DESIGN WORKSHOP

SMALL SCALE WATER CONTROL STRUCTURES

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PREFACE

A Transfer of Knowledge Program is included as part of the CIDA financed Small Scale Water Control Structures III (SSWCS III) Project. This Program is aimed primarily at the technical and engineering staff of the Government of Bangladesh (GOB) and Bangladesh Water Development Board (BWDB). It is intended to enhance technical skills in engineering, agriculture, economics, management, and to increase the understanding of related disciplines such as environment and fisheries. To achieve these ends, several levels of training will be undertaken including design workshops for BWDB officials.

The objective of the design workshops is to provide training to the BWDB design engineers on principles and methodologies for design of water control structures.

In order to achieve the above objective, these manuals have been prepared in which principles and step-by-step computations for design of a drainage regulator and embankment are provided.

The engineers of Bangladesh Water Development Board and Northwest Hydraulic Consultants Ltd. prepared materials for these volumes. The Canadian International Development Agency provided funds, under the SSWCS III project, for development and publication of these training manuals and for organizing the design workshop.

Any suggestion to improve the quality of contents will be greatly appreciated.

HRKhan

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CHAPTER SIX

FOUNDATION DESIGN

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PREFACE

The topic has been divided into two parts. Part A and Part B.

Part A deals with quick and simplified methods of foundation design on based SPT (Standard Penetration Test). Most of the foundations that are small and not very sophisticated can be designed by these methods.

Part B deals with detail foundation design based on soil test results and empirical formulas. For large and important structures, the quick and simplified methods should be used for preliminary design only. Final design should be based on detail test results.

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PART - A

SIMPLIFIED DESIGN BASED ON STANDARD PENETRATION TEST (SPT)

1.0 BACKGROUND

In most of the cases detail soil test results are not available during design stage. However, a bore log with N values of SPT is usually submitted for preliminary assessment of the subsoil condition. In absence of detail test results this 'N' value of SPT can be correlated to different strength parameters of soil to calculate the allowable bearing pressure and to check the permissible settlement.

2.0 INTRODUCTION

The art of designing the best and the most economical foundation for a structure depends on careful investigation of the subsoil condition. The sub-soil report should provide detail information about the qualitative index properties and mechanical characteristics of the soil on which the structure will be constructed. Foundation designer should consider two important factors: the bearing capacity of the soil for the applied load, and the anticipated total and differential settlements that should not exceed certain limit.

3.0 SOIL CLASSIFICATION

Soils can be classified into two general groups each with two principal subgroups.

Coarse Grained Soils - Gravel and Sand
Fine Grained Soils- Silt and Clay.

Most natural soils consists of a mixture of two or more of the above constituents and many of them contain an admixture of organic matters. The mixture is given the name of the constituent that appears to have the most dominant influence on its behaviour. Thus a silty clay has predominantly the properties of a clay but contains significant amount of silt.

Since particle size is probably the most obvious characteristics of a soil, it is natural that the classification system would be based on soil texture alone. Many such systems as shown in Table 1 have been suggested. The MIT and the UNIFIED systems are most commonly used by foundation engineers. According to ASTM and UNIFIED system the following criteria are used for soil classification by size:

Gravel	:	Larger than	4.760 mm
Sand	:	Coarse,	4.76 - 2.000 mm
		Medium,	2.00 - 0.420 mm
		Fine,	0.42 - 0.074 mm
Silt	:		0.074- 0.005 mm
Clay	:	Smaller than	0.005 mm.

As already stated, natural soil is a mixture of different grain sizes. For example, such a mixed soil can be described as a mixture of " 20 percent sand, 40 percent clay and 40 percent silt", etc.

Table 1. Soil Classification based on Grain Size

Grain size scale (mm): 100, 10, 1, 0.1, 0.01, 0.001, 0.0001

Classification System	Grain size mm			
MIT, 1931	Gravel	Sand	Silt	Clay
	2	0.06	0.002	
AASHTO, 1970	Gravel	Sand	Silt	Clay
	75	2	0.05	0.002
UNIFIED 1953	Gravel	Sand	Fines (silt & Clay)	
	75	4.75	0.075	
ASTM	Gravel	Sand	Silt	Clay
	2	0.075	0.005	

Depending on particle size distribution or from the triangular textural classification chart as shown in **Fig.** 1, the mixed grain soil can be given a single name that would indicate the most dominant properties of soil.

Soil Properties

Rather than assessing a soil from its description only, the foundation engineer must know the numerical values of the soil properties either by laboratory tests or field tests and these values should be used in the design. These properties may be classified into two major heads: index properties and physical properties.

Index Properties of Soil

Index properties of soil are used primarily to identify, classify and reflect the general character of soils. Index properties are divided into two classes: soil grain properties and soil aggregate properties.

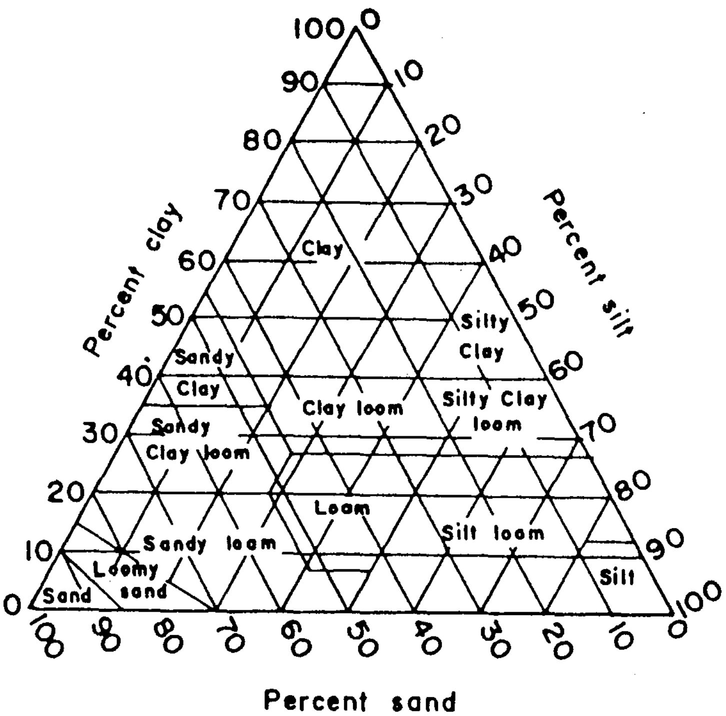


Fig. 1   Triangular soil classification chart (U.S Department of Agriculture)

Soil grain properties are the size, shape and composition of the individual particles irrespective of how the particles are arranged in a soil mass. The most important grain property of a coarse grained soil is the particle size distribution. This is determined by mechanical analysis using standard set of sieves. To determine the particle size distribution of fine grained soil, wet method of mechanical analysis is used.

All wet methods of analysis are based on stoke's law. The results of a mechanical analysis are usually presented in the form of a particle size distribution or gradation curve. From the gradation curve it is possible to know whether a soil is well graded, uniformly graded or poorly graded. The plot of the gradation curve for well graded soils takes an S-curve shape and that of uniform soil takes the shape of almost a vertical line.

Usually the words Well Graded, Graded and Uniform are used as a means of describing the gradation of a granular soil. The results of grain size distribution analysis are expressed in terms of some numerical values. These numerical values are referred to as Effective Grain Size (D_{10}), Uniformity Coefficient(Cu) and Average Grain Size (D_{50}).

The Effective Grain Size (D_{10}) is the diameter that corresponds to a particle size on the grain size distribution curve from which 10 percent of the particles are finer and 90 percent particles are coarser.

The uniformity coefficient Cu, is the ratio of D_{60} size to the effective size (D_{60}/D_{10}). D_{60} size is the grain size on the gradation curve from which 60 percent of the particles are finer and 40 of the particles are coarser. For uniform soils Cu is less than 4, for graded soils it is in between 4 and 6, and for well graded soils it is more than 6.

Average Grain Size (D_{50}) is the diameter that corresponds to a particle size on the grain size distribution curve from which 50 percent of the particles are finer and 50 percent of the particles are coarser.

For clayey soils index properties mostly define soil consistency and plasticity properties.

Soil aggregate properties describe the structure of the soil mass i.e arrangement and orientation of individual soil particles or grains. The aggregate properties of a soil mass greatly influence its stability and strength.

Physical Properties of Soil

Physical properties of soil include strength properties or mechanical properties like shear strength, unconfined compressive strength, cohesion, and compressibility etc. These properties are very important for foundation design and should be determined correctly through field tests and or laboratory tests.

4.0 TYPE OF FOUNDATION

All structures located on land are supported on a foundation system at or below the ground surface. Selection of foundation system to be used is essentially an economic study of alternatives.

In general, foundations can be grouped as shallow foundations and deep foundations. The most common types of shallow foundations are spread footings, continuous footings and mat foundations. These footings are used to spread highly concentrated column or bearing wall loads over a soil mass near the ground surface. Deep foundations include piles and pier foundations. Such foundations are used when the soils at shallow depth are unable to support the load from super structure. The load is transferred to deeper firm strata through piles and piers.

5.0 BEARING CAPACITY CONCEPT

The bearing capacity of a soil is defined as the maximum load that can be supported per unit of area of the soil without rupture or detrimental settlement. It is necessary to investigate both shear resistance and settlement for any structure. In many cases settlement criteria will control allowable bearing capacity. The recommendation for allowable bearing capacity to be used for design is based either on settlement consideration or on the ultimate bearing capacity. The ultimate bearing capacity is divided by a suitable factor of safety based on soil type and the accuracy of input data.

5.1 Standard Penetration Test

"Standard Penetration Test" is a widely used field test method for preliminary assessment of soil properties. In this method a hammer weighing 64 kg is dropped from a height of 75 cm. The hammer slides down a drill rod which is used as a guide. The number of blows N, necessary to produce a penetration of 30 cm. is regarded as penetration resistance.

Based on experience and/or other confirmatory tests the results of standard penetration tests can broadly be correlated to relevant soil properties.

Table 2, 3 & 4 show such correlations. The results given in the tables are approximate but more or less conservative and should be used with caution. The correlation for clays can be regarded as no more than a crude approximation, but that for sand is quite reliable. If the sand is very fine or contains large amounts of silt, and is located beneath water table, the correlation tables may indicate a relative density considerably greater than the actual relative density of the deposits. Under these conditions N value should be corrected for saturation before using the above tables.

Table 2. Relative Density of Granular Soil

Compactness	Relative Density (%)	SPT Values (N)	Angles of Internal Friction(ϕ)	Field Identification
Very Loose	0 - 15	0 - 4	0 - 28°	Reinforcing rod can be pushed into soil several cm.
Loose	15 - 35	4 - 10	28° - 30°	
Medium Dense	35 - 65	10 - 30	30° - 36°	
Dense	65 - 85	30 - 50	36° - 41°	Difficult to drive 50 x100mm stakes with a sledge hammer.
Very Dense	85 - 100	50 & Above	41° and above	

Table 3. Relationship Between Relative Density, Penetration Resistance and Angle of Internal Friction of Cohesionless Soils

State of Packing	Relative Density Dr	Standard Penetration Resistance N	Static Cone Resistance qc	Angle of Internal Friction ϕ
		blows per 30 cm	Kn/m^2	degrees
Very loose	<0.2	<4	196	<30
Loose	0.2 - 0.4	4 - 10	196 - 392	30 - 35
Compact	0.4 - 0.6	10 - 30	392 - 1176	35 - 40
Dense	0.6 - 0.8	30 - 50	1176 - 1960	40 - 45
Very Dense	>0.8	> 50	1960	>45

Table 4. Consistency of Cohesive Soil

Consistency	Unconfined Compressive Strength Kn/m^2	Standard Penetration N Blows/30 cm	Field Identification
Very Soft	Less than 2.45	Below 2	Easily penetrated several inches by fist
Soft	2.45 - 4.90	2 - 4	Easily penetrated by thumb
Medium Stiff	4.90 - 9.80	4 - 8	Can be penetrated by thumb with moderate effort.
Stiff	9.80 - 19.00	8 - 15	Readily indented by thumb but penetrated only by great effort.
Very stiff	19.60 - 39.21	15 - 30	Readily indented by thumb nail
Hard	over 39.21	over 30	Indented with difficulty by thumb nail.

NOTE : The measurement of consistency by means of the standard penetration test can be unreliable.

Although the standard penetration test cannot be regarded as a highly refined and completely reliable method of investigation, the "N" value gives a useful preliminary indication of the consistency or relative density of most soil deposits.

Standard penetration tests are generally made at 1.5 m intervals in the bore holes. An approximate depth and width of the foundation is assumed, and the average value of N is taken over a depth below foundation level equal to the width of footing. If several bore holes show different values of N, the lowest average should be taken to assess the value of N and to determine allowable bearing pressures.

During the process, number of hammer blows for each 15 cm of penetration is recorded. For standard penetration tests made at shallow depth, the number of blows are usually too low. At a greater depth, the same soil with same relative density would give higher penetration resistance. Influence of the weight of soil above (which is called over burden pressure) on the standard penetration resistance may be approximated by the following equation.

$$N' = N \frac{(50)}{P' + 10} \quad \text{.....(1)}$$

Where, N' = adjusted values of Standard Penetration resistance.
N = Standard Penetration resistance as actually recorded.
P' = effective overburden pressure, PSI, not exceeding 40, (Use buoyant weight for soil below water table).

In SI units the correction factor, $C_N = \frac{N'}{N}$

Can be obtained from Figure 4.

Standard penetration tests are not only useful for granular soils, they are also extensively used for other types of soils. For large and moderate jobs, both standard penetration tests and thin walled tube samples should be used. For smaller jobs, the foundation design may be based on the conservative values derived from standard penetration tests.

5.2 Bearing Capacity of Soils under Footing & Mat

The bearing capacity of granular soil depends upon its unit weight and the angle of internal friction. Compact soils are not very compressible and therefore, cause little settlement.

A) The allowable bearing pressure is based on ultimate capacity: The allowable pressure is equal to the ultimate bearing capacity divided by a factor of safety. A factor of safety equal to 3 is usually used under normal loading conditions and a factor of safety equal to 2 is used under combined maximum load. Teng suggested following empirical formula for determining the bearing capacity based on SPT values.

For sq. footing,

$$q_{ult} = 2\ N^2\ B\ Rw + 6\ (100 + N^2)\ DRw' \qquad \ldots.(2)$$

For very long footing,

$$q_{ult} = 3\ N^2\ B\ Rw + 5\ (100 + N^2)\ DRw' \qquad \ldots.(3)$$

Where,

q_{ult} = net ultimate bearing pressure in psf

N = Corrected SPT value

B = Width of footing (ft)

D = Depth of footing (ft), measured from ground surface to bottom of footing.

D should be measured from lowest ground level, if D > B, use D = B.

R_w', R_w : Correction factors for water level position. When water level is located at a depth 'B' below the bottom of footing R_w' = 1

When at or above the base R'_w = 0.5. Use interpolated values if located in between.

B. Allowable bearing pressure based on tolerable settlement (Peck and Terzaghi):

$$q_a = 720\ (N-3) \left[\frac{B + 1}{2B} \right]^2 R_w' \qquad \ldots.\ (4)$$

Where q_a = net allowable bearing pressure in psf for maximum settlement of 1 inch (2.5 cm). It should be taken as the pressure at the bottom of the footing in excess of the weight of soil immediately surrounding the footing. If the maximum tolerable settlement is different from 1 inch (2.5 cm), the equation may be modified on the assumption that settlement is proportional to the bearing pressure. The value of q_a may be increased linearly with depth of footing upto 100 percent when the depth is equal to the width of the footing.

In other words the above equation may be multiplied by a factor (1 + D/B) with a limiting value of 2 where D/B exceeds unity. The bearing capacity of a footing is largely affected by the characteristics of the volume of soil within a depth equal to about 1 to 1½ times the width of footing.

N should be corrected for overburden pressure.

5.3 Bearing Capacity of Clay and Clayey Soils

The ultimate bearing capacity of clayey soils depends primarily upon their consistency. For small jobs where a better economy can be achieved by using a conservative design based on simple test results, the Standard Penetration test is used. The N values of SPT should be adjusted for overburden pressure. The relationship between standard penetration resistance, consistency and allowable bearing capacity of clayey soils as indicated in **Table** 5 (Terzaghi and Peck, 1948) is approximate.

Table 5. Allowable bearing pressure

Consistency of soil	N (SPT)	Allowable bearing pressure Kn/m^2	
		Square Footing	Continuous Footing
Very soft	0-2	0.00 - 31.60	23.0
Soft	2-4	31.00 - 63.10	23.00- 47.41
Medium	4-8	63.1 -126.42	47.41- 94.81
Stiff	8-15	126.42 -252.84	94.81- 189.63
Very stiff	15-30	252.84 -505.68	189.63- 380.00
Hard	30+	505.68+	380.00+

Ultimate bearing capacity is equal to 3 times the allowable capacity.

5.4 Soil Pressure Calculation from Chart (for sand)

The chart in **Fig. 2** gives the allowable bearing capacity for sand based on SPT. The width 'B' may be taken as the side of a square footing, the smaller dimension of a rectangular footing, the width of a long continuous footing, or the diameter of a circular footing.

The chart applies to shallow footings ($D_f \le B$) resting on a uniform sand for which γ = 15.7 Kn/m^3 (100 lb/ft^3) and in which the water table is at too great a depth to influence the behaviour of footing. N values should be adjusted for overburden pressure.

The soil pressure can also be found from **Fig.** 3. Here settlement is considered to be 1 inch (2.5 cm). It is assumed that the water table is at least at a distance B below the foundation.

5.5 Influence of Water Table

Holtz and Gibbs (1957) in their Laboratory Study observed that N values have decreased by 25% in coarse sand and by 50-65 percent in fine sand due to submergence. If a saturated sand located beneath a footing is very loose ($N < 5$), the footing should be supported on piles, or sand should be compacted.

In order to determine the allowable soil pressure q_a by means of the chart (**Fig.** 2) the effect of submergence on settlement must be considered. According to theory the submergence of sand located beneath the base of a footing would approximately double the settlement if the base is located at or near the surface of the sand. If D_f/B is small, the values obtained in chart should be reduced by 50%, and if it is close to unity, the values in the chart may have to be reduced only by 33% because, the positive effect of surcharge on the settlement of footing is partly compensated by the negative effect of saturation.

For fine sands or silty soils under water the observed N values should be corrected by using the following formula.

$$N' = 15 + \frac{N - 15}{2} \qquad \text{.....(5)}$$

N' = computed equivalent value to be adopted in designs

N = actually observed number of blows

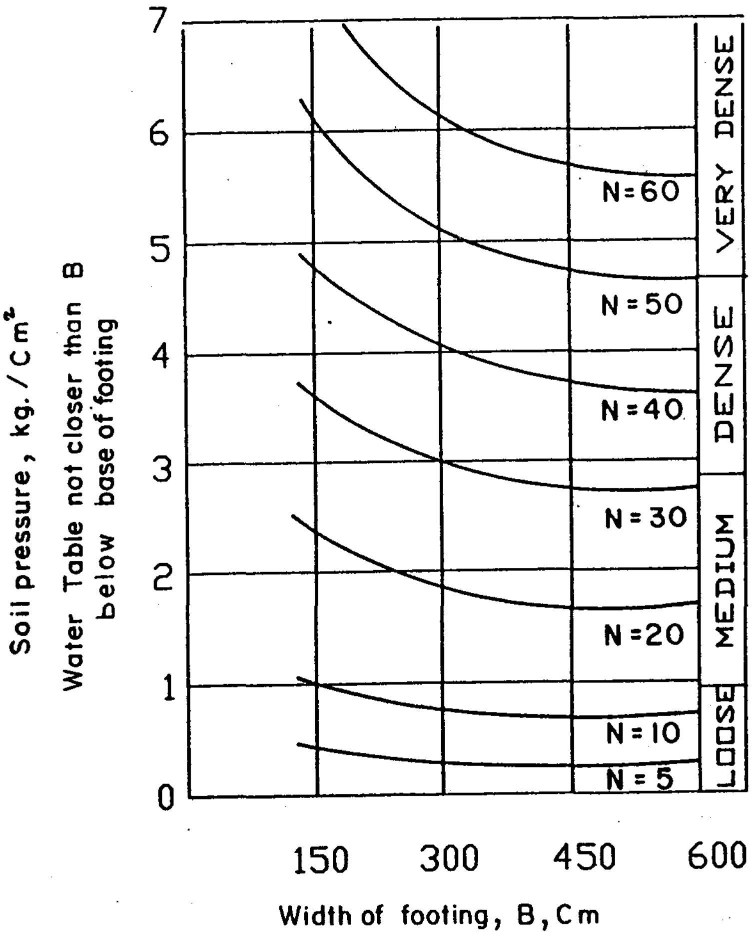


Fig. 2 Soil pressure corresponding to 2.5 cm settlement of footing on sand.

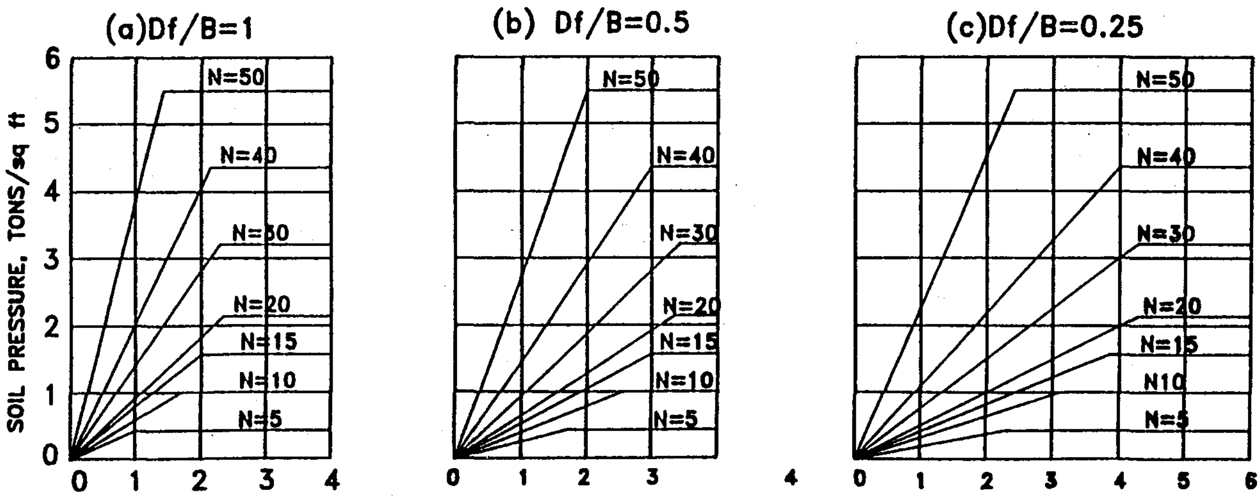


Fig. 3. Allowable Bearing Pressure of Footing on Sand from Settlement Consideration

Some engineers do not accept that it is necessary to halve the bearing pressures if the water table is close to foundation level. They consider that the test results themselves automatically allow for reduction in bearing pressure. Certainly some N-values when plotted with depth do show a reduction of about 15 percent when they pass from a dry or damp sand to a saturated sand. However, there is no conclusive evidence to suggest that it is safe to reject the established procedure of halving the bearing pressure for foundations at or below the water table.

5.6 Raft on Sand

Differential settlement for a raft foundation is less than that for a spread footing. So it is reasonable to permit larger total settlement and hence, higher allowable soil pressures for raft foundation. Experience has shown that a pressure approximately twice as great as that allowed for individual footings may be used because it does not lead to detrimental differential settlements. The maximum allowable settlement for raft may be about 2 inch (5 cm) instead of 1 inch (2.5 cm) as for a isolated footings.

The following empirical formula based on N value of SPT gives the allowable bearing capacity for a mat foundation:

$$q_a = 0.22\ N\ C_w, \quad \text{TSF} \qquad (5 < N < 50) \qquad \ldots\ldots(6)$$

The following correction factor for presence of water table should be used.

$$C_w = 0.5 + 0.5\ \frac{D_w}{D_f + B} \qquad \ldots\ldots.(7)$$

Where,

D_f = Depth of surcharge

D_w = Depth of water table

B = Width of footing

5.7 Footing on Clay

For maximum sustained loads that can reasonably be expected, F.S. should be taken as 3. The allowable bearing capacity for a footing on clay is given by:

$$q_d = 2.85\ q_u\ (1+0.3\ B/L) \quad .....(8)$$

$$q_a = 0.95\ q_u\ (1 + 0.3\ B/L) \quad .....(9)$$

q_d = net ultimate bearing capacity

q_a = net allowable bearing capacity

B = width, L = length

As a rough estimate, safe soil pressure may be considered equal to the unconfined compressive strength. If one or more soft layers are located even at a depth greater than 'B' below the base of footing, settlement computation should be made to ascertain whether the pressure at the top of any such soft layer exceeds the safe value for that layer. It may be assumed that the pressure at the base of the footing spreads out uniformly within the confines of a truncated prism with sides sloping outward from the edges of the footing at an angle of 60 with horizontal.

5.8 Raft on Clay

A raft on clay may fail in shear. The net ultimate bearing capacity may be determined from the following expression.

$$q_d = 5\ C\ (1 + 0.2\ B/L)\ (1 + 0.2\ D_f/B) \quad .....(10)$$

A factor of safety of 3 should be used to arrive at the safe bearing capacity.

By increasing the depth of excavation, the pressure that can safely be exerted by the structure is correspondingly increased. If a raft foundation is to be constructed at a site underlain by a clay deposit too soft to provide support at the normal basement level, the only practicable method for safely constructing the raft is to lower the elevation of its base.

5.9 Settlement from Standard Penetration Test

Terzaghi and Peck (1948) presented a family of curves that related the N value of SPT to the bearing pressure that would cause a settlement of 25 mm (1 in). Peck, Hansen and Thornburn (1974) have somewhat revised the Terzaghi and Peck curves to accommodate the findings of more recent researches and observations. The modified relationship is as follows:

$$q_a = C_w\ (0.41)\ N\ \Delta H \qquad .....(11)$$

When q_a = Allowable net bearing pressure in Kpa that will cause a settlement of ΔH in millimetres.

N = Average corrected Standard Penetration blow count

ΔH = Settlement in millimetre

Cw = Water table correction factor

$$Cw = 0.5 + 0.5\ \frac{Dw}{Df + B} \qquad .....(12)$$

Where Cw = Water table correction (0.5 - 1.00)

D_w = Depth of water table from ground surface

D_f = Depth at the base of footing from ground surface

B = Width of footing

Gibbs and Holtz (1957) showed that for a constant relative density the Standard Penetration blow count increased with increasing effective overburden pressure. Peck, Hansen and Thornburn (1974) suggested an effective overburden stress correction factor C for the N values to be used in Eq. 11. The correction is shown in **Fig. 4**. The recorded field value of N should be multiplied by C for use in Eq. 11. Since soil deposits are generally somewhat erratic, the 'N' value used in Eq. 11 should be based on the results of several test borings. When several borings are made the lowest average value should be used. Generally, safety factor of 3 is used for dead load plus the normal or sustained live load, and 2 for dead load plus maximum live load.

Meyerhof's (1956) studies reveal that allowable bearing pressures for a given settlement of shallow foundation on sand and gravel when estimated from standard or static cone penetration tests, are rather conservative. He observed that standard penetration tests are widely used to estimate the relative density of sand and they have been correlated with the results of plate loading tests and settlement observations on structures. On this basis the allowable bearing pressure in tons per square foot of spread footings on sand can be written approximately as:

$$Pa = \frac{N'}{8} Sa, \qquad \text{for } B < 4 \qquad .....(13)$$

$$\text{or } Pa = \frac{N'}{12} \left[\frac{B+1}{B}\right]^2 (Sa), \qquad \text{for } B > 4 \text{ ft} \qquad .....(14)$$

$$\text{and for rafts on sand, } Pa = \frac{N'}{12} (Sa) \qquad .....(15)$$

N' = corrected N value

$N/12 \left(\frac{B+1}{B}\right)^2 Sa$

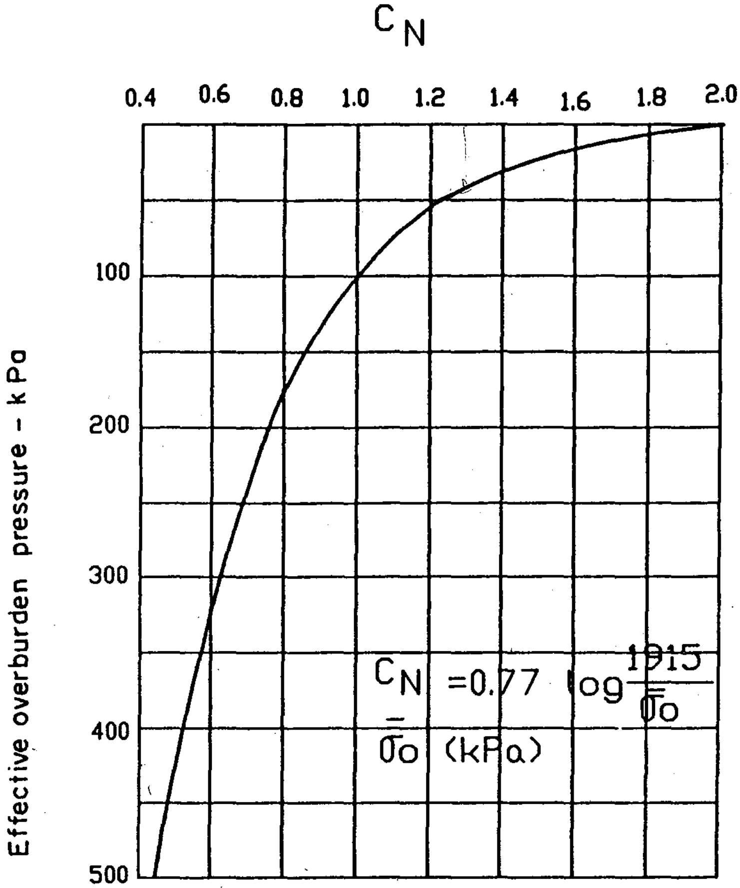


Fig. 4 Blow count correction factor for overburden pressure

Where,

B = foundation width, in feet,

N = SPT blow count per foot (corrected),

Sa = allowable settlement in inches.

Pa = allowable bearing pressure, TSF

This relationship is independent of the shape of the foundation. For a foundation depth approaching the width B, an increase of the allowable pressure by approximately 33% can be made.

As already mentioned SPT resistance depends not only on the relative density of the soil but also on the effective overburden pressure, ground-water conditions, and other factors. However, the analyses of settlement observations for buildings on sand as summarized in **Table 6** show that the calculated settlements were conservative even under submerged condition for which no correction was made. Comparison of observed and estimated settlements using Eqs. 13 to 15 is given in **Fig.** 5, which shows that the estimated settlements vary from approximately 1.5 to 3 times the observed values.

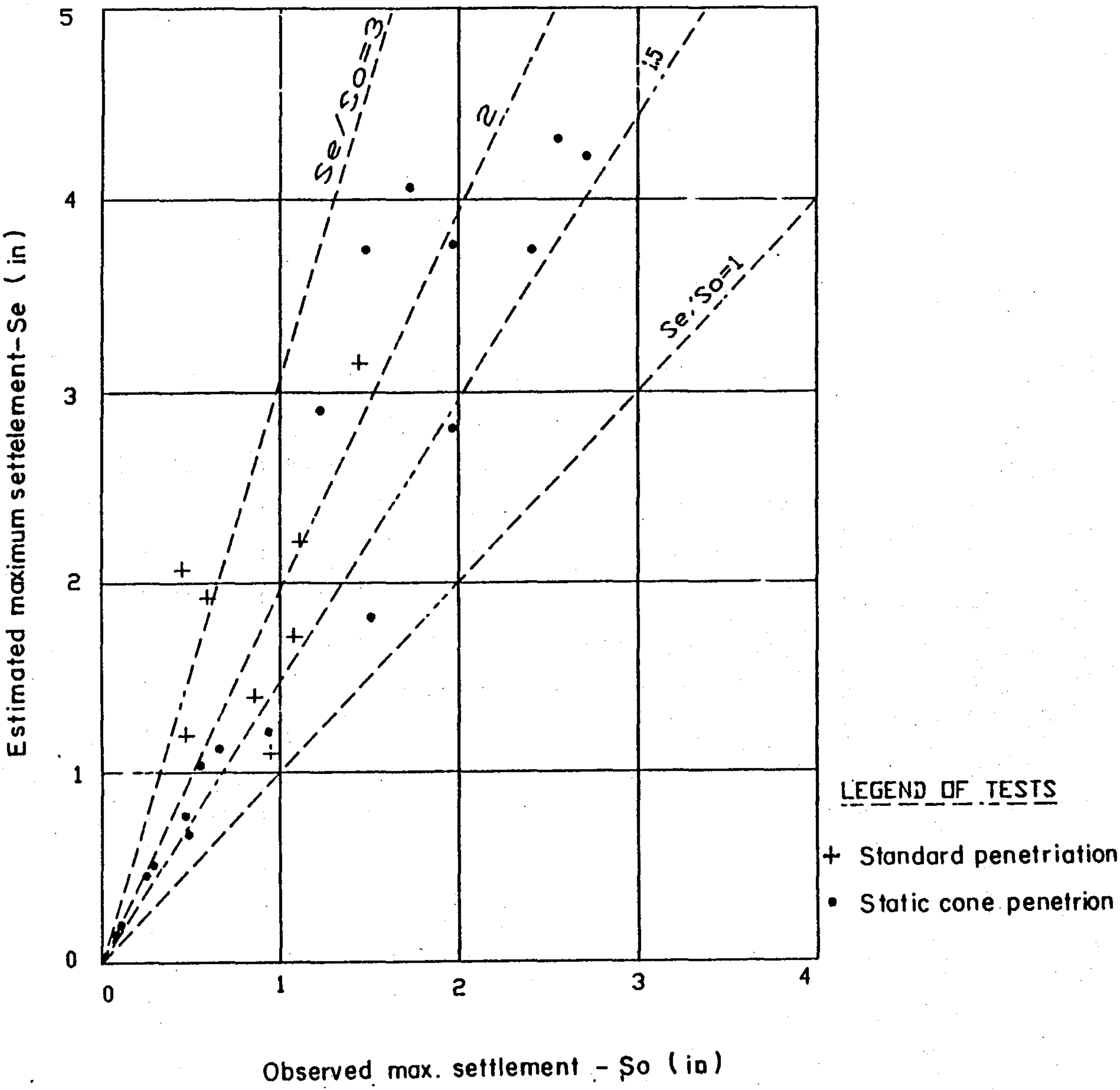


Fig. 5 Comparison of Observed and Estimated Settlement

Table 6. Comparison of Observed and Estimated Settlements of Shallow Foundations on Sand and Gravel

Structure	Reference	Foundation width in meter	Soil Type	Average standard penetration resistance in number of blows per 30 cm	Bearing pressure in Kn/m^2	Maximum Settlement		Ratio of estimated to observed settlement
						Observed, in cm	Estimated, in cm	
T.Edison, Edison, Sao Paulo	6	18.30	Fine clayey sand	15	254.84	1.52	4.80	3.2
Banco do Brasil Sao Paulo	6,8	22.86	Fine Clayey Sand	18	263.38	2.80	4.52	1.5
Imparanga, Sao Paulo	7	9.15	Fine-Medium Clayey Sand	9	242.31	3.56	7.87	2.2
C.B.I. Esplanada, Sao Paulo	8	14.63	Fine-Medium Clayey Sand	22	421.4	2.80	5.59	2.0
Thyssen, Dusseldorf	12	22.56	Sandy Gravel	25	263.38	2.41	3.00	1.3
Ministry, Dusseldorf	12	15.85	Coarse Gravel	20	242.31	2.16	3.50	1.6
Chimney, Cologne	12	20.42	Sandy Gravel	10	189.63	1.01	5.25	5.3

Static cone penetration (deep-sounding) tests have been used to estimate a compressibility index of sand.

5.10 Bearing Capacity of Caisson

A caisson is, in effect, a rigid mat foundation and is used to transfer load to a deeper strata. The allowable bearing capacity for caissons can also be estimated from the N value of SPT. The following equation are applicable for caissons too:

$$q_{ult} = 4 N^2 B R_w + 12 (100 + N^2) D R_w' \quad \text{(Granular soil)} \quad .....(16)$$

$$q_{ult} = C N_c \quad \text{(Cohesive soil)} \quad .....(17)$$

Where,

q_{ult} = Ultimate bearing capacity in psf.

N = corrected SPT value, D = Depth of caisson in feet.

R_w & R_w' = Reduction factor for water level.

C = Cohesion in psf.

N_c = Bearing capacity factor.

A factor of safety between 2 to 3 is used to obtain allowable capacity.

5.11 Design Examples:

Example: 1 vent regulator B = 2.45 m (width)

G. L. = + 6.10 m

F. L = 3.05 m

G W T max = + 2.45 m (Note soil is sandy).

