe to be installed, but does not address the quantity of excavated earth or the stone pipe bedding that is required to complete this item. These take-off quantities are ancillary to the length of pipe, on which payment is calculated, and can only be determined from the drawings.



B.2 COST ESTIMATING

Reliable and accurate project cost and schedule estimates are critical to staying in business in today's competitive environment of changing technology, greater project complexity, and faster cycle time. In this section, the discussion of how to manage the cost estimation process, understand the advantages and disadvantages of different cost estimation methods, avoid common errors and pitfalls in cost estimation, and properly use cost estimating "rules of thumb".

SCOPE (what)
COST $
SCHEDULE (time)

Cost estimates are done for different reasons, and the purpose of the estimate usually imparts a bias to the numbers. "Marketing estimates" are likely to be low, while good "budget estimates" are likely to be high. When judging the accuracy of an estimate, you need to know the source of the estimate and the purpose for which it was derived.

B.2.1 Direct versus Indirect Costs:

The following table shows a comparison between direct and indirect costs:

Table B-1 Direct and Indirect Costs

	Direct Costs	Indirect Costs
Definition	Are those which can be immediately associated in the field with work directly contributing to the physical completion of the permanent facility contracted by the owner.	Are those which necessarily contribute to the support of a project as a whole, but cannot be identified directly with specific work items in the permanent facility.
Examples	1- finishing labor for a concrete floor slab, 2- materials for a structural steel frame, 3- equipment for a foundation excavation, and 4- a subcontractor's charges for installing the air-conditioning system.	1- job and office personnel salaries; 2- materials, supplies, and utilities for the temporary warehouse, field office building and change facilities; 3- staff vehicles; 4- safety and first-aid expenses; and 5- the portion of home-office support required for the project.

B.2.2 Fixed and Variable Costs:

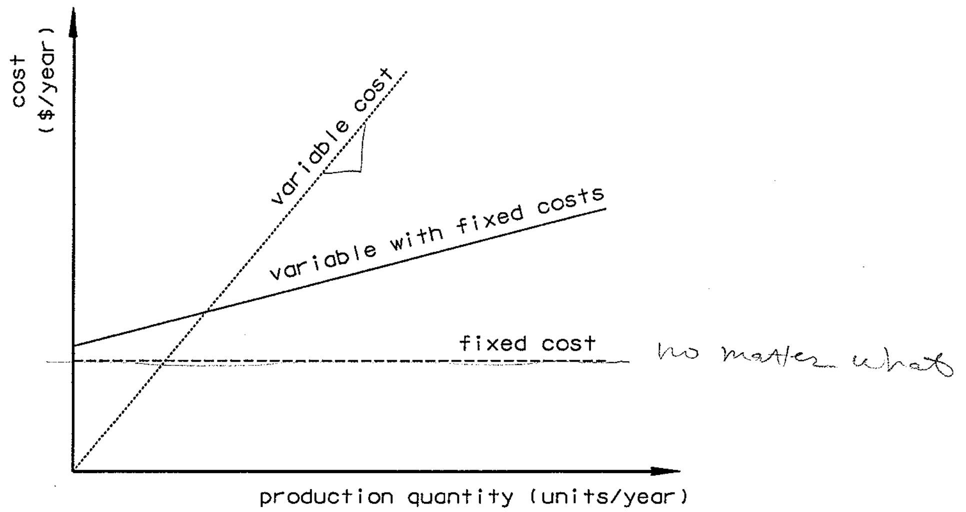


Figure B-1 Fixed and Variable Costs

Table B-2 Fixed and Variable Costs

	Fixed Costs	Variable Costs
Definition	➢ cost at zero production ➢ they are NOT a function of an independent variable	➢ they increase in proportion to the number of units produced ➢ they are a function of an independent variable ➢ the slope of the variable cost function (slope of the line) is known as incremental cost
Examples	1- interest on loans 2- insurance 3- rent	1- direct labor costs 2- direct material costs 3- supervision costs

B.2.3 Estimating Methods:

Several techniques are available to help the estimator estimate the cost of a project. Based on the project's scope, the purpose of the estimate, and the availability of estimating resources, the estimator can choose one or a combination of techniques when estimating an activity or a project.

Generally speaking there are ***three types of cost estimates***:

1- Order of magnitude
2- Budget
3- Detailed

When a cost estimate is **NOT** done well, the most likely problems will be:

1- scope left out, not well defined.
2- not understanding the technical difficulty, and
3- changes.

The following briefly describes techniques used to estimate:

I. Bottoms-Up Technique

Generally, a work statement and set of drawings or specifications are used to "take-off" material quantities required to perform each discrete task performed in accomplishing a given operation or producing an equipment component. From these quantities, direct labor, equipment, and overhead costs are derived and added. This technique is used as the level of detail increases as the project develops.

II. Specific Analogy Technique

Specific analogies depend upon the known cost of an item used in prior systems as the basis for the cost of a similar item in a new system. Adjustments are made to known costs to account for differences in relative complexities of performance, design, and operational characteristics.

III. Parametric Technique

Parametric estimating requires historical data bases on similar systems or subsystems. Data is derived from the historical information or is developed from building a model scenario. Statistical analysis is performed on the data to find correlations between cost drivers and other system parameters, such as design or performance parameters. The analysis produces cost equations or cost estimating relationships that can be used individually or grouped into more complex models. This technique is useful when the information available is not very detailed.

IV. Cost Review and Update Technique

An estimate is constructed by examining previous estimates of the same project for internal logic, completeness of scope, assumptions, and estimating methodology and updating them with any changes.

V. Trend Analysis Technique

A contractor efficiency index is derived by comparing originally projected contract costs against actual costs on work performed to date. The index is used to adjust the cost estimate of work not yet completed.

VI. Expert Opinion Technique

When other techniques or data are not available, this method may be used. Several specialists can be consulted repeatedly until a consensus cost estimate is established.

B.2.4 Data Collection and Normalization:

When estimating, cost data is collected. Data may be collected from similar projects, data bases, and published reports. The basis of the cost data should be documented as part of the detailed backup for the estimate. The amount of data collected will depend on the time available to perform the estimate and the type of estimate, as well as the budget allocation for the estimate's preparation.

When using the collected cost data, the estimator must be aware of the source of the data and make adjustments where necessary. Data from one project may not be consistent or comparable with data from a different project. For example, if historical costs data is used, the costs may not be applicable due to escalation, regulatory changes, or geographical differences. The data should be reviewed and adjustments (normalization) should be made before it is used in the estimate.

B.2.5 How To Estimate Direct Costs?:

In the initial stages of project development, estimates must be derived by using various relationships. As the project develops and more detail is available, the estimate also will be in more detail. Following are some general steps that may be used for developing the direct costs of a detailed estimate.

A. Material Take-off

A material, labor, and equipment take-off is developed from the drawing and specification review. The amount of detailed takeoff will vary with the amount of design detail. The takeoffs are divided into categories or accounts, and each account has sub-accounts. Each project or program should have an established code of accounts. By listing the accounts, a checklist of potential items, and activities that should be included in an estimate is formed.

Each account should be considered, even when developing planning estimates, to help eliminate any omissions or oversights.

B. Pricing the Material and Equipment

On fixed price or lump sum contracts, the material cost should be the cost a contractor will pay for the material and does not include any markup for handling by the contractor. Freight at the job site is included in the material cost. Material and equipment that is specified as Government or State furnished equipment or material (GFE or SFM) should be identified and kept separate from contractor furnished material.

Once the quantity takeoff is complete, the next step is to price the individual items. Several acceptable ways of pricing material are by verbal or written vendor quotations, up-to-date catalog price sheets, estimating manuals, and historical data. The current material price should be used whenever possible. If old prices are used, escalation must be added to make the prices current as of the estimate date. Escalation beyond the date of the estimate is included as a separate item.

C. Construction Equipment

Equipment and tools are required to install the materials. Databases can be used to obtain an equipment usage relationship with the materials. Large equipment may be estimated on an activity basis or may be estimated for the duration of the project. Pricing can be obtained from verbal or written vendor quotes, estimating manuals, and from historical data. Current prices should be used whenever possible, or prices should be adjusted to reflect prices at the time of the estimate date.

Some fixed price or lump sum contract projects require special tools or equipment for completion of the work. An example of this is a heating, ventilation, and air conditioning project that might require a large crane for setting an air handling unit on the roof of a building. The cost of the crane would be considered a direct cost. Examples of construction equipment are small tools and pickup trucks. These costs would be included as an indirect cost.

On cost-plus-fixed-percentage contracts, all costs for construction equipment and small tools are considered direct costs.

D. Labor

Several good publications provide an estimate of the labor hours required for a task that the estimator should use unless adequate experience has given the estimator a more accurate base for determining labor hours required. One important item that must be remembered when using general estimating publications is that these publications are based on a national average construction project for private industry.

When estimating labor costs, the worker's base rate plus all payroll indirect costs, such as Federal Insurance Contributions Act and payroll insurance, are multiplied by the estimated labor hours to generate the labor cost. Typically, this sum is handled as a direct labor cost. For ease of estimating, an average crew rate can be used and rounded to the nearest even dollar hourly rate. The following equations are used to calculate the labor costs:

$$\text{Work hours} = \text{Quantity takeoff} \times \text{productivity rate} \tag{20}$$

$$\text{Adjusted work hours} = \text{Work hours} \times \text{productivity factor} \tag{21}$$

E. Special Conditions

Consideration must be given to all factors that affect construction. Some of these factors are:

- availability of skilled and experienced manpower and their productivity;
- the need for overtime work;
- the anticipated weather conditions during the construction period;
- work in congested areas;
- work at night to avoid or minimize the impact on the public;
- restrictions imposed on the work area; and
- use of respirators and special clothing.

Special conditions may be estimated by applying a factor; for example, 10 percent is applied to the labor hours for loss of productivity due to work in a congested area. Other items may be calculated by performing a detailed takeoff. An example of this would be an activity that could only be performed over a 2-day period. Overtime would be required to complete the activity and the number of hours and rates could be calculated.

F. Government Furnished Materials (GFM) Or State Furnished Materials (SFM)

Labor and equipment costs for installation of GFM or SFM must be included for each item. They may be estimated as previously discussed. These costs should be kept separate from the labor and equipment costs for the materials the contractor is supplying.

G. Sampling and Analysis Costs

In some remediation projects, sampling and analysis costs are a part of the operations. They may be estimated by using the technical scope requirements to determine the type of sampling and analysis that will be performed and the project schedule to calculate the quantity of samples that will be collected. Costs can be obtained from current vendor quotes or from historical data.

H. Transportation and Waste Disposal

If waste disposal is required as part of a project, the waste classification must be identified. Based on the waste classification, disposal options can be identified. If waste is land filled, the nearest appropriate landfill can be identified so transportation and disposal costs can be calculated.

I. Environmental Management (EM) Considerations

The same principles used for the determination of direct costs associated with construction estimates can be applied to environmental restoration and waste management estimates. In addition to the factors to consider for construction projects, the following factors should also be considered for an EM project.

Some projects may have on-site capabilities such as laboratories. This can affect the project cost if it is to be set up for this project or if it is an existing facility. Special facilities such as decontamination units are required. These are a direct cost to the project.

B.2.6 Estimating Indirect Costs:

A. Each Indirect Cost Account

The indirect costs may be included as part of the code of accounts for a project. One method to estimate the indirect costs is to assign a cost to each cost account. This must be based on the size and type of contract and could be a lengthy list. This method requires a great deal of experience and a working knowledge of the construction firm's experience.

B. Percentage

A multiplier from a local data base or from published cost manuals can be developed.

B.2.7 Guidelines For Management Costs:

The estimate for management costs is largely a function of the duration of the project from the start through completion of construction.

A. Construction Management

A construction manager (CM) is responsible for construction activities. This responsibility includes subcontracting, purchasing, scheduling, and a limited amount of actual construction. Generally, CM costs are approximately 5 to 15 percent of the sum of the direct costs, indirect costs, and GFE whose installation is under the direction of the CM.

B. Project Management

The estimate for project management must consider the time element from start of design through completion of the construction for the project. Other factors to consider are the complexity of the project, the design group, the organization for which the project is to be performed, and the extent of procured items. Projects involving travel must also include those costs. Typically, project management costs range from 2 percent to 5 percent of the total project cost.

C. Construction Coordination

Construction coordination includes a field engineer. The field engineer should be involved in the review documents, as well as coordinate field construction. This function is generally estimated to be about 0.5 to 1 percent of the construction costs.

D. Quality Engineering

For the quality engineering estimate, the tasks for the project must be defined, and the man-hour effort with the quality organizations should be negotiated. The estimate will depend upon the quality level of the project, the amount of procurement effort, and level of involvement of the quality inspection organization. Where the latter is involved, quality engineering is responsible for the preparation of the quality plan. Travel must also be considered.

E. Health and Safety

This function is involved with the review and approval of the design package as well as the safety audits and health physics surveillance throughout the course of the construction period. Factors affecting this element are the type of project, operational area where the construction takes place, the amount of work requiring radiation surveillance, and any other special health and safety requirements. The portion of health safety that is an audit function is not funded by construction and need not be included in the estimate. This is typically estimated by taking from 0.5 to 1 percent of the total construction costs for conventional projects and would be more than that for a remediation job.

F. Environmental Restoration Management Contractor

Construction management costs for environmental restoration projects are those activity management services required to manage construction or cleanup activities, including review and approval of cleanup bid packages, review and acceptance of construction test procedures, control of field design change requests, and review and approval of contractor pay requests.

G. Program Management

Activity management associated with environmental restoration parallels construction project management. However, when estimating activity management, consideration must also be given to program management.

Program management consists of those services provided to the agency of a company on a specific program for planning, organizing, directing, controlling, budgeting, and reporting on the program. Program management will be provided as multiple levels within the EM program including the Headquarters, Operations Office, and installation.

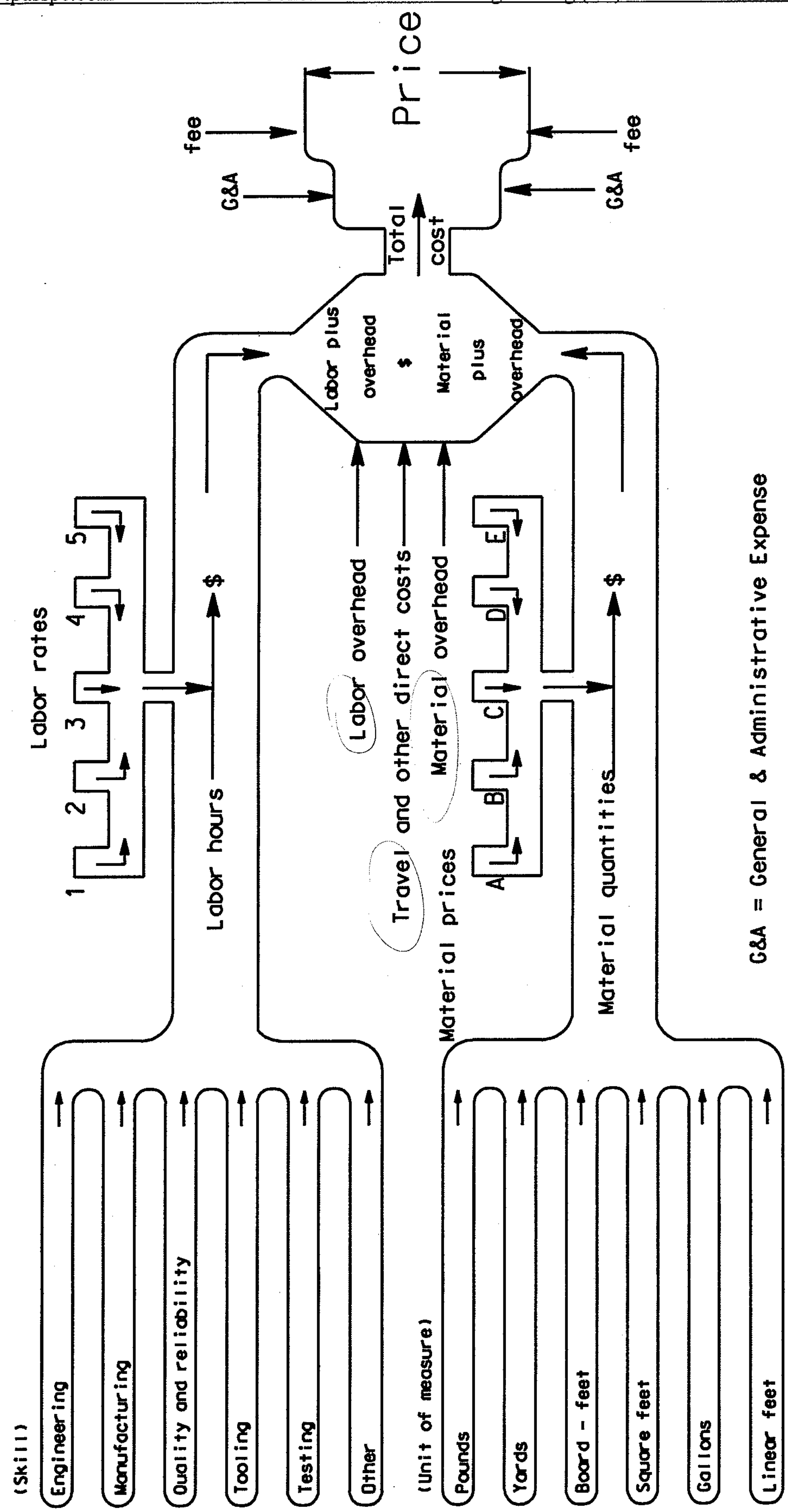


Figure B-2 Anatomy of an Estimate

ON TEST

Sample Problem B-1

***Given*:** A masonry wall is to be constructed using 8″×8″×16″ Concrete Masonry units (CMUs). The productive rate is 0.11 work hours /square foot and the Quantity takeoff (QTO) is 1,000 square foot. The crew consists of 3 masons and 2 helpers. It was determined that sixty percent of the hours will be worked by masons and forty percent by helpers.

Find: The mason and helper work hours are most nearly:

(A) 66 & 44 hours
(B) 44 & 66 hours
(C) 55 & 55 hours
(D) 60 & 50 hours

Solution:

Total work hours = (1000 sf/0.11 hr per sf) = 110 work hours
Mason work hours = 0.60 × 110 = 66 hours
Helper work hours = 0.40 × 110 = 44 hours

Answer: A ⇐

Sample Problem B-2

***Given*:** A masonry wall is to be constructed using 8″× 8″ × 16″ Concrete Masonry units (CMUs). The productive rate is 0.11 work hours per square foot and the quantity takeoff (QTO) is estimated to be 1,000 square foot. The crew consists of 3 masons and 2 helpers. It was determined that sixty percent of the hours will be worked by masons and forty percent by helpers. The bare cost per hour for mason and helper are $13.25and $10.00, respectively.

Find: The bare labor cost for constructing the masonry wall is most nearly:

(A) $1100.00
(B) $1215.00
(C) $1315.00
(D) $1415.00

Solution: The bare labor cost is calculated as:

Bare labor cost = Adjusted work hours × weighted average rate

The labor cost = (66 hrs × $13.25/hr) + (44 hrs × $10.00/hr) = $1314.50

Answer: C ⇐

Note: It was assumed that the productivity factor (PF) is 1.
The productivity factor is a combination of several variable such as:

i) Availability and productivity of workers (when work is plenty, workers are scarce and contractor will accept less-trained workers which means low productivity factor and vice versa when projects are scarce, contractor can be selective and hire only the most qualified workers, i.e. higher PF)
ii) Climatic Conditions (hot, cold, rain, snow, ..etc …all affect the PF)
iii) Working conditions as the location of the project.

Sample Problem B-3

Given: A gravity retaining wall is to be constructed at a new mall site in order to reduce the right of way take. The wall is 72 ft long × 12 ft high × 1 ft thick and will be built in three equal pours. Ignore overhead and profit. The following data will be used:

Table A- Productivity Rate

Item	Productivity per labor hr (LH)
Erect forms	5.5 ft^2/LH
Strip forms	15.00 ft^2/LH
Place concrete	2.2 yd^3/LH

Table B- Labor Cost

Item	$/hr
Carpenter	32.73
Laborer	26.08
Supervisor	35.37

Table C- Crews Required

Item	description
Erect and strip	4 carpenters 2 laborers 1 supervisor
Place Concrete	1carpenters 3 laborers 1 supervisor

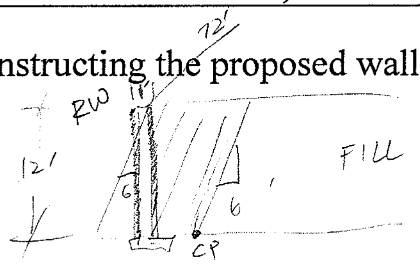
Table D- Materials Cost

Item	$/unit
Formwork Initial erection reuse	 2.66/ ft^2 0.34/ ft^2
Ready mix concrete	$97.20/$yd^3$
Reinforcing	$120.00/ yd^3

Assumptions: a) 10% waste for concrete b) material costs include taxes and delivery

Find: The total cost of constructing the proposed wall is most nearly:

(A) $17,231
(B) $19,853
(C) $20,675
(D) $22,993



Solution:

The sequencing of operations as follows: Know SCOPE OF WORK?

1- Erect form , 2- place rebar, 3- place concrete (1st pour), 4- Strip forms ...... and so on

Volume of Concrete = $(72 \times 12 \times 1)/27 = 32\ yd^3$ Actual wall

Volume of Concrete with 10 % waste = $1.10 \times 32 = 35.00\ yd^3$

- Average wage rate for erect and strip crew
  $= (4 \times \$32.73 + 2 \times \$26.08 + \$35.37)/7\,laborers = \$31.21/LH$
- Average wage rate for concrete crew
  $= (3 \times \$26.08 + \$32.73 + \$35.37)/5\ laborers = \$29.27/LH$

I) Erect forms: $72 \times 12 \times 2$ sides $= 1{,}728\ ft^2 / 5.5\ ft^2/LH = 314.18$ LH @ $31.21/LH = $9,806

II) Strip forms: 1,728 ft^2 /15 ft^2/LH = 115.2 LH @ $31.21/LH = $3,595

III) Reinforcing: 32 $yd^3 \times$ $120/ yd^3 = $3,840

(continued on next page)

+ waste

IV) Concrete cost: $35.00\ yd^3 \times \$97.20/\ yd^3 = \$3,402$

V) Place concrete: $(32\ yd^3/2.2\ yd^3/LH) \times \$29.27/LH = \$426$ cheaper because on site

pour 1st round

VI) Forms: $\frac{1}{3}(72\times12\times2)\$2.66/ft^2 + \frac{2}{3}(72\times12\times2)(\$0.34/ft^2) = \$1,924$

Total cost = \$9,806 + \$3,595 + \$3,840 + \$3,402 + \$426 + \$1,924 = \$22,993 Ans. D⇐

Comments Regarding to the Solution of Sample Problem B-3

The surface area of the retaining wall was calculated ignoring the two sides areas i.e. $1\times12 \times 2 = 24\ ft^2$ which is about 1.4 % more than the $1728\ ft^2$ used in the problem. This would change the cost as follows:

I) Erect forms: $72\times12\times2 + 1\times12\times2 = 1,752 ft^2/5.5\ ft^2/LH$
$= 318.54$ LH @ \$31.21/LH = \$9,942
II) Strip forms: $(1,728 + 24)\ ft^2/15\ ft^2/LH = 115.2$ LH @ \$31.21/LH = \$3,645
III) Reinforcing: $32\ yd^3 \times \$120/\ yd^3 = \$3,840$
IV) Concrete cost: $35.00\ yd^3 \times \$97.20/\ yd^3 = \$3,402$
V) Place concrete: $(32\ yd^3/2.2\ yd^3/LH) \times \$29.27/LH = \$426$

VI) Forms: $\frac{1}{3}(72\times12\times2+12\times1\times2)\$2.66/ft^2 + \frac{2}{3}(72\times12\times2+12\times1\times2)(\$0.34/ft^2) = \$1,951$

Total cost = \$9,942 + \$3,645 + \$3,840 + \$3,402 + \$426 + \$1,951 = \$23,206

This is less than 1% of the cost ignoring the two sided areas of the retaining wall.

B.2.8 Transportation and Handling Materials:

Although material suppliers generally deliver to the jobsite the construction materials that have been purchased by the contractor, sometimes the materials must be obtained by the contractor at the storage yard of the material supplier. Also, the contractor must move material from stockpiles on the jobsite to the location where the material will be permanently installed.

Some projects require the use of aggregates, sand and gravel, or crushed stone, which are produced from natural deposits or quarries and hauled to the project in trucks. A contractor using his or her laborers and equipment may do the handling and hauling, or it may be accomplished through a subcontractor. Regardless of the method used, it will involve a cost that must be included in the estimate for a project.

When estimating the time required by a truck for a round-trip, the estimator should divide the round-trip time into four elements:

1- Load
2- Haul, loaded
3- Unload
4- Return, empty

These four elements define the cycle time for transporting material. The time required for each element should be estimated. If elements 2 and 4 require the same time, they can be combined. Since the time required for hauling and returning will depend on the distance and effective speed, it is necessary to determine the probable speed at which a vehicle can travel along the given haul road for the conditions that will exist. Speeds are dependent on the vehicle, traffic congestion, condition of the road, and other factors. An appropriate operating factor should be used in determining production rates. For example, if a truck operate only 45min/hr, the time should be used in determining the number of round – trips the truck make in 1 hr.

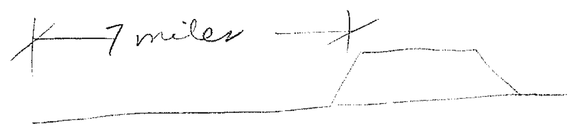
Sample Problem B-4

***Given*:** 175 tons of sand with a density of 100 lb/cf must be transported 7 mi using a 12-cy dump truck. Two laborers and a driver, at a rate of 1.5 cy/hr each, will load the truck. Assume a haul speed of 30 mph, return speed of 40 mph, and 3 min to dump the load. The cost of the truck is $25/hr, the driver is $18/hr, and the laborers cost $15/hr each. Assume 45 min/hr as productive.

Find: The total time to haul the sand is most nearly:

(A) 32.1 hrs
(B) 38.2 hrs
(C) 45.1 hrs
(D) 48.1 hrs

Solution:



***Quantity of work*:**

Volume of sand = 175 T × 2,000 lb/T × cf/100 lb × cy/27cf = 129.63 = 130 cy
Cycle time:

Load	= 12 cy/(3×1.5 cy/hr)	=	2.667 hrs
Haul	= 7 mi/30 mph	=	0.233 hrs
Dump	= 3 min/(60 min/ hr)	=	0.050 hrs
Return	= 7 mi/40 mph	=	0.175 hrs
	Total cycle time	=	3.125 hr/trip

production rate:

Number of trips per hour = 1.0/(3.125 hr/trip) = 0.32 trips/hr
Quantity hauled per trip = 12 cy/trip × 0.32 trips/hr = 3.84 cy/hr
Production rate = 3.84 cy/hr × 45/60 = 2.88 *cy/hr*

Time:

Using 1 truck and 2 laborers = 130 cy/2.88 cy/hr = 45.1 hrs

Answer: C ⇐

Sample Problem B-5

Given: 175 ton of sand with a density of 100 lb/cf must be transported 7 mi using a 12-cy dump truck. Two laborers and a driver, at a rate of 1.5 cy/hr each, will load the truck. Assume a haul speed of 30 mph, return speed of 40 mph, and 3 min to dump the load. The cost of the truck is \$25/hr, the driver is \$18/hr, and the laborers cost \$15/hr each.

Find: The total cost to haul the material is most nearly:

(A) \$3,292.30
(B) \$3,392.30
(C) \$3,492.30
(D) \$3,592.30

Solution:

Using the time calculated in the previous problem 45.1 hr, the cost is calculated as follows:

Truck	= 45.1 hr × 1 truck @ \$25/hr	= \$1,127.50
Driver	= 45.1 hr × 1 driver@\$18/hr	= \$ 811.80
Laborers	= 45.1 hr × 2 laborers @ \$15/hr	= \$1,353.00
Total cost		= \$3,292.30

Answer A ⇐

Sample Problem B-6

Given: 175 tons of sand with a density of 100 lb/cf must be transported 7 miles using a 12-cy dump truck. Two laborers and a driver, at a rate of 1.5 cy/hr each, will load the truck. Assume a haul speed of 30 mph, return speed of 40 mph, and 3 minutes to dump the load. The cost of the truck is \$25/hr, the driver is \$18/hr, and the laborers cost \$15/hr each.

Find: The cost per ton for transporting the material is most nearly:

(A) \$18.61
(B) \$18.81
(C) \$18.91
(D) \$19.21

Solution:

Unit cost:

Cost per cubic yard = \$ 3,292.30 / 130 cy = \$ 25.33/ cy
Cost per ton = \$ 3,292.30 / 175 T = \$ 18.81/ *T*

The time, cost, and cost per unit of work are high using only laborers to load the sand. These high costs are a result of the truck sifting idle while the laborers are loading the truck

Answer: B ⇐

B.2.9 CONSTRUCTION EQUIPMENT COSTS

The hourly cost for a piece of construction equipment consists of the following two cost components:

1- Owning costs, and
2- Operating costs.

Table B- 3 Construction Equipment Cost Components

Owning Costs	Operating Costs
1) Investment (interest) cost 2) Depreciation 3) Insurance 4) Taxes 5) Storage costs	1) Operators's wages 2) Fuel cost 3) Services cost (filters, grease, oil, hydraulics fluids, ..) 4) Tire cost 5) Repair cost

Investment Cost

Investment cost is the annual cost of the capital invested in the piece of equipment. This is equal to the interest paid on the loan each year. If no loan was used to purchase the equipment, this cost is the interest that could be realized by the owner if the assets were invested.

Insurance, Taxes, and Storage Costs

The cost of owning equipment includes fire, theft, accident, and liability insurance; property taxes; any required licenses; and the cost to store the equipment. Actual costs should be used, but for estimating purposes, a percentage can be added to the investment rate and multiplied by the average investment.

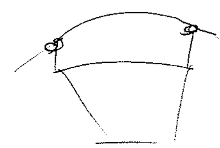
Chapter C: Scheduling

Scope
* Schedule
* Costs

C.1 CONSTRUCTION SEQUENCING

A construction sequence schedule is a specified work schedule that coordinates the timing of ALL activities to ensure that the work would be performed correctly in a minimum amount of time and least cost. To illustrate the construction sequencing, the following study cases are given:

(pg 172)



Study Case # 1: Installation of Erosion and Sedimentation Control

Purpose

- To reduce on-site erosion and off-site sedimentation from land-disturbing activities by installing erosion and sedimentation control practices in accordance with a planned schedule.

- Preserving the natural vegetation on-site to the maximum extent.

Effectiveness

All land development that clears, grades or fills a significant land area. The removal of existing surface ground cover leaves a site vulnerable to accelerated erosion. Good planning will: a) Reduce land clearing b) Provide necessary controls and c) Restore protective cover.

Planning

The purpose of the construction sequence schedule is to address erosion prevention and sediment control in an efficient and effective manner. Appropriate sequencing of construction activities can be a cost-effective way to help accomplish this goal. The plan can be open to changes that should be discussed at the erosion control project meetings.

Construction

Many timely construction techniques, such as shaping earthen fills daily to prevent overflows and constructing temporary diversions ahead of anticipated storms, can reduce the erosion potential of a site. These type of activities cannot be put on the construction sequence but should be used whenever possible.

Maintenance

Follow the construction sequence throughout project development. When changes in construction activities are needed, amend the sequence schedule in advance to maintain management control. Orderly modification assures coordination of construction and erosion-control practices to minimize erosion and sedimentation problems. When major changes are necessary, you may want to send a copy of the modified schedule to the local permitting authority.

#	Construction Activity	Schedule Consideration
1	Identify and label protection areas (e.g., buffer zones, filter strips, trees).	Site delineation should be completed before construction begins.
2	Construction access. Construction entrance, construction routes, equipment parking areas and cutting of vegetation (necessary perimeter controls).	First land-disturbing activity: Establish protected areas and designated resources for protection. Stabilize bare areas immediately with gravel and temporary vegetation as construction takes place.
3	Sediment traps and barriers. Basin traps, sediment fences, and outlet protection (necessary perimeter controls).	Install principal basins after construction site is accessed. Install additional traps and barriers as needed during grading.
4	Runoff control. Diversions, silt fence, perimeter dikes, water bars, and outlet protection.	Install key practices after principal sediment trap sand before land grading. Install additional runoff control measures during grading.
5	Runoff conveyance system. Stabilize stream banks, storm drains, channels, inlet and outlet protection, and slope drains.	Where necessary, stabilize stream banks as early as possible. Install principal runoff conveyance system with runoff-control measures. Install remainder of system after grading.
6	Grubbing and grading. Site preparation : cutting, filling and grading, sediment traps, barriers, diversions, drains, surface roughening.	Begin major grubbing and grading after principal sediment and key runoff control measures are installed. Clear borrow and disposal areas only as needed. Install additional control measures as grading progresses.
7	Surface stabilization: temporary and permanent seeding, mulching, sodding and installing riprap.	Apply temporary or permanent stabilization measures immediately on all disturbed areas where work is delayed or complete.
8	Building construction: buildings, utilities, paving.	Install necessary erosion and sedimentation control practices as work takes place.
9	Landscaping and final stabilization: topsoiling, planting trees and shrubs, permanent seeding, mulching, sodding, installing riprap.	Last construction phase - Stabilize all open areas, including borrow and spoil areas. Remove and stabilize all temporary control measures.
10	Maintenance	Maintenance inspections should be performed weekly, and maintenance repairs should be made immediately after periods of rainfall.

Study Case # 2: Repairs for Historic Cladding Systems

Construction Sequencing: Understanding the Implication of Failure and Effective Repairs for Historic Cladding Systems

To effectively define and repair problems that develop in facade cladding systems, the design professional must understand the original construction sequencing, and how the repair construction sequencing will influence the cost, performance, and risk of damage to the facade cladding system. Repair details that do not consider the execution of the construction may lead to short-term difficulties including damage and cost concerns during the assembly and long term durability concerns including increased distress that may cause increased maintenance requirements, localized failures or even catastrophic failures. All too often designers detail repairs without thought as to how and when the work will actually be performed. As a result, difficulties arise during construction because of lack of forethought on critical issues such as tolerances, access requirements, temperature restrictions, owner requirements, interface between trades, and material and system compatibility issues. Anticipating potential constructability conflicts during the design of the repairs reduces the number of issues that develop, and minimize delays and extra costs.

Study Case # 3: Construction Sequencing Alternatives

Formalization of Construction Sequencing Rationale and Classification Mechanism to Support Rapid Generation of Sequencing Alternatives

Resequencing construction activities is a critical task for project planners for effective project control. Resequencing activities require planners to determine the impact or "role" an activity has on successor activities. They also need to determine the status of activities, i.e., which activities may or may not be delayed. Distinguishing the role and status of activities in turn requires planners to understand the rationale for activity sequences. The current critical path method (CPM) framework, however, represents sequencing rationale using precedence relationships and distinguishes activities only with respect to their time-criticality. Thus, planners find it difficult to keep track of individual sequencing logic, and manually inferring the role and status of activities becomes practically prohibitive in complex project schedules. The research presented in this paper addressed this limitation of the CPM framework by formalizing a constraint ontology and classification mechanism. The ontology allows planners to describe their rationale for activity sequences in a consistent and intuitive way, whereas the classification mechanism leverages the ontology to automatically infer the role and status of activities. The ontology and mechanisms were implemented in a prototype tool. With this tool, users can quickly verify which activities to delay to expedite critical milestone or bottleneck activities, thus making it possible to quickly evaluate and generate sequencing alternatives in CPM-based schedules.

Study Case # 4: Soil Erosion and Sediment Control

TYPICAL CONSTRUCTION SEQUENCING

1) Installation of soil erosion and sediment control

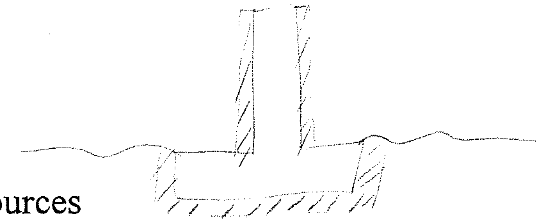
    a) Selective vegetation removal for silt fence installation
    b) Silt fence installation
    c) Construction fencing around areas not to be disturbed
    d) Stabilized construction entrance

2) Tree removal where necessary (clear & grub)
3) Construct sediment trapping devices (sediment traps, basins…)
4) Construct detention facilities and outlet control structure with restrictor & temporary perforated riser
5) Strip topsoil, stockpile topsoil and grade site
6) Temporarily stabilize topsoil stockpiles (seed and silt fence around toe of slope)
7) Install storm sewer, sanitary sewer, water and associated inlet & outlet protection
8) Permanently stabilize detention basins with seed and erosion control blanket
9) Temporarily stabilize all areas including lots that have reached temporary grade
10) Install roadways
11) Permanently stabilize all outlot areas
12) Install structures and grade individual lots
13) Permanently stabilize lots
14) Remove all temporary SE/SC measures after the site is stabilized with vegetation

- Soil erosion and sediment control maintenance must occur every two weeks and after every ½″ or greater rainfall event.

C.2 RESOURCE SCHEDULING

Definition: Resource scheduling (or resource leveling) is a way of determining schedule dates on which activities should be performed.



Objectives of resource scheduling:

1- to smooth demand for resources
2- to minimize the variations in resources demand
3- to avoid exceeding the limits or availability of resources
4- to improve efficiency

How to perform resource scheduling?

1- examine resource requirements during specific periods of the project
2- modify activities within available float, i.e. modify resource loading for each unit of time (day, week or month)
3- redistribute or substitute resources within all activities

Effect of constraints on resource scheduling:

- if the project is time limited → the total cost may increase
- if the project is cost limited → you may have to "find" more resources when needed or even "cut" the "project scope"
- if the project is resource limited → you may have to negotiate more time, more money, or both

How to improve resource scheduling:

- apply minimum late start time
- apply ~~minimum~~ max late finish time
- look for the highest resource demand
- redistribute resources to achieve the best usage of the resources
- look for opportunities for parallel activities i.e. provide more float
- cascade activities that need similar skills to improve efficiency and avoid "downtime"

Integrated Cost/Schedule/Work:

The relationship between time and work for any project could be established by generating the relationships between cost/time and work/time. However, evaluating these relationships separately does not provide an accurate status of the project. A cost/schedule/work graph can be prepared that shows the integrated relationship of the three basic components (***three constraints***) of the project:

a- scope (work)
b- cost(budget)
c- schedule (time)

The following figures (***4 cases***) link costs on the left hand ordinate, time on the abscissa, and the work on the right hand ordinate. The upper curve is the cost/time relationship which is known as ***S-Curve*** and the lower work/time curve shows the relationship between work and time throughout the duration of the project.

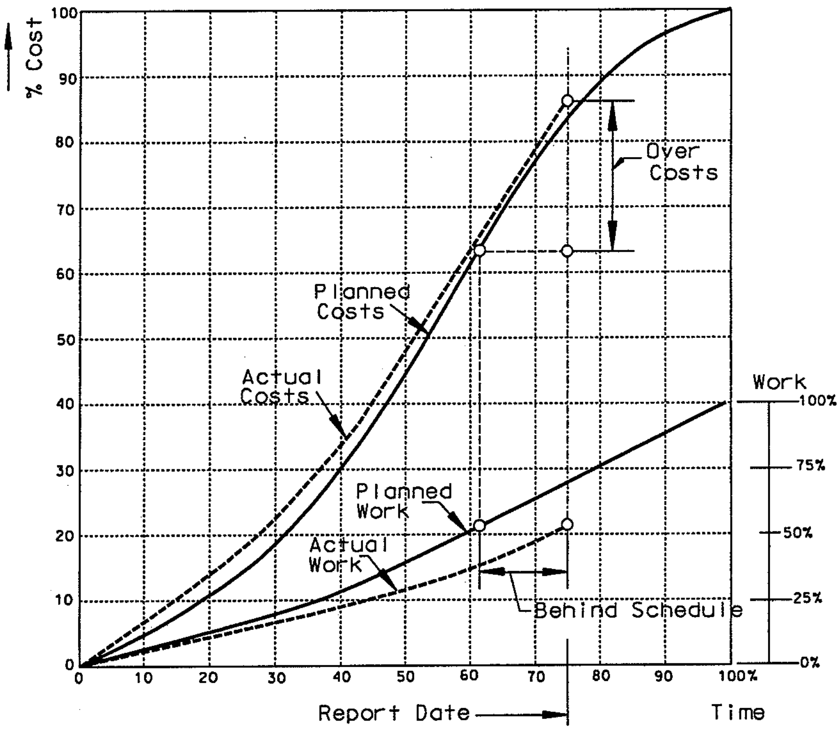


Figure C-1 (Case# 1) Over Costs and Behind Schedule

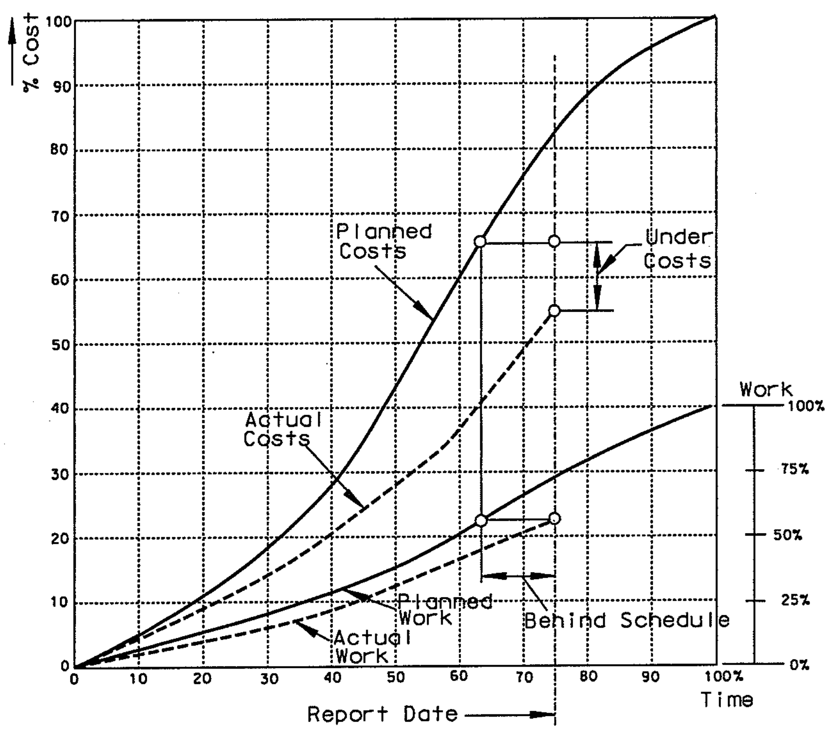


Figure C-2 (Case# 2) Under Costs and Behind Schedule

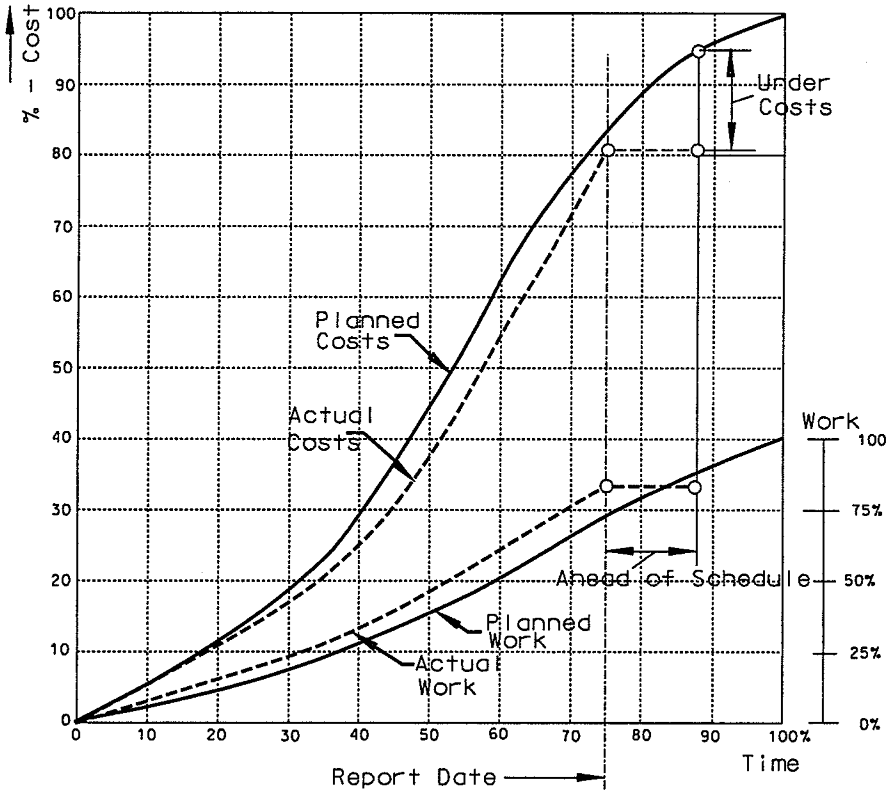


Figure C-3 (Case# 3) Under Costs and Ahead of Schedule

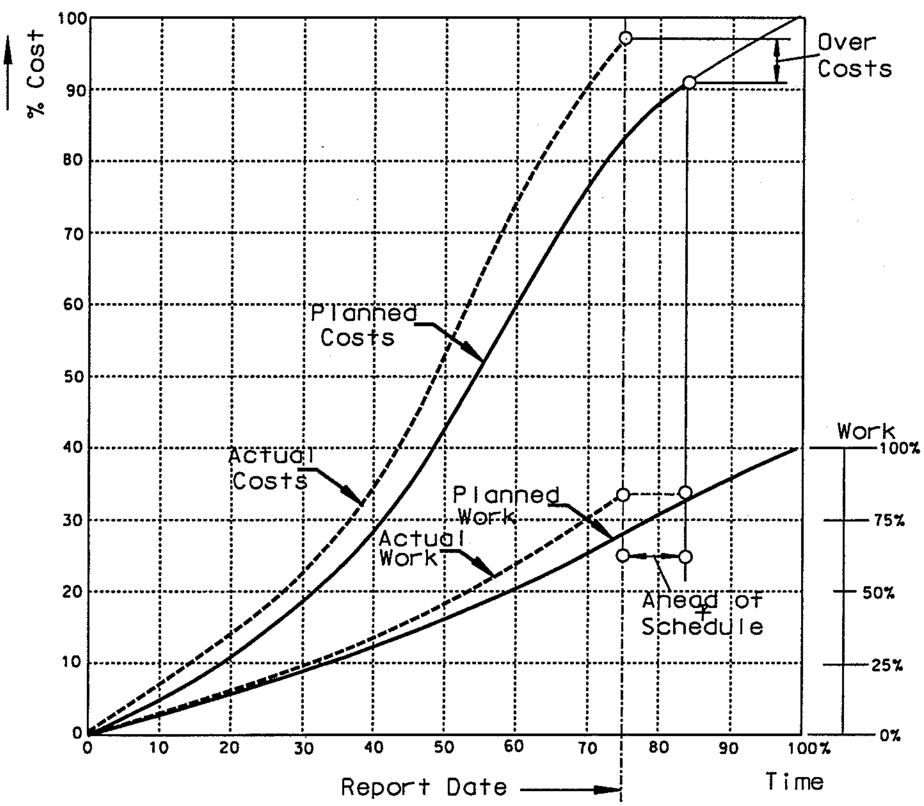


Figure C-4 (Case# 4) Over Costs and Ahead of Schedule

Cost and Schedule Variance: (Pg. 265 #21)

Performance against budget is measured by comparing what was done to what was paid. This compares ***earned-hours*** to actual work-hours or cost. If more was paid than done, then the project would have a cost overrun of the budget. The following variance and index equations can be used to calculate these values as follows:

Scheduled Variance (SV) =

(Earned work-hours or dollars) – (Budgeted work-hours or dollars)

$$SV = BCWP - BCWS \tag{22}$$

Schedule Performance Index (SPI) =

$$\frac{(Earned\ work - hours\ or\ dollars\ to\ date)}{(Budgeted\ work - hours\ or\ dollar\ to\ date)} = \frac{BCWP}{BCWS} \tag{23}$$

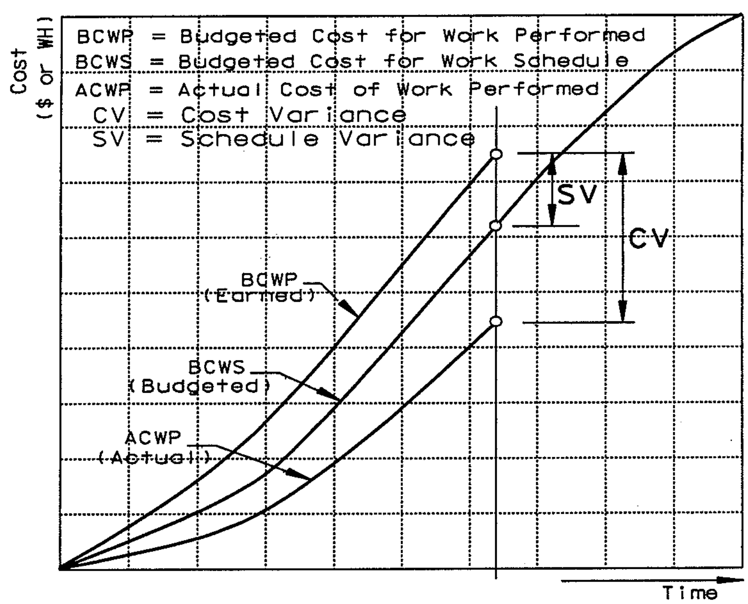


Figure C-5 Cost and Schedule Variance Curve

Cost Variance (CV) =

(Earned work-hours or dollars) – (Actual work-hours or dollars)

$$CV = BCWP - ACWS \tag{24}$$

Cost Performance Index (CPI) =

$$\frac{(Earned\ work - hours\ or\ dollars\ to\ date)}{(Actual\ work - hours\ or\ dollar\ to\ date)} = \frac{BCWP}{ACWS} \tag{25}$$

C.3 TIME-COST TRADE-OFF

Definition: time/cost trade-off analysis is the compression of the project schedule to achieve a more favorable outcome in terms of project duration, cost, and projected revenues.

Objectives of time/cost trade-off analysis:

1- compress project to an acceptable duration
2- minimize total project costs

How to perform time/cost trade-off analysis?

Done by selectively crashing specific activities to shorten project duration

Steps to perform time/cost trade-off analysis:

1- estimate project-based (indirect) cost per unit time

2- select critical activity (or activities, if there are multiple critical paths) that are good candidates for crashing (use lowest acceleration cost heuristic)

3- incrementally crash (i.e., shave a day off of) the selected activity where that is possible (or activities, if there are multiple critical paths and no bottleneck activities that are economical to crash)

4- keep track of the activity-based (direct) cost of crashing selected activity (or activities) and indirect cost savings associated with reducing overall project duration

5- recalculate the forward pass and check for changes in critical path

6- continue until acceptable duration is reached or it becomes uneconomical to continue crashing the project

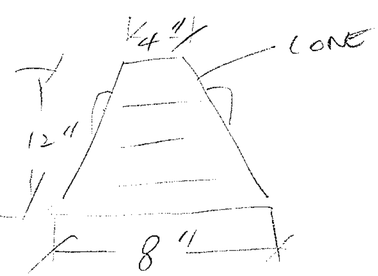
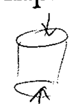
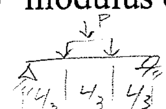
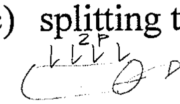
Chapter D: Material Quality Control and Production

D-1 MATERIAL TESTING (e.g., CONCRETE, SOIL, ASPHALT)

D-1.1 Concrete

Concrete testing may be performed for ***freshly mixed*** concrete or for ***hardened concrete*** (after curing had been completed fully or partially). The following table summarizes these properties and the factors related to these properties.

Table D-1 Fresh and Hardened Concrete Properties

	Freshly Mixed Concrete	**Hardened Concrete (mechanical properties)**
Properties	a) workability(slump test/ Kelly ball test), b) unit weight (w_c , γ_c) Normal Concrete 150 lb/ft3 c) temperature ($^\circ C$, $^\circ F$) 	a) compressive strength (cylinder test 6″ ×12″), f_c'  $f_c' = \frac{P}{A}$ (lab testing) b) modulus of rupture (beam test), f_r Ca 48 Lindeburg  $f_r = \frac{Mc}{I}$ (lab testing) $f_r = 7.5\sqrt{f_c'}$ (empirical formula ACI 318 Eq. 9-10) c) splitting tensile strength (Brazilian test), f_{ct}  $f_{ct} = \frac{2P}{\pi DL}$ (lab testing) The following three empirical equations given by the ACI 318 ***(§5.1.4 & §11.2)*** $f_{ct} = 6.7\sqrt{f_c'}$ (*normal weight concrete*) $f_{ct} = 5.7\sqrt{f_c'}$ (*sad weight concrete*) $f_{ct} = 5.0\sqrt{f_c'}$ (*all weight concrete*)
Factors Affected by the Properties	1- method of construction, 2- transporting, and 3- finishing methods and techniques.	1- type of the structure 2- functions of the structures 3- environmental conditions

D-1.2 Soil

$\frac{wt}{V} = \frac{lb}{ft^3}$

***Proctor Testing*:**

In order to achieve proper soil density, the results of a laboratory Proctor (AASHTO T99) or modified Proctor (T180) compaction test are plotted. Both tests involve compacting a sample of the soil at various moisture contents. The Proctor test utilizes a 5.5 lb hammer dropping 12 inches 25 times on each of three equal layers of soil.

The soil is placed in a 4-inch-diameter compaction cylinder with a total volume of 1/30 cubic foot. This produces 12,400 ft-lb/cu ft of compactive effort. The modified Proctor test uses a 10 lb hammer dropping 18 inches on each layer in the same mold. This produces 56,200 ft-lb/cu ft of compaction.

After compaction, the sample is weighed, and the ***wet density*** is calculated as:

$$Wet\ Density = \frac{Weight\ of\ soil\ and\ mold - weight\ of\ mold}{(\text{Volume of mold})} \qquad (26)$$

The ***moisture content (M.C.)*** is determined by taking a sample (from the Proctor test specimen), weighed and dried and weighed again.

$$Mositure\ Content\ (w) = \frac{Weight\ of\ wet\ soil\ - weight\ of\ dry\ soil}{(\text{weight of dry soil})} \qquad (27)$$

The dry density is calculated as follows:

$$Dry\ Density = \frac{Wet\ density}{(1 + w/100)} \qquad (28)$$

Samples are compacted at five different moisture contents, and the dry densities are plotted versus moisture content. The peak of the curve is designated as the ***Maximum Dry Density (MDD)***, while the moisture content corresponding to this density is the ***Optimum Moisture Content (OMC)***. It is this moisture content that helps to achieve the maximum density. The OMC ranges from 6% to 12% for coarse-grained soils, and from 10% to 25% for fine-grained soils. The following figure shows compaction curves for a soil compacted by standard and Curves modified Proctor test.

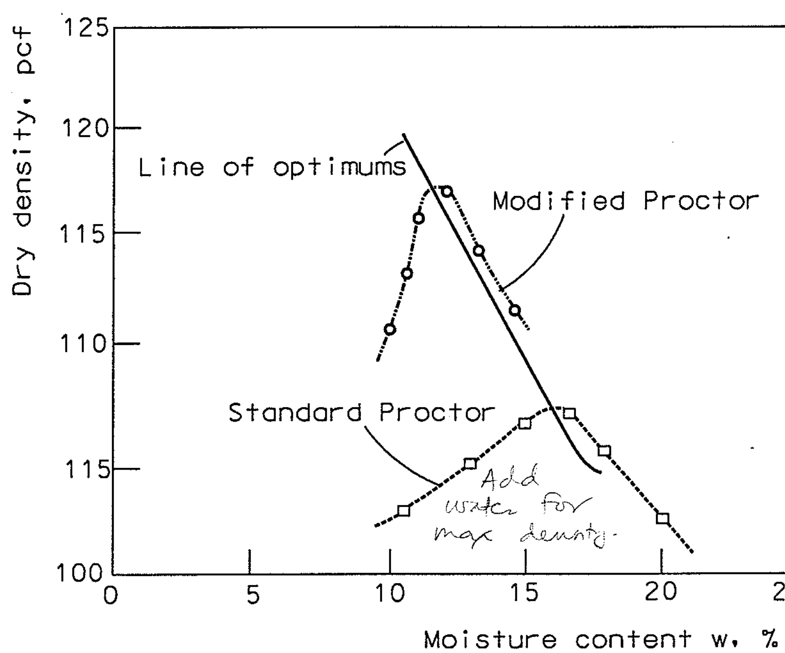


Figure D.1 Standard and Modified Proctor Compaction

The following information are to be used for problems D-1 to D-3

A sample has been compacted using standard Proctor compaction. The soil and mold weigh 12.95 lb, and the empty mold weighs 9.1 lb. A sample of this soil weighs 0.24 lb wet and 0.21 lb after drying.

Sample Problem D-1: Wet Density of Soil

Find: the wet density of this sample is most nearly:

(A) 115.5 pcf
(B) 117.5 pcf
(C) 119.5 pcf
(D) 121.5 pcf

Solution:

The volume of the mold for a standard Proctor is = 1/30 ft^3

$$Wet\ Density = \frac{Weight\ of\ soil\ and\ mold - weight\ of\ mold}{\text{(Volume of mold)}} = \frac{(12.95-9.1)\ lb}{(1/30)\ ft^3} = 115.5\ pcf$$

Assume! Answer: A ⇐

Sample Problem D-2: Moisture Content of Soil

Find: the moisture content of this sample is most nearly:

(A) 13.50 %
(B) 14.30 %
(C) 15.40 %
(D) 17.50 %

Solution:

$$Mositure\ \ Content\ (w) = \frac{Weight\ \ of\ wet\ soil\ \ - weight\ of\ dry\ soil}{\text{weight of dry soil}}$$

$$= \frac{0.24-0.21}{0.21} \times 100 = 14.29\ \%$$

Answer: B ⇐

Sample Problem D-3: Dry Density of Soil

Find: the dry density of this sample is most nearly:

(A) 90.98 pcf
(B) 100.35 pcf
(C) 101.10 pcf
(D) 115.65 pcf

Solution:

The volume of the mold for a standard Proctor is = $1/30\, ft^3$

$$\boxed{Dry\ Density} = \frac{Wet\ density}{(1+w/100)} = \frac{115.5\ pcf}{(1+14.29/100)} = 101.1\ pcf$$

Answer: C ⇐

Optimum Moisture Content (OMC):

In the field, OMC is too narrow a target to achieve. Therefore, a ***four percentage point*** range of moisture is typically specified, centered on the OMC. Additionally, a density range is specified based on the nature of the soil and the desired strength. A minimum density is specified, ***usually 90% of the maximum for a cohesive, fine-grained soil or 95% of the maximum for a noncohesive soil***. Although the minimum value is usually designated, a maximum density 5% above the minimum is often designated. This upper limit avoids shear problems in clay type soils.