N value at F. L. = 6

Gross pressure = 85 Kn/m^2

Soil unit wt. moist = 18.06 Kn/m^3

Soil unit wt. sat. = 18.85 Kn/m^3

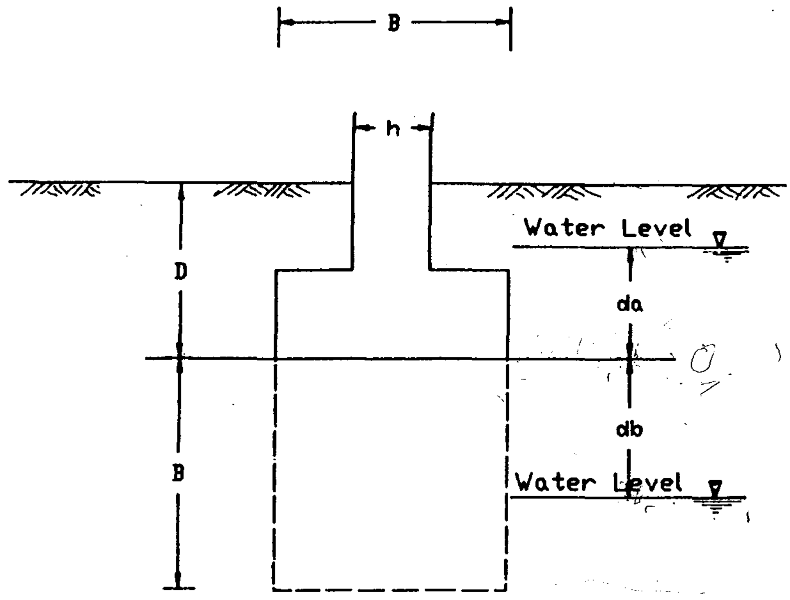
Overburden pressure = 3.05 x 18.06 = 55.09 Kn/m^2

Ref. **Fig.** 4, C_N = 1.20

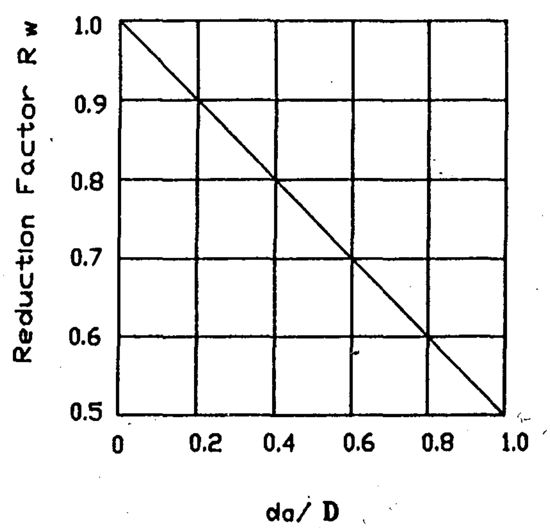
Corrected, 'N' value = 6 x 1.20 = 7.20

$q_a = C_w\ (0.41)\ N\ \Delta H$

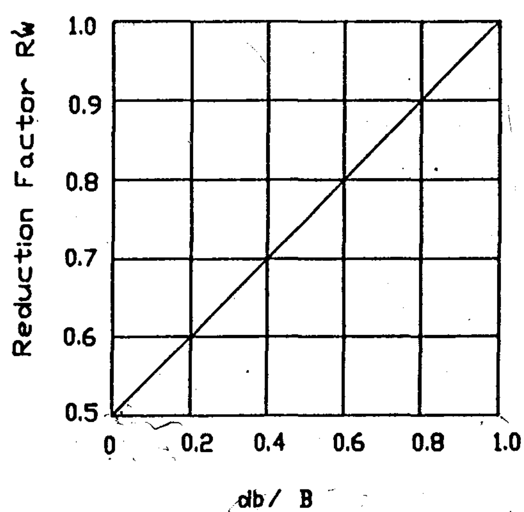
ΔH = 25 mm



a) Depth of water level with respect to dimension of footing



b) Water level above base of footing



c) Water level below base of footing

Fig. 6 Reduction Factor for Ground Water Level

$$C_w = 0.5 + 0.5 \frac{D_w}{D_f + B}$$

$$= 0.5 + 0.5 \frac{3.66}{3.05 + 2.45} = 0.50 + 0.33 = .83$$

$q_a = 0.83 \times 0.41 \times 7.2 \times 25 = 61.25 \text{ Kn/m}^2$

Net allowable foundation pressure for 25 mm settlement = 61.25 KN/m^2

Net foundation pressure = 85.00 - 55.09 = 29.91 Kn/m^2

So it is O.K. from settlement consideration.

If water table is located at 1.52 m below G.L.

Overburden pressure = 1.52 x 18.06 + 1.52 x 9.04 = 41.19 KN/m^2

C_N = 1.30 (From Fig. 4)

Corrected N value = 6 x 1.30

= 7.80

$$C_w = 0.5 + 0.5 \frac{1.52}{3.04 + 2.45} = 0.638$$

$q_a = .638 \times 0.41 \times 7.80 \times 25 = 51.08 \text{ Kn/m}^2$

Net foundation pressure = 85.00-41.19 = 43.81 Kn/m^2

It is just sufficient in consideration of 1 inch (25 m) settlement.

Considering raft foundation : (Trial - 2)

qa = 0.22 N = .22 x 7.8 = 1.72 Ton/sq ft.

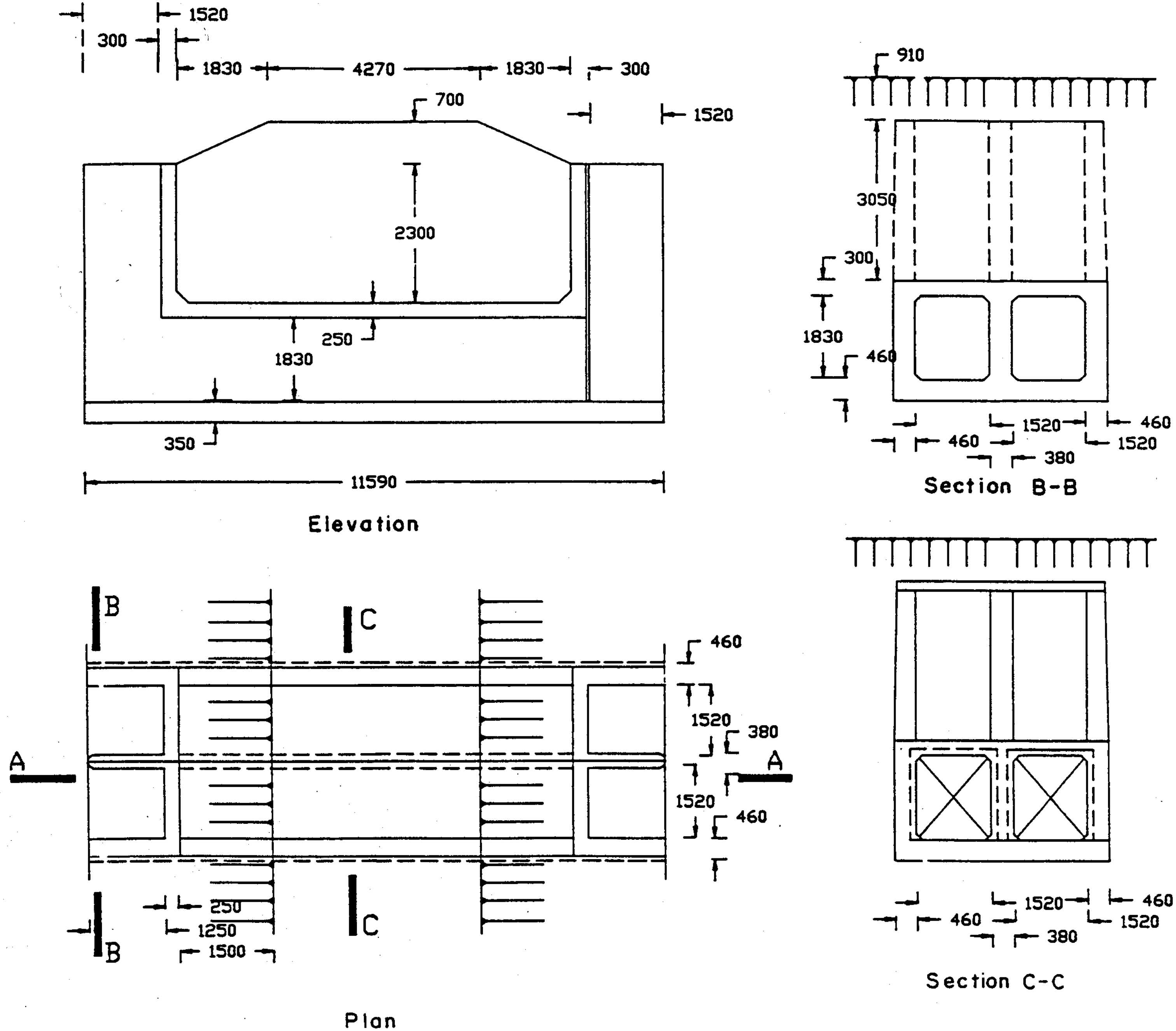


Fig. 7 Main part of a typical 2 v-(1.52 m x 1.83m) Sluice

Net soil pressure = 1.72 x 0.638 = 1.09 ton/sft (with water table correction).
= 114.83 Kn/m^2 So O.K.

(Trial - 3)

$$q_a = 720\ (7.8-3) \left[\frac{8+1}{16}\right]^2 \times 0.5$$

$$= 720 \times 4.8 \times 0.316 \times 0.5 = 546 \text{ lb/sft}$$

Increasing by a factor of $\left[1 + \frac{D_f}{B}\right]$

This factor is maximum 2

q_a = 546 x 2 $\simeq$ 1092 psf (Net)

For 37 mm settlement (1.5 inch).

q_a net =1638 psf

= 79 Kn/m^2

Net foundation pressure = 85.00 - 55.09 = 29.91 Kn/m^2

So it is O.K.

Again q_{ult} = $2\ N^2\ BR_w + 6(100 + N^2)\ DR_w'$

= 2 x 60.84 x 8 x 0.75 + 6(100 + N^2) x 10 x 0.50, da/D = 0.5, R_w = 0.75,

= 730 + 4825 = 5555 lb/sft R_w' = 0.50

qa = 5555/3 lb/sft = 1852 lb/sft = 87.10 Kn/m^2.

Again from equation 13,

$$P_a = N'S_a \left[\frac{B+1}{B}\right]^2$$ (N corrected for overburden)

$$P_a = \frac{7.8}{12} \times 1 \times \left[\frac{8+1}{8}\right]^2 = 1.64 \text{ Kip/sft.}$$

Increasing by a factor 1.30

Pa = 1.64 x 1.3 = 2100 lb/sft = 100.96 Kn/m^2. O.K.

Example: 2

3 vent regulator B = 21' - 0" (6.40m)

L = 24' - 0" (7.32m)

(Note : Soil is sandy)

Depth of surcharge 6' - 0" (1.83m)

N value = 5, G.W.L. at base i.e. 6'- 0" below

Gross pressure = 1800 lb/sft (85 Kn/m^2)

Solution 1:

Overburden pressure = 18.06 x 1.83 = 33.05 Kn/m^2

Net pressure = 85.00 - 33.05 = 51.95 Kn/m^2

C_N = 1.40

Corrected N value = 7.00

$q_a = C_W$ (0.41) 7.00 x 25

$$C_W = 0.5 + 0.5 \frac{1.83}{1.83 + 6.40} = 0.5 + 0.11 = 0.61$$

q_a = 0.61 x 0.41 x 7.00 x 25 = 43.76 Kn/m^2 <51.95 Kn/m^2.

This structure is not safe considering 25 mm settlement.

Considering the structure as raft more settlement may be allowed.

$$q_a = 720\ (7-3)\left[\frac{(21 + 1)}{42}\right]^2 \times 0.50 = 720 \times 4 \times 0.53^2 \times 0.5$$

$$= 395 \text{ lb/sft.}$$

Increasing by a factor $\left(1 + \dfrac{D_f}{B}\right) = \left(1 + \dfrac{6}{21}\right) = 1.28$

q_a = 395 x 1.28 = 506 lb/sft = 23.79 Kn/m^2, Not OK.

Again from settlement consideration,

$$P_a = N' \frac{S_a}{12} \times \left[\frac{B + 1}{B}\right]^2$$ (P_a = gross pressure in tons/sft)

$$= \frac{7 \times 1}{12} \times \left[\frac{21 + 1}{21}\right]^2 = 1280 \text{ lb/sft} = 61.54 \text{ Kn/m}^2, \quad \text{Not O.K.}$$

So, with N value 5 the foundation is not safe. It is suggested to examine the deeper soil layers. If the deeper layers are sandy with increasing SPT value, the top layer with lower N value may be replaced with medium dense sand.

The values obtained from the above empirical relations may be conservative, but when other information are not available these relations provide a very good tool for foundation design. While using these equations it has to be kept in mind that some of these relations are based on settlement consideration and some are based on bearing capacity failure.

PART B

DETAIL DESIGN BASED ON SOIL TEST RESULTS

1.0 BACKGROUND

Bangladesh Water Development Board is associated with the implementation of Flood Control, Drainage and Irrigation Projects of both large and small dimension. Activities of the Board are spread over the whole country.These Flood Control, Drainage and Irrigation schemes include construction of embankments and appurtenant structures.

In many cases during construction of these structures serious foundation problems had been encountered. During the early period of implementation of Coastal Embankment Project, much stress on foundation was not given because of very limited scope of soil exploration. At present Ground Water Circle of BWDB is wholly responsible for collection of sub-soil data. RRI Soil Testing Laboratory is responsible for performing soil tests analyzing data. However, specific opinion regarding the type of foundation is not incorporated in the report and in some cases no information of soil parameters other than bore-logs are supplied. As such, some times foundation design becomes very difficult.

1.1 General Type of Soil

Soils in different regions of Bangladesh are not same. In the northern part foundation soil is better and mostly contains sandy soils at shallow depths. As such special treatment is not required for normal structure. Moreover, this region has no tidal effect and foundation level is normally above ground water table. In the coastal zone foundation soil creates major problem. Soils of Satkhira and Khulna are very poor and need lot of attention and careful judgement during foundation design. In some parts of Chittagong which is also in coastal embankment project, soil condition is very poor and 0 SPT value has been encountered even at a depth of 25 m from ground level. In locations of poor soil special care is to be taken for design of foundation.

2.0 SHALLOW AND DEEP FOUNDATION

Foundation is broadly classified into two types; Shallow Foundation and Deep Foundation. A shallow foundation is one which is placed immediately beneath the lowest part of the super structure which it supports. The deep foundation is one in which the foundation must be placed at considerable depth below the lower part of the superstructure. Obviously, there is no sharp dividing line between the two classes. Generally foundation that has a width (B) equal to or greater than the depth of the foundation (D_f) is considered a shallow foundation. Shallow foundation includes spread footings and mats which transfer structural loads to the underlying soil at a relatively shallow depth below the lowest part of the super structure. Deep foundation includes piles and caissons which transfer structural loads through a foundation member to a deeper soil strata.

When soils occurring immediately below the structure are unsuitable to support shallow foundations, it is sometimes possible to use shallow foundations after the unsuitable soil is excavated and replaced by compacted fill, or after the unsuitable material is improved or stabilized in some manner by densification or by pre-compression.

3.0 REQUIREMENTS FOR A SATISFACTORY FOUNDATION

A satisfactory foundation for any given structure must meet the following:

a) It must be adequately safe against structural failure.
b) Differential settlement must not exceed certain limit beyond which the structure will be damaged.
c) Foundation must be feasible technically and economically and be practicable to build.

4.0 SELECTION OF PROPER FOUNDATION

The foundation of a structure is the interfacing element between the superstructure and the soil. The selection of foundation type depends on the mechanical behaviour of the subsoil encountered at the site, nature of loads to be supported and the allowable total and differential settlements. Following foundation types may be considered depending on soil type.

i) When the soil is of the expansive type a raft foundation or short piles foundation may be contemplated. Total and differential settlements should be carefully assessed.

ii) In sandy clayey silts, continuous footings or raft foundations are indicated.

iii) On river plains where the finest alluvial sediments are encountered, compensated foundations may be used and in some cases, use of piles or caissons may prove to be necessary.

iv) Soils consisting of fine or very fine sediments like silts and clays are encountered in deltaic areas. These deposits may be of medium to high and very high compressibility. Compensated foundation with or without friction piles may be used in compressible deposits extending to great depth.

In case of hydraulic structures in the coastal areas base of the foundation is in most cases below ground water table which reduces allowable bearing pressure. Moreover, due to surface pumping during construction, soil becomes more disturbed and does not reflect the condition as shown in the bore logs. Box part i.e, middle part of hydraulic structures are kept separated from the u/s apron and d/s apron part as there is considerable difference of loading. These parts with different loading are separated by a separation joint or an expansion joint. A PVC water stop is provided across the joint to check seepage. If the differential settlement is more than 25 mm, it may cause tearing of PVC water stop and erosion of the underlying soil. So, the foundation should be so designed that the differential settlement does not exceed certain specified limit.

5.0 SOME COMMON TERMS USED

TRUE SPECIFIC GRAVITY (G) is defined as the ratio of the weight of the solid constituents of a soil mass, exclusive of voids, to the weight of an equal volume of water. Average specific gravity of the predominating solids in a soil is 2.65 for sandy soils and 2.70 for clays.

POROSITY (n) is the ratio of the volume of voids (V_v) to the volume of soil aggregate (V)

$$n = \frac{Vv}{V} \qquad Vv = \text{Volume of voids} \qquad \text{....(18)}$$

$$V = \text{Volume of soil}$$

VOID RATIO (e) is the ratio of the volume of voids (V) to the volume of solids (Vs) in a soil mass

$$e = \frac{Vv}{Vs} = \frac{Vv}{V - Vv}, \quad \text{Also, } e = \frac{n}{1 - n} \qquad \text{....(19)}$$

NATURAL MOISTURE CONTENT (w) is the amount of water expressed as a percentage of the weight of moisture in a soil (W) to the dry weight of soil solids (Ws).

$$w\ \% = \frac{W}{W_s} \times 100 \qquad \text{....(20)}$$

DEGREE OF SATURATION (Sr) is the ratio of the volume of water (V_w) in a soil to the volume of voids (V_v) expressed as a percentage.

$$Sr(\%) = \frac{Vw}{Vv} \times 100 \qquad \text{....(21)}$$

UNIT WEIGHT OR DENSITY (γ) is the weight of soil per unit volume. The unit weight of soil may vary depending compactness of the soil. The correlation between blow count and relative density for sands is quite reliable. For the purpose of foundation design, the relationships shown in **Table** 2 between the relative density, penetration resistance, N and the angle of internal friction ϕ, may be used.

CONSISTENCY is the most significant aggregate property of fine grained cohesive soil and is comparable to the relative density of coarse grained soils. Expressed qualitatively, the consistency of a soil is referred to as very soft, medium, stiff or hard. Quantitatively, it is related to the unconfined compressive strength of the soil.

PLASTICITY of cohesive soils is defined by arbitrary indices. These are liquid limit, plastic limit and plasticity index.

LIQUID LIMIT (LL) is the water % content at which the soil changes from liquid state to plastic state.

PLASTIC LIMIT (PL) is the water % content at which soil changes from plastic state to solid state.

PLASTICITY INDEX (PI) is the difference between the liquid limit and plastic limit i.e. , PI = LL - PL. It represents the range of water content in which the soil remains plastic.

The liquid limit and the plasticity index together constitute a measure of the plasticity of a soil. Soils possessing large values of LL and PL are said to be highly plastic or fat. Such soils are highly compressible.

<u>**SENSITIVITY**</u> is a property of cohesive soils. Cohesive soil often lose a portion of its shear strength upon disturbance. The amount of strength loss due to disturbance is expressed in terms of sensitivity. Sensitivity is the ratio of unconfined compressive strength of an undisturbed clay sample to the unconfined compressive strength of the same sample when remoulded. A cohesive soil is described as insensitive, sensitive and extra sensitive according to its sensitivity. The sensitivity of most clays range between 2 and 4. Clays having an index greater than 4 are called sensitive and those over 8 are called extra-sensitive. Sensitive soils may lose their strength due to construction operations.

<u>**COMPRESSIBILITY**</u> defines the settlement properties of soil. The amount of settlement a structure will experience is related to the stress-strain characteristics of the foundation sub-soil. When a cohesive soil is subjected to compression, water and air from the voids are extruded very slowly and the process of compression called consolidation continues for a long period of time. The amount of such compression depends upon the compression index C_c, and initial void ratio which are determined by laboratory consolidation test. For the purpose of approximate calculations of C_c, the following empirical relation may be used for normally loaded clay.

$C_c = 0.009\ (LL - 10)$, Where LL = Liquid limit (Peck, 1953) or

$C_c = 0.30\ (e_o - 0.27)$.

e_o = natural void ratio of soil in place (Hough, 1957)

6.0 BEARING CAPACITY CONCEPT

The bearing capacity of a soil is defined as the maximum load that can be supported per unit area of soil without rupture. It is necessary to investigate both base shear resistance and settlement for any structure. In many cases settlement criteria will control allowable bearing capacity. Allowable bearing capacity to be used for design is based either on settlement considerations or on the ultimate bearing capacity. The ultimate bearing capacity is divided by a suitable factor of safety based on the type of soil and accuracy of input data.

The factor of safety is used to safeguard against:

(a) Natural variations in the shear strength of the soil

(b) Uncertainties in the accuracy or reliability of theoretical or empirical methods for calculating bearing capacities.

(c) Effects of local loose pockets or unsuitable soils on the presumed bearing capacity during or subsequent to construction.

(d) Excessive settlement caused by yielding of the soil when the foundation is approaching failure in shear.

Of the above, variation in soil condition is the main reason that call for a factor of safety. A high degree of judgement is required for selecting design values of shear strength of soil on sites where test results are widely scattered. A safety factor of 2.5 to 3, usually 3 is generally used to cover the variations or uncertainties mentioned above. A value of 2 would only be adopted on sites where very uniform soil conditions are found. Low safety factors of 1.5 to 2 are also used in the design of temporary works such as cribs supporting construction plant, or for calculation of the allowable bearing pressure for earthworks where the settlement is not detrimental to temporary works or adjoining property.

It should be noted that safety factors used in calculations to determine safe bearing pressures are not normally intended to cover accidental increases in structural loading.

7.0 BEARING CAPACITY EQUATIONS

In the past, foundation design procedures were mainly empirical and were based largely on allowable bearing pressures for various soil types. The study of foundation behaviour in the field amplified by laboratory research has led to more rational approach. It is now recognized that the sub-soil investigation and soil test results should form the basis of foundation design.

One of the early set of bearing capacity equations was proposed by Terzaghi (1943). He modified a bearing capacity equation developed for an infinite strip by Prandlt (1920) based on the theory of plasticity. No exact mathematical approach has been devised for the analysis of bearing capacity failure in soil. Many methods have been formulated but all involved some simplifying approximations regarding soil properties and soil movement. An analysis of the condition of complete bearing capacity failure can be made by assuming that the soil behaves like an ideally plastic material. Equations were further developed by Meyerhof (1951, 1963), Hansen (1957,1970), Bousine and others.

The general bearing capacity formula given by Terzaghi is

$$q_d = \alpha\ C\ N_c + \gamma\ D_f\ N_q + \beta\ \gamma\ B\ N_\gamma \qquad ....(22)$$

Where, C = cohesion of soil

N_c, N_q & N_γ are dimensionless bearing Capacity factors that depend on the angle of internal friction of soil, ϕ .

Values of N_c, N_q and N_γ as suggested by Terzaghi are presented in tables 7 & 8. The above equation is suitable for the case of general shear failure where the soil is initially more compact but gradual settlements develop full plastic zones. For local shear failure in soft and loose soils, Terzaghi suggested modified values for bearing capacity factors as N_c', N_q' and N_γ'. Generally for N values 5 or less, local shear failure is considered.

Terzaghi has suggested following semi- empirical equations for circular and square footings.

For circular footing of diameter B,

$$q = 1.2\ C\ N_c + \gamma\ D_f\ N_q + 0.3\ \gamma\ B\ N_\gamma \qquad ....(23)$$

for square footing of size B,

$$q = 1.3\ C\ N_c + \gamma\ D_f\ N_q + 0.4\ B\ \gamma\ N_\gamma \qquad ....(24)$$

The above equations are not applicable for very deep foundations.

Table 7. Shape Factors for Terzhagi's Equation

Shape of footing	Continuous	Square	Rectangular	Circular
α	1.0	1.3	$1 + 0.3 \frac{B}{L}$	1.20
β	0.5	0.4	$0.5 - 0.3 \frac{B}{L}$	0.3

Table 8. Bearing Capacity Factors for Terzhagi's Equation

ϕ	N_c	N_q	N_γ
5	5.7	1.0	0.0
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
34	52.6	36.5	35.0
35	57.8	41.4	42.4
40	95.7	81.3	100.4
45	172.3	173.3	297.5
48	258.3	287.9	780.1
50	347.5	415.1	1153.2

Since bearing capacity depends on the shape and depth of footing, and the inclination of load, Hansen introduced some parameters to account for these factors.

$$q = C N_c S_c d_c i_c + \gamma D_f N_q S_q d_q i_q + 1/2 \gamma B N_\gamma S_\gamma D_\gamma i_\gamma \quad ....(25)$$

Where c = cohesion of soil

γD_f = surcharge

d = depth factor

i = inclination factor

s = shape factor

These values are given in **Table** 9 and 10.

Many other equations have been developed for calculating the bearing capacity of soil but these are very less commonly used and hence not mentioned here.

The ultimate bearing capacity equation has three independent terms. The 1st term is a function of soil shear strength parameters C & ϕ, the 2nd term is a function of the vertical confining effective stress at the footing grade elevation, and the third term is a function of the width of the footing. If the water table is at foundation level the reduction factor will be taken as 0.50. Suggested values of bearing capacities for different soils are given in **Table** 11 (IS).

Table 9. Bearing Capacity Factors for Hansen's Equation

ϕ	Nc	Nq	$N\gamma$	Nq/Nc	tan ϕ
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.03	1.20	0.13	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.16
10	8.36	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.40	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.69	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.91	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84

Table 10. Shape and Depth Factors for Hansen's Equation

Shape Factors :	$S_c' = 0.2\ (B/L)$	
	$S_c' = 1 + N_qB/N_cL$	
	$S_q = 1 + (B/L) \tan\phi$	
	$S_\gamma = 1 - 0.4\ (B/L)$	
Depth Factors :	$d_c' = 0.4\ (D/B)$	$D < B$
	$d_c' = 0.4 \tan^{-1}(D/B)$	$D > B$
	$d_c = 1 + 0.4\ (D/B)$	$D < B$
	$d_c = 1 + 0.4 \tan^{-1}(D/B)$	$D > B$
	$d_q = 1 + 2 \tan\phi\ (1-\sin\phi)^2\ (D/B)$	$D < B$
	$d_q = 1 + 2 \tan\phi\ (1-\sin\phi)^2 \tan^{-1}(D/B)$	$D > B$
	$d_\gamma = 1.00$ for all ϕ	

Table 11. Values of Safe Bearing Capacity According to Indian Standard (IS 1904 - 1961)

Coarse sand, compact & dry	45 t/m^2
Medium Sand, compact & dry	25
Fine sand & silt	15
Loose* Coarse and medium sand	25
Loose fine and dry sand	10
Hard or stiff clay	45
Medium clay (indented with thumb nail)	25
Moist clay (strong thumb pressure)	15
Soft clay (moderate thumb pressure)	10
Very soft clay (penetrated several inches by thumb)	5

* If a pointed picket 5 cm x 5 cm pushed down by a man, penetrates more than 20 cm, the sand is said to be loose.

* 1 t/m^2 = 9.80 Kn/m^2

8.0 CHOICE OF FOUNDATION

If the sub-soil is loose and bearing capacity is not adequate to withstand the load of the structure, foundation design becomes rather complicated. Soil condition at a particular site is not generally found ideal and economic foundation design may not be possible for all cases. This observation applies not only to design but also to construction problems. Moreover an economic solution may not be feasible from the construction view point. Foundation soil may be composed of medium dense sand, or may be very soft in the top layers underlain by dense strata or there may be a dense soil mass in between comparatively weaker layers. There may be other combinations too. Each situation will demand specific consideration.

Very often in coastal areas loose sand of shallow depth overlying deep layers of soft clay is found. In such case the type of foundation to be adopted will depend upon the load and the type of structure. For light structure, whose foundation width is very narrow and there is no interference between two foundations, individual footings might offer the most economical solution. But when the load is very heavy and the sand layer is very thin, the settlement of clay layer below may cause large differential settlements of magnitudes exceeding the allowable limit. So it may be necessary to carry the load on bearing piles passing through the clay strata to the dense sand below.

In some cases a stiff crust underlain by soft clays may be found. The top crust may be formed due to over consolidation. Foundation on such ground depends upon load. If the structure is comparatively light the foundation could be placed on the hard soil in the form of individual footings. However, settlement in the clay layer under the hard soil should be checked. If the load is very high, deep foundation may be be necessary.

If there are loose fills resting on hard strata, there is no question of individual footing. In such case pile foundation or pier foundation is the suggested.

9.0 ALLOWABLE SOIL PRESSURE ON SUB SOIL

As the load on a given foundation increases, the soil underneath deforms and the foundation settles. At low pressure the settlement may increase in direct proportion to the load but with increasing load it will be higher. The relation between settlement and average load per unit area may be represented by a load settlement curve shown in **Fig.** 8. If the soil is fairly dense or stiff, the settlement curve is similar to curve 1 and if it is loose or fairly soft, the settlement curve is similar to curve 2.

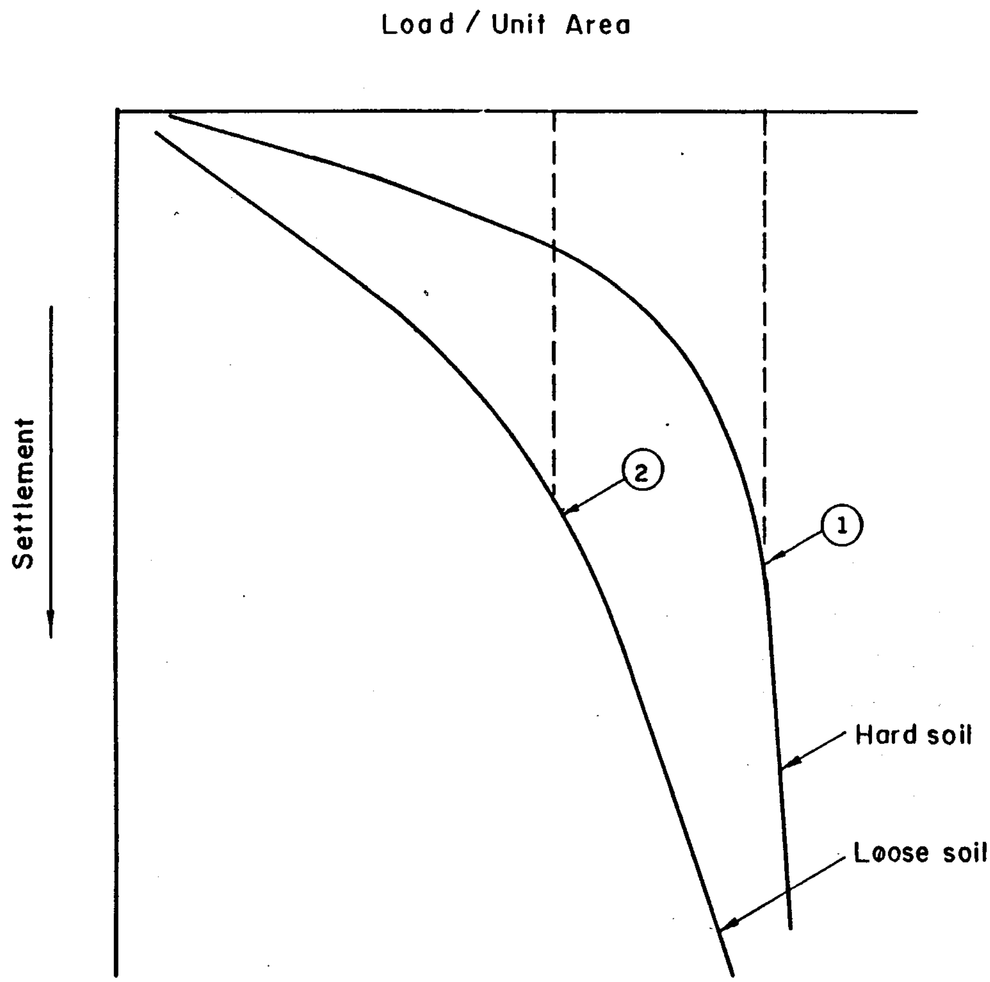
If the load becomes sufficiently great, the movement of settlement may be excessive or uncontrollably large and the foundation is said to have broken into the ground or to have experienced a bearing capacity failure.

Every foundation, therefore, should satisfy two conditions:

a) The factor of safety with respect to breaking into the ground should not be less than about 3.

b) The deformation of the base of the structure due to unequal settlement should not be so great as will damage the structure.

The allowable soil pressure on cohesionless soil is usually determined from settlement consideration.



Fig, 8 Load settlement curve

10.0 RATIONAL DESIGN METHOD

The rational approach to foundation design is essentially a trial evaluation - revision technique. A trial design is assumed on the basis of experience. It is analyzed to determine how well it meets the requirements at a minimum cost. The use of this method requires extensive and accurate information regarding the soil and the super structure. The soil data includes the depth and thickness of soil strata, location of the ground water table and physical properties of the soil particularly its strength and compressibility. The variations in the properties of the soil from one part of the site to another govern the way in which the analysis for bearing capacity and settlement are made. The following sections will concentrate on discussion of footings on sand, clay and mixed soils.

11.0 FOOTING ON SAND

In order to establish reliable criteria for design of footings on sand, the allowable soil pressure must be correlated to the properties and conditions that have significant influences on the behaviour of sand under load. The relative density of the sand and the position of water table with respect to the footing play very important role in this respect.

$$\text{Relative density, } D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 \qquad \ldots(26)$$

e_{max} = void ratio in the loosest state

e = in place void ratio

e_{min} = void ratio in the densest state

The relative density has a decisive influence on the angle of internal friction and the shape of the load settlement curve. The position of water table with reference to the base of footing also has an influence on both the ultimate bearing capacity and settlement. The allowable load for cohesive soil based on settlement consideration is determined by means of consolidation test on undisturbed samples and computing settlement by Terzaghi's theory and thereby limiting the settlement within permissible limits. For sandy soils, undisturbed samples being not possible, the allowable load is found out by standard Terzaghi's curves (Terzaghi & Peck, 1948) based upon in-situ penetration tests and load tests. These curves (**Fig.** 2) have been drawn on the supposition that total and differential settlement of the footing would be limited to 2.5 cm and 1.25 cm respectively, and that the water table exists at a depth equal to twice the base width B of the footing below the foundation level. Terzaghi assumes that if sand beneath the base of a footing is submersed the settlement would be double. Meyerhof (1956), has pointed out that due to water table the bearing capacity should be reduced linearly from 0 to 0.5, if the water table rises from a depth of 2B to zero below the base of footing.

This process of determining bearing capacity does not take into account the reduction in the values of penetration resistance due to ground water. Thus the allowable load obtained on the basis of settlement criteria gives somewhat conservative results.

12.0 FOOTING ON HOMOGENEOUS CLAY

It has already been mentioned that the allowable soil pressure on clay, as well as that for sand, should satisfy two requirements that the factor of safety against bearing capacity failures should be adequate and the settlement produced by the load should be within tolerable limits.

The net ultimate bearing capacity q_d, per unit area of footing resting on clay may be calculated from

$$q_d \text{ net} = 5c(1 + 0.2\ D_f/B)(1 + 0.2\ B/L) \qquad \text{....(27)}$$

where
- B = width of footing
- L = length of footing
- D_f = depth of foundation

If Df/B is greater than 2, it should be taken as 2. For a circular footing, diameter D = B = L. Under normal condition the factor of safety for footing on clay shall not be smaller than 3. For computing bearing capacity of clay, it is necessary to determine the undrained shear strength of the clay below the proposed footing. The undisturbed samples are collected by using thin walled Shelby tubes of 50 to 75 mm diameter. The average shear strength C of the soil can be estimated from unconfined compression test. The value of ultimate bearing capacity is divided by factor of safety to find the allowable bearing capacity.

The allowable soil pressure q_a, for clay can be assumed to be equal to one third of the value of q_d net determined from equation 27.

If the footing rests on a normally loaded clay, the magnitude of both total and the differential settlement can be very large. Fortunately foundations on normally loaded clay are not usual. In most localities, even the soft clays are pre-compressed to some extent, either by densification or by temporary lowering of water table.

12.1 Foundation located on firm soil above soft clays

Let us consider the case of a footing resting on a firm stratum, located above a soft Stratum. If the upper boundary of the soft stratum is located close to the base of the footing, the footing may break through the firm layer into the soft deposit. This can be avoided by so selecting the footing dimension that the pressure on the upper boundary of the soft stratum does not exceed the allowable bearing value for that stratum. The pressure at a certain depth below the footing can be calculated by Bousinesq's equations or Newmark influence chart (**Fig.** 9). The total footing load is assumed to be uniformly distributed over the base of a truncated pyramid where the sides slope from the edges of the footing at an angle of 60° with horizontal (**Fig.** 10).

If the upper boundary of the soft stratum is located at a considerable depth below the base of the footing, failure by breaking into ground may not occur since, in this case, the upper stratum would act like a thick mat or raft.

12.2 Stability analysis for foundation on clay

For analyzing the stability of foundations on clay it is necessary to understand the basic difference in the behaviour of soft clays and stiff over consolidated clays. In soft clays positive pore pressures are developed in a relatively short time when the load is applied.

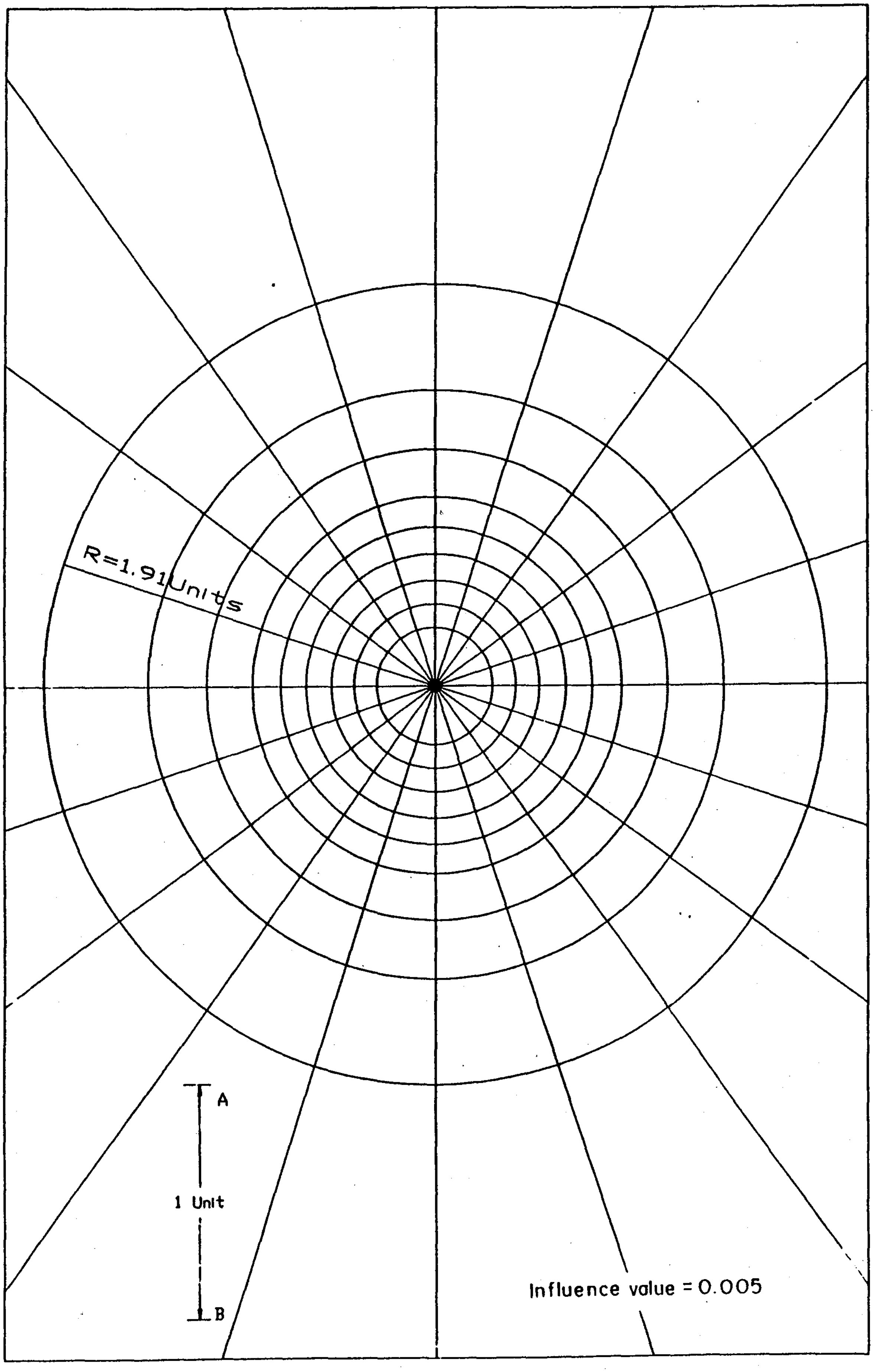


Fig. 9 Newmark Influence Chart.

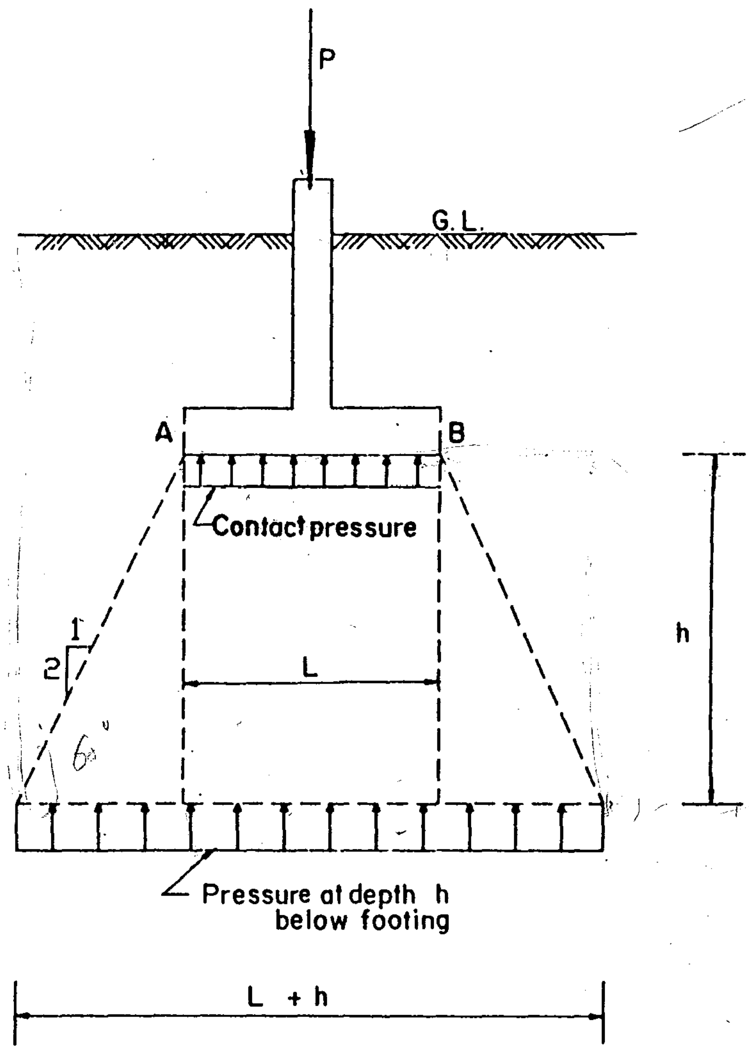


Fig. 10 Approximate distribution of pressure within soil mass under footing

Hence the critical condition generally occurs immediately after the loading. In over consolidated clays the pore pressures are generally small and sometimes negative and long term stability is dictated. In soft clays, influence of creep is compensated by dissipation of pore pressure. There are many examples of failure by creep in foundations resting on soft saturated clays.

Cohesive soils are virtually incompressible in undrained shear and the angle of internal friction is close to zero. Hence the basic assumptions underlying theory of plasticity are rigorously valid.

The dispersion of load through a sandy layer overlying a soft clay layer is not clearly understood. According to work of Yuan Teheng (1957) the influence of the sand layer on the ultimate bearing capacity could be approximated by increasing the bearing capacity by a factor 1 / (1 - 0.3 x h/b), where h is the depth of sand layer and b is the width of footing. Experience suggests that dispersion is considerably greater. Model test has indicated that dispersion angle is at least 30 to 45 for ultimate failure. However, it is understood that sand in model under small confining pressure behaves quite differently from sand layers at considerable depth under field conditions.

13.0 SETTLEMENT ANALYSIS

Magnitude of vertical displacements is important for the foundation engineer. . When a mass of soil is subjected to a foundation load it undergoes some settlement which may not occur instantaneously. Though a part of the total settlement takes place as soon as the load is imposed on the soil, usually the greater part develops slowly.

During settlement analysis it should be remembered that :

a) The soil should not fail in shear under the external loads.

b) Total and differential settlements of the structure resulting from the compression and displacements of the soil below should not endanger the safety of the structure or damage the utility services.

The most important causes of the settlement of structures are those involving the mechanical properties of the soil under loads which are good enough to produce shear distortion and consolidation. This can be predicted with reasonable limits by an analysis of the stress developed in the soil and of the reaction of the soil to stresses. Any settlement calculation at best be approximated. In addition, all the settlement calculations are generally based on assumption that foundation loads are known in magnitude and distribution.

13.1 Evaluation of Settlements

Presently settlement calculations are based on the assumption of one dimensional compression of all soil layers. The assumption is correct for deep seated layers only. In case of very shallow and small sized foundations, the settlement may be due to lateral bulging of the soil immediately below the foundation. In such cases there is no simple and reliable method of estimating settlement. In such case the bearing capacity criteria mostly governs the design.

Computations for Settlement of Clay

For saturated clay the settlement may be computed by the equation.

$$S = \frac{H}{1 + eo} \, Cc \log_{10} \frac{Po + \Delta P}{Po} \qquad \ldots.(28)$$

where S = Settlement of the layer in cm

ΔP = Increase in vertical stress in the layer in Kn/m^2

H = Thickness of the layer in cm

eo = Initial void ratio

Po = Initial vertical pressure in Kn/m^2

Cc = compression index of soil layer

Value of ΔP always corresponds to the middle of the layer. This equation is applicable only to normally consolidated soils i.e, the maximum previous consolidation pressure is equal to present initial overburden pressure. If the soil is pre-consolidated, a more realistic value of settlement may be computed by using the pre-consolidation pressure Po' rather than the existing overburden pressure Po in the settlement equation.

Terzaghi and Peck proposed that if the ratio of $\frac{\Delta P}{(Po' - Po)} < 0.5$, the compression of stratum would be from 10 to 25 percent of the computed value using overburden pressure Po, and if the ratio $\frac{\Delta P}{(Po' - Po)} > 1$ the computed value of settlement could be used. (Po' = consolidation pressure from e-log P curve).

13.2 Primary Consolidation Settlements

Primary consolidation settlement occurs due to excess pore pressure dissipation caused by stress change in the stratum of interest. Additional settlement termed as secondary compression continues for longer time.

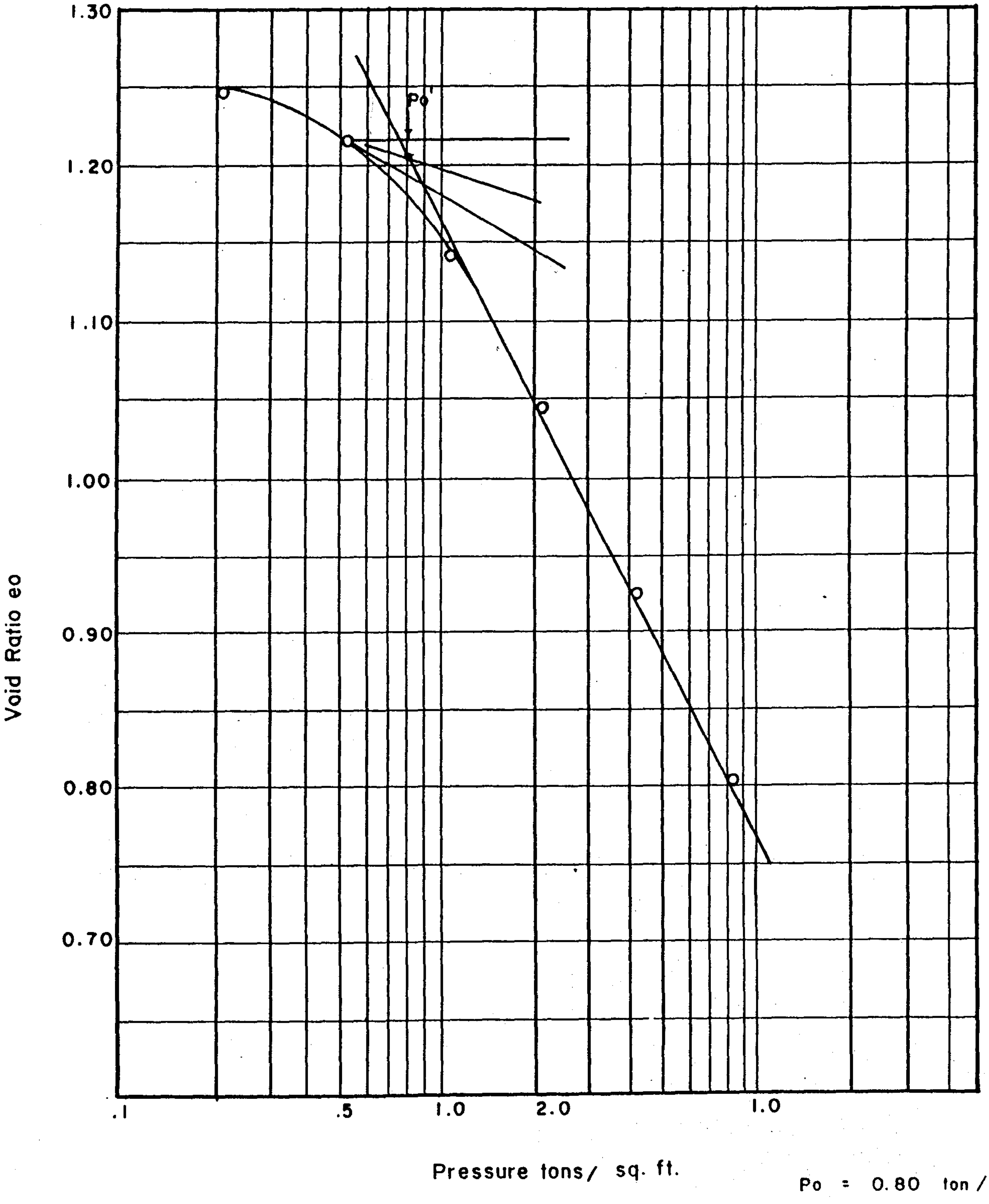
13.3 Secondary Compression

When the excess pore-pressure due to consolidation has been dissipated, the change of void ratio continues but generally at a reduced rate. During secondary compression some of the highly viscous water in contact with soil particles is forced. In many soil deposits secondary compression is so small that it can be neglected.

13.4 How to Estimate whether the Soil is Pre-consolidated or not

Casagrande (1936) developed a method for estimating the maximum past effective stress from the laboratory compression curve of an undisturbed sample. He developed this method from observations during slow cyclic compression tests on undisturbed clay samples. The basic steps in Casagrande's method are illustrated in **Fig.** 11 & 12. The step by step procedures are:

1. Plot the laboratory compression curve on semi-log paper.
2. Determine by eye estimation the sharpest curvature.
3. Draw a tangent at the curvature point.
4. Draw a horizontal line through the tangent point and bisect the angle produced.



Po = 0.80 ton/sft

Cc = 0.32

eo = 1.28

Fig. 11 Estimating Preconsolidation Pressure (1)

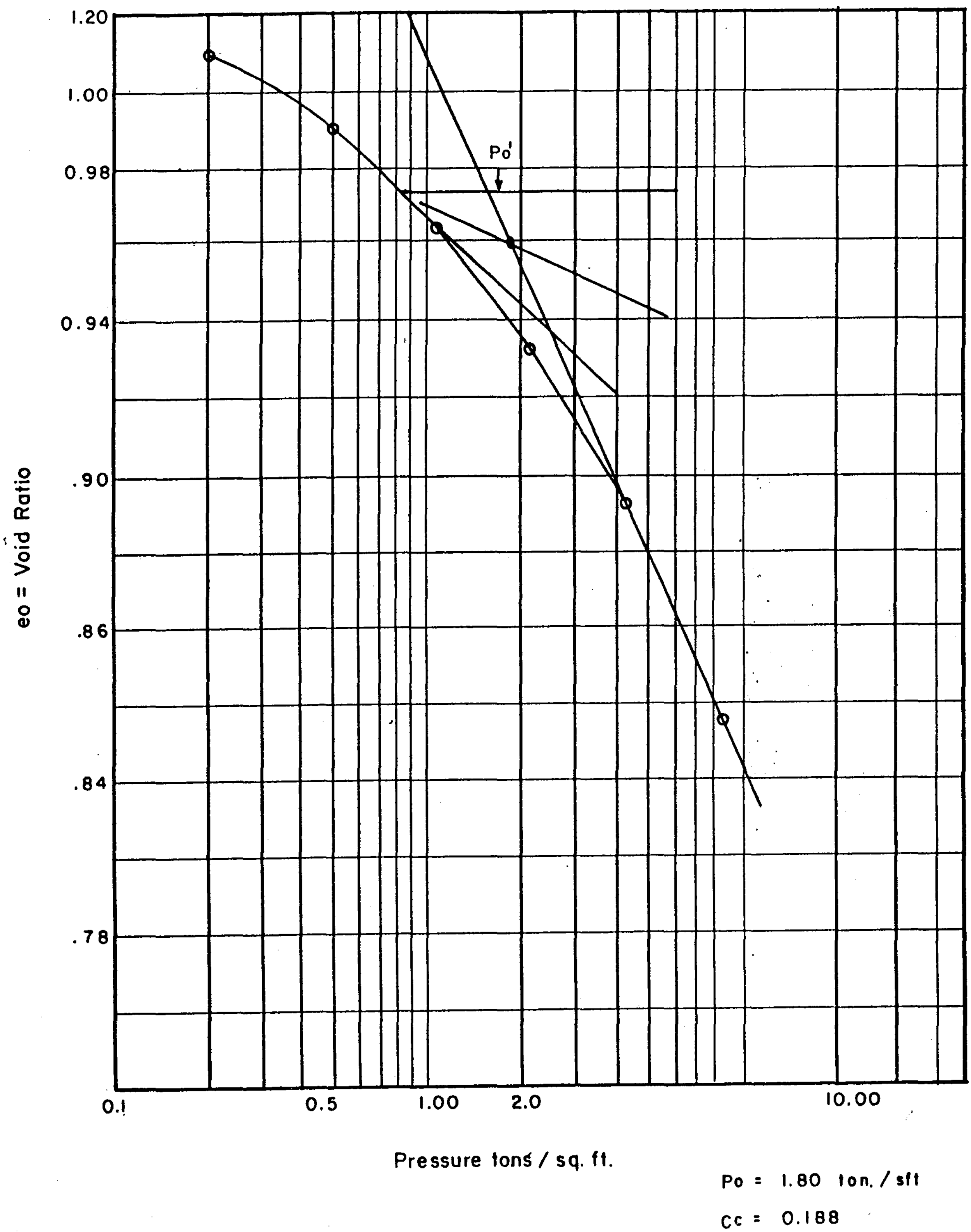


Fig. 12 Estimating Preconsolidation Pressure (2)

5. Project the straight-line portion of the laboratory curve to its intersection with the line bisecting the angle. This point of intersection approximates the maximum past effective stress to which the soil has been subjected.

If the maximum past effective stress is approximately equal to existing effective stress, then the soil is normally consolidated. If the maximum past effective stress is greater than the existing effective stress, then the soil is over consolidated. Settlement computations for pre-consolidated soil are almost similar. If $Po + \Delta P > Po'$ the amount $P_1 = Po + \Delta P - Po'$ is used together with Po to compute the compression.

Computed settlement is less in pre-consolidated soil than normally loaded soil. If e-log P curve does not clearly identify pre-consolidation other factors should be examined. The natural moisture content is one indicator of pre-consolidation. If it is closer to plastic limit (PL) than to liquid limit (LL), the clay is almost certainly pre-consolidated. This is based on the reasoning that a pre-consolidated soil is more dense and thus if it is saturated the moisture content will be lower than the same soil in a less dense state. Consolidation settlements predicted on the above basis have been found to be reliable if the laboratory samples were of good quality.

14.0 PILE FOUNDATION

Introduction : When the surface soil deposits exhibits low bearing capacity and the compressibility of the soil is found high for the required load, the foundation engineer has to investigate deeper strata to support the proposed load with smaller vertical displacements, therefore making use of a deep foundation with piles and piers. The design and behaviour of pile and pier foundation have always been the concern of foundation engineers and may be considered as one of the less investigated fields in soil mechanics. The design of pile foundation, in most cases, is done by means of empirical rules and experience supplemented by simple pile tests.

Pile may be of timber, concrete or steel. A pile is defined as a rigid and strong structural member driven by means of hammer causing compaction of soil in the neighbourhood of the pile. The subsoil usually encountered in pile and pier problems is complex, since these elements are driven through stratified subsoils of different mechanical properties. Piles should be designed according to their ability to carry load under an allowable settlement in group action, and not by the individual ultimate bearing capacity and vertical displacement obtained in customary single unit tests.

The pile foundation problem will be divided into two parts, the stability problems and elasticity problem. The first concerns the pile or pile group breaking into the ground because of exceeding the shear strength of the soil material where it is supported. The second problem concerns total and differential settlements of a pile or pile group.

The stability of a pile is governed by its ultimate load capacity, i.e. a) by the ultimate capacity and b) by the ultimate friction load capacity.

a) The ultimate bearing capacity is known to be a function of the geometrical dimensions, shape and roughness of the point, confining pressure at the point, shear strength, density and compressibility of the soil under and around the point of the pile.

b) The ultimate friction load capacity is a function of the shear strength of the soil along the pile shaft. In clays the time element is important. Highly sensitive clays never regain their total strength lost during pile driving.

14.1 Pile Capacity

The pile capacity i.e. the load taken by a pile of given length and cross section can be determined from the followings :

- by using the soil properties and determining the supporting strength
- by pile driving formula
- by load test

Whatever may be the procedure, the capacity of individual pile should not exceed the structural strength of pile. Load test is the best and reliable method for determining the ultimate load capacity but this should be limited to large jobs only. Alternatively conservative design may be made by using soil properties and verifying the capacity of pile during actual driving by using pile driving formula. Pile capacity depending on the soil properties are mentioned below.

14.2 Pile Capacity on Cohesionless Soil

Piles are frequently driven in cohesionless soil through loose cohesionless deposits or soft to very soft cohesive soil into a dense cohesionless soil for transmission of load into the firm strata. In this case the pile acts as end bearing pile and the capacity of pile depends partly on the end bearing and partly on the skin resistance of the portion driven into firm strata. The ultimate bearing capacity of pile can be computed as follows :

$$Qult = \pi R^2(\gamma D N_q + 0.6 R \gamma N\gamma) + 2 \pi R h fs \quad \text{(for round pile)....(29)}$$

$$Qult = B^2(\gamma D N_q + 0.4 B \gamma N\gamma) + 4 B h fs \quad \text{(for square pile)...(30)}$$

where Qult = ultimate bearing capacity of individual pile

R = radius of pile (round)

B = breadth of pile (Square)

C = cohesion of soil

D = total penetration of pile in soil

h = depth of penetration in supporting layer

ϕ = angle of internal friction of the supporting soil

$f_s = \gamma(D-h/2)\tan\phi$

For friction piles in cohesionless soil :

$Qult = 2\pi RL(\gamma Z + q) K \tan\phi$ (round pile) ....(31)

or $Qult = 4BL (\gamma Z + q) K \tan\phi$ (square pile) ....(32)

where L = total length of embedment of pile

K = coeff. of lateral earth pressure (0.5 to 1.00 for loose to dense sand)

q = permanent surcharge load

Z = depth of centre of gravity of the embedded portion of the pile.

14.3 Pile Capacity in Cohesive Soil

A friction pile in clay is supported by adhesion between the pile and the soil. The bearing capacity of such pile is approximately equal to the unit adhesion multiplied by the embedded surface area of pile. The adhesion is approximately equal to the cohesive strength of the clay. For friction piles in clay the ultimate bearing can be computed as follows :

$Qult = CLP$ ....(33)

where C = cohesion of soil

L = length of pile embedded in the soil

P = perimeter of pile

14.4 Pile Group

It has been recognized that groups of closely spaced piles in cohesive soils settle more than a single pile carrying the same load as the average for the group. Model tests have confirmed that at failure of a group the average load per pile is less than the ultimate bearing capacity of a single pile. Both these effects become more pronounced as the centre to centre spacing, 'S', of the pile is reduced. Realistic design of closely spaced groups is therefore difficult and empirical efficiency formula are frequently used in calculation of group bearing capacity. Some of the formulae take account of the spacing of piles generally expressed in the form of a ratio of spacing to the pile diameter, S/d. Alternatively the over all bearing capacity of the block of soil defined by the pile group is calculated. It is recommended that the group bearing capacity should be calculated both by efficiency formula and by the block method and the lower of the two values should be taken.

In case of pile foundations where piles transmit the load of the structure directly to the underlying soil partly by end bearing and partly by skin friction around the part of the pile embedded into the bearing layer, the foundation may be treated as deep footing considering one block. The capacity is based on the peripheral shear and bearing capacity of the block at the pile ends

$$Q_{ult} = SLP + A\, q_{ult}. \qquad \ldots.(34)$$

In case of friction piles in clay, group capacity is the peripheral shear only.

$$Q_{ult} = S\ L\ P \qquad \ldots.(35)$$

S = Shear strength = 1/2 qu

L = Length of pile embedded in the soil in case of friction pile and length of pile embedded in bearing layer in case of bearing pile.

P = Perimeter of area enclosing the piles in group.

A = Area enclosing piles in group.

Qult = Ultimate bearing capacity of soil at the level of pile tips and may be determined from Terzaghis' bearing capacity formula.

14.5 Function of end Bearing, Friction and Compaction Piles

End bearing piles are used to transfer loads through water or soft soil to a suitable bearing stratum.

Friction piles are used to transfer loads to a depth by means of skin friction along the length of piles.

Compaction piles are used to compact loose granular soil, thus increasing their bearing capacity. These piles do not carry any load. Hence they may be of weaker materials sometimes of sand only.

14.6 Pile Group and its Efficiency

When several closely spaced piles are grouped together, it is reasonable to expect that the pressure developed in the soil as resistance will overlap. The bearing capacity of a pile group may or may not be equal the sum of the bearing capacities of individual piles constituting a group. However, no reduction due to grouping occurs for end bearing piles. For the combined end bearing and friction piles, only the load carrying capacity of the frictional portion is reduced.

For cohesive soil, point bearing is generally neglected for individual pile action, since it is negligible as compared to frictional resistance. Bearing capacity of single pile in clay is mainly due to friction.

For point bearing piles no reduction need to be made on account of the close spacing of piles. However, the frictional resistance in such a case is completely neglected.

$$Eg = \frac{\phi}{90}\left[\frac{(n-1)m + (m-1)n}{mn}\right] \qquad ....(36)$$

When m = number of rows

n = " " pile in a column

ϕ = $\tan^{-1} d/s$

d = diameter of pile

S = spacing of pile (centre to centre)

14.7 Pile Foundation

Piles generally obtain support from combination of friction along the surface of pile shaft and from end bearing at the bottom of the shaft. In many cases however, either the friction component or the end bearing component will be predominant and the pile will be referred to as friction pile or as an end bearing pile.

$$Q_{ult} = Q_e + Q_f \qquad ....(37)$$

In dense sands, plastic clayey soil and saturated silts a close spacing may cause objectionable upheaval or lateral displacement of ground, whereas in loose sand a smaller spacing may be desired because of the benefit of compaction. In any case pile spacing should be checked for group capacity and settlement, and efficiency factor should be taken into account.

14.8 Estimating Total Pile Capacity

Total pile capacity is the sum of frictional resistance and end bearing and can be estimated either by static and dynamic formulas or by load test.

Static formulas are used to estimate capacity of individual pile on the basis of strength properties of the soil.

Dynamic formulas are used to check the correctness of assumed or calculated pile capacities during pile driving.

Load tests are used for direct checking of pile capacity. This method is costly but very reliable.

Total pile capacity can be obtained from:

$$Q_{ult} = Q_e + Q_f \qquad ....(37)$$

Q_e = end bearing capacity

Q_f = friction component

Q_e = Q_{ult} X Area

= $[C\ N_c + \gamma\ D_f\ (N_q-1)+1/2\ \gamma\ B\ N\gamma]$ Area

Since depth of pile foundation is much greater than individual diameter, N is small compare to Nq.

$$Qe = [CNc + \gamma\, D_f\, (Nq-1)]\ \text{Area} \qquad \text{....(38)}$$

For end bearing in clay $\phi = 0$

In granular soil C is zero, Nq should be higher for pile than for shallow foundation.

Friction component of the pile capacity can be estimated by considering adhesion between soil and the pile since shear component vary with depth.

$$Q_f = \Sigma di\ \ li\ \ Sui \qquad \text{....(39)}$$

Where di = incremental diameter
li = " length
sui = shear strength

For sandy soil the friction component of pile resistance depends on the co-efficient of friction between the soil and the pile and the horizontal inter-granular stress adjacent to pile surface. The horizontal stress at any depth is equal to the vertical inter-granular stress times a lateral earth pressure co-efficient, K which varies between 1 to 3.

$$Q_f = \Sigma\ di\ li\ K\ \delta vi\ \tan\delta_1 \qquad \text{....(40)}$$

K = lateral earth pressure co-efficient
δvi = average vertical inter-granular pressure
$\tan\delta_1$ = co-efficient of friction between the soil and pile
δ_1 = $2\phi/3$ (where ϕ = Angle of internal friction)

14.9 Pile Spacing

a)

Function of piles	Minimum pile spacing (centre to centre)
Point bearing piles in hard stratum	2 to 2.50 times butt diameter or 75 cm
- do - on hard bed rock	2 times butt diameter or 60 cm
Friction piles	3 to 5 times butt diameter or 1.05 m

b)

Type of piles	Minimum spacing (centre to centre)
Friction	Perimeter of the pile
End bearing	Twice the least width

<u>Pile spacing as per Norwegian code of practice.</u>

Length of pile	Friction piles on sand	Friction piles in clay	Point bearing piles
Less than 12 m	3d	4d	3d
12 to 24 m.	4d	5d	4d
Greater than 24 m.	5d	6d	5d

d is the pile diameter or largest side.

15.0 SETTLEMENT OF PILE GROUPS IN COHESIVE SOIL:

The total settlements of pile groups may be calculated by making use of consolidation settlement equations. The problem involved here is to evaluate the increase in stress ΔP beneath the pile group when the group is subjected to a vertical load P. The computation of stresses depends on the type of soil through which the pile passes.

i) If the soil is of homogenous clay, the load is assumed to act on a fictitious footing at a depth of 2/3rd from the surface and distributed over the sectional area of the group. The load on the pile group acting at this level is assumed to spread out at 2:1 slope.

ii) Where the pile passes through a very weak layer of depth D and the lower portion of length D_2 is embedded in strong layer, the load is assumed to act at a depth equal to 2/3rd D_2 below the surface of that strong layer and spread at 2:1 slope.

iii) In case of point bearing pile supported on a firm strata, the load is assumed to act at the level of the firm strata and spread out at 2:1 slope.

Allowable settlement for piles should be same as shallow foundation.

For pile foundation on sands not underlain by more compressible soil a maximum settlement of 25 mm for isolated footings and 50 mm for raft foundations are frequently allowed. Using the concept of equivalent pier foundation preliminary estimates of the settlement of a pile group in a homogenous sand deposits can be made directly from the results of penetration tests as for spread foundation.

Meyerhof (56), Skempton and Mcdonald (56) established the following relationship for estimating the settlement pile foundation.

$$S = \frac{2 P\sqrt{B}}{N} \qquad \ldots.(41)$$

Where B = Width of pile group in ft.

P = Net foundation pressure ton/sft.

N = Average corrected SPT

Observations on foundations supported by driven and bored piles shows that observed maximum settlements are generally somewhat smaller than the estimated values, as had been found for spread foundation.

16.0 COMPARISON OF PILE CAPACITIES OBTAINED BY DIFFERENT FORMULAS

Ex: Pile bearing on sand, depth of penetration = 8 ft.=2.44m.
In the top layer soil is very loose with negligible cohesion. Only bearing of sand layer is considered. Calculate the capacity of 10.67 m (35 ft) pile and find the number of piles required for total superstructure load of 3000K = 13350 Kn.

Design Data : N = 10 (Sand layer). B = 0.30m

H = 2.44 m. γ = 110 Ib/ft^3 = 17.27 Kn/m^3 ϕ = 28°

<u>Method 1</u>,

From **Table** 8, $N_q = 18.58$ $N\gamma = 15.70$ $f_s = \gamma_s\left(D - \frac{h}{2}\right)\tan\phi$

$$= (17.28 - 9.81)\left(10.67 - \frac{2.44}{2}\right)\tan 28^\circ = 7.46 \times 9.45 \times 0.532 = 37.50\ \text{Kn/m}^2.$$

$$Q_{ult} = B^2(\gamma_s D N_q + 0.4\ B\gamma N\gamma) + 4\ Bhf_s = 0.30^2(7.46 \times 10.67 \times 18.58 + 0.4 \times 0.3 \times 7.46 \times 15.70) + 4 \times 0.30 \times 2.44 \times 37.50$$

$$= 0.09\ (1478.93 + 14.05) + 109.80 = 134.36 + 109.80$$

$$= 244.16\ \text{Kn} = 54.87\text{K}$$

Q_a = allowable load/pile = 27.43 K = 122.08 Kn

$$\text{No. of pile required} = \frac{13350}{122.08} = 109.35 \text{ Say 110 Nos.}$$

<u>Method-2</u>,

Qult = Qf (Sand) + Qe (Sand) + Qf (Clay).

Q_f clay is neglected. N = 10. $\phi = 28^\circ$ $2/3\phi = 18.67^\circ$.

Penetration in hard layer = 2.44 m

$\delta v = 9.45\ (17.28 - 9.81) = 70.50\ Kn/m^2$.

Q_f (Sand) = 1.20 m X 70.50 X 2.44 X tan 18.67°

= 1.20 X 70.50 X 2.44 X 0.338 = 69.75 Kn

Qe (Sand) = $[C\ N_c + \gamma D_f\ (N_q - 1)]$ Area. C = 0, for deep foundation

Nq = 28 (Refer **Fig.** 13)

$\gamma_{sub} = 7.46\ KN/m^3$. Qe(Sand) = 7.46 X 10.67 X 27 X .090 = 193.42 Kn.

Total capacity per pile = 69.75 + 193.42 = 263.17 Kn.

$$\text{Allowable load per pile with F.S. 2} = \frac{263.17}{2} = 131.58 \text{ Kn.}$$

$$\text{No. of pile required} = \frac{13350}{131.58} = 101.46$$ However no. of piles depend on their arrangement and spacing.

Method-1, 110 Nos. ¦ So safely 110 Nos.
Method-2, 102 Nos. ¦ can be used.

17.0 LOAD CAPACITY BY DYNAMIC METHODS

Perhaps the oldest and most frequently used method of estimating the load capacity of driven piles is to use a driving formula or dynamic formula. All such formulas relate ultimate load capacity to pile set and assume that the driving resistance is equal to the load capacity of the pile under static loading. They are based on an idealized representation of the action of the hammer on the pile in the last stage of its embedment. There are great number of driving formulas available of varying degrees of reliability. There are about of 450 of such formulas.

The primary objectives in using a pile-driving formula are usually either to establish a safe working load for a pile by using the driving record of the pile. Pile driving formulas attempt to relate the dynamic to the static resistance of a pile and have been established on an empirical or theoretical basis.

17.1 Reliability of Dynamic Formulas

Several investigations have been carried out to determine the reliability of the various pile driving formulas by comparing load capacity computed from the appropriate formula with the measured capacity from pile load test. Hansen (1957), Flaate (1964) and Olson (1967) found large reliability in Danish, Hiley and Janbu Formulas. Sorensen and Hansen used data from 78 load tests on concrete, Steel and wooden piles, most of them their points in sand. Flaate (1964) investigated the accuracy of the Janbu, Hiley and Engineering News formula in 116 cases with tests on timber, concrete and steel piles in sand. They found that there is little difference in Janbu and Hiley formulas although the former is had better correlations for timber and concrete piles. Hiley's formula is also reasonable for timber piles.

Olson and Flaate (1967) extended the studies reported by Flaate (1964) and examined the reliability of various pile driving formulas. The three formulas that yielded the highest average correlation coefficient were the Danish, Janbu and Gates formulas. Engineering News formula was found to be quite unsatisfactory.

The following formulas will be used to compare the load carrying capacity of piles.

1) Gates : $P_u = 104.5 \sqrt{(e_h E_h)}(2.4 - \log S)$ ...(42)

2) Janbu formula: $$P_u = \frac{1}{K_u} \left(\frac{e_h E_h}{S}\right) \quad \text{....(45)}$$

$$K_u = C_d \left[1 + \sqrt{\left(1 + \frac{\lambda}{C_d}\right)} \right]$$

$$C_d = 0.75 + 0.15 \frac{W_p}{W_r}, \quad \lambda = \frac{e_h E_h L}{A E S^2}$$

3) Hiley formula $$P_u = \frac{K(WH\eta)}{S + C/2} \quad \text{....(46)}$$

Where,

E_h = Manufacturers hammer rating, Kn.m.(input energy = WH)
W = Hammer wt., Kn; H = Drop in m
e_h = hammer efficiency,
= 0.75 for drop hammed
= 0.85 for others.
Pu = ultimate Load Capacity,
S = Final set mm (penetration/blow)
L = pile length in m.
η = eff. of blow, depends on co-efficient of restitution, e,
K = Hammer co-efficient,
C = sum off temporary elastic compressions of the pile, Cp, the pile head Cc, and the ground C_q.
S = total set mm/blow.

Example:

Single Acting drop hammer.

W_r = Weight of hammer = 8.90 Kn.

N = Average number of blows for last 30 cm penetration.
H = Drop of hammer = 1.52 m

S = Settlement = 12 inch/30 blows = 10 mm/blow
L = 9.416m

Gates Formula: $Pu = 104.5 \sqrt{(0.75 \times 9.8 \times 1.52)}\ (2.4 - \log 10)$

$= 104.5 \times 3.34 \times 1.40 = 488$ Kn.

Janbu Formula: (1953)

$$\frac{W_p}{W_r} = \frac{19.96}{9.8} = 2.036$$

W_p = Weight of pile + cap

W_p = " hammer

$$Cd = 0.75 + 0.15 \frac{W_P}{W_r} = 0.75 + .305 = 1.055$$

$$\lambda = \frac{e_h\ E_h\ L}{AES^2} = \frac{0.75 x 9.80 x 1.52 x 9.15}{0.09 x 14 x 10^6\ x\ (.01)^2} = 0.81$$

$$Ku = Cd\ [1 + (1 + \frac{\lambda}{Cd})^{\frac{1}{2}}] = 1.055\ [\ 1 + \left[\frac{1+0.81}{1.055}\right]^{\frac{1}{2}}] = 2.12$$

$$Pu = \frac{1}{2.12} \times \frac{.75 x 9.80 x 1.52}{.01}$$

$= 526.98$ Kn

Pu = ultimate load capacity,

E = modulus of elasticity of pile Material, Kn/m^2.

S = final set (pent./blow) (m), W = Weight in Kn

H = drop of hammer (m), e_h = 0.75 for good driving condition.

Value of Ku may be obtained from **Fig.** 14

<u>Hiley Formula:</u>

$$Pu = \frac{K\ W\ H\ \eta}{S + C/2}, \qquad C = C_c + C_p + C_q$$

Table 18, K = 0.40, from **Fig.** 15, η = 0.425

H = effective height of fall of hammer = 0.90 x 1.52m = 1370 mm.
(**Table** 17, K = 0.90)

Assume ultimate driving resistance = 390 KN

$$\text{Overall driving stress} = \frac{390}{.09} = 4.33\ Mn/m^2.$$

Fig. 16, Cc = 2.50, **Fig.** 17, Cp = 2.90

Fig. 18, Cq = 1.90

C = Cc+Cp+Cq = 2.50 + 2.90 + 1.90 = 7.30 mm.

$$Pu = \frac{WH\eta}{S + C/2} = \frac{0.90x9.80x1370x.425}{10 + 3.65} = 376\ Kn$$

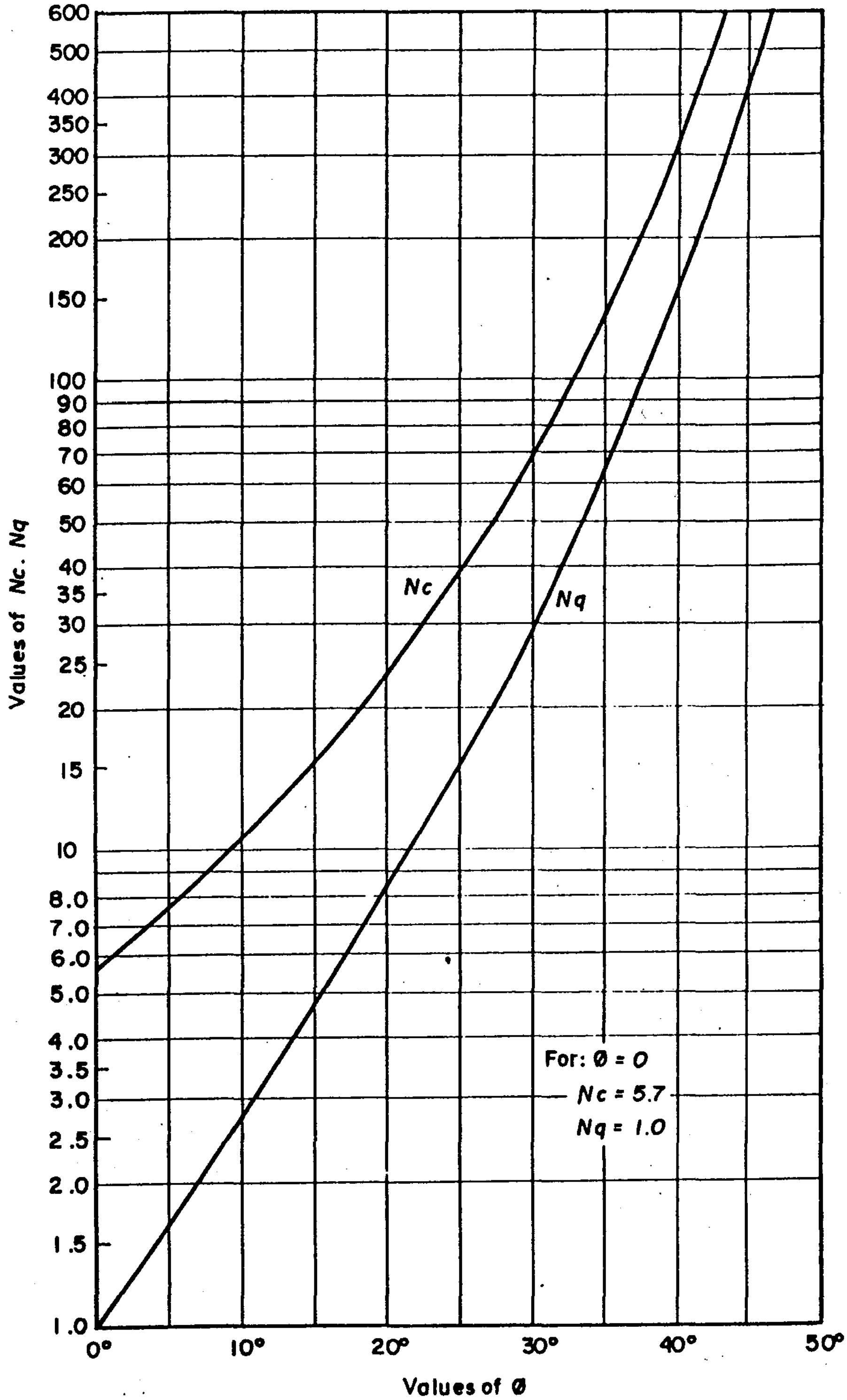
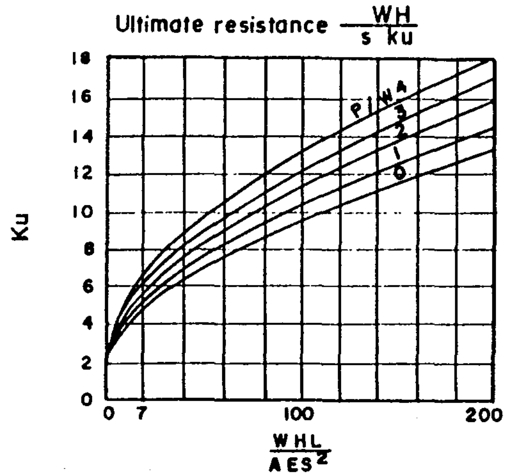


Fig. 13. Bearing capacity factors for a deep strip foundation.