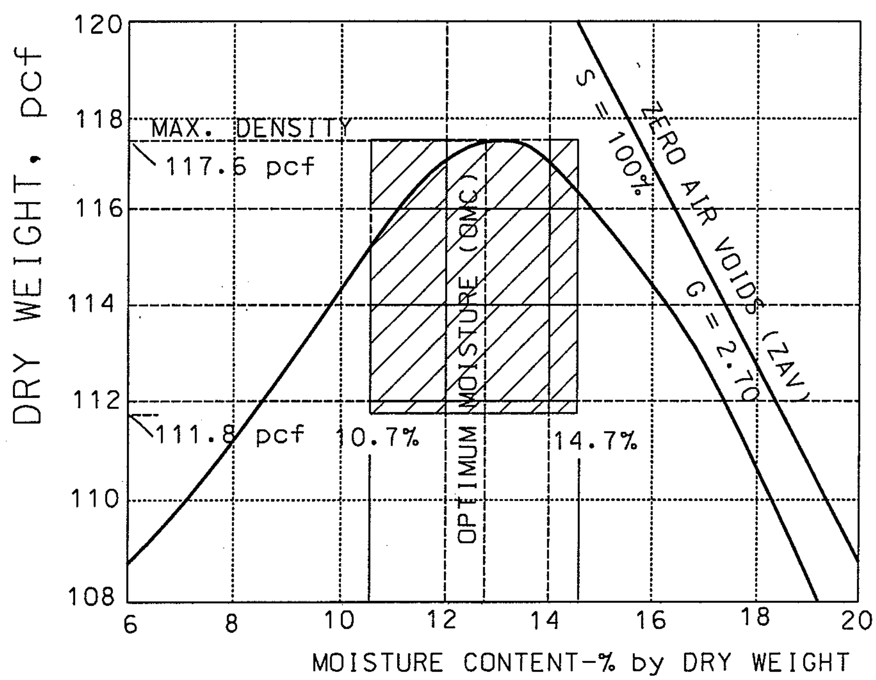


Figure D.2 Proctor Curve with Specific Tolerance (±4% of OMC)

Moisture Content and Density Determination in the Field:

Moisture content and density are measured in the field using the following two methods:

a) sand cone, or
b) nuclear moisture-density gauge.

The sand cone is used to determine the volume of a hole from which compacted soil has been removed. Dividing the weight of soil removed by the volume of the sample (hole) will produce wet density. If a small sample is dried to determine the moisture content, the dry density can be calculated using the equations listed above.

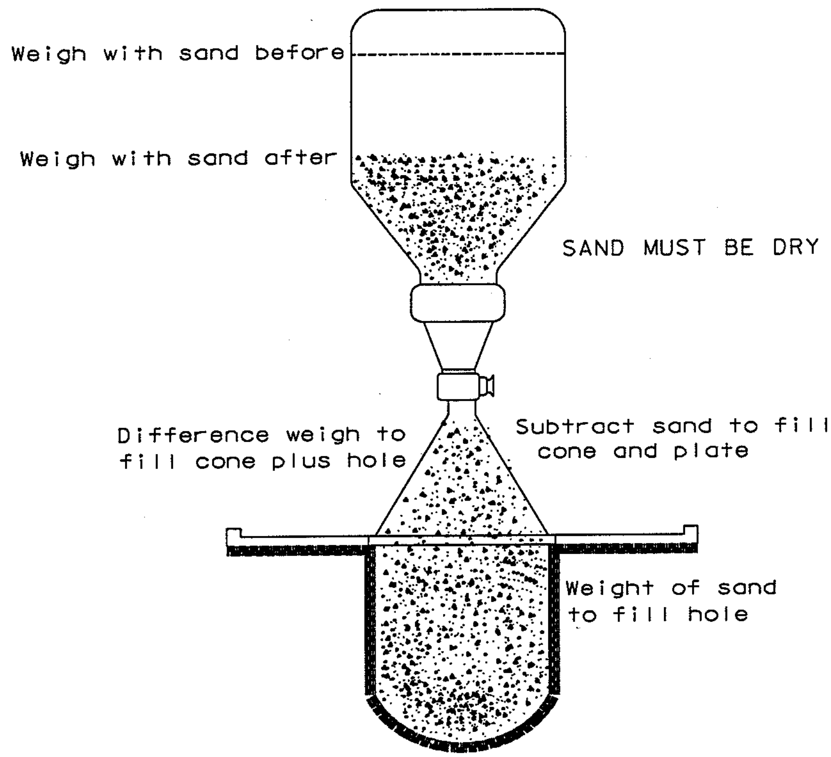


Figure D.3 Density Determination using Sand Cone in the Field

Quantity of Water Needed in the Field:

The quantity of water to be added is based on the dry weight of the soil and the compacted volume of the lift. This is usually measured by the number of gallons per 100-foot station, as calculated by the following equation:

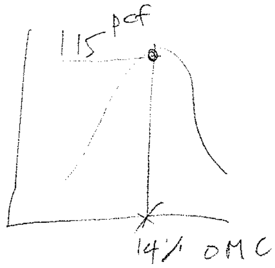
$$Gallons\ of\ Water = Desired\ dry\ density\,(pcf) \times \frac{Goal\ water\ content\ \% - Existing\ water\ content\,\%}{100} \times \frac{Compacted\ cubic\ feet\ of\ soil}{8.33\,lb/gal} \qquad (29)$$

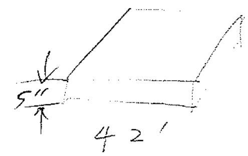
Sample Problem D-4: Amount of Water Needed in the Field

Given: A new highway construction project requires a dry density of 115 pcf at an Optimum Moisture Content (OMC) of 14%. The soil is compacted in a 5-inch-thick lift, 42 feet wide. In-place tests indicate a moisture content of 4%.

Find: The water must be added to attain the proper moisture content for compaction is most nearly:

(A) 3.67 gal/yd^2
(B) 4.55 gal/yd^2
(C) 5.44 gal/yd^2
(D) 6.54 gal/yd^2





Solution:

$$Gallons\,of\,Water = Desired\,dry\,density\,(pcf) \times \frac{Goal\,water\,content\,\% - Existing\,water\,content\,\%}{100}$$

$$\times \frac{Compacted\,cubic\,feet\,of\,soil}{8.33\,lb/gal}$$

$$= 115\,pcf \times \frac{\overset{OMC}{14\%} - 4\%}{100} \times \frac{42\,ft \times 100\,ft\,(sta.) \times (5\,in/12\,in/ft)}{\underset{water}{8.33\,lb/gal}} = 2416\,gal/sta.$$

$$Gallons/yd^2 = \frac{2416\,gal/sta}{(42\,ft \times 100\,ft\;sta)/9\,ft^2/yd^2} = \underset{5.18}{5.44}\,gal/yd^2$$

Answer: C ⇐

Compactor Selection & Production:

Different types of compaction equipment work well in different soil types. Compaction must be matched to the rate of spreading of the material. Typically, a test strip is rolled and tested to determine how many passes of the designated roller provide the desired density. Roller production is calculated by the following equation:

$$Compacted\ cubic\ yards/hour = \frac{16.3 \times W \times S \times L \times Efficiency}{N} \qquad (30)$$

Where:

W = compacted width in feet per pass
S = roller speed, mph
L = compacted lift thickness, inches
N = number of passes required

D-1.3 Asphalt (see the AC Pavement Design)

D-1.4 Nuclear determination of moisture density of soils:

Nuclear methods are used extensively to determine the moisture-density of soils. The instrument required for this test can be transported readily to the fill, placed at a location where a test is to be conducted, and within a few minutes the results can be read directly from the indicators.

The device utilizes the Compton effect of gamma-ray scattering for density determinations and hydrogenous thermalization of fast neutrons for moisture determinations. The emitted rays enter the ground, where they are partially absorbed and partially reflected. Reflected rays pass through Geiger-Miller tubes in the surface gauge. Counts per minute are read directly on a reflected-ray counter gauge and are related to moisture and density calibration curves.

Instruments are available that will measure moisture and density of soils at depths up to 200 ft or more below the surface of the ground. The measurements are performed by drilling holes in the soil to the desired depths, installing aluminum tubing in the holes temporarily, then lowering nuclear sensing probes down the tubing and making the tests at the desired depths

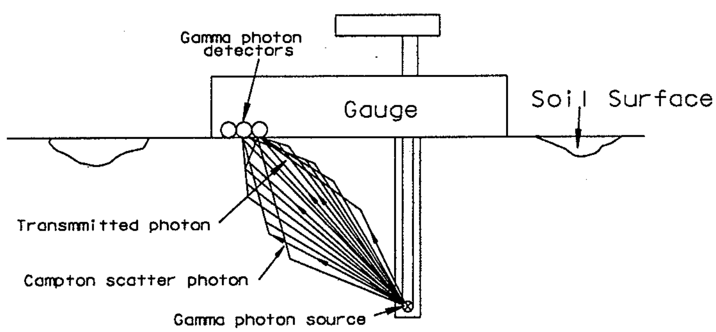


Fig D-4 Nuclear gage

Advantages of the nuclear method when compared with the Proctor method include the following:

1. Does not require the removal of soil samples from the site of the tests.
2. Decreases the time required for a test from as much as a day to a few minutes, thereby eliminating potentially excessive delays for the contractor, e.g., waiting for the results from Proctor tests.
3. Provides a means of performing density tests on soils containing large-sized aggregates and on frozen materials.
4. Reduces or eliminates the effect of the personal element, and possible errors, that may occur in performing Proctor tests.

Because nuclear tests are conducted with instruments that present a potential source of radiation, an operator should exercise reasonable care to assure that no harm can result from the use of the instruments. However, by following the instructions furnished with the instruments and exercising proper care, exposure can be maintained well below the limits set by the Nuclear Regulating Commission (NRC). In the United States a license is required to own, possess, or use nuclear-type density- and moisture-measuring instruments. A license may be obtained from the Nuclear Regulating Commission, and where required from state and/or local government agencies.

Sample Problem D-5: Concrete, Soil and Asphalt Testing

Given: Which of the following statement (s) is (are) not true about the nuclear gages used for measuring soil compaction:

I) Nuclear gages use gamma-ray to measure density and moisture up to depth of 200 ft,
II) Nuclear gages should be transported in a properly labeled carrying case,
III) The results of the nuclear gage tests need 24 hrs to be available
IV) A license is required to own, possess, or use nuclear-type density and moisture-measuring instruments.

(A) I
(B) II & III
(C) I & IV
(D) III

Solution:

Statement I: TRUE, nuclear gages use low- level radioactive material as gamma-rays. Also the gages are used to measure density and moisture up to a depth of 200 ft.

Statement II: TRUE, nuclear gages should be handled with care and a protective case properly labeled should be used.

Statement III: NOT TRUE, the result of nuclear gages can be read directly from the indicators within a few minutes. In fact, this is one of the advantages of the nuclear gages.

Statement IV: TRUE, a license is required to own, possess, or use nuclear-type density- and moisture-measuring instruments. A license may be obtained from the Nuclear Regulating Commission (NRC), and where required from state and/or local government agencies.

Answer: D⇐

Sample Problem D-6: Concrete, Soil and Asphalt Testing

Given: Which of the following statement (s) is (are) true about the nuclear gages used for measuring soil compaction:

destructive vs non destructive

I. Nuclear gages can only measure density of soils
II. Tests using nuclear gages are considered destructive tests
III. Nuclear gages can be used only for fine graded soils with no presence of large- sized aggregates
IV. A license is required to own, possess, or use nuclear-type density and moisture-measuring instruments.

(A) I & II
(B) II & III
(C) I, II & III
(D) IV

Solution:

Statement I: NOT TRUE, nuclear gages are used to measure density and moisture up to a depth of 200 ft.

Statement II: NOT TRUE, nuclear gages are nondestructive tests because there is no need to remove soil samples from the site of the tests

Statement III: NOT TRUE, nuclear gages perform density tests on soils containing large-sized aggregates and on frozen materials.

Statement V: TRUE, a license is required to own, possess, or use nuclear-type density- and moisture-measuring instruments. A license may be obtained from the Nuclear Regulating Commission (NRC), and where required from state and/or local government agencies.

Answer: D ⇐

Chapter E: Temporary Structures

E-1 CONSTRUCTION LOADS

Construction loads could be applied to both temporary and permanent structures where these loads will be applied during the construction phase of these structures. Construction loads are to be combined with other applicable loads related to the structure under consideration. Stairs, ladders, and elevators are not addressed in the SEI/ASACE 37-02 Standards.

§ 4.1.1 ASCE 37-02 Definitions :

Construction loads: those loads imposed on a partially completed or temporary structure during and as a result of the construction process. Construction loads include, but are not limited to, materials, personnel and equipment imposed on the temporary or permanent structure during the construction process.

Construction dead load, C_D : the dead load of temporary structures that are in place at the stage of construction being considered. The dead load of the permanent structure, either partially complete or complete, is not included in the C_D; the dead load of the permanent structure is defined as dead load, D.

LL **Individual personnel load**: a concentrated load of 250 lb that includes the weight of one person plus equipment carried by the person or equipment that can be readily picked up by a single person without assistance.

Working surfaces: floors, decks, or platforms of temporary or partially completed structures which are or are expected to be subjected to construction loads during construction.

2 TYPES OF CONSTRUCTION:

Shored (False work support) — Unshored, No false work.

§ 4.2. **Materiel Loads**

The material dead loads consist of two categories:

Table E-1 FML versus VML

Fixed material loads (FML)	Variable material loads (VML)
➢ Fixed in magnitude ➢ Examples:	➢ Varies in magnitude during the construction process ➢ Examples:

The FML is the load from materials that is fixed in magnitude. The VML is the load from materials that varies in magnitude during the construction process. If the local magnitude of a material load varies during the construction process, then that load must be considered a VML.

§ 4.2.1 Concrete Load

The weight of concrete placed in a form for the permanent structure is a material load. When the concrete gains sufficient strength so that the formwork, shoring, and reshoring are not required to support, the concrete becomes a dead load.

§ 4.2.2 Materials Contained in Equipment

Materials being lifted by or contained in equipment are part of the equipment load, not a material load. Once such materials have been discharged from the equipment, they become a material load.

§ 4.3 Personnel and Equipment Load, C_p

Personnel and equipment loads shall be considered in the analysis or design of a partially completed or temporary structure. The design or analysis of the structure shall be governed by either a uniformly distributed or a concentrated personnel and equipment load, whichever creates the most severe strength and/or serviceability condition. The governing load shall be assumed to be placed in the pattern or location that creates the most severe strength and/or serviceability condition.

The personnel and equipment loads used in the design or analysis of a partially completed or temporary structure shall be the maximum loads that are likely to be created during the sequence of construction.

§ 4.3.2 Uniformly Distributed Loads

Uniform loads shall be selected to result in forces and moments that envelope the forces and moment that would result from the application of concentrated loads that could occur and are not separately considered.

§ 4.3.3 Concentrated Loads

The personnel and equipment concentrated loads shall be the actual maximum loads expected in the construction process but shall be no less than those given in Table E-2. The concentrated load shall be located to produce the maximum strength and/or serviceability conditions in the structural members. The designer shall consider each category of minimum concentrated personnel and equipment load that is likely to occur during the construction process.

Concentrated loads from equipment shall be determined in accordance with Section 4.6. of ASCE 37-02

$N = kg \frac{m}{s^2}$ $Pa = \frac{N}{m^2}$

Table E-2 Minimum Concentrated Personnel and Equipment Loads

Action	Minimum Load[a] *lb (kN)*	Area of load Application in. × in. (mm × mm)
Each person	250 (1.11)	12 × 12 (300 × 300)[b]
Wheel of manually powered vehicle	50 (2.22)	Load divided by tire pressure[c]
Wheel of powered equipment	2000 (8.90)	Load divided by tire pressure[c]

a Use actual loads when they are larger than the tabulated here.
b. Need not less than 18 in. (457 mm) c. to c.
c. For hard rubber tires, distribute load over an area 1 in. (25 mm) by the width of the tire.

§ 4.3.4 Form Pressure:

The concentrated loads specified in Table 1 include adequate allowance for ordinary impact conditions. Provision shall be made in the structural design for loads that involve unusual vibration and impact forces.

§4.4 Horizontal Construction Load, C_H

One of the following horizontal load criteria, where appropriate, shall be applied to temporary or partially complete structures as a minimum horizontal loading, whichever gives the greatest structural effects in the direction under consideration.

1. For wheeled vehicles transporting materials, 20% for a single vehicle or 10% for two or more vehicles of the fully loaded vehicle weight. Said force shall be applied in any direction of possible travel, at the running surface.
2. For equipment reactions as described in Section 4.6, the calculated or rated horizontal loads, whichever is the greater.
3. 50 lb per person (0.22 kN/person), applied at the level of the platform in any direction.
4. 2% of the total vertical load. This load shall be applied in any direction and shall be spatially distributed in proportion to the mass. This load need not be applied concurrently with wind or seismic load.

This provision shall not be considered as a substitute for the analysis of environmental loads.

§4.5 Erection and Fitting Forces, C_F

Forces caused by erection (alignment, fitting, bolting, bracing, guying, and so on) shall be considered.

§4.6 Equipment Reactions, C_R

The reactions from equipment, with due consideration to all loading, conditions, shall be used in the design of the temporary or partially completed structure. The equipment reactions shall include the full weight of the equipment operating at its maximum rated load in conjunction with any applicable environmental loads, unless the use is restricted and revised reactions are developed.

§ 4.6.1 General

The structure shall be designed to safely support the full weight of the equipment and associated worst-case load effects caused by its operation. The design shall include the consideration of support deflections or movements, out-of-level supports, vertical misalignment, and environmental loads on the equipment.

§ 4.6.2 Rated Equipment

The minimum equipments loads for design shall be those provided by the equipment manufacturer of supplier.

Unless loaders, such as front end loaders or forklifts, are intentionally restricted from tipping on one axle, the loader self weight plus tipping load shall be applied to the front axle.

The designer shall verify the basis of the rating and the rated reactions given by the equipment supplier. If the basis of the rating is different than the conditions under which the equipment will be used, the more severe reactions shall be used in design.

§ 4.6.3 Nonrated Equipment

The equipment loads for nonrated equipment shall be determined by analysis.

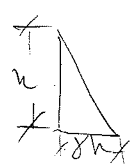
§ 4.6.4 Impact

The reaction of equipment shall be increased by 30% to allow for impact, unless other values (either larger or smaller) are recommended by the manufacturer, are required by the authority having jurisdiction, or are justified by analysis.

§ 4.7 Form Pressure:

Formwork shall be designed for the lateral pressure of the newly placed concrete given by Equation 4-1. ***Maximum and minimum*** values given for other pressure formulas do not apply to Equation 4-1.

$\sigma = \gamma h$



$$C_C = w \times h \qquad (4\text{-}1)$$

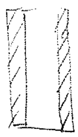
$$C_{C\,SI} = 23.5\, h_{SI} \qquad (4\text{-}1\ SI)$$

where C_C (C_{CSI}) is lateral pressure, psf (kPa); w is the unit weight of fresh concrete, pcf; and h (h_{SI}) is the depth of fluid or plastic concrete, ft(m).

For columns or other forms that may be filled rapidly before any stiffening of the concrete takes place, *h* shall be taken as the full height of the form or the distance between horizontal construction joints when more than one placement of concrete is to be made.

§ 4.7.1.1 For concrete made with Type I cement, weighing 150 pcf (23.6 kN/m3), containing no pozzolans or admixtures, having a slump of 4 in. (100 mm) or less, and normal internal vibration to a depth of 4 ft (1.22m) or less, formwork may be designed for a lateral pressure as follows:

For columns:



$$\boxed{C_C = (150 + 9{,}000\,R/T)} \qquad (4-2)$$

$$C_{C\,SI} = 7.2 + \frac{785\,R_{SI}}{T_{SI} + 17.8} \qquad (4-2SI)$$

with a maximum of 3,000 psf (144 kPa), a minimum of 600 psf (28.8 kPa), but in no case greater than 150h (23.5 h_{SI}).

For walls:

rate of placement less than 7 ft (2 m) per h

$$C_C = (150 + 9{,}000\,R/T) \qquad (4-3)$$

$$C_{C\,SI} = 7.2 + \frac{785\,R_{SI}}{T_{SI} + 17.8} \qquad (4-3SI)$$

with a maximum of 2,000 psf (95.8 kPa), a minimum of 600 psf (28.8 kPa), but in no case greater than 150 h (23.5 h_{SI}).

For walls: rate of placement of 7 to 10 ft (2 to 3 m) per hour

$$C_C = 150 + 43{,}400/T + 2{,}800\,R/T \qquad (4-4)$$

$$C_{C\,SI} = 7.2 + \frac{244\,R_{SI}}{T_{SI} + 17.8} + \frac{1156}{T_{SI} + 17.8} \qquad (4-4SI)$$

Where R (R_{SI}) is the rate of placement, ft/h (m/h); and $T(T_{SI})$ is the temperature of concrete in the form, °F (°C).

Mathematically derived

Table E-3 Form Pressure

$$C_C = w \times h \qquad (4\text{-}1)$$

$$C_{C\,SI} = 23.5\, h_{SI} \qquad (4\text{-}1\ \text{SI})$$

No Limits

Where:

$Cc\ (C_{CSI})$ = is lateral pressure, psf (kPa);
w = is the unit weight of fresh concrete, pcf ; and
$h\ (h_{SI})$ = is the depth of fluid or plastic concrete, ft (m).

Table E-4 Form Pressure for concrete made with Type I cement (weight = 150 pcf, slump ≤ 4″ & normal internal vibration to a depth of 4′)

For columns	For walls (Empirical Eqns)		Slipform Pressure
	$R < 7\ ft/h$	$7\ ft/h \le R \le 10\ ft/h$	
$C_C = (150 + 9{,}000\, R/T) \quad (4-2)$ $C_{C\,SI} = 7.2 + \dfrac{785\, R_{SI}}{T_{SI} + 17.8} \quad (4-2\,SI)$ $600\ psf \le C_c \le 3000$ psf $C_c \le 150\, h$ Where: R (R_{SI}) = rate of concrete placement, ft/h ; and $T\ (T_{SI})$ = the temperature of concrete in the forms, °F (°C).	$C_C = (150 + 9{,}000\, R/T) \quad (4-3)$ $C_{C\,SI} = 7.2 + \dfrac{785\, R_{SI}}{T_{SI} + 17.8} \quad (4-3\,SI)$ minimum $600\ psf \le C_c \le 2000$ psf $C_c \le 150\, h$ Where: R (R_{SI}) = rate of concrete placement, ft/h ; and $T\ (T_{SI})$ = the temperature of concrete in the forms, °F (°C).	$C_C = 150 + 43{,}400/T + 2{,}800\, R/T \quad (4-4)$ $C_{C\,SI} = 7.2 + \dfrac{244\, R_{SI}}{T_{SI} + 17.8} + \dfrac{1156}{T_{SI} + 17.8} \quad (4-4\,SI)$ Where: R (R_{SI}) = rate of concrete placement, ft/h ; and $T\ (T_{SI})$ = the temperature of concrete in the forms, °F (°C).	$C_C = c + 6{,}000\, R/T \quad (4-5)$ $C_{C\,SI} = c_{SI} + \dfrac{524\, R_{SI}}{T_{SI} + 17.8} \quad (4-5\,SI)$ where: c (c_{SI}) = 100 psf for concrete placed in 6 to 10-in. lift with slight vibration or no revibration, and = 150 psf for concrete that requires additional vibration, such as gastight or containment structures; Cc($C_{C\,SI}$) = lateral pressure, psf R(R_{SI}) = the rate of concrete placement, ft/h ; and $T\ (T_{SI})$ = the temperature of concrete in the forms, °F (°C).

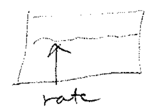
Sample Problem E-1: Form Pressure

Given: A reinforced concrete wall 1 ft × 15 ft × 40 ft (thickness × height × length) will be constructed using Type I cement without any admixture added. The concrete has a 3000 psi 28-day compressive strength and a 4-inch slump. The concrete will be delivered at a rate of 6 cubic yards per hour and the concrete has a temperature of 70° F. The concrete has a unit weight of 150 pcf.	**Find:** The maximum pressure at the base of the form is most nearly: (A) 670 psf (B) 680 psf (C) 945 psf (D) 1045 psf

Solution:

The rate of concrete placement =

$$R = \frac{6.0\ cy/hr}{40\ ft \times 1\ ft\ (1\ cy/27\ cu\ ft)} = 4.05\ ft/hr$$



For walls:

rate of placement less than 7 ft (2 m) per h

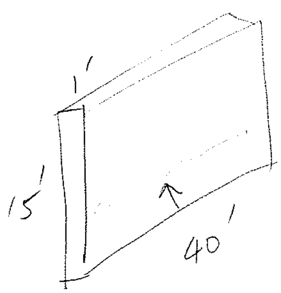
$$C_C = (150 + 9{,}000\,R/T) \qquad (4-3)$$

$$C_{C\,SI} = 7.2 + \frac{785\ R_{SI}}{T_{SI} + 17.8} \qquad (4-3SI)$$

R (R_{SI}) = rate of concrete placement, ft/h ; and
T (T_{SI}) = the temperature of concrete in the forms, °F (°C).

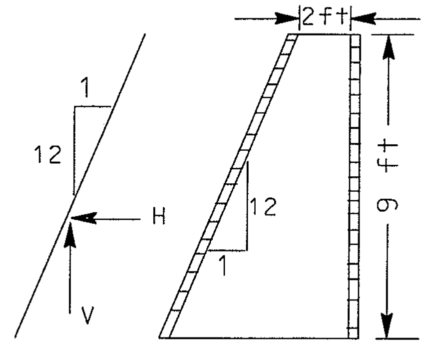
$$C_C = (150 + 9{,}000\,R/T) = (150 + 9{,}000 \times 4.05/70) = 670.71\,psf$$

Answer: A ⇐



The following information are to be used for problems E-2 to E-6

A 9-ft high concrete retaining wall, battered on one side at a slope of 1:12 is shown. The wall will be poured with concrete (a unit weight of 145 pcf) to its full height in less than one hour.



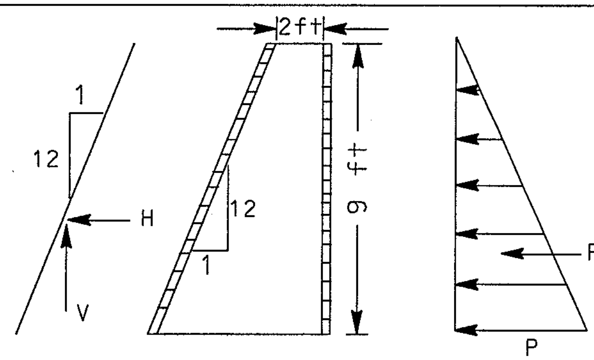
Sample Problem E-2: Form Pressure
Given: The maximum pressure at the base of the form is most nearly: (A) 653 psf (B) 980 psf (C) 1305 psf (D) 1350 psf

Solution:

The maximum lateral pressure at the base is:

$P = C_C = w \times h$ (4-1)

$= 145 \text{ pcf} \times 9 \text{ ft} = 1305 \text{ psf}$



Answer: C ⇐

Sample Problem E-3: Form Lateral Force
Given: The maximum lateral force per linear foot of the wall is most nearly: (A) 5873 plf (B) 4657 plf (C) 3500 plf (D) 2950 plf

Solution:

The maximum lateral force F is:

$$F = \tfrac{1}{2} \times P \times h = \tfrac{1}{2} \times 1305 \text{ psf} \times 9 \text{ ft} = 5872.5 \text{ plf}$$

Answer: A ⇐

Sample Problem E-4: Location of the Lateral Force

Given: The lateral force is located at what distance from the base?

(A) 6.0 ft
(B) 4.5 ft
(C) 3.0 ft
(D) 0 ft

Solution:

The lateral force *F* is acting at the centriod which is located at ⅓ from the base i.e. 3 ft

Answer: C ⇐

Sample Problem E-5: Uplift Force on the Form

Given: The uplift force the form will be subjected to is most nearly:

(A) 0 lb/ft
(B) 200 lb/ft
(C) 382 lb/ft
(D) 482 lb/ft

Solution:

The uplift force *V* is $= H/12 = F/12 = 5782.5\ plf/12 = 481.9\ plf$

Answer: D ⇐

Sample Problem E-6: Uplift Force on the Form

Given: If the slope of battered side of the retaining wall is changed from 1:12 to 1:10 keeping everything else the same. What effect would be on the uplift force on the form.

(A) will increase
(B) will decrease
(C) no change at all
(D) not enough information

Solution:

Slope 1:12 (85.23°) is steeper than 1:10 (84.29°). The uplift force (V) is a function of the slope as indicated in sample problem E-5. The steeper the slope the smaller the uplift force

Answer: B ⇐

§ 4.7.1.2 Alternatively, a method based on appropriate experimental data may be used to determine the lateral pressure used for form design.

§ 4.7.1.3 If concrete is pumped from the base of the form, the form shall be designed for full hydrostatic head of concrete. $C_C = w \times h$, plus a minimum allowance of 25% for pump surge pressure. In certain instances, pressure may be as high as the face pressure of the pump piston.

§ 4.7.2 Slipform Pressure

For a slipform concreting operation, the lateral pressure of fresh concrete to be used in designing the forms, bracing, and wales shall be calculated as:

$$C_C = c + 6{,}000\, R/T \qquad (4-5)$$

$$C_{C\,SI} = c_{SI} + \frac{524\, R_{SI}}{T_{SI} + 17.8} \qquad (4-5\,SI)$$

where c (c_{SI}) is 100 psf (4.79 kPa) for concrete placed in 6 to 10-in. (150 to 250-mm) lift with slight vibration or no revibration and 150 psf (7.19 kPa) for concrete that requires additional vibration, such as gastight or containment structures; Cc ($C_{C\,SI}$) is lateral pressure, psf (kPa); R (R_{SI}) is the rate of concrete placement, ft/h (m/h); and T (T_{SI}) is the temperature of concrete in the forms, °F (°C).

§ 4.7.3 Shoring loads

When shores are required to support the load of newly placed concrete, these shores shall be maintained until the concrete has gained enough strength to be self -supporting. When shoring is continuous over several floors, the calculated loads on these shores shall be cumulative unless and until the shores have been released and reset to allow the slab in question to carry its own dead weight. Such release should not occur until the concrete is capable of carrying its own dead load.

§ 4.8 Application of Loads

§ 4.8.1 Combined Loads

The design construction load shall include the critical combination of personnel, equipment and material loads.

§ 4.8.1.1 Working Surfaces. Structures supporting working surfaces as defined in Section 4.1 shall be designed for the combined material, personnel, equipment, and other applicable construction loads.

When the construction operation fits the definition in Table E-4, the designer is permitted to design for the tabulated uniform loads as the vertical load from the combination of personnel, equipment, and material in transit or staging. When the construction operation does not fit the definitions in Table E-4, the design shall be for the actual loads. Concentrated loads shall be considered separately

Table E-4 Classes or Working Surfaces for Combined Uniformly Distributed Loads

Operational Class	Uniform Load[a] psf (kN/m^2)
Very light duty; sparsely populated with personnel; Hand tools; *very small amounts of construction materials*	20 (0.96)
Light duty: sparsely populated with personnel; hand Operated equipment; staging of materials for lightweight *construction*	25 (1.20)
Medium duty: concentrations of personnel; staging of materials *for averag construction*	50 (2.40)
Heavy duty: material placement by motorized buggies; staging of materials for heavy construction	75 (3.59)

[a]Loads do not include dead load, D; construction dead lad, C_D; or fixed material loads, C_{FML}.

§ 4.8.1.2 Specification of Temporary Structures. When temporary structures are specified by load name, the names of the load class and the magnitude of design loads shall be as given in Table 2.

§ 4.8.2 Partial Loading

The full intensity of the construction load applied only to a portion of the length of a structure of member shall be considered if it produces a more unfavorable effect than the same intensity applied over the full length of the structure of member.

§ 4.8.3 Reduction in Construction Loads

§ 4.8.3.1 Material loads. No reduction is allowed for fixed or variable material loads, except to the extent that small amounts of material in transit at staging are included in uniformly distributed personnel, equipment, and material loads, such as those in Table 2.

§ 4.8.3.2 Personnel and Equipment Loads. When justified by an analysis of the construction operations, members having an influence area of 400 ft^2 (37.16 m^2) or more may be designed for a reduced uniformly distributed personnel and equipment load determined by applying the following formula:

$$C_p = L_o\left(0.25+\frac{15}{\sqrt{A_I}}\right) \qquad (4-6)$$

$$C_{P\,SI} = L_o\left(0.25+\frac{4.57}{\sqrt{A_I}}\right) \qquad (4-6\ SI)$$

Where C_P ($C_{P\ SI}$) is the reduced design uniformly distributed personnel and equipment load per ft^2 (m^2) f area supported by the member; L_o is the unreduced uniformly distributed personnel and equipment design load per ft^2 (m^2) of area supported by the member; and A_I is the influence area, ft^2 (m^2). The influence area A_I is normally four times the tributary area for a column, two times the tributary area for a beam, and equal to the panel area for a two-way slab.

The reduced uniformly distributed personnel and equipment design load, regardless of influence area, shall not be less than 50% of the unreduced design load for members supporting one level or 40% of the unreduced design load for members supporting more than one level, except that where the uniformly distributed personnel and equipment load is 25 psf (1.2 kN/m^2) or less, the reduced load shall not be less than 60% of the unreduced design load, unless justified by an analysis of the construction operations.

§ 4.8.3.3 Personnel and Equipment Loads on Sloping Roofs

A reduction in gravity construction loads for personnel and equipment on a roof is also permitted based upon the slope of the roof. The reduction factor R is:

$$R = 1.2 \; - 0.05F$$

in/ft

where F is the slope of the roof expressed in inches per foot (*in SI system.* $F = 0.12 X$ *slope of the roof expressed in percentage points*). R need not exceed 1.0 and shall not be less than 0.6. This reduction may be combined by multiplication with the reduction based on area, but the reduced load shall not be less than 60% of the basic unreduced load.

§ 4.8.4 Restriction of Loads

The following working surfaces shall have their use and access restricted by posting of the permitted loads and load conditions or by operational control by the entity that has jurisdiction over their use.

1. Scaffolds with working surfaces of 40 ft^2 (3.72 m^2) or less shall be rated for the number of individual personnel loads that they can support, and the working surfaces shall be restricted accordingly. When designing, the individual personnel loads shall be placed in such locations so as to maximize their effects on the structural members of the scaffold; however, they need not be spaced closer than 2ft (0.61 m) on center.
2. Working surfaces designed for superimposed uniform loads of 25 psf (1.20 kN/m^2) or less shall be rated for both their superimposed uniform load capacity and the number and location of the individual personnel loads that they can support. These working surfaces shall be restricted accordingly.
3. Working surfaces designed for loads less than what could reasonably be expected to be placed thereon shall be restricted to the design loads.

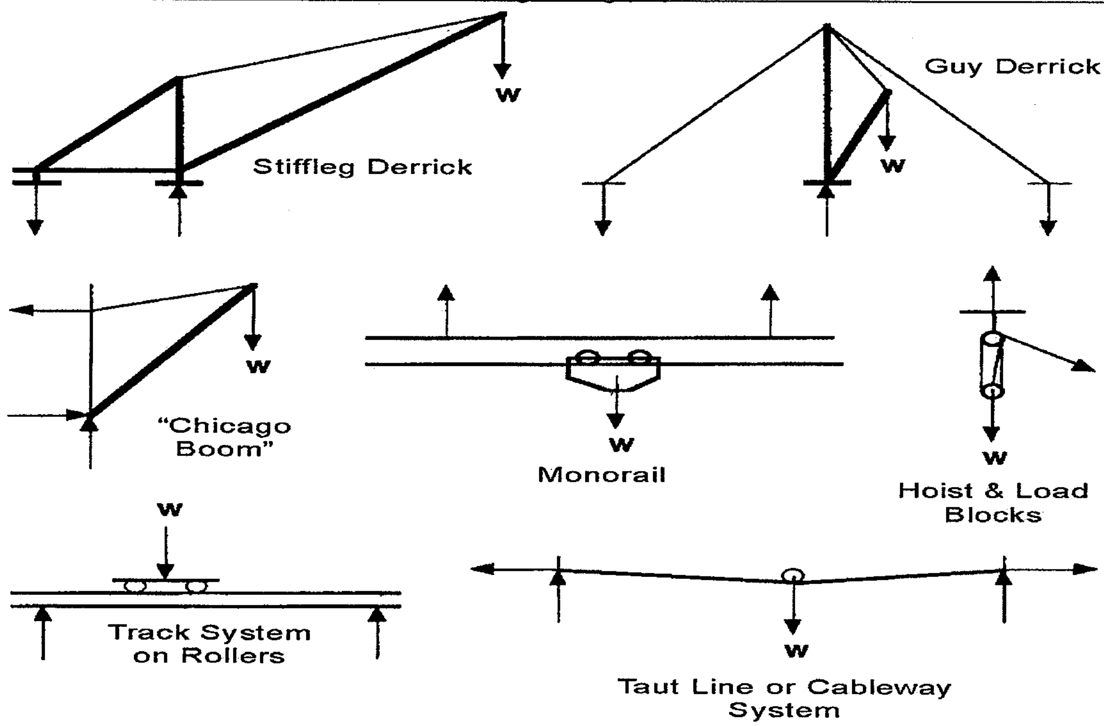


Figure E-1 Typical Rigging Configurations and Construction Loads

Construction loads produced during the handling operations of heavy component erection, govern the design of all elements of the rigging and handling system, and affect both the existing infrastructure and the respective component being erected. These loads are a function of the gross rigging weight of the component, its center of gravity location, required handling operations, and the configuration of the rigging and handling system being used.

Construction Breadth (A.M.) Practice Problems and Solutions

Construction Breadth A.M. Practice Problems

1- The following data of the cross section for a roadway is given in the following table.

556

472

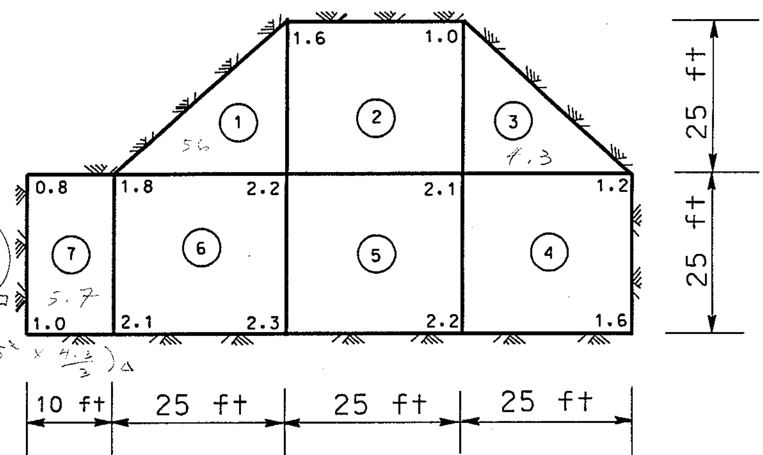
Station	Cut (ft^2)	Fill (ft^2)
1 + 25	0	90
2 + 75	200	80
4 + 50	100	0

150 175 472 259

The amount of imported borrow (deficit) or waste (excess) using the average end area method is most nearly:

(A) 731 yd^3 deficit
(B) 796 yd^3 excess cut more than fill.
(C) 1528 yd^3 excess
(D) 1528 yd^3 deficit

2- A borrow pit grid is shown. The numbers at the grid points represent the depth of cut, in feet. The excavated volume, in cubic yards is most nearly: $\left(\frac{31.2}{4} \times 25^2\right)$



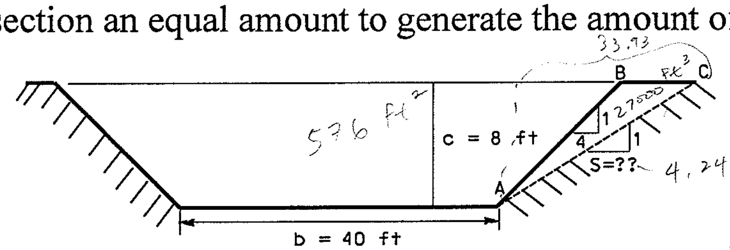
A) 228 yd^3
B) 328 yd^3 + $\left(\frac{1}{2} \times 25^2 \times \frac{5.6}{3}\right)$ + $\left(\frac{1}{2} \times 25^2 \times \frac{4.3}{3}\right)$
C) 338 yd^3 + $\left(\frac{5.7}{4} \times 10 \times 25\right)$
D) None of the above

→ [(4875) + (314 + 313) + (356)] / 27 → ~ 231

3- The cross section of a cut section of a proposed conventional highway is 40 feet wide at subgrade and 8 feet deep with side slopes 4H : 1V. This section of cut has a length of 3500 ft. The mass diagram of the entire project indicates that 2000 yd^3 of borrow is required. In order to avoid imported borrow, the designer decided to lay back both side slopes of the cut section an equal amount to generate the amount of borrow needed.

2000 × 27 = 54000

54000 / 2 = 27000 ft^3



576 ft^2 4.24

The new side slopes are most nearly:

27000 / 3500

(A) 4.06 :1
(B) 4.10:1
(C) 4.24:1
(D) 4.48:1

7.714 × 2 / 8 = 1.93

4- An area of a parcel was measured from a map using a planimeter and it was found to be 51.2 in^2. The map scale is 1in. = 100 ft. The area of the parcel is most nearly:

$51.2\ in^2 \times \frac{(100\ ft)^2}{1\ in^2} \rightarrow 512{,}000\ ft^2$

A) 11.42 acres
B) 11.75 acres
C) 12.16 acres
D) 12.45 acres

5- A construction company can manufacture a manhole frame with off-the-shelf hand tools. Costs will be \$4000 for tools and \$11.50 manufacturing cost per frame. As an alternative, an automated system will cost \$115,500 with a \$4.50 manufacturing cost per frame with an annual anticipated volume of 10,000 frames. Neglecting interest, the break-even point is most nearly:

$[4000 + 11.50(10{,}000)] = [115{,}500 + 4.50(10{,}000)]$

(A) 1.0 year
(B) 1.6 years
(C) 3.6 years
(D) 4.6 years

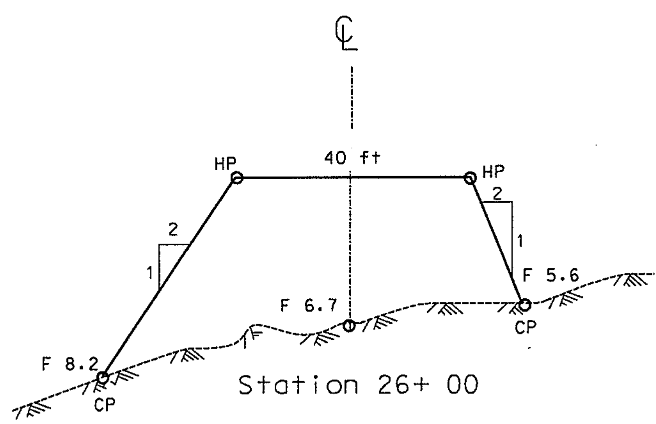
6- A compressor costs \$900. After 5 years its salvage value is \$300. Annual maintenance is \$50. The interest rate is 8%. The equivalent uniform annual cost is most nearly:

$\$900\ A/P - \$300\ A/F + \$50$

(A) \$224
(B) \$300
(C) \$327
(D) \$350

7- A fill section at Station 26 + 00 is shown. The area of the fill is most nearly:

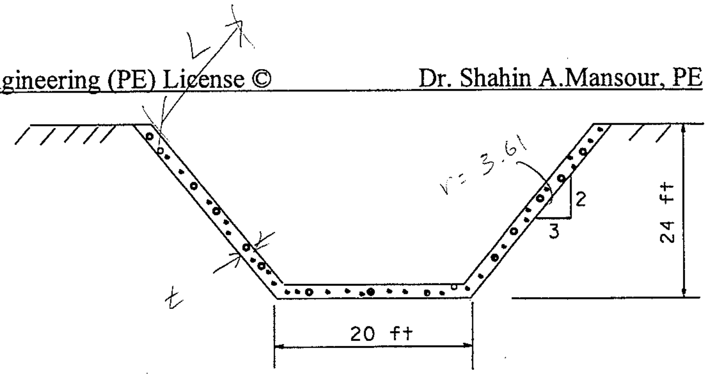
(A) 364 ft^2
(B) 374 ft^2
(C) 384 ft^2
(D) 394 ft^2



8- A masonry wall 8 feet high and 15 feet long will be constructed of brick 2 ¼″× 3 ¾″ × 8″. The thickness of the mortar is ½″. The number of bricks needed to construct this wall is most nearly:

A) 573 bricks
B) 690 bricks
C) 723 bricks
D) 743 bricks

9- The cross section of an 800 ft long irrigation channel is shown. The walls and bottom of the channel have a uniform thickness of 8 inches. Assume 10% waste. The volume of the concrete delivered to build this channel is most neatly:



(A) 2104.40 yd^3
(B) 2114.80 yd^3
(C) 2214.80 yd^3
(D) 2314.80 yd^3

10- Slope staking information for a new road with a 22 foot wide is given below:

C7.2	C12.6	C13.1
18.6	0.0	24.1

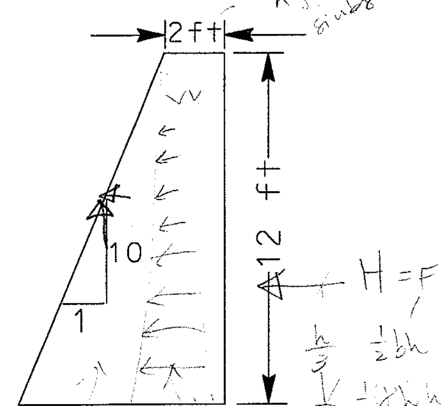
The future side slope of the new road is most nearly:

A) 0.75:1
B) 1: 1
C) 1.5 : 1
D) 2 : 1

11- A reinforced concrete wall 2 ft×20 ft×60 ft (thickness × height × length) will be constructed using Type I cement without any admixture added. The concrete has a 28-day compressive strength of 3500 psi and a 4-inch slump. The concrete will be delivered at a rate of 12 cubic yards per hour and the concrete has a temperature of 75° F. The maximum pressure at the base of the form is most nearly:

A) 375 psf
B) 475 psf
C) 645 psf
D) 945 psf

12- A 12-ft high concrete retaining wall, battered on one side at a ratio of 1:10, will be poured with 150 pcf concrete to its full height in less than one hour. The calculated maximum concrete pressure in the form is 1,800 psf. The uplift force the form will be subjected to is most nearly:



A) 0 lb/ft
B) 600 lb/ft
C) 800 lb/ft
D) 1080 lb/ft

Pg. 93 σh = 150 (12')

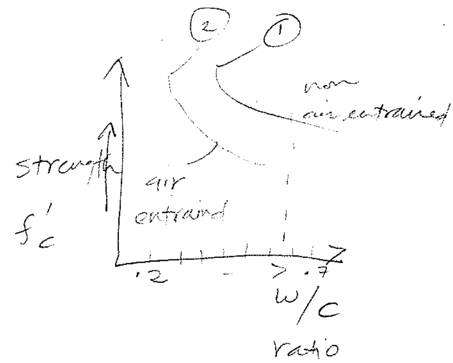
13- If the water-to-cement ratio of concrete is decreased, which statements about the concrete are true? Pg.203

i. Workability is decreased.
ii. Durability is increased.
iii. Water tightness is decreased.
iv. Strength is increased.

$\frac{H_2O \text{ lb}}{\text{Cement lb}}$

(pg. 210)

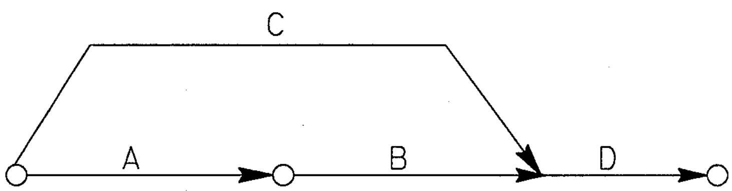
$\frac{8.33 \text{ lb/gal}}{94 \text{ lb}}$



(A) i & ii
(B) ii, iii, & iv
(C) i, ii & iv
(D) iii & iv

14- An activity on arrow network is shown below. The variables are defined as follows:

TF = Total float
LF = Late finish
LS = Late start
ES = Early start
EF = Early finish
D = Duration



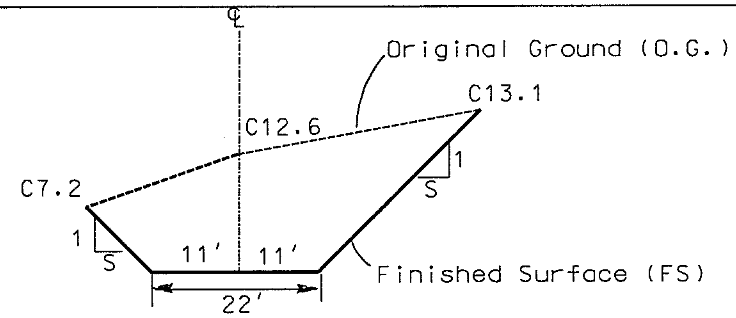
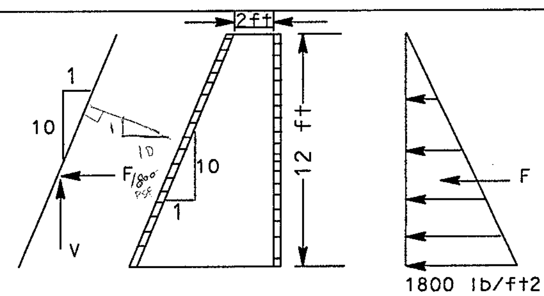
Using subscription to denote the different activities, the expression for the total flat of Activity B, TF_B, is most nearly:

(A) $LS_D - ES_B - D_B$
(B) $LS_B - EF_A$
(C) $ES_D - ES_B - D_B$
(D) $LF_B - ES_B - D_B$

Construction Breadth (A.M.) Practice Problems Solutions

#	Ans.	Solution or Reference
1	B	Using the average end area method even though [triangle sketch] $V_{Cut}=\left(\frac{0+200}{2}\right)(\frac{150}{27})+\left(\frac{200+100}{2}\right)(\frac{175}{27})=1527.78\ CuYd$ $V_{Fill}=\left(\frac{90+80}{2}\right)(\frac{150}{27})+\left(\frac{80+0}{2}\right)(\frac{175}{27})=731.48\ CuYd$ $V_{Net}=V_{Cut}-V_{Fill}=1527.78-731.48=796.30\ Cu\,Yd$
2	A	The grid figures can be grouped as 4 adjacent 25-ft squares (2, 4, 5, and 6) , two separate triangles (1 and 3), and a 10 x 25 ft rectangle (7). I - *Squares*: For the group of squares, the sum of the corners is (1.8) + (3 × 2.2) + (2.1) + (2 × 2.3) + (2 × 2.2) + (1.6) + (1.2) + (3 × 2.1) + (1.0) + (1.6) = 31.2 ft The total volume excavated within those grid squares is: (31.2/4)(25 × 25) = 4,875 ft^3 *II - Triangles*: For triangle 1, the average cut is: (1.8 + 2.2 + 1.6)/3 = 1.87 ft 5.6 The area of the triangle is: ½ × 25 × 25 = 312.5 ft^2 The approximate volume, then = area × height = (1.87 ft)(312.5 ft^2) = 584.4 ft^3 For triangle 3, the volume is: = ½ (2.1 + 1.2 + 1.0)/3 × 25 × 25 = 447.9 ft^3 *III – Rectangle*: 4.3 For the rectangle 7, the volume is: = (2.1+1.8 + 0.8 + 1.0)/4 × 10× 25 = 356.3 ft^3 5.7 Finally, the total volume = 4,875 + 484.4 + 447.9 + 356.3 = 6,163.6 ft^3 = 6,163.6 ft^3/(27 ft^3/yd^3) = 228.3 yd^3
3	C	The volume that will be generated from the proposed excavation equals to the area of the triangle ABC times the length times TWO: $V=\frac{2\times(1/2)(BC)(8ft)(3500ft)}{27\ ft^3/yd^3}=2000\ yd^3$ Solve for the length BC = 1.93 ft Distance S = horizontal distance from A to B + BC = (8 × 4) + 1.93 = 33.93 ft New side slope = 33.93 ft/ 8 ft = 4.24 :1 H :V

4	B	Based on the scale given in the problem: $1\ in^2 = (100ft)^2$ Therefore, the area is $51.2\ in^2 = (100ft)^2(51.2in^2) = 512{,}000\ ft^2 / 43{,}560\ ft^2/acre = 11.75\ acres$
5	B	Annual saving = $(10{,}000 \times \$11.50) - (10{,}000 \times \$4.5) = \$70{,}000$ difference Annual investment = $\$115{,}500 - \$4000 = \$111{,}500$ Pay back period = $111{,}500 / 70{,}000 = 1.59\ years$
6	A	$EUAC = \$900\left(A/P\right)_{n=5}^{i=8\%} + \$50 - \$300\left(A/F\right)_{n=5}^{i=8\%}$ $= \$900(0.2505) + \$50 - \$300(0.1705)$ $= \$224.30$
7	A	There are three methods to calculate an area : 1- divide the area into simple shapes (triangles, rectangles, trapezoids) 2- criss-cross (coordinates) 3- DMD Obviously method 1 is the easiest and fastest and it will be used here: Use side slopes to find the horizontal distances and calculate the areas by subtracting the area of a triangle from a trapezoid for each side of the section as follows: $Area = \left[\frac{(5.6+6.7)}{2}(20+2\times5.6) - \frac{(5.6)(11.2)}{2}\right] + \left[\frac{(6.7+8.2)}{2}(20+2\times8.2) - \frac{(8.2)(16.4)}{2}\right]$ $= 364\ ft^2$
8	D	The given dimensions of a brick are: 2 ¼ ″ (height) × 3 ¾″ (thickness) × 8″ (length) Now adding the ½ ″ mortar to the face dimensions of the brick Height = 2 ¼ ″ + ½ ″ = 2 ¾ ″ = 2.75″ Length = 8″+ ½″ = 8.5″ The surface area of the wall = 8 ft ×15 ft = 120 ft^2 The area of the face of the brick = 2.75 × 8.5 = 23.275 in^2 The number of bricks needed to construct the wall = $= \frac{120\ ft^2}{23.275\ in^2 \times (ft^2/144\ in^2)} = 742.4$

9	D	$V_{Concrete}=\left(\frac{8\,in.}{12}\right)\left(20\,ft+2\sqrt{(24)^2+(\frac{3}{2}\times 24)^2}\right)(800\,ft)(1.10)$ waste Height L Depth $=62499.50\,ft^3=2314.80\,yd^3$
10	B	 $Slope=\frac{Rise}{Run}=\frac{7.2}{(18.6-11)}=0.94$
11	B	The rate of concrete placement = $R=\frac{12.0\,cy/hr}{60\,ft\,x\,2\,ft\,(1\,cy/27\,cu\,ft)}=2.70\,ft/hr$ ***For walls***: rate of placement less than 7 ft (2 m) per hour $C_C=(150+9{,}000\,R/T)$ (4−3) $C_{C\,SI}=7.2+\frac{785\,R_{SI}}{T_{SI}+17.8}$ (4−3SI) R (R_{SI}) = rate of concrete placement, ft/h ; and T (T_{SI}) = the temperature of concrete in the forms, °F (°C). $C_C=(150+9{,}000\,R/T)\;=(150+9{,}000\times 2.7/75)=474\,psf$
12	D	 The vertical force (uplift) on the form is: $V=F/10=\frac{(1/2)(1800)(12)}{10}=1080\,lb/ft$

13	C	Water cement ratio is the ratio of water to cement by weight. Decreasing the w/c ratio will: Workability is decreased → true Durability is increased→ true Water tightness is decreased→ NOT true Strength is increased → true
14	D	For any activity, the following relationships are true: $TF = LF - EF$ But, $EF = ES + D$ $\therefore TF = LF - (ES + D) = LF - ES - D$

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PART 2

Civil DEPTH (P.M.) Exam Specifications

CONSTRUCTION

Construction Depth (P.M.) Topics

Chapter I- Earthwork Construction and Layout

A. Excavation and embankment (cut and fill) [A.M.]....................................
B. Borrow pit volumes [A.M.]..
C. Site layout and control [A.M.]...
PM D. Earthwork mass diagrams ....

Chapter II- Estimating Quantities and Costs

A. Quantity take-off methods [A.M.]..
B. Cost estimating [A.M.]..
PM C. Engineering economics ....
C.1 Value engineering and costing ....

Chapter III- Construction Operations and Methods

A. Lifting and rigging ....
B. Crane selection, erection, and stability ....
C. Dewatering and pumping ....
D. Equipment production ....
E. Productivity analysis and improvement ....
F Temporary erosion control ....

Chapter IV- Scheduling

A. Construction sequencing [A.M.]..
PM B. CPM network analysis ....
PM C. Activity time analysis ....
D. Resource scheduling [A.M.]...
E. Time-cost trade-off [A.M.]...

Chapter V- Material Quality Control and Production

A. Material testing (e.g., concrete, soil, asphalt) [A.M.]...........................
B. Welding and bolting testing ....
PM C. Quality control process (QA/QC) ....
D. Concrete mix design ....

Chapter VI- Temporary Structures

Chapter VII- Worker Health, Safety, and Environment

Chapter VIII- Other Topics

Chapter I: Earthwork Construction and Layout

I-D EARTHWORK MASS DIAGRAM

I-D.1 Introduction

The distribution analysis of earthwork involves determining balance points along the roadway center line between cut and fill. On a simple job, one could make separate subtotals of cuts and fills. The balance points would occur where these subtotals are equal. On larger projects, this method is inadequate and a more comprehensive method is necessary.
Three procedures can be used to analyze earthwork operations:

2) the station-to-station method. Details concerning the station-to-station method.
3) by study of the mass diagram. The mass diagram, a general graphical approach that provides an excellent introduction to earthwork operations, and
3) by optimization using linear programming. Optimization by linear programming is a numerical method beyond the scope of this manual.

On the mass diagram, the abscissa, or horizontal measurement, represents the stationing (distance) of the roadway. The ordinate, or vertical measurement, represents the accumulated (cumulative) net cubic yards (cubic meters), excavation (yardage gained) being positive, and embankment (yardage used) being negative. At the beginning of the curve the ordinate is zero, and the ordinates are calculated continuously to the end of the project.

The mass diagram is used to determine the following:

- The accumulated difference between cut (supply) and fill (demand) at any station.
- Amounts of haul, freehaul, and overhaul.
- Direction of haul.
- Amount and location of borrow.
- Amount and location of waste.

Figure I.1 illustrates a section of highway constructed by means of cuts and fills. At the lower portion of Figure 1.1 the profile of the original ground before the road was built is shown. The alignment of this profile coincides with the alignment of the highway. The "proposed grade" is the grade or profile to which the road will be constructed and is shown on the sketch as the finished road.