Fig, 14. Design chart for the Jumbu pile driving formula

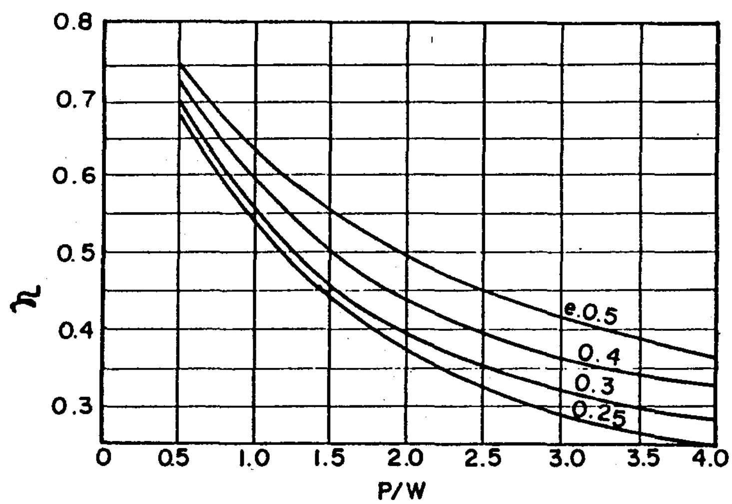


Fig. 15. Determination of efficiency factor, h.for use in the Hiley pile driving formula, after BSP Pocket book in (1969).

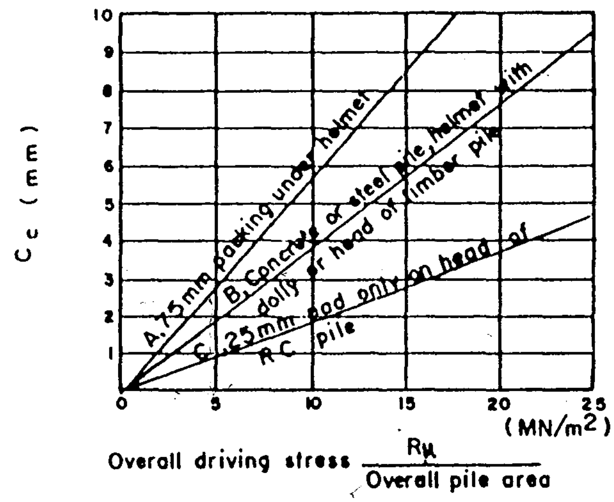


Fig. 16 Determination of temporary elastic compression Cc, after BSP Pocket book in (1969)

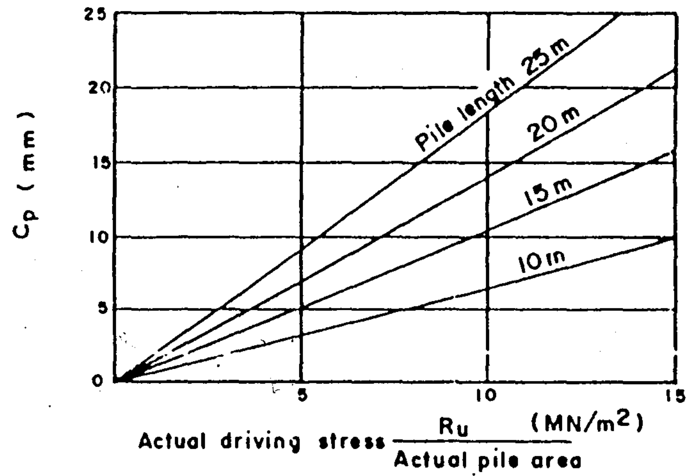


Fig.17a Determination of temporary elastic compression Cp for concrete piles, after BSP Pocket book in(1969)

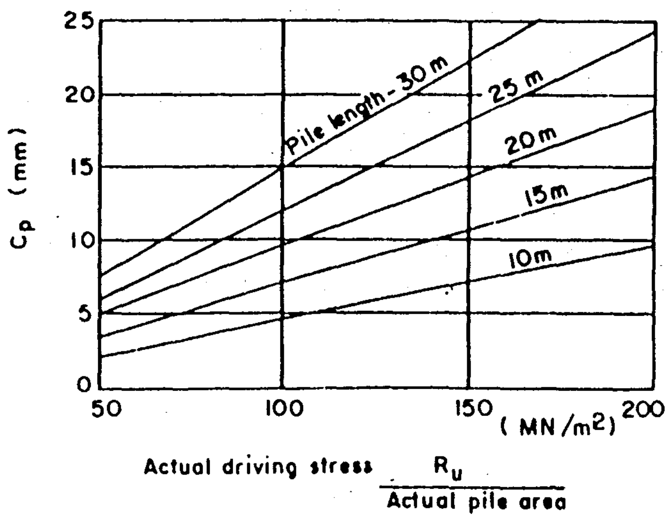


Fig. 17 b Determination of temporary elastion compression Cp for steel piles, after BSP Pocket book in (1969)

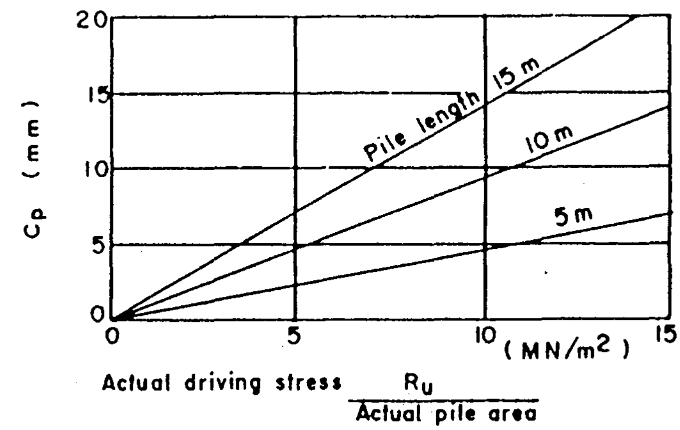


Fig. 17c Determination of temporary elastic compression Cp for timber piles after BSP Pocket book (1969)

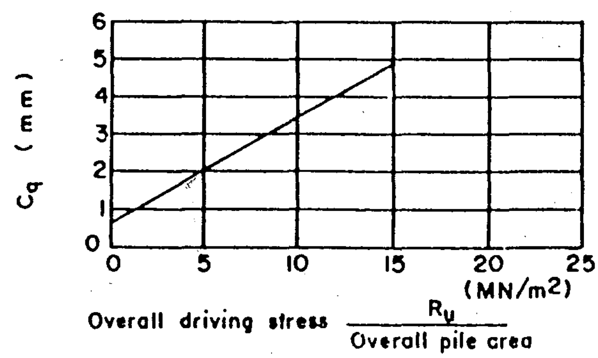


Fig. 18 Determination of temporary elastic compression Cq., after BSP Pocket book (1969)

It is nearly equal to assumed value 390 KN, hence the calculation need not to be repeated.

Allowable load per pile according to different formulas are given below:

Gates	Hilley	Janbu	Av. Load
(F=3.00)	(F=3)	(F=3)	
162.66 Kn	125.33 Kn	175.66 Kn	154.55 Kn

Pile foundation (Special note):

a) The design and behaviour of pile and pier foundations have always been the concern of the foundation engineer and may be considered as one of the less investigated fields in soil mechanics.

b) The design of pile foundation in most cases is achieved by means of empirical rules, and experience and pile test.

c) Sub soil conditions are far from being homogenous and isotropic and therefore, the theories and working hypothesis are no better than the knowledge of the foundation engineer obtained from the engineering characteristics of sub soil materials.

d) The sub soil usually encountered in pile and pier problems is complex, since these elements be driven through stratified soil conditions where the layers may have different mechanical properties.

e) In clays the sensitivity is an important factor. High sensitive clays never regain their total strength lost during pile driving.

f) Theories for determining capacities of point bearing of piles and piers are not satisfactory. However, they may be used conveniently if properly adjusted to sub soil conditions.

18.0 TOLERANCE OF STRUCTURES TO SETTLEMENT

Every structure needs a foundation to transfer its weight to the underlying soils. The construction of foundation introduces new conditions in the soils. Determination of what amounts of differential settlements can be tolerated by a structure has been studied by evaluation of conditions under which measured settlements have caused distress. Different results found with rigid and flexible structures shows that no rules can be generalized for all structures. No answers can be expected to the question of how much settlement a structure can tolerate. For each type of structure there will be different answers which must consider settlement, and amount and rate of differential settlement as measured by the slope generated by adjacent supports.

Karl Terzaghi states "Differential Settlement must be considered inevitable for every foundation, unless the foundation is supported by solid rock. Consequently, it is necessary to find out by careful inspection of buildings with known settlement records how much distortion different types of elements can withstand without any harm".

18.1 Permissible Maximum Settlements

The calculated differential settlements are usually not same as actual differential settlements. This is due to stiffness of the structure which results in a redistribution of foundation load with settlement. Skempton and Mcdonald (1956) has observed maximum settlement in case of individual footing and 40 to 60% in case of rafts. On the basis of those observations they have suggested the following values as in **Table 12.**

Table 12. Permissible Settlement

Foundation type	Soil	Permissible Settlement in (cm)
Individual	Clay	6
Footing	Sand	4
Raft	Clay	6 - 10
Rigid Box Type	Sand	4 - 6
Structure	Clay	10 and more

The Russian practice is to allow high values from settlement observation studies, it is concluded that the structures can withstand much higher rotation and differential settlements than that suggested by Statistical Analysis (**Table 13**).

In case of piles settlement also occur where foundation is located in hard soil below which there is compressible layer of soil. Depending upon loading and settlement, the compressible layer may have to be penetrated. The procedure for settlement computation of piles is the same as that for rigid footing. Once the location of footing is established for the pile type, the stresses are approximated by sixty degree distribution method.

Table 13. **Maximum Differential Settlement Permitted by U.S.S.R. Building Code**

Item	Description of standard value	Subsoil: Sand and hard clay	Subsoil: Plastic clay
1.	Slope of crane way as well as tracks for bridge crane truck	0.003	0.003
2.	Difference in settlement of civil and industrial building column foundations:		
	(a) for steel and reinforced concrete structures,	0.002L	0.002L
	(b) for undraws of columns with brick cladding,	0.007L	0.001L
	(c) for structures where auxiliary strain does arise during nonuniform settlement of foundations (L=distance between column centres)	0.005L	0.005L
3.	Relative deflection of plain brick walls:		
	(a) for multi-story dwellings and civil buildings at L/H≤3	0.0003	0.0004
	at L/H≥5	0.0005	0.0007
	(L=length of deflected part of wall; H = height of wall from foundation footing.		
	(b) for one-story mills	0.0010	0.0010
4.	Pitch of solid or ring-shaped foundations of high rigid structures (smoke stacks, waters, towers, silos, etc.) at the most unfavourable combination of loads	0.004	0.004

For structures supported on fine-grained soils, settlement takes place only under long time loading. Transient loading, if applied in very short duration or durations, brings little additional settlement. In such cases, the settlement should be calculated only under those loads which remain on the structure for a period, or a number of periods, long enough to cause consolidation settlement of the soil. This load may be referred to as the service load.

---

* From Polshin and Tokar.

18.2 Control of Settlements

Since vertical movements of structures placed on a soil are caused by the imposed loads, two solutions to reduce the settlements are immediately obvious. One method is to pre-load the soil with a load as large as or larger than the probable future load and allow settlement to take place. The second method is to excavate as much material as the structure is estimated to weigh so that when the structure is built the soil undergoes no net increase in pressure.

The second method of reducing the total and differential settlement of a foundation by decreasing the net applied load through excavation is called floating condition. The floating idea is partly applied to water retention structures. The principle of floating foundation is that the net load applied to the foundation should be small enough so that virgin consolidation must not occur.

19.0 CONSTRAINT

In Water Development Board most of the hydraulic structures are constructed in isolated places where pre-loading arrangement is difficult to maintain. Moreover, it is also costly. Floating principles are not applicable in all cases.

On the other hand, in many occasions the foundation has to be designed based only on the 'SPT' value, which gives very approximate correlations. For clay soil detail analysis is required before taking any decision. Excessive settlement in the pier part of a regulator may destroy P.V.C. resulting in soil piping & foundation failure.

It is suggested that, in future, soil investigation should be arranged in such a way that complete analysis of soil conditions is available during design, otherwise design done with speculation or mere estimate will not only be uneconomical and risky, total cost of exploration will also be fruitless.

20.0 DESIGN EXAMPLES

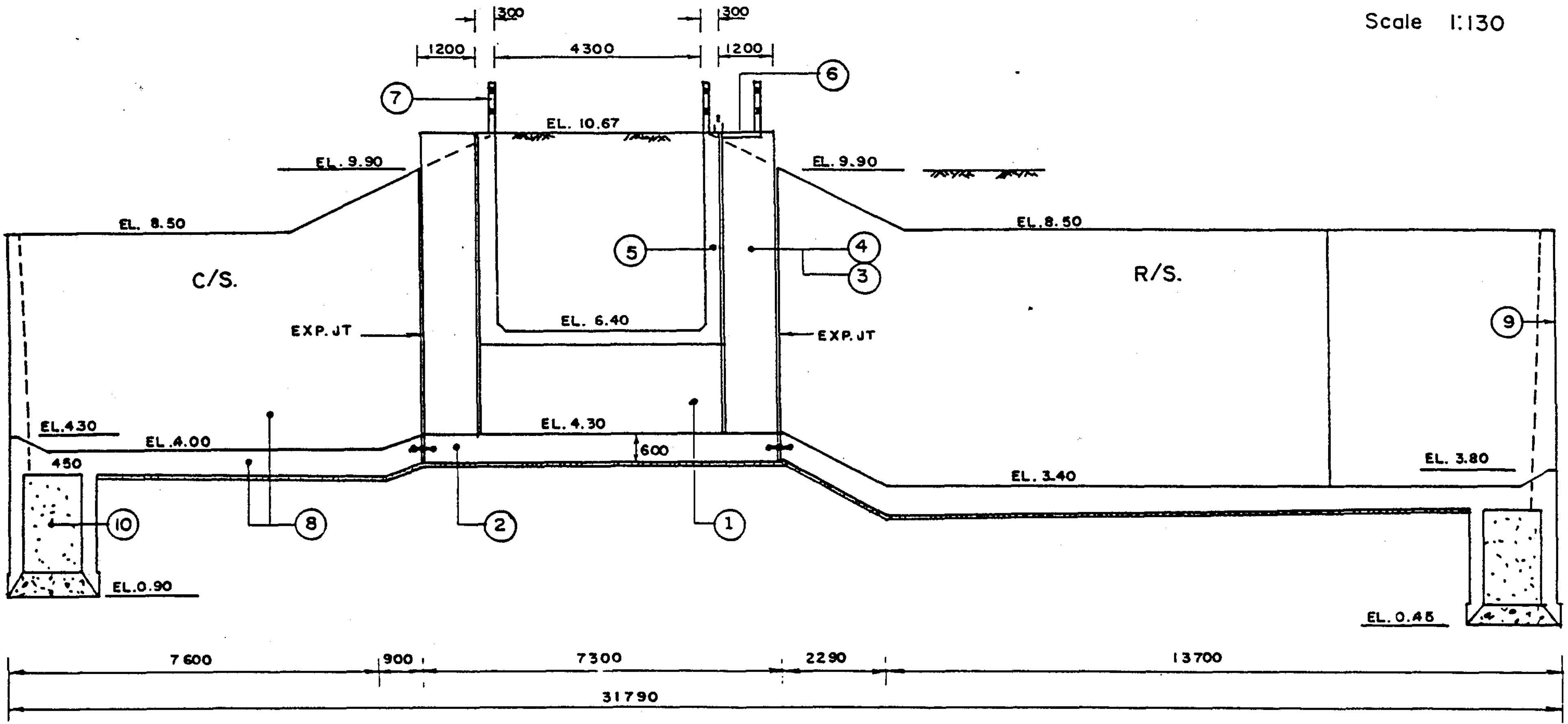
Example 1.

Main part of 3 vent (1.52 x 1.83m) Mrigi Khal sluice is shown in **Fig.** 19 and 20. For this structure four possible locations will be tried. The bore logs of those four locations are given in **Figs.** 21, 22, 23 & 24.

The steps for foundation design are follows:

a) Calculate the loads imposed on the structure.
b) Calculate gross and net foundation pressure.
c) Find out the level of foundation, position of ground water table and the existing ground level.
d) Set the design criteria for foundation design.
e) Check each bore hole and collect detail soil report.
f) Calculate bearing capacity and settlement of the structure
g) Find the suitability of foundation.

Scale 1:130



Longitudinal profile

LEGEND:

1. Box Conduit
2. Extended Part of Conduit Base Slab
3. Extended Part of Conduit Abutment
4. Extended Part of Conduit Pier
5. Head Wall
6. Operating Platform
7. Railing
8. Wing wall & Apron (Monolithic)
9. Return wall
10. Caisson cut off wall

Fig. 19 Longitudinal Profile of Mrigi Khal Sluice

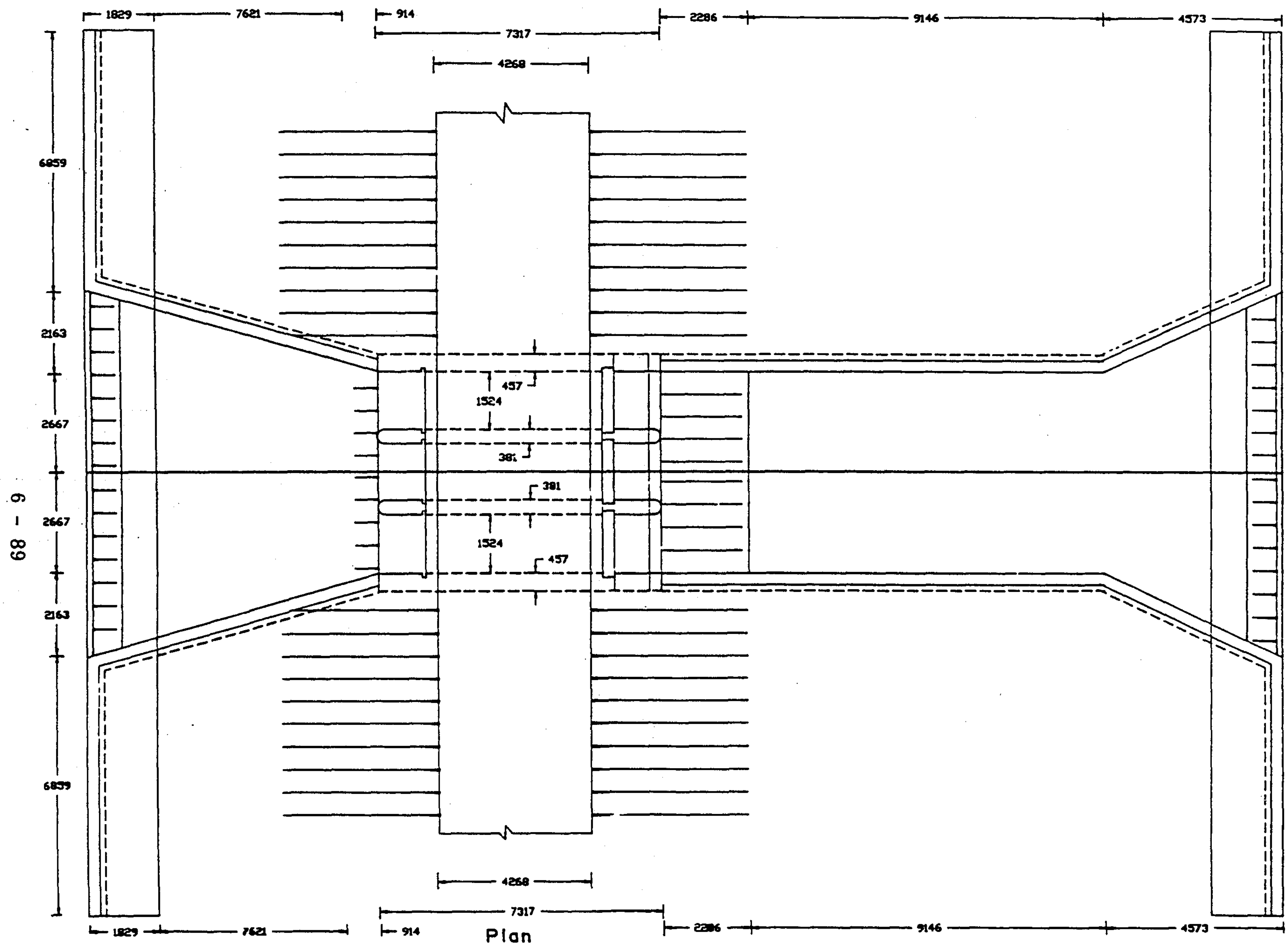


Fig. 20 Plan of Mrigi Khal Sluice

PROJECT :-	HOLE NO. :-
LOCATION :-	GROUND LEVEL :-
DRILLED BY :-	GROUND WATER LEVEL :-

NO.OF SAMPLE	TYPE OF SAMPLE	SCALE	LOG.	DESCRIPTION OF METERIALS	GENERAL: PERMEABILITY	GENERAL: DENSITY	GENERAL: COLOUR	GENERAL: MOISTURE	GENERAL: PLASTIC	BLOWS ON SPOON PER 6 INCH PENETRATION: 0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE: BLOWS PER FOOT OF PENETRATION	INDEX: DISTURBED SAMPLES / UNDISTURBED SAMPLES / REMARKS
D1	Disturbed			MEDIUM TO FINE SAND TRACE SILT TRACE MICA		LOOSE	GREY	MOIST	NON PLASTIC	4	4	5	6		
D2	Disturbed	10				MED DENSE				4	4	6	7		
D3	Disturbed									5	6	6	6		
D4	Disturbed	20								6	7	7	8		
D5	Disturbed									6	7	8	8		
D6	Disturbed	30								6	8	8	9		
D7	Disturbed			CLAY TRACE FINE SAND		STIFF	LIGHT BROWN		MED PLASTIC	5	7	8	10		
D8	Disturbed	40								6	7	8	9		
D9	Disturbed									5	6	7	8		
D10	Disturbed	50								6	6	7	8		
D11	Disturbed									6	7	8	9		
				MED.TO FINE LITTLE SILT TRACE MICA		MED DENSE	GREYISH BROWN		NON PLASTIC						
D12	Disturbed	60		FINE SAND LITTLE SILT TRACE MICA						7	0	11	13		
D13	Disturbed									8	10	11	12		
D14	Disturbed	70								9	11	18	15		
D15	Disturbed									10	12	14	16		
D16	Disturbed	80		CLAY TRACE FINE SAND		V ST				11	13	16	16		

Fig. 21 Bore Log.

PROJECT :-

LOCATION :-

DRILLED BY :-

HOLE NO. :-

GROUND LEVEL :-

GROUND WATER LEVEL :-

NO. OF SAMPLE	TYPE OF SAMPLE	SCALE	LOG.	DESCRIPTION OF METERIALS	GENERAL: PERMEABILITY	GENERAL: DENSITY	GENERAL: COLOUR	GENERAL: MOISTURE	GENERAL: PLASTIC	BLOWS ON SPOON PER 6 INCH PENETRATION: 0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE / BLOWS PER FOOT OF PENETRATION	REMARKS
D1	Disturbed			CLAY, TRACE FINE SAND		V SOFT	GREY	MOIST	MPL	0	0	1	1		
D2	Disturbed	10'								0	0	1	1		
D3	Disturbed			CLAY, TRACE FINE SAND		LOOSE			HPN	1	1	2	3		
D4	Disturbed	20'		FINE SAND SOME SILT		MED			PLASTIC	2	3	3	5		
D5	Disturbed														
D6	Undisturbed	30'		CLAY, TRACE, FINE SAND, TRACE ORGANIC		M STIFF				Shelby					
D7	Disturbed			CLAY AND FINE SAND					MP	2	3	5	6		
D8	Disturbed	40'		FINE SAND TRACE		MED DENS			NPLASTIC	4	6	8	10		
D9	Disturbed									5	7	9	11		
D10	Disturbed	50'		FINE SAND SOME SILT						5	7	11	16		
D11	Disturbed			CLAY, LITTLE FINE SAND					HP	2	2	2	2		
D12	Undisturbed	60'		CLAY, TRACE, FINE SAND AND ORGANIC		M ST			HP	Shelby					
D13	Disturbed			CLAY, LITTLE FINE SAND		V STIFF			MPL	3	7	11	13		
D14	Disturbed	70'								3	8	10	13		
		80'													

INDEX: DISTURBED SAMPLES (hatched); UNDISTURBED SAMPLES (lined)

Fig. 22 Bore Log

PROJECT :-

LOCATION :-

DRILLED BY :-

HOLE NO. :-

GROUND LEVEL :-

GROUND WATER LEVEL :-

NO.OFSAMPLE	TYPE OF SAMPLE	SCALE	LOG.	DESCRIPTION OF METERIALS	PERMEABILITY	DENSITY	COLOUR	MOISTURE	PLASTIC	BLOWS ON SPOON PER 6 INCH PENETRATION 0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE — BLOWS PER FOOT OF PENETRATION	INDEX / REMARKS
D1	Disturbed	10'		SILT,TRACE FINE SAND		SOFT	LT. BR.		MED COM	1	2	2	3		
D2	Disturbed			SILT,TRACE FINE SAND		V.L.			N.P.	0	1	1	2		
D3	Disturbed	20'				V. SOFT			MED COM	0	0	1	2		
D4	Disturbed						YGREY			0	0	1	2		
D5	Disturbed	30'		SILT,TRACE FINE SAND		V. LOOSE		M	NON PLASTIC	0	1	2	2		
U1	Undisturbed									Shel	by				
D6	Disturbed	40'								1	2	2	3		
D7	Disturbed			FINE SAND		MEDIUM DENSE		O		4	5	7	10		
D8	Disturbed	50'								4	6	8	10		
D9	Disturbed									7	9	11	12		
D10	Disturbed	60'		LITTLE SILT						8	11	14	16		
D11	Disturbed			TRACE MICA			Y			8	12	16	20		
D12	Disturbed	70'								9	12	13	20		
D13	Disturbed			DO						11	13	15	21		
D14	Disturbed	80'		FINE SAND TRACE SILT & MICA						11	12	15	20		
D15	Disturbed									12	14	16	22		

INDEX: DISTURBED SAMPLES (hatched); UNDISTURBED SAMPLES (lined)

PROJECT :-

LOCATION :-

DRILLED BY :-

HOLE NO. :-

GROUND LEVEL :-

GROUND WATER LEVEL :-

INDEX: DISTURBED SAMPLES (hatched); UNDISTURBED SAMPLES (horizontal lines)

NO.OF SAMPLE	TYPE OF SAMPLE	DESCRIPTION OF METERIALS	DENSITY	COLOUR	MOISTURE	PLASTIC	0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE (BLOWS PER FOOT OF PENETRATION)	REMARKS
D1	Disturbed	SILT, TRACE FINE SAND	T F O S			MED. COMPRESS	1	2	2	2		
D2	Disturbed	SILT, TRACE FINE SAND					0	0	2	2		
D3	Disturbed	SILT, TRACE FINE SAND					0	1	1	2		
U1	Undisturbed	CLAY, TRACE FINE SAND		Y E R G	M O I S T	M PL	Shelby					
D4	Disturbed	SILT, LITTLE FINE SAND				M E D. COMPRESS	0	1	2	2		
D5	Disturbed	SILT, TRACE FINE SAND,					1	1	2	3		
D6	Disturbed	SILT, TRACE FINE SAND,	MED.				1	2	3	3		
D7	Disturbed	SILT, TRACE FINE SAND,					1	2	3	4		
D8	Disturbed	SILT, TRACE FINE SAND,	STIFF				2	2	3	3		
D9	Disturbed	SILT, TRACE FINE SAND,					2	2	4	6		
D10	Disturbed	SILT, TRACE FINE SAND,					3	3	4	7		
D11	Disturbed	SILT, LITTLE FINE SAND				MED. PLASTIC	3	4	5	7		
D12	Disturbed	SILT, LITTLE FINE SAND					4	5	6	6		
D13	Disturbed	CLAY, TRACE, FINE SAND					4	5	6	6		
D14	Disturbed	CLAY, TRACE, FINE SAND					5	6	8	9		
D15	Disturbed	CLAY, TRACE, FINE SAND					6	7	7	10		

Fig. 24 Bore Log.

The calculated total load of the structure as shown in the **Fig.** 19 and 20 under major heads are as follows :

D.L. : Concrete & others	= 2345.15	Kn
Embankment soil	= 1966.90	Kn
L.L. = H10 truck over	= 89.00	Kn

Total	= 4401.05	Kn

The gross foundation pressure on a bearing area of 7.32 m x 6.25m = 45.75 Sq.m. under different condition of the applied load are given below :

D.L.(Concrete & others) only = 51.44 Kn/m^2.

D.L.(Concrete & Embankment soil) = 94.71 Kn/m^2.

Full D.L. + L.L. = 96.63 Kn/m^2 .

(As the surcharge is 4.57 m, L.L. effect may be neglected).

The net foundation pressure for full D.L. + L.L. condition

= 96.63 Kn/m^2 - γD_f

= 96.63 Kn/m^2 - 17.27x4.57

= 96.63 - 78.93 = 17.70 Kn/m^2

The factor of safety (F.S.) of safe net bearing pressure against the net foundation pressure shall be as follows :

For cohesive soil F.S = 3

For cohesionless soil F.S = 2

As the foundation is raft, the settlement shall be limited to 50 mm total and 25 mm differential.

The ground water table is considered at foundation level as this will give the critical condition of design as full structural load is imposed but bearing capacity is still reduced to saturation of foundation soil.

BEARING CAPACITY : (Ref. Bore Log in Fig. 21)

Below the foundation, soil is sandy, and minimum value of "N" is 10. So corresponding ϕ value is taken 30°. (This is taken from **Table** 9).
Hansen bearing capacity Equation is adopted.

Q_{ult} = 0.5 B γ_{sub} N_γ S_γ d_γ + γ_{moist} D_f N_q S_q d_q

γ_{moist} = 110 lb/cft. = 17.27 KN/m^3

γ_{sub} = 52.60 lb/cft. = 8.26 KN/m^3

B = 6.25 m, N_q = 18.40 & N_γ = 22.40 for ϕ value = 30°

$$S_\gamma = 1 - 0.4 \left(\frac{B}{L}\right) = 1 - 0.4 \left(\frac{6.25}{7.32}\right) = 1 - 0.34 = 0.66$$

d_γ = 1.0, Df = 15.0″ = 4.57 m

$$S_q = 1 + \left(\frac{B}{L}\right) \tan\phi = 1 + \left(\frac{6.25}{7.32}\right) \tan 30^\circ = (1 + 0.49) = 1.493$$

$$d_q = 1 + 2 \tan\phi \, (1 - \sin\phi)^2 \; D/B.$$

$$= 1 + 2 \tan 30^\circ \, (1 - \sin 30^\circ)^2 \left(\frac{4.57}{6.25}\right)$$

$$= 1 + 2 \times .577 \, (1 - 0.50)^2 \times 0.73^2$$

$$= 1 + 1.154 \times .25 \times .73$$

$$= 1.211$$

Incorporating the above values in the same equation.

$$Q_{ult} = \frac{0.5 \times 6.25 \times 8.26 \times 22.40 \times 0.66 \times 1.0 + 17.27 \times 4.57 \times 18.40 \times}{1.493 \times 1.211}$$

$$= 381.61 + 2625.61 = 3007.22 \text{ Kn/m}^2.$$

Before back filling and construction of embankment, the safe bearing capacity (without surcharge)

$$q_a = \frac{381.61}{2} = 190.80 \text{ Kn/m}^2 > 51.44 \text{ Kn/m}^2 \quad \text{OK.}$$

After back filling and construction of embankment (with surcharge), the net safe bearing capacity $= \dfrac{Q_{ult} - \gamma D_f}{F}$

$$= \frac{3007.22-17.27 \times 4.57}{2} = 1464.15 \text{ Kn/m}^2 \gg 17.70 \text{ Kn/m}^2 = \text{OK.}$$

Settlement:

For N = 10, B = 4.57m, net soil pressure for 1" (2.54 cm)

settlement = 1.0 ton/sft. = 96 Kn/m^2 (from **Fig.** 3)

with water level correction, net soil pressure = 0.5t/sft. = 52.67 Kn/m^2.

For 25 mm settlement in case of raft,

$q_a = 2 \times 52.67 = 105.34 \text{ Kn/m}^2 > 17.70 \text{ Kn/m}^2$ O.K.

Safe bearing capacity is controlled by settlement. Foundation should be safe against allowable bearing capacity failure and also from detrimental settlement.

Example 2.

The same structure placed in a site whose bore log is given in **Fig. 22.** The soil is loose consisting of clay and trace sand upto 6.10 m. Below that minimum 'N' value is 6. F.L. is at 4.57 m below G.L. So, below the foundation there will be 1.52 m loose layer. This layer should be replaced by sand to get adequate bearing pressure.

It is seen that after removing top 1.52 m of soil below foundation there is one layer of clay located between 9.15-11.85 m. Settlement of this layer is to be checked.

For N value 6, ϕ is taken as 28°, Using Hansen's formula:

$N_q = 14.72$
$N_\gamma = 16.72$ From **Table 8.**

$$Q_{ult} = 0.5\ B\ \gamma_{sub}\ N_\gamma\ S_\gamma\ d_\gamma + \gamma_{moist}\ D_f\ N_q\ d_q\ S_q$$
$$= 0.5 \times 6.25 \times 8.26 \times 16.72 \times 0.66 \times 1.00 + 17.27 \times 14.72 \times 1.493 \times 1.211 \times 4.57$$
$$= 284.85 + 2100.49 = 2385.34\ Kn/m^2 .$$

Before back filling.

$$q_a = \frac{284.85}{2} = 142.42\ Kn/m^2 > 51.44\ Kn/m^2. \quad OK.$$

It is also found safe after construction of embankment.

Settlement :

Settlement is given by $S = \frac{Cc\ H}{1 + eo} \log 10 \frac{Po + \Delta P}{Po}$

In this example

$Cc = 0.263$ ¦ From test result.

$eo = 1.11$

$P_{net} = 17.70\ KN/m^2$

The existing overburden pressure at mid-height of compressive layer located at 10.36m below G.L.(5.79m below structure base).

$Po = 4.57 \times 17.27 + 5.97\ (18.05-9.81) = 78.92 + 47.72$

$= 126.63\ Kn/m^2$.

Net foundation pressure without L.L. = 17.70 - 1.94 = 15.76 Kn/m^2.

Increase in pressure at the middle of compressible layer in short direction considering 2 : 1 pressure distribution.

$$\Delta P = \frac{15.76 \times 6.25}{6.25 + 5.79} = 8.18\ Kn/m^2. \quad H = 2.44m = 244\ cm.$$

$$S = \frac{0.263 \times 244}{1 + 1.11} \log_{10} \frac{126.63 + 8.18}{126.63} = 30.41 \times 0.027\ cm$$

$$= 0.82\ cm$$

Allowable 50 mm. Hence OK. The foundation is safe from bearing capacity and as well as settlement consideration.

Example 3.

If the same structure is located in soil type given by **Fig.** 23, shallow foundation for the structure is not possible to provide due to excessive settlement of under lying compressible soil.

It is suggested to give 9.15 m timber pile with average dia 20 cm. At the bottom (1.83 m) end bearing is taken and out of 7.29 m friction for 5.46 m is considered. Top 1.83 m is omitted for safety.
Cohesion = 40 Kn/m^2 = 832 lb/sft.

The ultimate bearing capacity from consideration of end bearing.

$Q_{ult} = \pi R^2(\gamma\ D\ N_q + 0.6\ R\ \gamma\ N\gamma) + 2\pi\ Rhfs$

D = 9.15 m.

h = 1.83m, fs = $\gamma_s(D-h/2)\tan\phi$ from SPT $\phi = 28^\circ$

From chart Nq = 18.70, $N\gamma$ = 15.70.

γ_s = 8.24 KN/m^2.

$$f_s = 8.24\left(9.16 - \frac{1.83}{2}\right)\tan\phi$$

f_s = 8.24 x 8.23 x .53 = 35.96 Kn/m^2.

Q_{ult} = 0.0314(8.24 x 9.15 x 18.70 + 0.6 x .10 x 8.24 x 15.70)
+ 2 x 3.14 x .10 x 1.83 x 35.96
= 0.0314(1408.36 + 7.76) + 41.33 = 44.47 + 41.33 = 85.80 Kn

$$q_a = \frac{85.80}{2} = 42.90\ Kn$$

C = 40 Kn/m^2.

Group efficiency:

$$E_g = 1 - \frac{12.52}{90} \times \left[\frac{8 \times 7 + 6 \times 9}{7 \times 9}\right]$$

$$= 1 - \frac{12.52}{90} \times \left[\frac{110}{6}\right] = 1 - 0.24 = 0.76$$

Therefore, total load carrying capacity of piles (**Fig.** 25)

= 42.9 x 63 + 45.88 x .76 x 63 ?

= 2702.70 + 2196.73

= 4899.43 Kn > 4401 Kn O.K.

Example-4:

The same structure is located in a soil as shown by bore log in **Fig. 24.**

Below foundation there is continuous clay & silt soil of 'N' value from 3 to 7 upto 45 ft (13.72 m), So friction pile is suggested.

C = 23 Kn/m^2. Use 0.30 m dia pile

Try with R.C.C. Pile:

Provide 10.67 m pile

Qult = 23 x 10.67 x 1.22 = 299.40 Kn.

Q all = 99.80 Kn.

Consider Eff.factor 0.70. = Qall = 69.92 Kn.

$$\text{No. of piles required} = \frac{4401}{69.92} = 63 \text{ Nos.}$$

Group Action:

Perimeter of group = P = 2(6.25 + 7.23) = 27.14 m.

L = 10.67 m. c = 23 Kn/m^2.

Load carrying capacity = 10.67 x 23 x 27.14 = 6660 Kn

$$Q_{all} = \frac{6660}{3} = 2220 \text{ Kn from skin friction.}$$

Bearing at end : CNC. Area = 23x6x6.25x7.32 = 6313 Kn.

Q_{all} = 2104 Kn.

Total load carrying capacity from group action.

= 2220+2104 = 4324 Kn. < 4401 Kn.

Change the length of pile as no. of pile can not be increased due to small foundation area.
Provide : 13.25 m pile.

Individual Action:

C = 23 Kn/m^2 CLP = 23 x 13.25 x 1.22 = 371.79 Kn.
P = 1.2 m^2. Q_{all} : 371.79/ 3.0 = 123.93 Kn ; (FS = 3)
L = 13.25m

Assume efficiency factor = 0.70. Q all = 86.75 Kn.

$$\text{No of piles required} = \frac{4401}{86.75} = 51 \text{ Nos.}$$

Provide 56 Nos. (8 x 7).

Group Action :

Qult = SLP + CN_cA.

SLP = 23 x 13.25 x 27.14 = 8270.91 Kn.

CN_cA = 23 X 6 X 6.25 X 7.32 = 6313.50 Kn

Total = 14584.41 Kn.

Allowable = $\frac{14584.41}{F.S(3)}$ = 4861.47 KN

Allowable load = 4861.47 Kn.> 4401 Kn OK.

Check for efficiency factor:

S = 0.96 m to 0.939 m.

m = 8

$$\phi = \tan^{-1}\left[\frac{d}{s}\right] = \tan^{-1}\left[\frac{0.30}{0.95}\right]$$

= 17.50

n = 7

d = .30 m

$$Eg = 1 - \frac{\phi}{90}\left[\frac{(n-1)\,m + (m-1)n}{mn}\right] = 17.53$$

$$Eg = 1 - \frac{(17.50)}{90} \times \frac{(6 \times 8 + 7 \times 7)}{8 \times 7}$$

= 1 - . 337 = 0.663

Q total = 123.93 x 56 x .663 = 4601 Kn > 4401 Kn. OK.

Example 5.

In the same site (Bore log Fig. 23) provide 30 ft. (9.15 m). R.C.C. pile (0.30 m x 0.30 m) with depth of penetration 2.13 m in cohesionless soil.

Individual Action:

Ultimate bearing capacity from consideration of end bearing.
Minimum N value on the bearing layer = 12, for N = 12, $\phi = 30^\circ$,

From Terzhaghi's table value of $N_q = 22.50$, $N_\gamma = 19.70$

$fs = \gamma s(D - 1/2\ h) \tan \phi$

$$= \gamma s(9.15 - \frac{2.13}{2}) \tan 30^\circ = 8.24 \times 8.08 \times 0.577 = 38.42\ Kn/m^2.$$

$Q_{ult} = B^2 (\gamma\ D\ N_q + 0.4\ B\ \gamma\ N_\gamma) + 4\ B\ h\ fs$

$= .09(8.24 \times 9.15 \times 22.5 + 0.4 \times .30 \times 8.24 \times 19.20) + 4 \times .30 \times 2.13 \times 28.42$

$= .09(1696.41 + 18.98) + 98.20.$

$= 154.38 + 9820 = 252.58$ Kn.

$Q_{all} = 126.29$ Kn = 28.38K.

Friction Part (For the portion above bearing layer).

C = 40 Kn/m^2

CLP = 40 x 7.02 x 1.22 = 342 .57 Kn.

q_{all} = 342/3 = 114 Kn.

Assume eff = 0.70.

Qu = 239.80 Kn.

q_{all} = 79.93 Kn,(after correction).

Total load carrying capacity of each pile both from bearing and frictional consideration. = 79.93 + 126.29 = 206.22 Kn.

$$\text{No of piles required} = \frac{4401 \text{ Kn}}{206.22} = 21.34 = \text{provide 24 Nos.}$$

<u>Group Action :</u>

Perimeter = (6.25 + 7.32) x 2 = 27.14 m.

Area of pile group = 6.25 x 7.32 = 45.75 m^2

C = 40 KN/m^2.

Ultimate bearing capacity of pile group at tips.

$$Q_{all} = \beta\ B\ D\ N\gamma + \gamma\ D_f\ N_q \quad , \beta = 0.5 - \frac{0.1\ B}{L} = 0.5 - 0.08 = 0.42.$$

= 0.42 x 6.25 x 9.15 x 19.70 + 8.24 x 9.15 x 22.50

= 473.17 + 1696.41 . = 2169.58 Kn/m^2.

Ultimate group capacity of pile = SLP + qult xA.

= 40 x 9.15 x 27.14 + 2169.58 x 45.75

= 4933.24 + 99258.28 = 109101.53 Kn

Allowable group capacity = 54595 Kn. >> 4401 Kn. OK.

$$E_g = 1 - \frac{\phi}{90} \times \frac{\{(n-1)m+(m-1)n\}}{mn}$$

d = 0.30m, S = 1.29m.

$\tan^{-1} d/s = 0.232$, $\phi = 13.09^\circ$, m = 6, n = 4,

$$Eg = 1 - \frac{13.90}{90} \times \frac{(3 \times 6 + 5 \times 4)}{6 \times 4} = 1 - \frac{13.09}{90} \times \left[\frac{38}{24}\right]$$

$$= 1 - 0.23 = 0.77$$

Total load carrying capacity. = 114.19x0.77 x 24 + 126.29x24

= 2110.23 + 3030.96 = 5141 Kn> 4401 Kn. OK.

(See **Fig.** 26)

Example 6.

Foundation is located 18 ft (5.49m) below ground level. At foundation level there is medium dense sand. Below the sandy layer there is soft clay of 10 ft (3.05m). This layer is likely to settle. Laboratory consolidation curve is given. (**Fig.** 11).

The following parameters are found from test results.

e_o = 1.28,

c_c = 0.350.

Foundation pressure P = 96.15 Kn/m^2

Size of structure = 7.62m x 10.67m.

γ_{sat} = 18.05 Kn/m^3.

γ_{moist} = 17.27 Kn/m^3.

a) Check whether the soil is pre-loaded.

b) Find out settlement.

c) Upper sand layer is 4.57m deep.

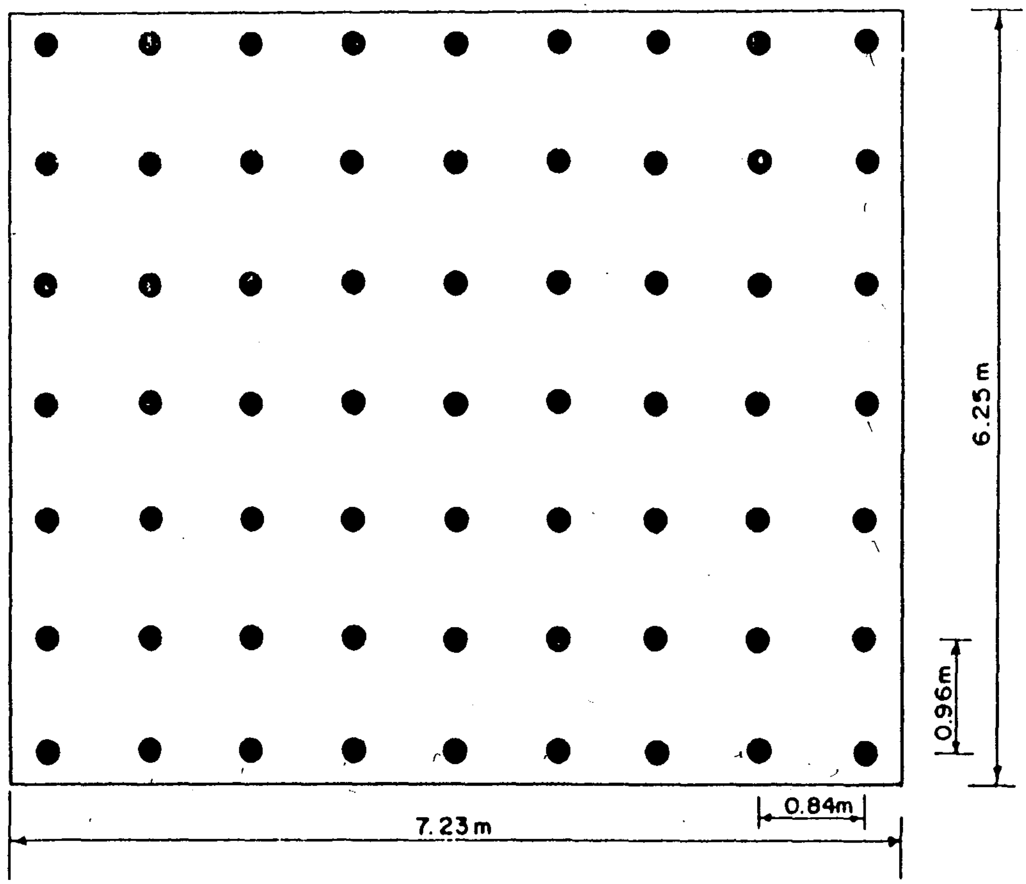


Fig. 25 Pile arrangements in ex. 3 Scale 1:64

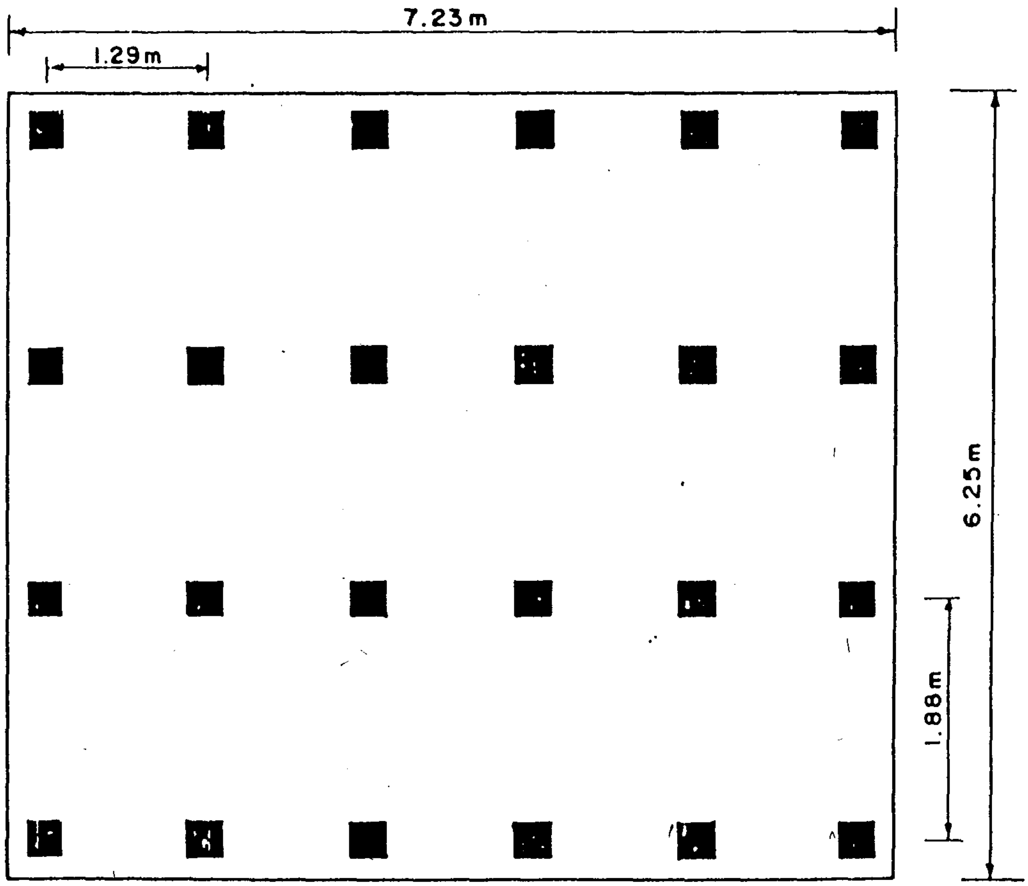


Fig. 26 Pile arrangements in ex. 5 Scale 1:64

Solution :

A tangent is drawn at maximum curvature point. A horizontal line is also drawn. The angle is divided and intersection of bisector with virgin curve is found out. $Po' = 0.8$ ton/sft. $= 76.92$ Kn/m².

Water level is 3.66m below base. At centre of clay layer settlement is to be calculated.

$Po = 17.27 \times 3.66 + (18.05-9.81) \times 7.93 = 128.55$ Kn/m² = 1.34 Ton/sft

As $Po > Po'$, it is not pre-loaded.

$\gamma D_f = 17.27 \times 3.66 + 8.24 \times 1.83 = 78.28$ Kn/m².

Net load/sft = $96.15 - 78.28 = 17.87$ Kn/m².

ΔP at centre of compressive layer

$$\Delta P = \frac{17.87 \times 7.62 \times 10.67}{(7.62 + 6.40)(10.671+6.40)} = 6.07 \text{ Kn/m}^2.$$

$$S = \frac{Cc\ H}{1 + eo} \log_{10} \frac{(\Delta P + Po)}{Po,} \quad H = 10 \text{ ft.} = 305 \text{ cm.}$$

$$S = \frac{0.32 \times 305}{1 + 1.28} \log_{10} \frac{6.07 + 128.53}{128.53}$$

$$= 42.80 \times 0.20 = 0.86 \text{ cm} = 0.34 \text{ inch.}$$

Beyond central part, load is approximately 0.375 T/sft.

or 36.05 Kn/m².

No net pressure, settlement is not likely to occur.

Differential settlement = 0.86 cm > 3.81 cm. O.K.

Example 7.

Same soil condition. Consolidation curve as in **Fig.** 12.

Po'= 1.8 Ton/sft. = 173 Kn/m².

Cc = 0.188, eo = 1.05, Po = 128.55 Kn/m².

$$Po < Po'$$

So the soil is pre-consolidated. There will be less settlement than calculated. Use Po' instead of Po, H = 305 cm.

$$S = \frac{0.188 \times 305}{1 + 1.05} \log 10 \frac{(6.07 + 128)}{128}$$

$$= 27.07 \times 0.020 = 0.54 \text{ cm} = 0.165 \text{ inch.}$$

$$Po' - Po = 173 - 128 = 45 \text{ Kn}$$

$$\frac{\Delta P}{Po' - Po} = \frac{6.07}{45} = 0.13 \ll 0.5,$$ estimated settlement would be 15% of the calculated settlement.

i.e, S = 0.15 x 0.54 = 0.081 cm; (15%).

So differential settlement is very low.

Pre-consolidated clay causes less settlement.
Although the foundation soil has high bearing capacity, it will cause settlement below.

For checking foundation, two considerations are to be given:

i) whether foundation can take care of superimposed load.

ii) whether there will be settlement in compressible layer located below, (At a maximum depth of 2B from foundation level).

Example 8.

A bridge of span 120 ft (36.58m) is to be located in a place where foundation soil is very poor (bore log enclosed **Fig.** 27) and no alternate site with better subsoil condition is available.

Solution:

As the soil is poor the bridge is designed with a few simply supported spans. Any settlement if occurs will affect to a lesser degree than that of continuous bridge.

Dead load at central column = 190 K = 845.50 Kn.
Soil parameters : N value = 4.
q_u = 0.45 T/sft. C = 450 lb/sft = 21.63 Kn/m^2
Average pile dia = 8 inch = 20 cm.
Length of pile = 40 ft. = 12.20 meters.
Timber pile will be used with joint. Allowable cohesion
= 150 lb/sft = 7.21 Kn/m^2.

PROJECT :– Bridge Over Chankhali Khal	HOLE NO. :– H5
LOCATION :– Chankhali	GROUND LEVEL :– N/S
DRILLED BY :– GWD – 1	GROUND WATER LEVEL :– RIVER BORING

INDEX: DISTURBED SAMPLES; UNDISTURBED SAMPLES

NO.OFSAMPLE	TYPE OF SAMPLE	SCALE	LOG.	DESCRIPTION OF METERIALS	PERMEABILITY	DENSITY	COLOUR	MOISTURE	PLASTIC	BLOWS ON SPOON PER 6 INCH PENETRATION 0/6	6/12	12/18	18/24	STANDARD PENETRATION RESISTANCE BLOWS PER FOOT OF PENETRATION	REMARKS
D1	Disturbed			SILT,TRACE FINE SAND		V SOFT	GREY	MOIST	M COMPRESS	0	0	0	1		
D2	Disturbed	10'								0	0	0	1		
U1	Undisturbed	20'		CLAY,LITTLE FINE SAND					MPL	Shelby tube 3"x24" Recovery is 19"					
D3	Disturbed			CLAY & FINE SAND		SOFT			LPLASTIC	1	1	2	3		
D4	Disturbed									1	1	3	4		
D5	Disturbed	30'				MST				2	3	3	6		
D6	Disturbed			SILT & FINE SAND		LOOSE			NP	3	3	4	7		
D7	Disturbed	40'		SILT,LITTLE FINE SAND		MST			MC	3	3	5	8	Bottom Level of Foundation	
D8	Disturbed			CLAY,LITTLE FINE SAND		SOFT			MPLASTIC	1	1	1	2		
D9	Disturbed	50'								1	1	1	2		
D10	Disturbed									1	1	1	3		
D11	Disturbed	60'		FINE SAND, TRACE SILT, TRACE MICA		MD			MP	5	6	6	8		
D12	Disturbed			CLAY, LITTLE FINE SAND		SOFT			MED PLASTIC	2	2	2	3		
D13	Disturbed	70'								1	2	2	3		
D14	Disturbed									2	2	2	3		
D15	Disturbed	80'								2	2	2	5		

Fig. 27 Bore Log

Load carrying capacity/pile.

a) Individual action = πDCL = 3.14 x 0.20m x 7.21x 12.20 = 53.20 Kn.
Number of pile required = 845.50/5~~5~~3.20 = 15.30. Considering group efficiency and for arranging group use 21 nos.

m = No. of column = 7

n = No. of rows = 3

S = c/c spacing = 0.86 m.

D = Av. dia = 0.20 m.

b) Group action:

Perimeter of block = 2 (5.18 + 2.13) = 14.62 m.

Length of pile = 12.20 m.

Cohesion = 150 Psf = 7.21 Kn/m^2.

Allowable = 14.62 x 12.20 x 7.21 = 1285 Kn > 845 Kn. O.K.
(Without considering load carrying capacity of plan area).

$$\text{Group efficiency : } E_g = 1 - \frac{\phi}{90}\left[\frac{(n-1)m + (m-1)n}{m\cancel{m}n}\right]$$

$$\phi = \tan^{-1} d/s = 13.09^\circ.$$

$$E_g = 1 - \frac{13.09}{90}\left[\frac{(3-1)\,7 + (7-1)3}{3 \times 7}\right] = 1 - 0.22 = 0.78.$$

Total allowable load on piles considering individual action.
= 53.20 x 21 x 0.78 = 871 Kn > 845 Kn. So the pile design is O.K.

Settlement:

As it is located in homogenous clay, load is assumed to be distributed at 2/3rd from tip of pile. Thickness of clay layer = 20 ft = 6.10m.

Depth of centre of layer = 7.12 m from 2/3rd pile depth.

Therefore ΔP = 845.50/[(3.56 + 3.56 + 2.13) (3.56 + 3.56 + 5.18)]

$$\Delta P = \frac{845.50}{9.25 \times 12.30} = 7.43 \text{ Kn/m}^2 \quad .$$

eo = 1.10, Cc = 0.28.

As there is one value of Cc and eo at 6.10 meter depth, the whole layer is considered homogenous and settlement is considered in one layer only.

$$Po = \gamma_{sub} (8.13 + 4.07 + 3.05) = 8.24 \times 15.25 = 125.66 \text{ Kn/m}^2.$$

$$S = \frac{H}{1 + eo} \; Cc \log_{10} \frac{Po + \Delta P}{Po}, \quad H = 610 \text{ cm.}$$

$$S = \frac{610}{1 + 1.10} \times 0.28 \times \log_{10} \frac{125.66 + 7.43}{125.66}$$

$$= 290.48 \times 0.28 \times 0.0249 = 2.03 \text{ cm} = 0.80 \text{ inch.}$$

This is actual settlement. Adjoining column may also settle. So it is considered safe from settlement consideration.

21.0 PILE TYPE, DESIGN AND DRIVING

21.1 Timber Piles

Timber piles are frequently used in North America, China, USSR, and in Scandinavian countries in the form of trimmed tree trunks. When used in foundation work their lightness gives buoyancy to the foundation. Their lightness, flexibility, and resistance to shock makes timber piles very suitable for temporary works.

If timber piles are kept permanently wet or permanently dry i.e driven wholly below or wholly above water level, they can have a very long life. However, they are liable to decay in the zone of a fluctuating water table. Care during selection and treatment of timber can prevent or minimize attack by wood destroying insects.

Those parts of timber piles which are permanently buried in the ground and below the lowest ground water level may not need treatment. Whenever possible, the concrete pile caps should be taken down so that their under sides are below ground water level and the portion of the pile embedded in the caps should be cut off square to sound wood and liberally coated with creosote or other preservative.

Mohr has suggested that working loads on timber piles should be restricted to 100-250 Kn in order to avoid unseen damage to the buried portions of the pile as a result of hard driving to achieve high carrying capacity. It has been stated by Menzies that the timber piles are usually used as friction piles, but on occasions, as point bearing piles. The weight of the hammer should be at least equal to the weight of the pile for hard driving condition and not less than half the weight of the pile for easy driving.

Pile lengths upto 20 m and loads upto 600 Kn are usual. Where end bearing piles are driven into dense or hard materials, a shoe is required to prevent splitting of the pile point. A typical shoe for timber is shown in **Fig.** 28a. Shoe is not required when piles are driven wholly in soft ground. It is usual in British practice to provide a steel hoop around the head of the pile to prevent damage during driving. Piles are generally in 6-12 meter lengths but they may be spliced if longer piles are required (**Fig.** 28e).

21.2 Precast Concrete Piles

Precast concrete piles are widely used for structures such as wharves or jetties where the piles are required to be carried above ground level in the form of a structural column. They are also used if the soil conditions are unfavourable to cast-in-place piles, and if the situation demands a high resistance to lateral forces for example, in foundations for crane gantries or heavy reciprocating machinery or as anchor piles to the ties of retaining walls.

Precast concrete piles are normally of square section for short and moderate lengths, but hexagonal, octagonal or circular piles are usually preferred for long lengths. If soil conditions require a large cross-sectional area, precast concrete tubes are used. The tubes are driven hollow and can be filled with concrete after driving.

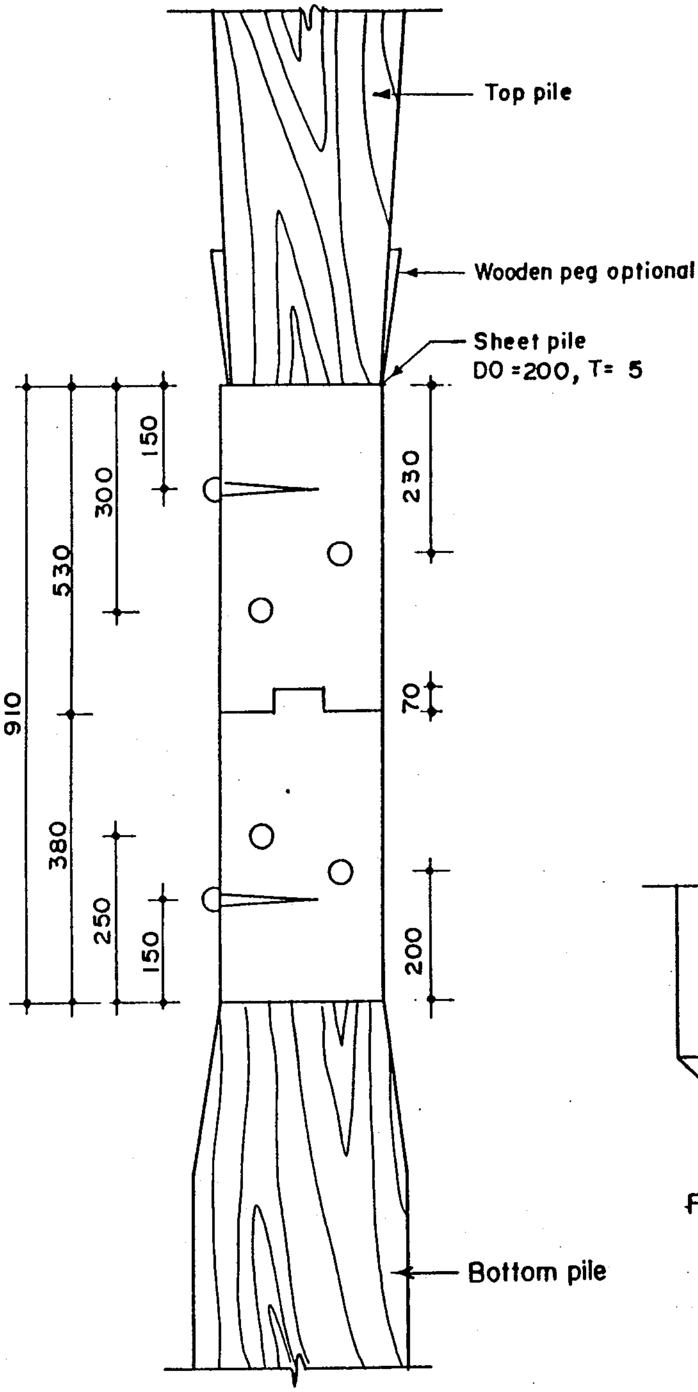


Fig. 28e. Typical joint details

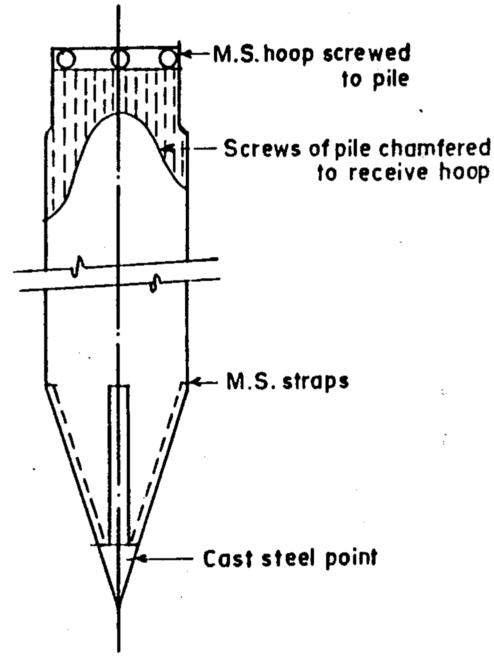
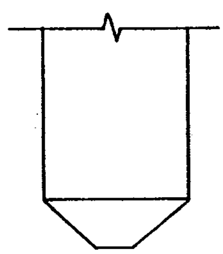
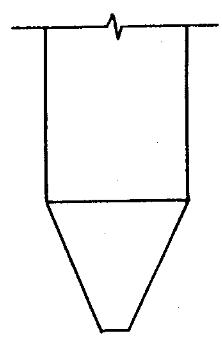


Fig. 28a. Head and Toe of Timber pile



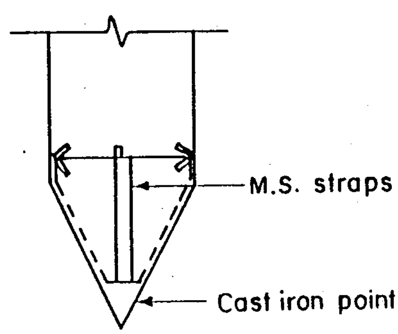
A

Fig. 28b.



B

Fig. 28c.



C

Fig. 28d. Types of pile for various ground conditions

A = soft, B = stiff

C = contaning boulders

Note:-
All dimensions are in mm.

To avoid complications during handling and driving, the usually accepted maximum lengths for various square-section piles are as in **Table 14.**

Table 14. Relationship between size and length of precast concrete piles.

Pile size (mm square)	Maximum length (m)
250	12
300	15
350	18
400	21
450	25

21.3 Design

The structural design of precast concrete piles is governed by stresses caused by lifting, handling and subsequent driving. Once the piles are driven to their final position the stresses caused by foundation loading are likely to be much lower than those induced by handling and driving.

The design of piles to resist driving stresses is mostly based on the researches of Glanville, Grime and Davies at the Building research station. They embedded stress-recorders in piles to measure the magnitude and velocity of the stress waves, which after each blow travels from the head of the pile to the toe where it is partly reflected to return to the head. The researches showed that the stresses produced in a pile by the hammer blows far exceed those given by the driving resistance (calculated by dynamic formula) divided by the cross-sectional area of the pile. The driving stresses were found to depend almost wholly on the fall of the hammer and the nature of the packing between the helmet and the pile. They made the following recommendations for the provision of reinforcement.

(a) The quantity of longitudinal steel should be proportional to the stresses caused during lifting and handling. Research showed that the proportion of main steel did not seem to have any effect on the resistance to driving stresses.

(b) The quantity of transverse reinforcement, where hard driving is expected, should not be less than 0.40 percent of the concrete volume.

(c) The proportion of link steel in the head of the pile should be 1.0 percent.

The lateral reinforcement is of particular importance in resisting driving stresses and should be in the form of hoops or links and of diameter not less than 6 mm (CP 2004). The volume of lateral reinforcement should not be less than 0.6 percent of the gross concrete volume for a distance of about 3 times the pile width from both ends of the pile. In the body of the pile the lateral reinforcement should be not less than 0.2 percent, spaced at not more than half the width of the pile. The transition between the close spacing near the ends and the maximum spacing in the middle should be made gradually over a length equal to three times the width.

Clear cover over all kinds of reinforcement, including binding wire, should not be less than 40 mm (1.5 in) of concrete but where the piles are exposed to sea water or other corrosive environment, the cover should not be less than 50 mm (2 in). The hoops and links should fit tightly against the longitudinal bars and be bound to them by welding or by soft iron wire, the free ends of which should be turned into the interior of the pile. The longitudinal bars may be held apart by spreader forks not more than 1.5 m (5 ft) apart.

It should be noted that the percentage of lateral reinforcement in the form of links or ties in the head and body of the pile as recommended by the code is only about half the quantity recommended by Glanville, Grime and Davies. However, the latter recommendations were made for hard driving conditions, whereas the figure given in the code are minimum value for easy driving conditions. Bending moments for a variety of conditions of support in lifting and handling a pile of weight W and length L are given in **Table** 15.

Table 15. Bending Moments for lifting of precast of piles

	Condition			Maximum static moment
a)	Lifting by two points	1/5xL	from either end	± WL/40
b)	- do -	1/4xL	- do -	± WL/32
c)	Pitching by one point	3/10xL	from the head	± WL/22
d)	" - do -	1/3xL	- do -	± WL/18
e)	- do -	1/4xL	- do -	± WL/18
f)	- do -	1/5xL	- do -	± WL/14
g)	Pitching by one end			± WL/8
h)	Balancing by the centre.			± WL/8

The above conditions are illustrated in **Fig.** 29. From design charts published by the cement and concrete association maximum length for square sectioned piles for various lifting conditions are shown in **Table** 16 & **Fig.** 30. The design charts also give the bending moments due to self weight while lifting, and ultimate resisting moments for various square piles. Typical detail of R.C.C. pile is shown in **Fig.** 31.

21.4 Pile Casting

CP 2004 recommends that for piles under very hard driving conditions including piles in marine works, a concrete mix with a minimum cement content of 400 kg/m^3 (cube strength 25 N/mm^2 at 28 days) should be used. For normal and easy driving conditions the code suggests that a cement content of 300 kg/m^3 (cube strength 20 N/mm^2) can be used. Where hard driving is expected it may be advantageous to adopt even richer mixes in the head and toe of the pile, say of the order of 600 kg cement per m^3 of cement.

Normally Portland cement is used for piles. High alumina cement concrete is advantageous from the point of view of early release from the form work and a reduced curing period but there is evidence of a substantial retrogression in strength of high alumina cement concrete during or subsequent to the curing period.

Piles may be cast directly on to a concrete bed or bottom forms may be laid on rigid timber supports. The casting bed should be laid on sufficient depth of well compacted hard filling to prevent settlement of the bed under the weight of the pile. The side forms should be removed as soon as practicable and the piles should be kept continuously wet for at least four days in normal air temperature conditions. At some stage after casting, the piles should be clearly marked with a reference number, their length and the date of casting. It will prevent driving too soon after casting. Before being driven, the piles should be carefully inspected

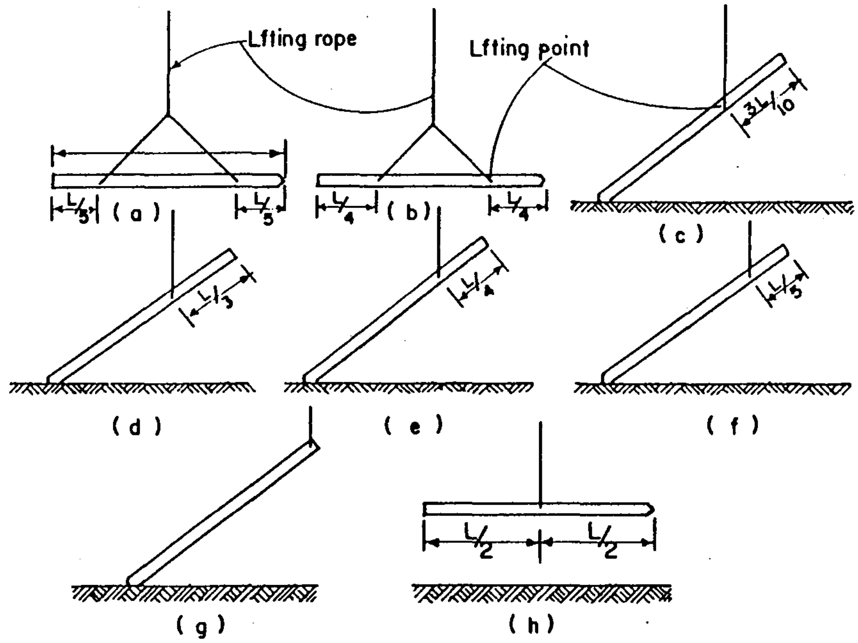


Fig. 29 Method of lifting reinforced concrete piles

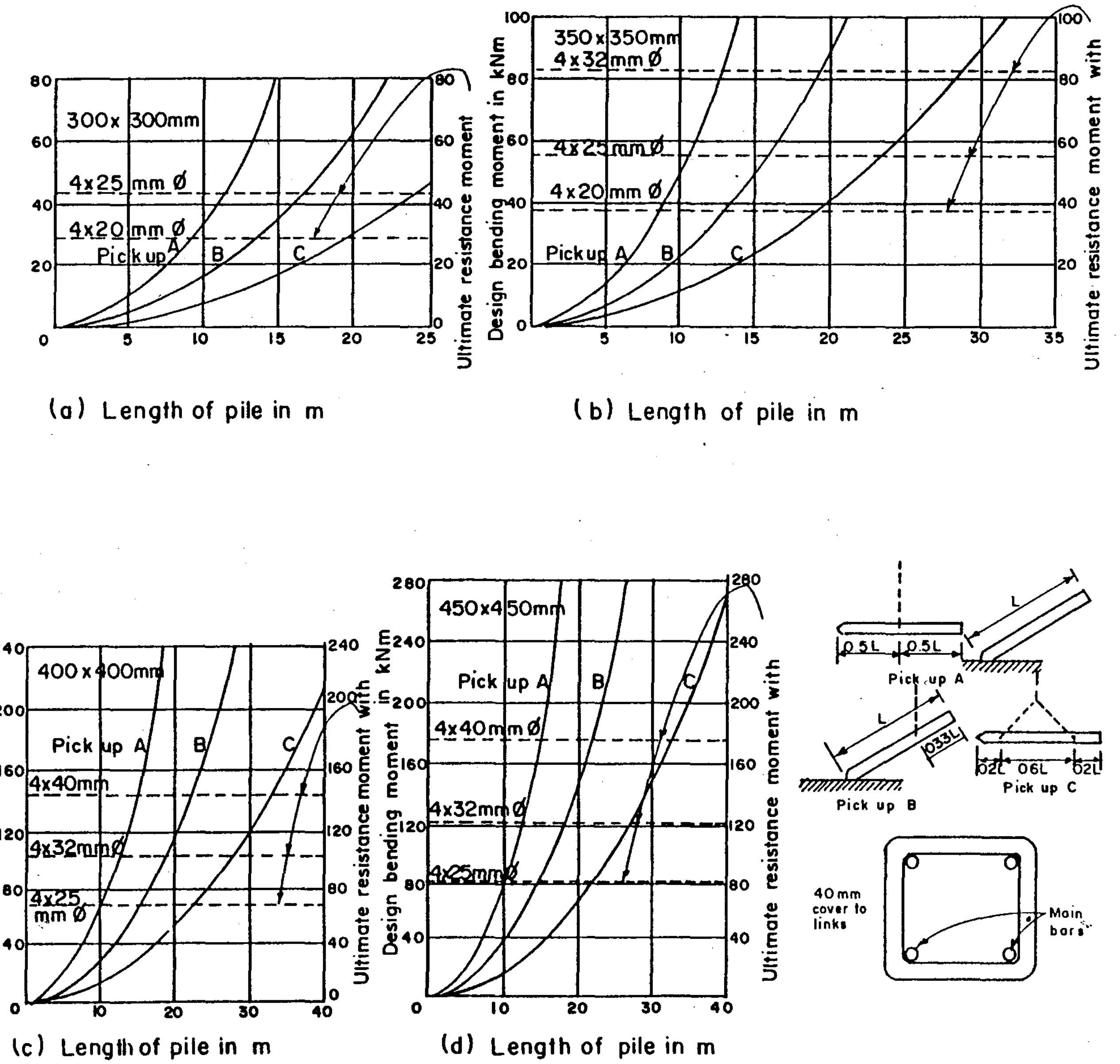


Fig. 30 diagrams showing required lifting points for reinforced concrete piles of various cross - sections.