A review of the sketch and the profiles would indicate that the fills would be constructed with material obtained from the cuts. If we were to raise the grade, or build the road a little higher, the cuts would be smaller and the fills would be higher. That is, there would be less roadway excavation and more roadway embankment. If we were to lower the profile of the proposed road, the opposite would be true.

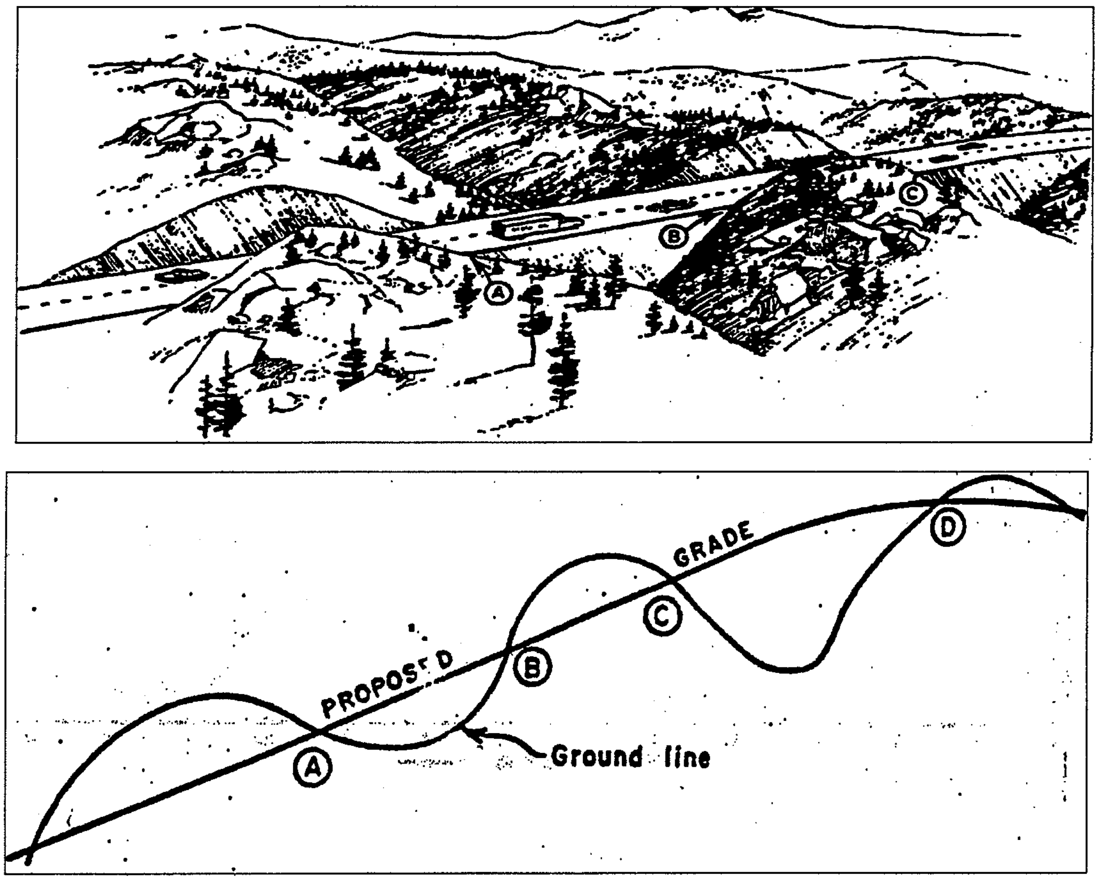


Figure I-1 Original Ground versus Proposed Profile

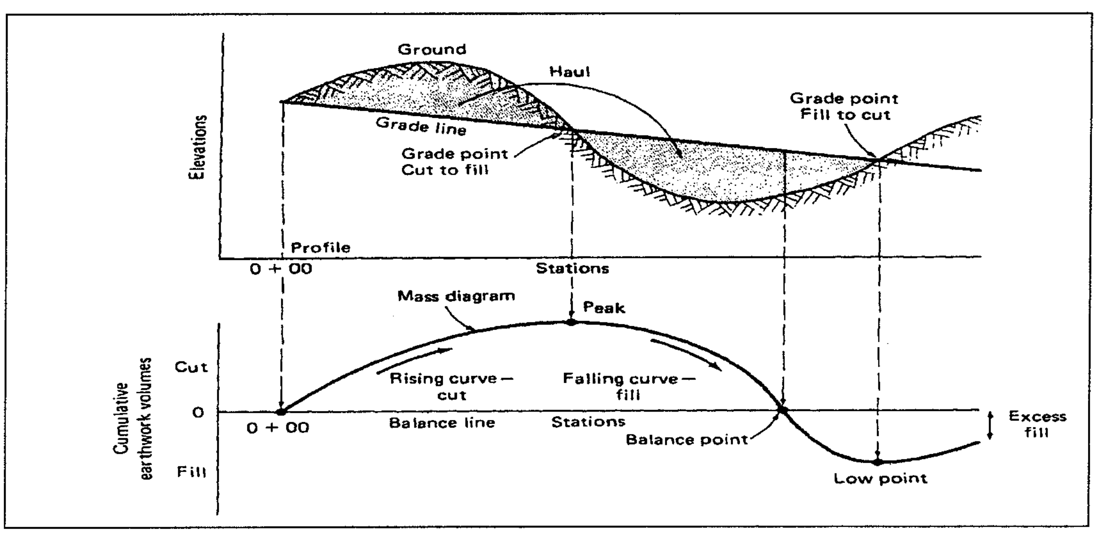


Figure I-2 Mass Diagram

I-D.2 Definitions:

Abscissa: Represents the centerline station along the facility (100 ft or 100 m).

Area under the diagram (curve): Represents haul or work.

Ordinate: Represents the cumulative (algebraic sum) quantities in yd^3 or m^3.

Freehaul: The Standard Specifications establish a centerline distance of 1,000 feet as the "freehaul distance". This means that material hauled 1,000 feet or less is hauled free; the cost of haul is included in the excavation price. For haul longer than 1,000 feet, the first 1,000 feet is freehaul.

Overhaul: This is simply total haul less freehaul. Overhaul is measured along the main centerline.

Average Haul: Since each load moved from a cut to a fill is hauled a different distance, average haul is the average distance between the centers of mass of a cut and a fill.

Station Yards Overhaul: The unit of measurement of the hauling operation is the "station yard" which is one cubic yard of material hauled a distance of one station. Thus 5 cubic yards hauled 15 stations would equal 75 station yards of total haul, deducting 10, stations for freehaul gives a result of $(15 - 10) \times 5 = 25$ station yards of overhaul.

Local Borrow: Borrow material excavated from sources within the right of way as designated by the engineer, and not at the option of the contractor, is called local borrow. It is handled exactly as ordinary roadway excavation as regards excavation and hauling costs.

Imported Borrow: Borrow material obtained from sources outside the right of way is called imported borrow. It is paid for by the ton or cubic yard in place, including the cost of hauling. Thus it does not enter into the mass diagram calculations for overhaul

Waste or Excess: Waste is material excavated from roadway cuts that is either unsuitable or not requited for making embankments. Excess material that is suitable for roadway embankments is usually used to widen embankment uniformly or to flatten fill slopes.

Slope of the Curve: Represents the magnitude of net material per station, (+) for net excavation (cut) and (–) for net embankment (fill) per station.

Maxima Points: Indicate a change from cut to fill (haul forward).

Minima Points: Indicate a change from fill to cut (haul back).

Balance Line: A horizontal line that balances cut and fill quantities.

Grading Factor: Is the volume in fill divided by the volume in cut.

Swell Factor: Excavated materials such as rock when placed, as fill would require a large volume to occupy than it would in its natural state.

Shrinkage Factor: Excavated materials such as clay when placed as fill occupies less volume than it would in its natural state.

I-D.3 Properties of the Mass Diagram:

Figure I-3 shows a mass curve, corresponding to the accompanying profile of an existing ground line and of a new grade line.

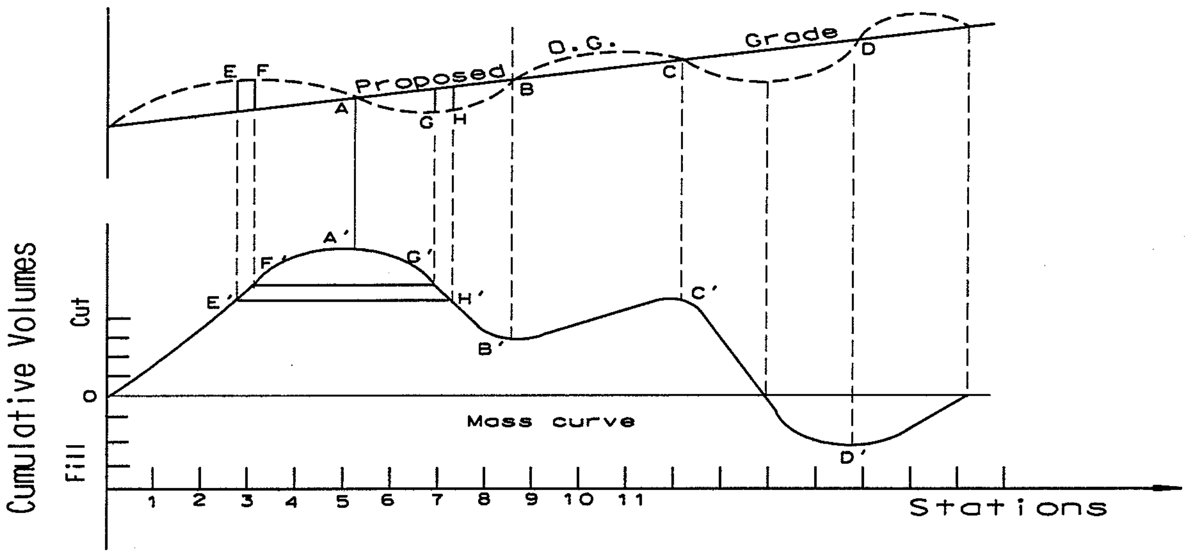


Figure I-3 Properties of Mass Diagram

(a) An upward slope on the mass curve indicates excavation (cut), the slope of the curve varying with the volume of excavation per linear foot along the line. That is the steeper the line the heavier the yardage.

(b) A downward slope on the mass curve indicates embankment (fill, the slope as before, varying with volume per linear foot.)

(c) Since the mass curve is plotted from excavation quantities and embankment quantities, it may be seen that the cut and fill between the points at which any horizontal line cuts off a loop of the mass curve will exactly balance. Such lines are called balancing lines. Note for example that line F'G' shows a balance of cut and fill volumes between the two points of intersection since the difference in ordinate, F' to A', is just equal to the difference in ordinate A' to G'. Transferring these points to the corresponding stations on the profile F and G, the cut FA makes the fill AG.

(d) Loops below a balancing line indicate hauling from right to left.

(e) Loops above a balancing line indicate hauling from left to right.

(f) The area between a balancing line and its corresponding loop of the mass curve is a measure of haul because it is the product of the volume and distance.

For example, the area between a horizontal. line F'G' and the curve F'A'G' is a measure of total haul (volume x distance) necessary to move the cut volume FA to make the fill AG. The area between the two horizontal line F'G' and E'H' is the measure of the total haul (volume × distance) necessary to move excavated material from EF to fill GH.

I-D.4 Free Haul on Mass Diagram:

To establish the two points fixing the limits of freehaul, the procedure is to fix on the mass diagram a line such as AB, in Figure I-4, which represents to scale a distance of 1000′. The material excavated between A and E may be deposited in fill between E and B without payment to the contractor for haul of this material. The distance beyond the freehaul (in excess of 1,000') is overhaul and is paid for at the price per station yard for hauling excavated material beyond the freehaul distance.

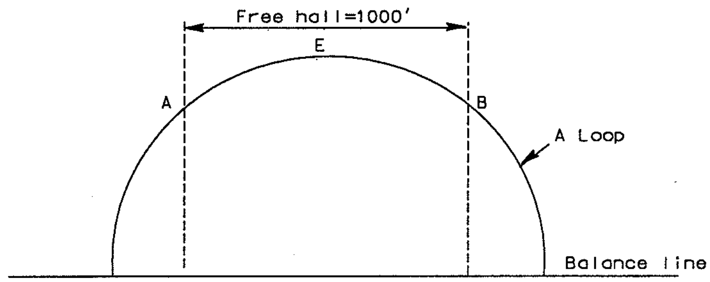


Figure I-4 Free Haul Distance

Consider a single loop of a mass curve as illustrated in Figure I.5, having a long cut on one side and a long fill on the other. The line AB represents a scaled distance of 1,000'. The excavated material between A and E will be used to fill the embankment EB, all within the freehaul distance.

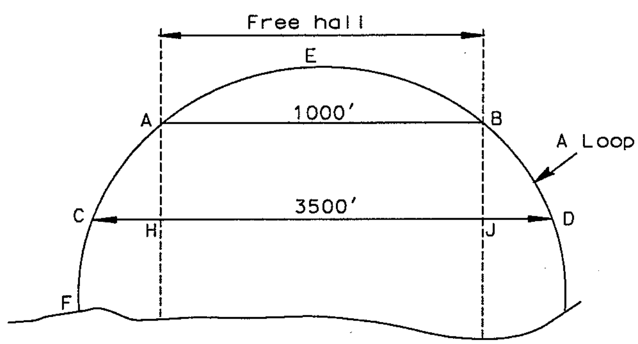


Figure I-5 Calculation of Overhaul

The line CD represents a scaled distance of 3500'. The cut CA will be used to make the fill BD. They distance involved in moving this material will range from 1,000' to 3,500'.

The average haul distance will be: $\dfrac{1{,}000 + 3{,}500}{2} = 2{,}250\,feet$

The average overhaul distance for which the transportation of the material will have to be paid for will be:

$$2{,}250' - 1{,}000' = 1{,}250' \text{ or } 12.5 \text{ station}$$

The yardage involved is the difference in ordinate between line CD and line AB. In order to obtain the station yards overhaul, the average overhaul distance should be multiplied by this yardage and the pay quantity is expressed in station yards.

The most common method of determining the station yards of overhaul, however, is to measure the area between the mass curve and the balance line. Since this area is made up of the horizontal measurement (stations) multiplied by the vertical measurement (yardage), the area represents station yards of total haul directly.

The area is usually measured with a planimeter. The square inches of area are then multiplied by the vertical-scale of so many yards, per inch and the horizontal scale of so many stations, per inch. The answer is the total haul in station yards. From this total haul, however, the freehaul must be subtracted.

The obvious short cut is to exclude the area representing the freehaul from the area measured. To do this, perpendicular lines are drawn from the ends of the 1,000' freehaul line to the balance line as shown in Figure I-5. The enclosed area, AEBJH, is the freehaul and need not be measured. The areas AHC and BDJ represent overhaul in station yard units.

I-D.5 Determination of Overhaul:

The amount of overhaul can be determined from the mass diagram. Portions of one loop of a mass diagram and the corresponding profile are shown in Figure I-6. Assume that balance line AB is the free-haul distance, at the scale of the mass diagram, and balance line A'B' is the limit of economic haul. Material from cut aa'd'd, excavated, hauled, and placed in fill at bb'e'e, is overhaul, which is represented under the mass diagram by areas AA'D and BB'D'. When AA′ and BB' are relatively straight, the sum of these two areas equals AD (EE'–AB), where EE' lies midway between AB and A'B'.

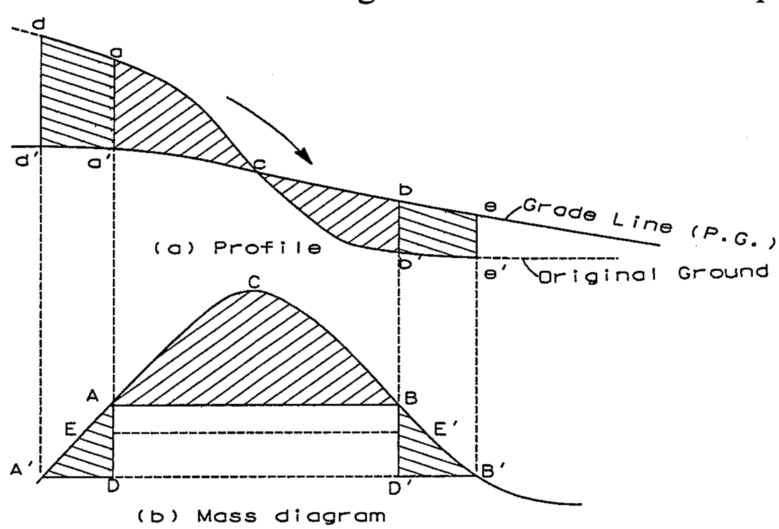


Figure I-6 Calculation of Overhaul

Consequently, the overhaul in station units (station m or station yd) is found by multiplying the difference in ordinates between the free-haul balance line and the overhaul balance line by the difference between the overhaul distance and free-haul distance in stations. This graphical method of determining overhaul quantities is valid when the two overhaul areas (AA'D and BB'D′ in Figure I-6) are nearly equal and of similar configuration. When the mass diagram is very curved or irregular, the areas that represent overhaul can be measured by a planimeter, an analog device for measuring area. Another, more rigorous numerical approach consists of approximating the profile and cross slopes by higher-order polynomials and integrating to obtain volumes and respective centroids of the volumes.

I-D.6 Balancing Procedure:

So far, only the case involving one loop of the mass diagram has been studied. In practice, several or many loops of varying magnitude may occur consecutively, and the mass diagram is very useful in determining the most economical solution by applying certain balancing procedures.

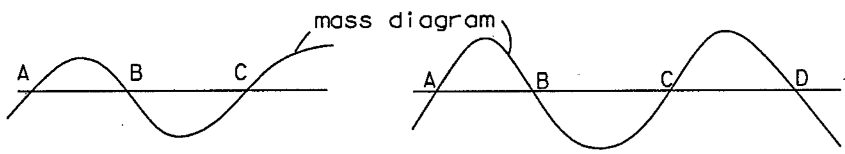


Figure I-7 Balancing Procedures for two and three loops

When two loops occur, as in Figure I-7 (left) , the most economical position for the balance line ABC is that in which AB = BC, providing the distances AB and BC are each less than the limit of economic haul (LEH).

If three consecutive loops occur, as in Figure I-7 (right) the most economical position for balance line ABCD is that in which (AB + CD) - BC = L.E.H., provided that AB, CD, and BC each is less than the limit of economic haul.

In general, the most economical position for a balance line cutting an even number of loops is where the sum of the segments cutting concave loops equals the sum of the segments cutting convex loops. Each segment should be less than the limit of economic haul. The most economical position for a balance line cutting an odd number of loops is where the sum of the segments cutting concave (or convex) loops minus the sum of the segments cutting loops in the opposite direction equals the limit of economic haul. As before, each segment should be less than the limit of economic haul (LEH).

I-D.7 Limit of Economic Overhaul (L.E.H.):

Under certain conditions, it may pay to borrow or waste. In other words, the overhaul distance is profitable only to a limit.

Let:

C_E = cost to excavate 1 unit volume (includes free haul)

C_{EW} = cost to excavate and waste 1 unit volume

C_B = cost to borrow 1 unit volume

C_{OH} = cost for overhaul per station unit

F = free-haul distance, stations

$L.E.H.$ = limit of economic haul, stations

The cost to excavate and waste plus the cost to borrow equals the cost of excavation plus the cost of overhaul:

$$C_{EW} + C_B = C_{OH}\ (L.E.H. - F) + C_E \qquad (31)$$

Assume that

$$C_{EW} = C_E$$

Then,

$$L.E.H. = \frac{C_B}{C_{OH}} + F \qquad (32)$$

Equation (32) governs the limit to economic overhaul given the free-haul distance and unit costs for borrow and overhaul.

Sample Problem I-1: L.E.H Calculations	
Given: The free-haul distance is specified as 10 stations (1000 ft) and the unit costs are C_E = \$11.80/ yd^3, C_B = \$ 14.50/yd^3, and C_{OH} = \$ 2.90/station yd.	***Find:*** The limit of economic haul (L.E.H.) is most nearly: (A) 10 Stations (B) 14 Stations (C) 15 Stations (D) 20 Stations

Solution:

$$L.E.H. = \frac{C_B}{C_{OH}} + F = \frac{14.50}{2.90} + 10 = 15\,stations$$

Answer: C⇐

I-D.8 Swell and Shrinkage:

Swell and shrinkage of excavated material are important parameters that should be considered in earthwork calculations. Consideration of these properties is of vital importance in computing a mass diagram to achieve a balance between cuts and fills.

Ordinary earth occupies from 5 to 26 percent less space after compaction than it did before excavation. Rock after excavation occupies a larger volume than it did in its original state. For example, if excavation rock is assumed to swell 25%, a factor of 1.25 is applied to the excavation yardage for use in the mass ordinates; should excavation be expected to shrink in embankments, say 10%, then those excavation quantities would be given a factor of 0.90 for use in the ordinates and if there will be neither swell nor shrinkage the factor is 1.00 <u>It is common practice to keep embankment quantities at unity and apply the swell or shrinkage factors to excavation only.</u>

Although it is obvious that the factors must be applied to make the cuts and fills balance, it should be remembered that overhaul is paid on the volume of the natural material in the cut, not compacted volume in the fill. <u>Since the station yards of overhaul as compacted in the mass diagram is adjusted yardage, it is necessary to apply the factor in reverse; that is divide the station yards of haul by the factor to get it back into transported yardage.</u>

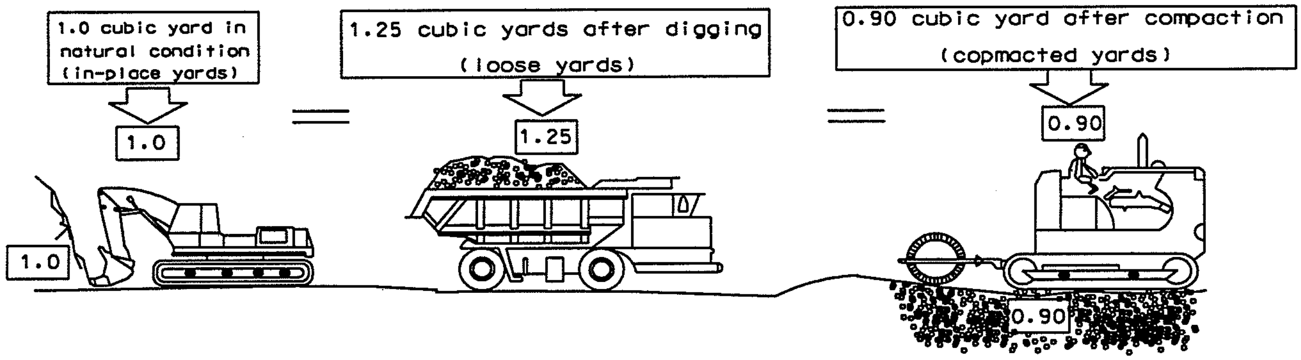


Figure I-8 Shrinkage and Swell of Soil During Earthmoving

Following are the equations for swell & shrinkage factors and compacted & bank volumes

Table I-4 Swell and Shrinkage Factors, BCY, LCY & CCY Relationships

Swell factor		Shrinkage factor	
$= \dfrac{Loose\ unit\ \ weight\ \ (Luw)}{Bank\ unit\ weight\,(Buw)}$	(33)	$= \dfrac{Bank\ unit\ \ weight\ (Buw)}{Compacted\ unit\ weight\ (Cuw)}$	(35)
$= \dfrac{1}{(1+Swell)}$	(34)	$= \dfrac{1}{(1-Shrinkage)}$	(36)
$Compacted\ \ Volume(CCY) = \ Bank\,Volume(BCY) \times Shrinkage\ Factor$			(37)
$Bank\ Volume\ (BCY) = Loose\ Volume\ (LCY) \times Swell\ Factor$			(38)

The following figure shows the swell and shrinkage factors in relation to the condition of the soil.

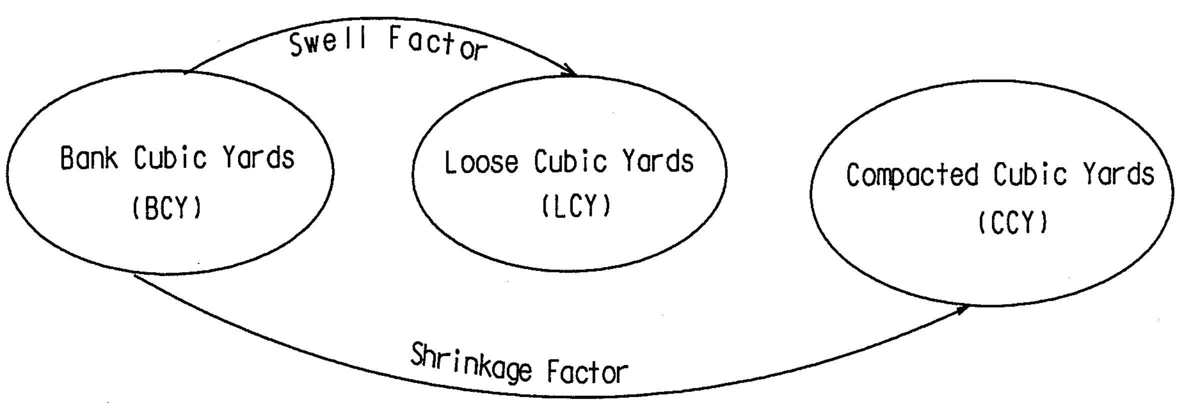
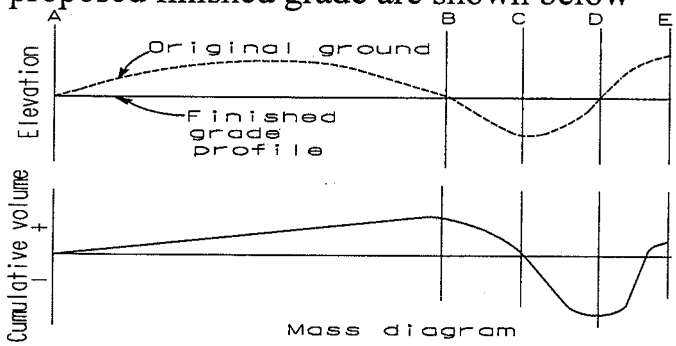


Figure I-9 Shrinkage and Swell Factors

Sample Problem I-2: Properties of Mass Diagram

Given: The original ground and the proposed finished grade are shown below



Find: Which of the following statement(s) is (are) true:

I. Section "B-D" represents a fill operation.
II. The job is balanced.
III. Section "D" represents a transition point between fill and cut.

(A) I
(B) II
(C) III
(D) I, III

Solution:

<u>Statement I</u> is true where the transition from cut to fill is occurring at section "B" because the OG is below the finished grade.

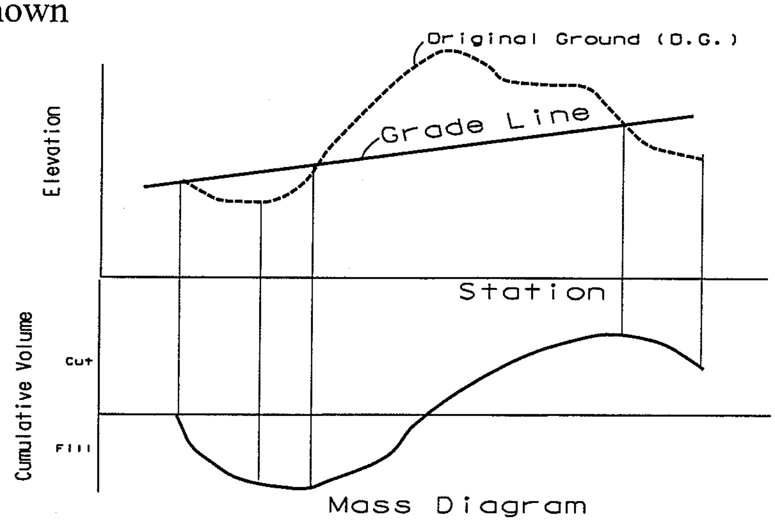
<u>Statement II</u> is NOT true because the mass diagram ended above the balance line which means there is cut more than fill i.e. there is EXCESS soil resulting from the project.

<u>Statement III</u> is true because at section D is the low point on the mass diagram. A low point represents a change from FILL to CUT.

Answer: D⇐

Sample Problem I-3: Properties of Mass Diagram

Given: The original ground and the proposed finished grade as shown



Find: Sketch the mass diagram showing low and high points. Also, indicate the key points on the diagram.

Sample Problem I-4: Volume and Shrinkage Calculations

Given: The following data of the cross section for a new highway is given in the table below. The lab tests of a soil sample taken from the site indicated a 15 % shrinkage.

Station	Cut (m^2)	Fill (m^2)
7 + 50	0	50
8 + 70	28	70
9 + 85	100	0

Find: The new waste or borrow for this segment of the highway is most nearly:

(A) borrow 1403.33 m3
(B) waste 1403.33 m3
(C) borrow 2675.33 m3
(D) waste 2675.33 m3

Solution:

➢ *Concept*:

1- The volume of cut between Sta. 7 + 50 and Sta. 8 + 70 is calculated by the pyramid method because one end area is ZERO. Similarly, the fill between Sta. 8 + 70 and 9 + 85.

2- The volume of cut and fill between other station will be calculated using the average end area method.

3- The adjustment for the shrinkage will be applicable only to CUT

➢ *Steps:*

1- Cut Volume Calculations:

$$V_{Cut} = \frac{1}{3} \times 28 \times \{(8+70)-(7+50)\ \} + \left(\frac{28+100}{2}\right)\{(9+85)-(8+70)\} = 1120 + 7360 = 8480\, m^3$$

2- Adjustment for the Cut Volume:

$$(V_{cut})_{adjusted} = (8480)(1-0.15) = 7208\, m^3$$

3- Fill Volume Calculations:

$$V_{Fill} = \left(\frac{50+70}{2}\right)\{(8+70)-(7+50)\} + \frac{1}{3} \times 70 \times \{(9+85)-(8+70)\} = 7200 + 2683.33$$

$$= 9883.33\, m^3$$

4- Net Volume Calculations:

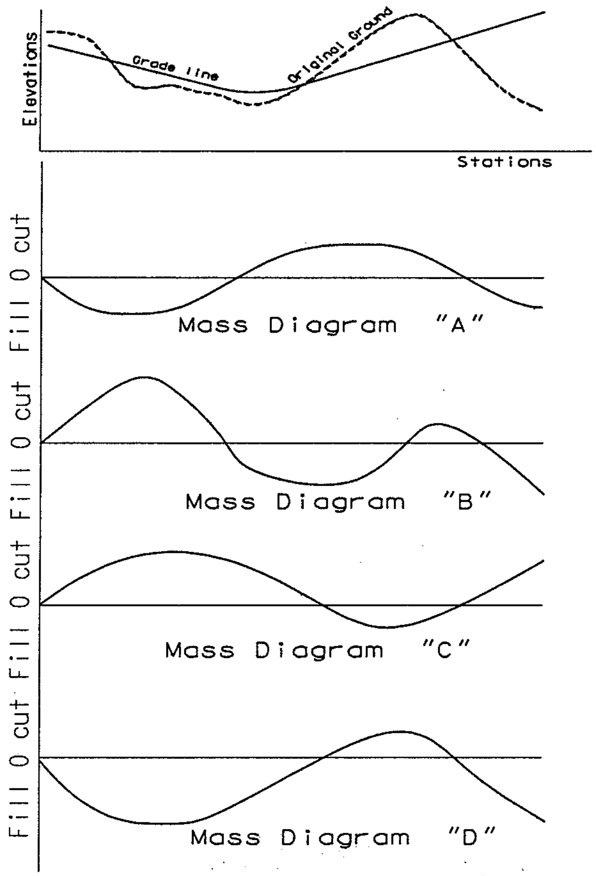
The volume of fill is greater than the adjusted volume of cut. Therefore, a borrow is needed for the project.

$$V_{Net} = V_{Fillt} - V_{cut} = 9883.33 - 7208 = 2675.33\, m^3$$

Answer: C ⇐

Sample Problem I-5: Properties of Mass Diagram

Given: O.G. & grade line as shown. Four mass diagrams are given below.



Find: The mass diagram that most nearly represents the given information is:

(A) Mass Diagram A
(B) Mass Diagram B
(C) Mass Diagram C
(D) Mass Diagram D

Solution:

Answer: B ⇐

Chapter II: Estimating Quantities and Costs

II-C ENGINEERING ECONOMICS

Factor Name	Converts	Symbol	Formula
Single Payment Compound Amount	to F given P	(F/P, i%, n)	$(1+i)^n$ (39)
Single Payment Present Worth	to P given F	(P/F, i%, n)	$(1+i)^{-n}$ (40)
Uniform Series Sinking Fund	to A given F	(A/F, i%, n)	$\dfrac{i}{(1+i)^n-1}$ (41)
Capital Recovery	to A given P	(A/P, i%, n)	$\dfrac{i(1+i)^n}{(1+i)^n-1}$ (42)
Uniform Series Compound Amount	to F given A	(F/A, i%, n)	$\dfrac{(1+i)^n-1}{i}$ (43)
Uniform Series Present Worth	to P given A	(P/A, i%, n)	$\dfrac{(1+i)^n-1}{i(1+i)^n}$ (44)
Uniform Gradient Present Worth	to P given G	(P/G, i%, n)	$\dfrac{(1+i)^n-1}{i^2(1+i)^n}-\dfrac{n}{i(1+i)^n}$ (45)
Uniform Gradient Future Worth*	to F given G	(F/G, i%, n)	$\dfrac{(1+i)^n-1}{i^2}-\dfrac{n}{i}$ (46)
Uniform Gradient Uniform Series	to A given G	(A/G, i%, n)	$\dfrac{1}{i}-\dfrac{n}{(1+i)^n-1}$ (47)

* $F/G = (F/A) \times (A/G)$ (48)

NOMENCLATURE AND DEFINITIONS

A .... Uniform amount per interest period

B .... Benefit

BV........ Book value

C.......... Cost

d........... Combined interest rate per interest period
D_j ....Depreciation in year j
F.......... Future worth, value, or amount
f .... General inflation rate per interest period
G.......... Uniform gradient amount per interest period
i .... Interest rate per interest period
i_e .... Annual effective interest rate
m.......... Number of compounding periods per year
n........... Number of compounding periods; or the expected life of an asset
P.......... Present worth, value, or amount
r........... Nominal annual interest rate
S_n .... Expected salvage value in year n

Subscripts

j .... at time j
n........... at time n

NON-ANNUAL COMPOUNDING:

$$i_e = \left(1 + \frac{r}{m}\right)^m - 1 \qquad (49)$$

BREAK-EVEN ANALYSIS:

By altering the value of any one of the variables in a situation, holding all of the other values constant, it is possible to find a value for that variable that makes the two alternatives equally economical. This value is the break-even point.

Break-even analysis is used to describe the percentage of capacity of operation for a manufacturing plant at which income will just cover expenses.

The payback period is the period of time required for the profit or other benefits of an investment to equal the cost of the investment.

INFLATION:

To account for inflation, the dollars are deflated by the general inflation rate per interest period f, and then they are shifted over the time scale using the interest rate per interest period i. Use a combined interest rate per interest period d for computing present worth values P and Net P. The formula for d is:

$$d = i + f + (i \times f) \qquad (50)$$

DEPRECIATION:

Depreciation is the decline in market value of the piece of equipment due wear and tear and age of the equipment. Any depreciation calculation must correspond to the IRS guideline for equipment life, which is five years. It should be noted that the cost of tires (for wheeled equipment) is subtracted from the purchase price due to different years of life between equipment and tires.

1- Straight Line:

The straight line method of depreciation provides uniform annual depreciation for each year of equipment life. The annual depreciation is calculated as follows:

$$D_j = \frac{C - S_n}{n} \qquad (51)$$

2- Accelerated Cost Recovery System (ACRS):

$$D_j = (factor)\ C \qquad 52)$$

A table of modified factors is provided below.

MODIFIED ACRS FACTORS				
	Recovery Period (Years)			
	3	5	7	10
Year	Recovery Rate (Percent)			
1	33.3	20.0	14.3	10.0
2	44.5	32.0	24.5	18.0
3	14.8	19.2	17.5	14.4
4	7.4	11.5	12.5	11.5
5		11.5	8.9	**9.2**
6		5.8	8.9	7.4
7			8.9	6.6
8			4.5	6.6
9				6.5
10				6.5
11				3.3

3- Sum of the Years Digits (SOYD):

The sum of the years digits method (SOYD) of depreciation provides a nonuniform depreciation, with higher amounts in the first years, decreasing in amount through the fifth year. The calculation involves multiplying the digit for the year in inverse order times the amount to depreciate, and then dividing by the sum of the years' digits (1 + 2 + 3 + 4 + 5 = 15 for a five-year life).

$$D_j = \frac{n+1-j}{\sum_{j=1}^{n} j}(C - S_n) \tag{53}$$

BOOK VALUE:

$$BV = initial\ cost - \Sigma D_j \tag{54}$$

TAXATION:

Income taxes are paid at a specific rate on taxable income. Taxable income is total income less depreciation and ordinary expenses. Expenses do not include capital items, which should be depreciated.

CAPITALIZED COSTS:

Capitalized costs are present worth values using an assumed perpetual period of time.

$$\text{Capitalized Costs} = P = \frac{A}{i} \tag{55}$$

BONDS:

Bond Value equals the present worth of the payments the purchaser (or holder of the bond) receives during the life of the bond at some interest rate i. Bond Yield equals the computed interest rate of the bond value when compared with the bond cost.

RATE-OF-RETURN:

The minimum acceptable rate-of-return (MARR) is that interest rate that one is willing to accept, or the rate one desires to earn on investments. The rate-of-return on an investment is the interest rate that makes the benefits and costs equal.

BENEFIT-COST ANALYSIS:

In a benefit-cost analysis, the benefits B of a project should exceed the estimated costs C.

$$B - C \geq 0,\ or\ B/C \geq 1 \tag{56}$$

Compound Interest Factors
$i = 0.50$ %

n	P/F	P/A	P/G	F/P	F/A	A/P	A/F	A/G
1	0.9950	0.9950	0.0000	1.0050	1.0000	1.0050	1.0000	0.0000
2	0.9901	1.9851	0.9901	1.0100	2.0050	0.5038	0.4988	0.4988
3	0.9851	2.9702	2.9604	1.0151	3.0150	0.3367	0.3317	0.9967
4	0.9802	3.9505	5.9011	1.0202	4.0301	0.2531	0.2481	1.4938
5	**0.9754**	**4.9259**	**9.8026**	**1.0253**	**5.0503**	**0.2030**	**0.1980**	**1.9900**
6	0.9705	5.8964	14.6552	1.0304	6.0755	0.1696	0.1646	2.4855
7	0.9657	6.8621	20.4493	1.0355	7.1059	0.1457	0.1407	2.9801
8	0.9609	7.8230	27.1755	1.0407	8.1414	0.1278	0.1228	3.4738
9	0.9561	8.7791	34.8244	1.0459	9.1821	0.1139	0.1089	3.9668
10	**0.9513**	**9.7304**	**43.3865**	**1.0511**	**10.2280**	**0.1028**	**0.0978**	**4.4589**
11	0.9466	10.6770	52.8526	1.0564	11.2792	0.0937	0.0887	4.9501
12	0.9419	11.6189	63.2136	1.0617	12.3356	0.0861	0.0811	5.4406
13	0.9372	12.5562	74.4602	1.0670	13.3972	0.0796	0.0746	5.9302
14	0.9326	13.4887	86.5835	1.0723	14.4642	0.0741	0.0691	6.4190
15	**0.9279**	**14.4166**	**99.5743**	**1.0777**	**15.5365**	**0.0694**	**0.0644**	**6.9069**
16	0.9233	15.3399	113.4238	1.0831	16.6142	0.0652	0.0602	7.3940
17	0.9187	16.2586	128.1231	1.0885	17.6973	0.0615	0.0565	7.8803
18	0.9141	17.1728	143.6634	1.0939	18.7858	0.0582	0.0532	8.3658
19	0.9096	18.0824	160.0360	1.0994	19.8797	0.0553	0.0503	8.8504
20	**0.9051**	**18.9874**	**177.2322**	**1.1049**	**20.9791**	**0.0527**	**0.0477**	**9.3342**
21	0.9006	19.8880	195.2434	1.1104	22.0840	0.0503	0.0453	9.8172
22	0.8961	20.7841	214.0611	1.1160	23.1944	0.0481	0.0431	10.2993
23	0.8916	21.6757	233.6768	1.1216	24.3104	0.0461	0.0411	10.7806
24	0.8872	22.5629	254.0820	1.1272	25.4320	0.0443	0.0393	11.2611
25	**0.8828**	**23.4456**	**275.2686**	**1.1328**	**26.5591**	**0.0427**	**0.0377**	**11.7407**
30	0.8610	27.7941	392.6324	1.1614	32.2800	0.0360	0.0310	14.1265
40	0.8191	36.1722	681.3347	1.2208	44.1588	0.0276	0.0226	18.8359
50	0.7793	44.1428	1,035.6966	1.2832	56.6452	0.0227	0.0177	23.4624
60	0.7414	51.7256	1,448.6458	1.3489	69.7700	0.0193	0.0143	28.0064
100	**0.6073**	**78.5426**	**3,562.7934**	**1.6467**	**129.3337**	**0.0127**	**0.0077**	**45.3613**

Compound Interest Factors
$i = 1.00$ %

n	P/F	P/A	P/G	F/P	F/A	A/P	A/F	A/G
1	0.9901	0.9901	0.0000	1.0100	1.0000	1.0100	1.0000	0.0000
2	0.9803	1.9704	0.9803	1.0201	2.0100	0.5075	0.4975	0.4975
3	0.9706	2.9410	2.9215	1.0303	3.0301	0.3400	0.3300	0.9934
4	0.9610	3.9020	5.8044	1.0406	4.0604	0.2563	0.2463	1.4876
5	**0.9515**	**4.8534**	**9.6103**	**1.0510**	**5.1010**	**0.2060**	**0.1960**	**1.9801**
6	0.9420	5.7955	14.3205	1.0615	6.1520	0.1725	0.1625	2.4710
7	0.9327	6.7282	19.9168	1.0721	7.2135	0.1486	0.1386	2.9602
8	0.9235	7.6517	26.3812	1.0829	8.2857	0.1307	0.1207	3.4478
9	0.9143	8.5650	33.6959	1.0937	9.3685	0.1167	0.1067	3.9337
10	**0.9053**	**9.4713**	**41.8435**	**1.1046**	**10.4622**	**0.1056**	**0.0956**	**4.4179**
11	0.8963	10.3676	50.8067	1.1157	11.5668	0.0965	0.0865	4.9005
12	0.8874	11.2551	60.5687	1.1268	12.6825	0.0888	0.0788	5.3815
13	0.8787	12.1337	71.1126	1.1381	13.8093	0.0824	0.0724	5.8607
14	0.8700	13.0037	82.4221	1.1495	14.9474	0.0769	0.0669	6.3384
15	**0.8613**	**13.8651**	**94.4810**	**1.1610**	**16.0969**	**0.0721**	**0.0621**	**6.8143**
16	0.8528	14.7179	107.2734	1.1726	17.2579	0.0679	0.0579	7.2886
17	0.8444	15.5623	120.7834	1.1843	18.4304	0.0643	0.0543	7.7613
18	0.8360	16.3983	134.9957	1.1961	19.6147	0.0610	0.0510	8.2323
19	0.8277	17.2260	149.8950	1.2081	20.8109	0.0581	0.0481	8.7017
20	**0.8195**	**18.0456**	**165.4664**	**1.2202**	**22.0190**	**0.0554**	**0.0454**	**9.1694**
21	0.8114	18.8570	181.6950	1.2324	23.2392	0.0530	0.0430	9.6354
22	0.8034	19.6604	198.5663	1.2447	24.4716	0.0509	0.0409	10.0998
23	0.7954	20.4558	216.0660	1.2572	25.7163	0.0489	0.0389	10.5626
24	0.7876	21.2434	234.1800	1.2697	26.9735	0.0471	0.0371	11.0237
25	**0.7798**	**22.0232**	**252.8945**	**1.2824**	**28.2432**	**0.0454**	**0.0354**	**11.4831**
30	0.7419	25.8077	355.0021	1.3478	34.7849	0.0387	0.0277	13.7557
40	0.6717	32.8347	596.8561	1.4889	48.8864	0.0305	0.0205	18.1776
50	0.6080	39.1961	879.4176	1.6446	64.4632	0.0255	0.0155	22.4363
60	0.5504	44.9550	1,192.8061	1.8167	81.6697	0.0222	0.0122	26.5333
100	**0.3697**	**63.0289**	**2,605.7758**	**2.7048**	**170.4814**	**0.0159**	**0.0059**	**41.3426**

Compound Interest Factors
i = 1.50 %

n	P/F	P/A	P/G	F/P	F/A	A/P	A/F	A/G
1	0.9852	0.9852	0.0000	1.0150	1.0000	1.0150	1.0000	0.0000
2	0.9707	1.9559	0.9707	1.0302	2.0150	0.5113	0.4963	0.4963
3	0.9563	2.9122	2.8833	1.0457	3.0452	0.3434	0.3284	0.9901
4	0.9422	3.8544	5.7098	1.0614	4.0909	0.2594	0.2444	1.4814
5	0.9283	4.7826	9.4229	1.0773	5.1523	0.2091	0.1941	1.9702
6	0.9145	5.6972	13.9956	1.0934	6.2296	0.1755	0.1605	2.4566
7	0.9010	6.5982	19.4018	1.1098	7.3230	0.1516	0.1366	2.9405
8	0.8877	7.4859	26.6157	1.1265	8.4328	0.1336	0.1186	3.4219
9	0.8746	8.3605	32.6125	1.1434	9.5593	0.1196	0.1046	3.9008
10	0.8617	9.2222	40.3675	1.1605	10.7027	0.1084	0.0934	4.3772
11	0.8489	10.0711	48.8568	1.1779	11.8633	0.0993	0.0843	4.8512
12	0.8364	10.9075	58.0571	1.1956	13.0412	0.0917	0.0767	5.3227
13	0.8240	11.7315	67.9454	1.2136	14.2368	0.0852	0.0702	5.7917
14	0.8118	12.5434	78.4994	1.2318	15.4504	0.0797	0.0647	6.2582
15	0.7999	13.3432	89.6974	1.2502	16.6821	0.0749	0.0599	6.7223
16	0.7880	14.1313	101.5178	1.2690	17.9324	0.0708	0.0558	7.1839
17	0.7764	14.9076	113.9400	1.2880	19.2014	0.0671	0.0521	7.6431
18	0.7649	15.6726	126.9435	1.3073	20.4894	0.0638	0.0488	8.0997
19	0.7536	16.4262	140.5084	1.3270	21.7967	0.0609	0.0459	8.5539
20	0.7425	17.1686	154.6154	1.3469	23.1237	0.0582	0.0432	9.0057
21	0.7315	17.9001	169.2453	1.3671	24.4705	0.0559	0.0409	9.4550
22	0.7207	18.6208	184.3798	1.3876	25.8376	0.0537	0.0387	9.9018
23	0.7100	19.3309	200.0006	1.4084	27.2251	0.0517	0.0367	10.3462
24	0.6995	20.0304	216.0901	1.4295	28.6335	0.0499	0.0349	10.7881
25	0.6892	20.7196	232.6310	1.4509	30.0630	0.0483	0.0333	11.2276
30	0.6398	24.0158	321.5310	1.5631	37.5387	0.0416	0.0266	13.3883
40	0.5513	29.9158	524.3568	1.8140	54.2679	0.0334	0.0184	17.5277
50	0.4750	34.9997	749.9636	2.1052	73.6828	0.0286	0.0136	21.4277
60	0.4093	39.3803	988.1674	2.4432	96.2147	0.0254	0.0104	25.0930
100	0.2256	51.6247	1,937.4506	4.4320	228.8030	0.0194	0.0044	37.5295

Compound Interest Factors
i = 2.00 %

n	P/F	P/A	P/G	F/P	F/A	A/P	A/F	A/G
1	0.9804	0.9804	0.0000	1.0200	1.0000	1.0200	1.0000	0.0000
2	0.9612	1.9416	0.9612	1.0404	2.0200	0.5150	0.4950	0.4950
3	0.9423	2.8839	2.8458	1.0612	3.0604	0.3468	0.3268	0.9868
4	0.9238	3.8077	5.6173	1.0824	4.1216	0.2626	0.2426	1.4752
5	0.9057	4.7135	9.2403	1.1041	5.2040	0.2122	0.1922	1.9604
6	0.8880	5.6014	13.6801	1.1262	6.3081	0.1785	0.1585	2.4423
7	0.8706	6.4720	18.9035	1.1487	7.4343	0.1545	0.1345	2.9208
8	0.8535	7.3255	24.8779	1.1717	8.5830	0.1365	0.1165	3.3961
9	0.8368	8.1622	31.5720	1.1951	9.7546	0.1225	0.1025	3.8681
10	0.8203	8.9826	38.9551	1.2190	10.9497	0.1113	0.0913	4.3367
11	0.8043	9.7868	46.9977	1.2434	12.1687	0.1022	0.0822	4.8021
12	0.7885	10.5753	55.6712	1.2682	13.4121	0.0946	0.0746	5.2642
13	0.7730	11.3484	64.9475	1.2936	14.6803	0.0881	0.0681	5.7231
14	0.7579	12.1062	74.7999	1.3195	15.9739	0.0826	0.0626	6.1786
15	0.7430	12.8493	85.2021	1.3459	17.2934	0.0778	0.0578	6.6309
16	0.7284	13.5777	96.1288	1.3728	18.6393	0.0737	0.0537	7.0799
17	0.7142	14.2919	107.5554	1.4002	20.0121	0.0700	0.0500	7.5256
18	0.7002	14.9920	119.4581	1.4282	21.4123	0.0667	0.0467	7.9681
19	0.6864	15.6785	131.8139	1.4568	22.8406	0.0638	0.0438	8.4073
20	0.6730	16.3514	144.6003	1.4859	24.2974	0.0612	0.0412	8.8433
21	0.6598	17.0112	157.7959	1.5157	25.7833	0.0588	0.0388	9.2760
22	0.6468	17.6580	171.3795	1.5460	27.2990	0.0566	0.0366	9.7055
23	0.6342	18.2922	185.3309	1.5769	28.8450	0.0547	0.0347	10.1317
24	0.6217	18.9139	199.6305	1.6084	30.4219	0.0529	0.0329	10.5547
25	0.6095	19.5235	214.2592	1.6406	32.0303	0.0512	0.0312	10.9745
30	0.5521	22.3965	291.7164	1.8114	40.5681	0.0446	0.0246	13.0251
40	0.4529	27.3555	461.9931	2.2080	60.4020	0.0366	0.0166	16.8885
50	0.3715	31.4236	642.3606	2.6916	84.5794	0.0318	0.0118	20.4420
60	0.3048	34.7609	823.6975	3.2810	114.0515	0.0288	0.0088	23.6961
100	0.1380	43.0984	1,464.7527	7.2446	312.2323	0.0232	0.0032	33.9863

Compound Interest Factors
$i = 4.00\ \%$

n	P/F	P/A	P/G	F/P	F/A	A/P	A/F	A/G
1	0.9615	0.9615	0.0000	1.0400	1.0000	1.0400	1.0000	0.0000
2	0.9246	1.8861	0.9246	1.0816	2.0400	0.5302	0.4902	0.4902
3	0.8890	2.7751	2.7025	1.1249	3.1216	0.3603	0.3203	0.9739
4	0.8548	3.6299	5.2670	1.1699	4.2465	0.2755	0.2355	1.4510
5	0.8219	4.4518	8.5547	1.2167	5.4163	0.2246	0.1846	1.9216
6	0.7903	5.2421	12.5062	1.2653	6.6330	0.1908	0.1508	2.3857
7	0.7599	6.0021	17.0657	1.3159	7.8983	0.1666	0.1266	2.8433
8	0.7307	6.7327	22.1806	1.3686	9.2142	0.1485	0.1085	3.2944
9	0.7026	7.4353	27.8013	1.4233	10.5828	0.1345	0.0945	3.7391
10	0.6756	8.1109	33.8814	1.4802	12.0061	0.1233	0.0833	4.1773
11	0.6496	8.7605	40.3772	1.5395	13.4864	0.1141	0.0741	4.6090
12	0.6246	9.3851	47.2477	1.6010	15.0258	0.1066	0.0666	5.0343
13	0.6006	9.9856	54.4546	1.6651	16.6268	0.1001	0.0601	5.4533
14	0.5775	10.5631	61.9618	1.7317	18.2919	0.0947	0.0547	5.8659
15	0.5553	11.1184	69.7355	1.8009	20.0236	0.0899	0.0499	6.2721
16	0.5339	11.6523	77.7441	1.8730	21.8245	0.0858	0.0458	6.6720
17	0.5134	12.1657	85.9581	1.9479	23.6975	0.0822	0.0422	7.0656
18	0.4936	12.6593	94.3498	2.0258	25.6454	0.0790	0.0390	7.4530
19	0.4746	13.1339	102.8933	2.1068	27.6712	0.0761	0.0361	7.8342
20	0.4564	13.5903	111.5647	2.1911	29.7781	0.0736	0.0336	8.2091
21	0.4388	14.0292	120.3414	2.2788	31.9692	0.0713	0.0313	8.5779
22	0.4220	14.4511	129.2024	2.3699	34.2480	0.0692	0.0292	8.9407
23	0.4057	14.8568	138.1284	2.4647	36.6179	0.0673	0.0273	9.2973
24	0.3901	15.2470	147.1012	2.5633	39.0826	0.0656	0.0256	9.6479
25	0.3751	15.6221	156.1040	2.6658	41.6459	0.0640	0.0240	9.9925
30	0.3083	17.2920	201.0618	3.2434	56.0849	0.0578	0.0178	11.6274
40	0.2083	19.7928	286.5303	4.8010	95.0255	0.0505	0.0105	14.4765
50	0.1407	21.4822	361.1638	7.1067	152.6671	0.0466	0.0066	16.8122
60	0.0951	22.6235	422.9966	10.5196	237.9907	0.0442	0.0042	18.6972
100	0.0198	24.5050	563.1249	50.5049	1,237.6237	0.0408	0.0008	22.9800

Compound Interest Factors
i = 6.00 %

n	*P/F*	*P/A*	*P/G*	*F/P*	*F/A*	*A/P*	*A/F*	*A/G*
1	0.9434	0.9434	0.0000	1.0600	1.0000	1.0600	1.0000	0.0000
2	0.8900	1.8334	0.8900	1.1236	2.0600	0.5454	0.4854	0.4854
3	0.8396	2.6730	2.5692	1.1910	3.1836	0.3741	0.3141	0.9612
4	0.7921	3.4651	4.9455	1.2625	4.3746	0.2886	0.2286	1.4272
5	0.7473	4.2124	7.9345	1.3382	5.6371	0.2374	0.1774	1.8836
6	0.7050	4.9173	11.4594	1.4185	6.9753	0.2034	0.1434	2.3304
7	0.6651	5.5824	15.4497	1.5036	8.3938	0.1791	0.1191	2.7676
8	0.6274	6.2098	19.8416	1.5938	9.8975	0.1610	0.1010	3.1952
9	0.5919	6.8017	24.5768	1.6895	11.4913	0.1470	0.0870	3.6133
10	0.5584	7.3601	29.6023	1.7908	13.1808	0.1359	0.0759	4.0220
11	0.5268	7.8869	34.8702	1.8983	14.9716	0.1268	0.0668	4.4213
12	0.4970	8.3838	40.3369	2.0122	16.8699	0.1193	0.0593	4.8113
13	0.4688	8.8527	45.9629	2.1329	18.8821	0.1130	0.0530	5.1920
14	0.4423	9.2950	51.7128	2.2609	21.0151	0.1076	0.0476	5.5635
15	0.4173	9.7122	57.5546	2.3966	23.2760	0.1030	0.0430	5.9260
16	0.3936	10.1059	63.4592	2.5404	25.6725	0.0990	0.0390	6.2794
17	0.3714	10.4773	69.4011	2.6928	28.2129	0.0954	0.0354	6.6240
18	0.3505	10.8276	75.3569	2.8543	30.9057	0.0924	0.0324	6.9597
19	0.3305	11.1581	81.3062	3.0256	33.7600	0.0896	0.0296	7.2867
20	0.3118	11.4699	87.2304	3.2071	36.7856	0.0872	0.0272	7.6051
21	0.2942	11.7641	93.1136	3.3996	39.9927	0.0850	0.0250	7.9151
22	0.2775	12.0416	98.9412	3.6035	43.3923	0.0830	0.0230	8.2166
23	0.2618	12.3034	104.7007	3.8197	46.9958	0.0813	0.0213	8.5099
24	0.2470	12.5504	110.3812	4.0489	50.8156	0.0797	0.0197	8.7951
25	0.2330	12.7834	115.9732	4.2919	54.8645	0.0782	0.0182	9.0722
30	0.1741	13.7648	142.3588	5.7435	79.0582	0.0726	0.0126	10.3422
40	0.0972	15.0463	185.9568	10.2857	154.7620	0.0665	0.0065	12.3590
50	0.0543	15.7619	217.4574	18.4202	290.3359	0.0634	0.0034	13.7964
60	0.0303	16.1614	239.0428	32.9877	533.1282	0.0619	0.0019	14.7909
100	0.0029	16.6175	272.0471	339.3021	5,638.3681	0.0602	0.0002	16.3711

Compound Interest Factors
$i = 8.00$ %

n	P/F	P/A	P/G	F/P	F/A	A/P	A/F	A/G
1	0.9259	0.9259	0.0000	1.0800	1.0000	1.0800	1.0000	0.0000
2	0.8573	1.7833	0.8573	1.1664	2.0800	0.5608	0.4808	0.4808
3	0.7938	2.5771	2.4450	1.2597	3.2464	0.3880	0.3080	0.9487
4	0.7350	3.3121	4.6501	1.3605	4.5061	0.3019	0.2219	1.4040
5	0.6806	3.9927	7.3724	1.4693	5.8666	0.2505	0.1705	1.8465
6	0.6302	4.6229	10.5233	1.5869	7.3359	0.2163	0.1363	2.2763
7	0.5835	5.2064	14.0242	1.7138	8.9228	0.1921	0.1121	2.6937
8	0.5403	5.7466	17.8061	1.8509	10.6366	0.1740	0.0940	3.0985
9	0.5002	6.2469	21.8081	1.9990	12.4876	0.1601	0.0801	3.4910
10	0.4632	6.7101	25.9768	2.1589	14.4866	0.1490	0.0690	3.8713
11	0.4289	7.1390	30.2657	2.3316	16.6455	0.1401	0.0601	4.2395
12	0.3971	7.5361	34.6339	2.5182	18.9771	0.1327	0.0527	4.5957
13	0.3677	7.9038	39.0463	2.7196	21.4953	0.1265	0.0465	4.9402
14	0.3405	8.2442	43.4723	2.9372	24.2149	0.1213	0.0413	5.2731
15	0.3152	8.5595	47.8857	3.1722	27.1521	0.1168	0.0368	5.5945
16	0.2919	8.8514	52.2640	3.4259	30.3243	0.1130	0.0330	5.9046
17	0.2703	9.1216	56.5883	3.7000	33.7502	0.1096	0.0296	6.2037
18	0.2502	9.3719	60.8426	3.9960	37.4502	0.1067	0.0267	6.4920
19	0.2317	9.6036	65.0134	4.3157	41.4463	0.1041	0.0241	6.7697
20	0.2145	9.8181	69.0898	4.6610	45.7620	0.1019	0.0219	7.0369
21	0.1987	10.0168	73.0629	5.0338	50.4229	0.0998	0.0198	7.2940
22	0.1839	10.2007	76.9257	5.4365	55.4568	0.0980	0.0180	7.5412
23	0.1703	10.3711	80.6726	5.8715	60.8933	0.0964	0.0164	7.7786
24	0.1577	10.5288	84.2997	6.3412	66.7648	0.0950	0.0150	8.0066
25	0.1460	10.6748	87.8041	6.8485	73.1059	0.0937	0.0137	8.2254
30	0.0994	11.2578	103.4558	10.0627	113.2832	0.0888	0.0088	9.1897
40	0.0460	11.9246	126.0422	21.7245	259.0565	0.0839	0.0039	10.5699
50	0.0213	12.2335	139.5928	46.9016	573.7702	0.0817	0.0017	11.4107
60	0.0099	12.3766	147.3000	101.2571	1,253.2133	0.0808	0.0008	11.9015
100	0.0005	12.4943	155.6107	2,199.7613	27,484.5157	0.0800		12.4545