Table 16. **Maximum lengths of square section precast concrete piles for given reinforcement**

Pile size (mm)	Main reinforcement	Maximum length in meters for pick up at: head and toe	0.33 x length from head	0.2 x length from head and toe	Transverse reinforcement: Head and toe	Body of pile
300 x 300	4 x 20 mm	9.0	13.5	20.0	6 mm at	6 mm at
	4 x 25 mm	11.0	16.0	24.0	40 mm crs	130 mm crs
350 x 350	4 x 20 mm	8.5	13.0	19.0	8 mm at	8 mm at
	4 x 25 mm	10.5	16.0	23.5	70 mm crs	175 mm crs
	4 x 32 mm	12.5	19.0	28.5		
400 x 400	4 x 25 mm	10.0	15.0	22.5	8 mm at	8 mm at
	4 x 32 mm	12.5	18.5	28.0	60 mm crs or	200 mm crs
	4 x 40 mm	15.0	22.0	33.0	10 mm at 100 mm crs	
450 x 450	4 x 25 mm	10.0	14.5	22.0	8 mm at	8 mm at
	4 x 32 mm	13.0	18.5	27.0	60 mm crs	180 mm crs
	4 x 40 mm	15.0	22.0	32.0	or 10 mm at 90 mm crs	or 10 mm at 225 mm crs

Notes: Piles designed in accordance with CP 110 and CP 2004.
Characteristic strength of mild steel reinforcement = 250N/mm²
Cover to link steel = 40 mm.
Characteristic strength of concrete = 25 N/mm²

to ensure that they are free from any cracks. Fine transverse cracks can result from careless handling or lifting when they are of an immature age. Some code of practice permit piles which are cracked during driving to remain in position. The German code does not regard cracks narrower in width than 0.15 mm to be detrimental. The Swedish Code also permits cracks upto 0.2mm width with a length not exceeding one half of the pile circumference for transverse cracks or 100mm for longitudinal cracks.

After driving the pile to the desired level, it is usual to strip the pile head to expose the reinforcement which is then bonded into the pile cap. If piles are required to be driven deeper than anticipated it may be necessary to extend them by splicing the reinforcement and casting on an additional length. The concrete in the head of the pile already driven should be broken away to expose the reinforcement for a distance of 40 times the diameter of the longitudinal bars, and the reinforcing bars for extension length should be welded at the joints or lapped for full 40 diameters.

21.5 Helmet, Driving Cap, Dolly and Packing

During pile driving helmets or caps are placed on the top of the pile to receive the blows of the hammer and to prevent damage to the head of the pile. The purpose of the cast steel helmets placed on top of concrete piles (**Fig.** 32) is to hold the resilient dolly and packing which are interposed between the hammer and the pile to prevent shattering of the latter at the head. The helmet should fit loosely around the pile, so that the latter can rotate slightly without binding on the helmet. The dolly is placed in a square recess on top of the helmet. It is square at the base and rounded at the top. Elm dollies are used for easy to moderate driving and for hard driving a hard wood is used. Plastic dollies can withstand much heavier driving than timber dollies. In moderately hard driving conditions, plastic dollies will last for several hundred piles.

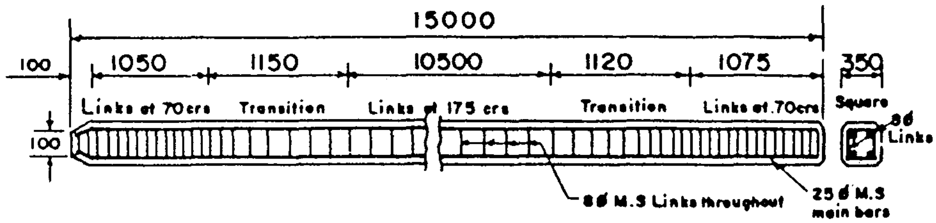


Fig. 31 Typical details of precast reinforced concrete piles

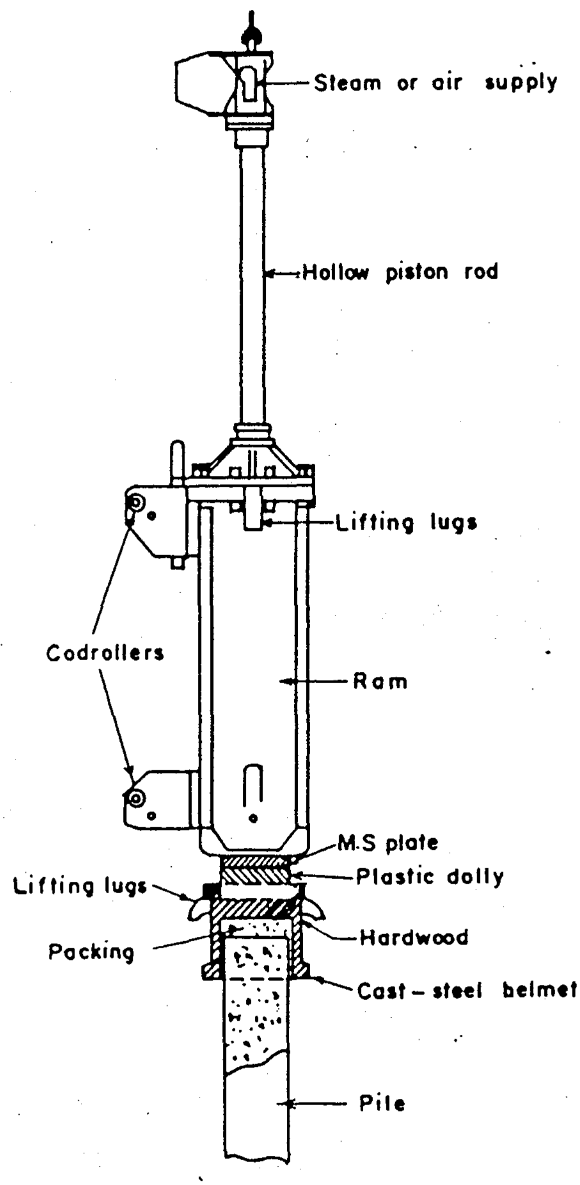


Fig.32 B.S.P. single-acting hammer driving pile
with cast steel helmet and plastic dolly.

Packing is placed between the helmet and the top of the pile to cushion the blow between the two. Various type of materials like hessian, thin timber sheets, coconut matting and wall board etc. are used as packing.

Driving caps are used to protect the heads of the steel bearing piles. They are specially shaped to receive particular type of pile to be driven and are fitted with a recess for a hard wood dolly and with steel wedges to keep the caps tight on the pile. If the caps are allowed to work loose they well damage the pile head. Generally, great care is needed in selecting materials, and the thickness of dollies and packing, since lack of resilience will lead to excessive damage to the pile head.

21.6 Pile Driving

Pile are commonly driven by means of a hammer supported by a crane or by a special device known as a pile driver. The hammer is guided between two parallel steel members known as leads. The loads are carried in a frame in such a way that they can be adjusted at various angles for driving vertical and batter piles. Several type of hammers are in use and they have different sizes too. The common types are:

1. Drop hammer : If a hammer(ram or monkey) is raised by winch and allowed to fall by gravity on the top of a pile, it is called a drop hammer. This type is slow and therefore, not economical. It is used for cases where small numbers of piles are driven.

2. Single acting hammer: If the hammer is raised by steam compressed air or by internal combustion engines and is allowed to fall by gravity it is called a single acting hammer. The charge of single acting hammer is equal to weight of the ram times the height of the fall.

3. Double acting hammer: The double acting hammer employs steam or air for lifting the ram and for accelerating the downward stroke. It operates with a succession of rapid blows.

4. Diesel hammer : The diesel hammer is small, light weight, self contained and self acting type. Total driving energy is the sum of the impact of the ram plus the energy delivered by explosion.

5. Vibratory hammer : The driving unit vibrates at high frequency.

21.7 Selection of Pile Hammer

Pile hammers are available in a wide range of weights, types and driving energy. The choice of hammer type and size for a given job depends upon many factors. Generally, the size is more important factor than type. A heavy pile should be driven by a heavy hammer delivering large energy. Preferably the weight of hammer should be at least one-half the total weight of the pile, and the driving energy should be at least one foot-pound for each pound of pile weight. **Table** 17, 18, & 19 show the suggested hammer coefficients and hammer weights for different driving conditions.

Single acting hammers are advantageous when driving heavy piles in compact or hard soils, the heavy ram striking at low velocity produces least damage due to impact. For single acting hammer the maximum height of drop is usually 1.37 m and the hammer can be operated at a rate up to about 60 strokes per minute. Single acting hammers are made with cylinder masses varying from 2500 to 20,000 kg. A useful rule to determine a suitable weight of drop hammer or single acting hammer is to select a hammer weighing approximately the same as the pile. This will not be possible in case of a heavier reinforced concrete piles when the hammer is more likely to be half of the weight of pile. However, it should not weigh less than a third of the pile. In order to avoid damage to the pile, the height of drop should be limited to 1.20 m.

Table 17. Values of hammer coefficient K after BSP Pocket Book (1969)

Hammer	K
Drop hammer, which operated	0.8
Drop hammer, tagger release	1.0
Single-acting hammer	0.9
BSP double-acting hammer	1.0
McKiernan-Terry diesel hammer	1.01

Table 18. Values of the hammer coefficient e, after BSP Pocket Book (1969)

Type of pile	Head condition	Single-acting or drop hammer or diesel hammer	Double acting hammer
Reinforced concrete	Helmet with composite plastic or Greenheart dolly, and packing on top of pile	0.4	0.5
	Helmet with timber dolly (not Greenheart) and packing on top of pile	0.25	0.4
	Hammer directly on pile with pad only	-	0.5
Steel	Driving up with composite plastic or Greenheart dolly	0.5	0.5
	Driving up with timber (not Greenheart) dolly	0.3	0.3
	Hammer directly on pile	-	0.5
Timber	Hammer directly on pile	0.25	0.4

Table 19. Data for Selection of Pile Hammers for Driving Concrete, Timber, and Steel Sheet piling Under Average and Heavy Driving Conditions*

Length of pile (ft)	Depth of penetration (per cent)	Sheet pile Light	Sheet pile Medium (ft-lb per blow)	Sheet pile Heavy	Timber pile Light (ft-lb per blow)	Timber pile Heavy	Concrete pile Light (ft-lb per blow)	Concrete pile Heavy
1. Driving through Earth, Sand, Loose Gravel-Normal Frictional Resistance								
25	50	1000-1800	1000-1800	1800-2500	3600-4200	3600-7250	7250-8750	8750-150
	100	1000-3600	1800-3600	1800-3600	3600-7250	3600-7250	7250-8750	13000-15000
50	50	1800-3600	1800-3600	3600-4200	3600-8750	7250-8750	8750-15000	13000-25000
	100	3600-4200	3600-4200	3600-7500	7250-8750	7250-15000	13000-15000	15000-25000
75	50		3600-7500	3600-8750		13000-15000		19000-36000
	100			3600-8750		15000-19000		19000-36000
2. Driving through Stiff Clay, Compacted Gravel - Very Resistant								
25	50	1800-2500	1800-2500	1800-4200	7250-8750	7250-8750	7250-8750	8750-15000
	100	1800-3600	1800-3600	1800-4200	7250-8750	7250-8750	7250-15000	13000-15000
50	50	1800-4200	3600-4200	3600-8750	7250-15000	7250-15000	13000-15000	13000-25000
	100		3600-8750	3600-13000		13000-15000		19000-36000
75	50		3600-7500	3600-13000		13000-15000		19000-36000
	100			7500-19000		15000-25000		19000-36000
Weight (per in,ft)		20 lb	30 lb	40 lb	30 lb	60 lb	150 lb	400 lb
Pile size (approx.)		15 in.	15 in.	15 in.	13 in. diam	18 in. diam.	12 in.^{2}	20 in.^{2}

Tennesse Valley Authority. + Energy required in driving single sheet pile. Double these when driving two piles at a time.

Double acting hammers are used mainly for sheet pile driving, and piles of light and moderate weight in soils of average resistance. The rate of driving ranges from 300 blows per minute for the heavy types and 100 blow per minute for light types. The mass of the ram is generally in the range of 90-2300 kg, imparting a driving energy of 165 - 2700 kg per blow. Double acting hammers can be driven by steam or compressed air.

Diesel hammers are of similar application as double acting hammers. They provide an efficient means of pile driving if the ground condition is favourable, but are not effective for all types of soil. These hammers are economical as they do not need steam or compressed air plants and are entirely self contained. Diesel hammers are ineffective in soft or yielding soils where the impact of the blow is insufficient to atomize the fuel.

21.8 Pile Shoes

When piles are driven wholly in soft soils no shoe need to be provided. The ends of the piles are usually cast in the shape of a blunt point. A sharper point (**Fig.** 28c) is preferred for driving into hard clays or compact sands and gravels. Where the piles are to be driven into soil containing large cobbles or boulders, shoe as shown in **Fig.** 28d is needed to split the boulders to one side. The area of the top of the metal shoe in contact with the pile should be large enough to ensure that the compressive stress on the concrete is within safe limits.

22.0 BORED PILES

If the length of pile is more than usual (above 15m), the handling stress is high and there is possibility of cracking during handling. In such cases possibility of using bored piles may be examined. It will reduce number of piles. In bored piles nominal reinforcement is provided to take care of moments due to soil movement (0.5 to 1%).

Advantages

1) Length can be readily varied to suit varying ground condition.
2) Soil removed in boring can be inspected and if necessary sampled or in situ tests made.
3) Can be installed in very large diameters.
4) End enlargements upto 2 to 3 times the diameter are possible in clays.
5) Material of pile is not dependent on handling or driving conditions.
6) Can be installed in very long lengths.
7) Can be installed without appreciable noise or vibration.
8) No risk of ground heave.

22.1 Concreting Bored Piles

In dry boreholes or in holes where water can be removed by bailing, the piles are concreted simply by tipping a reasonably workable mix (not leaner than 300 kg cement per cubic meter of concrete) down the bore hole from a borrow or dumper. A hopper is to be provided at the mouth of the hole to receive the charge of concrete and to prevent contamination with earth. Before any concrete is placed, the hole should be bailed dry of water and loose or softened soil should be cleaned out and the bottom of the hole rammed.

If the bottom of the hole is wet, a layer of dry concrete should first be placed and rammed well. Then a workable mix (75-100 mm slump) of concrete is poured in such a way that segregation is minimum.

In concreting bored cast insitu piles in water bearing soils, in squeezing ground and or a combination of two (for sand or soft clays) the pile casting demands greater care. The concrete should be rich and easily workable (125-180 mm slump). When the bore hole cannot be bailed or pumped dry before placing the concrete any of the following two methods are may be adopted.

<u>Method - 1</u>

The bore hole must be lined with a casing throughout its depth and a plug of concrete should then be deposited under water in the base of the hole by tremie pipe.

As soon as concrete has hardened sufficiently, the hole should be pumped free of water. The casing should then be gently turned and lifted slightly to free it from the plug. The remainder of the concrete in the shaft should be placed in dry condition upto the surface. The casing should then be lifted entirely from the bore hole (a casing vibrator is a useful device to assist lifting), additional concrete being placed as necessary after the concrete has slumped to fill the void left by withdrawing the casing.

<u>Method - 2</u>

The entire shaft may be concreted under water using a tremie pipe. Concrete placed through a tremie pipe should be easily workable (125-180 mm slump) and should have a minimum cement content of 400kg kg/m^3. A retarder should be used if there is a risk of the concrete setting before the casing is lifted out of the hole. The finished level of concrete placed by tremie pipe should be higher than the design cut off level to permit the removal of the thick layer of laitance which always forms on the rising surface of concrete placed by tremie method. It may be necessary to remove 300 mm or more of laitance and contaminated concrete.

If bentonite mud instead of casing is used to support the wall of a pile bore hole, concrete should be placed beneath bentonite by means of tremie pipe. It is necessary to keep an adequate head of concrete in the tremie pipe and to keep it moving continuously in order to overcome the pressure of high density thixotropic bentonite fluid which is displaced by the out flowing concrete.

22.2 Design Example

In example 4, (Ref. Bore log - **Fig.** 24) 56 Nos. of 13.25 m long piles have been used. If it is replaced by bored pile, the number will be reduced. Find out the size, number and length.

Dia = 0.45 m (assume).

L = 16.70 m (assume).

Top 5.20 meter is clay, qu = 23 Kn/m^2

Next 8.50 m is non plastic silt.

Next 3.00 m is sand.

Design

First 5.20 m clay. Average qu = 23 Kn/m^2.

Allowable skin friction = 0.45 x 23 = 10.35 Kn/m^2.

(Adhesion factor 0.45).

Total skin friction = π x 0.45 x 5.20 x 10.35 = 76.05 Kn.

For middle part

$f_s = K_s \gamma_s D \tan\delta$, $K_s = 1$

consider loose silt. $\phi = 20^\circ$, $3/4\ \phi = 15^\circ = \delta$

$f_s = 1 \times 9.45 \times 9.13 \tan 15 = 86.28 \times 0.268 = 23.12\ Kn/m^2$

Total resistance in the mid part = 8.50 x 3.14 x 0.45 x 23.12 = 279 Kn.

For last 3.00 m. N = 10, $\phi = 28^\circ$ $\delta = 21^\circ$. fs = 15.20 x 9.13 x tan 21°

= 53.27 Kn/m^2.

Total resistance = 3.00 x 3.14 x 0.45 x 53.27 = 225.82 Kn.

Base resistance

$Q_b = A_b \gamma_s D (N_q-1)$. Where Q_b = Total base resistance. $\gamma_s D$ = Overburden pressure at pile base level. Nq = Berezantev's bearing capacity factors (Fig. 33).

$N_q = 10$

$$Q_b = \pi \times \frac{0.45^2}{4} \times 16.70 \times 9.13 \times 9 = 218.53 \text{ Kn.}$$

Total resistance of pile = 76.05 + 279.00 + 225.82 + 218.53 = 799.40 Kn.

Design by volume displacement method:

(Nordlund empirical method for calculation of skin resistance for cohesionless soil). $V = 0.45^2 \times 1 = 0.20$ m^3/m. N value = 5,
$\phi = 20^\circ$, $\delta/\phi = 0.85$ $\delta = 17^\circ$. **Fig.** 34.

From curve in **Fig.** 35, $K_\delta = 0.88$, Correction factor 0.95 (**Fig.** 36).
Corrected $K_\delta = 0.84$

$Q_s = \Sigma K_\delta \gamma_s D_f \sin\delta\, C_d.D$; K_δ = A dimensionless factor, D = Depth of pile over which skin friction is calculated, $\gamma_s D_f$ = Average effective overburden pressure over D, δ = Angle of wall friction, Cd = Minimum perimeter of pile.

$Q_s = 0.84 \times 86.28 \times \sin 17^\circ \times 3.14 \times 0.45 \times 8.50 = 254.50$ Kn.
Last part. $\phi = 28$. $\delta/\phi = 0.85$, $\delta = 23.80$, $K_\delta = 1.20$ (**Fig.** 35).
Correction factor = 0.95. Corrected $K_\delta = 1.12$ (**Fig.** 36).
$Q_s = 1.12 \times 138.32 \times \sin 23.80^\circ \times 3.14 \times 0.45 \times 3.00$
$= 656.70 \times \sin 23.80^\circ = 265.00$ Kn.

Total capacity = 76.05 + 254.50 + 265.00 + 218.53 = 814.08 Kn.
By the 1st method total capacity = 799.40 Kn.
By the 2nd " " " = 814.08 Kn.

Allowable load/pile with F.S = 2.50 is,
All Capacity = 799.40 ÷ 2.50 = 319.76 Kn.

$$\text{No of piles} = \frac{4401}{319.76} = 13.76 \text{ , use 16 nos.}$$

This will increase F.S to 2.90. OK.

Use nominal reinforcement between 0.5 to 1%, use 6 nos. 19 mm dia bars.

Tie. 8 mm ϕ @ 10 cm c/c for top 1.5 m and bottom 1.50 m.

Rest 8 mm ϕ @ 20 cm c/c.

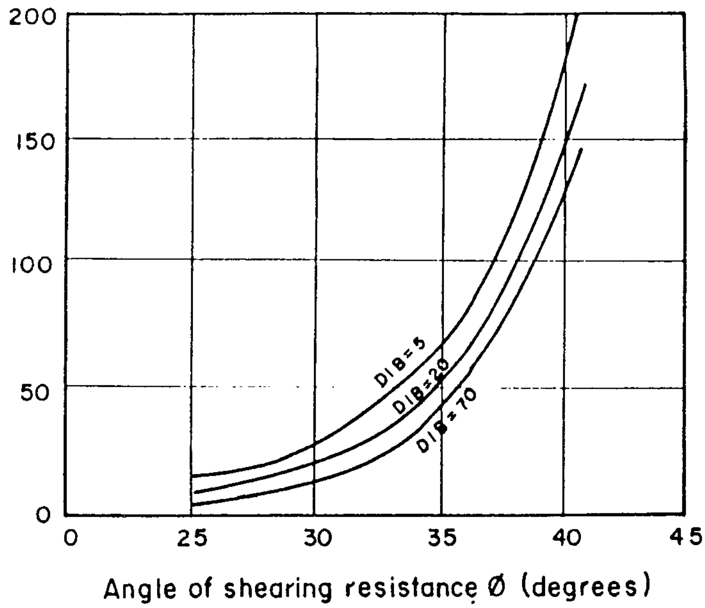


Fig. 33. Berezantev's bearing capacity factor Nq.

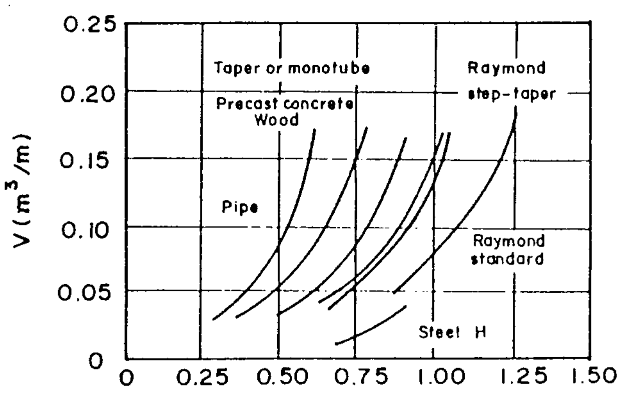


Fig. 34. Design curves for calculating skin friction on piles in cohensionless soils with relationship between V & δ/ϕ for various types of piles.

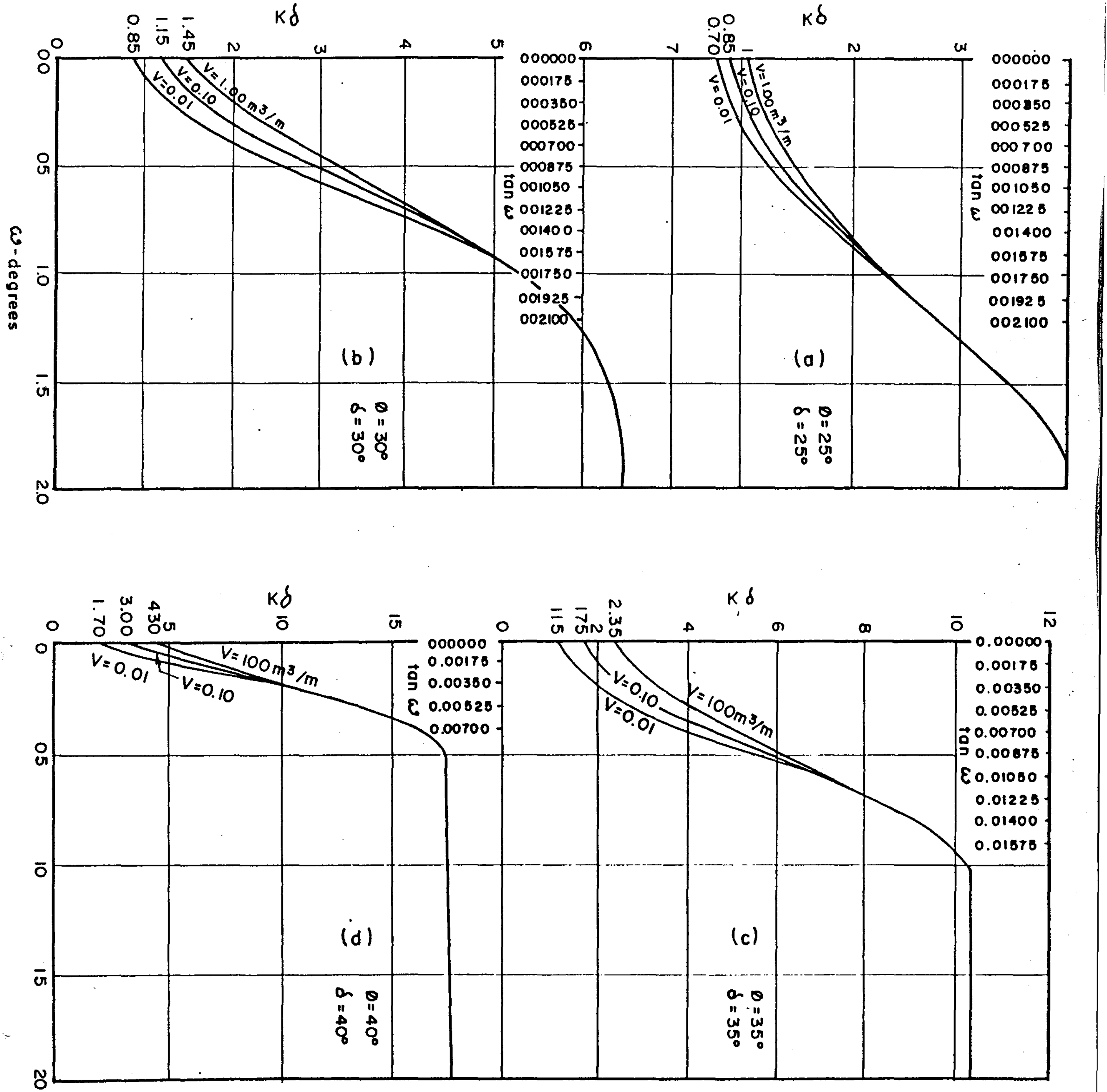


Fig. 35. Design curves for calculating skin friction on piles in cohensionless soils relating Kδ to Ø

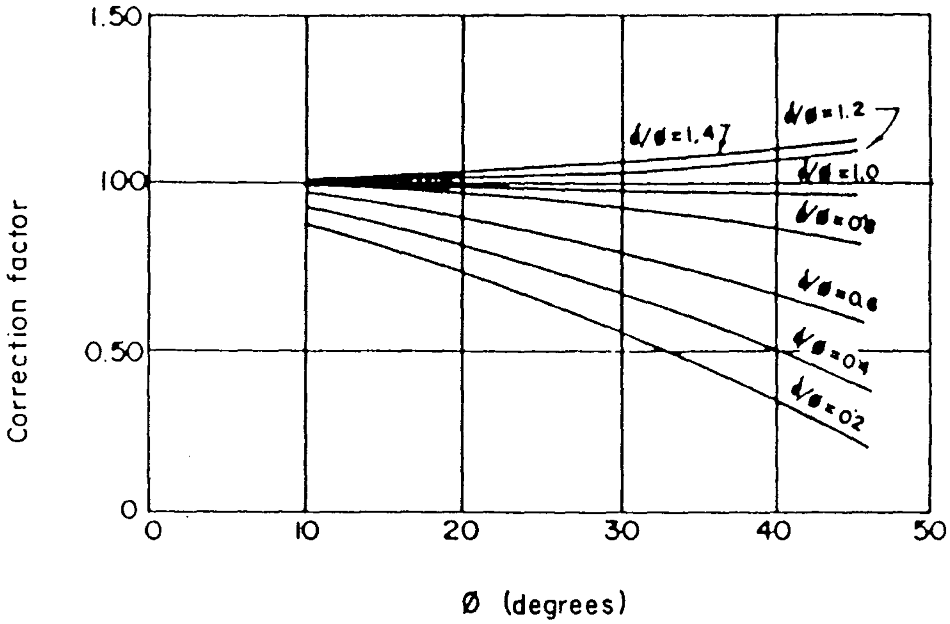


Fig. 36 Design curves for calculating skin friction on piles in cohensionless soils with correction factors $K\delta$ when δ is not equal to $\emptyset$

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CHAPTER SEVEN

STRUCTURAL DESIGN OF REGULATOR

A. DESIGN CRITERIA

B. DESIGN CONSIDERATIONS

C. DESIGN EXAMPLE

A. H. BHUIYAN

STRUCTURAL DESIGN
OF REGULATOR

A. DESIGN CRITERIA

PART - A: DESIGN CRITERIA

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1.0 GENERAL DESIGN REQUIREMENTS

Any structure should be so designed that it remains stable against external loads and pressures and different members of the structure are strong enough to resist external loads, forces and internal stresses. Considering this, the structural design of regulator should be undertaken after stability analysis.

Stability : The stability design of the structure should fulfil the following four requirements:

a) It must be safe against overturning at any horizontal plane.

b) It must be stable against sliding or shearing force on any horizontal plane, including planes in foundation materials.

c) It must be stable against water uplift.

d) It shall not over stress the underlying foundation material or soil.

Structural : An efficient design of a reinforced concrete structure requires that the materials be economically selected, proportioned and arranged to carry the required loads and pressures without developing stresses which are in excess of the allowable working stresses. Correct estimation of all types of loads should be made before taking up the design proper.

The structural design requirements may be summarised as follows :

a) The sectional area of concrete should be sufficient to take care of the compressive stresses caused by bending or direct load.

b) Sufficient longitudinal reinforcements should be provided to take care of the tensile stresses caused by bending or direct load.

c) Sufficient anchorage should be provided for all reinforcement bars so that bond and anchorage requirements are met.

d) Sufficient shear reinforcement should be provided to keep the shearing stresses within permissible limits.

2.0 STRUCTURAL DESIGN CRITERIA

2.1 Code of Practice

For structural design, the codes of ACI, USBR and AASHTO may be followed. Where they do not fit local conditions engineering judgement should be exercised to determine suitable criteria.

2.2 Permissible Stresses

In this text working stress design method has been followed for all structural designs. The permissible stresses of concrete and steel are as given in the following sub-sections:

2.2.1 Concrete

i) Modulus of elasticity of concrete

$$Ec = 4.73 \times 10^3 \ f'c^{\frac{1}{2}} \ (5.7 \times 10^4 \ f'c^{\frac{1}{2}} \text{ psi})$$

ii) Flexural stress

$$fc = 0.40 \ f'c$$

where f'c = extreme fibre stress of concrete in compression

iii) Shear stress

<u>Flexural (without web reinf.)</u>

$$Vc = 9.10 \times 10^{-2} \ f'c^{\frac{1}{2}} \ \text{N/mm}^2 \ (1.1 \ f'c^{\frac{1}{2}} \text{ psi})$$

Flexural (with web reinf.)

$Vc = 0.42\ f'c^{\frac{1}{2}}\ N/mm^2\ (5\ f'c^{\frac{1}{2}}\ psi)$

$Punching = 0.17\ f'c^{\frac{1}{2}}\ N/mm^2\ (2\ f'c^{\frac{1}{2}}\ psi)$

iv) Bond Stress

Top bar : $3.6\ \frac{f'c^{\frac{1}{2}}}{d} \leq 1.10\ N/mm^2\ (1.7\ \frac{f'c^{\frac{1}{2}}}{d} \leq 160\ psi)$

Other bar: $5.1\ \frac{f'c^{\frac{1}{2}}}{d} \leq 1.10\ N/mm^2\ (2.4\ \frac{f'c^{\frac{1}{2}}}{d} \leq 160\ psi)$

Where d = diameter of the bar (mm or in)

v) Bearing Stress

Load on full area:

$= 0.25\ f'c\ N/mm^2\ (0.25\ f'c\ psi)$

Load on one third area or less:

$= 0.375\ f'c\ N/mm^2\ (0.375\ f'c\ psi)$

2.2.2 Reinforcing Steel

(i) Modulus of elasticity of steel

$Es = 2.00 \times 10^5\ N/mm^2\ (2.90 \times 10^7\ psi)$

(ii) Working Stress

$$fs = 0.45\ fy$$

where, fy = yield stress

2.2.3 Design constants

$$n = Es/Ec$$

$$r = fs/fc$$

$$k = \frac{fc}{(fs/n) + fc}$$

$$j = 1 - k/3$$

$$R = 0.5\ fc\ kj$$

Table 1 shows the values of design constants for different coarse aggregates.

Table 1 . Design Constants

Design Para-meters	Coarse aggregate type: Shingle/Crushed Stone	Coarse aggregate type: Jhama/1st. class brick chips
f'c	19.3 N/mm^2 (2,800 psi)	17.3 N/mm^2 (2,500 psi)
fy	276 N/mm^2 (40,000 psi)	276 N/mm^2 (40,000 psi)
fs	124 N/mm^2 (18,000 psi)	124 N/mm^2 (18,000 psi)
fc	7.7 N/mm^2 (1,120 psi)	6.9 N/mm^2 (1000 psi)
n	9.6	10
r	16	18
k	0.373	0.357
j	0.876	0.881
R	1.30 N/mm^2 (183 psi)	1.1 N/mm^2 (157 psi)
Es	2.00×10^5 N/mm^2 (2.90×10^7 psi)	2.00×10^5 N/mm^2 (2.90×10^7 psi)
Ec	2.08×10^4 N/mm^2 (3.02×10^6 psi)	1.97×10^4 N/mm^2 (2.85×10^6 psi)
Vc	0.40 N/mm^2 (58 psi)	0.377 N/mm^2 (55 psi)

2.3 Reinforcement

2.3.1 Reinforcement bar

The most commonly used plain mild steel reinforcement bar has the following sizes, areas and weights (**Table** 2.)

Table 2. Details of Reinforcement Bar

Bar Dia (inch)	Bar Dia (mm)	Area (mm^2)	Wt (kg/metre)
1/4"	6	28	0.25
3/8"	10	78	0.56
1/2"	12	113	1.00
5/8"	16	201	1.56
3/4"	19	283	2.24
7/8"	22	380	3.05
1"	25	491	3.98
1-1/8"	28	616	5.03
1-1/4"	32	804	6.21

2.3.2 Spacing of Reinforcement

The maximum reinforcement spacing is twice the thickness of the member for main reinforcement (stress bars) and three times the thickness of the members for distribution reinforcement (temperature bars). In either case the maximum spacing is 460 mm (18 in) with a preferred limit of 300 mm (12 in).

Distribution bars in the vertical direction on the water face of a hydraulic structure shall always be 12 mm (½ in) diameter at 450 mm (18 in) centre to centre. The clear distance between parallel bars (except in columns and between multiple layers of bars in beams) should not be less than the nominal diameter of the bars, 1.33 times the maximum size of the coarse aggregate or 25 mm (1 in).

In spirally reinforced and tied columns, the clear distance between longitudinal bars should not be less than $1\frac{1}{2}$ times the bar diameter, $1\frac{1}{2}$ times the maximum size of the coarse aggregate or 40 mm ($1\frac{1}{2}$ in).

2.3.3 Splice and Bond Length

For a fully stressed plain bar spliced in the tension zone, a splice length of 48 bar diameter is to be used. When spliced in compression zone, a splice length of 40 bar diameter is to used. The bond or development length of bars should not be less than fs. d/4u, where d is the diameter of bar and u is 0.8 times the allowable bond stress. The splice and bond lengths of different bars are given in Table 3 and Table 4 respectively.

Table 3. Splice length for Plain Reinforcing Bars without Hooks

Bar diameter	Splice length (mm)	
	Tension zone	Compression zone
10 (3/8")	480	400
12 (1/2")	576	480
16 (5/8")	768	640
19 (3/4")	912	760
22 (7/8")	1056	880
25 (1")	1200	1000
28 (1-1/8")	1344	1120
32 (1-1/4")	1536	1280

Table 4. Bond or development length (mm) of Plain Reinforcing Bar without Hook

Bar Dia	$f'c = 17.3\ N/mm^2$		$f'c = 19.3\ N/mm^2$	
	* Top Bar	Other Bar	* Top Bar	Other Bar
10	353	353	353	353
12	423	423	423	423
16	665	564	627	564
19	937	670	885	669
22	1256	887	1186	837
25	1622	1145	1531	1081
28	2035	1436	1921	1356
32	2658	1876	2509	1771

* Top bar are horizontal bar with 300 mm or more concrete poured below the bar

2.3.4 Web Reinforcement

Vertical or inclined stirrups must be designed to resist the shear stress in excess of that which can be taken by the concrete alone i.e.

$$v' = (v - v_c)$$

where, v' = excess shear stress for which web reinforcement has to be provided

v = developed shear stress = V/bd

v_c = allowable shear stress

V = total shear

b = width of the section

d = effective depth of the section

Longitudinal spacing S of the stirrups can be computed as follows:

For vertical stirrups, $S = \dfrac{A_u f_u}{v' b}$

For inclined stirrups, $S = \dfrac{A_u f_u (\sin\alpha + \cos\alpha)}{v' b}$

Where, S = centre to centre spacing of the stirrups

A_u = cross-sectional area of the stirrups

f_u = allowable stress of the steel

v' = excess shear stress for which stirrups are to be provided.

b = width of the section

α = angle of inclination of the stirrups to the horizontal.

Where stirrups are required, under no circumstances should the spacing exceed d/2. Spacing should not exceed d/4 when

$$v > 3\, f_c'^{\frac{1}{2}}$$

Where web reinforcement is required the following conditions must be met:

- A_u shall not be less than 0.0015 bS for any interval
- No single type of web reinforcement shall resist a total shear force greater than 2V'/3 nor shall the total shear stress exceed $5\, f_c'^{\frac{1}{2}}$.

2.3.5 Minimum Reinforcement or Temperature Reinforcement

Except for very small structures, the following criteria should be used to determine the cross-sectional area of temperature or minimum reinforcement required in hydraulic structures. The percentages indicated are based on the gross cross-sectional area, not including fillets, of the concrete to be reinforced. Where the thickness of the section exceeds 380 mm (15 in), a thickness of 380 mm should be used in determining the temperature or minimum reinforcement.

(i) Single layer reinforcement

- slabs not exposed to direct sun with spacing of joints not exceeding 9.15 m (30 ft): 0.25%
- slabs exposed to direct sun with spacing of joints not exceeding 9.15 m (30 ft): 0.30%
- slabs exceeding 9.15 m (30 ft) between joints and not exposed to direct sun: 0.35%
- slabs exceeding 9.15 m (30 ft) between joints and exposed to direct sun: 0.40%

ii) Double layer reinforcement (in each direction)

- face adjacent to earth with spacing of joints not exceeding 9.15 m (30 ft): 0.10%
- face not adjacent to earth nor exposed to direct sun and with spacing of joints not exceeding 9.15 m (30 ft): 0.15%
- face not adjacent to earth but exposed to direct sun and with spacing of joints not exceeding 9.15 m (30 ft): 0.20%
- if member exceeds 9.15 m (30 ft) in any direction parallel to reinforcement, add to the reinforcement requirement in that direction because of the increased length: 0.05%

Table 5 may be used to select the desired minimum reinforcement or temperature reinforcement.

Table 5. Minimum reinforcement or Temperature Reinforcement (Ref: USBR)

Member Length <9.15m

Member Thickness	**Location of Reinforcement**	**Quantity of Reinf. cm^2/m**	**Dia and Spacing provided**
30 cm	Adjacent to earth	0.001x30x100 = 3.0	12mm @300 c/c
	Exposed not to direct sun	0.0015x30x100 = 4.5	12mm @250 c/c
	Exposed to direct sun	0.002x30x100 = 6.0	12mm @180 c/c or 16mm @300 c/c
35 cm	Adjacent to earth	0.001x35x100 = 3.5	12mm @300 c/c
	Exposed not to direct sun	0.0015x35x100 = 5.25	12mm @200 c/c
	Exposed to direct sun	0.002x35x100 = 7.0	12mm @150 c/c or 16 mm @280 c/c
38 cm or greater	Adjacent to earth	0.001x38x100 = 3.8	12mm @250 c/c
	Exposed not to direct sun	0.0015x38x100 = 5.7	12mm @180 c/c or 16mm @300 c/c
	Exposed to direct sun	0.002x38x100 = 7.6	16mm @250 c/c

Table 5. Minimum reinforcement or Temperature Reinforcement (Contd.)

Member Length > 9.15 m

Member Thickness	Location of Reinforcement	Quantity of Reinf. cm^2/m	Dia and Spacing provided
30 cm	Adjacent to earth	0.0015x30x100 = 4.5	12mm @250 c/c
	Exposed not to direct sun	0.002x30x100 = 6.0	12mm @180 c/c or, 16mm @300 c/c
	Exposed to direct sun	0.002x30x100 = 7.5	12mm @150 c/c or, 16mm @250 c/c
35 cm	Adjacent to earth	0.0015x35x100 = 5.25	12mm @200 c/c
	Exposed not to direct sun	0.0015x35x100 = 7.0	12mm @150 c/c or, 16mm @280 c/c
	Exposed to direct sun	0.0025x35x100 = 8.75	16mm @200 c/c
38 cm or greater	Adjacent to earth	0.0015x38x100 = 5.7	12mm @180 c/c or 16mm @300 c/c
	Exposed not to direct sun	0.002x38x100 = 7.6	16mm @250 c/c
	Exposed to direct sun	0.0025x38x100 = 9.5	16mm @200 c/c

2.3.6 Hooks and Bend Dimensions in Reinforcing Bar

The standard dimensions and bend radii for hooks standardized by ACI code (Refer Figure 1) are as follows:

- A semi-circular turn plus extension of atleast four diameters, but not less than 63.50mm at the free end of the bar.

- A 90° turn plus an extension of at least 12 bar diameters at the free end of the bar.

- For stirrups and ties only either a 90° or 135° turn plus an extension of at least 6 bar diameters, but not less than 63.50mm at the free end of the bar.

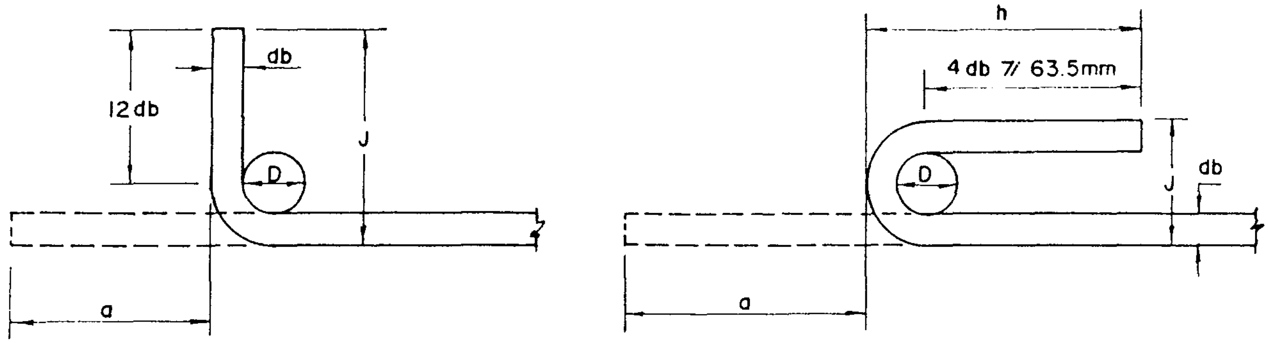
The minimum radius for bends are as follows:

Bar dia (mm), d_b	Min. Diameter, D
10, 12, 16, 20, 22, 25,	6 bar dia
28, 32	8 bar dia

The standard bend and hook dimensions for reinforcement bar and ties & stirrups according to the above mentioned criteria, have been shown in table 6 and table 7 respectively.

Table 6. Standard Hook and Bend Dimensions (mm) for Reinforcing Bar

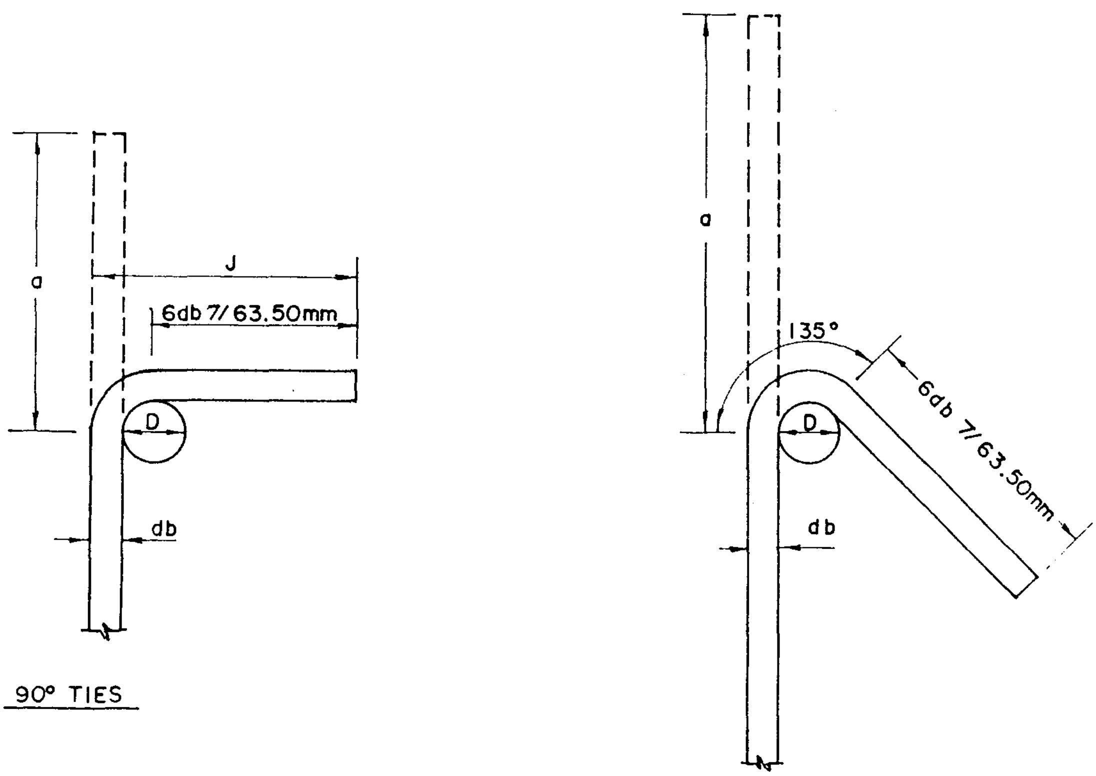
Bar size (mm)	Bend Dia (mm)	180° Hook			90° Bend	
		a	j	h	a	j
10	60	134	80	104	130	160
12	72	148	96	112	156	192
16	96	176	128	128	208	256
19	114	209	152	152	247	304
22	132	242	176	176	285	352
25	150	275	200	200	324	400
28	224	368	280	252	136	170
32	256	420	320	288	163	204



90° BEND

180° HOOK

Bend & Hook Dimensions for Reinforcing Bars



135° STIRRUPS

Bend Dimensions for Ties and Structure

Fig. - 1 Bend and Hook Dimensions for Reinforcing Bars

Table 7. Standard Hook and Bend Dimension (a) for Ties and Stirrups

Bar Size (mm)	6	10	12	16	19
90° Bend	68	73	84	111	132
135° Bend	85	106	123	164	195

2.3.7 Clear Concrete Cover to Reinforcement

- Wall and Floor Slab in Contact with Earth
  - All Structure except Minor one : 80 mm
  - Minor Structure : 50 mm
- Pier : 65 mm
- Beam/Girder/Bridge Deck slab : 40 mm
- Column : 50 mm
- Floor/Roof slab, Railing, Pre-cast pile : 25 mm
- Cast-in-situ pile : 75 mm

2.4 Wall and Slab

Wall having two layers of reinforcement normally have minimum thickness of 25 cm (10 inch). Cantilever walls shall have a minimum thickness at the base equal to 2.5 cm (1 inch) per 30 cm (12 inch) of height, 12 cm (5 inch) minimum upto 2.45 m (8 feet); above 2.45 m (8 feet) the minimum thickness at the base shall be 20 cm (8 inches) plus 2 cm (3/4" inch) for each 0.30 m (1 foot) increment in height above 2.45 m (8 feet).

In cases where the thickness of the wall at the base and that of floor vary appreciably, judgement should be applied to maintain consistency in the section of wall thickness with the floor and in use of reinforcement.

2.5 Joints in Structure

2.5.1 Construction Joints

Construction joints are the joints which are either purposely placed in structures to facilitate construction or which occur in structures as a result of inadvertent delays in concrete placing operation. Bond is required at the construction joints regardless of whether or not reinforcement is continuous across the joint.

2.5.2 Contraction Joints

Contraction joints are the joints placed in structures or slabs to permit for volumetric shrinkage of monolithic units. The joints shall be so constructed that there will be no bond between the concrete surfaces forming the joint. To take care of both shrinkage effects and differential settlements the structure should have contraction joints at intervals of 8 m to 12 m. The upper face of the joints should be chamfered and a gap of 1.5 cm should be provided. Also the joints are to be provided with water stops of approved material and normally flexible PVC water stops.

2.5.3 Expansion Joint

Expansion joints may also be provided at intervals of 18 m to 30 m. The joint may be formed using a 2 cm thick compressible filler with a water stop centrally located. The exposed edges of the joint should be sealed with an approved sealant.

It may be noted that the structure must slide on the soil underlying the base and simultaneously shear the soil underneath and the soil behind the earth retaining wall, if any, in order for any expansion (or contraction) to take place. These resisting forces may be large enough to cancel the expansion forces and no expansion joint at all may actually be required.

2.6 Fillets

Fillets are used to provide increased strength or to relieve stress concentration at points of maximum stress. **Table 8** gives the dimensions of fillets which are to be provided at the inside corners of box sections and at bases of vertical cantilever walls.

Table 8. Dimensions of Fillet

Size of fillet	Size of box section	Clear vertical height of cantilever
50 mm x 50 mm (2 in x 2 in)	0 to 1.25 m (0 to 4 ft)	-
75 mm x 75 mm (3 in x 3 in)	1.25 m to 1.85 m (4 ft to 6 ft)	0 to 2.45 m (0 to 8 ft)
100 mm x 100 mm (4 in x 4 in)	1.80 m to 2.45 m (6 ft to 8 ft)	2.45 m to 3.70 m (8 ft to 12 ft)
150 mm x 150 mm (6 in x 6 in)	Over 2.45 m (8 ft)	Over 3.7 m (12 ft)
No reinforcement is needed for fillets upto 150 mmx150 mm in size.		

3.0 STABILITY DESIGN CRITERIA

3.1 Forces considered in the analysis of stability

The following forces may be considered as affecting the stability:

a) Dead Load
b) Imposed Load
c) Water Load
d) Water Pressure
e) Uplift Pressure
f) Earth Pressure
g) Wind Pressure
h) Foundation Pressure

The forces to be resisted by a structure fall into the following two categories:

a) Forces, such as weight of the structure and water pressure which can directly be calculated from the unit weights of the materials and properties of the fluid pressures: and

b) Forces, such as uplift, earthquake loads, silt pressure for which realistic assumptions must be made.

3.2 Load Combinations

Design should be based on the most adverse combination of probable load conditions, but should include only those loads having a reasonable probability of simultaneous occurrence. Depending on the various project components, site conditions and construction programme, the most adverse combination should be considered for the design of a hydraulic structure. This shall include loads to which the structural component is subjected during construction and after completion.

a) Load Combination 'A' (Construction Condition)

Structure completed but no water in the upstream and no tail water in the downstream.

b) Load Combination 'B' (Normal operating condition)

Full upstream water level, normal dry weather tail water (downstream), normal uplift and silt pressure (if applicable)

c) Load Combination 'C' (Flood Discharge Condition)

Upstream elevation at maximum flood pool elevation, all gates open, tail water at flood elevation (normally in post-monsoon), normal uplift, silt pressure (if applicable).

d) **Load Combination 'D'**

Reverse of load condition 'B' and river side elevation at maximum (design) flood level, normal uplift, silt pressure (if applicable).

3.3 Factor of Safety

The factor of safety against the above stability requirements shall be taken as follows:

Overturning	:	Normal case	: 1.50
		Extreme case	: 1.30
Sliding	:	Normal case	: 1.50
		Extreme case	: 1.30
Uplift	:	1.10	
Bearing	:	Sandy Soil	: 2
		Clayey Soil	: 3

4.0 COMPUTATION OF LOADS AND PRESSURES

Accurate computation of loads and pressure for analysis and design of any structure is of prime importance. The loads and pressures given in the following subsections may be used.

4.1 Dead Loads (Unit Weights)

Structural concrete	:	23.6 kN/m^3 (150 lb/ft^3)
Plain concrete	:	22.6 kN/m^3 (144 lb/ft^3)
Sea water	:	10.1 kN/m^3 (64 lb/ft^3)
Fresh water	:	9.8 kN/m^3 (62.4 lb/ft^3)
Brick Masonry	:	18.9 kN/m^3 (120 lb/ft^3)
Timber (hard)	:	7.1-12.6 kN/m^3 (45-80 lb/ft^3)
Timber (soft)	:	4.7-7.1 kN/m^3 (30-45 lb/ft^3)

Stone	:	25.5 kN/m^3(162 lb/ft^3)
Bitumen	:	13.7 kN/m^3(87 lb/ft^3)
Steel	:	77.0 kN/m^3(490 lb/ft^3)
Cement	:	14.1 kN/m^3(90 lb/ft^3)

For preliminary design only, the unit weights of soil given below may be used. Final designs are to be based upon actual soil data which have been obtained from site investigations.

Dry soil	:	15.7 kN/m^3 (100 lb/ft^3)
Moist soil	:	17.3 kN/m^3 (110 lb/ft^3)
Saturated soil	:	18.9 kN/m^3 (120 lb/ft^3)

4.2 Imposed loads

The imposed load (live load) for human occupancy shall be taken as follows:

Operating deck : 7.2 kN/m^2 (150 lb/sft, with stop log operation)

Regulator deck : 7.2 kN/m^2 (150 lb/sft, for pedestrian movement)

4.3 Truck Load

i) The truck loading shall be selected on the basis of location of structure in relation to the type of road or embankment it crosses. The following type of truck loading may be selected.

Village road	: H10
Feeder road and District Council road	: H15
Highway	: H20 S16

For Truck loading it should be noted that 1 ton = 2000 lb = 8.9 Kn

ii) The distribution of truck loading on concrete slab and filled earth shall be as per AASHTO design standard code of practice. Where the depth of earth fill is less than 200 mm, the wheel load is treated as a concentrated load in direct contact with concrete slab.

iii) The distribution of loads for wheels resting directly on slab can be computed as follows:

$E = (1.219 + 0.06S)$ with maximum of 2.134 m.

Where

E = distribution width (m) & S = design span of slab (m)

iv) The distribution of wheel loads transmitted through fills are as follows:

a) When the depth of fill is 0.60 m or more, concentrated loads shall be considered as uniformly distributed over a square, the sides of which are equal to 1 3/4 times the depth of fill. When such areas from several concentrations overlap, the total load shall be considered as uniformly distributed over the area defined by the outside limits of the individual areas.

b) For single span the effect of live load may be neglected when the depth of fill is more than 2.44 m and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between end supports or abutments.

c) In no case shall the stresses for load as determined by the above be greater than those obtained where the wheel is applied directly on the structure.

d) In no case shall the total width of distribution exceed the total width of supporting slab.

v) The percentage impact of vehicle on concrete slab should not exceed 30% and is given by the equation.

$$I = 15.2/(L+38)$$

Where, L = span of the bridge deck (m)

vi) The percentages impact of vehicle on earth fill of various height are as follows:

For fills	upto 0.30 m,	impact	= 30%
" "	0.30 m to 0.60 m,	impact	= 20%
" "	0.60 m to 0.90 m,	impact	= 10%
" "	0.90 m and over,	impact	= 0%

vii) The depth of earth surcharge equivalent to truck loading shall be a maximum of 0.90 m.

4.4 Earth Pressure

4.4.1 Earth Pressure Against Vertical Wall

The active earth pressure against vertical wall with different types of dry or moist backfill material can be computed as follows:

a. Cohesionless Soil (ϕ-Soil) (Refer to **Fig. 2a**)

Pressure at any depth, $p = ca\ \gamma\ h$, $Ca = \dfrac{1-\text{Sin}\phi}{1+\text{Sin}\phi}$

Total Pressure, $P = \frac{1}{2}\ ca\ \gamma\ h^2$

b. Cohesive Soil (C - Soil) (Refer to **Fig.** 2b)

Pressure at any depth, $p = \gamma h - 2C$

Depth of potential tension crack, $Z = 2C/\gamma$

Design total pressure, $P = 1/2\ ph$

c. Mixed Soil ($C - \phi$) Soil: (Refer to **Fig.** 2b)

Pressure at any depth, $p = ca\ \gamma\ h - 2C - (Ca)^{\frac{1}{2}}$

Depth of potential tension Crack, $Z = \dfrac{2C\ Ca}{\gamma}$

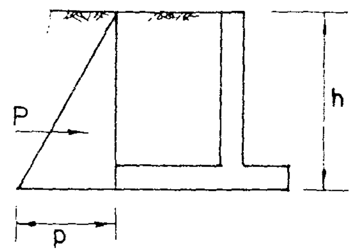


Fig. a

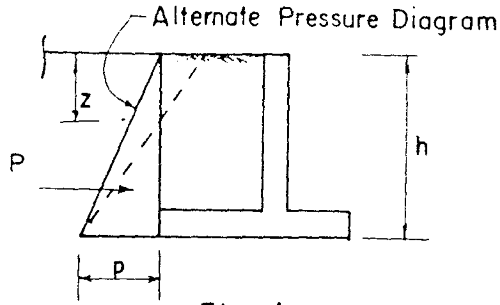


Fig. b

Earth pressure with moist backfill of horizontal surface

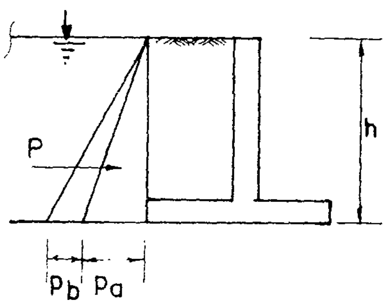


Fig. c

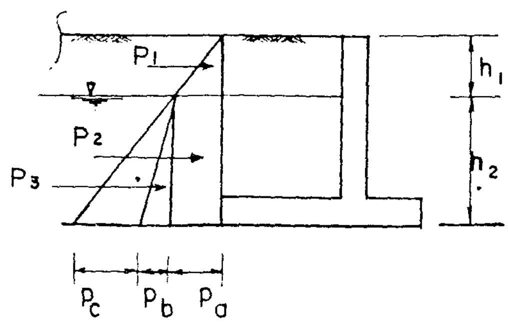


Fig. d

Earth pressure with saturated backfill of horizontal surface

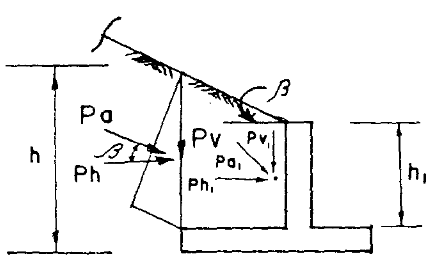


Fig. e

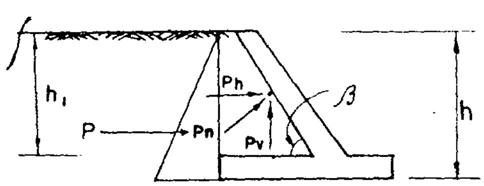


Fig. f

Earth pressure on vertical wall with inclined backfill
& inclined wall with horizontal backfill (moist soil)

Fig. 2 Earth pressure on retaining walls

Design total pressure, $P = \frac{1}{2}\ ph$

where, Ca = Rankin's active earth pressure co-efficient
ϕ = Angle of internal friction of soil
C = Cohesion
γ = Unit weight of backfill soil.

In the upper region of cohesive and mixed backfill soil, a tension crack will form when the soil dries out. Formation of tension zone will reduce the lateral earth pressure but it should not be relied upon. Instead, one should assume that it will form and will possibly be filled with water. The depth of water however, will increase the lateral pressure considerably due to hydrostatic pressure in excess of earth pressure. It is thus recommended to use alternate pressure diagram conservatively as shown in **Fig.** 2b and compute wall pressure.

The active earth pressure against vertical wall with fully or partly saturated backfill soil of level surface can be computed as follows:

a. <u>Fully Saturated Soil:</u> (Refer to **Fig.** 2c)

Pressure at any depth,

$p = pa + pb = Ca\ \gamma_{sub}\ h + \gamma_w h$

or $p = (Ca\ \gamma_{sub} + \gamma_w)\ h$

Total pressure, $P = \frac{1}{2}\ (Ca\ \gamma_{sub} + \gamma_w)\ h^2$

b. <u>Partly Saturated Soil</u> (Refer to **Fig.** 2d)

Pressure at any depth below saturation level,

$p = pa + pb + pc$

Where, $p_a = C_a \gamma_{moist} h_1$

$\& \ p_b + p_c = C_a \gamma_{sub} h_2 + \gamma_w h_2$

$= (C_a \gamma_{sub} + \gamma_w) h_2$

Total Pressure, $P = P_1 + P_2 + P_3$

Where, $P_1 = \frac{1}{2} C_a \gamma_{moist} h_1^2$

$P_2 = C_a \gamma_{moist} h_1 h_2$

$P_3 = \frac{1}{2} (C_a \gamma_{sub} + \gamma_w) h_2^2$

The active earth pressure against vertical wall with dry or moist backfill soil having inclined surface can be computed as follows (Refer **Fig.** 2e):

$P_a = \frac{1}{2} C_a \gamma h^2$, or $P_{a1} = \frac{1}{2} C_a \gamma h_1^2$, where,

$$C_a = \cos\beta \, \frac{\cos\beta - \sqrt{(\cos^2\beta - \cos^2\phi)}}{\cos\beta + \sqrt{(\cos^2\beta - \cos^2\phi)}}$$

$P_h = P_a \cos\beta$, $P_v = P_a \sin\beta$ or $P_{h1} = P_{a1} \cos\beta$, $P_{v1} = P_{a1} \sin\beta$

Where, β = angle of inclination of backfill with horizontal

4.4.2 Earth Pressure Against Inclined Wall (Fig. 2f)

The earth pressure against inclined wall with dry or moist backfill soil of level surface can be computed as follows:

$P = \frac{1}{2} C_a \gamma h^2$ (same as fig. 2a)

$P_n = \frac{1}{2} C_a \gamma h_1^2$, P_n = Pressure normal to the back of wall

$P_h = P_n \cos(90^\circ - \beta)$, P_h = Horizontal component of pressure

$P_v = P_n \sin(90^\circ - \beta)$, P_v = Vertical component of pressure

$$Ca = \left[\frac{\sin(\beta - \phi)}{(n + 1)\sin\beta} \right]^2 \times \frac{1}{\sin\beta}$$

Where,

$$n = \frac{\sin\phi}{\sin\beta}$$

β = Angle between wall & horizontal

ϕ = Angle of internal friction of soil

4.5 Pressure Due to Surcharge

a) Earth pressure due to uniform surcharge p = CaW
Where, Ca = Rankin's active earth pressure Co-efficient which depends on soil characteristics.

W = Weight of surcharge material in unit area.

b) When the ground surface is subjected to a uniformly distributed surcharge load, the earth pressure computation is often made by substituting the load by an equivalent surcharge layer. The thickness of this surcharge layer is equal to the distributed load divided by the unit weight of backfill soil. In that case, pressure due to surcharge.

$p = Ca\, \gamma_{soil}\, h'$, where $h' = W/\gamma_{soil}$

c) Equivalent surcharge depth for truck loading shall be taken a maximum depth of earth = 0.90 m.

4.6 Water Pressure

Water pressure may act laterally against walls and vertically against base slabs of any structure.

4.6.1 Lateral Water Pressure

The lateral water pressure at any depth, $p = \gamma_w h$

Total lateral water pressure = $\frac{1}{2} \gamma_w h^2$

Where, γ_w = unit weight of water

h = depth of water

4.6.2 Vertical Water Pressure (Uplift Pressure)

The uplift pressure under the base slab of structure can be computed by Khosla's or Lane's theory. In such case, the total uplift pressure (U) is the difference in elevations (h) between hydraulic grade line and bottom level of base slab multiplied by unit weight of water (γ_w).

Thus total uplift pressure, $U = \gamma_w h$.

4.7 Wind Pressure

Wind pressure may be determined by the following formula:

$$P = \frac{wv^2}{2g} \times \text{gust factor} \times \text{shape factor} \times \text{height factor}$$

Where, P = pressure per unit area

w = weight of air per unit volume

v = highest wind velocity (miles per hour)

g = acceleration due to gravity

If V is in miles per hour, the foregoing equation is reduced to:
$P = .00256\ v^2$ x gust factor x shape factor x height factor.

The values of different factors may be obtained from the USBR publication "Design of Low Head Hydraulic Structures."

Instead of generous estimation of wind pressure, the wind pressure of 2.39 kn/m² (50 lb/sft) may be used.

4.8 Intensity of Pressure on Foundation

For application of different loads and pressures under different situations, the maximum (P_1) and minimum (P_2) intensity of pressure on foundation base shall be computed as follows:

When the resultant is at the middle third

$$P = \frac{\Sigma V}{L}\left[1 \pm \frac{6e}{L}\right], \qquad \begin{array}{l} + \text{ Sign gives } p = p_1 \\ - \text{ Sign gives } p = p_2 \end{array}$$

When the resultant is at the edge of middle third

$$P_1 = \frac{2\Sigma V}{L}, \qquad P_2 = 0$$

When the resultant is outside middle third

$$P_1 = \frac{2\Sigma V}{3a}, \qquad a \leq L/3$$

Where, V = Vertical load

L = Base length

e = Eccentricity

STRUCTURAL DESIGN
OF REGULATOR

B. DESIGN CONSIDERATIONS

PART - B : DESIGN CONSIDERATIONS

Table of contents **Page**

1.0 STRUCTURAL PARTS OF A REGULATOR

Regulator or Sluice structures normally built in this country consist of either a pipe of maximum diameter of 0.90 meter or a Box Conduit of 1.83 m x 1.52 m size with inlet and outlet transitions. Pipe sluice is selected when a minor amount of discharge is to be drained from a small catchment area. The Box sluices are usually single box or multi box structures and are constructed for drainage of large areas. They are usually provided with two central joints separating the transition part with main conduit part of the structure. In large structure, additional joints are provided along the longitudinal direction of wingwall to separate the wingwall and apron slab.

There are three major parts of a regulator or a sluice with earth fill construction over box conduit. The various structural elements for each of these major parts are as follows:-

a) **Central part**

- Railing
- Head wall
- Operating Deck
- Conduit
- Abutment as extended part of conduit
- Pier as extended part of conduit
- Base slab as extended part of conduit

b) **Transition/Stilling basin part**

- Wingwall
- Floor or Apron Slab

c) **Return wall part**

- Return wall

The above mentioned parts may be seen in a typical profile of a regulator in page 7-57.

2.0 DESIGN CONSIDERATIONS OF VARIOUS PARTS

2.1 Central part

2.1.1 Railing

Two lines of railing, one at the edge of roadway or embankment and other at the edge of the operating deck are required for protection of the pedestrians and the gate operators respectively. The materials for railing at the edge of roadway shall be concrete. The materials for railing on operating deck may be either concrete or G.I Pipe. In designing railing the following considerations shall be taken into accounts:

a) Railing at the edge of road shall not be less than 0.75 m in height measured from top of roadway. Railing on operating deck shall not be less than 0.90m in height.

b) Rail members i.e. rails & posts shall be designed for a moment due to specified horizontal and vertical loads. The railing may be designed for a moment of $0.1\ wL^2$ at the centre of the panel and at the posts where w is the uniform load on rail and L is the post spacing.

c) In general, the design section and reinforcement of railing can be standardized. As for example, railing and post size of 15 cm x 15 cm are sufficient. A reinforcement equivalent to 4 numbers 10 mm ϕ bars for rail and 4 numbers 12 mm ϕ bars for post with 6 mm ϕ bars @ 15 cm c/c as ties are sufficient if the post spacing does not exceed 1.50 m on centres. In case of G.I Pipe railing, the post and railing members may be made of pipe of 38 mm diameter.

2.1.2 Head Wall

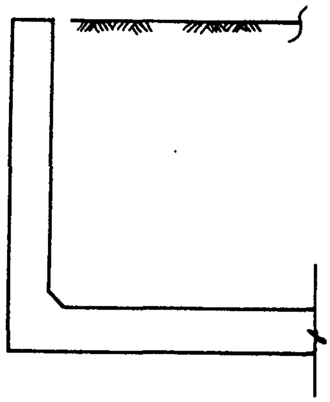
The headwall retains the earth of roadway or embankment and is supported by the piers and abutments. As in usual construction, the headwall is fixed at piers and abutments at the sides and at top slab of the conduit
at bottom. Accordingly, each panel of headwall in between piers or in between pier and abutment is a plate fixed on three sides and can be designed on the basis of theory of plates. However, as in most of the cases, the desired section and reinforcement for headwall conform to the minimum standard, no accurate analysis is necessary. Therefore, rather easy and conservative design procedures, that are outlined below are usually followed.

a) For computing earth pressure on headwall, the soil parameters of road or embankment shall be considered. Preferably, an angle of internal friction of soil of 20° may be adopted.

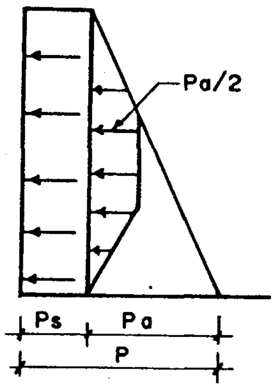
b) A maximum of 0.90 m of earth may be considered over the road/ embankment for computing earth pressure due to live load surcharge.

c) Design earth pressure for horizontal bending moment of headwall slab shall be considered equal to the sum of surcharge pressure & half of earth pressure at the base as shown in sketch below.

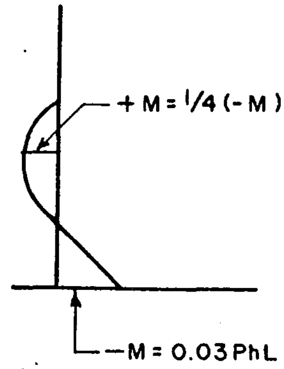
d) Maximum positive and negative bending moments for horizontal bending = $wL^2/10$, where w is the uniform load and L is the clear spacing of pier/abutment. Maximum vertical bending moment shall be calculated as shown in the following sketch.



Head Wall



Load Diagram



Vertical Bending Moment

2.1.3 Operating Deck

The operating deck is the platform on which the gate hoisting equipments rest and from which gates are operated. The deck member may be either a continuous slab supported over the piers and abutments or a simply supported slab over them. For regulators of standard vent width of 1.52 m, the operating deck may be constructed as continuous slab. The following points should be considered for designing the same.

a) A design concentrated load consisting of gates & hoist including operating load shall be considered at the centre of each panel of deck slab. In case of lift gate, this load may be considered equally shared by the headwall and deck slab.

b) Maximum shear & moment for the uniformly distributed and concentrated dead load & live load may be computed by using plate 1.

2.1.4 Conduit

The conduit is a RCC box over which road/embankment rests and through which irrigation and drainage water is conveyed. It is designed as a frame structure. The following considerations may be adopted for its design.

a) The conduit section at the centre of the road/embankment with all loads above shall be considered for design.

b) The full height of earth fill above the top of conduit at the selected section shall be considered.

c) Live load shall not be considered if the fill height is over 2.44 m. For fill height less than 2.44 m, design procedure explained in Part-A shall be followed.

d) The earth pressure on side slab shall be computed by taking account of the coefficient of earth pressure at rest corresponding to the angle of internal friction of road/embankment soil. Preferably an angle of internal friction of 20° of soil shall be considered.

e) An earth surcharge depth of 0.90 m shall be considered for computing earth pressure on side wall due to live load.

f) All critical loading situations should be examined to compute the maximum shear and moment for each member of the conduit. The loading conditions as in Plate 2 may be considered.

g) For all imposed loads, a uniform base pressure may be considered.

h) The conduit frame may be analyzed by the method of moment distribution.

2.1.5 Extended part of Conduit

The extended part of conduit is composed of sidewall, pier and base slab. The design considerations for each of these members are outlined below:

i) **Sidewall**

The sidewall may be treated as a plate fixed at two ends i.e. at bottom slab and at headwall/conduit sidewall. Accordingly, Plate 3 showing coefficients for shear & moment can be used for design. This procedure is recommended where the proportion of wall height and length is not less than at least 2.0. In case of sidewall having a height and length proportion less than 2.00, the sidewall shall be designed as cantilever wall fixed at bottom slab only. As in such case horizontal bending is appreci- able it is considered that if it is designed as plate fixed at two ends, the design might not be safe as there is doubt about adequate fixity of wall with head walls for resisting horizontal bending. Therefore, in order to be on the safe side, the wall may be designed considering the vertical bending only.

Different loading situations should be examined to compute the maximum shear & moment on the sidewall.

The soil parameters of the embankment or road may be considered for design. Preferably, an angle of internal friction of 20° of soil shall be considered.

As the sidewall supports the headwall which in turn is acted upon by earth pressure, the sidewall shall also be designed as cantilever beam considering fixity at top of conduit. In such case, full earth pressure including surcharge pressure shall be considered between the end of wall and centre of clear width between pier and abutment.

The sidewall may also be designed as column as it takes the load of operating deck and the gate hoisting equipment etc. However, design as column is not critical considering its section and reinforcement obtained from other design considerations.

ii) Pier

The pier supports the deck, gate hoists & operating load etc. So it may be designed as column. However, considering its section from other requirements, the design as column may not always be critical specially for a regulator of standard size.

It shall be designed as cantilever beam fixed at the top slab of the conduit with earth pressure on head wall between centres of headwall bays as considered in the case of sidewall.

During maintenance of any conduit or gate by closing the ends of piers by stop log, the pier may be acted by differential water head when the maintenance chamber is made dry and there is flowing water outside the chamber or there is difference of water levels. In such case, the wall is designed as plate fixed at two ends or as purely cantilever wall with vertical bending only for considering the limiting conditions mentioned in case of side wall above.

iii) Base Slab

The shear and moment of base slab in between piers and pier and abutment and at their supports shall be computed for all vertical loads and end moments.

Different loading situations should be examined to design the base slab for maximum shear & moment, for both top and bottom tension of slab.

2.2 Stilling Basin/Transition Part

The transition part of a regulator may be one of the various forms as shown in Plate 4 and may be divided into three groups as below. The choice of selecting any type depends on various factors such as wall height, wall to wall distance, type of material to be used in wingwall construction, allowable bearing capacity of soil, complexity in design computation and also economy in construction.

Monolithic wingwall and apron

For the monolithic type of construction, the wall and apron are made of R.C.C. and may be a frame of U shape with or without projected base at ends.

Separate Wingwall & Apron

For the separate type of construction, the apron is separated from wingwall by expansion joint. The apron in this case is either a purely mass concrete or mass concrete with minor reinforcement which simply rests on the ground creating no shear and moment when the apron is completely horizontal. In this case, the thickness of apron is computed by balancing the weight with uplift pressure. Where part of the apron is a sloping glacis, the glacis part is subjected to horizontal water pressure and uplift, which can cause shear and thrust on the slab. The R.C.C. wingwall in such case may be a cantilever or a counterfort or a L type retaining wall.

Combined wing wall & apron

For the combined type, the wall is a masonry structure which either completely or partly rest on both ends of R.C.C. apron.

The design consideration of wingwall and apron has been described below:

The wingwall of a stilling basin or/and of a transition of regulator is usually a vertical wall of R.C.C. or masonry construction. The R.C.C. wall may be either a usual cantilever or a counterfort retaining wall. In case of low height, it may a L shaped wall. In masonry construction, the wing is a gravity wall. In all such cases, the wingwall is constructed with apron by a construction or contraction joint. In monolithic construction of wingwall and apron, the wall is simply a cantilever member fixed with the apron at its end.

General design considerations for the stability of a wingwall are outlined below.

- The wingwall shall be stable with respect to overturning, sliding and uplift under all loading conditions and shall have specified factor of safety.

- The imposed foundation pressure shall not exceed the allowable bearing capacity of soil and should have specified factor of safety.

- The wall members shall be so proportioned that the pressures at toe and heel become nearly equal i.e. pressure shall be nearly uniformly distributed. This is specially desired in case of high walls and with soils of low bearing capacity.

Structural design considerations for various forms of wingwalls are outlined below. Various loading conditions should be examined to design different members for the largest shear & moment.

a) Usual or L-Shaped Cantilever Retaining Wall

Stem : It shall be designed as cantilever slab with full maximum load imposed on the back of wall. The reinforcement shall be placed at earth face with gradual curtailments.

Toe slab : It shall be designed as cantilever slab with net foundation pressure acting upward. The main reinforcement shall be placed at bottom.

Heel slab: It shall be designed as cantilever slab with net foundation pressure acting downwards and main reinforcement at top.

b) Counterfort retaining wall

General

For walls above 6.00 m in height, the cantilever type of retaining wall requires a greater stem thickness. In such high walls, the cost may be reduced by placing counterforts at suitable intervals on the back side of the wall. The stem behaves as a vertical slab, extending continuously over the counterforts and is loaded by the horizontal thrust. The function of the counterforts is to tie the vertical wall slab to the base of the wall.

Design of Stem

The stem is designed as a continuous vertical slab, spanning over the counterforts. For the end spans the negative moment is taken as $WL^2/10$ and for the inner spans the same is taken as $WL^2/12$ where, L is spacing of counterforts.

The maximum bending moment in the stem wall occurs at its junction with the base slab and decreases proportionately towards top of the stem. The thickness of the stem, however, is often kept uniform throughout the height of the wall, as calculated from the maximum bending moment and reinforcement is curtailed towards top of the stem.