Compound Interest Factors
i = 10.00 %

n	*P/F*	*P/A*	*P/G*	*F/P*	*F/A*	*A/P*	*A/F*	*A/G*
1	0.9091	0.9091	0.0000	1.1000	1.0000	1.1000	1.0000	0.0000
2	0.8264	1.7355	0.8264	1.2100	2.1000	0.5762	0.4762	0.4762
3	0.7513	2.4869	2.3291	1.3310	3.3100	0.4021	0.3021	0.9366
4	0.6830	3.1699	4.3781	1.4641	4.6410	0.3155	0.2155	1.3812
5	0.6209	3.7908	6.8618	1.6105	6.1051	0.2638	0.1638	1.8101
6	0.5645	4.3553	9.6842	1.7716	7.7156	0.2296	0.1296	2.2236
7	0.5132	4.8684	12.7631	1.9487	9.4872	0.2054	0.1054	2.6216
8	0.4665	5.3349	16.0287	2.1436	11.4359	0.1874	0.0874	3.0045
9	0.4241	5.7590	19.4215	2.3579	13.5735	0.1736	0.0736	3.3724
10	0.3855	6.1446	22.8913	2.5937	15.9374	0.1627	0.0627	3.7255
11	0.3505	6.4951	26.3962	2.8531	18.5312	0.1540	0.0540	4.0641
12	0.3186	6.8137	29.9012	3.1384	21.3843	0.1468	0.0468	4.3884
13	0.2897	7.1034	33.3772	3.4523	24.5227	0.1408	0.0408	4.6988
14	0.2633	7.3667	36.8005	3.7975	27.9750	0.1357	0.0357	4.9955
15	0.2394	7.6061	40.1520	4.1772	31.7725	0.1315	0.0315	5.2789
16	0.2176	7.8237	43.4164	4.5950	35.9497	0.1278	0.0278	5.5493
17	0.1978	8.0216	46.5819	5.0545	40.5447	0.1247	0.0247	5.8071
18	0.1799	8.2014	49.6395	5.5599	45.5992	0.1219	0.0219	6.0526
19	0.1635	8.3649	52.5827	6.1159	51.1591	0.1195	0.0195	6.2861
20	0.1486	8.5136	55.4069	6.7275	57.2750	0.1175	0.0175	6.5081
21	0.1351	8.6487	58.1095	7.4002	64.0025	0.1156	0.0156	6.7189
22	0.1228	8.7715	60.6893	8.1403	71.4027	0.1140	0.0140	6.9189
23	0.1117	8.8832	63.1462	8.9543	79.5430	0.1126	0.0126	7.1085
24	0.1015	8.9847	65.4813	9.8497	88.4973	0.1113	0.0113	7.2881
25	0.0923	9.0770	67.6964	10.8347	98.3471	0.1102	0.0102	7.4580
30	0.0573	9.4269	77.0766	17.4494	164.4940	0.1061	0.0061	8.1762
40	0.0221	9.7791	88.9525	45.2593	442.5926	0.1023	0.0023	9.0962
50	0.0085	9.9148	94.8889	117.3909	1,163.9085	0.1009	0.0009	9.5704
60	0.0033	9.9672	97.7010	304.4816	3,034.8164	0.1003	0.0003	9.8023
100	0.0001	9.9993	99.9202	13,780.6123	137,796.1234	0.1000		9.9927

Compound Interest Factors
i = 12.00 %

n	P/F	P/A	P/G	F/P	F/A	A/P	A/F	A/G
1	0.8929	0.8929	0.0000	1.1200	1.0000	1.1200	1.0000	0.0000
2	0.7972	1.6901	0.7972	1.2544	2.1200	0.5917	0.4717	0.4717
3	0.7118	2.4018	2.2208	1.4049	3.3744	0.4163	0.2963	0.9246
4	0.6355	3.0373	4.1273	1.5735	4.7793	0.3292	0.2092	1.3589
5	0.5674	3.6048	6.3970	1.7623	6.3528	0.2774	0.1574	1.7746
6	0.5066	4.1114	8.9302	1.9738	8.1152	0.2432	0.1232	2.1720
7	0.4523	4.5638	11.6443	2.2107	10.0890	0.2191	0.0991	2.5515
8	0.4039	4.9676	14.4714	2.4760	12.2997	0.2013	0.0813	2.9131
9	0.3606	5.3282	17.3563	2.7731	14.7757	0.1877	0.0677	3.2574
10	0.3220	5.6502	20.2541	3.1058	17.5487	0.1770	0.0570	3.5847
11	0.2875	5.9377	23.1288	3.4785	20.6546	0.1684	0.0484	3.8953
12	0.2567	6.1944	25.9523	3.8960	24.1331	0.1614	0.0414	4.1897
13	0.2292	6.4235	28.7024	4.3635	28.0291	0.1557	0.0357	4.4683
14	0.2046	6.6282	31.3624	4.8871	32.3926	0.1509	0.0309	4.7317
15	0.1827	6.8109	33.9202	5.4736	37.2797	0.1468	0.0268	4.9803
16	0.1631	6.9740	36.3670	6.1304	42.7533	0.1434	0.0234	5.2147
17	0.1456	7.1196	38.6973	6.8660	48.8837	0.1405	0.0205	5.4353
18	0.1300	7.2497	40.9080	7.6900	55.7497	0.1379	0.0179	5.6427
19	0.1161	7.3658	42.9979	8.6128	63.4397	0.1358	0.0158	5.8375
20	0.1037	7.4694	44.9676	9.6463	72.0524	0.1339	0.0139	6.0202
21	0.0926	7.5620	46.8188	10.8038	81.6987	0.1322	0.0122	6.1913
22	0.0826	7.6446	48.5543	12.1003	92.5026	0.1308	0.0108	6.3514
23	0.0738	7.7184	50.1776	13.5523	104.6029	0.1296	0.0096	6.5010
24	0.0659	7.7843	51.6929	15.1786	118.1552	0.1285	0.0085	6.6406
25	0.0588	7.8431	53.1046	17.0001	133.3339	0.1275	0.0075	6.7708
30	0.0334	8.0552	58.7821	29.9599	241.3327	0.1241	0.0041	7.2974
40	0.0107	8.2438	65.1159	93.0510	767.0914	0.1213	0.0013	7.8988
50	0.0035	8.3045	67.7624	289.0022	2,400.0182	0.1204	0.0004	8.1597
60	0.0011	8.3240	68.8100	897.5969	7,471.6411	0.1201	0.0001	8.2664
100		8.3332	69.4336	83,522.2657	696,010.5477	0.1200		8.3321

Compound Interest Factors
i = 18.00 %

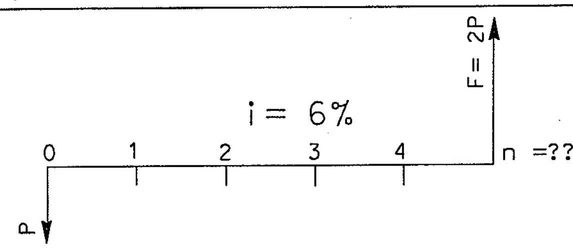
n	P/F	P/A	P/G	F/P	F/A	A/P	A/F	A/G
1	0.8475	0.8475	0.0000	1.1800	1.0000	1.1800	1.0000	0.0000
2	0.7182	1.5656	0.7182	1.3924	2.1800	0.6387	0.4587	0.4587
3	0.6086	2.1743	1.9354	1.6430	3.5724	0.4599	0.2799	0.8902
4	0.5158	2.6901	3.4828	1.9388	5.2154	0.3717	0.1917	1.2947
5	0.4371	3.1272	5.2312	2.2878	7.1542	0.3198	0.1398	1.6728
6	0.3704	3.4976	7.0834	2.6996	9.4423	0.2859	0.1059	2.0252
7	0.3139	3.8115	8.9670	3.1855	12.1415	0.2624	0.0824	2.3526
8	0.2660	4.0776	10.8292	3.7589	15.3270	0.2452	0.0652	2.6558
9	0.2255	4.3030	12.6329	4.4355	19.0859	0.2324	0.0524	2.9358
10	0.1911	4.4941	14.3525	5.2338	23.5213	0.2225	0.0425	3.1936
11	0.1619	4.6560	15.9716	6.1759	28.7551	0.2148	0.0348	3.4303
12	0.1372	4.7932	17.4811	7.2876	34.9311	0.2086	0.0286	3.6470
13	0.1163	4.9095	18.8765	8.5994	42.2187	0.2037	0.0237	3.8449
14	0.0985	5.0081	20.1576	10.1472	50.8180	0.1997	0.0197	4.0250
15	0.0835	5.0916	21.3269	11.9737	60.9653	0.1964	0.0164	4.1887
16	0.0708	5.1624	22.3885	14.1290	72.9390	0.1937	0.0137	4.3369
17	0.0600	5.2223	23.3482	16.6722	87.0680	0.1915	0.0115	4.4708
18	0.0508	5.2732	24.2123	19.6731	103.7403	0.1896	0.0096	4.5916
19	0.0431	5.3162	24.9877	23.2144	123.4135	0.1881	0.0081	4.7003
20	0.0365	5.3527	25.6813	27.3930	146.6280	0.1868	0.0068	4.7978
21	0.0309	5.3837	26.3000	32.3238	174.0210	0.1857	0.0057	4.8851
22	0.0262	5.4099	26.8506	38.1421	206.3448	0.1848	0.0048	4.9632
23	0.0222	5.4321	27.3394	45.0076	244.4868	0.1841	0.0041	5.0329
24	0.0188	5.4509	27.7725	53.1090	289.4944	0.1835	0.0035	5.0950
25	0.0159	5.4669	28.1555	62.6686	342.6035	0.1829	0.0029	5.1502
30	0.0070	5.5168	29.4864	143.3706	790.9480	0.1813	0.0013	5.3448
40	0.0013	5.5482	30.5269	750.3783	4,163.2130	0.1802	0.0002	5.5022
50	0.0003	5.5541	30.7856	3,927.3569	21,813.0937	0.1800		5.5428
60	0.0001	5.5553	30.8465	20,555.1400	114,189.6665	0.1800		5.5526
100		5.5556	30.8642	15,424,131.91	85,689,616.17	0.1800		5.5555

Sample Problem II-1

Given: An investment at a 6% effective annual rate?	***Find:*** The number of years it will take to double the money is most nearly: (A) 10 years (B) 12 years (C) 15 years (D) 17 years

Solution:

Two ways to solve: Interest tables or Equation



Interest Tables:

$$\left(F/P\right)_{n=??}^{i=6\%} = 2$$

$\Rightarrow$ n is between 11 (1.8983) and 12 (2.0122)

Equation: The relationship between P & F is

$$F = 2P = P(1+i)^n \qquad \Rightarrow 2 = (1+i)^n$$

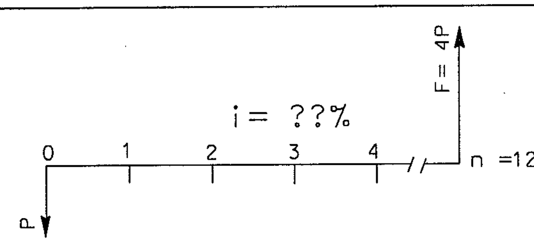
$$\log 2 = n\log(1+i) \qquad \Rightarrow n = \frac{\log 2}{\log(1+0.06)} = 11.8956 \cong 12$$

Answer: B $\Leftarrow$

Sample Problem II-2

Given: An investment with unknown amount	***Find:*** The rate of annual interest that will quadruple an investment in 12 years is most nearly? (A) 10.1 % (B) 11.2 % (C) 12.2 % (D) 13.1 %

Solution:



Interest Tables: $\left(F/P\right)_{n=12}^{i=??\%} = 4$

$\Rightarrow$ i is closer to 12% (F/P = 3.8960)

Equation: The relationship between P & F is:

$$F = 4P = P(1+i)^n \qquad \Rightarrow 4 = (1+i)^{12}$$

$$\log 4 = 12\log(1+i) \qquad \Rightarrow i = 10^{\log 4/12} - 1 \cong 12.2$$

Answer: C $\Leftarrow$

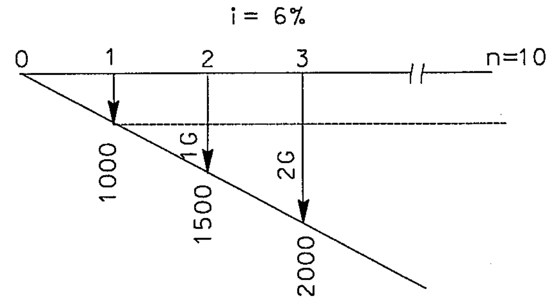
Sample Problem II-3

Given: A construction equipment is expected to have a maintenance cost of \$1000 the first year. It is believed that the maintenance cost will increase \$500 per year. The interest rate is 6% compounded annually.

Find: The effective annual maintenance cost over a 10-year period is most nearly:

(A) \$1900
(B) \$3000
(C) \$3500
(D) \$3800

Solution:



Note that G = 0 @ n = 1 & 1G @ n = 2

$$A = A + G\left(A/G\right)_{n=10}^{i=6\%}$$

$$= \$1000 + 500(4.0220)$$

$$= \$3011$$

Answer: B ⇐

Sample Problem II-4

Given: A scraper costs \$42,000 and can be depreciated over a period of 5 years, after which its salvage value will be \$12,000.

Find: The straight-line depreciation in year 3 is most nearly:

(A) \$6,000
(B) \$12,000
(C) \$18,000
(D) \$24,000

$$D_j = \frac{C - S_n}{n} \qquad (51)$$

$$D_3 = \frac{42{,}000 - 12{,}000}{5} = \$6{,}000$$

The depreciation using straight line method is <u>uniform</u> over the entire life of the asset.

Answer: A ⇐

Sample Problem II-5

Given: A concrete supplier purchased 10 concrete pumping trucks. The total cost is \$2,500,000. It is expected to operate a total of 260,000 hours over a period of 10 years and then have a\$250,000 salvage value. During its first year in service, it is operated for 31,200 hours.

Find: The depreciation in the first year using the MACRS method is most nearly:

(A) \$115,000
(B) \$125,000
(C) \$225,000
(D) \$250,000

Solution:

The depreciation using the MACRS method depends only on the original cost, not the salvage cost or hours of operations.

$$D_j = (factor)\ C \qquad (52)$$

The "factor" in Eq. (52) is obtained from the table shown below the equation. Enter the recovery period as "10" (row) and year "1" (column), the recovery percent is 10:

$$D_1 = (10\%)\ 2{,}500{,}000 = \$250{,}000$$

Answer: D⇐

Sample Problem II-6

Given: A construction company is considering replacing its air conditioning system. The owner has narrowed the choices to two systems A & B. The effective annual interest rate is 8%

	A	B
Initial Cost	\$7000	\$9000
Annual Savings	\$1500	\$1900
Salvage Value	\$500	–\$1250
Life	15 years	15 years

Find: The benefit-cost ratio of the better system is most nearly:

(A) 1.73
(B) 1.76
(C) 1.84
(D) 1.88

Solution:

Note: Salvage value should be counted as a decrease in cost, not as benefit.

System A:

$$\text{Benefit} = B = \$1500 \left(P/A\right)_{15}^{8\%} = \$1500\ (8.5595) = \$12{,}839.25$$

$$\text{Cost} = C = \$7000 - \$500 \left(P/F\right)_{15}^{8\%} = \$7000 - \$500\,(0.3152) = \$6{,}842.40$$

$$\left(\frac{B}{C}\right)_{Sys\ A} = \frac{\$12{,}839.25}{\$6{,}842.40} = 1.88$$

(continued on next page)

System B:

Benefit =B = $1,900 $\left(P/A\right)_{15}^{8\%}$ = $1,900 (8.5595) = $16,263.05

The salvage value of system "B" is "negative" which means that additional expense is required to salvage this system.

Cost = C = $9,000 + $1,250 $\left(P/F\right)_{15}^{8\%}$ = $9000+$1,250(0.3152)= $9,394.00

$$\left(\frac{B}{C}\right)_{Sys\ B} = \frac{\$16,263.05}{\$9,394.00} = 1.73$$

Comparing Alternatives Based on an Incremental Analysis:

The alternatives cannot be compared to one another based on their benefit-cost ratios. Instead, an incremental analysis is required.

$$\frac{B_{Sys\ B} - B_{Sys\ A}}{C_{Sys\ B} - C_{Sys\ A}} = \frac{\$16,263.00 - \$12,839.25}{\$9,394.00 - \$6,842.40} = 1.34$$

Because the incremental analysis is GREATER than ONE, System "B" is better than System "A"

Answer: A ⇐

Sample Problem II-7

Given: The internal rate of return (ROR) of a proposed new construction project is 18%. Five contractors are considering the project. The following are minimum attractive rates of return (MARR) for the five contractors.

Contractor	1	2	3	4	5
MARR, %	12	16	18	19	21

Find: Which contractor will be most likely to accept the project:

(A) Contractor 2
(B) Contractor 1
(C) Contractor 3
(D) Contractor 5

Solution:

Let us have these two definitions:

ROR = The estimated interest rate produced by an investment.

MARR = The lowest rate of return than a contractor (company, organization) will accept.

Based on these two definitions and the given tabulated values of the MARR, the project is good for contractors 1 & 2 because the ROR is higher than the MARR. Contractors 4 & 5 will not accept the project because the ROR is less than the MARR. Contractor 3 will break even if he accepted the project.

The project is the best for Contractor 1

Answer: B ⇐

II-C.1 Value engineering and costing:

A- Introduction:

Value engineering emerged during World War II when shortages of critical resources necessitated changes in methods, materials, and traditional designs; many of these changes resulted in superior performance at a lower cost. After the war, the General Electric Company pioneered in the development and implementation of an organized value analysis program for industry, and this technique was soon adopted by several other companies and government agencies. In 1962, value engineering became a mandatory requirement in the Armed Services Procurement Regulations (ASPR). This change in ASPR introduced value engineering to two of the largest construction agencies in the country, the U.S. Army Corps of Engineers and the U.S. Navy Bureau of Yards and Docks. During the 1960s and the 1970s, several other government agencies and jurisdictions adopted value engineering, including the Bureau of Reclamation, the National Aeronautics and Space Administration (NASA).

During the past decade, the term contractibility analysis has seen increasing use in the private sector and in some universities. Constructibility analysis is similar to value engineering in some or all respects. Some use the term to connote review of plans and specifications from the viewpoint of the constructor, which would include opportunities for prefabrication, preassembly, modularization, special construction methods and other considerations aimed at lessening the cost or improving the completion schedule. Others use the term to include both design and construction analysis in a manner similar to traditional value engineering but omitting the rigid certification and other requirements as recommended by the Society of American Value Engineers. For purposes of this section, value engineering, value analysis and constructibility analysis are used synonymously. Under any of these names, the technique represents a creative and organized approach whose objective is to optimize cost and/or performance of a construction facility or component.

B- Potential Savings:

L. D. Miles book on value analysis and engineering includes the following definition:

"Value analysis/engineering is an organized, creative approach which has for its purpose the effective identification of unnecessary costs, i.e., costs which provide neither quality nor use nor life nor appearance nor customer features."

But to whom do these savings accrue, and what are they really worth?

In 1974 the Army Corps of Engineers estimated that the total cumulative savings through value engineering was almost $234 million. The Public Buildings Service indicated that its value-engineering program had generated savings of $4.53 for every dollar spent, for total savings to GSA of $1.8 million in fiscal 1973. During fiscal year 1970, the Department of Defense estimated a saving of about $4.40 from contractor-sharing incentives for each $1 spent on the program.

Alphonse Dell' Isola's book on value engineering[3] established potential savings guidelines as follows:

On total budget 1 to 3%
On large facilities 5 to 10%
On high-cost areas 15 to 25%

C-Value Engineering Phases:

Realizing the potential savings from the value analysis study, a systematic and innovative approaches are required to achieve such savings. Generally accepted techniques include a job plan for value engineering which will have a number of phases:

i) Develop information and requirements.
ii) Speculate on alternatives.
iii) Analyze and evaluate alternatives.
iv) Develop the program.
v) Proceed with proposal, presentation, and selling.

This plan can be implemented over the life cycle of a construction project, and will have potential savings related to time and cost in varying degrees, depending upon the project development phase.

Conception
Development
Detail design
Construction
Start-up and use

In his book, Dell' Isola designed a value-engineering job plan accomplished in four phases:

Phase I- **Information**: Get facts
Phase II- **Speculative**: Brainstorm
Phase III- **Analytical**: Investigate, evaluate
Phase IV- **Proposal**: Sell

Phase I- **Information**: Get facts :

This phase includes these purposes:

1. To gather and tabulate data concerning the item as presently designed
2. To determine the item's function(s)

3. To evaluate the basic function(s)

During information gathering, certain questions must be answered:

1. What is the item?
2. What does it do?
3. What is the worth of the function?
4. What does it cost?
5. What are the needed requirements?
6. What is the cost/worth ratio?
7. What high-cost or poor-value areas are indicated?

Considerable effort, ingenuity, and investigation are required to answer these questions. The value-engineering group must determine what criteria and constraints existed at the time of the original design and whether they still apply at the present time.

Other important questions may be:

1. How long has this design been used?
2. What alternative systems, materials, or methods were considered during the original concept?
3. What special problems were or are unique to this system?
4. What is the total use or repetitive use of this design each year?

Phase II- **Speculative**: Brainstorm

The purpose of this phase is to generate numerous alternatives for providing the item's basic function(s). By definition, a brainstorming session is a problem-solving conference wherein each participant's thinking is stimulated by others in the group. A team may consist of four to six people of different disciplines, sitting around a table and spontaneously generating ideas. Production of the maximum number of ideas is encouraged, and no idea is criticized.

Phase III- **Analytical**: Investigate & Evaluate

The purposes in this phase are:

1. To evaluate, criticize, and test the alternatives generated during the speculation phase
2. To estimate the dollar value of each alternative
3. To determine the alternatives which offer the greatest potential for cost savings

During this phase, also known as the evaluation and investigation phase, the group examines alternatives generated during the brainstorming and tries to develop lower-cost solutions. The principal tasks are:

1. To evaluate
2. To refine

3. To cost-analyze
4. To form a possible list of alternatives in order of descending savings potential

James J. O'Brien[5] believes a value index such as "worth divided by cost" or "utility divided by cost" can be very beneficial during this phase.

The route of ideas is the following:

1. Eliminate ideas which do not meet environmental and operating conditions.
2. Set aside, for future discussion, ideas with potential but which are beyond present capability or technology.
3. Cost-analyze remaining ideas.
4. List ideas with useful savings, including their potential advantages and disadvantages.
5. Select ideas where advantages outweigh disadvantages and offer the greatest cost savings. (Often dollar values are not readily assignable and must be considered using statistical approaches.)
6. Finally, consider weighted constraints, such as aesthetics, durability, and salability, in order to produce a completed list.

Phase IV- **Proposal**: Sell

This is the last phase of the value engineering study and it must accomplish three things:

1. A thorough review of all alternate solutions must be prepared to assure that the highest value and significant savings are really being offered.
2. A sound proposal must be made to management.
3. The group must present a plan for implementing the proposal. If the proposal will not convince management to act, no savings will result.

D- Life-Cycle Cost Analysis

Accurate cost measurement is one of the most important requirements of a successful value-engineering program. Most cost estimates and cost records used in the construction industry deal with capital costs from the viewpoint of the contractor or the ultimate user of the facilities. Yet the life of the building or facility will extend over 20 to 50 or more years. During this period the cost of maintaining and servicing the facility, including the cost of utilities such as fuel oil, electric power, or natural gas, will equal or exceed the capital cost.

Value analysis from the viewpoint of the owner must therefore take into account both capital and future operation and maintenance costs if maximum value is to be achieved for minimum overall investment. In the final analysis, we are trying to find out how much additional capital expenditure is warranted today to achieve future cost benefits over the life of the facility.

However, as we endeavor to estimate the cost of future events, our cost estimates for the life cycle become less reliable when compared with capital or construction cost estimates.

To illustrate, consider some of the items that are important in analyzing future life-cycle costs for a project:

- Maintenance and operating costs
- Energy and utility costs
- Value of money
- Cost of insurance
- Anticipated future income growth
- Ease and timing of expansions
- Fringe benefits difficult to analyze, including aesthetics, durability, and overall future image
- Effect of the facilities on the productivity of operating, administrative, and maintenance personnel
- Present and future trends in real estate and property taxes, income taxes, and investment credits
- Location and operational costs based upon community growth, competitive patterns, and other factors

Each of the items includes choices among multiple alternatives, uncertain forecasts of future costs, and uncertain effects of future events. To help solve these general problems, a number of specialized tools is available; of these, the following are among the most important:

- Present worth analysis
- Sensitivity analysis
- Break-even analysis
- Discounted cash-flow and rate-of-return analysis

E- Example of a Successful Value-Engineering Study

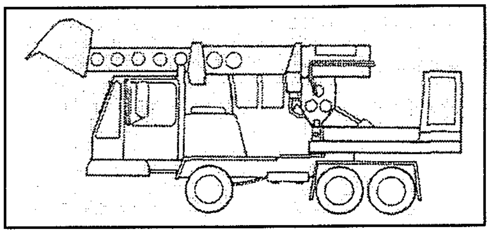
This study involved in a multibillion-dollar prison construction program. In the mid-1980s the Department of Corrections of the State of California embarked upon an unprecedented major prison construction program. The Department acted as overall project director employing a program management consultant and a number of construction management firms to manage individual prisons which were to be built using a phased construction schedule to minimize overall design-construction time. The program manager with the enthusiastic participation and backing of the Department introduced a structured value-engineering program which consisted of value-engineering studies held in parallel with scheduled design reviews for each design package along with designer participation. These meetings were held during the conceptual stage and periodically throughout design development and were chaired by the program manager.

In February 1989, the Auditor General of the State of California published a report which presented the results of a management audit covering the performance of six major prison construction projects which were completed by 1989. The report found, in part that:
"The indicated final cost for the six new prisons is two percent less than the initial budgets prepared before detailed design and construction work began. This represents an outstanding performance for a public sector program pioneering a new conceptual approach designed to shorten design-construction schedules as well as to decrease costs."

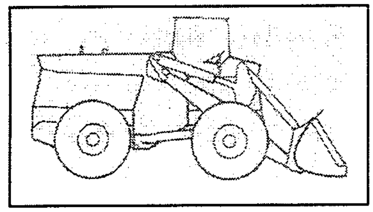
The report attributed much of this success to the value-engineering program developed by the program manager and embraced by the Department of Corrections which helped to effectively manage the ultimate construction costs throughout the design development and detail-design phases of each new prison. The program manager utilized a traveling team under the leadership of the project manager who visited each contract package designer to conduct a design review and value-engineering study at fixed points throughout the design period. In addition to the project manager, the team consisted of one each civil, mechanical and electrical engineer with estimating support. The team also included a correctional officer representing the Department who had the responsibility for the preservation or improvement of Department correctional and security requirements.

Chapter III: Construction Operations and Methods

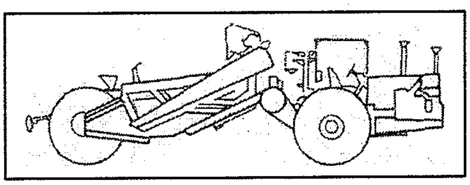
Excavation Equipment



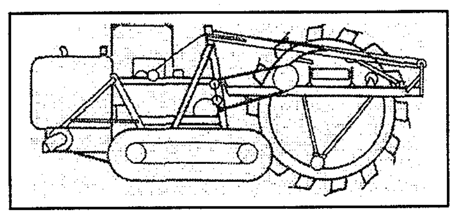
Hydraulic Excavator



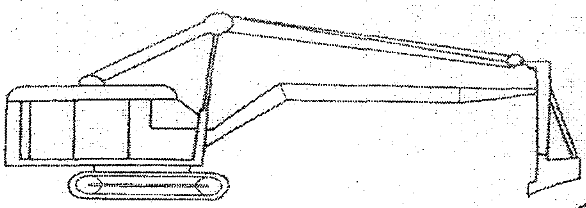
Front-End Loader (Wheel- Mounted)



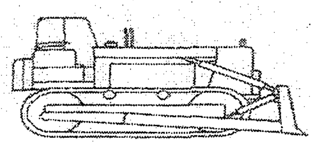
Scraper



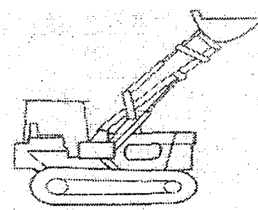
Trencher



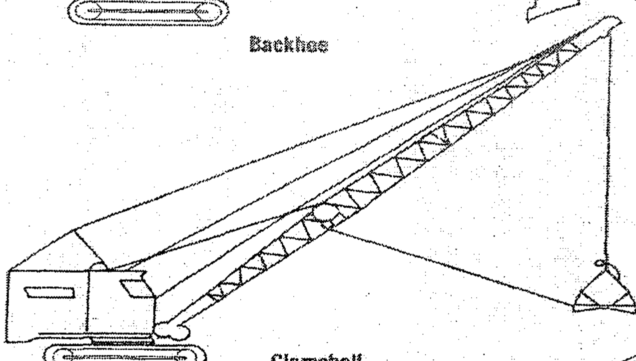
Backhoe



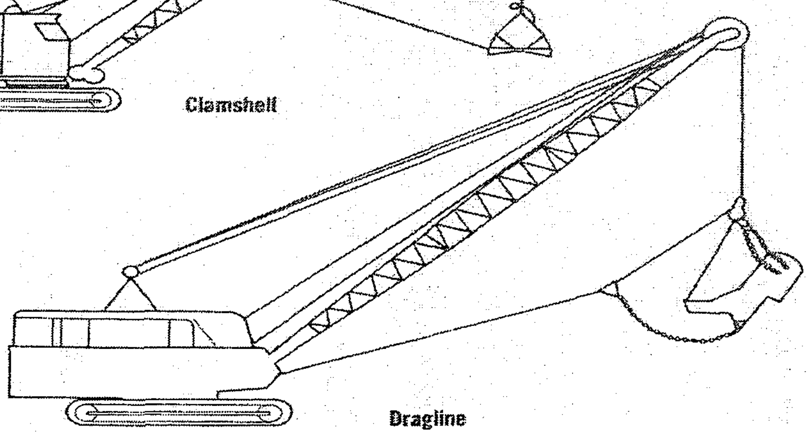
Bulldozer



Front-End Loader
(Crawler Mounted)



Clamshell



Dragline

Figure III-1 Typical Excavation Equipment

III-A LIFTING AND RIGGING

A crane is designed to pick (or lift) a load by means of a hoisting mechanism using ropes. The load must be properly attached to the crane by a rigging system. To properly attach the load, it is necessary to determine the forces that will affect the job, and then to select and arrange the equipment that will move the load safely. The forces involved in rigging will vary with the method of connection and the effects of motion. The rigger must, by proper application of mechanical laws and by resolving load-movement-induced stresses, correctly determine the weight and center of gravity of the load.

III-A.1 Weight:

The most important step in any rigging operation is to correctly determine the weight of the load. If this information cannot be obtained from the shipping papers, design plans, catalog data, or other dependable sources, it may be necessary to calculate the weight. It is good practice to verify the load weight as stated in the documents. Weights and properties of structural members can be obtained from:

i. American Concrete Institute
ii. Manual of Steel Construction, American Institute of Steel Construction
iii. Cold Formed Steel Design Manual, American Iron and Steel Institute
iv. Aluminum Design Manual, American Aluminum Association

III-A.2 Center of Gravity:

The center of gravity of an object is that location where the object will balance when lifted. When the object is suspended freely from a hook, this point will always be directly below the hook. Thus, a load that is slung above and through its center of gravity will be in equilibrium. It will not tend to slide out of the hitch or become unstable.

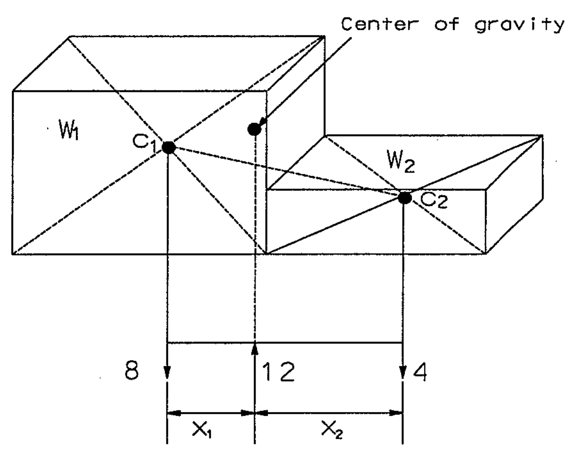


Figure III-2 Center of Gravity for Loads

One way to determine the center of gravity of an odd-shaped object is to divide the shape into simple masses and determine the resultant balancing load and its location at a point where the weights multiplied by theft respective lever arms are in equilibrium. Thus,

$$W_1 \times x_1 = W_2 \times x_2 \tag{57}$$

where W_1 and W_2 are the weights of the larger part and the smaller part, respectively, and x_1 and x_2 are the lever arms of the larger part and the smaller part, respectively.

III-A.3 Forces in Slings:

To calculate the stress developed by the load on a rigging arrangement, it must be remembered that all forces must be in equilibrium. If a 5- ton load is supported by a set of slings in such a manner that the individual sling legs make a 10° angle with the load, the sling has a load of 14.4 tons by applying the equilibrium equations in the Y- direction as follows:

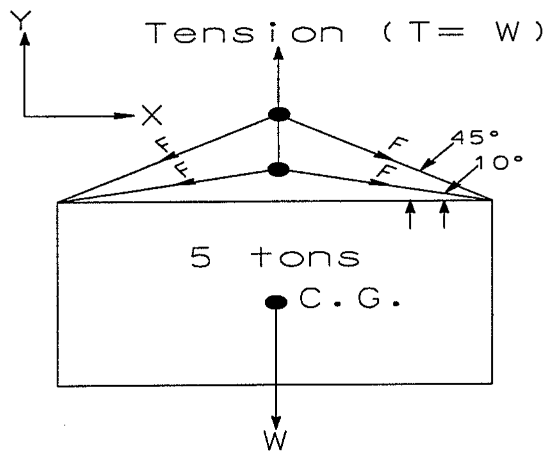


Figure III-3 Forces in Slings and Slope Angles

$$F = \frac{5}{2 \times Sin\,10^{\circ}} = 14.40\ tons$$

Changing the sling angle to 45° will reduce the load in the sling to 3.54 tons:

$$F = \frac{5}{2 \times Sin\,45^{\circ}} = 3.54\ tons$$

Therefore, changing the sling angle have a direct effect on the magnitude of the forces in the sling. Consequently, this will affect the sling size (diameter or the cross-sectional area) and the stress that will be developed in the sling. The following Figure shows the change of the forces in the slings as a result of changing the sling angles.

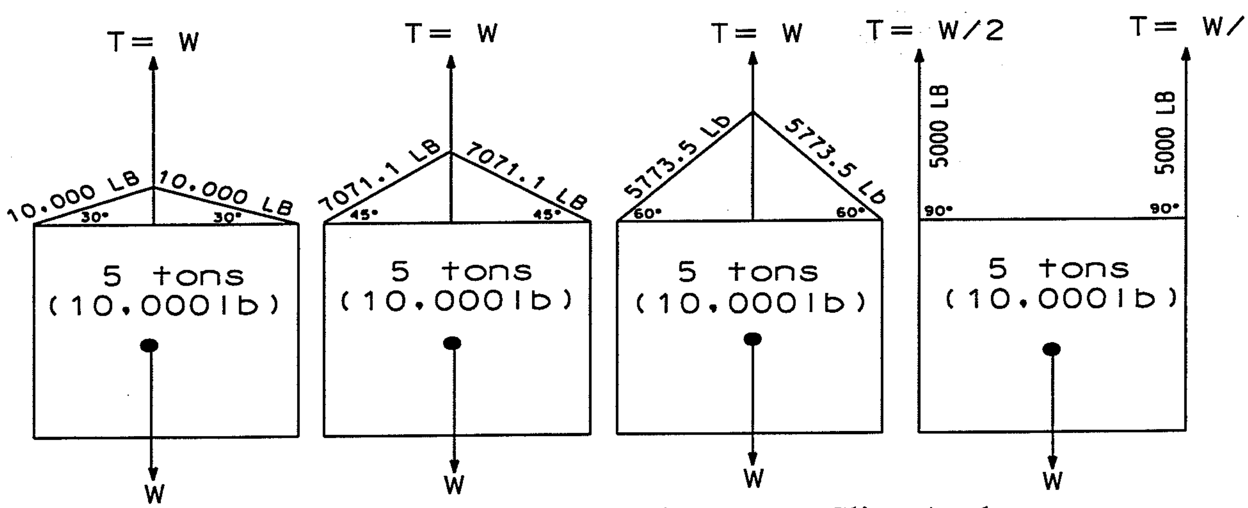


Figure III-4 Load Variation Versus Sling Angle

The general formula is given by the following equation:

$$F = \frac{T}{2\ Sin\ \theta} = \frac{W}{2\ Sin\ \theta} \tag{58}$$

Where:

W = weight of the load
T = tension in the cable
F = force in the sling
θ = horizontal angle of the sling

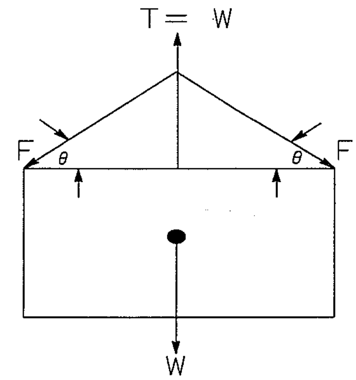
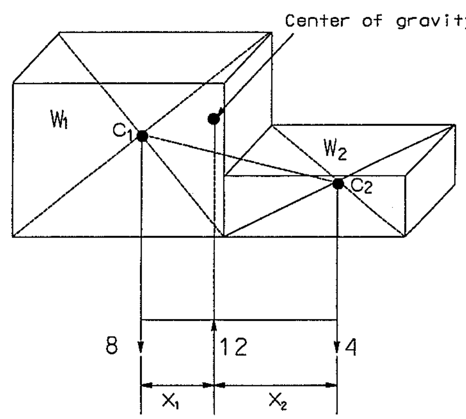


Figure III-5 Load Versus Sling Angle

Sample Problem III-1: Center of Gravity Calculations

Given: Rigging operation is required to lift a precast concrete block shown. The weight of the larger portion (W_1) is 16,000 lbs (8 tons) and the smaller portion (W_2) is 8,000 lbs (4 tons). The dimensions (width, height, depth) are 8 × 6 × 4 ft and 6 × 2× 4 ft for the larger and smaller portions respectively.



Find: The center of gravity of this load measured from center of gravity of the big load is most nearly:

(A) 2.33 ft
(B) 2.67 ft
(C) 3.67 ft
(D) 4.48 ft

Solution:

Using equation 3.1 (Taking moment about the center of gravity of the entire body)

$$W_1 \times x_1 = W_2 \times x_2$$

$$16 \times x_1 = 8 \times x_2$$

But: $x_1 + x_2 = (4+3) = 7\,ft$

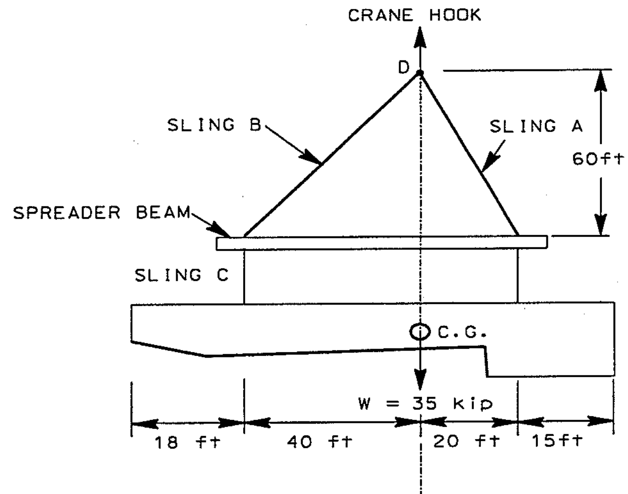
Solving the above two equations yields $x_1 = 2.33\,ft$

Answer: A ⇐

Note: The center of gravity is closer to the concentration of mass (or area). Therefore, it is closer to W_1. This problems could be solved by inspection since $W_1 / W_2 = 2.0$ and hence the center of gravity is 1/3 and 2/3 (of 7 ft) from W_1 and W_2 respectively.

Sample Problem III-2: Tension in Cables (Slings) of Rigging System

Given: The rigging shown will be used to lift a 35 kip precast concrete element. The center of gravity (C.G.) was determined for this irregular shape and is shown on the diagram.

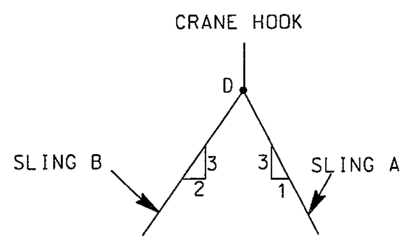


Find: The force in the sling "B" is most nearly:

(A) 33.03 kips
(B) 24.60 kips
(C) 19.03 kips
(D) 14.02 kips

Solution:

Consider joint D as free body diagram and using 2-D equilibrium equations will yield the following:



$$\sum F_x = 0$$

$$\frac{F_B}{\sqrt{13}}(2) = \frac{F_A}{\sqrt{10}}(1) \qquad Eq.(1)$$

$$\sum F_y = 0$$

$$35.0\,kips = \frac{F_B}{\sqrt{13}}(3) + \frac{F_A}{\sqrt{10}}(3) \qquad Eq.(2)$$

Solving the above two equations for the two unknowns:

$F_B = 14.02\ kips$

$F_A = 24.60\ kips$

Answer: D⇐

Note: Slope of the sling A is steeper than the slope of sling B. Therefore, the tension in sling A is greater than the tension in sling B.

III-B CRANE SELECTION, ERECTION, AND STABILITY

Construction cranes are classified into two major categories:

1- Mobile cranes
2- Tower cranes

Mobile cranes are the most commonly type used in the US because the contractors favored them over tower cranes due to mobility of these cranes. Tower cranes are usually used only when *job-site conditions make mobile crane movement impossible*, or for high-rise construction.

The rated load for a crane, as published by the manufacturer, is based on ideal conditions, a level machine, calm air and no dynamic effects. It is critical that the load chart being consulted before the actual crane configuration is used.

The following table shows different types of cranes used in the US

Table III-1 Mobile versus Tower Cranes

Mobile Cranes	**Tower Cranes**
1- Crawler 2- Telescoping-boom-truck-mounted 3- Lattice-boom-truck mounted 4- Rough-terrain 5- All terrain 6- Modified cranes for heavy lifting	1- Top - slewing 2- Bottom - slewing

Tower cranes provide high lifting height and good working radius, while taking a very limited area. These advantages are achieved at the expense of lower lifting capacity and limited mobility, as compared to mobile cranes.

Safety of cranes at the job site is very important. **Crane safety** program should addressed as required depending on the **site conditions** and **loads to be lifted**. The following are the items that should be included in a crane safety plan:

1- Equipment inspection
2- Hazard analysis- concern for the public, power lines, …etc.
3- Crane location
4- Crane movements
5- Determination of responsibility zones and line of control reporting
6- Post-accident reporting and investigation procedures,

III-C DEWATERING AND PUMPING

In excavating below the surface of the ground, contractors may encounter groundwater before reaching the required depth of excavation. In the case of an excavation into sand and gravel, the flow of water will be large if some method is not adopted to intercept and remove the water. In order to lower the level of groundwater at the construction site, dewatering is required.

Dewatering: temporarily lowering the level of groundwater at the construction site. After construction is completed, the dewatering action can be discontinued and the groundwater will return to its normal level. It should be noted that the groundwater levels change from season to season.

Methods of Dewatering:

There are three methods commonly used for dewatering of groundwater at construction sites. These methods are:

1- Collector ditches
2- Wellpoint system
3- Deep wells

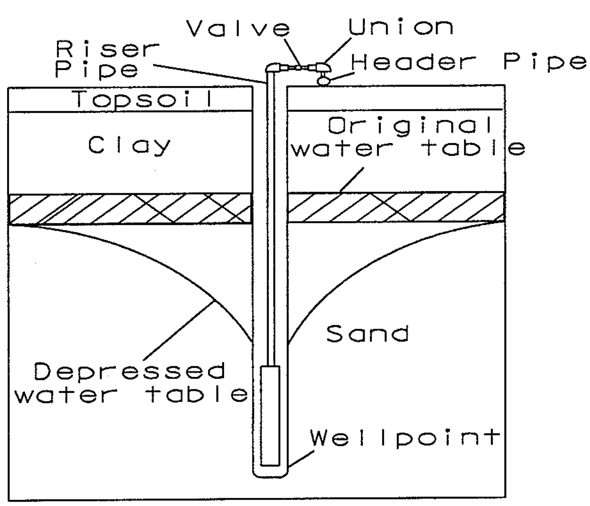


Figure III-5 Lowering Water Table by Dewatering (Wellpoints)

Table III-2 Dewatering Methods

Ditches	Wellpoint System	Deep Wells
➢ Collector ditches within the limits of the excavation are digged out to collect and divert the flow of groundwater into pumps where it can be removed by pumping ➢ The disadvantages of collector ditches method is that the presence of ditches may interferes with the construction operations	➢ Perforated tubes in a screen that are installed below the surface of the ground to collect water in order to lower the piezometeric level of groundwater. ➢ A wellpoint system consists of: a- Small vertical riser pipes extend a short distance above the ground surface connected to a larger pipe. Normally spaced at 2 to 5 feet. b- A larger pipe (called "header") usually between 6 to 10 inches in diameter where its connected to a centrifugal pump where the groundwater is pumped outside the construction site ➢ This method is suitable for a permeable soil as sand or gravel. If the soil is less permeable as silt or clay, a permeable well will be needed.	➢ Deep wells are large diameters wells installed outside the construction site. ➢ They are suitable when soil becomes more pervious with depth. ➢ The advantageous of deep wells is that can be installed outside the construction zone and therefore does not interfere with construction equipment and operations

III-D EQUIPMENT PRODUCTION

There are three definitions related to the earthwork condition as follows:

Bank cubic yard (BCY): material in natural state (in place yards)
Loose cubic yard (LCY): material after excavation (loose yards)
Compacted cubic yard (CCY): material in compacted state (also known as net in-place cubic yard)

Equipment productivity shall be calculated based on the loose cubic yard (LCY) as follows:

$$\text{Production (LCY) per hour} = \text{Bucket payload (LCY) per cycle} \times \text{Cycles per hour} \quad (59)$$

The payload and cycles per hour depend on:

1- type of equipment, and
2- manufacturer

Most manufacturers have developed performance/ productivity curves for their equipment.

III-E PRODUCTIVITY ANALYSIS AND IMPROVEMENT

III-E.1 Loaders:

Loaders are used extensively in construction work to transport and handle bulk material such as rock and earth. A loader is a versatile piece of equipment designed to excavate at or above wheel level. There are two types of loaders as follows:

1- Crawler-tractor-mounted type
2- Wheel-tractor-mounted type

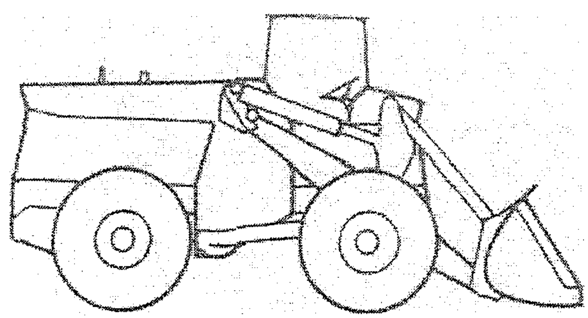


Figure III-6 Front-End Loader (Wheel- Mounted)

A- Loader Production Rates:

In general, there are two factors affecting the production rates for loaders:

2- the type of the material, and
3- the volume of the materials to be handled

The production rate for wheel loader will depend on the following:

1- Fixed cycle time required to load the bucket, maneuver with four reversals of direction, and dump the load,
2- Time required to travel from the loading to the dumping position,
3- Time required to return to the loading position, and
4- Volume of material hauled each cycle including bucket fill factor

$$\text{Production Cycle Time} = \text{load}_t + \text{travel}_t + \text{dump}_t + \text{return}_t \qquad (60)$$

The following table shows typical loader cycle times for tracked and wheeled loaders:

Table III-3 Approximate Loader Cycle Times for Different Materials

Loading Conditions	**Basic Cycle Time (minutes)**	
	Tracked Loader	**Wheeled Loader**
Loose material	0.30	0.35
Average material	0.35	0.50
Hard material	0.45	0.65

The following Table gives the fixed cycle times for both wheeled and tracked loaders for different bucket capacity.

Table III-4 Fixed Cycle Times for Loaders

Loader Size, heaped bucket capacity (CY)	**Wheeled loader Cycle Time* (Sec)**	**Tracked Loader Cycle Time* (Sec)**
1.00 – 3.75	27 – 30	15 – 21
4.00 – 5.50	30 – 33	–
6.00 – 7.00	33 – 36	–
14.00 – 23.00	36 – 42	–

*Loader cycle = load + maneuver with four reversals of direction (minimum travel) + dump

When travel distance is more than **minimum**, it will be necessary to **add travel time** to the **fixed cycle time** indicated in the above Table.

The following figure shows typical travel times for wheeled loader. The manufacturers' performance curve should be used when a specific loader is selected.

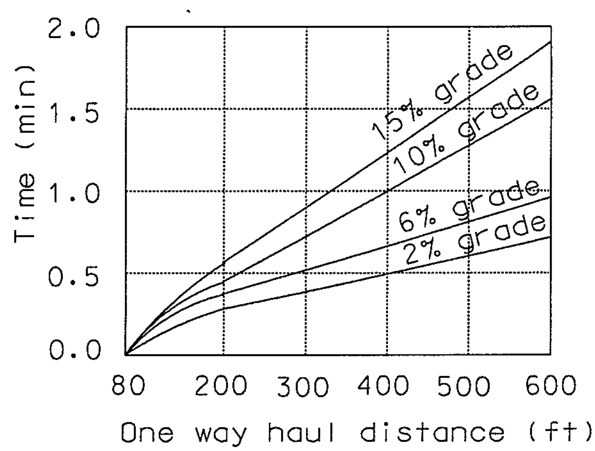


Figure III-7 Typical Loader Travel Time

B- Fill Factors for Loader Buckets:

The fill factor for a loader bucket adjusts heaped capacity which is based on:

1- type of material being handled, and
2- type of loader.

The following Table shows typical fill factors for wheeled and tracked loaders

Table III-5 Bucket Fill Factor for Loaders

Material		Wheeled Loader Fill Factor %	Tracked Loader Fill Factor %
Loose material	Mixed moist aggregates	95 – 100	95 – 100
	Uniform aggregates		
	Up to 1/8 in.	95 – 100	95 – 110
	1/8 - 3/8 in.	90 – 95	90 – 110
	½ - ¾ in.	85 – 90	90 – 110
	1 in. and over	85 – 90	90 – 110
Blasted rock	Well blasted	80 – 95	80 – 95
	Average	75 – 90	75 – 90
	Poor	60 – 75	60 – 75
Other	Rock dirt mixture	100 – 120	100 – 120
	Moist loam	100 – 110	100 – 120
	Soil	80 – 100	80 – 100
	Cemented materials	85 – 95	85 – 100

Source: Caterpillar Inc. (with permission)

Sample Problem III-3: Loader Productivity

Given: A wheeled loader with a 4 cubic yard bucket is used to excavate a site consists of loose material. The loader is moving the excavated material 400 feet away from the site. The road has + 5% effective grade. Assume 50-min hour efficiency factor and 100% fill factor.

Find: The production of the loader in LCY per hour is most nearly:

(A) 161 LCY/hr
(B) 171 LCY/hr
(C) 181 LCY/hr
(D) 191 LCY/hr

Solution:

From Table III-3 and for a bucket volume of 4 cubic yards
→ basic cycle time = 30 second = 0.50 minutes

From Figure III-7, travel time for a distance of 400 feet @ + 5% grade
→ travel time = 0.60 minutes (interpolation)

Total cycle time = 0.50 + 0.60 = 1.10 minutes

$$\text{Production} = 4.0\, LCY/cycle \times \frac{50 \min/hr}{1.10 \min/cycle} = 181.82\ LCY/hr$$

Answer: C ⇐

Note: A common mistake is that the units are not balancing (checking) out. Always place the units for each parameter into the equation to ensure the final units.

III-E.2 Dozers:

A dozer is a tractor unit has a blade attached to the machine's front. Dozers are classified on the basis of running gear as:

1- Crawler type (tracklaying machines)
2- Wheel type

A dozer can perform different construction tasks as:

1- Stripping: removal of a thin layer of covering materials. Normally, dozers are economical machines for moving material a maximum of 300 feet.
2- Backfilling: moving material to backfill pipes and culverts
3- Spreading: spreading material dumped by trucks or scrapers

The following table shows the comparison between the two types:

Table III- 6 Comparison Between Wheel and Crawler Dozers

Wheel Dozer	Crawler Dozer
1- Good on firm soils and concrete 2- Best for level and downhill work 3- Wet weather, causing soft and slick surface conditions, will slow or stop operation 4- Good for long travel distances 5- Can handle only moderate blade loads	1- Can work on a variety of soils 2- Can work over almost any terrain 3- Can work on soft ground and over mud-slick surfaces 4- Good for short distances 5- Can push large blade loads

A- Dozer Production Estimating:

The dozer production is controlled by the following factors:

1- Blade type,
2- Type and condition of materials, and
3- Cycle time: time required to push a load, back-track and maneuver into position to push again

The following formula calculates a dozer pushing production in loose cubic yards (LCY) per a 60 – minute

$$\text{Production (LCY/hr)} = \frac{60\,\text{min} \times \textit{blade load}}{\textit{push time}\ (\text{min}) + \textit{return time}\ (\text{min}) + \textit{maneuver time}\ (\text{min})} \qquad (61)$$

The production of a dozer could also be calculated using the ***rule-of-thumb formula*** proposed by equipment manufacturer International Harvester (IH)

$$\text{Production (LCY per 60-min hr)} = \frac{\textit{net hp} \times 330}{D+50} \qquad (62)$$

Where:

net hp = net horsepower at the flywheel for a power-shift crawler dozer
D = one-way push distance, in feet

Sample Problem III-4: Dozer Production Rate (LCY/hr)

Given: A track-type dozer has a blade load of 6.25 LCY is used to push a silty sand soil to back fill a new irrigation pipeline. The average push distance is 110 feet. The push, return and maneuver time are calculated to be 0.53, 0.28 and 0.05 minutes respectively.

Find: The production of the dozer in LCY per hour is most nearly:

(A) 461 LCY/hr
(B) 436 LCY/hr
(C) 416 LCY/hr
(D) 396 LCY/hr

Solution:

$$\text{Production (LCY/hr)} = \frac{60\,\text{min} \times blade\ \ load}{push\ \ time\ (\text{min}) + return\ \ time\ (\text{min}) + maneuver\ \ time\ (\text{min})}$$

$$= \frac{60\,\text{min} \times 6.25\,lcy}{0.53\,(\text{min}) + 0.28\,(\text{min}) + 0.05\,(\text{min})} = 436\ LCY/hr$$

Answer: B ⇐

Sample Problem III-5: Dozer Production Rate (BCY/hr)

Given: A tracked-type dozer has a production of 496 loose cubic yards (LCY/ hr). The percent swell for the soil was found to be 25%. Assume a job efficiency of 50-min hour.

Find: The actual production of the dozer in bank cubic yards per hour (BCY/hr) is most nearly:

(A) 237 BCY/hr
(B) 300 BCY/hr
(C) 331 BCY/hr
(D) 397 BCY/hr

Solution:

<u>Bank cubic yard (BCY)</u>: material in natural state (in place yards)

$$\text{Production (BCY/hr)} = \frac{496\,lcy/hr}{1.25} \times \frac{50\,\text{min}}{60\,\text{min}} = 330.67\ bcy/hr$$

Answer: C ⇐

Sample Problem III-6: Dozer Production Rate (Rule-of-thumb formula)

Given: A 200-hp tracked-type dozer is used for a spreading operation at a new highway facility. The push distance is 120 feet. Assume a job efficiency of 55-min hour.

Find: The production of the dozer in loose cubic yards per hour (LCY/hr) using the rule-of-thumb formula is most nearly:

(A) 325 lcy/hr
(B) 335 lcy/hr
(C) 356 lcy/hr
(D) 388 lcy/hr

Solution:

Production (LCY per hr) =

$$\frac{net\,hp \times 330}{D+50} = \frac{200\times330}{120+50}\times\frac{55\,\text{min}}{60\text{ min}} = 355.88\ lcy/hr$$

Answer: C⇐

III-E.3 <u>Excavators</u>:

Hydraulic excavators may be either crawler or pneumatic-tire-carrier–mounted. The basic production formula is:

Production = material carried per load × cycles per hour

The above formula will be used to estimate the production of an excavator using the following steps:

1- Obtain the heaped bucket load volume from the manufacturers' data sheet.
2- Apply a bucket fill factor based on the type of machine and the class of material being excavated. *Fill factor is a numerical value to adjust rated heaped excavator bucket capacity based on the type of material.* The following table shows typical values for backhoe bucket fill factor:

Table III-7 Backhoe Fill Factor

Material	**Fill Factor (% of heaped bucket capacity)**
Sand and gravel	95-110
Hard, tough clay	80-90
Moist loam /sandy clay	100- 110
Well blasted rock	60-75
Poorly blasted rock	40-50

3- Estimate a peak cycle time. This is a function of machine type and job conditions to include angle of swing, depth or height of cut.
4- Apply an efficiency factor
5- Conform the production units to the desired volume or weight units (LCY, tons)
6- Calculate the production rate using the following formula:

$$\text{Production} = \frac{3{,}600 \text{ sec} \times Q \times F \times (AS:D)}{t} \times \frac{E}{60-\min hr} \times \frac{1}{volume\ \ correction} \quad (63)$$

Where:

Q = heaped bucket capacity, lcy
F = bucket fill factor
$AS{:}D$ = angle of swing and depth (height) of cut
t = cycle time in seconds
E = efficiency (minutes worked per hour)

$$\text{Volume correction (convert LCY to BCY)} = \frac{1}{1+swell\ \ factor}$$

For backhoe, the production equation is given as:

Production (backhoe - excavation) =

$$= \frac{3{,}600 \text{ sec} \times Q \times F}{t} \times \frac{E}{60-\min hr} \times \frac{1}{volume\ \ correction} \quad (64)$$

Where:

Q = heaped bucket capacity, lcy
F = bucket fill factor
t = cycle time in seconds
E = efficiency (minutes worked per hour)

$$\text{Volume correction (convert LCY to BCY)} = \frac{1}{1+swell\ \ factor}$$

The following table gives cycle times for hydraulic track hoes based on bucket and average conditions:

Table III-8 Cycle Time for Hydraulic Hoes

Bucket Size (cy)	Load Bucket (sec)	Swing Loaded (sec)	Dump Bucket (sec)	Swing Empty (sec)	Total Cycle (sec)
<1	5	4	2	3	14
1–1 ½	6	4	2	3	15
2 – 2 ½	6	4	3	4	17
3	7	5	4	4	20
3 ½	7	6	4	5	22
4	7	6	4	5	22
5	7	7	4	6	24

Sample Problem III-7: Backhoe Production

Given: A backhoe having a 3 cubic yard bucket is used to excavate hard clay. The swell for this hard clay is 35%. Assume a job efficiency of 50-min hour.

Find: The production of the backhoe is most nearly:

(A) 283.33 bcy/hr
(B) 293.33 bcy/hr
(C) 301.33 bcy/hr
(D) 310.00 bcy/hr

Solution:
Production (backhoe - excavation) =

$$= \frac{3{,}600 \text{ sec} \times Q \times F}{t} \times \frac{E}{60-\min hr} \times \frac{1}{volume\ correction}$$

$$= \frac{3{,}600 \text{ sec} \times 3cy \times 0.85}{20 \text{ sec}/cycle} \times \frac{50 \min}{60-\min hr} \times \frac{1}{1+0.35} = 283.33\, bcy/hr$$

Answer: A ⇐

III-E. 4 Hauling Equipment (Scrapers and Trucks):

The production of hauling equipment as scrapers and trucks is calculated based on the *production cycle* of each equipment.

A- Scrapers:

The production cycle for a scraper consists of the following six operations:

1- loading
2- hauling travel
3- dumping and spreading
4- turning
5- return travel
6- turning and positioning to pick another load

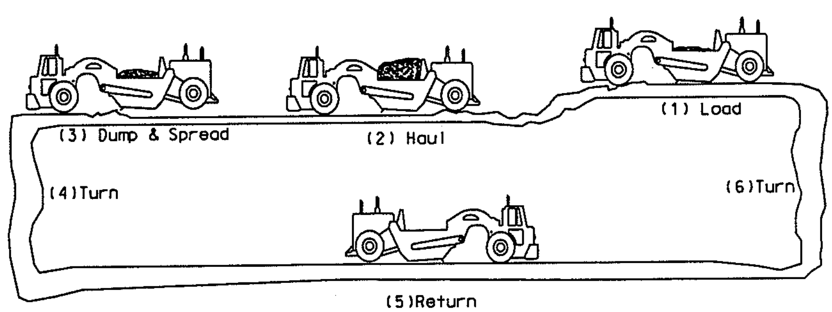


Figure III-7 Production Cycle for a Scraper

Production Cycle Time = $load_t + haul_t + dump_t + turn_t + return_t + turn_t$ (65)

It should be noted that scarpers are independent loading equipment not like the trucks which are loaded by another equipment as loaders.

B- Trucks:

The following equation calculates the required number of trucks for hauling operation:

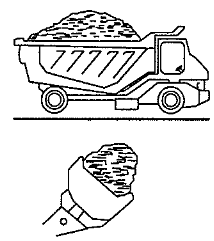
$$Number\ of\ Trucks = \frac{Truck\ Cycle\ Time}{Load\ Time} \qquad (66)$$

Where:

Load Time = Truck Capacity/ Loader Production at 100% Efficiency, or
Load Time = Number of Bucket loads × Loader Cycle Time

Another important factor when calculating the truck production is to match the excavator or loaders with trucks to have compatible capacities to yield the maximum loading efficiency. For example the number of excavator or loader bucket loads to load a truck is given as:

$$\text{Number of bucket loads} = \frac{Truck\ \ Capacity\,(LCY)}{Bucket\ Capacity\,(LCY)} = \qquad (67)$$



Sample Problem III-8: Tuck Production Rate
Given: A hauling truck has a loading capacity of 18 loose cubic yards (LCY) and a cycle time of 0.60 hr. The truck is loaded by a loader which has a capacity of 340 loose cubic yards (LCY) per hour
Find: The number of trucks needed to keep up with the loader production is most nearly: (A) 10 trucks (B) 11 trucks (C) 12 trucks (D) 13 trucks
Solution: Time required to load one truck = $\frac{18\ LCY}{340\,LCY/hr} = 0.053\,hr/truck$ Total truck cycle time = 0.60 hr + 0.053 hr = 0.653 hr Number of trucks (@ 100 % efficiency) = $\frac{0.653\,hr}{0.053\,hr/truck} = 12.3\,trucks \Rightarrow say\,13\,trucks$ Answer: D ⇐

III-E. 5 Finishing Equipment (Graders):

Finishing, finish grading, and fine grading are all terms used to refer to the process of shaping materials to the required line and grade specified in the contract documents. Finishing operations follow closely behind excavation (rough grading) operations or compaction of embankments.

Graders are multipurpose machines used for finishing, shaping, bank sloping, and ditching. They are also used for mixing, spreading, leveling and crowing.

A- Spreading Productivity of Graders: Graders are often used to spread and mix dumped loads. The materials to be mixed and spread by a grader should be free-flowing materials (loose & dry materials as sand). The following equation is used to calculate the spreading and mixing production for a grader:

$$\text{Production (BCY) per hr} = 3.0 \times hp \times efficiency \qquad (68)$$

Where:

hp = flywheel horsepower

$efficiency$ = 1.0 for 50-min working hour

Sample Problem III-9: Grader Spreading Production Rate
Given: A grader is used for mixing and spreading sand dumped from haul trucks. The flywheel horse power of the grader is rated at 240 hp. Assume 50-min working hour.
Find: The spreading and mixing production for the grader in bank cubic yards per hour is most nearly: (A) 720 BCY/hr (B) 620 BCY/hr (C) 520 BCY/hr (D) 510 BCY/hr
Solution: Using the above equation: $\text{Production (BCY) per hr} = 3.0 \times hp \times efficiency = 3.0 \times 240 \times 1.0 = 720\ \text{BCY/hr}$ Answer: A $\Leftarrow$

I- Productivity Time for Graders: The total time required to complete a grader operation is given by the following formula:

$$Total\ Time = \frac{P \times D}{S \times E} \quad (69)$$

Where:

P = number of passes required
D = distance traveled in each pass, miles or feet
S = speed of grader, in mph or feet per minute (fpm)
E = grader efficiency factor

Sample Problem III-10: Grader Productivity Time

Given: It is required to spread, level and reshape sand materials dumped on a new street alignment. The length of the street is 6 miles. The work required for spreading and leveling as follows:

a- two passes for spreading at 2.4 mph
b- two passes for leveling at 3.8 mph
c- three passes for reshaping at 10.0 mph

The flywheel horse power of the grader is rated at 240 hp. Assume 40-min working per hour.

Find: The total time in hours for spreading, leveling and reshaping the street is most nearly:

A) 7.50 *hr*
B) 12.23 *hr*
C) 14.93 *hr*
D) 16.93 *hr*

Solution:

Using the above equation :

$$Total\ Time = \frac{P \times D}{S \times E} =$$

$$\frac{2\,passes \times 6\,miles}{2.4\,mph \times (40\,\text{min}/60\,\text{min})} + \frac{2\,passes \times 6\ \ miles}{3.8\,mph \times (40\,\text{min}/60\,\text{min})} + \frac{3\,passes \times 6\,miles}{10\,mph \times (40\,\text{min}/60\,\text{min})}$$

$$= 7.50\ hr + 4.73\ hr + 2.70\ hr = 14.93\ hr$$

Answer: C ⇐

II- Fine Grading Productivity for Graders: Graders are used for finishing and fine grading to the surface layer on construction projects. The surface area in square foot or yard will be used to calculate the productivity as follows:

$$\text{Production (square yard per hr)} = \frac{5{,}280 \times S \times W \times E}{N \times 9\, sf/sy} \tag{70}$$

Where:

S = speed of grader, in mph or feet per minute (fpm)
W = effective width per grader pass, in feet
E = grader efficiency factor
N = number of passes over the same area

Sample Problem III-11: Grader Productivity Time

Given: A grader is used for fine grading of the subgrade of a new highway. The effective width of the blade per pass is 9 feet. Two passes are required to produce the finished surface. The length of the project is 6 miles and the width of the proposed highway is 34 feet.
The flywheel horse power of the grader is rated at 232 hp and operates at 4.0 mph. Assume 36-min working per hour.

Find: The productivity of the grader in square yard per hour (*sy/hr*) is most nearly:

A) 5340 *sy/hr*
B) 6336 *sy/hr*
C) 7340 *sy/hr*
D) 7636 *sy/hr*

Solution:

Production (square yard per hr) =

$$\frac{5{,}280 \times S \times W \times E}{N \times 9\, sf/sy} = \frac{5{,}280\, ft/mile \times 4\, mph \times 9\, ft \times (36\min/60\min)}{2\, passes \times 9\, sf/sy} = 6{,}336.0\, sy/hr$$

Answer: B ⇐

III-F Temporary Erosion Control

Erosion control is ***mandatory*** on all construction projects. The National Pollution Discharge Elimination System (NPDES) program requires projects disturbing areas larger than ***one acre*** in size to apply for a storm water discharge permit. This requirement will force contractors on smaller projects to address and implement temporary erosion control measures in order to be in compliance. Designers need to be aware and include Best Management Practices (BMPs) in the design phase of the project. All contractors and inspectors need to prepare, implement, and maintain a Storm Water Pollution Prevention (SWPP) Plan that will address the discharge of pollutants (such as sediment) to surface water bodies. Proper use of BMPs will protect *the environment and save the user time and money lost to erosion damage.*

In 1972, The Federal Water Pollution Control Act, also referred to as the Clean Water Act (CWA) was amended to provide that the discharge of pollutants to waters of the United States from any point source is unlawful, unless the discharge is in compliance with an National Pollutant Discharge Elimination System. Amendments to the CWA added Section 402 in 1987. Section 402 established the NPDES program to regulate municipal separate storm sewer systems (referred to MS4s and are systems owned or operated by State, City, town, county …), construction sites, and industrial discharges of pollutants from point sources. In 1990, the EPA published further regulations under the NPDES program that defined the term "storm water discharge associated with industrial activity" to include storm water discharges from construction activities that disturb five or more acres. Regulations (Phase II Rule) that became final in 1999, lowered the permitting threshold from five (5) acres to one (1) acres. ***In California, the EPA delegated its authority to the State Water Resources Control Board (SWRCB) to issue NPDES permits.***

Due to the burden associated with permitting individual construction sites throughout California, the SWRCB elected to adopt a single statewide general permit for construction activities (General Permit) (Order No. 99-08-DWQ) (CAS000002) that applies to all storm water discharges from land where clearing, grading, and excavation result in soil disturbances of at least one (1) acre or more. Construction activities that result in soil disturbances of less than one (1) acre is subject to this Construction General Permit if the construction activity is part of a larger Common Plan of Development totaling one (1) acre or more of soil disturbing activities, or if there is the potential for significant water quality impairment resulting from the activity as determined by the Regional Water Quality Control Board.

To obtain coverage under the Construction General Permit, dischargers must file a Notice of Intent (NOI) and requires owners of land where construction activities occur to develop and implement a Storm Water Pollution Prevention Plan (SWPPP). The SWPPP should include the following:

- site maps which shows the construction site perimeter
- existing and proposed buildings, lots, roadways, stormwater collection and discharge points,
- general topography before and after construction, and drainage patterns
- list of BMPs the discharger will use to protect storm water runoff and the placement of the BMPs
- a sediment monitoring plan if the site discharges directly to water body listed on the 303 (d) list for sediment

The authorization to discharge stormwater requires stormwater that contains sediment to remain on site. This means that the runoff must be detained onsite to allow sediment to settle or be filtered out.

The core of the storm water permit process is the SWPP Plan, is a listing of all planned erosion and sediment control practices on site. The SWPP Plan also addresses inspection and maintenance procedures. The SWPP Plan must be kept on the project site along with the record of inspection forms.

Upon completion of the project and stabilization of the disturbed areas, the permittee files a Notice of Termination (NOT). The NOT signifies that the site is stabilized when vegetation has been established on 70 percent of the disturbed area and coverage under the NPDES permit is no longer required.

What is erosion? Erosion is the process in which, by the actions of wind or water, soil particles are detached and transported. Erosion problems can be accelerated when natural erosion and human activities such as unrestricted development, removal of surface cover, increased imperviousness (paving that increases runoff) and poor stewardship occur. Sedimentation is the eroded material or soil suspended in wind or water. Construction related erosion and sedimentation can cause problems for downslope property owners, create nuisance problems on adjacent streets, clog streams and storm drains, produce turbidity plumes in surface water bodies, and can cover sensitive habitat areas with sediment. Uncontrolled erosion can be costly, violates state and federal pollution laws, and exposes developers, contractors, and landowners to legal liabilities.

When developing a temporary erosion-control plan for a specific site, you must decide which of the following ***three design objectives*** is most suitable:

- **Keep the soil at its original location.** ←
- **Keep the soil close to its original location.** ←
- **Keep the soil on site.** ←

Keeping the soil at its original location is the ***preferred objective*** because it causes the ***least amount of harm to the environment***. This option not only protects the surrounding land and water, but also prevents costly re-grading and redressing of slopes and ditches. However, keeping the soil at its original location is not always possible due to challenging topography and other site variables. If the soil can't be kept at its original location, it should be kept close. This option will require some re-grading and redressing of slopes and ditches. Finally, if site conditions are such that neither of the first two objectives can be met, then every effort should be made to keep the soil from leaving the site.

The use of soil stabilization practices and sediment control practices should be considered on all construction projects to prevent the discharge and transport of sediment from the construction site.

Temporary Soil Stabilization: Purpose is to protect disturbed soil from erosion by raindrop impact or wind.

- Preservation of Existing Vegetation (ESA fence)
- Hydraulic Mulch
- Hydroseeding
- Soil Binders
- Straw Mulch
- Geotextiles, plastic covers, erosion control blankets/mats
- Earth Dikes/drainage swales to direct flows away from disturbed soil areas
- Outlet protection/velocity dissipation devices (rock slope protection)

Sediment Control Practices: Purpose is to slow water velocities, to allow soil particles to settle out, attenuate flood peak by detaining flow and releasing water at a slower rate.

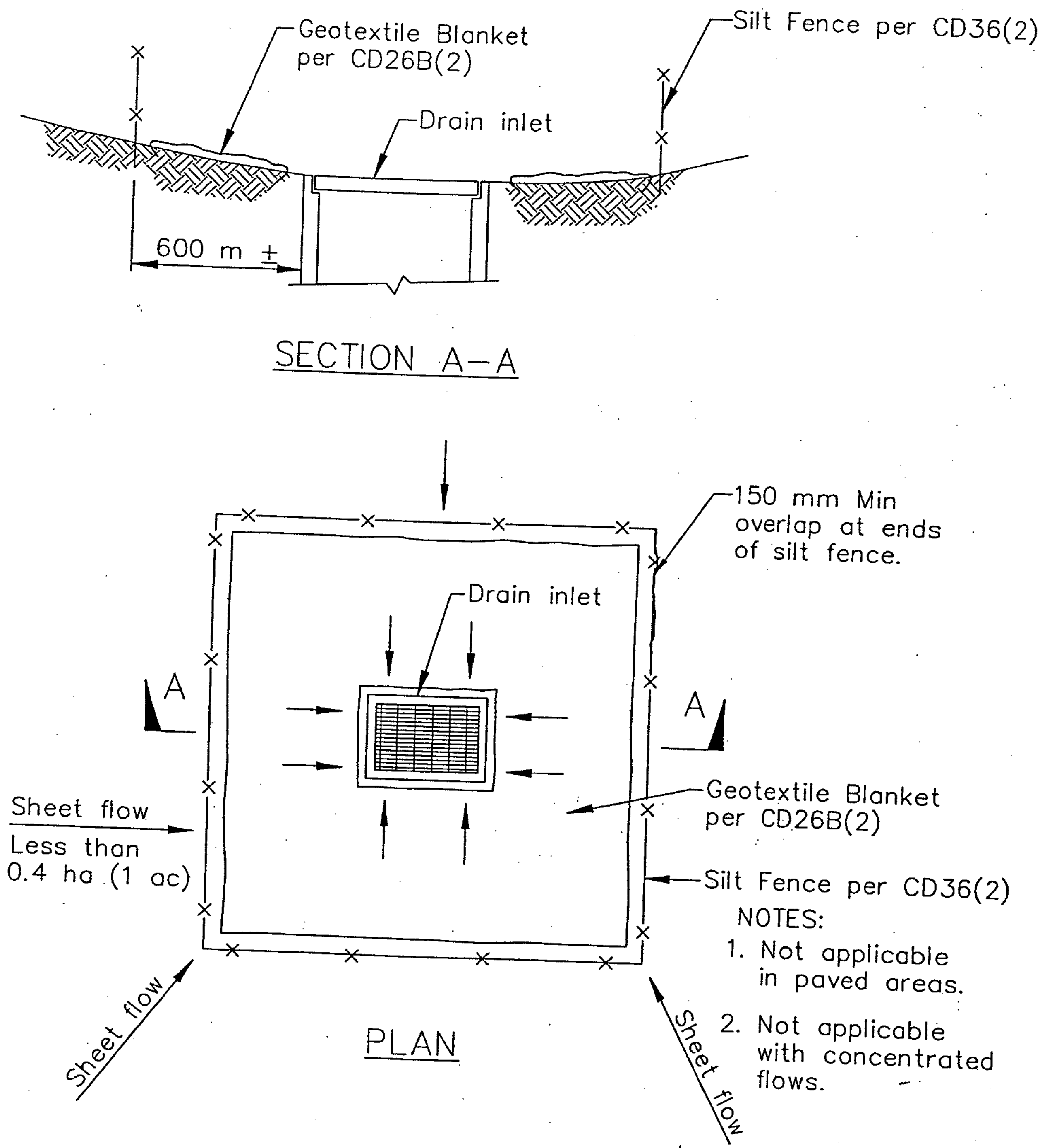
- Silt fence
- Sediment/desilting basins
- Sediment trap
- Check dams (composed of rock, gravel, sand, straw, fiber roll barriers)
- Fiber rolls
- Gravel bag berm
- Street sweeping and vacuuming
- Sand bag barrier
- Straw bale barrier

The following table provides general guidance for the selection of the most appropriate temporary erosion-control measures. The selection of temporary erosion control measures for some situations must be based upon good judgment and past experience under similar conditions.

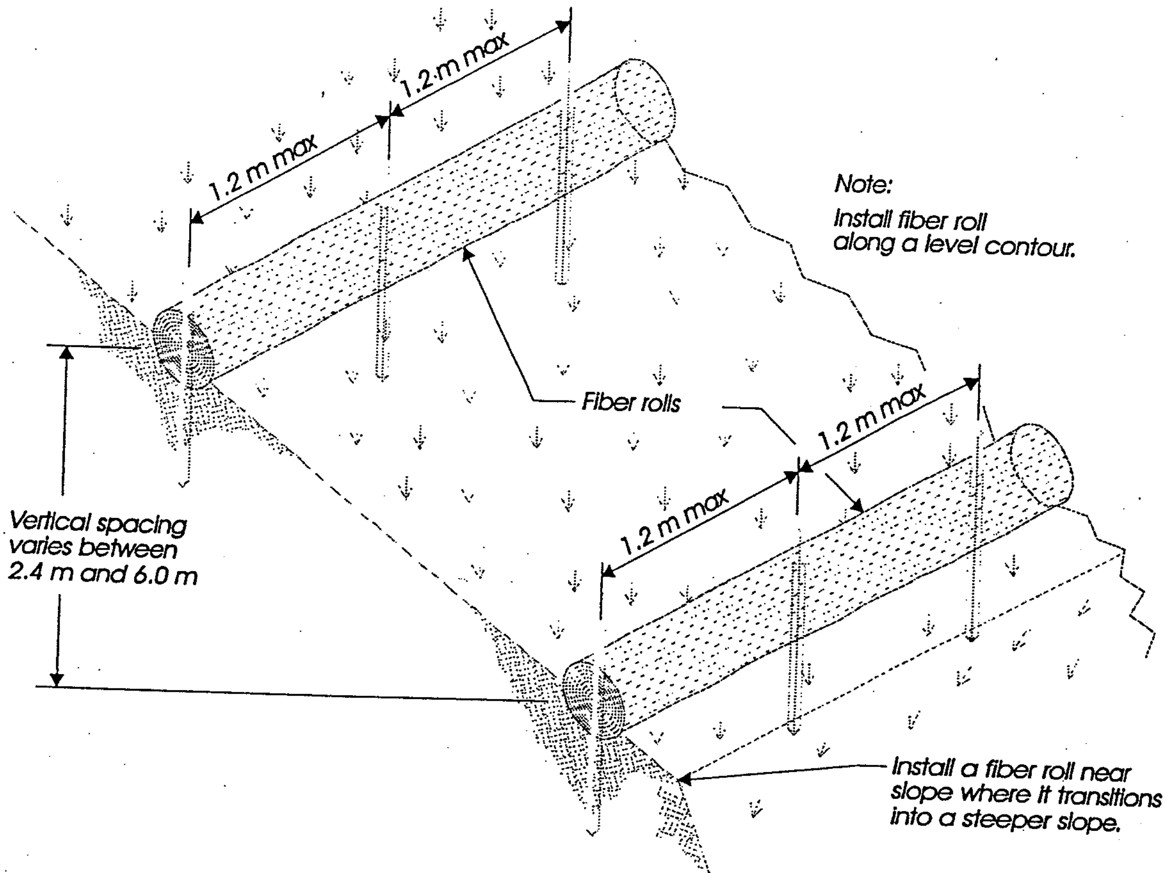
Table III-9 Temporary Erosion Control

Area to Protect	Grade or Control Needed	BMP to Use
Ditches	Grade ≤ 6%	Straw bale barrier
		Silt fence check
	Grade > 6%	Rock ditch check
		Erosion-control blankets
	High flows expected	Rock ditch check
		Erosion-control blankets
Slopes	Erosion - control	Temporary seeding
		Erosion-control blankets
	Sediment control	Fiber roll slope barriers
		Silt fence slope barriers
Drop-inlet Protection	(no decision needed)	Bale drop –inlet barrier
		Silt fence drop –inlet barrier

Following are typical illustrations for temporary erosion control examples.

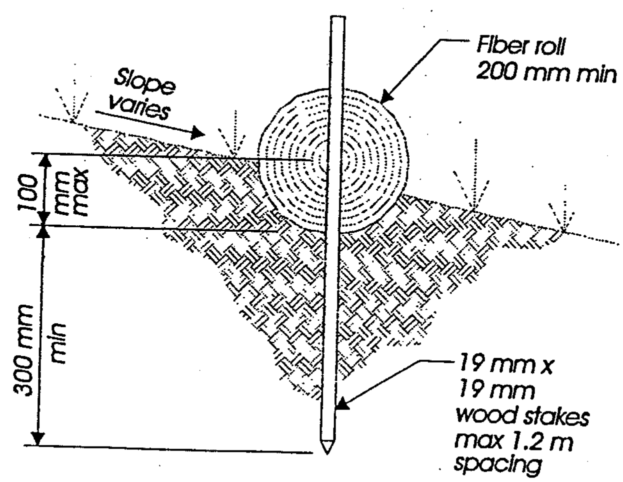


TYPICAL FILTER FABRIC FENCE
NOT TO SCALE



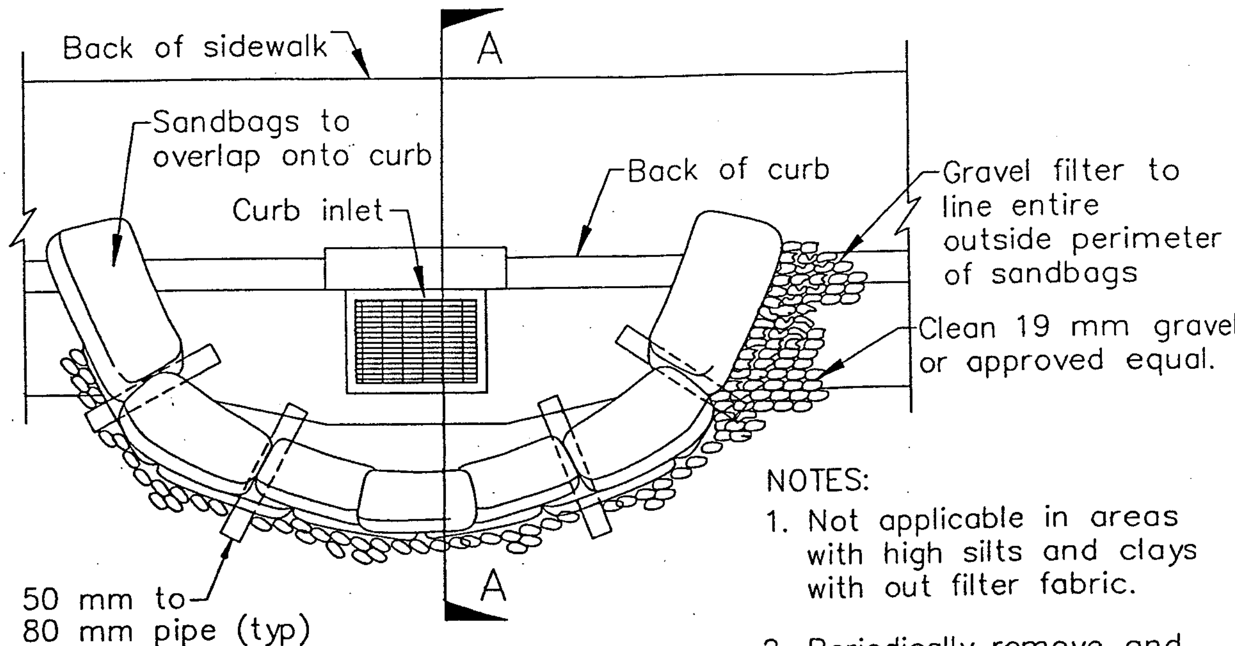
TYPICAL FIBER ROLL INSTALLATION

N.T.S.



ENTRENCHMENT DETAIL

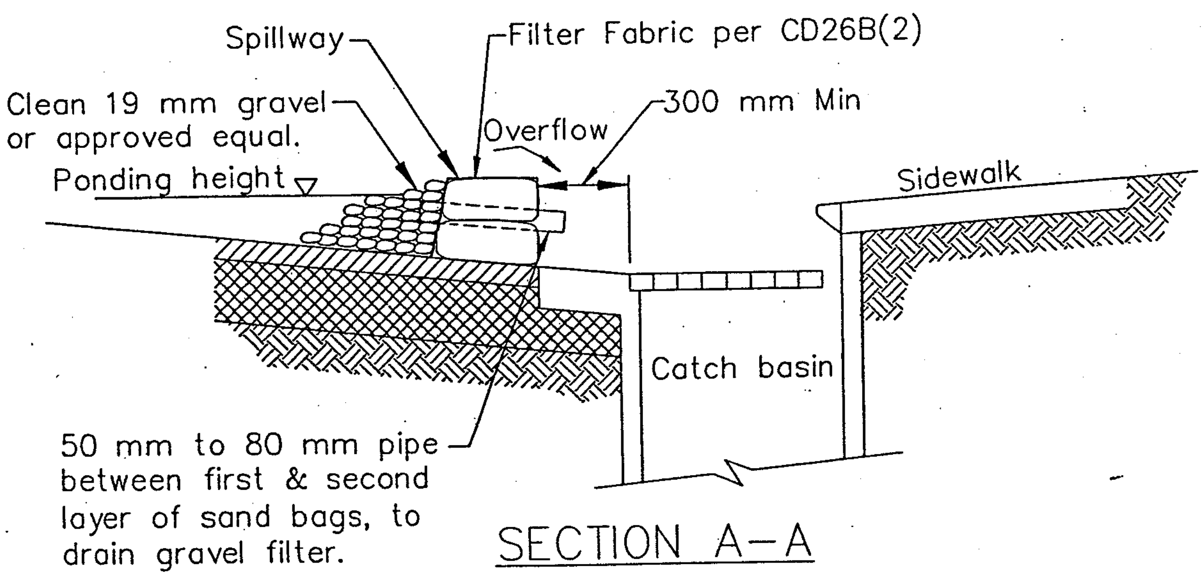
N.T.S.



PLAN

NOTES:

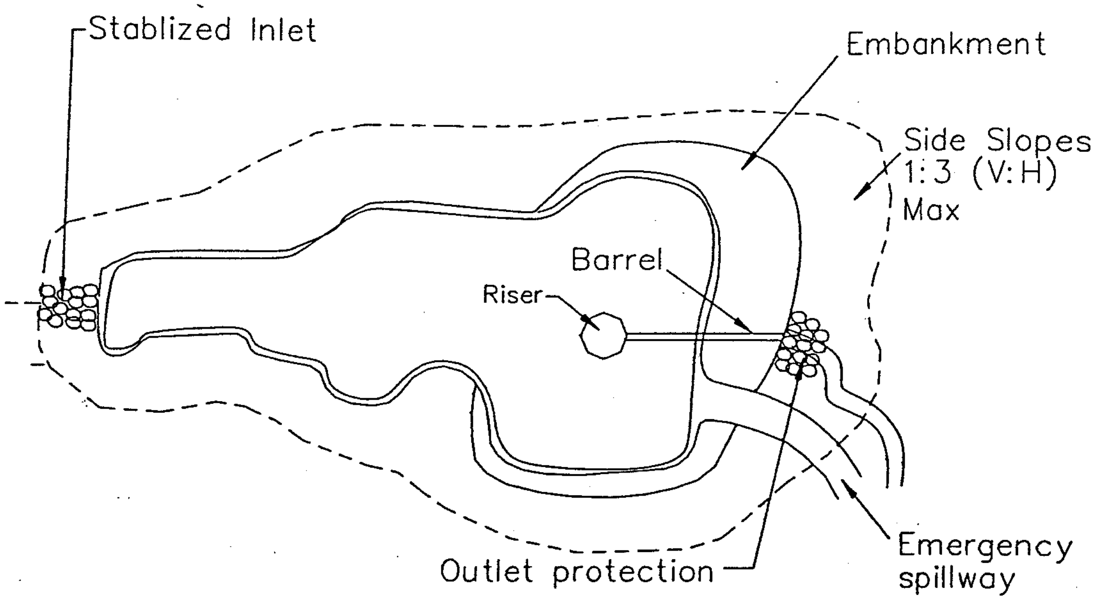
1. Not applicable in areas with high silts and clays with out filter fabric.
2. Periodically remove and replace gravel. Old gravel may be used as backfill material if approved by Engineer.



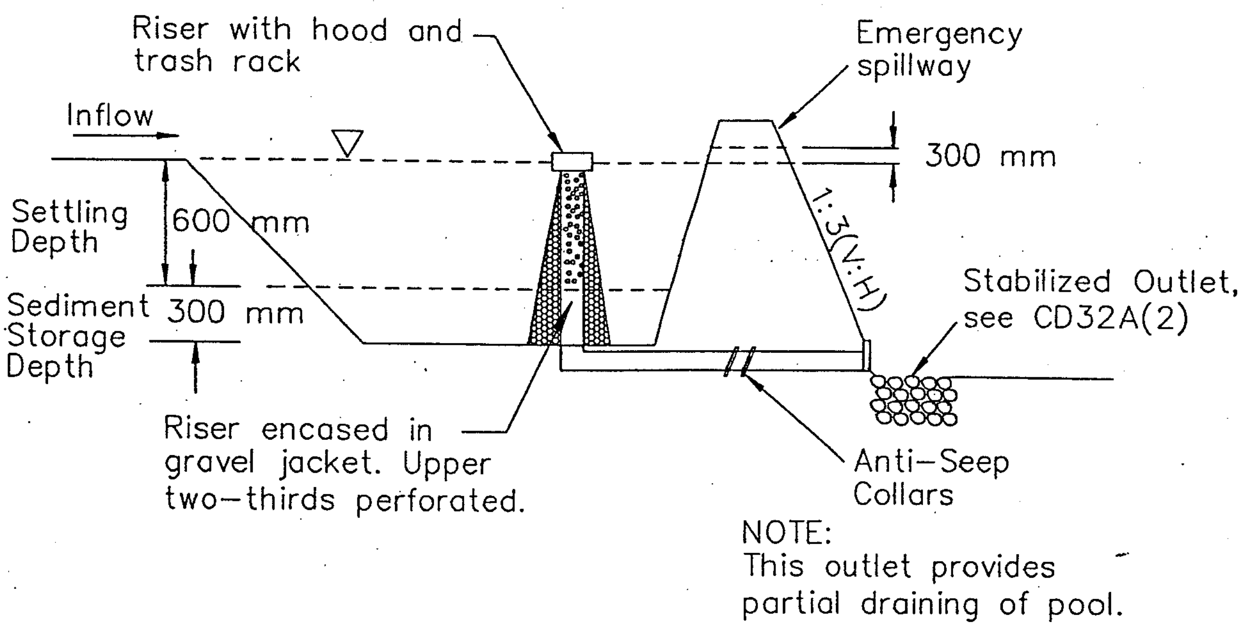
SECTION A–A

TYPICAL SAND BAG BARRIER

NOT TO SCALE



TOP VIEW



TYPICAL TEMPORARY SEDIMENT BASIN – OUTLET #1

NOT TO SCALE

Sample Problem III-12: Erosion for Large Projects

Given: Which of the following is **not** correct. Large construction projects have high potential for erosion and adverse impact on water quality for the following reason:

B) existing drainage is altered
C) slope steepness is increased
D) existing vegetation is preserved
E) impervious surfaces are increased

Solution:

Answer: C ⇐

Sample Problem III-13: Functions of Check Dams

Given: Check dams placed at intervals along channel will help reduce flows.

(A) True
(B) False

Solution:

Answer: B ⇐

Sample Problem III-14: Responsibility

Given: The ___________ is ultimately responsible for the performance of the erosion and sediment control plan on the site.

(A) contractor
(B) property owner
(C) developer
(D) designer

Solution:

Answer: B ⇐

Chapter IV- Scheduling

IV-B CPM NETWORK ANALYSIS

IV-B.1 The Value of Scheduling:

Any project-whether it involves manufacturing, research and development, construction, or even a small personal project such as the organization of personal chores-requires some level of management. The level and skills required depend on the complexity of the project. As the project grows in size, and more and larger organizations become involved, the sophistication of the control system also tends to increase.

The project manager of today is primarily concerned with three objectives:

Scope: Does the project meet the owner's needs?
Cost: Is the project within budget?
Time: Is the project proceeding in a timely manner?

To meet these objectives most construction projects proceed along a somewhat predictable path. In the early stages of a project, the owner assembles the key people that will be the ultimate users of the project and develops the scope of the project. This stage of the project is generally termed the ***Conceptual Phase***.

As the project moves along to the design phase, the criteria are defined in more detail, and the owner's needs are translated into written documents that respond to the physical nature of the project, and to issues such as government and safety concerns. The project is rebudgeted and the timing is reviewed again.

The next and final stage of the project is the ***Construction Phase***. This is when the physical construction of the project begins, the ideas and requirements of the owner are identified, and the design communicated in the preparation of the drawings and specifications becomes a reality. The success of the project also begins to become evident at this point.

- Is the project proceeding within budget?
- Does the project meet the owner's practical, aesthetic, and quality requirements?
- Will the project be completed on time?

The fact is that the further the project proceeds in time, the less opportunity the owner/designer and builder have to influence the project in a cost- effective and timely manner.

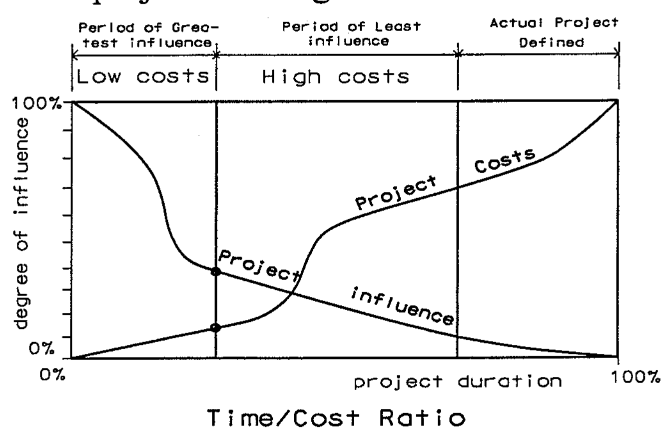


Figure C-1 Relationship Between Time, Degree of Influence and Cost

IV-B.2 Definitions:

1-*Project*: A temporary endeavor undertaken to create a unique product, service or result.

2-*Activity*: A subdivision of a project whose execution requires time and other resources.

3-*Task*: Lowest level of effort on a project

4-*Project schedule*: The planned dates for performing activities and planned dates for meeting milestones.

5-*Milestone*: A significant event in the project, usually completion of a major deliverable e.g. design

6-*Critical Path*: A path connecting all activities that have minimum or zero slack time (float). The critical path is the longest path through the network.

7-*Duration*: The number of work periods required to complete an activity, usually expressed in workday or workweeks.

8- *Effort*: The number of labor units required to complete an activity, usually expressed in staff hours or days

9- *Total Float (TF)*: The amount of time that an activity may be delayed from its early start without delaying the project's finish date.

10-*Free Float*: The amount of time that an activity can be delayed without delaying the early start of any immediately following activities

11-*Triple Constraints*: Scope, Cost, Schedule

12- *Slack*: Comparison between the actual finish and late finish

13- *Schedule*: The addition of time to the activities on a plan or network

14- *Resource Allocation*: Analyzing the manning, interim financing, and material delivery requirements for a project based on estimated production.

15- *Dummy*: A restraint with no activity and no time.

16- *Lag Time*: A restraint used to restrict the finish of an activity or group of activities.

17- *Direct Cost*: Labor, material, and equipment costs used to complete an activity without overhead.

18- *Fast Track*: A method of breaking the project into segments to speed up construction by early contract awarding phases.

C.2.3 Scheduling Tools:

- Gantt (Bar)Chart
- Milestone Chart
- Network Diagrams

  CPM- Critical Path Method

PERT- (Program Evaluation and Review Technique)- Stochastic; Beta distribution
GERT- Graphical Evaluation and Review Technique

- Gantt (Bar) Chart

Bar charts are schedules on which activities are shown graphically on a calendar time scale, each bar representing the start and finish date, with the length of the bar representing the probable duration. Bars show activities in their entirety, disregarding dependency on other activities or deliveries. Figure C-2 is a bar chart, or "GANTT" chart as it is also called, with activities generally classified in the Uniformat system and not necessarily in the sequence of actual construction.

Many project managers, owners, and supervising authorities find the bar chart easy to decipher and prefer this type of presentation. Nevertheless, this is not the most efficient scheduling method. Duration times of the activities are often "backed-up" from an allotted project completion time with little regard for manpower requirements and delivery lead time dependencies. The bar chart in its simplified format, without modifications, shows the project on a broad general basis, and causes the project manager to think in generalities.

The chart does not give the manager the ability to visualize, at a given time, the exact state of progress of a project or to anticipate delays or problems soon enough to correct them.

Furthermore, the bar chart does not show how the overall project duration will be affected by changes. Failure to obtain a delivery on time, a slippage due to unusual weather

conditions, or a change in scope of one or more phases of the project may change the length of one or more bars, but the effect on other activities is not visually apparent.

Experienced project managers can efficiently use bar charts to manage large, complex projects by relying heavily on past experience to identify possible problem areas. However, activities must be mentally monitored continually to ensure all components of the activity are sequentially in order and in phase with other activities in the schedule. If the project manager is relying on bar charts and handling several projects, confusion or omissions are probable without additional information.

- Milestone Chart

Project planning and control has as its broad and overall objective the prescribing, and field attainment, of an orderly progression, within budget and time, toward the completion of the project facilities. These overall project planning and control efforts establish the sequence, time, and cost frameworks within which construction activity is constrained. Thus, the construction planning and control framework is influenced by decisions at the owner, contract administrator, and contractor levels.

From the owner's point of view, there is a need to establish the time and cost framework for the various project components so that the completion of each can be funded, scheduled, and integrated into the project's start-up and business development programs.

Thus, there is management interest in the identification of significant project **milestones** that can serve as indicators of project progress. Accordingly, the monitoring of the progressive achievement of **milestones**, especially in relation to the interaction of anticipated and actual cash flows on budgets and available finance, is of vital interest. sequence and time frame for the completion of each project component significantly affect the definition of the construction plan and the rate at which construction will proceed

- Network Diagrams

In the late 1950s and early 1960s two similar management tools were developed–PERT (Program Evaluation Review Technique) and CPM (Critical Path Method). PERT is event-oriented, and CPM is activity-oriented, but differences between them have all but disappeared in the process of evolution. Both systems use interconnected activities, restraints, and duration times to produce a schedule in keeping with today's sophisticated construction techniques.

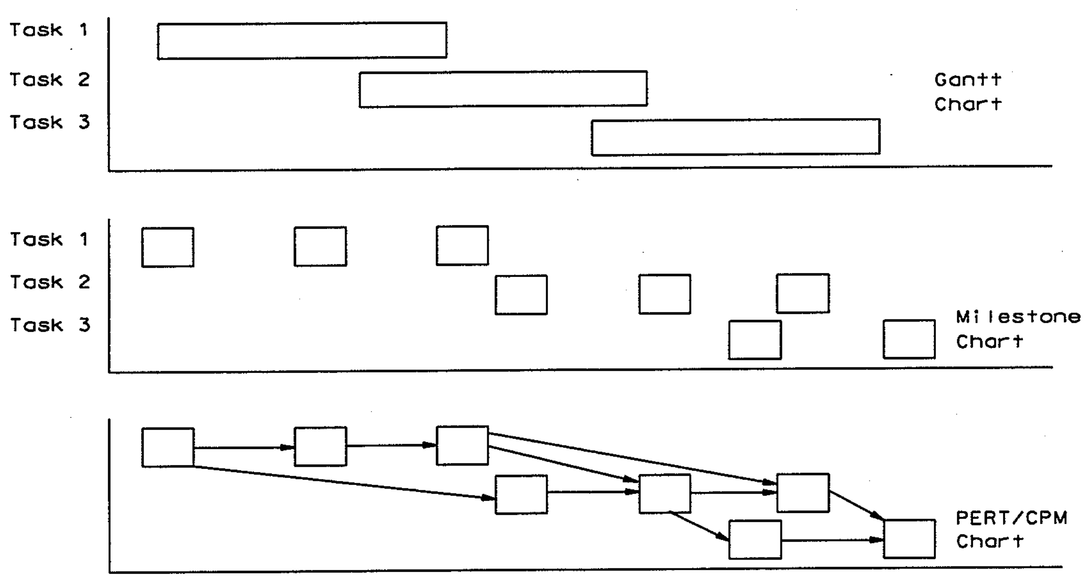


Figure C-2 Bar (GANTT), Milestone and PERT/CPM Charts

Advantages of PERT/CPM

1. Useful at several stages of project management
2. Straightforward in concept, not mathematically complex
3. Uses graphical displays employing networks to help user perceive relationships among project activities
4. Critical path and slack time analyses help pinpoint activities that need to be closely watched
5. Networks generated provide valuable project documentation and graphically point out who is responsible for various project activities
6. Applicable to a wide variety of projects and industries
7. Useful in monitoring not only schedules, but costs as well

Limitations of PERT/CPM

1. Project activities must be clearly defined, independent, and stable in their relationships
2. Precedence relationships must be specified and networked together
3. Time activities in PERT are assumed to follow the beta probability distribution -must be verified
4. Time estimates tend to be subjective, and are subject to fudging by managers
5. There is inherent danger in too much emphasis being placed on the critical path

Six Steps Common to PERT and CPM

1. Define the project and all significant activities/tasks.
2. Develop relationships among the activities. Identify precedence relationships.
3. Draw the network.
4. Assign time and/or cost estimates to each activity.
5. Compute the longest time path (critical path) through the network.
6. Use the network to help plan, schedule, monitor, and control the project.

Program Evaluation Review Technique (PERT)

PERT is a system of scheduling using key events and uncertain time estimates. It is used primarily for projects never before attempted, such as the development of the first nuclear submarine or a new rocket. Duration of activities are estimated using a three-point estimate.

Let:

a = optimistic time (minimum),
m = most likely time, and
b = pessimistic time (maximum)

The expected time, $t_e = \dfrac{a+4m+b}{6}$

OR

Estimate or mean $\mu = \dfrac{pessimistic + 4\,most\ likely + optimistic}{6}$ (71)

Risk is measured by standard deviation (σ)

$$\sigma = \frac{Pessimistic - Optimistic}{6} = \frac{Maximum - Minimum}{6} \quad (72)$$

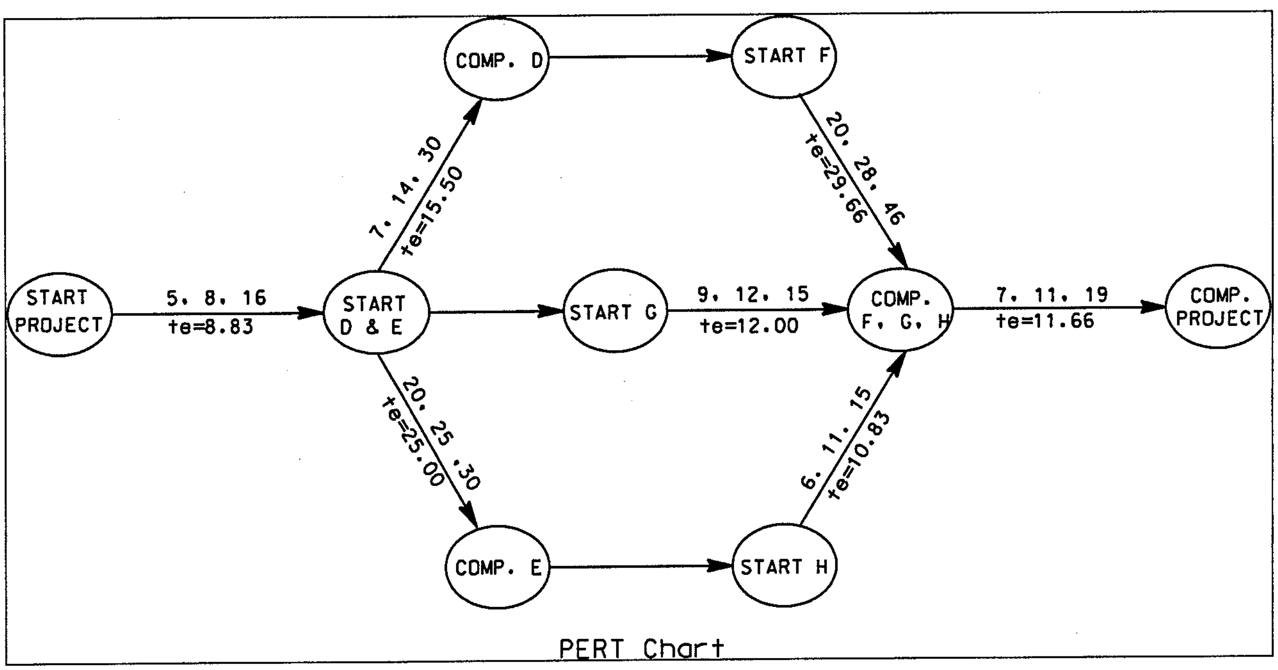


Figure C-3 PERT Chart

Critical Path Method (CPM)

In the CPM network, each activity is represented by a no-scale arrow. The arrows are placed on a diagram and each activity is examined for the following logic:

a) What activities must be completed before this activity may start?
b) What activities may run concurrent with this activity?
c) What activities cannot begin until this activity is complete?

The activity arrow may go from left to right, right to left, up or down, depending on the whim of the scheduler. However, for ease of calculating and recording, it is preferable to work the network from left to right. If a new activity is introduced after completion of the network, such as a work stoppage, it is necessary to add another activity arrow. This added activity arrow may go from right to left, up or down, or at an angle due to space limitations. (See activities 6-8 and 12-14 in Figure C-4

Activity On Arrow (AOA)	Activity on Node (AON ; PDM; CPM)
➢ Activity and duration shown on arrow ➢ All relationship are Finish-Start. ➢ No lags or leads ➢ Uses dummy activity as a dashed line to represents relationship	➢ Nodes represent activities ➢ Arrows represent relation

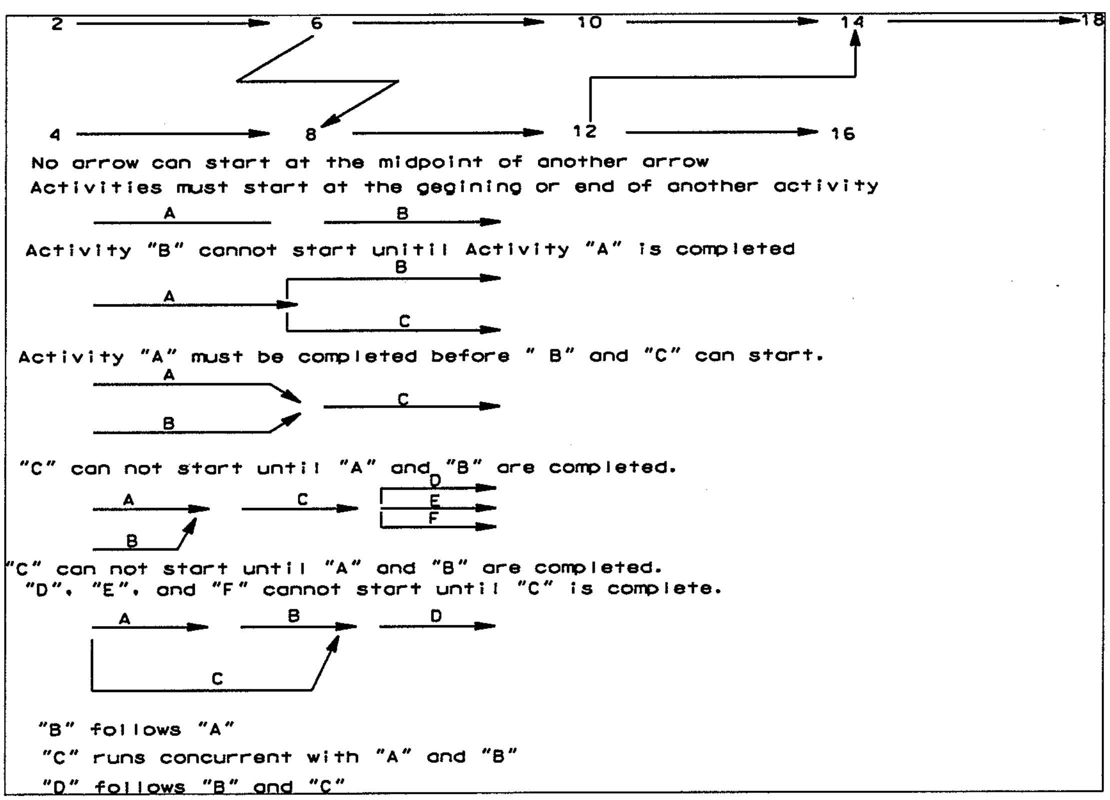


Figure C-4 Interpretations of Activity Arrows of CPM

Abbreviations & Definitions for CPM:

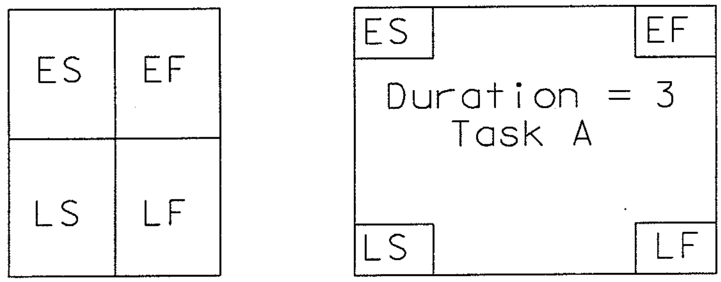


Figure C-5 Abbreviations for CPM

ES: Earliest Start
EF: Earliest Finish
LS: Late Start
LF: Latest Finish

Float = LF – EF (72)

OR

Float = LS – ES (73)

- Float > 0 Time is available
- Float = 0 Situation is critical
- Float < 0 Project is behind schedule

Crashing: Applying more resources to reduce duration

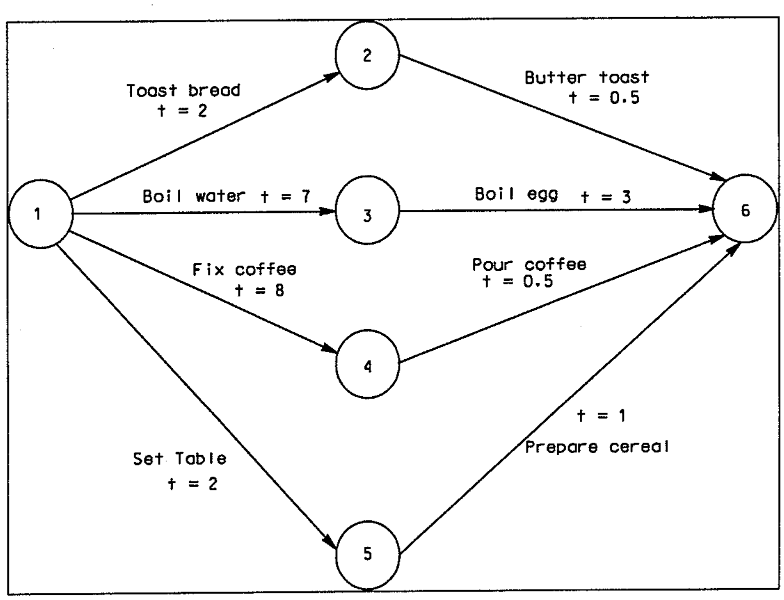


Figure C-6 Activity on Arrow Diagram

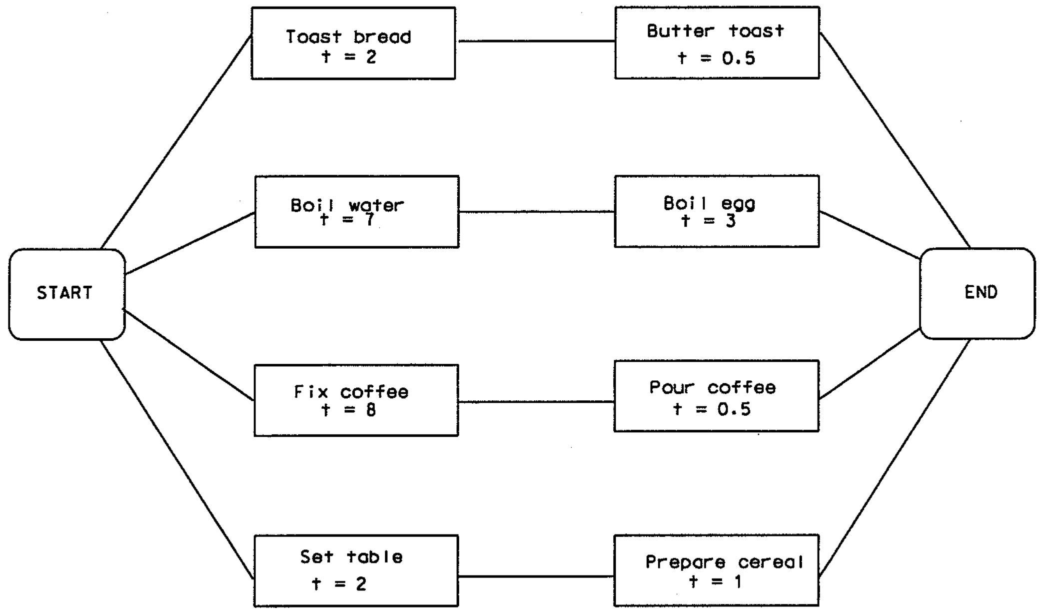


Figure C-7 Precedence Diagram

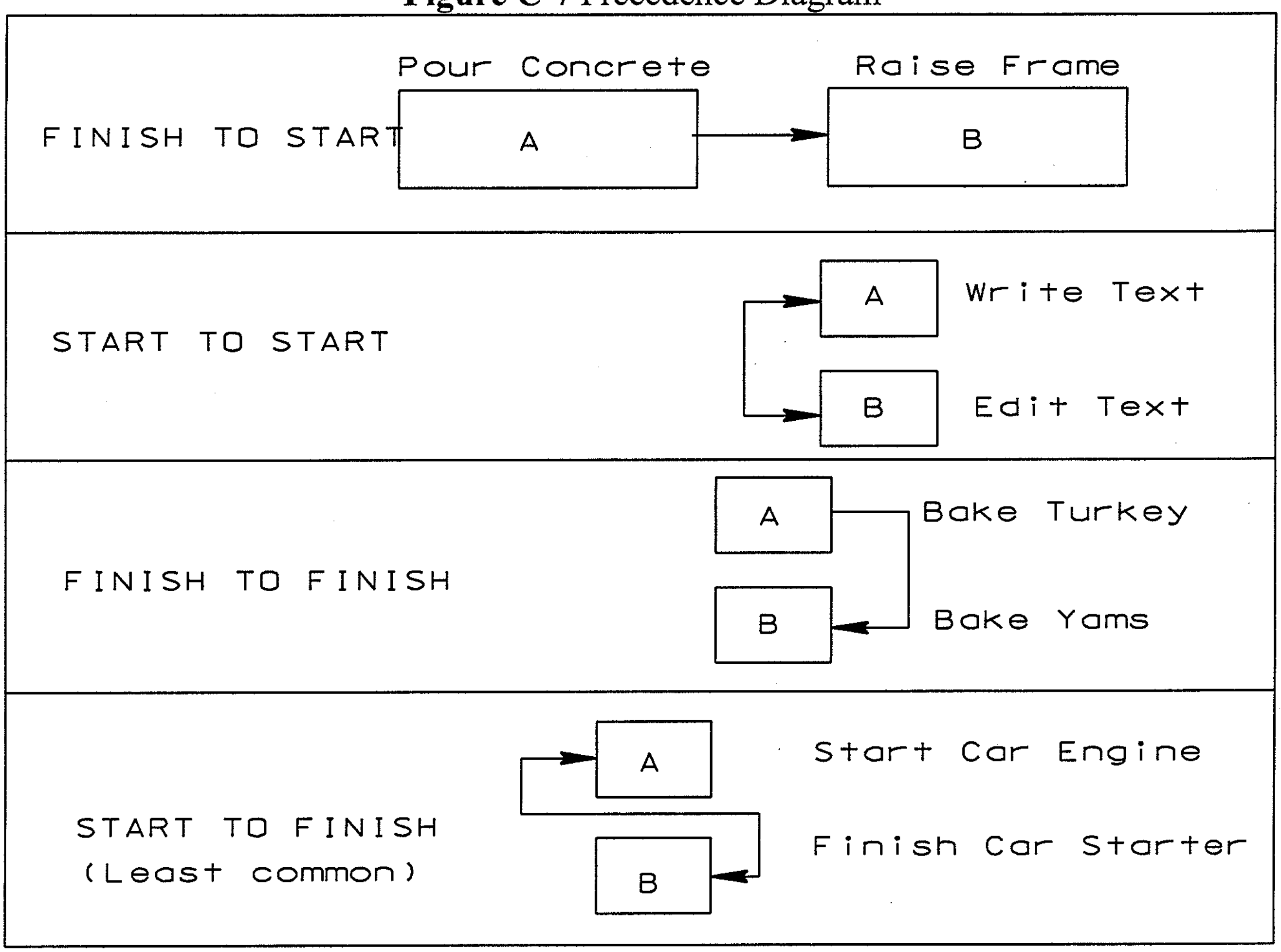


Figure C-8 Types of Relationships

Sample Problem IV-1

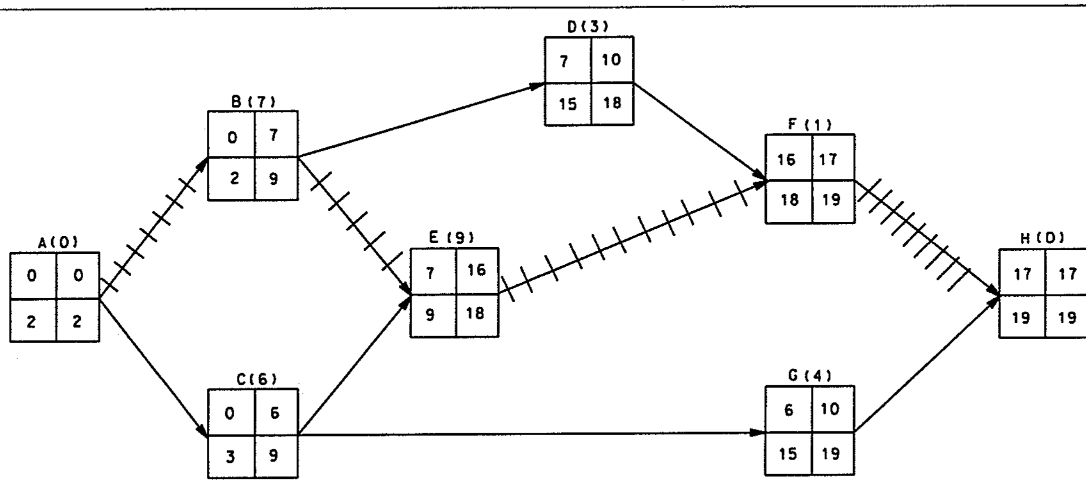
Given: The following table shows the activities and duration for a certain project. Assuming that the completion date is in 19 days.

Activity	*Duration*	*Predecessors*
A, Start	0	----
B	7	A
C	6	A
D	3	B
E	9	B, C
F	1	D, E
G	4	C
H	0	F, G

Find:

The critical path is:

(A) "ABDFH"
(B) "ACEGH"
(C) "ABEFH"
(D) "ACEFH"



NOTE: There are two ways to follow the CPM:

Forward Pass – Finding ES and EF for all activities
Backward Pass – Finding LF and LS for activities

After drawing all the activities with all constraints, the critical path can be determined based on the definition given below:

The definition of "Critical Path" is: <u>A path connecting all activities that have minimum or zero slack time (float). The critical path is the longest path through the network</u>

Answer C⇐

Sample Problem IV-2

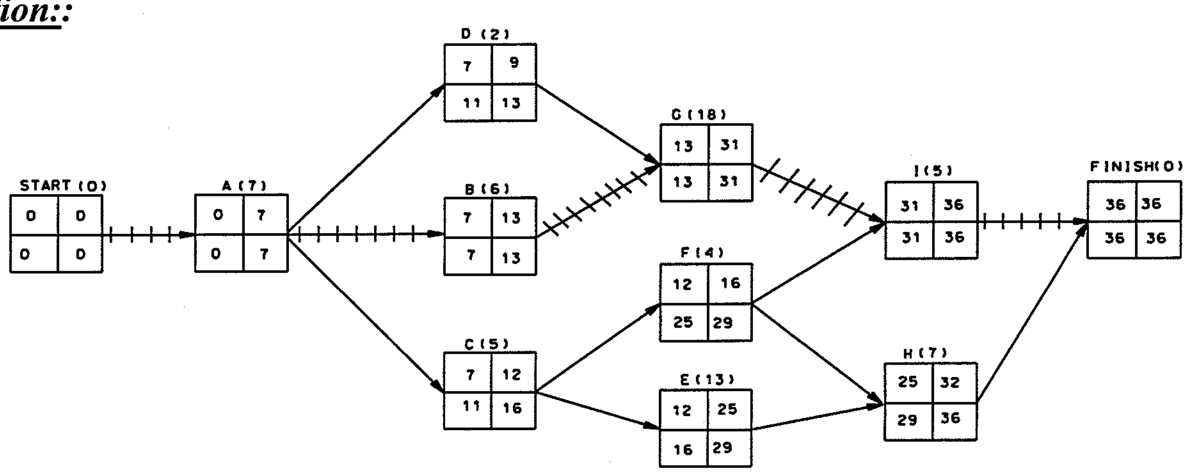
Given: The following table shows the activities and duration for a certain project.

Activity	*Duration*	*Predecessors*	*Successors*
Start	0	----	A
A	7	Start	B,C,D
B	6	A	G
C	5	A	E, F
D	2	A	G
E	13	C	H
F	4	C	H, I
G	18	D, B	I
H	7	E, F	Finish
I	5	F, G	Finish
Finish	0	H, I	-----

Find: The critical path is:

(A) "Start-ABGI-Finish"
(B) "Start-ADGI-Finish"
(C) "Start-ACFI-Finish"
(D) "Start-ABFH-Finish"

Solution::



NOTE: There are two ways to follow the CPM:
Forward Pass - Finding ES and EF for all activities
Backward Pass — Finding LF and LS for activities
After drawing all the activities with all constraints, the critical path can be determined based on the definition given below:
The definition of "Critical Path" is: <u>A path connecting all activities that have minimum or zero slack time (float). The critical path is the longest path through the network</u>

Answer A ⇐

Sample Problem IV-3

Given: Sample Problem IV-2

Find: The earliest finish is most nearly:

(A) 31
(B) 32
(C) 36
(D) 38

Solution:
From the CPM network drawn, the earliest finish is 36

Answer C ⇐

Sample Problem IV-4

Given: Sample Problem IV-2

Find: The Latest Finish is most nearly:

(A) 31
(B) 32
(C) 34
(D) 36

Solution:
From the CPM network drawn, the latest finish is 36

Answer D ⇐

Sample Problem IV-5

Given: Sample Problem IV-2

Find: The slack along the critical path is most nearly:

(A) 0
(B) 2
(C) 4
(D) 13

Solution:
From the CPM network drawn, the slack along the critical path is the difference between actual finish and late finish

Answer A ⇐

Sample Problem IV-6

Given: Sample Problem IV-2

Find: The float along the critical path is most nearly:

(A) 13
(B) 4
(C) 2
(D) 0

Solution:

Float = LF – EF *OR* LS – ES

From the CPM network drawn, the float along the critical path is the difference between late finish and early finish or late start and early start. In both cases the float is ZERO

Answer D ⇐

IV-C ACTIVITY TIME ANALYSIS

See Chapter C of the A.M. Section

Chapter V- Material Quality Control and Production

V-B WELDING AND BOLTING TESTING

Welding-testing is usually understood as a destructive testing.

What do we gain from Welding-testing?

- Proof of performance, peace of mind.
- Use it to certify your welders.
- Know and prove to others adequacy of procedures and of performance,
- show to customers that the operation is successful as expected and needed.

How do you know if there is a test requirement in the weld job?

Welding-testing has to be spelled out on the drawings, or on a technical document, or on a called for Specification or on the purchase order. The purchaser may be able to help you out with the test, as he knows what will satisfy engineering.
Details on standard Welding-testing methods can be found in AWS B4.0-98 Standard Methods for Mechanical Testing of Welds (US Customary Units)

The New York Times

Tuesday, January 1, 2008

3 Plead Guilty In Bolt Testing

A military contracting company and three individuals pleaded guilty this week in Federal court to charges of failing to test bolts for use in combat aircraft, including the F-16 fighter and the B-1B bomber. A military contracting company and three individuals pleaded guilty this week in Federal court to charges of failing to test bolts for use in combat aircraft, including the F-16 fighter and the B-1B bomber.

Norman McHaffie and two workers at his company, McHaffie Inc., also pleaded guilty to submitting fraudulent certificates to the military that the bolts had been tested and met all specifications. Mr. McHaffie, James Hick and William Whitman entered the pleas on Monday, and sentencing was set for June 18, said Grace Denton, a spokeswoman for the United States Attorney's office.
McHaffie Inc., in Sylmar, Calif., could face a maximum of $1.5 million in fines. Mr. McHaffie, who is 56 years old, could be sentenced to 15 years in prison and a fine of $750,000.
Mr. Hicks, 45, and Mr. Whitman, 37, cooperated with the Government in the case, prosecutors said. Each faces a possible maximum sentence of 10 years in prison and $500,000 in fines.

V-C QUALITY CONTROL PROCESS QA/QC

Quality assurance, or "QA" for short, is the activity of providing evidence needed to establish quality in work, and that activities that require good quality are being performed effectively. All those planned or systematic actions necessary to provide enough confidence that a product or service will satisfy the given requirements for quality.

Quality assurance activities

Quality assurance covers all activities from design, development, production, installation, servicing and documentation. This introduced the rules: "fit for purpose" and "do it right the first time". It includes the regulation of the quality of raw materials, assemblies, products and components; services related to production; and management, production, and inspection processes.
One of the most widely used paradigms for QA management is the PDCA (Plan-Do-Check-Act) approach, also known as the Shewhart cycle.

Failure testing

A valuable process to perform on a whole consumer product is failure testing, the operation of a product until it fails, often under stresses such as increasing vibration, temperature and humidity. This exposes many unanticipated weaknesses in a product, and the data is used to drive engineering and manufacturing process improvements. Often quite simple changes can dramatically improve product service, such as changing to mould-resistant paint or adding lock-washer placement to the training for new assembly personnel.

Statistical control

Many organizations use statistical process control to bring the organization to 6σ levels of quality, in other words, so that the likelihood of an unexpected failure is confined to six standard deviations on the normal distribution. This probability is less than four one-millionths. Items controlled often include clerical tasks such as order-entry as well as conventional manufacturing tasks.

Traditional statistical process controls in manufacturing operations usually proceed by randomly sampling and testing a fraction of the output. Variances of critical tolerances are continuously tracked, and manufacturing processes are corrected before bad parts can be produced.

Total quality control

Total Quality Control is the most necessary inspection control of all in cases where, despite statistical quality control techniques or quality improvements implemented, sales decrease.

The major problem which leads to a decrease in sales was that the specifications did not include the most important factor, "What the customer required".

The major characteristics, ignored during the search to improve manufacture and overall business performance were:

1. Reliability
2. Maintainability
3. Safety

As the most important factor had been ignored, a few refinements had to be introduced:

1. Marketing had to carry out their work properly and define the customer's specifications.
2. Specifications had to be defined to conform to these requirements.
3. Conformance to specifications i.e. drawings, standards and other relevant documents, were introduced during manufacturing, planning and control.
4. Management had to confirm all operators are equal to the work imposed on them and holidays, celebrations and disputes did not affect any of the quality levels.
5. Inspections and tests were carried out, and all components and materials, bought in or otherwise, conformed to the specifications, and the measuring equipment was accurate, this is the responsibility of the QA/QC department.
6. Any complaints received from the customers were satisfactorily dealt with in a timely manner.
7. Feedback from the user/customer is used to review designs.
8. Consistent data recording and assessment and documentation integrity.
9. Product and/or process change management and notification.

If the original specification does not reflect the correct quality requirements, quality cannot be inspected or manufactured into the product. For instance, all parameters for a pressure vessel should include not only the material and dimensions but operating, environmental, safety, reliability and maintainability requirements.

V-D CONCRETE MIX DESIGN

V-D.1 Introduction and Definitions:

The properties of concrete could be classified as:

Table V-1 Hardened and Fresh Concrete Properties

	Hardened Concrete	Freshly Mixed Concrete
Properties	a) compressive strength ($f_c^{'}$), b) modulus of rupture (f_r), c) durability, and d) water tightness	d) workability(slump test), e) unit weight, and f) temperature
Factors Affected by the Properties	4- type of the structure 5- functions of the structures 6- environmental conditions	4- method of construction, 5- transporting, and 6- finishing methods and techniques.

Based on the required properties of fresh and hardened concrete, the engineer can determine the amount of each ingredient of the concrete mix.

Definition: mix design is the process of selecting the appropriate ingredients and their quantities to produce an economical concrete mix having certain properties in the fresh and the hardened state.

The following definitions will be used throughout this Section:

Bulk specific gravity (saturated surface dry): The ratio of the mass of a volume of a material including the mass of water within the voids in the material (but excluding the voids between particles) at a stated temperature, to the mass of an equal volume of distilled water at a stated temperature.

Bulk specific gravity: The ratio of the mass of a volume of a material (including the permeable and impermeable voids in the material, but excluding the voids between particles of the material) at a stated temperature of the mass of an equal volume of distilled water at a stated temperature.

Cement factor: The number of bags or cubic feet of cement per cubic yard of concrete.

Dry rodded unit weight of coarse aggregate: The unit weight of coarse aggregate obtained by rodding three equal layers 25 times, lb/ft^3 (kg/m^3)

Fineness modulus: An empirical factor obtained by adding the cumulative percentages by weight retained on each of a specific series of sieves and dividing the sum by 100.

Saturated surface dry (SSD): A condition of an aggregate which holds as much water as it can without having any free surface water between the aggregate particles.

Water-cement ratio: The ratio of the amount of water, exclusive only of that absorbed by the aggregates, to the amount of cement in a concrete or mortar mixture; preferably stated as a decimal by weight. Alternatively, the number of gallons of water per 94 pound sack of cement.

Workability of concrete: That property determine the effort required to transport, place and finish a freshly mixed quantity of concrete without segregation or bleeding.

V-D.2 Components (Ingredients) of Concrete:

Concrete is a mixture of four basic materials: fine aggregate, coarse aggregate, cement and water. The following is a brief discussion of each ingredient:

Fine aggregates: Fine aggregates generally consist of natural sand or crushed stone with most particles smaller than 0.2 in (5 mm). Aggregates either fine or coarse must conform to certain standards to achieve the best engineering properties; they must be strong, clean, hard, free of absorbed chemicals. Fine aggregates must meet the particle -size distribution (grading). The seven standard ASTM C33 sieves for fine aggregate have openings ranging from No. 100 sieve (150 μ m) to 3/8 in. Another requirement for the fine aggregate is the fineness modulus. The fine aggregate shall have not more than 45% passing any sieve and retained on the next consecutive sieve and its fineness modulus shall be not less than 2.3 nor more than 3.11.

Coarse aggregates: Coarse aggregates consist of one or a combination of gravels or crushed aggregate with particles predominantly larger than 0.2 in. (5 mm) and generally between 3/8 and 1 1/2 in (9.5 and 38.1 mm). Also coarse aggregates should meet the gradation requirements as stated in the ASTM C33. The coarse aggregate has three main functions in a concrete mix: (1) to provide a relatively inexpensive filler, (2) to provide a mass of particles that are suitable for resisting the action of applied loads, and (3) to reduce the volume changes caused by the setting and hardening process and the drying of the cement-water paste.

Cement: Portland cement is produced by burning in a rotary kiln a properly proportioned mixture of lime and clay. There are five types of portland cement, and the choice is dependent on the application:

Type I: Normal portland cement: This is a general-purpose cement used whenever sulfate hazards are absent and when the heat of hydration will not produce significant rises in temperature. Typical uses are <u>sidewalks, pavement, beams, columns, and culverts.</u>

Type II: Modified portland cement: This cement has a moderate sulfate resistance, but is generally used in hot weather in the construction of large concrete structures. Its heat rate and total heat generation are lower than for normal Portland cement.

Type III: High-early strength portland cement: This type develops its strength quickly. It is suitable for use when structure must be put into early use or when long-term protection against cold temperature is not feasible. Its shrinkage rate, however, is higher than for types I and II; and extensive cracking may result.

Type IV: Low-heat portland cement: For extensive concrete structures, such as gravity dams, low-heat cement is required to minimize the curing heat. The ultimate strength also develops more slowly than for the other types.

Type V: Sulfate-resistance portland cement: This type of cement is applicable when exposure to sulfate concentration is expected. This typically occurs in States having highly alkaline soils.

The American Society for Testing and Materials (ASTM) Designation C150, Standard Specification for Portland Cement, provides further classification for Portland cement as follows:

Type I	Normal
Type IA	Normal, air-entraining
Type II	Moderate sulfate resistance
Type IIA	Moderate sulfate resistance, air-entraining
Type III	High early strength
Type IIIA	High early strength, air-entraining
Type IV	Low heat of hydration
Type V	High sulfate resistance

The composition of the three types of air-entraining portland cement (Types IA, IIA, and III A) is similar to ASTM Types I, II and III, respectively, with the exception that during manufacture of these types an amount of air-entraining material is added. This air-entraining material will improve durability of concrete expose to freeze-thaw action and will reduce scaling caused by chemicals during removal of ice and snow.

Water: The presence of water in a concrete mix has ***three functions***:

a) To react chemically with the cement. This chemical reaction is known as hydration. The strength of hardened concrete depends on the completeness of this chemical reaction between water and cement.

b) To wet the aggregate, and

c) The mix of water and cement which is known as the paste will lubricate the mixture for easy workability.

Water used for concrete mixing could be any natural water that is drinkable (potable) that has no pronounced odor or taste. Mixing water that contains impurities will affect the setting time of concrete, concrete strength and corrosion of reinforcing bars or wires. In a summary, water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salt, organic materials, or other substances that may be deleterious to concrete or reinforcement.

The ACI Code allows for a nonpotable water to be used in concrete mixing providing that:

a) A water from the same source will be used for concrete proportions, and

b) The 7-day and 28-day compressive strength of mortar cubes made with nonpotable water equal at least 90 percent of strengths of similar specimens made with potable water.

Admixtures:

Concrete consists of four basic ingredients: fine aggregate, coarse aggregate, portland cement and water. Admixtures are ingredients other than the four basic ingredients that added to the concrete mix before or during mixing.

Admixtures normally added to concrete mixtures to achieve one or all of the followings:

i) Lower construction cost of concrete.

ii) Maintain the quality of fresh concrete during mixing, transporting, placing and curing concrete under upnormal weather conditions, and

iii) Achieving certain properties of concrete instead of normal means or techniques.

At the present time there are different types of admixtures that could be categorized according to their functions. The following is a brief discussion of each type:

1- **Air-entraining admixtures**: This type of admixture will create million of microscopic air bubbles in concrete. This will improve the durability of concrete subject to freezing and thawing conditions. In addition to the improved durability of hardened concrete, the workability of freshly mixed concrete is also improved. The bleeding and segregation of an air-entrained concrete are also reduced.

2- ***Water-reducing admixtures***: The functions of these types of admixtures are:

a) To reduce water/cement ratio which will increase the strength of hardened concrete.

b) Increase the slump of freshly mixed concrete. Concretes with higher slump are easy to place, finish and the probability of honeycombs existence will be small. The reduction in the mixing water varies between 5 and 15 %.

3-***Retarding admixtures***: These admixtures delay the setting of the cement paste in a concrete mix. This delay is very important when mixing, transporting and placing concrete in hot weather which tends to speed up the setting of concrete. Also, retarders are used to delay the initial set of concrete when unusual conditions exist.

4- ***Accelerating admixtures***: The rate of development of strength of concrete increases by the addition of calcium chloride. Accelerating admixtures used when:

a) In cold regions where concrete to be placed at low temperatures (about 35° to 40° F).

b) An urgent repair work to be done as in case of bridges or highways damaged due to an earthquake.

The most commonly material used as an accelerating admixture is calcium chloride ($CaCl_2$). The ACI code does not allow chloride to be added to concrete used in prestressed construction or in concrete containing aluminum embedments because of corrosion that could occur for steel and aluminum.

5- Superplasticizers: Superplasticizers or high-range water reducers are another type of water reducing admixtures. They are used mainly to make concrete more workable (high slump concrete). High slump concrete is suitable for concrete sections heavily reinforced and areas where consolidation can not be attained by the regular techniques. Also, more workable concrete can be pumped at low pump pressure and consequently the lift and the distance of pumping concrete increase. Overall, superplasticizers reduce the cost of mixing, pumping and finishing concrete.

V-D.3 Proportioning Concrete Mixtures:

When a designer proportions a concrete mix, the following objectives should be taken into consideration:

1- The concrete mix in the fresh state should have an acceptable workability. The acceptable workability depends on the types and methods of construction

2- The concrete in the hardened state should provide the required design compressive strength, modulus of rupture and durability, and

3- The mix should be proportion to produce the most economical concrete.

The concrete in the hardened state could be controlled by either the compressive strength or durability. Therefore, the designer should determine which one; strength or durability, is the controlling property when a concrete mix to be proportioned. The design compressive strength of concrete, f_c', is defined as the strength of standard concrete cylinders (6″ diameter by 12″ height) tested at specified age (normally at 28 days) under uniaxial (the simple compressive stress in one direction) compressive strength according to the ASTM C39. According to the ACI Code, the compressive strength of concrete is considered satisfactory if both of the following criteria are met:

1- No single test (average of two cylinders) shall be more than 500 psi (3.45 MPa) below the specified compressive strength, and

2- The average of any three consecutive test strength must equal or exceed the specified compressive strength.

Durability of concrete is defined as the ability of concrete in the hardened state to resist deterioration from the environment or service where the concrete will be subjected to. One of the most destructive environmental factor that deteriorates concrete is freezing and thawing. Deterioration is caused by the freezing of the water in the paste, the aggregate particles, or both.

The ASTM C 666, Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing, is the standard method to determine the freeze-thaw durability for hardened concrete in the laboratory. From this test, a durability factor is calculated which indicates the number of cycles of freezing and thawing required to produce a certain amount of deterioration.

When durability of concrete is the controlling factor, the water cement ratio for mixing concrete will be selected according to the following table:

Table V-2 Maximum Water-Cementitous Material Ratios and Minimum Design Strengths for Various Exposure Conditions

Exposure condition	**Maximum water-cement ratio by weight for normal-weight concrete**	**Minimum design compressive strength, f_c', MPa (psi)**
Concrete protected from exposure to freezing and thawing or application of deicing chemicals, or aggressive substances	Select water-cement ratio on basis of strength, workability, and finishing needs	Select strength based on structural requirements
Concrete intended to have low permeability when exposed to water	0.50	28 (4000)
Concrete exposed to deicers or freezing and thawing in a moist condition	0.45	31(4500)
For corrosion protection for reinforced concrete exposed to deicing salts, brackish water, seawater, or spray from these sources	0.40	35(5000)

To improve the durability of concrete subject to freezing and thawing or deicer chemicals, an air-entrained concrete shall be used. An air entrained concrete is a concrete containing millions of small air bubbles intentionally created in the mix by using either an air-entraining cement or an air-entraining admixture to the mixing water

Table V-3 Bulk Volume of Coarse Aggregate per Unit Volume of Concrete

Nominal maximum size of aggregate in. (mm)	Bulk volume of dry-rodded coarse aggregate per unit volume of concrete for different fineness moduli of fine aggregate			
	2.40	**2.60**	**2.80**	**3.00**
⅜ (9.5)	0.50	0.48	0.46	0.44
½ (12.5)	0.59	0.57	0.55	0.53
¾ (19)	0.66	0.64	0.62	0.60
1 (25)	0.71	0.69	0.67	0.65
1 ½ (37.5)	0.75	0.73	0.71	0.69
2 (50)	0.78	0.76	0.74	0.72
3 (75)	0.82	0.80	0.78	0.76
6 (150)	0.87	0.85	0.83	0.81

Table V-4 Recommended Slumps for Various Types of Construction

Concrete construction	Slump, in (mm)	
	Maximum[a]	**Minimum**
Reinforced foundations, walls,	3 (75)	1 (25)
Plain footings, caissons, and substructure walls	3 (75)	1 (25)
Beams and reinforced walls	4 (100)	1 (25)
Building columns	4 (100)	1 (25)
Pavements and slabs	3 (75)	1 (25)
Mass concrete	3 (75)	1 (25)

a: May be increased 1 in (25 mm) for consolidation by hand methods such as rodding or spading

The amount of entrained air will depend on the exposure conditions and the size of coarse aggregate as shown in Table V-6.

Water-cement ratio and strength: The relationship between water-cement ratio and strength of concrete is inversely proportional. If the selection of the ingredients of concrete will be controlled by the compressive strength of concrete, Table V-5 could be used for the selection process. It should be noted that Table V-5

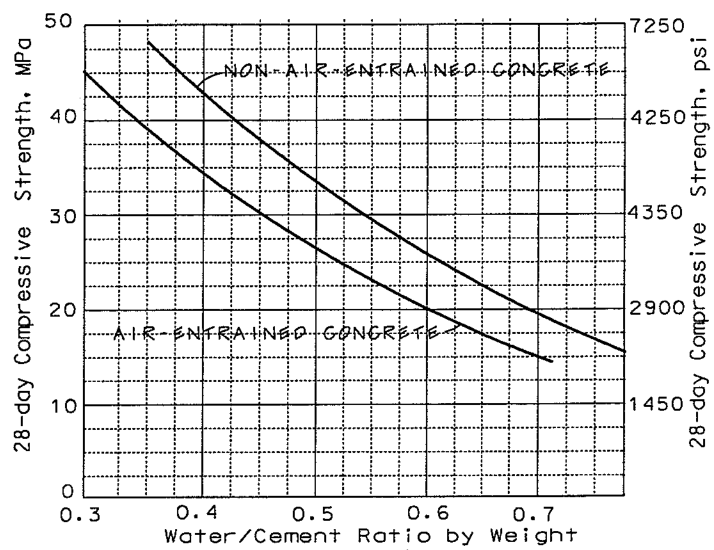


Figure V-1 W/C Ratio & Compressive Strength will be used when past data is not available and trial mixes will not be made. As indicated before, if durability not the strength of concrete is the controlling factor, then the water-cement ratio will be selected according to the following Table V-2.

Table V-5 Relationship Between Water to Cementitious Material Ratio and Compressive Strength of Concrete

compressive strength at 28 days, f_c', (lb/in^2)	Water-cementitious materials ratio by mass	
	Non-air entrained concrete	**Air-entrained concrete**
7000	0.33	-
6000	0.41	0.32
5000	0.48	0.40
4000	0.57	0.48
3000	0.68	0.59
2000	0.82	0.74

Table V-6 Approximate Mixing Water and Target Air Content Requirements for Different Slumps and Nominal Maximum Sizes of Aggregate

Slump, in.	lb/yd^3 of concrete for indicated maximum sizes of aggregate							
	3/8 in.	**1/2 in.**	**3/4 in.**	**1 in.**	**1 1/2 in.**	**2 in.**	**3 in.**	**6 in.**
Non-air-entrained concrete								
1 to 2	350	335	315	300	275	260	220	190
3 to 4	385	365	340	325	300	285	245	210
6 to 7	410	385	360	340	315	300	270	-
Approximate amount of entrapped air in non-air-entrained concrete, percent	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
1 to 2	305	295	280	270	250	240	205	180
3 to 4	340	325	305	295	275	265	225	200
6 to 7	365	345	325	310	290	280	260	-
Recommended average total air content, percent, for level of exposure:								
Mild exposure	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
Moderate exposure	6.0	5.5	5.0	4.5	4.5	3.5	3.5	3.0
Severe exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5	4.0

Unit weight and yield: The unit weight (also known as the weight density) of freshly mixed concrete is the weight of a specified volume expressed in kilograms per cubic meter (kg/ m^3) or pounds per cubic feet (pcf). The unit weight of concrete can vary from 100 pcf to 160 pcf (1598-2557 kg/m^3) depending on the mixture ratios and the specific gravities of the constituents. ***Normal weight concrete*** has a unit weight of ***145-150 pcf*** (2317- 2397 kg/m^3).

The yield is the volume of fresh concrete produced in a batch and is equal to:

$$yield = \frac{Total\ weight\ of\ material\ per\ batch}{Unit\ weight\ of\ concrete} \tag{74}$$

The units of the yield are cubic feet or cubic meters

V-D.4 Concrete proportioning methods:

1- Arbitrary volume or weight method

This is the oldest method of proportioning concrete where a ratio of cement, fine aggregate, and coarse aggregate is designated. For example, ***1: 2: 3 means that one part of cement, two parts of fine aggregate, and three parts of coarse aggregate*** are combined. The ratio can be either in terms of weight or volume. Weight ratio are more common. The amount of water used is called out in terms of gallons of water per 94 pound sack of cement which is known as the water-cement ratio.

2- Absolute volume method

The amount of concrete that can be made from mixing known quantities of ingredients can be found from the absolute volume method. It involves the use of specific gravity values for all the ingredients to calculate the absolute volume each will occupy in a unit volume of concrete. The absolute volume is calculated according to the following equation:

$$Absolute\ volume = \frac{Weight\ of\ the\ material}{Specific\ gravity\ of\ the\ material \times Unit\ weight\ of\ water} \tag{75}$$

The absolute volume assumes that for granular materials as cement and aggregates, there will be no voids between particles. Therefore, the amount of concrete is the sum of the solid volumes of the cement, sand, coarse aggregate, and water.

To use the absolute volume method, it is necessary to know the solid densities of the constituents. In the absence of other information, the following data can be used for solid densities: ***Cement 195 pcf; fine aggregate, 105 pcf; coarse aggregate, 105 pcf; water, 62.4 pcf. A sack of cement weights 94 pounds, and 7.48 gallons of water make a cubic foot. Alternatively, there are 239.7 gallons per ton of water.***

If the mix proportions are volumetric, it will be necessary to multiply the ratio values by the bulk densities to get the weights of the constituents. Then, weight ratios may be calculated and the absolute volume method applied directly.

The problem may be complicated by air entrainment and/or the water content of the aggregate. The following guidelines should be observed:

* The yield is increased by the addition of air. This can be accounted for by dividing the solid yield by (1- air percentage)
* Any water in the aggregate above the saturated surface dry (SSD) water content must be subtracted from the water requirements.
* Any porosity (water affinity) below the SSD water content must be added top the water requirements.
* The densities used in the calculation of yield should be the SSD densities.

V-D.5 Batching, Mixing, Transporting, and Handling Concrete:

Batching

In order to obtain quality concrete, all ingredients must be weighed accurately before being mixed together. The process of weighing (or volumetrically measuring) the ingredients for a batch of concrete is known batching. The weighing method is the most common technique used for batching of concrete because it is simple and accurate. However, ingredients like water and liquid admixtures could be measured accurately by either volume or weight. The following percentages of accuracy are commonly specified in the concrete industry: cement 1%, aggregates 2%, water 1% and admixtures 3%.

Mixing Concrete

The objective of mixing is to coat the surface of all aggregate particles with cement paste, and to blend all the ingredients of concrete into a uniform mass. All ingredients of concrete should be mixed together to produce a uniform mixture where the ingredients are well distributed throughout the entire batch. The mixing method depends on many factors such as: the quantity of concrete required for the job, location of the job, availability of the ingredients, and transportation costs. Below is a brief discussion of ***three mixing methods***:

1- ***Stationary Mixing***: Stationary mixer are mixing units that can be moved from one jobsite to another. Stationary mixers can also be found in ready mix plants. They are available in sizes from 2 cubic foot to 12 cubic yard.

2- ***Ready Mixed Concrete***: Concrete can be mixed in ready mix plant where concrete is proportioned and mixed off the jobsite and is transported to the jobsite in a freshly mixed state.

3- ***Mobile Batcher Mixed Concrete***: Mobile batcher mixers are special trucks that proportion each batch by volume and continuously mix concrete as the aggregates, water, and cement are continuously fed into the mixer.

CONCRETE MIX DESIGN
CALCULATION BY ABSOLUTE VOLUME METHOD

REQUIRED INFORMATION

Specific gravity of Cement ______________________________

Specific gravity of Fine aggregate ______________________________

Specific gravity of Coarse Aggregate ______________________________

Dry-rodded unit weight of Coarse Aggregate ______________________________

Fineness Modulus of Fine Aggregate ______________________________

Maximum Size of Aggregate ______________________________

☐ Absorption or ☐ Free Moisture content of *Fine Aggregate* ______________

☐ Absorption or ☐ Free Moisture content of *Coarse Aggregate* ______________

CONCRETE REQUIREMENTS

Strength @ 28 days ______________ Non-air-entrained

Slump ______________

FROM TABLES OR CHARTS

Water Cement Ratio ______________

Strength ______________

Exposure Conditions ______________

Weight of Water per cu. yd ______________

Entrapped-Entrained- Air ______________

Volume Calculations (Saturated Surface-Dry Aggregate, SSD)

INGREDIENTS	WEIGHT(LBS)	VOLUME (cu ft/ cu yd)
Cement	______________	______________
Water	______________	______________
Coarse Aggregate	______________	______________
Air	______________	______________
Fine Aggregate	______________	______________
TOTAL		

Adjustments for Surface Water

INGREDIENTS	CORRECTED WT. (LBS)
Coarse Aggregate	______________
Fine Aggregate	______________
Water	______________
Cement	______________
TOTAL	

$$Unit\ weight = \frac{Total\ weight}{Volume} = \_\_\_\_\_\_\_\_$$

CONCRETE MIX DESIGN
CALCULATION BY ABSOLUTE VOLUME METHOD

REQUIRED INFORMATION

Specific gravity of Cement ____________________

Specific gravity of Fine aggregate ____________________

Specific gravity of Coarse Aggregate ____________________

Dry-rodded unit weight of Coarse Aggregate ____________________

Fineness Modulus of Fine Aggregate ____________________

Maximum Size of Aggregate ____________________

☐ Absorption or ☐ Free Moisture content of *Fine Aggregate* ____________

☐ Absorption or ☐ Free Moisture content of *Coarse Aggregate* ____________

CONCRETE REQUIREMENTS

Strength @ 28 days ____________ Non-air-entrained

Slump ____________

FROM TABLES OR CHARTS

Water Cement Ratio ____________

Strength ____________

Exposure Conditions ____________

Weight of Water per cu. yd ____________

Entrapped-Entrained- Air ____________

Volume Calculations (Saturated Surface-Dry Aggregate, SSD)

INGREDIENTS	WEIGHT(LBS)	VOLUME (cu ft/ cu yd)
Cement	____________	____________
Water	____________	____________
Coarse Aggregate	____________	____________
Air	____________	____________
Fine Aggregate	____________	____________
TOTAL		

Adjustments for Surface Water

INGREDIENTS	CORRECTED WT. (LBS)
Coarse Aggregate	____________
Fine Aggregate	____________
Water	____________
Cement	____________
TOTAL	

$$Unit\ weight = \frac{Total\ weight}{Volume} = \_\_\_\_\_\_\_\_$$

CONCRETE MIX DESIGN
CALCULATION BY ABSOLUTE VOLUME METHOD

REQUIRED INFORMATION

Specific gravity of Cement ______________________

Specific gravity of Fine aggregate ______________________

Specific gravity of Coarse Aggregate ______________________

Dry-rodded unit weight of Coarse Aggregate ______________________

Fineness Modulus of Fine Aggregate ______________________

Maximum Size of Aggregate ______________________

☐ Absorption or ☐ Free Moisture content of *Fine Aggregate* ______________

☐ Absorption or ☐ Free Moisture content of *Coarse Aggregate* ______________

CONCRETE REQUIREMENTS

Strength @ 28 days ______________ Non-air-entrained

Slump ______________

FROM TABLES OR CHARTS

Water Cement Ratio ______________

Strength ______________

Exposure Conditions ______________

Weight of Water per cu. yd ______________

Entrapped-Entrained- Air ______________

Volume Calculations (Saturated Surface-Dry Aggregate, SSD)

INGREDIENTS	WEIGHT(LBS)	VOLUME (cu ft/ cu yd)
Cement	______	______
Water	______	______
Coarse Aggregate	______	______
Air	______	______
Fine Aggregate	______	______
TOTAL		

Adjustments for Surface Water

INGREDIENTS	CORRECTED WT. (LBS)
Coarse Aggregate	______
Fine Aggregate	______
Water	______
Cement	______
TOTAL	

$$Unit\ weight = \frac{Total\ weight}{Volume} = \_\_\_\_\_\_$$

Table V-7 Units and Constants used in Concrete Mix Design

Units and Constants At a Glance

WATER

8.33 lb / gallon
62.4 lb / cubic feet
7.48 gallons / cubic feet

PORTLAND CEMENT (CONVENTIONALLY USED VALUES)

94 lb / sack
94 lb / cubic foot (bulk- loose)
1 cubic feet / sack (bulk-loose)
Specific Gravity 3.15 (unless given)

CONCRETE

Normal Concrete Unit Weight ....145 –150 lb / cubic foot
Lightweight Concrete Unit Weight (Range) .... 105 – 120 lb / cubic foot

AGGREGATE

Fine Aggregate (Sand) Specific Gravity.. 2.50 – 2.65
Coarse Aggregate (Gravel) Specific Gravity.......... ....2.55 – 2.70

FLY ASH

Class F .... 2.40 – 2.60
Class C .... 2.20 – 2.50

STANDARD UNITS AND MEASURES

1 Cubic yard = 27 cubic feet
1 ton = 2,000 lb

V-D.6 Transporting and Handling Concrete:

The uniformity of the concrete mix must not be disturbed by the process of transporting and handling concrete at the jobsite. To achieve the predetermined properties of fresh and hardened concrete, concrete should be transport to the jobsite without delays and segregation. Delays may cause concrete to harden which make the process of consolidation difficult with the possibility of forming honeycomb of the finished product. Segregation occurs when coarse aggregate separates from the sand-cement mortar. Segregation will produce nonuniform mix where the solids settle at the bottom of the member which will reduce the strength of that member.

V-D.7 Placing and Finishing Concrete:

Preparation and Depositing Concrete

Before placing concrete, all surfaces must be clean and free from any foreign materials including debris, oil, ice and snow. All surfaces must be moistened especially in hot weather and when concrete to be placed on subgrades. All reinforcing steel bars and other embedded items should be secured in their place and free from rust, oil and scale.

Concrete should be placed in a manner to eliminate or minimize segregation. It should not be dumped in a form of piles and then leveled, but rather it should be placed continuously in horizontal layers.

V-D.8 Consolidating of Concrete

The fresh placed concrete should be well compacted to eliminate the entrapped air, rock pockets and honeycomb. Consolidation could be done by hand or mechanical methods depending on the workability of concrete, shape and size of formwork and spacing between reinforcing bars. Workable concrete can be consolidated easily by hand rodding while stiff mixtures with low water-cement ratios must be consolidated using mechanical methods such as vibration. Vibration is the most widely technique for consolidating concrete. Vibration is either internally or externally. Both types of vibration are characterized by their frequency of vibration expressed in vibrations per minute (vpm) and by the amplitude of vibration. Internal vibration is normally used to consolidate concrete in columns, beams, and slabs. External vibration is accomplished by using vibrating tables, surface vibrators such as vibratory screeds, plate vibrators or vibratory hand floats or trowels. External vibration is used in case of thin and heavily reinforced concrete members. Also it is used for stiff mixed where internal vibrators cannot be used.

V-D.9 Curing Concrete

Curing is the process of controlling temperature and humidity of concrete for prompting the hydration of cement. The objective of curing is to keep concrete saturated, or as nearly saturated as possible, until the chemical reaction between water and cement is completed. The strength of concrete depends on the success of curing process. Concrete strength increases with age as long as moisture and a favorable temperature are present for hydration of cement Loss of water will reduce the hydration process and also cause the concrete to shrink creating tensile stresses which can cause cracking of the surface of the finished product.

Curing Methods

In order to keep concrete moist at a favorable temperature, one or combination of the following methods can be used:

1- Maintain the presence of mixing water in the concrete especially during the early hardening process. This can be accomplished by ponding or immersion of flat surfaces as pavements, and floors. Spraying or fogging are another techniques to keep concrete moist. Saturated fabric coverings as cotton mats, burlap, and rugs could also be used to keep concrete moist. Selecting one technique over the others depend on the size and shape of concrete, availability of materials and economics.

2- Prevent the loss of mixing water by sealing the surface. Impervious paper, plastic sheets and membrane -forming curing compounds are the commonly methods used to prevent loss of mixing water.

3- Accelerate the rate of strength gain which can be accomplished by providing heat and moisture. The sources for heat and moisture are live steam, heating coils, or electrically heated forms or pads.

V-D.10 Hot-Weather Concreting

Concrete should be mixed, transport, placed and finished at a temperature that has no effect on it properties in the fresh or hardened state. Hot weather will increase water demand, decrease the slump and increase the rate of setting. To counteract the effect of hot weather by adding more water to concrete will decrease strength and durability of concrete in the hardened state. Previous studies showed that increasing concrete temperature from 50° F to 100° F would require an additional 33 lb of water per cubic yard to maintain the same 3 in slump that mix was designed for. This additional amount of water would decrease the compressive strength by about 12% to 15 %. This loss in the strength could disqualify this concrete mix.

Concrete having a temperature between 50° F and 60° F is desirable but not always practical. Many specifications require that concrete to be placed when its temperature less than 85° F or 90° F.

V-D.11 Cold -Weather Concreting

Concrete temperature has a direct effect on the strength and workability of concrete. Therefore, in cold weather all necessary precautions must be taken to ensure the safe and desirable temperature for concrete during mixing, placing, finishing and curing. According to ACI 306 C cold weather is defined as "a period when for more than 3 consecutive days the mean daily temperature drops below 40° F."

At low temperatures, the rate of hydration of cement decreases which delay the strength gain of concrete with time. Concrete should be delivered at the proper temperature and the temperature of the forms, reinforcing steel and the ground must be considered. Concrete should not be placed on frozen concrete or on frozen ground. The temperature at the time of casting can be raised by heating the ingredients of the mix. Water can be heated easily, but it is not advisable to exceed a temperature of 140° to 180° F as flash set of the cement may result Aggregate can also be heated by passing steam through coils.

Sample Problem V-3: Weight & Volume of Concrete	
Given: Given a water cement ratio of 0.54 and 340 pound of water per cubic yard.	***Find:*** The volume of the cement needed for a 10 cubic yard concrete mix is most nearly: (A) (B) (C) (D)
Solution:	

Sample Problem V-4: Sacks of Cement	
Given: Mr. Smith wants to pour a concrete slab 5-in thick by 14 ft by 5 ft. Water cement ratio is 6 gallon of water per sack of cement and 325 pounds of water per cubic yard. Assume 6 % waste of concrete.	***Find:*** The number of sacks of cement required for this slab is most nearly: (A) (B) (C) (D)
Solution:	

Sample Problem V-5: Sacks of Cement Calculations

Given: A mix is designed as 1:1.9:2.8 by weight. The water cement ratio is 7 gallons of water per sack of cement.	**Find:** The concrete yield in cubic feet is most nearly: (A) (B) (C) (D)

Solution:

V- 6 10 cubic feet of 1: 2.5: 4 (by weight) concrete are required for a potion of a highway in northern US. The ingredients have the following properties:

Ingredient	SSD density (pcf)	Moisture (dry basis from SSD)
Cement	197	N/A
Fine aggregate	164	5% excess (free moisture)
Coarse aggregate	168	2% deficit (absorption)

5.5 gallons of water are to be used per sack and the mixture have 6% entrained air.

Sample Problem V-6: Mixing Concrete based on the Absolute Volume

Find: The weight and volume of each ingredient after the adjustment for surface water

Solution:

Chapter VI- Temporary Structures

VI- B FORMWORK:

VI-B.1 Introduction

The need for a formwork standard and increased knowledge concerning the behavior of formwork was evident from the rising number of failures, sometimes resulting in the loss of life. The first report by the committee, based on a survey of current practices in the United States and Canada, was published in the ACE JOURNAL in June 1957. The second committee report was published in the ACI JOURNAL in August 1958.

The first standard was ACI 347-63. Subsequent revisions were ACI 347-68 and ACI 347-78. Two subsequent revisions, AC1347R- 88 and ACI 347R-94, were committee reports because of changes in the ACI policy.

Examples of Formwork for Concrete Structures:

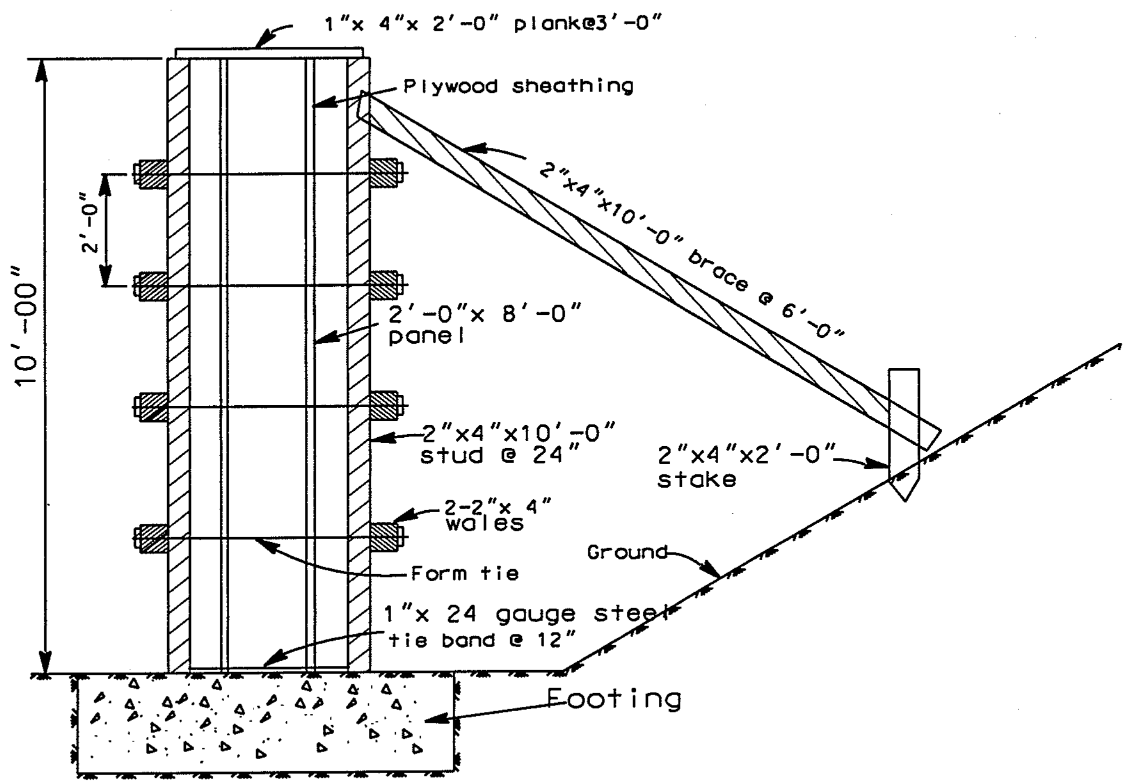


Figure VI-1 Typical Form for Concrete Wall

§ 1.2 ACI 347-04: Definitions

Backshores: shores placed snugly under a concrete slab or structural member after the original formwork and shores have been removed from a small area at a time, without allowing the slab or member to deflect; thus, the slab or other member does not yet support its own weight or existing construction loads from above.

Bugholes: surface air voids: small regular or irregular cavities, usually less than 0.6 in. (15 mm) in diameter, resulting from entrapment of air bubbles in the surface of formed concrete during placement and consolidation. Also called blowholes.

Centering: specialized temporary support used in the construction of arches, shells, and space structures where the entire temporary support is lowered (struck or decentered) as a unit to avoid introduction of injurious stresses in any part of the structure.

climbing form: a form that is raised vertically for succeeding lifts of concrete in a given structure.

diagonal bracing: supplementary formwork members designed to resist lateral loads.

engineer/architect: the engineer, architect, engineering firm, architectural firm, or other agency issuing project plans and specifications for the permanent structure, administering the work under contract documents, or both.

flying forms: large prefabricated, mechanically handled sections of formwork designed for multiple reuse; frequently including supporting truss beam, or shoring assemblies completely utilized. Note: Historically, the term has been applied to floor forming systems.

Form: a temporary structure or mold for the support of concrete while it is setting and gaining sufficient strength to be self-supporting.

Formwork: total system of support for freshly placed concrete including the mold or sheathing that contacts the concrete and all supporting members, hardware, and necessary bracing.

Formwork engineer/contractor: engineer of the formwork system, contractor, or competent person in charge of designated aspects of formwork design and formwork operations.

ganged forms: large assemblies used for forming vertical surfaces; also called gang forms.

horizontal lacing: horizontal bracing members attached to shores to reduce their unsupported length, thereby increasing load capacity and stability.

Preshores: added shores placed snugly under selected panels of deck-forming system before any primary (original) shores are removed. Preshores and the panels they support remain in place until the remainder of the complete bay has been stripped and backshored, a small area at a time.

reshores: shores placed snugly under a stripped concrete slab or other structural member after the original forms and shores have been removed from a large area, requiring the new slab or structural member to deflect and support its own weight and existing construction loads to be applied before installation of the reshores.

Scaffold: a temporary elevated platform (supported or suspended) and its supporting structure used for supporting workers, tools, and materials; adjustable meal scaffolding cm be used for shoring in concrete work provided its structure has the necessary load-carrying capacity and structural integrity.

Shores: vertical or inclined support members designed to carry the weight of the formwork, concrete, and construction loads above.

Slipform: a form that is pulled or raised as concrete is placed; may move in a horizontal direction to lay concrete paving or on slopes and inverts of canals, tunnels, and siphons; or may move vertically to form walls, bins, or silos.

§ 1.3 ACI 347-04: Achieving economy in formwork

The engineer/architect can help overall economy in the structure by planning so that formwork costs are minimized. The cost of formwork in the United States can be as much as **60% of the total cost** of the completed concrete structure in place and sometimes greater. This investment requires careful thought and planning by the engineer/architect when designing and specifying the structure and by the formwork engineer/ contactor when designing and constructing the formwork.

Formwork drawings, prepared by the formwork engineer/ contractor, can identify potential problem and should give project site employees a clear picture of what is required and how to achieve it. The following guidelines show how the engineer/architect can plan the structure so that formwork economy may best be achieved:

- To simplify and permit maximum reuse of formwork, the dimensions of footings, columns, and beams should be of standard material multiples, and the number of sizes should be minimized;
- When interior columns are the same width as or smaller than the girders they support, the column form becomes a simple rectangular or square box without boxouts, and the slab form does not have to be cut out at each corner of the column
- When all beams are made one depth (beams framing into beams as well as beams framing into columns), the supporting structures for the beam forms can be carried on a level platform supported on shores;
- Considering available sizes of dressed lumber, plywood, and other ready-made formwork components and keeping beam and joist sizes constant will reduce labor time;
- The design of the structure should be based on the use of one standard depth wherever possible when commercially available forming systems, such as one - or two-way joist systems, are used;

- The structural design should be prepared simultaneously with the architectural design so that dimensions can be better coordinated. Room sizes can vary a few inches to accommodate the structural design
- The engineer/architect should consider architectural features, depressions, and openings for mechanical or electrical work when detailing the structural system, with the aim of achieving economy. Variations in the structural system caused by such items should be shown on the structural plans. Wherever possible, depressions in the tops of slabs should made without a corresponding break in to elevations of the soffits of slabs, beams, or joists;
- Embedments for attachment to or penetration through the concrete structure should be designed to minimize random penetration of the formed surface; and
- Avoid locating columns or walls, even for a few floors where they would interfere with the use of large formwork shoring units in otherwise clear bays.

VI- C FALSEWORK AND SCAFFOLDING

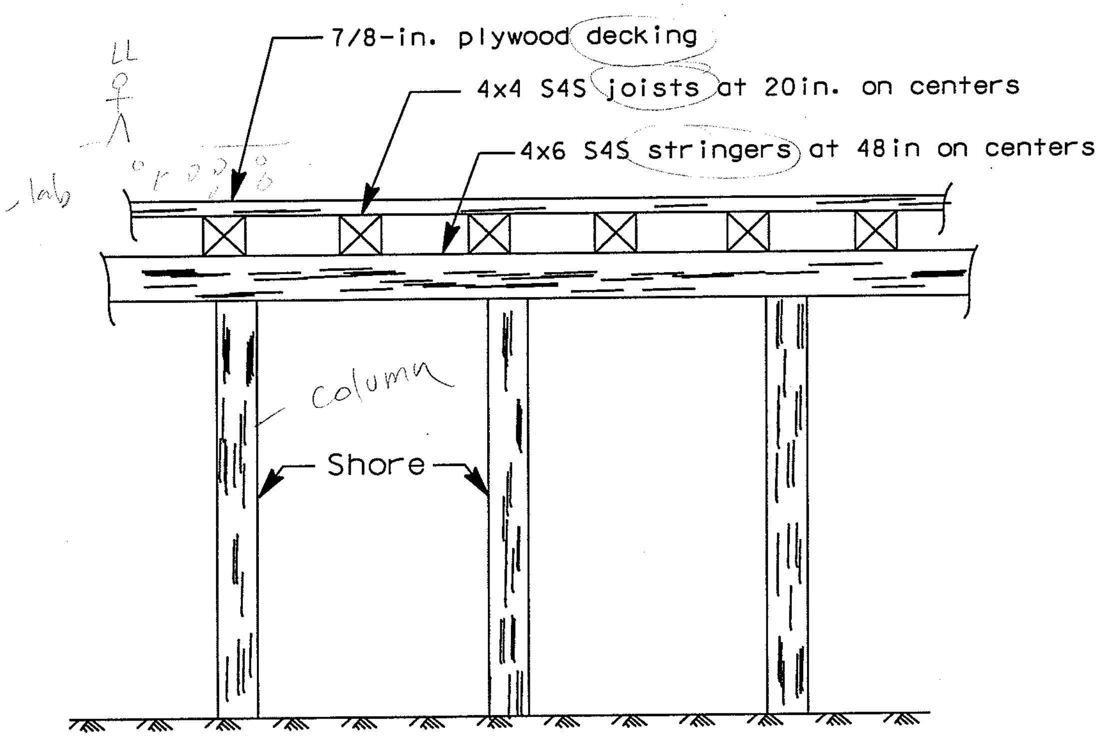


Figure VI-2 Typical Slab Framing System

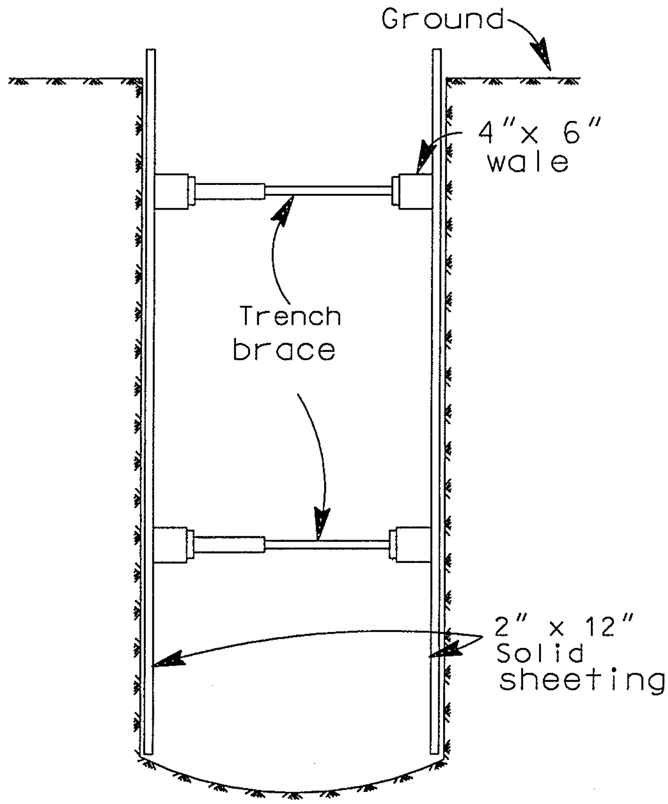


Figure VI-3 Trench Braces, Wales, and Solid Bracing

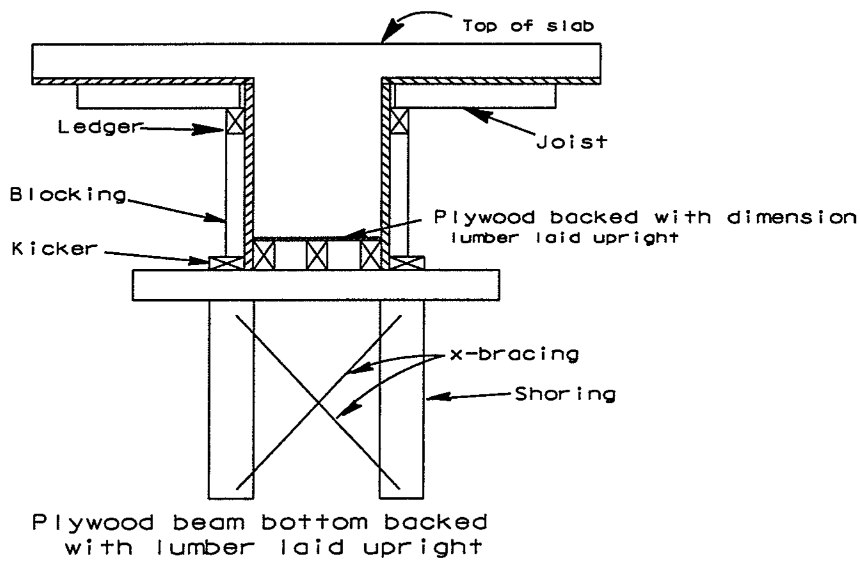


Figure VI-4 Forms for a Concrete Beam and Slab

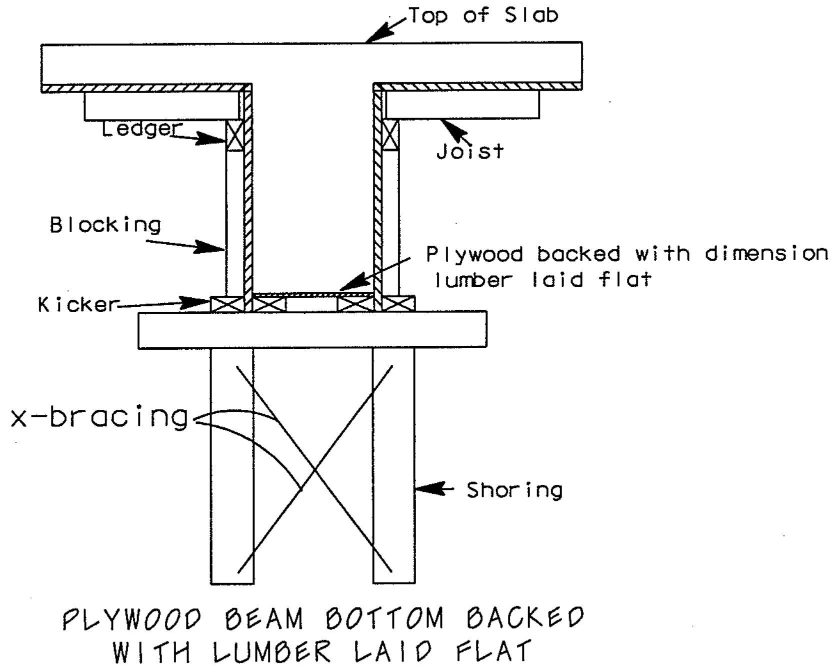


Figure VI-5 Forms for a Concrete Beam and Slab

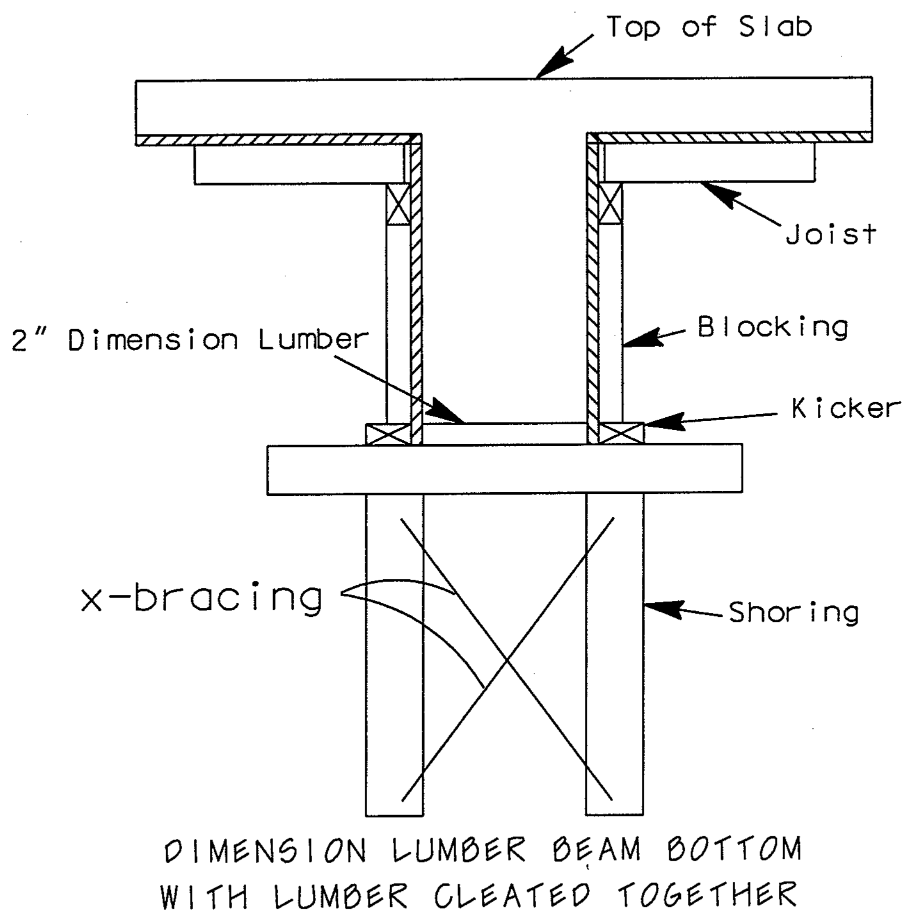


Figure VI-6 Forms for a Concrete Beam and Slab

Chapter VII- Worker Health, Safety, and Environment

VII- A OSHA REGULATIONS

The Occupational Safety and Health Administration (OSHA) has developed regulations involving trench/excavation safety. Failure to comply with these regulations can result in criminal and civil penalties.

OSHA classifies soils into four categories as follows:

Category I: Stable Rock: This is a natural solid mineral matter that can be excavated with vertical sides and remain intact while exposed. It is usually identified by a rock name such as granite or sandstone. Determining whether a deposit is of this type may be difficult unless it is known whether cracks exist and whether or not the cracks run into or away from the excavation.

Category II: Type "A" Soils: Those are cohesive soils with an unconfined compressive strength of 1.5 tons per square foot (tsf) (144 kPa) or greater. Examples of Type "A" cohesive soils are often: clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. No soil is Type A if it is fissured, is subject to vibration of any type, has previously been disturbed, is part of a sloped, layered system where the layers dip into the excavation on a slope of 4 horizontal to 1 vertical (4H:1V) or greater, or has seeping water.

Category III: Type "B" Soils: Those are cohesive soils with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa). Examples of other Type "B" soils are: angular gravel; silt; silt loam; previously disturbed soils unless otherwise classified as Type C; soils that meet the unconfined compressive strength or cementation requirements of Type "A" soils but are fissured or subject to vibration; dry unstable rock; and layered systems sloping into the trench at a slope less than 4H:1V (only if the material would be classified as a Type B soil).

Category IV: Type "C" Soils: Those are cohesive soils with an unconfined compressive strength of 0.5 tsf (48 kPa) or less. Other Type "C" soils include granular soils such as gravel, sand and loamy sand, submerged soil, soil from which water is freely seeping, and submerged rock that is not stable. Also included in this classification is material in a sloped, layered system where the layers dip into the excavation or have a slope of four horizontal to one vertical (4H : 1 V) or greater.

LAYERED GEOLOGICAL STRATA. Where soils are configured in layers, i.e., where a layered geologic structure exists, the soil must be classified on the basis of the soil classification of the weakest soil layer. Each layer may be classified individually if a more stable layer lies below a less stable layer, i.e., where a Type C soil rests on top of stable rock. *(www.osha.gov/SLTC/trenchingexcavatiorVinclex.html)*

Once the soil type has been identified, the side slope of an excavation/trench can be determined. The following table presents the allowable slope for excavations in the different soil types. These slopes apply to trenches greater than or equal to 5 feet deep, up to 20 feet deep. In lieu of sloping excavations, trench boxes or benching of the cut may be utilized. Consult OSHA website for details on these methods.

Table 7-1 OSHA Maximum Allowable Side Slope (OSHA Table B-1)

Soil Type	Maximum Allowable Side Slope (H:V Ratio) for excavation < 20 feet deep(3)	Slope Angle(1) (Repose angle), degrees
Rock	0: 1 (Vertical)	90
Type A(2) (depth < 12 ft)	½ H : 1V	63
Type A	¾ H : 1V	53
Type B	1H:1V	45
Type C	1 ½ H : 1V	34

(3) Maximum allowable angles expressed in degrees from horizontal. Angles have been rounded off.
(4) A short term maximum allowable slope of ½ H: 1V is allowed in excavations in Type A soil that are 12 feet (3.67 m) or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet (3.67 m) in depth shall be ¾ H: 1V (53°).
(3) Shoring or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.

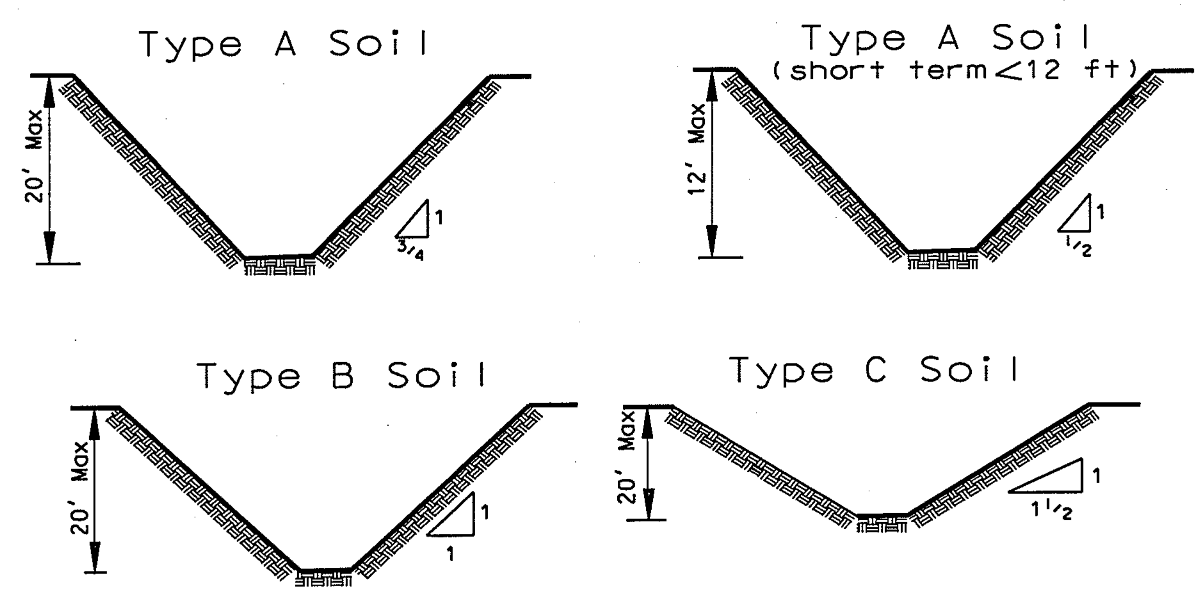


Figure VII-1 Allowable Side Slopes for Excavation < 20 feet

Additional requirements are that ***egress ladders be provided every 25 feet in trenches 4 feet or more in depth***, and that temporary soil be placed no closer than ***2 feet*** from an excavation as shown in the following figure:

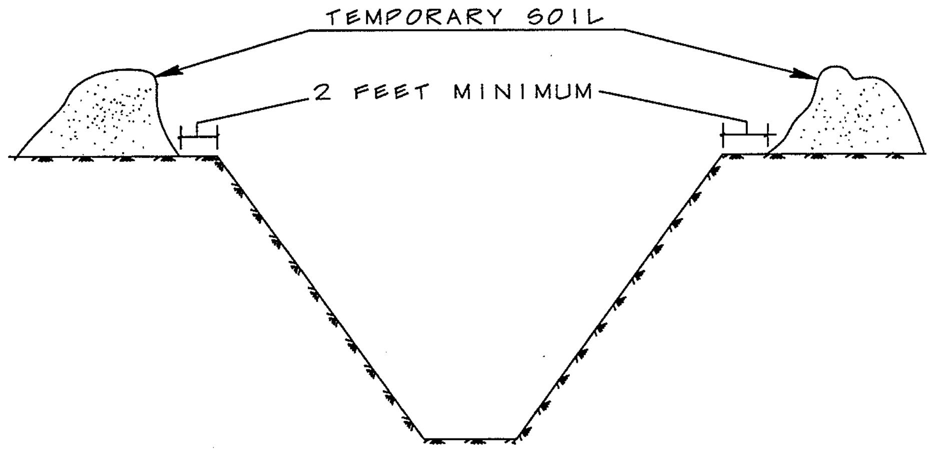


Figure VII-2 Temporary Soil Placement Next to an Excavation

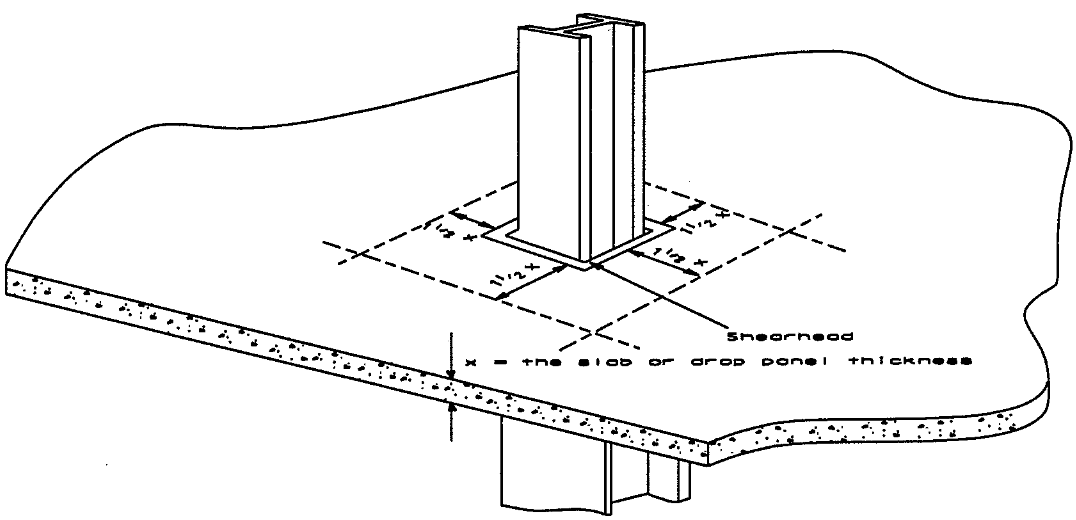


Figure VII-3 Column Head Area (Bottom)

VII- B SAFETY MANAGEMENT

VII-C: SAFETY STATISTICS (INCIDENT RATE, EMR)

VII-C.1 INCIDENT RATES

Incident rates are an indication of how many incidents have occurred, or how severe they were. They are measurements only of past performance or lagging indicators. Incident rates are also only one of many items that can be used for measuring performance. There are many items that should be used to measure performance, most of which are positive in nature; incident rates tend to be viewed as an indication of something that is wrong with a safety system, rather than what is positive or right about the system. In spite of this, for many companies, incident rates remain the primary indicator of safety performance measurement. This is primarily because incident rates are fairly easy to figure out, and can be easily compared between one company and another, and are used throughout industry.

The most difficult part about incident rates is that the five major types of rates are easily confused with one another. The most common rate used is the Recordable Incident Rate. This is commonly called either the "total case incident rate" or just the "incident rate". The "Lost Time Case Rate" (LTC) is the second most commonly used. The "Lost Workday Rate" and "Severity Rate" are primarily used only in larger companies that have a larger number of Lost Time Cases. The newest incident rate type is called the DART or "Days Away/Restricted or Transfer Rate".

VII-C.2 EFINITIONS

DART (Days Away/Restricted or Transfer Rate): A mathematical calculation that describes the number of recordable injuries and illnesses per 100 full-time employees that resulted in days away from work, restricted work activity and/or job transfer that a company has experienced in any given time frame.

LOST TIME CASE: Any occupational injury or illness which results in an employee being unable to work a full assigned work shift. (A fatality is not considered a LTC) Lost time cases result when there are no reasonable circumstances under which the injured employee could return to meaningful work. It is assumed that if an employee could work, even if it is not their normally assigned duties, alternate tasks that accommodate the restrictive nature of an injury would be assigned to the employee. In this situation, the days are recorded as *RESTRICTED WORK DAYS*, rather than Lost Work Days. (Note that working from home, on a computer or at other assigned tasks, is not considered restricted work activity unless the employee would normally perform this function from home as part of their assigned work. Situations like this would be considered lost work days. The incident, if employees can report to their normal workplace, and they can be assigned and complete productive tasks to benefit the company, can be considered restricted work days, rather than lost work days.)

***LOST WORKDAY RATE*:** a mathematical calculation that describes the number of lost work days per 100 full-time employees in any given time frame.

***LOST TIME CASE RATE*:** a mathematical calculation that describes the number of lost time cases per 100 full-time employees in any given time frame.

***OCCUPATIONAL INJURY*:** Any injury (including a fatality) which results from a work-related incident or exposure involving a single incident. Examples are:

- Thermal and chemical burns
- Cuts, abrasions and punctures
- Fractures or crushing injuries
- Respiratory irritations
- Instantaneous hearing loss
- Amputations
- Sprains or strains
- Broken bones

***OCCUPATION ILLNESS*:** Any abnormal condition or disorder (other than an injury) that resulted from a work-related exposure to a biological, chemical or physical agent. These include both acute and chronic illnesses or diseases that may be caused by inhalation, absorption, ingestion or direct contact. Some examples are:

CATEGORY	EXAMPLES
Skin diseases	Dermatitis, eczema or rash that is caused by plants, oil, acne, chromic ulcers, chemical contact, or inflammation.
Lung diseases	Silicosis, asbestosis, pneumoconiosis, or other similar disorder
Respiratory Conditions	Pneumonitis, rhinitis or acute congestion caused by work related exposures to chemicals, dusts, gases or fumes
Poisoning	Exposure to lead, mercury, cadmium, arsenic or other heavy metals; inhalation of carbon monoxide, hydrogen sulfide or other gases; exposure to benzene compounds, carbon tetrachloride, or other organic solvents; exposure to toxic levels of insecticide sprays; and exposure to other chemicals such as formaldehyde, plastics or other resins.
Physical disorders	Heatstroke, sunstroke, heat exhaustion, freezing, frostbite and other environmental effects; radiation exposure; and effects form non-ionizing radiation sources such as welding flashes, UV rays, microwaves and sunburn

Repetitive Trauma	Carpal Tunnel syndrome, synovitis, tendonitis and other conditions related to repeated motion, vibration or pressure; and noise induced hearing loss.
Other	Anthrax, infectious hepatitis, tumors, food poisoning, and gradual hearing loss

RECORDABLE INCIDENTS: Recordable incidents include all work related deaths, illnesses, and injuries which result in a loss of consciousness, restriction of work or motion, permanent transfer to another job within the company, or that require some type of medical treatment or first-aid. Companies with 10 or more employees need to report their incident rates, types of incidents and lost/restricted work days to OSHA every year. Recordable incidents are incidents that resulted from an exposure or event in the workplace and that required some type of medical treatment or first-aid.

Incidents are not recordable if the employee has symptoms that merely surfaced while at work but were the result of a non-work related event or exposure. For example, a cold or an infection from a cut that was received at home is not recordable. Additionally, "activities of daily living" are not normally recordable. For example, a heart attack is generally not considered a recordable injury, unless it was caused by a singular event or exposure at work that caused the attack. For more information on what is and is not recordable, the OSHA website (http://www.osha.gov/recordkeeping/index.html) has numerous resources that will guide a company through the recordability determination. The OSHA website also has forms and guidelines to assist a company in the proper reporting format.

SEVERITY RATE: a mathematical calculation that describes the number of lost days experienced as compared to the number of incidents experienced.

TOTAL INCIDENT RATE: a mathematical calculation that describes the number of recordable incident that a company experiences per 100 full-time employees in any given time frame.

WORK RELATED: Work relationship is established with the injury or illness results from an event or exposure in the work environment. The work environment is normally considered the company premises, or another location where the employee is present as a condition of employment (i.e. a construction site, or customer location). Driving to or from work is not normally considered work-related, unless the company requires the employee to drive or be transported to a specific location for a specific business purpose. The following flowchart is a simplified version to assist companies in determining work-relationship.

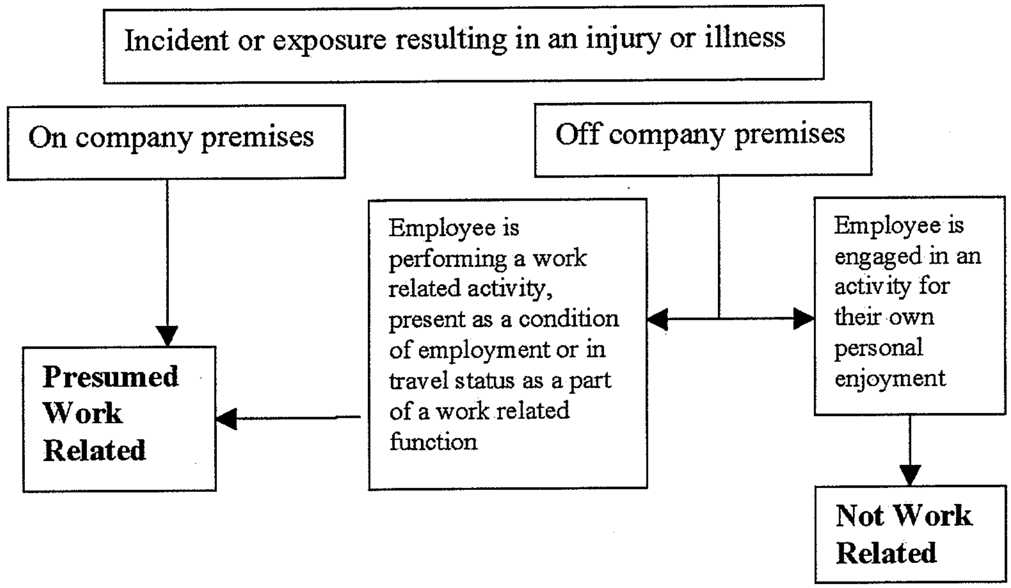


Figure C-1

VII-C.3 DETERMINING LOST WORK DAYS

Once a decision has been made that an injury or illness should be considered as a Lost Time Case (LTC), the number of days charged to that case is the number of days an employee lost work because of the incident. Days do NOT have to be consecutive. For example, if an employee breaks their leg on a Monday, and loses the rest of that day plus three additional days of work, then the employee comes back to work the following Friday and is given restricted or limited work tasks, and then loses another two days when their cast is removed, the total number of lost days would be five. The day the injury or illness occurred is not counted as a lost work day. For incidents that have lost time occurring over a longer period days is capped at 180 days of time, weekends are counted as working days, and the number of lost

CALCULATING RATES:

OSHA has established specific mathematic calculations that enable any company to report their recordable incident rates, lost time rates and severity rates, so that they are comparable across any industry or group. The standard base rate of calculation is based on a rate of 200,000 labor hours. This number equates to 100 employees, who work 40 hours per week, and who work 50 weeks per year. Using this standardized base rate, any company can calculate their rate(s) and get a percentage per 100 employees.

OSHA Recordable Incident Rate

The OSHA Recordable Incident Rate (or Incident Rate = IR) is calculated by multiplying the number of recordable cases by 200,000, and then dividing that number by the number of labor hours at the company.

$$IR = \frac{\text{Number of injuries and illnesses, or number of lost workdays} \times 200{,}000}{\text{Number of employee labor hours worked}} = \frac{N \times 200{,}000}{EH} \qquad (7\text{-}1)$$

Where:

N = Number of injuries and illnesses, or number of lost workdays

EH = Total hours worked by all employees during a month, a quarter, or a fiscal year

For example, a company has 17 full-time employees and 3 part-time employees that each work 20 hours per week. This equates to 28,400 labor hours each year. If the company experienced 2 recordable injuries, then the formula works like this:

$$IR = \frac{2 \times 200{,}000}{28{,}400} = 14.08$$

What is now known is that for every 100 employees, 14.08 employees have been involved in a recordable injury or illness.

Please note that smaller companies that experience recordable incidents will most likely have high incident rates, or the incident rates will fluctuate significantly from year to year. This is because of the small number of employees (and hence the lower number of labor hours worked) at the company. Calculations are more meaningful at larger companies that have a higher labor hour count.

Lost Time Case Rate

The Lost Time Case Rate (LTC) is a similar calculation, only it uses the number of cases that contained lost work days. The calculation is made by multiplying the number of incidents that were lost time cases by 200,000 and then dividing that by the employee labor hours at the company.

$$LTC\ Rate = \frac{\text{Number of Lost Time Cases} \times 200{,}000}{\text{Number of employee labor hours worked}} \qquad (7\text{-}2)$$

Using the previous company example, assume that one of the two recordable cases had lost work days associated with the incident. The calculations would look like this:

$$LTC\ Rate = \frac{1 \times 200{,}000}{28{,}400} = 7.04$$

What is now known is that for every 100 employees, 7.04 employees have suffered lost time because of a work related injury or illness.

Lost Work Day Rate (LWD)

The Lost Work Day rate is primarily used only at larger companies. This does not preclude a small business from using this calculation in their performance system, however. The LWD rate is calculated by multiplying the total number of lost work days for the year by 200,000, then dividing that number by the number of employee labor hours at the company.

$$LWD\ Rate = \frac{Total\ number\ of\ lost\ days \times 200{,}000}{Number\ of\ employee\ labor\ hours\ worked} \qquad (7\text{- }3)$$

Using the previous company example and the broken-leg example used earlier, there were 5 lost days due to the injury. The calculations would look like this:

$$LWD\ Rate = \frac{5 \times 200{,}000}{28{,}400} = 35.21$$

What is now known is that for every 100 employees, 35.21 days were lost from work due to work related injuries or illnesses.

DART Rate (Days Away/Restricted or Job Transfer Rate)

The DART rate is relatively new to industry. This rate is calculated by adding up the number of incidents that had one or more Lost Days, one or more Restricted Days or that resulted in an employee transferring to a different job within the company, and multiplying that number by 200,000, then dividing that number by the number of employee labor hours at the company.

$$DART\ \ Rate = \frac{Total\ number\ of\ DART\ incidents \times 200{,}000}{Number\ of\ employee\ labor\ hours\ worked} \qquad (7\text{- }4)$$

Using the previous company examples, assume that the second recordable incident resulted in limited or restricted work activity that necessitated a job transfer to a different position in the company. The first was a broken leg that had only lost time associated with it (no restriction or transfer). The calculations would look like this:

$$DART\ \ Rate = \frac{2 \times 200{,}000}{28{,}400} = 14.08$$

What is now known is that for every 100 employees, 14.08 incidents resulted in lost or restricted days or job transfer due to work related injuries or illnesses.

Severity Rate

The severity rate is a calculation that gives a company an average of the number of lost days per recordable incident. Please note, that very few companies use the severity rate as a calculation, as it only provides an average. The calculation is made by dividing the total number of lost work days by the total number of recordable incidents.

$$Sverity\ Rate\,(SR) = \frac{Total\ number\ lost\ work\ days}{Total\ number\ of\ recordable\ incidents} \tag{7-5}$$

Again, using our previous company as an example, there were 5 lost work days and two recordable incidents. So, the severity rate calculation would look like this:

$$Severity\ Rate\ (SR) = \frac{5}{2} = 2.5$$

What is now known is that for every recordable incident at the company, an average of 2.5 days will be lost due to those work related injuries and illnesses.

SUMMARY

Incident rates, of various types, are used throughout industry. Rates are indications only of past performance (lagging indicators) and are not indications of what will happen in the future performance of the company (leading indicators).

Incident rates have been standardized, so that OSHA and other regulatory agencies can compare statistically significant data, and determine where industries may need additional program assistance. OSHA uses the recordable incident rates to determine where different classifications of companies (manufacturing, food processing, textiles, machine shops, etc.) compare to each other with regard to past safety performance. Although OSHA could potentially use this data for enforcement action, unless incident rates are consistently high for a small company over a number of years, they will not normally target particular industries or companies for enforcement action.

In addition to the incident rate data, additional information may be sought by OSHA to assist them in determining the most common types of injuries, and consequently assisting them in determining what types of assistance programs are needed in various industries. If OSHA or the Bureaus of Labor Statistics (BLS) contacts your company to report additional data, you must follow through with their request for information.

Experience Modification Rates (EMR)

Three sources of information provide ways for owners to evaluate the probable safety performance of prospective contractors:

- Experience modification rates for workers' compensation insurance
- OSHA incidence rates for recordable injuries and illnesses
- Contractor safety attitudes and practices

The reliability of OSHA incidence rates is solely dependent on judicious reporting by the employer, while the EMRs are established by independent rating bureaus. Although the EMR is a more objective measure than the OSHA incidence rate, there is a correlation between them. Both will indicate past safety performance.

Experience Modification Rates for Workers' Compensation Insurance

The Experience Modification Rate is a widely used indicator of a contractors past safety performance. The insurance industry has developed experience rating systems as an equitable means of determining premiums for workers' compensation insurance. These rating systems consider the average workers' compensation losses for a given firm's type of work and amount of payroll and predict the dollar amount of expected losses to be paid by that employer in a designated rating period, usually three years. Rating is based on comparison of firms doing similar types of work, and the employer is rated against the average expected performance in each work classification. Losses incurred by the employer for the rating period are then compared to the expected losses to develop an experience rating.

Workers' compensation insurance premiums for a contractor are adjusted by this rate, which is called the experience modification rate (EMR). Lower rates, meaning that fewer or less severe accidents had occurred than were expected, result in lower insurance costs. A contactor's EMR is adjusted annually by using the rate for the first three of the last four years.

OSHA Incidence Rates

The Occupational Safety and Health Act (1970) requires employers to record and report accident information on Occupational Injuries and Illnesses Annual Survey Form No. 300 & 300A. The employer must retain completed forms for five years.

Information available from a contractor's OSHA Form No. 300 & 300A includes:

- Number of fatalities
- Number of injuries and illnesses involving lost workdays
- Number of injuries and illnesses involving restricted workdays
- Number of days away from work
- Number of days of restricted work activity

- Number of injuries and illnesses without lost workdays

Sample Problem VI-1: OSHA Incident Rate

Given: A construction company has 750,000 employee hours worked with the following safety record:

Incident Category	No. of Incidents
Minor injuries (first aid only)	10
Medical-only injuries (no lost time or light duty	4
Medical injuries resulting in "light duty" restrictions	3
Lost-time injuries	5

Find: The OSHA Incident Rate is most nearly:

A) 1.33
B) 2.13
C) 3.20
D) 5.86

Solution:

The term incident rate means the number of injuries and illness, or lost workdays, per 100 full-time workers.
The rate is given by the following equation:

$$IR = \frac{N \times 200{,}000}{EH}$$

Where:

IR = Incident Rate
N = Number of injuries and illness, or number of lost workdays
EH = Total hours worked by employees during a month, a quarter, or a fiscal year
200,000 = Base for 100 full-time equivalent workers (40 hrs/week, 50 weeks per year)

$$IR = \frac{(4+3+5) \times 200{,}000}{750{,}000} = 3.2$$

Answer: C$\Leftarrow$

Chapter VII: Other Topics

VIII- A GROUNDWATER AND WELL FIELDS

III-A.1 Groundwater control including drainage, construction dewatering

VIII- B SUBSURFACE EXPLORATION AND SAMPLING

VIII-B.1 Drilling and sampling procedures

VIII- C EARTH RETAINING STRUCTURES

VIII-C.1 Mechanically stabilized earth wall

VIII-C.2 Soil and rock anchors

VIII- D DEEP FOUNDATIONS

The foundations required to support a specific structure are selected based on many criteria such the magnitude and type of the load (s), the soil characteristics at the site, method of construction, availability of the materials at the site. Generally speaking foundations could be classified as:

8.4.1- <u>Shallow foundations such as footings, and rafts</u>:

A footing is an enlargement of a column or wall for the purpose of transmitting the load to the subsoil at a pressure suited to the properties of the soil. A footing that supports a single column is known as an individual column footing, an isolated footing or a spread footing. The footing beneath a wall is known as a wall footing or a continuous footing. If a footing supports several columns, it is called a combined footing. A particular form of combined footing commonly used if one of the columns supports an exterior wall is a cantilever footing.

A raft or mat foundation is a combined footing that covers the entire area beneath a structure and supports all walls and columns. Whenever the building loads are so heavy or the allowable soil pressure so small that individual footings would cover more than about half the building area, a raft foundation is likely to be more economical than footings.

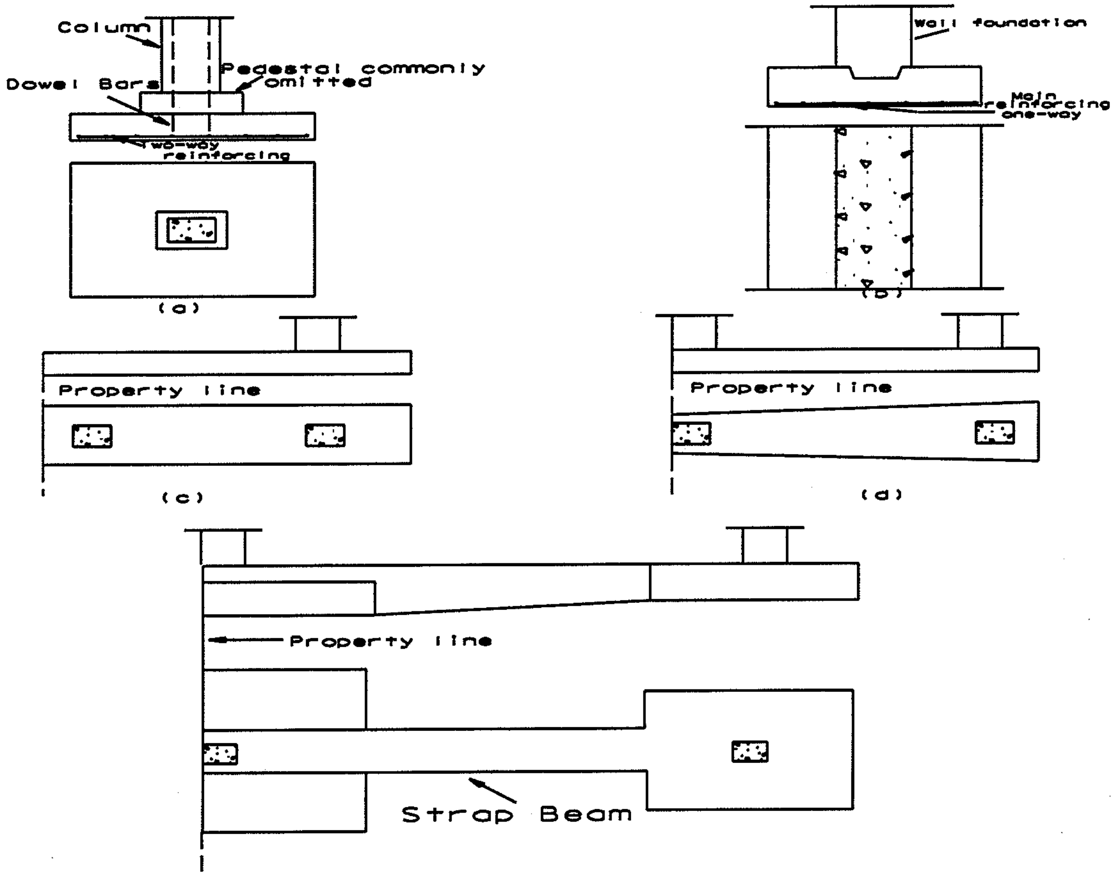


Figure 8-1 Types of Shallow Foundations

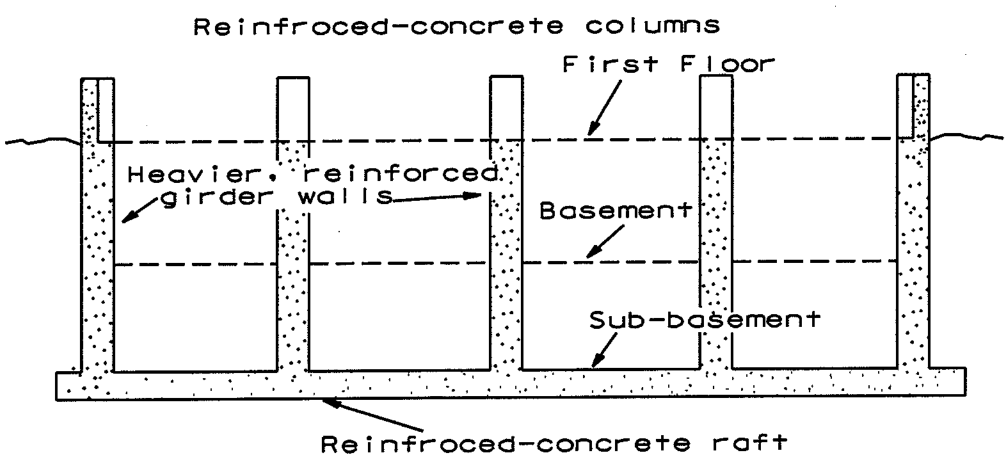


Figure 8-2 Mat Foundation

8.4.2- <u>Deep foundations such as piles, piers and caissons:</u>

The function of pier and caisson foundations is to <u>enable structural loads to be taken down through deep layers of weak soil on to a firmer stratum which will give adequate support in end bearing and resistance to lateral loads</u>. Pier and caisson foundations are also used in river and maritime construction to enable foundations to be taken below zones of soil affected by scour. They fulfill a similar function to piled foundations, the main difference being in the method of construction. In the strictest sense of the word, a pier is a heavy structural member acting as a massive strut, for example piers supporting a bridge over a waterway, or the supports to the heavy gate structures of a barrage or spillway. However, the term `pier foundation' is widely used to describe a pad foundation and the buried column above it which are constructed in situ in a deep -hich the foundation structure is built above ground level and sunk to the required founding level as a single unit.

The following definitions are given:

Caisson: a structure which is sunk through ground or water for the purpose of excavating and placing the foundation at the prescribed depth and which subsequently becomes an integral part of the permanent work.
Caisson, box: a caisson which is closed at the bottom but open to the atmosphere at the top.
Caisson, open: a caisson open both at the top and the bottom.
Caisson, pneumatic: a caisson with a working chamber in which the air is maintained above atmospheric pressure to prevent the entry of water into the excavation.
Monolith: an open caisson of heavy mass concrete or masonry construction, containing one or more wells for excavation.

Calculations to determine allowable bearing pressures and resistance to lateral and uplift loads are common both to pier and caisson foundations

Piles are relatively long and slender members used to transmit foundation loads through soil strata of low bearing capacity to deeper soil or rock strata having a high bearing capacity. They are also used in normal ground conditions to resist heavy uplift forces or in poor soil conditions to resist horizontal loads. Piles are a convenient method of foundation construction for works over water, such as bridge piers. Sheet piles perform an entirely different function; they are used as supporting members to earth or water in cofferdams for foundation excavations or as retaining walls.

If the bearing stratum for foundation piles is a hard material such as rock or a very dense sand and gravel, the piles derive most of their carrying capacity from the resistance of the stratum at the toe of the piles. In these conditions they are called end-bearing or point-bearing piles. On the other hand, if the piles do not reach an impenetrable stratum but are driven for some distance into a penetrable soil, their carrying capacity is derived partly from end-bearing and partly from the skin friction between the embedded surface of the pile and the surrounding soil. Piles which obtain the greater part of their carrying capacity by skin friction or adhesion are called friction piles

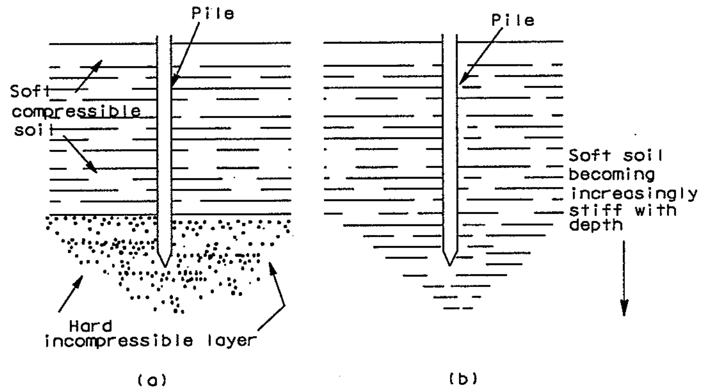


Figure VIII-3 Types of Deep (Piles) Foundations

The main types of pile in general use are as follows:

i) **Driven piles.** Preformed units, usually in timber, concrete, or steel, driven into the soil by the blows of a hammer.
ii) **Driven and cast-in-place piles.** Formed by driving a tube with a closed end into the soil, and filling the tube with concrete. The tube may or may not be withdrawn.
iii) **Jacked piles**. Steel or concrete units jacked into the soil.

iv) **Bored and cast-in-place piles**. Piles formed by boring a hole into the soil and filling it with concrete.
v) **Composite piles**. Combinations of two or more of the preceding types, or combinations of different materials in the same type of pile.

The first three of the above types are sometimes called displacement piles since the soil is displaced as the pile is driven or jacked into the ground. In all forms of bored piles, and in some forms of composite piles, the soil is first removed by boring a hole into which concrete is placed or various types of precast concrete or other proprietary units are inserted. This basic difference between displacement and non-displacement piles requires a different approach to the problems of calculating carrying capacity.

8.4.3- Behavior of Piles Under Load:

The load–settlement relationship for a single pile in a uniform soil when subjected to vertical loading to the point of failure is shown in Figure 8-4

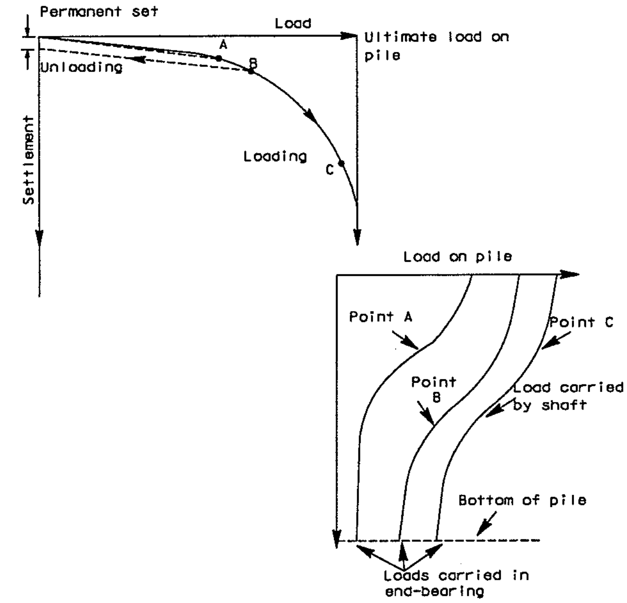


Figure VIII-4 Load –Settlement Curve and Strain-Gage Readings on Pile Shaft

At the early stages of loading, the settlement is very small and is due almost wholly to elastic movement in the pile and the surrounding soil. When the load is removed at a point such as A in Figure 8-4 the head of the pile will rebound almost to its original level. If strain gauges are embedded along the length of the pile shaft they will show that nearly the whole of the load is carried by skin friction on the upper part of the shaft. As the load is increased, the load–settlement curve steepens, and release of load from a point B will again show some elastic rebound but the head of the pile will not return to its original level, indicating that some `permanent set' has taken place. The strain gauge readings will show that the shaft has taken up an increased amount of skin friction but the load carried by the shaft will not equal the total load on the pile, indicating that some proportion of the load is now being carried in end-bearing. When the load approaches failure point C, the settlement increases rapidly with little further increase of load. The strain gauge readings show rather less load carried in skin friction than that just before failure, especially near the toe where the soil tends to flow away from the pile as failure takes place.

When piles are arranged in close-spaced groups as shown in Figure 8-5 the mechanism of failure is different from that of a single pile. The piles and the soil contained within the group act together as a single unit. A slip plane occurs along the perimeter of the group and `block failure' takes place when the group sinks and tilts as a unit. The failure load of a group is not necessarily that of a single pile multiplied by the number of piles in the group. In sand it may be more than this; in clays it is likely to be less. The `efficiency' of a pile group is taken as the ratio of the average load per pile when failure of the group occurs to the load at failure of a comparable single pile.

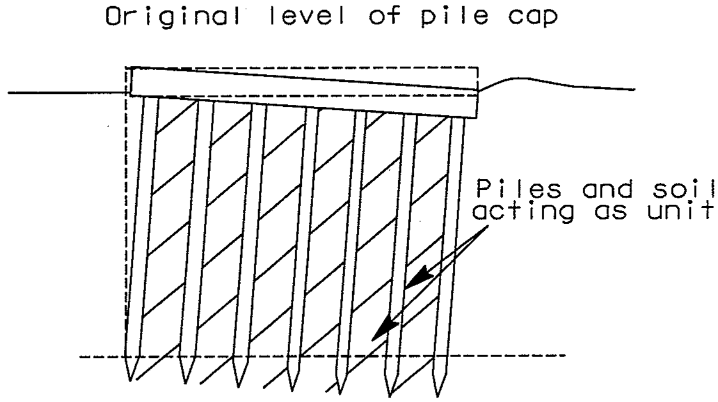


Figure VIII-5 Failure of Piles by Group Action

8.4.4- Ultimate Loads on Isolated Driven Piles:

When a pile is driven by hammering or jacking into a cohesionless soil it displaces the soil. A loose soil is compacted to a higher density by the pile and very little, if any, heave of the ground surface takes place. In very loose soils a depression will form in the ground surface around the pile due to the compaction of the soil by the pile driving. In dense cohesion-less soils very little further compaction is possible, with the result that the pile will displace the soil, and heaving of the ground surface will result. Such displacement involves shearing of the mass of soil around the pile shaft. Resistance to this is very high in a dense cohesionless soil, so that very heavy driving is required to achieve penetration of piles in dense sands or gravels. Heavy driving may lower the shearing resistance of the soil beneath the pile toe owing to degradation of angular soil particles. Thus no advantage is gained by over-driving piles in dense cohesionless soils. In any case this is undesirable because of the possible damage to the pile itself.

The shaft frictional resistance of piles in cohesionless soils is small compared with the end resistance. This is thought to be due to the formation of a ring or `shell' of compacted soil around the pile shaft, with an inner ring of soil particles in a relatively loose or `live' state. The skin friction is governed by this inner shell. In the case of straight-sided piles the soil particles may stay in a loose condition on cessation of driving. In tapered piles the ring is re-compacted with each blow of the pile hammer.

The basis of the `static' or soil mechanics method of calculating the ultimate carrying capacity of a pile is that the ultimate carrying capacity is equal to the sum of the ultimate resistance of the base of the pile and the ultimate skin friction over the embedded shaft length of the pile. This is expressed by the following equation:

$$Q_u = Q_b + Q_s \qquad (81)$$

where

Q_b = base resistance,

Q_s = shaft resistance.

Knowing the angle of shearing resistance of the soil at base level, Q_b can be calculated from Terzaghi's general equation in Section. Because the diameter of the pile is small in relation to its depth the term $0.4\gamma B N_\gamma$ can be neglected. Therefore:

Net unit base resistance, $q_{nf} = q_f = p = p_d (N_q - 1)$

Net total base resistance, $Q_b = A_b (N_q - 1)$ (82)

where p_d = effective overburden pressure at pile base level.

Pile material	δ	Value of $\overline{K}_s$	
		Low relative density	High relative density
Steel	20^o	0.5	1.0
Concrete	$3/4\phi$	1.0	2.0
Wood	$2/3\phi$	1.5	4.0

The total ultimate skin friction on the pile shaft is given by the general expression

Unit skin friction, $f = \overline{K}_s P_d \tan\delta$ (83)

Where:

$\overline{K}_s$ = an earth pressure coefficient,

δ = angle of wall friction.

VIII-D.1 Pile load test

Pile load tests are usually carried out for one or more of the following reasons:

I. To serve as a proof test to ensure that failure does not occur before a selected proof load is reached, this proof load being the minimum required factor times the working load.
II. To determine the ultimate bearing capacity as a check on the value calculated from dynamic or static approaches, or to obtain support soil data that will enable other piles to be denied.
III. To determine the load-settlement behavior of a pile, especially in the region of the anticipated working load. This data can be used to predict group settlements and settlements of other piles.
IV. To indicate the structural soundness of the pile.

The most common type of test is a compression test, although uplift, lateral-load, and even torsion-load tests are also performed.

A variety of test procedures have been developed for carrying out pile load tests; among the most common procedures for compression tests are

1- Maintained loading tests.
2- Constant-rate-of-penetration (C.R.P.) tests.
3- Method of equilibrium.

It must be emphasized that in many cases, the results of a test on a single pile cannot be extrapolated directly to predict the behavior of pile groups or other piles. As pointed out by many scientist that, the volume of soil influenced by a single pile is much less than that of a large group, so the influence of deep-seated compressible layers may not be apparent in a pile load test, although such layers may critically affect the behavior of a group. Pile load tests should therefore be accompanied by detailed site investigation to define accurately the entire soil profile.

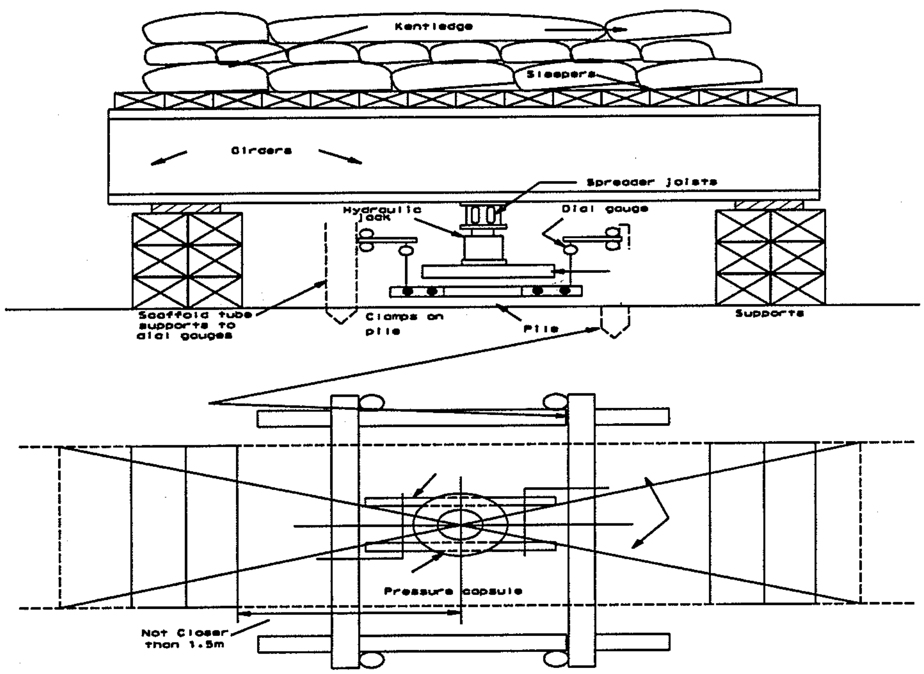


Figure 8-6 Load Test

two foundations sufficiently far from the test pile for the reaction system to be unaffected by the ground movement. In order to measure pile movements and loads at various points along the pile, displacement rods (sometimes termed "tell-tales") or strain gauges may be installed.

VIII-D.2 Pile installation

The structural features and methods of constructing piles depend on the type of piles. The following table shows types of piles according to the installation technique:

Type of Piles	Examples
I- Driven piles	Timber (round or square sections) Precast concrete (solid or hollow sections) Prestressed concrete (solid or hollow sections) Steel H-section, box and tube
II-Driven and cast-in-place piles	Withdrawable steel drive tube, end closed by concrete plug Withdrawable steel drive tube, end closed by detachable point Steel shells driven by withdrawable mandrel or drive tube Precast concrete shells driven by withdrawable mandrel
III-Bored piles	Continuous bored Cable percussion drilling Augered Large-diameter under-reamed Types incorporating precast concrete units Drilled-in tubes
IV-Composite piles	Combinations of two or more of the preceding types, or combinations of different materials in the same type of pile.

The above list might at first sight present rather a confusing or difficult choice to the engineer. However, in practice it is found that the following three main factors will narrow the choice to not more than one or two basic types:

1- location and type of structure;
2- ground conditions; and
3- durability

The final selection is then made from considerations of overall cost.

1-location and type of structure: the driven pile or the driven and cast-in-place pile in which the shell remains in position are the most favored for works over water such as piles in wharf structures or jetties. Structures on land present a wide choice of pile type, and driven and cast-in-place types are usually the cheapest for moderate loadings and unhampered site conditions. However, the proximity of existing buildings will often necessitate the selection of a type which can be installed without ground heave or vibration, e.g. some form of bored and cast-in-place pile. Jacked piles are suitable types for underpinning existing structures. Large-diameter bored piles are normally the most economical type for very heavy structures, especially in ground which can be drilled by power augers.

2- The ground conditions: influence both the choice of pile type and the technique for installing piles. For example, driven piles cannot be used economically in ground containing boulders and where ground heave would be detrimental. On the other hand, driven piles are preferred for loose water-bearing sands and gravels where compaction due to driving can develop the full potential bearing capacity of these soils. Steel H-piles, having a low ground displacement, are suitable for conditions where deep penetration is required in sands and gravels. Stiff clays favor the adoption of bored and under-reamed types. Under-reamed bases cannot be formed in cohesionless soils.

3- Durability often affects the selection of pile type. For piles driven in marine conditions, precast concrete piles may be preferred to steel piles from the aspect of resistance to corrosion. Timber piles may be rejected for marine conditions because of the risk of attack by destructive mollusc-type borers. Where soils contain sulphates or other deleterious sub-stances, piles incorporating high-quality precast concrete units are prefer-able to piles formed by placing concrete in situ in conditions where placing difficulties, such as the presence of ground water, may result in the concrete not being thoroughly compacted.

VIII-E LOADINGS

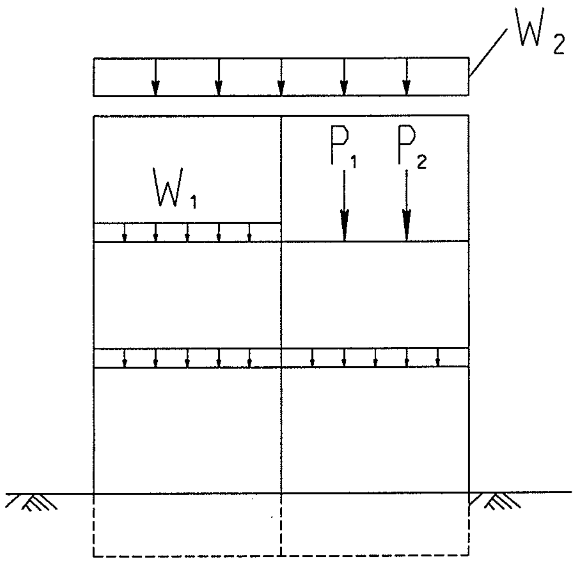
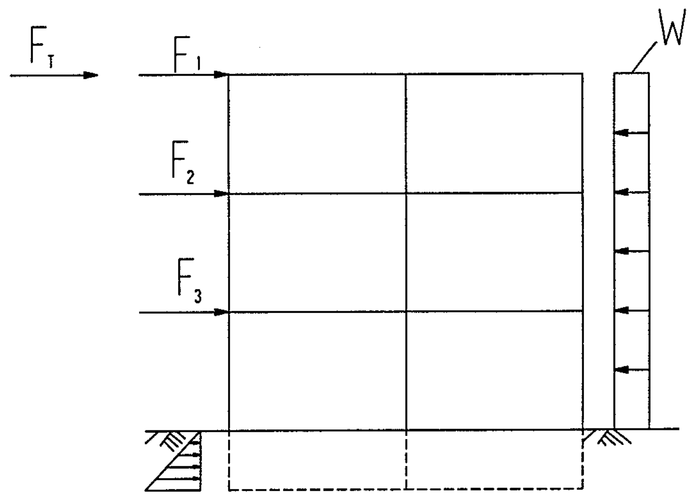
Structural loads usually are classified based on their characteristics and duration as follows:

Table VIII-1 Types of Loads

Dead loads	Live loads	Environmental Loads
➢ Constant in magnitude ➢ Remain in one position ➢ Include the weight of the structure and any fixtures that are permanently attached to it.	➢ Change in magnitude ➢ Change position ➢ Include occupancy, warehouse, construction and equipment operating loads.	➢ Change in magnitude ➢ Change position ➢ Include earthquake, rain, snow, wind, temperature loads. Also known as live loads caused by environment.

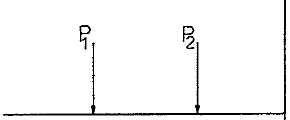
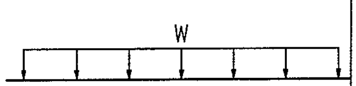
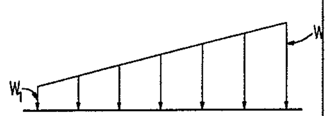
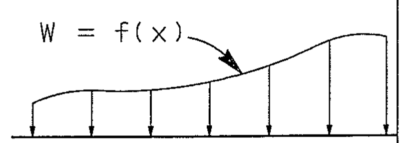
Loads can be also classified according to the direction of application as gravity and lateral loads as follows

Table VIII-2 Types of Loads

Gravity Loads	Lateral Loads
➢ All loads caused by gravity such as dead, live and snow loads.	➢ Loads acting in the lateral direction such as earthquake, wind, earth and fluids lateral pressure.
	

Also, loads could be classified according to the intensity and the area acting over as follows:

Table VIII-3 Types of Loads

Concentrated loads	Distributed Loads		
	Uniform	Linear	Nonlinear
➢Loads are assumed to be action over a point except as indicated in codes.	➢Loads are uniformly over an area or per unit length e.g. lb/ft^2 k/ft^2, lb/ft, N/m	➢Loads are vary linearly as shown below	➢ Loads are specified by a function
			

Finally, loads can be classified as service and factor loads:

Table VIII-4 Types of Loads

Service Loads	Factored Loads
➢ Used in the Allowable Stress Design (ASD). Also, known as Working Stress Design (WSD) or Service Load Design. ➢ Loads without factors (or a factor of ONE) except "0.7E" and "0.6D" in some of the load combinations equations.	➢ Used in Strength Stress Design (SD) and Load and Resistance Factor Design (LRFD). Also, known as Factored Load Design. ➢ All loads are multiplied by factors (≥ 1 or ≤ 1) depending on the type of the load. ➢ Load factor for earthquake load is ONE.

VIII-E.1 Wind loads

Wind loads and seismic or earthquake are lateral loads that caused by environmental sources. The determination of the wind loads acting on a building is often the subject of an entire course. The determination of the seismic load will be discussed in details in the following chapters.

When the code-prescribed wind design produces greater effects, the structure shall be:

1- Designed for the larger force (wind force); and

2- Detailed in accordance with the requirements and limitations prescribed by the IBC 2006/CBC 2007 seismic design provisions.

The following table shows the major differences between seismic and wind loads:

Table VIII-5 Seismic Loads Vs. Wind Loads

Seismic Loads	Wind Loads
➢ Result from the inertial response of a structure to the accelerations and displacements from the earthquake ground shaking. ➢ Tend to have somewhat unpredictable upper limits. ➢ Structures are assumed to resist the design seismic forces inelastically.	➢ Result from aerodynamic pressures applied to an exterior surface of a structure. ➢ Tend to have somewhat predictable upper limits (e.g., 100 year storm, etc.). ➢ Structures are designed to resist the design wind forces elastically.

A survey of engineering literature for the past 150 years reveals many references to structural failures caused by wind. Perhaps the most infamous of these have been bridge failures such as those of the Tay Bridge in Scotland in 1879 (which caused the deaths of 75 persons) and the Tacoma Narrows Bridge (Tacoma, Washington) in 1940. However, there were some disastrous building failures due to wind during the same period, such as the Union Carbide Building in Toronto in 1958. It is important to realize that a large percentage of building failures due to wind have occurred during their construction

Considerable research has been conducted in recent decades on the subject of wind loads. Nevertheless a great deal more study is needed, as the estimation of wind forces can by no means be classified as an exact science. The average structural designer would love to have a simple rule with which he or she could compute the magnitude of design wind loads, such as: The wind pressure is to be 20 psf for all parts of structures 100 ft or less above the ground and 30 psf for parts that are more than 100 ft above the ground.

The ASCE 7-02/05 standard provides equations with which wind pressures can be estimated for various parts of buildings. Though the use of these equations is complicated, the work is somewhat simplified with the tables and charts presented in the specification. Several of these charts and tables are shown in Appendix C of this book with the permission of the ASCE. The reader should particularly note that the information provided is for buildings of regular shape. Should domes, A-frames, or buildings with roofs sloped at angles greater than 45 degrees or buildings with unusual floor plans as H or Y shapes or others be encountered, it will be desirable to conduct wind tunnel studies.

Design Wind Speed, V

The basic wind speed to be used in design for the locality involved may be estimated from ASCE 7-02. The values provided in this figure are not applicable to mountainous areas, gorges, and other regions where unusual wind conditions may exist. For such areas special studies will have to be made. The velocities obtained are the estimated worst 3-second gust speeds in miles per hour (mph) that would occur at 33 feet above the ground surface during a 50-year period.

Importance Factor, I

The importance factor is intended to bring into the calculation of wind forces a measure of the consequences of failure. Critical buildings, such as schools and hospitals, will have a higher importance factor and therefore higher design wind forces. Buildings whose failure will have little consequence on human life, such as farm buildings, will have a lower importance factor and therefore lower design wind forces.

VIII-E.2 Snow loads

In the colder states, snow and ice loads are often quite important. One inch of snow is equivalent to approximately 0.5 psf, but it may be higher at lower elevations where snow is denser. For roof design, snow loads ranging from 10 to 40 psf are usually specified. The magnitude depends primarily on the slope of the roof, and to a lesser degree on the character of the roof surface. The larger values are used for flat roofs and the smaller values for sloped roofs. Snow tends to slide off sloped roofs, particularly those with metal or slate surfaces. A load of approximately 10 psf might be used for roofs with 45° slopes while a 40-psf load might be used for flat roofs. Studies of snowfall records in areas with severe winters may indicate the occurrence of snow loads much greater than 40 psf, with values as high as 100 psf in northern Maine.

Snow is a variable load that may cover an entire roof or only part of it. There may be drifts against walls or buildup in valleys or between parapets. Snow may slide off one roof onto a lower one. The snow may blow off one side of a sloping roof or it may crust over and remain in position even during very heavy winds.

The snow loads that are applied to a structure are dependent upon many factors, including geographic location, the pitch of the roof, sheltering, and the shape of the roof. The discussion that follows is intended to provide only an introduction to the determination of snow loads on buildings. When estimating these loads, consult ASCE 7-02 for information that is more complete.

VIII-E.3 Load paths

TRANSFER OF VERTICAL (GRAVITY) LOADS

The gravity loads are transmitted to the foundations through the floor/roof systems via the connections of the horizontal and vertical resisting elements. This is accomplished through joist, floor beams, beams, girders and columns or walls as shown in the following figures.

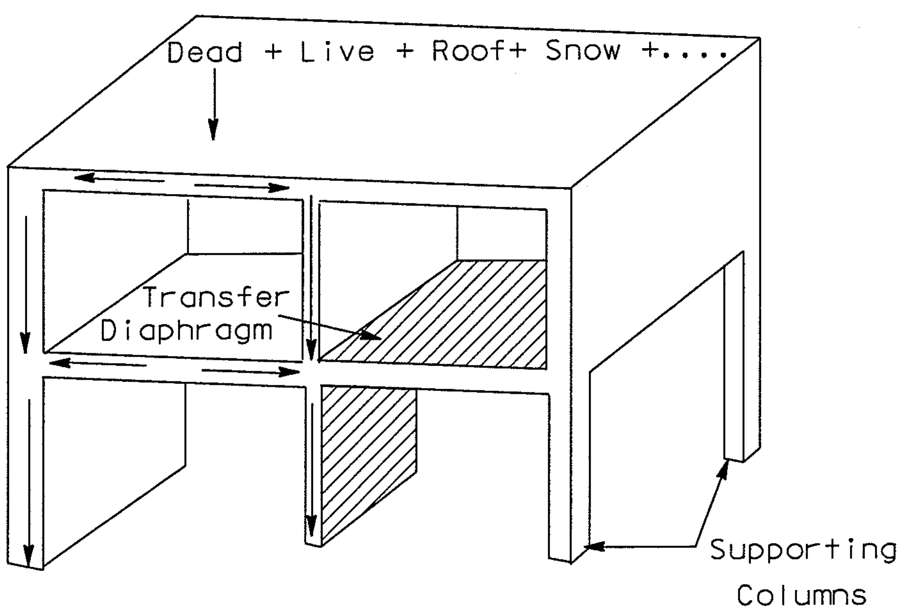


Figure VIII-3 Path of Gravity Loads to Foundations

TRANSFER OF LATERAL LOADS

A continuous load path is necessary to transfer the lateral loads (seismic or wind) from the upper portion of any structure to the foundation. The lateral loads as wind and earthquake are transferred (transmitted) to the foundations through the diaphragms (flexible or rigid), bracings of a braced frames or moment resisting frames.

The lateral-force-resisting-system (LFRS) may consist of shear walls, braced and unbraced frames or moment resisting frames. A combination of different systems could be used as in the case of dual systems. Also, a system may exist in one direction while another system in the orthogonal direction (see horizontal combination).

The total lateral farce is distributed throughout the building in a manner that simulates the behavior of the building during an earthquake.

The load-path concept involves the systematic analysis of loads throughout a structure, from points of origin or application, to the final points of resistance. This analysis can be done at a "macro" level; in which general forces carried by diaphragms, collectors, walls, etc. are determined; and at a "micro" level, in which forces through bolts, stiffeners and welds within a connection are traced.
The IBC 2006/CBC 2007 requires a continuous load path as one of its basic requirements. Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable o transferring all loads and forces from their point of origin to the load-resisting elements. Detailed provisions for identifying and designing this load path are given in the appropriate design and material sections of the UBC.

While providing a continuous load path is an obvious design requirement for vertical loads, experience has shown that it is sometimes overlooked or not completely developed when designing for lateral loads. The failure to review all members and connections for each combination of load can lead to weak-links in the load path.

Important aspects of developing and designing/detailing the lateral load path include:

1- All of the inertia forces originating from the masses on and within the structure must be transmitted from their source to the base of the structure.

2- Forces normal to the plane of a wall must be transferred either vertically to the floors above and below or horizontally to columns that are capable of transferring the forces vertically to the floors above and below.

3- Diaphragms acting as horizontal beams must transfer inertia forces to the frames and/or shear walls. Collector elements need to be provided when transferring loads from diaphragms (roof or floors) into walls, braces, or frames.

4- Frames and shear walls must transfer forces contributed from the diaphragms as well as their own inertia forces to the foundations.

5- Forces applied to the foundations by the shear walls and frames must be transmitted into the ground. Consideration needs to be given when transferring seismic loads through the foundation system into the surrounding soil. It is not sufficient to assume that seismic forces are "resolved" when they reach the soil system. A rational means, using each element of the foundation system (e.g., bearing, friction, passive pressure), must be employed to resist seismic loads.

6- Connections between all elements must be capable of transferring the applied forces from one element to another.

The following figure shows the load path for the lateral loads.

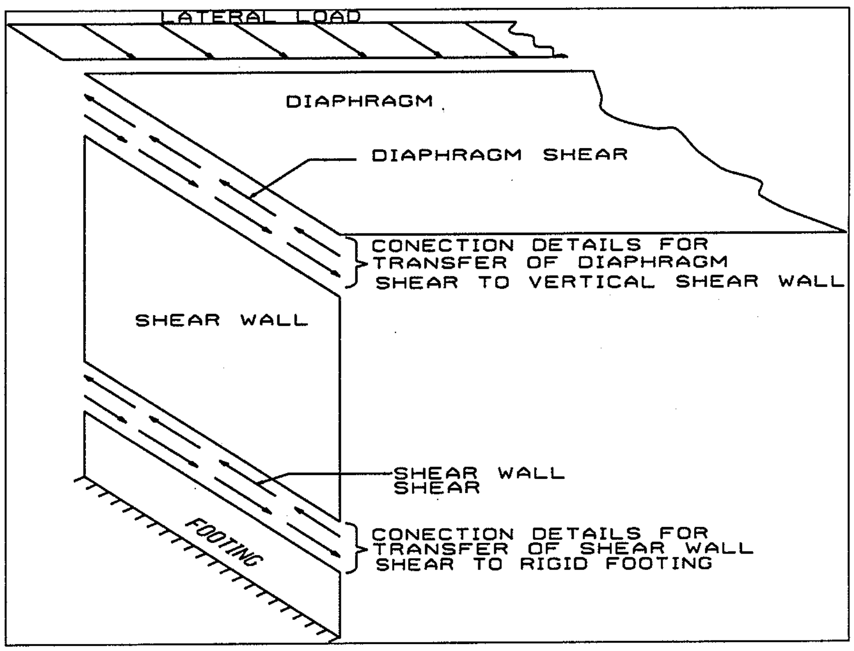


Figure VIII-4 Path of Lateral Loads to Foundation

VIII- F MECHANICS OF MATERIALS

Introduction:

Failure is defined as the "nonconformity with design expectations". Also, failure could be defined as "when materials, members, connections and structures do not perform the intended function(s)". If the definition of failure is limited to "catastrophic structural collapse", these will be only few failures. The Technical Council of Forensic Engineering of the American Society of Civil Engineers (ASCE) has adopted the definition of failure as

"Failure is an unacceptable difference between expected and observed performance." This definitions is similar or close to the previous definition mentioned above.

In addition to structural collapse, the definition adopted by the ASCE includes serviceability problems such as distress, excessive deformation, premature deterioration of materials, leaking roofs and facades, and inadequate interior environmental control systems. This relationship between expectations and performance given in the ASCE definition is helpful to explain the large number of claims field each year stemming from both realistic and unrealistic expectations.

Failure is not a new concept. The construction industry, from ancient times, has benefited from trial-and-error and trial-and-success experience. In fact, the science of archeology is dependent on the uncovered debris of engineering failures happened in the past. The Code of Hammurabi (1780 B.C.); the King of Babylonia, is one of the ancient regulations where builders, engineers and architects had NO opportunity to learn from their own mistakes. There was no such thing as "a second chance".

Laws 229-233 Hammurabi's Code of Laws

- If a builder build a house for someone, and does not construct it properly, and the house which he built fall in and kill its owner, *then the builder shall be put to death*.
- If it kill the son of the owner, *the son of that builder shall be put to death*.
- If it kill a slave of the owner, *then he shall pay slave for slave to the owner* of the house.
- If it ruin goods, he shall make compensation for all that has been ruined, and inasmuch as he did not construct properly this house which he built and fell, he shall re-erect the house fro his own means.
- If a builder build a house for some one, even though he has not completed it, if then the walls seem toppling, the builder must make the walls solid from his own means.

These five harsh rules of Hammurabi not likely stopped all failure, but certainly must have been a deterrent to shoddy construction practice. Also, these harsh rules would have discouraged many innovations and eliminated the probability of malpractice, at least by the same practitioner.

Failure Causes:

Failures may result from a single source or error. However, failure may be the result of several interrelated contributing factors. These factors could be related to technical problems or deficiencies in material performance. The following are the most common causes of failure:

1. Fundamental errors in concept
2. Site selection and site development errors
3. Programming deficiencies
4. Design errors
5. Construction errors
6. Material deficiencies
7. Operational errors

1. *Fundamental errors in concept:*

Some failed projects may be described as fundamental errors in basic concept. The project may be unique, an original attempt to build something beyond available technology. The scale of the project may be outside the envelope of past experience. The project may have been located in an unusual environment, where the prediction of environmental effects was unreliable. Some failures of this type are not engineering failures at all, but rather, economic failures. Those in control of the project may discover that their concept is flawed technically or that the resolution of evolving problems will require far more economic investment than originally anticipated. Such projects may be abandoned and will be considered failures by most observers.

The story of the U.S. Navy " Big Dish" project of 1948-1962 is the record of an example of failure due to error not in design or construction but in concept. In 1948 the Naval Research Laboratory suggested the construction of a radio telescope (600 ft diameter) for scientific work. Congress approved a budget of $20 million in 1956. By the end of 1957, feasibility studies reported that such a structure could be built but at a cost of $52.2 million. Soon thereafter, the Navy decided to combine military functions with the instrument's scientific capabilities, increasing the cost projections to $79 million. These cost estimates were subsequently revised to $126 million and then to over $200 million. In September 1961, however, some years after construction had begun, and with designs not yet fully complete, Congress set a ceiling of $135 million for the project.

In 1962, with expenditures of $63 million, work was stopped and the project was abandoned. The Controller General's report justified termination of the project as follows: "Our belief is based upon reports prepared in 1960 and earlier by scientists within and outside the government who reviewed specific [technical] problem areas and indicated serious doubts that the instrument, if completed, would have the desired capabilities."

2. *Site selection and site development errors:*

Failures often result from unwise land-use or site-selection errors. Certain sites are more vulnerable than others to failure. The most obvious examples are sites located in regions of significant seismic activity, in coastal regions, or in floodplains. Other sites pose problems related to specific soil conditions, such as expansive soils or permafrost in cold regions.

Recognition of the characteristics of particular site conditions through appropriate geotechnical studies can lead to decisions about site selection and site development that reduce the risk of failure. Unnecessary exposure to natural hazards is an unfortunate consequence of historic patterns of human settlement.

3. *Programming deficiencies:*

Failure has been defined as an unacceptable difference between expected and observed performance. This definition implies that the expectations of the client must be clearly understood by the designer, and that they must be realistic. When the project does not perform as expected, even if the expectations are unrealistic or unachievable within given economic restraints, the client is likely to define the project as a failure.

A considerable volume of construction litigation results from unclear or unrealistic expectations. This type of failure could have been avoided through communication during the programming phase of a project. A program should clearly define the scope and intent of a project at the outset, so that general agreement can be reached on a way to measure the success of the completed project.

4. *Design errors:*

Design errors have contributed to many failure cases. The sources of design errors include:

- Errors in design concept
- Lack of structural redundancy
- Failure to consider a load or a combination of loads
- Deficient connection details
- Calculation errors
- Misuse of computer software
- Detailing problems, including selection of incompatible materials or assemblies that are not constructable
- Failure to consider maintenance requirements or durability
- Inadequate or inconsistent specifications for materials or expected quality of work
- Unclear communication of design intent

Deficiency in the basic design of a structure, such as amount of reinforcing steel at points of maximum moment or incorrect dimensions of concrete or steel sections to provide sufficient resistances for normal loading, is a rare cause of failure. One case was caught in 1925, just before placing concrete in forms. for beams spanning 19.5 m (64 ft) over a school auditorium in Yonkers, New York. About 4 in^2 of reinforcement steel had been specified and placed, although the design should have required 40 in^t. At that time, girders of such size were unusual and the error was discovered by an inquisitive young engineer on the contractor's staff. The girders would undoubtedly have failed if the error had not been corrected.

On March 27, 1981, a cast-in-place concrete condominium project collapsed while under construction in Cocoa Beach, Florida, killing 11 construction workers. The most important factor was a design error. The designer never performed any calculations to check punching shear, the most common failure mode associated with this type of structure.

The failure of the Hartford, Connecticut, Civic Center Coliseum space truss roof in January 1978 has been attributed to design assumptions that were not executed in design details. The 110 by 92 m (360 by 300 ft) roof experienced a total collapse when compression chords of the space truss buckled. Bracing, assumed at midspan of the members for input to the sophisticated computer analysis, was not provided in the final design.

5. *Construction errors:*

Failures can result from construction errors. Such failures may involve:

- Excavation accidents
- Construction equipment failure
- Improper construction sequencing
- Inadequate temporary support
- Excessive construction loads
- Premature removal of shoring or formwork
- Nonconformance to design intent

Construction is a dangerous occupation. Many failures of cast-in-place concrete structures occur during construction due to inadequate temporary support, premature removal of shoring, and premature loading of concrete. Examples include the 1973 failure of a 26-story residential tower in Virginia (14 killed) and the Willow Island, West Virginia, cooling tower scaffold collapse of April 1978 (51 killed).

Precast concrete and steel frame structures often experience stability failures when temporary bracing is inadequate. Improper construction sequencing is also a source of failure. In 1985, a Denver, Colorado, highway structure collapsed during construction

when eight girders were placed on incomplete piers. One worker was killed and four were seriously injured. Construction sequence is absolutely critical for certain construction types, such as post-tensioned prestressed concrete. The designer makes assumptions regarding sequencing, and if these are not communicated clearly to those responsible for field operations, the results can be catastrophic.

Some failures are the result of gross violations of the design documents. The July 1983 collapse of the Magic Mart Department Store in Bolivar, Tennessee, is one example. Following the collapse, investigators were able to find very few similarities between the design documents and the as-built construction. Structural steel members were not the sections specified, and the structural grid was even rotated 90 degrees from that shown on the drawings.

6. *Material deficiencies:*

Some would claim that materials do not fail; people fail. Although it is true that most materials problems are the result of human errors involving a lack of understanding about materials or the ignorant juxtaposition of incompatible materials, there are failures that can be attributed to unforeseeable material inconsistencies.

Designers have come to rely on modern structural materials. However, manufacturing or fabrication defects may exist in the most reliable structural materials, such as standard structural steel sections or centrally mixed concrete. In 1980, for example, over 130 buildings in California's San Francisco Bay area experienced serious structural defects as a result of poor-quality aggregate that was inadvertently used by four concrete suppliers. Spalling of the concrete was attributed to several tons of expansive brick that was accidentally dumped onto an aggregate pile at a cement plant. Stone facade panels or glass curtain wall units may contain undetected critical flaws. Although these examples may, in fact, derive from human errors, they can hardly be considered design or construction errors.

7. *Operational errors:*

Failure may occur after the facility is opened and in use by the occupancy as the results of the owner or operator errors. This may include <u>alterations to the structure, change in use, operational judgment errors, inadequate maintenance and negligent overloading</u>. For example, <u>removing a shear</u> wall in a structure by the owner may result in reducing the lateral capacity of the entire structure and or shifting the centre of rigidity which may lead to an increase in the torsional moment and may lead to failure of the entire structure. The failure of Mianus River bridge in Connecticut in June 1983 was due to corroded connection.

VIII-F. 1 Progressive collapse

Progressive collapse could be defined as the collapse (failure) of the structure <u>due to the lack or fewer of structural redundancy.</u> The most prominent example describing this type of failure is the collapse of a 22-story precast concrete building in London (Ronan Point housing project) in the 1968. A gas explosion on the eighteenth floor started a progressive collapse that continued down to the first floor of the building, killing four occupants. This failure led the code requirements for <u>structural redundancy</u> in England and continues to have an influence on designers in the United States , with regard to structural integrity, provision of adequate connections and concerns for prevention of progressive collapse.

VIII- G MATERIALS

VIII-G. Concrete (prestressed, post-tensioned)

VIII-G.2 Timber

See Design Review

VIII- H TRAFFIC SAFETY

VIII-H.1 Work zone safety

See Traffic Review

Construction Depth (P.M.) Practice Problems and Solutions

Depth (P.M.) Problems

The following information are to be used for problems 1 to 4

A soil sample at a future imported site has a 12 % shrinkage and a swell factor of 0.85. An embankment of a new highway has the following dimensions:

width at the base = 70 ft,
with at the top = 38 ft,
side slopes = 4 : 1, and
length of the embankment = 2800 ft

A dump truck has a capacity of 18 loose cubic yards (LCY) of excavated soil and was used to haul the material to the construction site.

1- The shrinkage factor is most nearly:

A) 0.12
B) 0.68
C) 0.88
D) 0.95

2- The volume of the embankment is most nearly:

A) 22,100 yd^3
B) 22,200 yd^3
C) 22,300 yd^3
D) 22,400 yd^3

3- The volume in bank cubic yard (BCY) to build the proposed embankment is most nearly:

A) 24,155 yd^3
B) 25,255 yd^3
C) 25,455 yd^3
D) 26,455 yd^3

4- Assuming that a 30,000 cubic yards of embankment is needed for widening of an existing road. The number of trips the truck will make to haul the soil described above is most nearly:

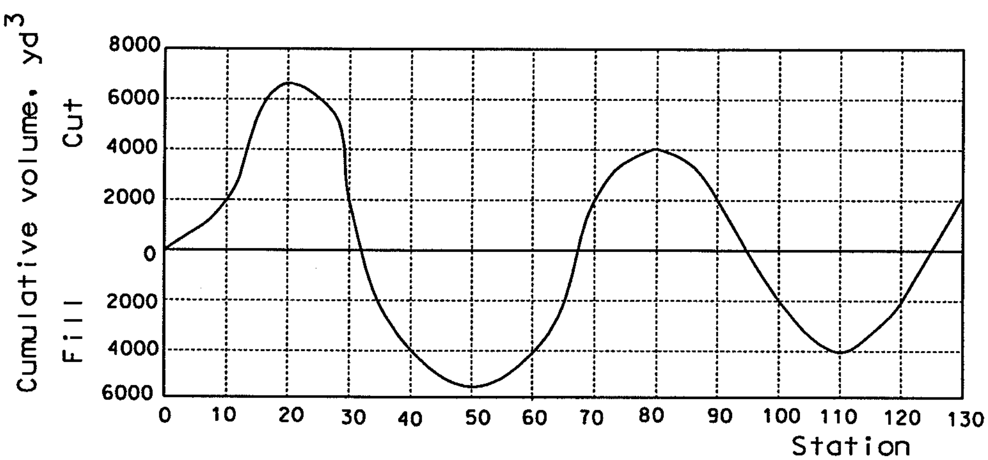
A) 1600
B) 1821
C) 1901
D) 1961

5- Federal OSHA standards required that supported scaffolds can be restrained from tipping when the height to base width ratio is more than.

(A) 5:1
(B) 4:1
(C) 3:1
(D) 2:1

The following information are to be used for problems 6 to 10

The mass diagram of a new construction project is shown below. The testing of the soil indicated that 1.176 yd^3 of excavation (bank cubic yard - BCY) yielded 1.00 yd^3 of embankment (compacted cubic yard - CCY). The limit of economic haul (L.E.H.) was calculated and found to be 2,000 ft (20 stations).



6- Which condition is indicated in the given mass diagram?

(A) Excess
(B) Borrow
(C) Neither
(D) Can't tell

7- If the project contains excess material, how much embankment could be constructed?

(A) 3,500 yd^3
(B) 3,000 yd^3
(C) 2,500 yd^3
(D) 2,000 yd^3

8- The shrinkage factor of the soil is most nearly:

(A) 0.75
(B) 0.80
(C) 0.85
(D) 0.95

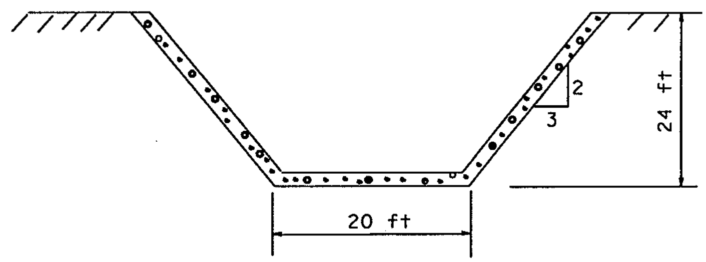
9- The volume of excess excavation actually exists is most nearly:

(A) 2,000 yd^3
(B) 2,253 yd^3
(C) 2,353 yd^3
(D) 3,520 yd^3

10- For the entire job, the total amount of borrow is most nearly:

(A) 2,000 yd^3
(B) 4,000 yd^3
(C) 6,000 yd^3
(D) 10,000 yd^3

11- The cross section of a 1000 ft long irrigation channel is shown. The walls and bottom of the channel have a thickness of 7 inches. Assume 10% waste. During a high flood season, a portion of the lining was damaged. This portion was 8 ft along the left lining of the canal. A 7 sack mix design will be used.



The volume of the cement used in the concrete needed to repair the damage is most nearly:

A) 636.44 ft^3
B) 654.60 ft^3
C) 664.60 ft^3
D) 674.60 ft^3

The following information are to be used for problems 12 &13

The following table shows activities and durations.

Activity	Action	Duration (minutes)	Predecessor
A	Buy groceries	30	
B	Drive home	15	A
C	Start charcoal	20	B
D	Pre-heat oven	5	B
E	Grill hamburgers	10	C
F	Bake beans	20	D
G	Open potato chips	1	B
H	Meal ready	0	Everything ready

12- The critical path is:

A) ABDFH
B) ABCEFH
C) ABCEH
D) ABDGH

13- Assume that the time for dinner is set at 7: 00 p.m.

A) What is the latest you can start buying groceries?
B) What is the latest you can start baking the beans?
C) What is the latest you can finish buying groceries?
D) What is the latest you can open the bag of chips?
E) What is the latest you can finish driving home?

14- A construction company is considering replacing its fueling system. Management has narrowed the choices to two alternatives that offer comparable performance and considerable saving over their present system. The effective annual interest rate is 8%. What is the benefit-cost ratio of the better alternative?

Item	System 1	System 2
Initial Cost	$7000	$11000
Annual Savings	$1500	$1900
Salvage Value	$500	$1250
Expected Life	12 years	12 years

A) 1.36
B) 1.46
C) 1.66
D) 1.86

The following information are to be used for problems 15 to 18

A construction company purchased a new loader for $60,000. The estimated salvage value of the loader is $10,000 at the end of five years.

15- The annual depreciation for the loader using the straight line method is:

A) $10,000
B) $12,000
C) $15,000
D) $16,000

16- The book value of the loader at the end of the third year based on straight line depreciation method is:

A) \$50,000
B) \$40,000
C) \$30,000
D) \$20,000

17- The depreciation of the loader for the third year using sum of the years' digits method is:

A) \$15,000
B) \$14,000
C) \$12,000
D) \$10,000

18- The book value of the loader at the end of the third year based on the sum of the years' digits depreciation method is:

A) \$50,000
B) \$40,000
C) \$30,000
D) \$20,000

19- A construction company has 650,000 employee hours worked with the following safety record:

Incident Category	No. of Incidents
Minor injuries (first aid only)	10
Medical-only injuries (no lost time or light duty	3
Medical injuries resulting in "light duty" restrictions	4
Lost-time injuries	4

The OSHA Incident Rate (IR) is most nearly:

A) 2.33
B) 3.13
C) 3.38
D) 6.46

20- Per OSHA classification, which type of soil or rock type could be excavated with vertical sides and remain intact for an excavation less that 20 feet deep:

A) Soil type A
B) Soil type B
C) Soil type C
D) Stable rock

21- After 6 months of starting the construction of a new highway, the following information is given:

The planned costs to date = $1,450,000
The reported charges to the job to date = $1,625,000
The earned value to date = $1,725,000

The cost and the schedule status of the project are most nearly:

A) behind schedule and under budget
B) ahead of schedule and over budget
C) behind schedule and over budget
D) ahead of schedule and under budget

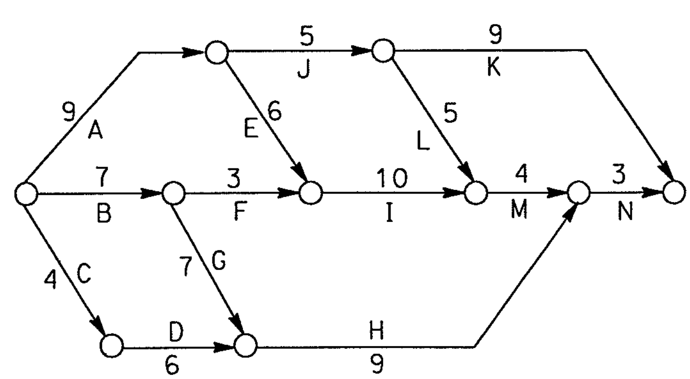
22-Per OSHA classification, which type of soil has the least unconfined compressive strength:

A) Soil type D
B) Soil type C
C) Soil type B
D) Soil type A

23- Which of the following statement is not correct regarding the installation of the temporary erosion control measures:

A) The straw bale barrier shall be used only when water can pond to allow the sediments to settle out.
B) A silt fence is to be installed along the toe of the slope below the high water level of the receiving channel.
C) Silt fences or straw bales need to be staked into place downstream of the maintenance site.
D) Sand bags will be placed where it can intercept and slow the flow to allow the sediments to settle out.

24- The network below shows activity-on-arrow where the duration for each activity is indicated.



The early start for activity N is most nearly:

A) 23
B) 24
C) 29
D) 31

25- Sloping or benching for excavations greater than 20 feet deep should be:

A) designed by a non registered engineer
B) designed by a registered professional engineer
C) designed by either a registered or non registered professional engineer
D) designed by a non registered engineer with 15 years experience in projects that have excavations greater than 20 feet

26- A new water line will be constructed eight feet below the original ground. The width of the trench at the bottom was determined to be five feet. The soil at the site has an unconfined compressive strength of 1.6 ton per square foot. The width of the trench at the top is most nearly:

A) 13 feet
B) 17 feet
C) 21 feet
D) 29 feet

The following information are to be used for problems 27 to 28

150 tons of sand with a density of 105 lb/cf must be transported 30 mi using a 15-cy dump truck. Two laborers and a driver, at a rate of 2.0 cy/hr each, will load the truck. Assume a haul speed of 40 mph, return speed of 50 mph, and 6 min. to dump the load. The cost of the truck is $30/hr, the driver is $24/hr, and the laborers cost $16 /hr each.

27- The total time to haul the sand is most nearly:

A) 35.15 hr
B) 37.15 hr
C) 39.15 hr
D) 42.15 hr

28- The total cost to haul the material is most nearly:
A) $3,195
B) $3,694
C) $4.654
D) $5,543

29- The maximum distance from the edge of a trench that an excavated material can be stored is:
A) 2 feet
B) 3 feet
C) 4 feet
D) 5 feet

30- A shoring system is required to construct a new sewer line. The contractor wants to substitute the vertical planks (uprights) by plywood. Which of the following statements is correct regarding to his request:

A) a plywood is permitted to substitute the vertical planks
B) a plywood is permitted to substitute the vertical planks if the plywood is a ¾-inch thick and is a structural I
C) plywood should not substitute the uprights (vertical planks)
D) plywood will be permitted if two layers of plywood are used

31- Temporary erosion control is:

A) required only for small projects less than $1,000,000
B) required only for the projects that have concrete operations
C) required for large projects greater than $25,000,000
D) mandatory for all projects

32- Which of the following is considered a temporary erosion-control measure:

A) Silt fence
B) Straw bale barrier
C) Sand bag barrier
D) All of the above

33- An excavation in a soil type B is requited to place a new water line. The bottom of the water line elevation is 14 feet below the original ground. The slope of the excavation should not be steeper than:

A) 1 H: 1V
B) ½ H:1V
C) 1½ H: 1V
D) ¾ H:1V

34- A construction company has 450,000 employee hours worked per year. The company has 16 injuries and illnesses during a particular year. The OSHA Incident Rate (IR) is most nearly:

A) 6.33
B) 7.11
C) 9.38
D) 12.46

35- Which of the following is the preferred objective when developing a temporary erosion-control plan:

A) Keep the soil at its original location.
B) Keep the soil close to its original location.
C) Keep the soil on site.
D) Keep the soil within ½ mile from its original location

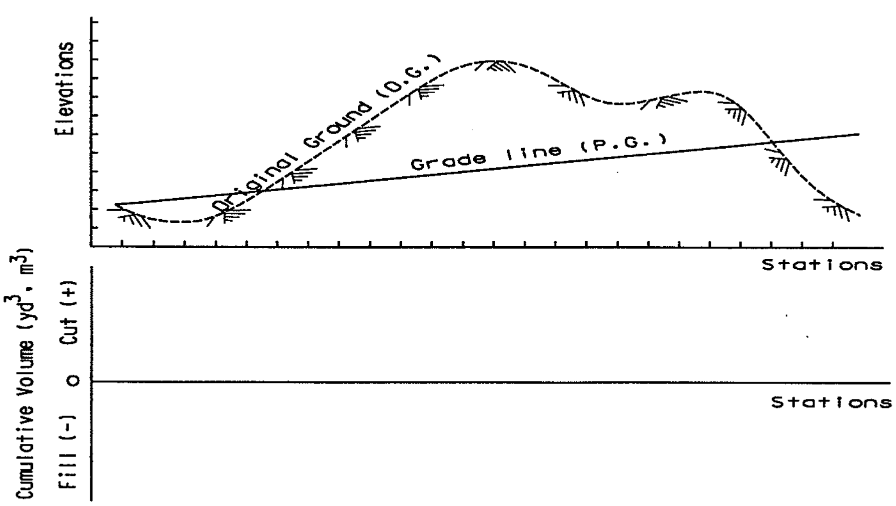
36- A driven pile foundation system will be used to support a new bridge. The subsurface exploration log of the site is shown on the following page. The designer decided to have the pile tip (end-bearing) at the top of the rock layer elevation. The elevation of the bottom of the pile cap is 978.5 and the pile embedment in the pile cap is 1′ – 6″. A length of 1′ – 6″ at the top of the pile will be cut-off due damage caused by the driving hammer. The minimum pile length to be ordered is most nearly:

A) 49 feet
B) 50 feet
C) 51 feet
D) 52 feet

37- In a mass diagram of earthwork, abscissas represent stations and ordinates represent:

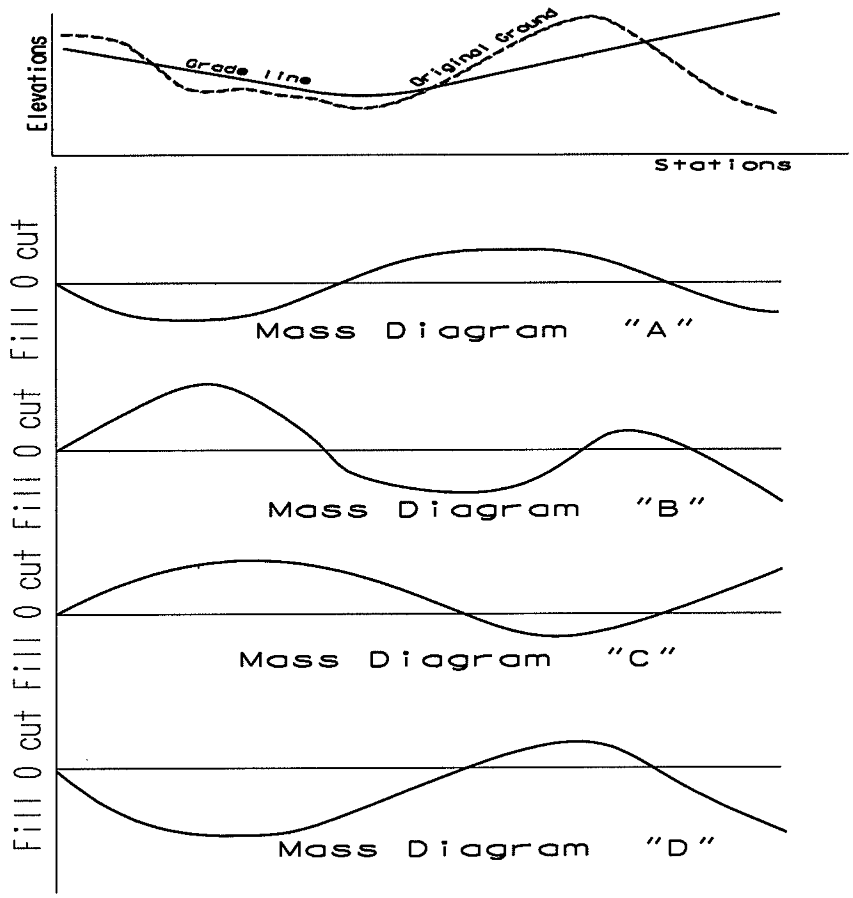
(A) Cut or fill at individual stations
(B) Cumulative corrected algebraic sums of volumes of earth work
(C) The volume of earth work without correction for shrinkage or swell
(D) The absolute volume of earth work after correction for shrinkage and swell

38- A proposed grade line (profile grade, P.G.) and original ground for a new facility is shown below. Construct the qualitative mass diagram below the given figure. Show low and high points. Also, indicate positive and negative slopes.



39- A proposed grade line (profile grade, P.G.) and the original ground for a new facility are shown below. Four mass diagrams "A" through "D" were drawn. Which mass diagram represents the given information? Explain

(A) Mass diagram "D"
(B) Mass diagram "C"
(C) Mass diagram "B"
(D) Mass diagram "A"

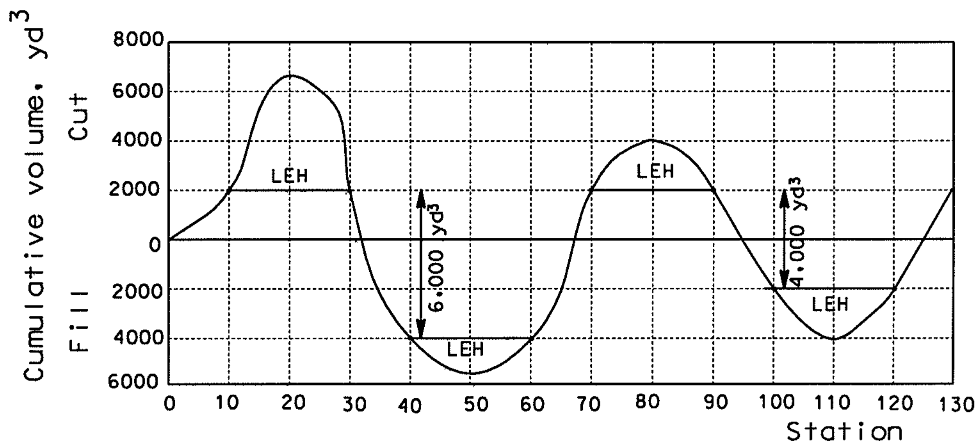


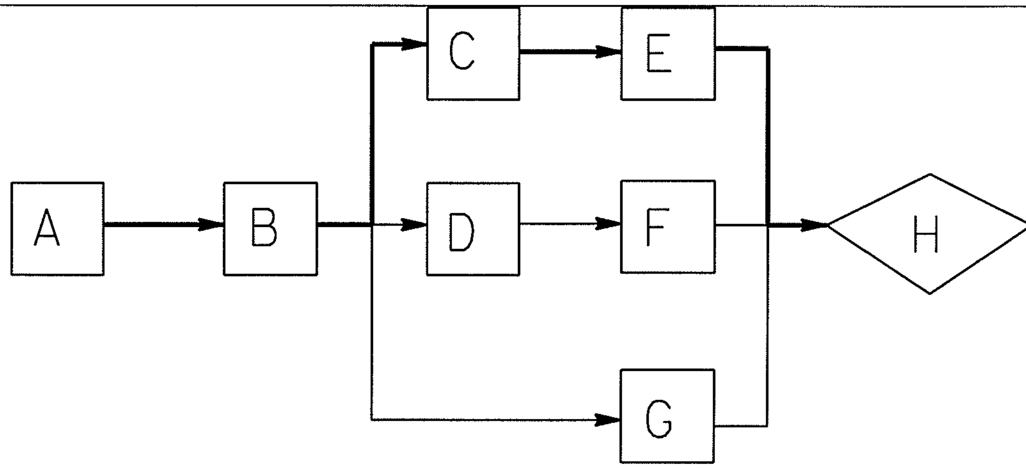
40- A construction company is expanding by adding two new scrapers at a total cost of \$75,000. Operating and maintenance costs for the new scrapers are projected to be \$20,000/year for the next eight years. After eight years, the scrapers will be sold for a total of \$10,000. Annual revenues are expected to increase by \$34,000 with the addition of the scrapers. The company's rate of return on the purchase is most nearly:

A) 10.0%
B) 10.3%
C) 12.7%
D) 15.2%

Depth (P.M.) Solutions

#	Ans.	Solution
1	C	Shrinkage factor = 1 – shrinkage = 1 – 12 % = 0.88
2	D	The height of the embankment = [(70′ – 38′) / 2] / 4 = 4 feet Volume of the embankment (CCY): $= [(70' + 38') / 2] \times 4' \times 2800' \times (ft^3 / 27\ yd^3) = 22{,}400\ yd^3$
3	C	Bank cubic yards (BCY) needed = 22,400 yd^3/0.88 = 25,455.5 yd^3
4	D	Number of trips = Loose cubic yards (LCY)/ truck capacity per trip Loose cubic yards (LCY) = Bank cubic yards (BCY) / swell factor = 30,000/0.85 = 35,294.1 yd^3 Number of trips = 35,294.1 yd^3 / 18 yd^3 = 1960.78
5	B	4:1 §1926.451 (c) on page 278
6	A	As can be seen from the mass diagram that it is ended above the balance line. Therefore, it is an excess (cut > fill) condition.
7	D	As can be seen from the mass diagram above that it is ended above the balance line. Therefore, there is an excess of 2000 yd^3
8	C	$Compacted\ Volume(CCY) = Bank\ Volume(BCY) \times Shrinkage\ Fact$ $Shrinkage\ factor = \frac{CCY}{BCY} = \frac{1}{1.176} = 0.85$
9	C	The mass diagram volume is the <u>compacted volume</u> after the necessary corrections for shrinkage or swell factors have been applied. The shrinkage factor is calculated to be 0.85. The excess excavation actually exists is = 2000 yd^3/ 0.85 = 2352.9 yd^3

<table>
<tr><td>10</td><td>D</td><td>The longest distance the material will be hauled is the limit of economical haul, in this problem given as 2000 ft. Therefore, any material represented by a negative slope (i.e. fill) portion of the mass diagram that is outside the portion of the loops cut off by the LEH lines must be borrowed.

➢ Steps:
1- Draw horizontal lines equal to the LEH specified (or calculated) in the problem. These horizontal lines should cut off ALL loops (a loop consists of a cut & a fill).
2- The volume of fill (negative slope) which is outside the LEH is the total borrow.
3- Thus: 6000 yd^3 + 4000 yd^3 = 10,000 yd^3</td></tr>
<tr><td>11</td><td>A</td><td>$V_{Concrete}=\left(7\,in./12\right)(8\,ft)(1000\,ft)(1.10)=5133.33\,ft^3=190.12\,yd^3$
$Weight\ of\ Cement=7\,Sack/yd^3\times 190.12\,yd^3\times 94\,lb/Sack=125{,}098.96\,lb$
Using the absolute volume method equation:
$V_{Cement}=\dfrac{Weight\ of\ Cement}{Specific\ Gravity\ of\ Cement\times Unit\ Weight\ of\ Water}$
$=\dfrac{125{,}098.96\,lb}{3.15\times 62.4\,pcf}=636.44\,ft^3$</td></tr>
<tr><td>12</td><td>C</td><td></td></tr>
</table>

		 The critical path is shown with dark lines (ABCEH)
13		A) What is the latest you can start buying groceries? 5:45 p.m. B) What is the latest you can start baking the beans? 6:40 p.m. C) What is the latest you can finish buying groceries? 6:15 p.m. D) What is the latest you can open the bag of chips? 6:59 p.m. E) What is the latest you can finish driving home? 6:30 p.m.
14	C	***Note***: Salvage value should be counted as a decrease in cost, not as benefit. System 1: Benefit = B = $\$1500 \left(P/A\right)_{12}^{8\%} = \$1500\ (7.5361) = \$11{,}304.15$ Cost = C = $\$7000 - \$500\left(P/F\right)_{12}^{8\%} = \$7000 - \$500(0.3971) = \$6{,}801.45$ $\left(\frac{B}{C}\right)_{Sys\ 1} = \frac{\$11{,}304.15}{\$6{,}801.45} = 1.66$ System 2: Benefit = B = $\$1{,}900 \left(P/A\right)_{12}^{8\%} = \$1{,}900\ (7.5361) = \$14{,}318.59$ Cost = C = $= \$11{,}000 - \$1{,}250\left(P/F\right)_{12}^{8\%} = \$11{,}000 - \$1{,}250(0.3971) = \$10{,}503.63$ $\left(\frac{B}{C}\right)_{Sys\ 2} = \frac{\$14{,}318.59}{\$10{,}503.63} = 1.36$ *(continued on next page)*

		Comparing Alternatives Based on an Incremental Analysis: *The alternatives cannot be compared to one another based on their benefit-cost ratios. Instead, an incremental analysis is required.* $\frac{B_{Sys2}-B_{Sys1}}{C_{Sys2}-C_{Sys1}}=\frac{\$14{,}318.59-\$11{,}304.15}{\$10{,}503.63-\$6{,}801.45}=0.8142$ Because the incremental analysis is LESS than ONE → ∴ System "1" is better than System "2"
15	A	Depreciation = $\frac{\$60{,}000-\$10{,}000}{5}=\$10{,}000$
16	C	Book Value = \$60,000 – 3 × \$10,000 = \$30,000
17	D	SOYD = $\frac{n(n+1)}{2}=\frac{5\times 6}{2}=15$ Depreciation = (\$60,000 –\$10,000) (3/15) = \$10,000 ***NOTE:*** depreciation is highest in the first year and decreases throughout the equipment's life. Also note that the total amount depreciated over the five years adds to \$50,000, the same as in straight line depreciation. This leaves the equipment value at \$10,000, the salvage value
18	D	Book Value = \$60,000 – (5/15 × \$50,000 + 4/15 × \$50,000 + 3/15 × \$50,000) = \$20,000

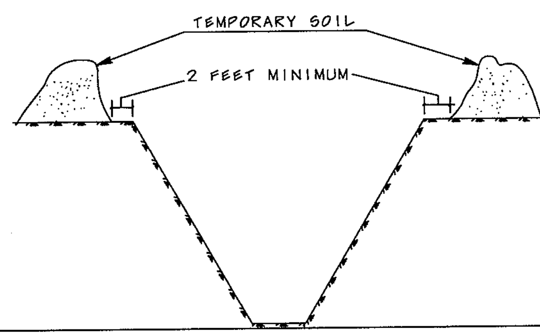
19	C	The term incident rate means the number of injuries and illness, or lost workdays, per 100 full-time workers. The rate is given by the following equation: $$IR = \frac{N \times 200{,}000}{EH}$$ Where: IR = Incident Rate N = Number of injuries and illness, or number of lost workdays EH = Total hours worked by employees during a month, a quarter, or a fiscal year 200,000 = Base for 100 full-time equivalent workers (100 employees × 40 hrs/week × 50 weeks per year) $$IR = \frac{(3+4+4) \times 200{,}000}{650{,}000} = 3.38$$
20	D	OSHA classifies soils into four categories as follows: **<u>Stable Rock:</u>** This is a natural solid mineral matter that can be excavated with vertical sides and remain intact while exposed. It is usually identified by a rock name such as granite or sandstone. Determining whether a deposit is of this type may be difficult unless it is known whether cracks exist and whether or not the cracks run into or away from the excavation. **<u>Type "A" Soils:</u>** Those are cohesive soils with an unconfined compressive strength of 1.5 tons per square foot (tsf) (144 kPa) or greater. Examples of Type "A" cohesive soils are often: clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. No soil is Type A if it is fissured, is subject to vibration of any type, has previously been disturbed, is part of a sloped, layered system where the layers dip into the excavation on a slope of 4 horizontal to 1 vertical (4H:1V) or greater, or has seeping water. **<u>Type "B" Soils:</u>** Those are cohesive soils with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa). Examples of other Type "B" soils are: angular gravel; silt; silt loam; previously disturbed soils unless otherwise classified as Type C; soils that meet the unconfined compressive strength or cementation requirements of Type "A" soils but are fissured or subject to vibration; dry unstable rock; and layered systems sloping into the trench at a slope

		less than 4H:1V (only if the material would be classified as a Type B soil). **<u>Type "C" Soils:</u>** Those are cohesive soils with an unconfined compressive strength of 0.5 tsf (48 kPa) or less. Other Type "C" soils include granular soils such as gravel, sand and loamy sand, submerged soil, soil from which water is freely seeping, and submerged rock that is not stable. Also included in this classification is material in a sloped, layered system where the layers dip into the excavation or have a slope of four horizontal to one vertical (4H:1V) or greater.
21	D	Budgeted Cost for Work Scheduled = BCWS = planned costs = \$1,450,000 Actual Cost of Work Performed = ACWP = actual spent = \$1.650,000 Budgeted Cost of Work Performed = BCWP = earned value = \$1,725,000 Cost Variance (CV) = (Earned work-hours or dollars) – (Actual work-hours or dollars) = BCWP – ACWP = \$1,725,000 – \$1,650,000 = \$75,000 → ∴ under budget Schedule Variance (SV) = (Earned work-hours or dollars) – (Budgeted work-hours or dollars) = BCWP – BCWS = \$1,725,000 – \$1,450,000 = \$275,000 → ∴ ahead of schedule Therefore, the project is ahead of schedule and under budget because the difference in both cases is positive. If the difference is negative, in either one of them, this indicates either over budget or behind schedule depending on which one has a negative value.
22	B	Soil type C (≤ 0.5 tsf) See the answer of question # 20
23	B	A silt fence is to be installed along the toe of the slope above the high water level of the receiving channel. If it is placed below the high water level of the stream channel, it will not serve its purpose of preventing sediment to the channel
24	C	Examining the different paths: Path AJLM = 9+ 5 + 5 + 4 = 23 Path AEIM = 9+ 6 + 10 + 4 = 29

<table>
<tr><td></td><td></td><td>Path BFIM = 7+ 3 + 10 + 4 = 24
Path CDH = 4 + 6 + 9 = 19
Path BGH =7 + 7 + 9 = 23
Path AJLM = 9 + 5 + 5 + 4 =23
The longest path to the start of activity N is defined by path AEIM and the duration is 29</td></tr>
<tr><td>25</td><td>B</td><td>See foot note 3 below

Table B-1 OSHA Maximum Allowable Slopes (modified)
<table>
<tr><th>Soil Type</th><th>Maximum Allowable Side Slope (H : V Ratio) for excavation < 20 feet deep(3)</th><th>Slope Angle(1) (Repose angle), degrees</th></tr>
<tr><td>Rock</td><td>0 : 1 (Vertical)</td><td>90°</td></tr>
<tr><td>Type A(2) (depth < 12 ft)</td><td>½ H : 1V</td><td>63°</td></tr>
<tr><td>Type A</td><td>¾ H : 1V</td><td>53°</td></tr>
<tr><td>Type B</td><td>1H:1V</td><td>45°</td></tr>
<tr><td>Type C</td><td>1½ H : 1V</td><td>34°</td></tr>
</table>
1. Maximum allowable angles expressed in degrees from horizontal. Angles have been rounded off.
2. A short term maximum allowable slope of ½ H: 1V is allowed in excavations in Type A soil that are 12 feet (3.67 m) or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet (3.67 m) in depth shall be ¾ H: 1V (53°).
3. Shoring or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.</td></tr>
<tr><td>26</td><td>B</td><td>The first step is to determine the soil type. Per OSHA CFR 1926 Subpart P, Appendix A:

<u>Type “A” Soils:</u> Those are cohesive soils with an unconfined compressive strength of 1.5 tons per square foot (tsf) (144 kPa) or greater.
Per Table B-1 (1926 Subpart P, Appendix B) , the width of the trench at the top for soil type A is :
= 5 feet (bottom) + ¾ (8 feet) × 2 (sides) = 17 feet</td></tr>
<tr><td>27</td><td>B</td><td>Volume of sand = 150 T × 2,000 lb/T × cf/105 lb × cy/27cf
= 105.82 cy</td></tr>
</table>

		Cycle time: Load = 15 cy/(3 × 2 cy/hr) = 2.50 hr Haul = 30 mi/40 mph = 0.75 hr Dump = 6 min/(60 min/ hr) = 0.10 hr Return = 30 mi/50 mph = <u>0.60hr</u> Total cycle time = 3.95 hr/trip ***production rate:*** Number of trips per hour = 1.0/(3.95 hr/trip) = 0.253 trips/hr Quantity hauled per trip = 15 cy/trip × 0.253 trips/hr = 3.795 cy/hr Production rate = 3.795 cy/hr × 45/60 = 2.85 *cy/hr* ***Time:*** Using 1 truck and 2 laborers = 105.82 cy/2.85 cy/hr = 37.13 hr
28	A	Using the time calculated in the previous problem 37.13 hr, the cost is calculated as follows: Truck = 37.13 hr × 1 truck @ $30/hr = $1,113.90 Driver = 37.13 hr × 1 driver@$24/hr = $ 891.12 Laborers = 37.13 hr × 2 laborers @ $16/hr = <u>$1,188.16</u> Total cost = $3,193.18
29	C	Per OSHA 29 CFRT 1926.651(j)(2), the answer is 2 feet minimum as shown below 
30	C	The spacing and size of the uprights (vertical planks) should be selected in accordance with the tables in Appendix C (CFR 1926, Subpart P, Appendix C). The use of plywood instead of the uprights is not permitted.
31	D	Erosion control is <u>*mandatory*</u> on all construction projects
32	D	All of them are considered temporary erosion-control measures. The selection of temporary erosion control measure must be based upon <u>good judgment and past experience under similar conditions.</u>

<table>
<tr>
<td>33</td>
<td>A</td>
<td>

Table B-1 OSHA Maximum Allowable Slopes (modified)

<table>
<tr><th>Soil Type</th><th>Maximum Allowable Side Slope (H : V Ratio) for excavation < 20 feet deep[3]</th><th>Slope Angle[1] (Repose angle), degrees</th></tr>
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</table>

</td>
</tr>
<tr>
<td>34</td>
<td>B</td>
<td>

The term incident rate means the number of injuries and illness, or lost workdays, per 100 full-time workers. The rate is given by the following equation:

$$IR = \frac{N \times 200{,}000}{EH}$$

Where:
IR = Incident Rate
N = Number of injuries and illness, or number of lost workdays
EH = Total hours worked by employees during a month, a quarter, or a fiscal year
200,000 = Base for 100 full-time equivalent workers (100 employees × 40 hrs/week × 50 weeks per year)

$$IR = \frac{16 \times 200{,}000}{450{,}000} = 7.11$$

</td>
</tr>
<tr>
<td>35</td>
<td>A</td>
<td>

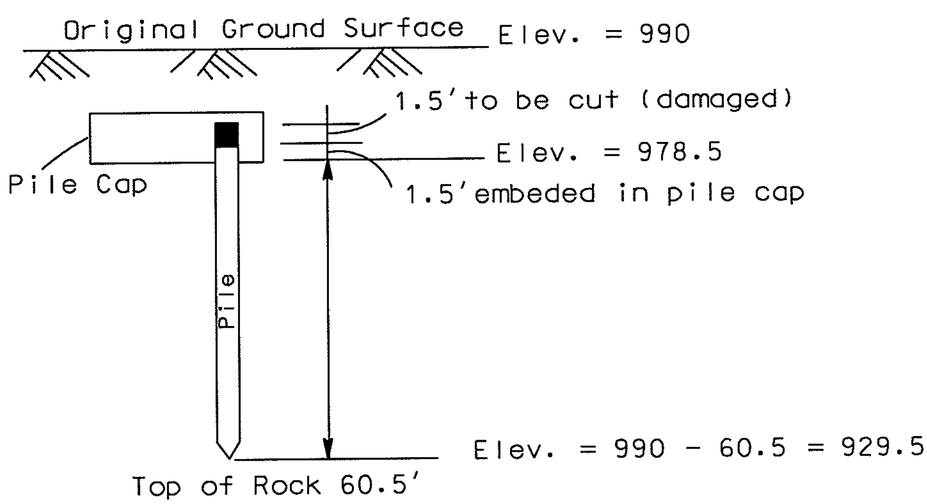
When developing a temporary erosion-control plan for a specific site, the following *three design objectives* should be analyzed:

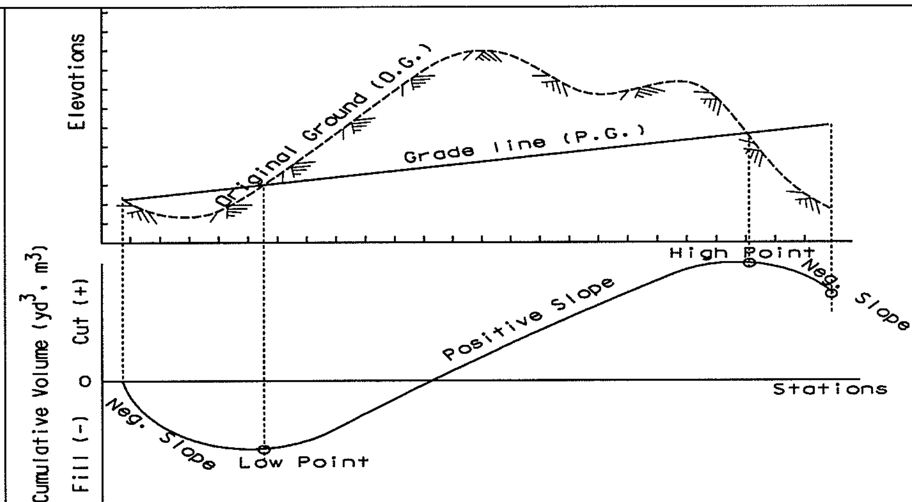
A) Keep the soil at its original location
A) Keep the soil close to its original location
B) Keep the soil on site

Keeping the soil at its original location is the <u>*preferred objective*</u> because it causes the <u>*least amount of harm to the environment*</u>. This option not only protects the surrounding land and water, but also prevents cost regrading and redressing of slopes and ditches.

Keeping the soil at its original location is not always possible due to

</td>
</tr>
</table>

		challenging topography and other site variables. If the soil can't be kept at its original location, it should be kept close. This option will require some re-grading and redressing of slopes and ditches. Finally, if site conditions are such that neither of the first two objectives can be met, at least try to keep the soil from leaving the site.
36	D	Here are the steps: 1- The rock layer is at 60.5 ft as shown in the boring log 2- The surface (O.G.) elevation is given to be = 990 ft 3- The elevation of the rock layer is = 990 – 60.5 = 929.5 ft 4- The elevation at the bottom of the pile cap is given = 978.5 ft 5- The pile will be embedded 1.5 ft into the pile cap 6- There will be a 1.5 ft at the top of the pile to be cut due to the damage of the driving hammer 7- The required length of the pile is: $= (978.5 - 929.5) + 1.5 + 1.5 = 52$ feet The following diagram shows the pile and pile cap in relation to the rock layer 
37	B	Cumulative corrected algebraic sums of volumes of earth work

38		
39	C	By the process of elimination, mass diagrams A & D are wrong because the slope at the beginning of these two diagrams is negative (i.e. fill) but it should be positive (cut). Mass diagram B is the best one to describe the given information where high and low points are located close to the transitions from cut to fill and from fill to cut.
40	B	Equate the equivalent uniform annual cost of the revenues and the expenses as follows: $\$40,000 = \$75,000\ (A/P)_{n=8}^{i=?} + \$20,000 - \$10,000\ (A/F)_{n=8}^{i=?}$ Try I = 10% and 12% Interpolation gives I = 10.3%

References:

1- OSHA. *www.osha.gov/SLTC/trenchingexcavation/index.html.*

2- *Caterpillar Performance Handbook.* Caterpillar Inc. Annual publication. Clough,

3- Atkins, Harold N. *Highway Materials, Soils, and Concretes.* Prentice-Hall, Upper Saddle River, N. J., 1997.

4- Richard, and Sears. *Construction Contracts.* John Wiley and Sons, New York, 1991.

5 - Callahan, Michael, Quackenbush, and Rowlings. *Construction Project Scheduling.* McGraw-Hill, New York, 1992.

6- Halpin, Daniel W. *Construction Management.* John Wiley and Sons, New York, 2006.

7- Nunnally, S. W. *Construction Methods and Management.* Prentice Hall, Upper Saddle River, N. J., 1998.