Design of Base slab

The heel is designed as a horizontal slab, supporting the counterforts. It is subjected to bending stresses induced by the backfill and the subsoil reactions to the loading. The toe is designed exactly in the same manner as used for cantilever retaining walls.

Design of counterforts

The earth pressure acting on the stem is transferred to the counterforts. The total load, to which each counterfort is subjected, is the total earth pressure per foot at any height of stem multiplied by the spacing of the counterforts. The back of the counterfort is in tension and is designed as a cantilever beam of non-uniform depth, fixed at the top of the base slab. The tensile reinforcement is provided along the sloping back and must be sufficiently anchored at both the ends.

Since the stem has a tendency to pull away from the counterfort, horizontal ties must be provided to tie the stem to the counterforts. Similarly, as the counterforts have also a tendency of pulling away from the base slab, the two must be tied by means of vertical ties.

In the design procedures mentioned above the effect of fixity of wall bottom has been neglected. In this regard, Huntington proposed the following modification to the design load (Plate 5).

The face slab is designed as a continuous vertical slab to resist the lateral pressure as shown in Plate 5. The slab should be designed to resist the negative and positive bending moments shown in the same plate. The heel slab is subjected to an equivalent downward force in addition to the forces. This equivalent forces is evaluated to include the effect of bending moment of the toe slab and assumed to be distributed parabolically with a maximum value Wa at top of the heel.

The total parabolic load is equal to 2/3(W.h.a) and the centroid is located at a distance of 5a/8 from the back of the face slab. This equivalent uniform load should be applied to the entire length of the heel slab, from counterfort to counterfort, in combination with other forces, for computing the bending moment and shear forces along the length of the wall.

ii) Apron/Floor

The design consideration of apron/floor of a stilling basin/transition having a monolithic construction with wingwall is outlined below:

a) <u>Loads and Pressures</u>

Fig. 3 shows a load diagram depicting all the loads acting on the U-frame of a monolithic wingwall and apron. The loads and pressures may be computed as below:

1) Dead load of Wingwall (WW):

$$WW = 0.5\ (h - t_5)\ (t_1 + t_2)\ \gamma_c$$

2) Dead load of Soil behind Wingwall (S):

$$S = 0.5\ h_1\ (t_3 + t_4)\ \gamma_m + 0.5\ (h_2 - t_5)\ t_4\ \gamma_s$$

Where $t_3 = t_2 - t_1$

$$\text{and } t_4 = \frac{t_3\ (h_2 - t_5)}{(h - t_5)}$$

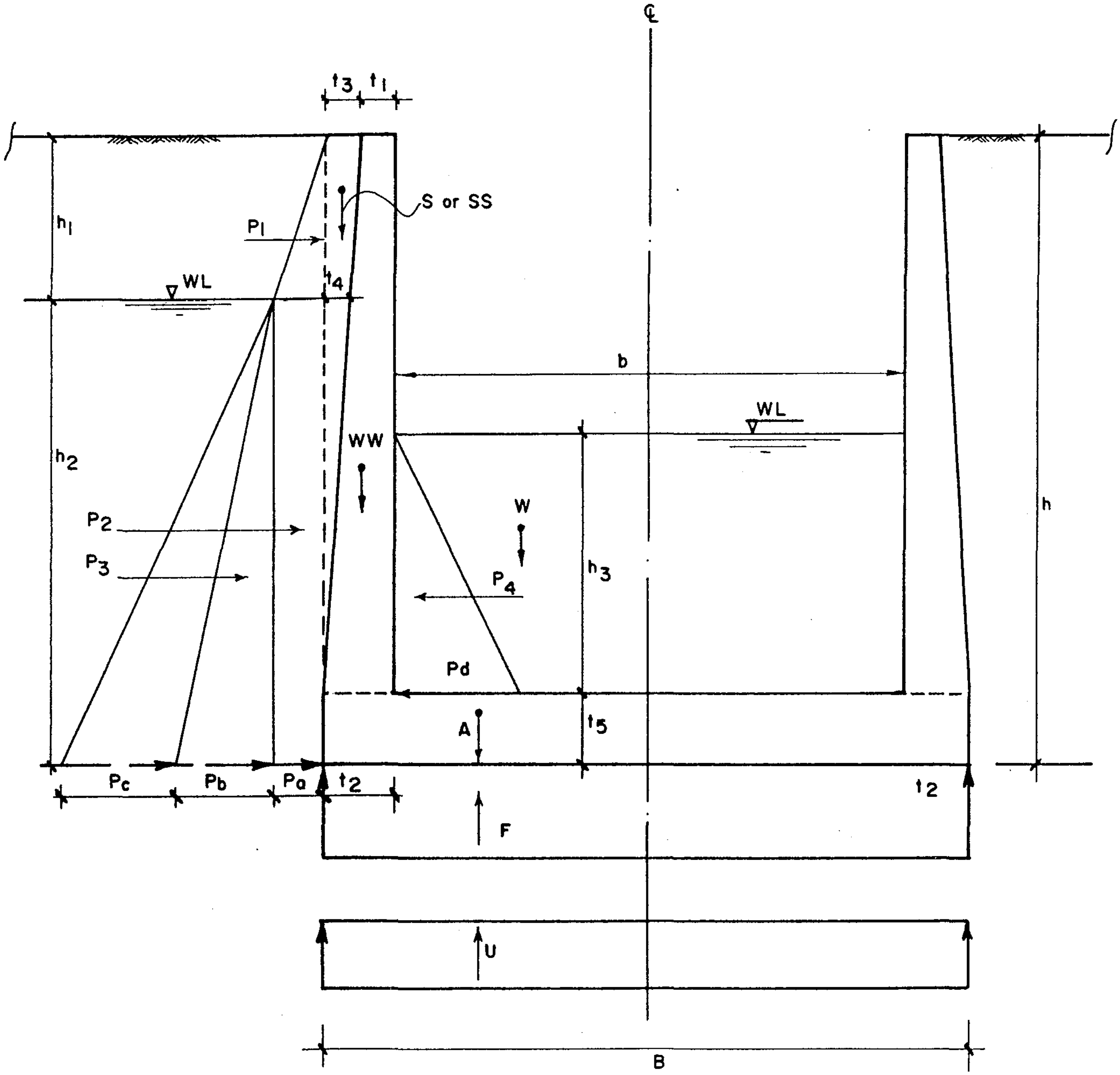


Fig. 3 Loads and Pressures

3) Dead load of Apron (A):

$$A = 0.5\ Bt_5\ \gamma_c$$

4) Weight of Water (W):

$$W = 0.5\ bh_3\ \gamma_w$$

5) Uplift (U):

$$U = 0.5\ Bh_2\ \gamma_w$$

6) Foundation Pressure (F):

$$F = WW + A + S + W - U$$

7) Earth Pressure ($P_1 + P_2 + P_3$)

$$P_1 = 0.5\ P_a h_1$$
$$P_2 = P_a h_2$$
$$P_3 = 0.5\ (P_b + P_c)\ h_2$$

Where $P_a = C_a \gamma_m\ h_1$ and $P_b + P_c = (C_a \gamma_{sub} + \gamma_w)\ h_2$

Total Earth Pressure = $P_1 + P_2 + P_3$

8) Water Pressure (P_4):

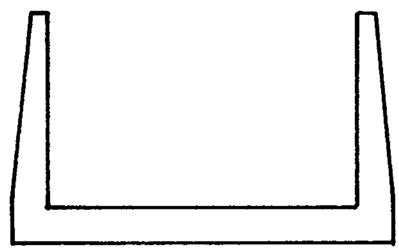
$P_4 = 0.5\ P_d\ h_3$ where, $P_d = \gamma_w h_3$

b) **The loading Conditions**

The U-frame may be subjected to various combinations of above mentioned loads during four different situations as below:

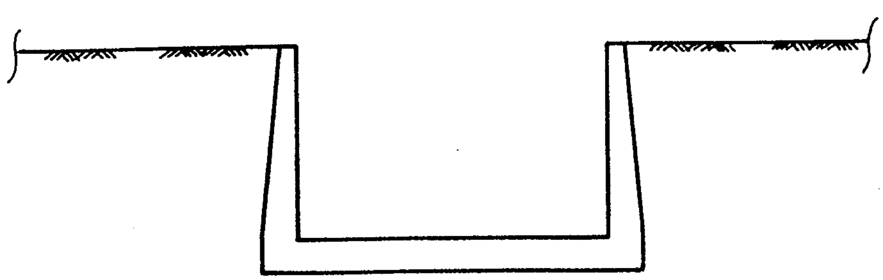
1) **Case A (During Construction)**

The U-frame has been constructed but no or part backfilling is done. In this situation, the loads WW and S are only acting. If there is partial backfill, load P_1 may also act.



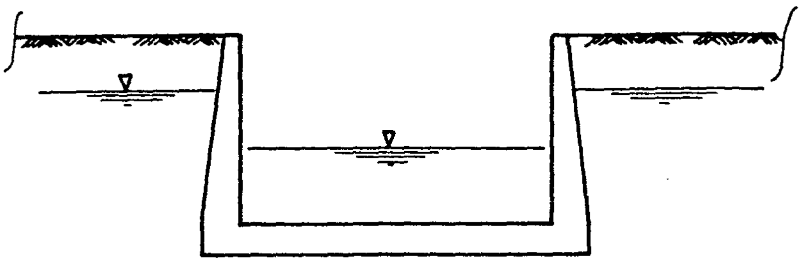
2) **Case B (After Construction)**

The U-frame and backfilling has just been completed. The loads that act under this situation are WW, S and P_1.



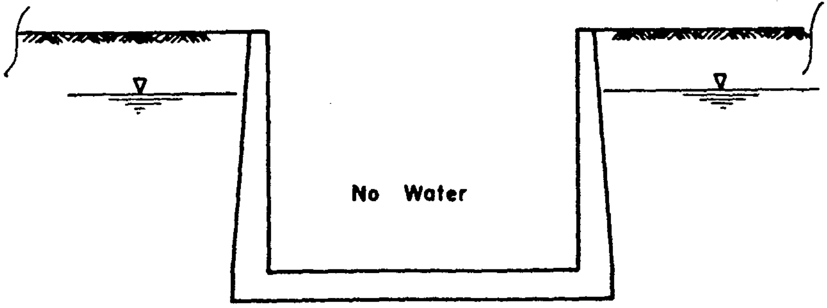
3) **Case C (During Operation)**

During operation of the structure with the under gate closed or opened, the structure is subjected to all the loads as alluded earlier. However, the critical design situation occurs when the structure is under maximum differential static head. Under static head at u/s of the structure, d/s wingwall shall be designed and vice versa. In this condition, the backfill is assumed to be saturated upto hydraulic grade line as computed during hydraulic design. The uplift at any location under the apron shall be the intercept between the hydraulic grade line and bottom elevation of apron. The U-frame trough may be partly or fully filled with water.



4) **Case D (During Maintenance)**

During maintenance, there might be situations when the transition/ stilling basin may require dewatering. Under this situation, the U-frame has no water in it and all the loads except P_4 and W act.



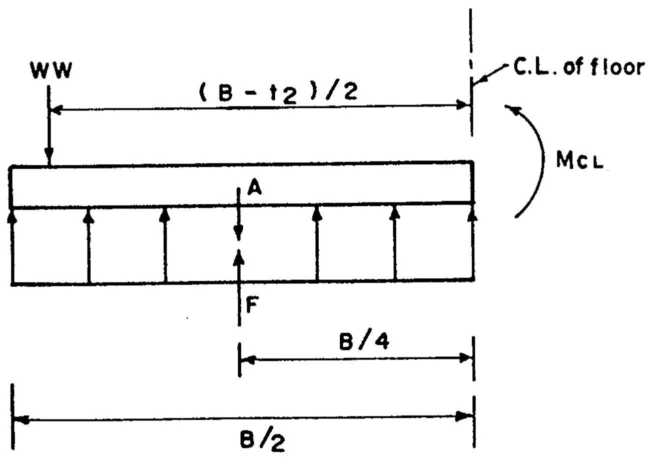
c) **Computation of loads & moments**

The maximum moment at the end of the floor which creates bottom tension occurs during operation situation when the Wingwall is also subjected to greatest moment.

The maximum span moment for both top tension and bottom tension needs to be investigated for each of the loading situations discussed earlier.

The loads, end moment, span moment, and axial force acting on the floor for each loading situation can be computed as follows (Refer to figure 3 and fig under various in cases considering half section).

1) **Case A (During Construction):**



$F \quad = WW + A$

$P \quad = 0$

$M_{end} \quad = 0$

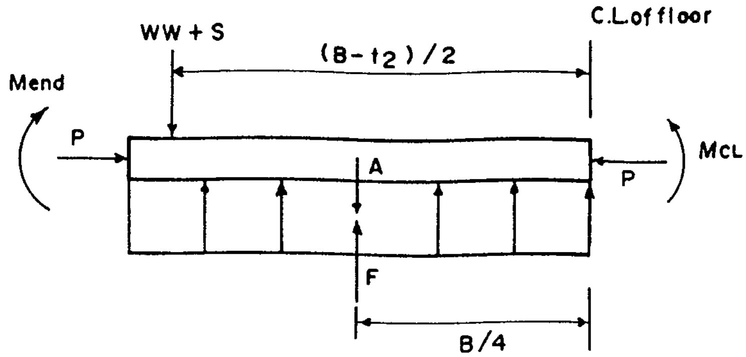
$M_{CL} \quad = WW \times [(B - t_2)/2] + A(B/4) - F(B/4)$

$\quad = WW\ [(B - t_2)/2] + A(B/4) - (WW + A)\ (B/4)$

$\quad = WW\ [(B - t_2)/2] - WW\ (B/4)$

$\quad = 1/2\ WW\ (B/2 - t_2)$

2) **Case B (After Construction):**



$F \quad = WW + S + A$

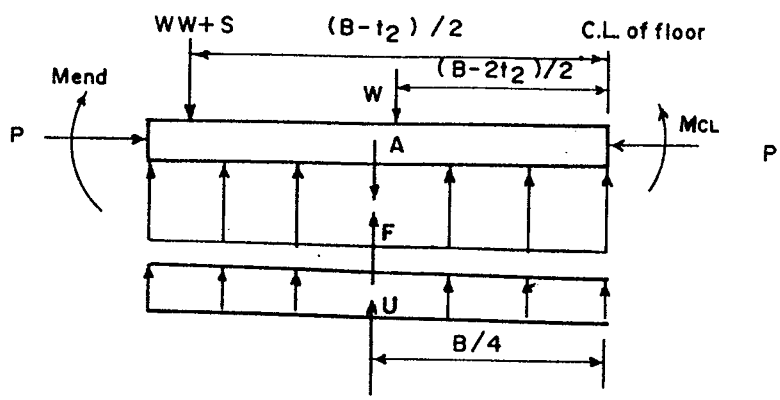
$P \quad = 0.5\ Ca\,\gamma_m\ h^2$

$M_{end} \quad = 1/3\ Ph$

$M_{CL} \quad = (WW + S) \times [(B - t_2)/2] - (F - A) \times B/4 - M_{end}$

$\quad = 0.5\ (WW + S)\ (B/2 - t_2) - M_{end}$

3) **Case C (During Operation)**



$F \quad = WW + S + A + W - U$

$P \quad = P_1 + P_2 + P_3 - P_4$

$M_{end} = P_1\ [(h_2 - t_5) + 1/3\ h_1)] + \frac{1}{2}\ P_2\ (h_2 - t_5)$

$\quad + 1/3\ P_3\ (h_2 - t_5) - 1/3\ P_4\ h_3$

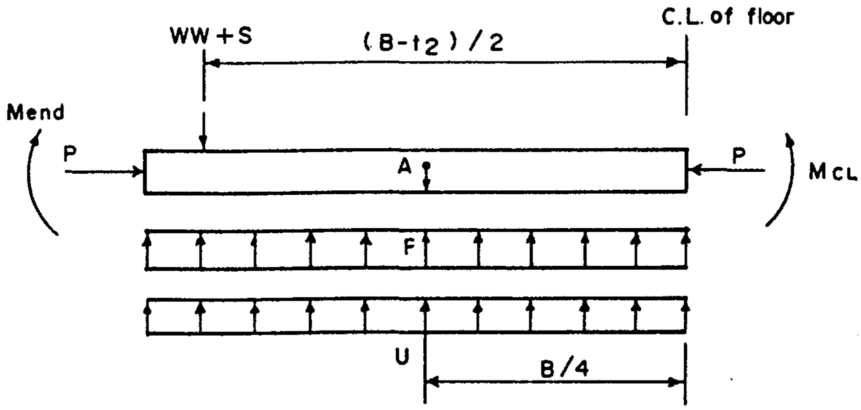
$M_{CL} \quad = [(WW + S)\ (B - t_2)/2] + W\ [(B - 2t_2)/2]$

$\quad - (F + U - A)\ (B/4) - M_{end}$

or,

$M_{CL} \quad = \frac{1}{2}\ (WW + S)(B/2 - t_2) - \frac{1}{2}\ Wt_2 - M_{end}$

4) **Case D (During Maintenance):**



$F \quad = WW + S + A - U$

$P \quad = P_1 + P_2 + P_3$

M_{end} = as in Case C except last term

$M_{CL} = (WW + S)\,[(B - t_2)/2] - (F + U - A)(B/4) - M_{end}$

or

$M_{CL} = [\frac{1}{2}(WW + S)][B/2 - t_2] - M_{end}$

2.3 Return wall

The return wall may be one of the various forms as below. The choice of any form depends on the wall height and allowable bearing capacity of soil.

- Usual Cantilever vertical retaining wall.
- Usual counterfort vertical retaining wall.
- L-shaped vertical cantilever wall.
- Cantilever sloped wall.

The general design considerations with respect to stability and structural strength are same as those of wingwalls as described under article 2.2.

Load	All spans loaded (e.g. dead load)	Imposed load (sequence of loaded spans to give max. bending moment or shearing force)
Uniformly distributed	0.125 0.071 0.071	0.125 0.096 0.096
	0.100 0.100 0.080 0.025 0.080	0.117 0.117 0.101 0.075 0.101
	0.107 0.072 0.107 0.077 0.036 0.036 0.077	(0.116) (0.107) (0.116) 0.121 0.107 0.121 0.099 0.081 0.081 0.099
	0.105 0.080 0.080 0.105 0.078 0.033 0.046 0.033 0.078	(0.116) (0.107) (0.107) (0.116) 0.120 0.111 0.111 0.120 0.100 0.080 0.086 0.080 0.100
Concentrated at midspan	0.188 0.156 0.156	0.188 0.203 0.203
	0.150 0.150 0.175 0.100 0.175	0.175 0.175 0.213 0.175 0.213
	0.161 0.107 0.161 0.169 0.116 0.116 0.169	(0.174) (0.160) (0.174) 0.181 0.160 0.181 0.210 0.183 0.183 0.210
	0.158 0.119 0.119 0.158 0.171 0.11 0.130 0.110 0.171	(0.174) (0.160) (0.160) (0.174) 0.179 0.167 0.167 0.179 0.211 0.181 0.191 0.181 0.211

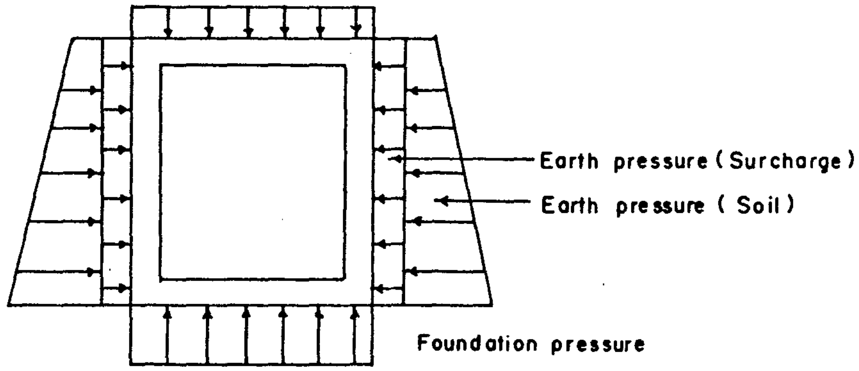
Bending moment = (coefficient) X (total load on one span) X (span)

Bending moment coefficient: above line apply to negative bending moment at supports.
below line apply to positive bending moment in span.

Plate No. I

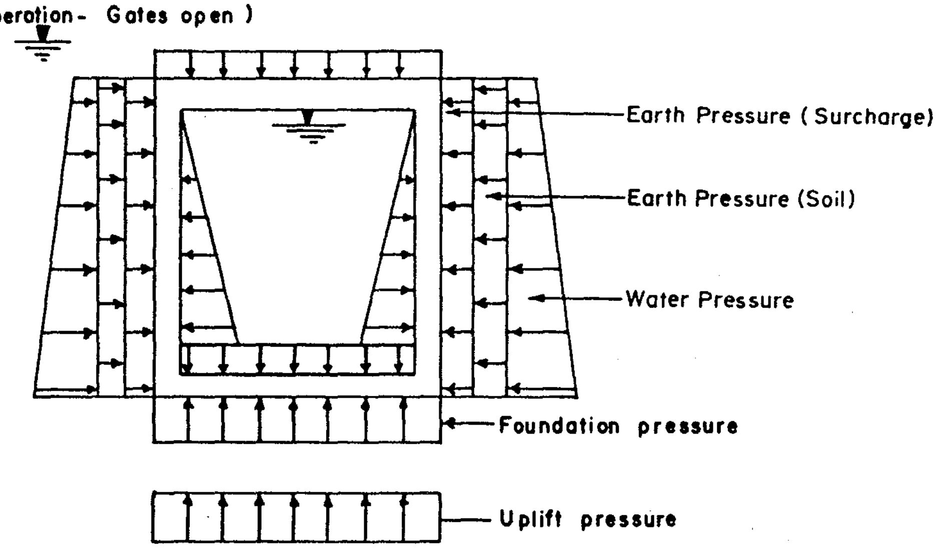
Loading Condition - I

(After Construction)



Loading Condition - 2

(During operation - Gates open)



Loading Condition - 3

(During operation - Gates - closed)

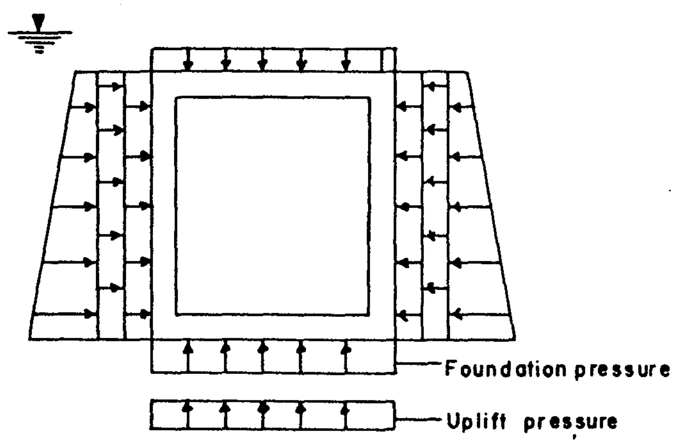
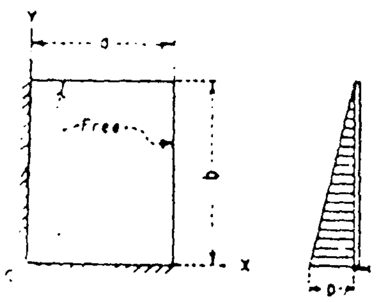
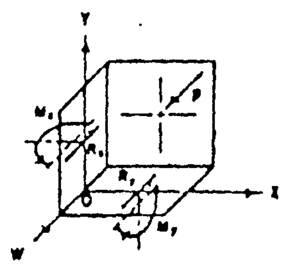


Plate No. 2   Loading Condition for the Design of Barrel

			M_x						M_y					
	y/b	R_x \ x/a	0	0.2	0.4	0.6	0.8	1.0	0	0.2	0.4	0.6	0.8	1.0
a/b = 1/8	1.0	+.0275	+.0027	+.0006	+.0004	+.0002	+.0001	0	0	0	0	0	0	0
	0.8	-.0252	+.0017	+.0011	+.0006	+.0003	+.0001	0	+.0003	+.0002	+.0001	+.0001	+.0001	+.0001
	0.6	+.0501	+.0031	+.0026	+.0011	+.0005	+.0001	0	+.0006	+.0001	+.0002	+.0001	+.0000	-.0000
	0.4	+.0755	[illegible]	+.0029	+.00[illegible]5	+.0007	+.0001	0	+.0009	+.0006	+.0003	+.0001	-.0000	-.0001
	0.2	+.0564	+.0052	+.[illegible]031	+.0015	+.0005	+.0000	0	+.0010	+.0006	+.0002	-.0001	-.0003	-.0004
	0	+.0056	0	+.[illegible]01	+.002[illegible]	+.0008	+.0011	0	0	+.0007	+.0021	+.0038	+.0056	+.0072
		R_x \ R_y	+.[illegible]56	.0008	+.0510	+.0996	+.1819	+.2706						
a/b = 1/4	1.0	+.0076	+.004[illegible]	+.0015	+.[illegible]	+.0016	+.0007	0	0	0	0	0	0	0
	0.8	+.055	+.0078	+.0052	.0031	+.0015	+.0005	0	+.0016	+.0011	+.0007	+.0005	+.0004	+.0004
	0.6	+.1026	+.0119	+.0074	+.0039	+.0015	+.0003	0	+.0024	+.0014	+.0007	+.0001	-.0004	-.0006
	0.4	+.1512	+.0150	+.0085	+.0040	+.0011	-.0002	0	+.0030	+.0015	+.0002	-.0009	-.0017	-.0022
	0.2	+.1475	+.0122	+.0060	+.0022	+.0001	-.0006	0	+.0024	+.0008	-.0006	-.0017	-.0026	-.0032
	0	-.05[illegible]	-	+.0005	+.0014	+.0024	+.0035	0	0	+.0024	+.0069	+.0121	+.0175	+.0221
		R_x \ R_y	-.0898	+.[illegible]	+.[illegible]020	+.1780	+.3009	+.4232						
a/b = 3/8	1.0	+.0840	+.0115	+.0095	+.0066	+.0037	+.0015	0	0	0	0	0	0	0
	0.8	+.0970	+.0180	+.0116	+.0066	+.0029	+.0008	0	+.0036	+.0023	+.0012	+.0004	-.0000	-.0002
	0.6	+.1553	+.0233	+.0133	+.0062	+.0018	-.0002	0	+.0047	+.0024	+.0003	-.0013	-.0026	-.0034
	0.4	+.2050	+.0246	+.0122	+.0045	+.0002	-.0012	0	+.0049	+.0017	-.0012	-.0037	-.0055	-.0067
	0.2	+.1717	+.0157	+.0063	+.0016	-.0005	-.0009	0	+.0030	+.0006	-.0014	-.0028	-.0037	-.0045
	0	+.0044	-	+.0009	+.0024	+.0040	+.0057	0	0	+.0044	+.0119	+.0202	+.0286	+.0354
		R_x \ R_y	+.0144	+.0309	+.1238	+.1911	+.3079	+.4185						
a/b = 1/2	1.0	+.0166	+.0271	+.0167	+.0100	+.0046	+.0013	0	0	0	0	0	0	0
	0.8	+.1402	+.0295	+.0177	+.0090	+.0032	+.0003	0	+.0059	+.0032	+.0010	-.0007	-.0019	-.0026
	0.6	+.1983	+.0334	+.0174	+.0069	+.0009	-.0013	0	+.0067	+.0027	-.0010	-.0041	-.0064	-.0079
	0.4	+.23	+.0309	+.0135	+.0037	-.0011	-.0021	0	+.0062	+.0012	-.0032	-.0067	-.0092	-.0109
	0.2	+.1413	+.0162	+.0058	+.0010	-.0005	-.0004	0	+.0051	+.0004	-.0012	-.0018	-.0019	-.0017
	0	-.0079	0	+.0015	+.0039	+.0064	+.0089	0	0	+.0074	+.0193	+.0318	+.0443	+.0546
		R_x \ R_y	-.0079	+.0573	+.1645	+.2446	+.3698	+.4827						
a/b = 3/4	1.0	+.0974	+.0465	+.0269	+.0107	+.0008	-.0028	0	0	0	0	0	0	0
	0.8	+.[illegible]67	+.0487	+.0242	+.0084	-.0002	-.0027	0	+.0097	+.0040	-.0004	-.0039	-.0065	-.0087
	0.6	+.2303	+.0459	+.0195	+.0046	-.0023	-.0036	0	+.0092	+.0020	-.0044	-.0093	-.0129	-.0155
	0.4	+.2383	+.0360	+.0121	+.0010	-.0030	-.0029	0	+.0072	-.0003	-.0060	-.0096	-.0117	-.0132
	0.2	+.1193	+.0160	+.0042	+.0005	+.0006	+.0016	0	+.0032	+.0008	+.0019	+.0050	+.0083	+.0106
	0	-.0194	0	+.0029	+.0070	+.0110	+.0148	0	0	+.0147	+.0352	+.0548	+.0740	+.0936
		R_x \ R_y	-.0194	+.1105	+.2399	+.3236	+.4489	+.5505						
a/b = 1	1.0	+.1917	+.0662	+.0291	+.0056	-.0059	-.0077	0	0	0	0	0	0	0
	0.8	+.2410	+.0613	+.0243	+.0035	-.0056	-.0063	0	+.0123	+.0039	-.0019	-.0059	-.0089	-.01[illegible]
	0.6	+.2354	+.0518	+.0173	+.0004	-.0059	-.0054	0	+.0104	+.0009	-.0084	-.0112	-.0144	-.0172
	0.4	+.2289	+.0368	+.0092	-.0015	-.0041	-.0028	0	+.0074	-.0015	-.0062	-.0078	-.0080	-.0044
	0.2	+.1047	+.0150	+.0030	+.0007	+.0021	+.0040	0	+.0030	+.0022	+.0073	+.0148	+.0216	+.0288
	0	-.0224	0	+.0046	+.0103	+.0152	+.0197	0	0	+.0232	+.0515	+.0759	+.0987	+.1157
		R_x \ R_y	-.0224	+.1598	+.2991	+.3794	+.4509	+.5566						

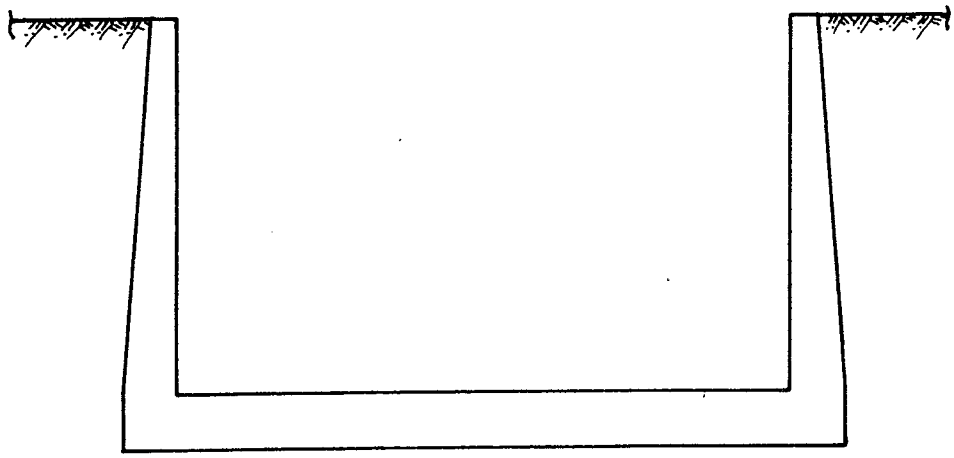


Moment = (Coefficient)(pb^2)
Reaction = (Coefficient)(pb)

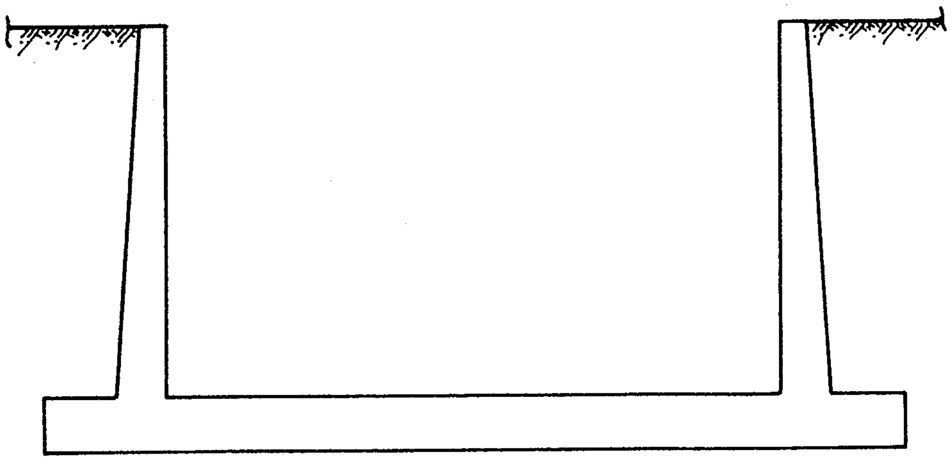


POSITIVE SIGN CONVENTION

PLATE NO 3 Plate fixed along two adjacent edges, moment and reaction coefficients, Load uniformly varying

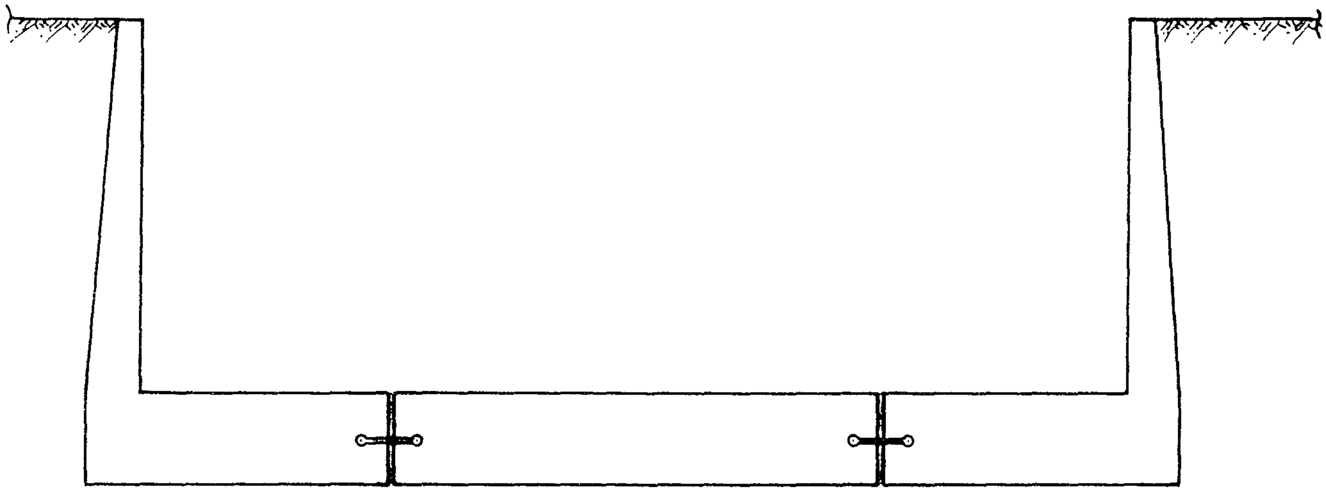


U Type Construction

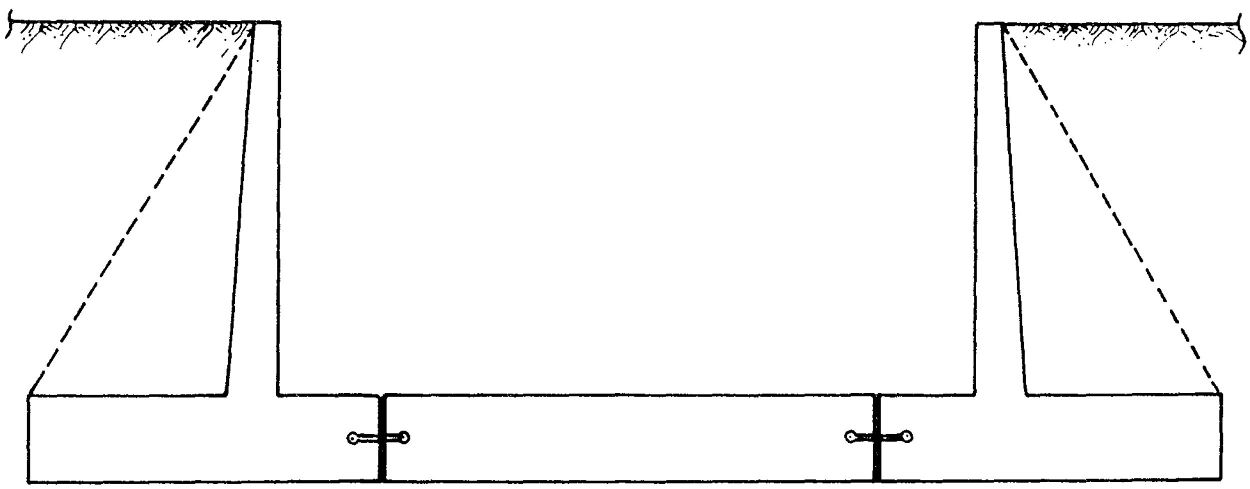


U Type With Projection of Base at ends
Monolithic Wingwall and Apron

Plate No. 4.



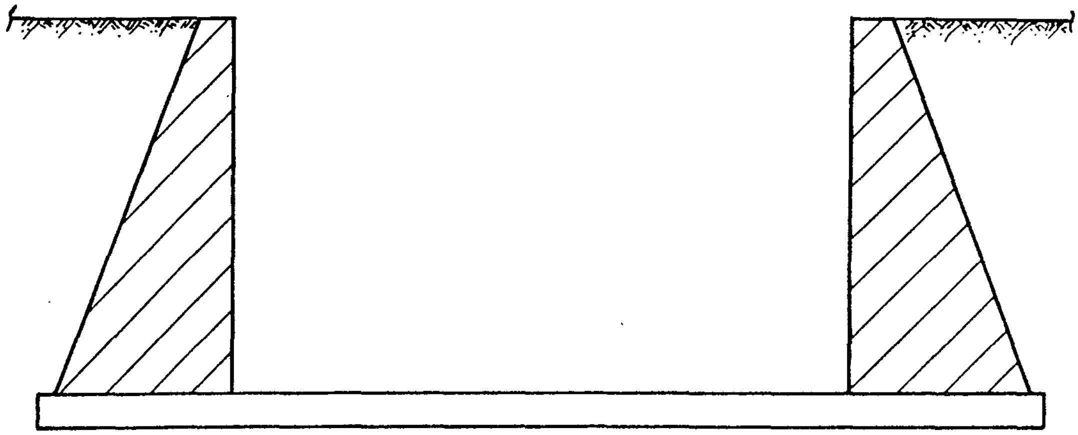
With L-Shaped Wall



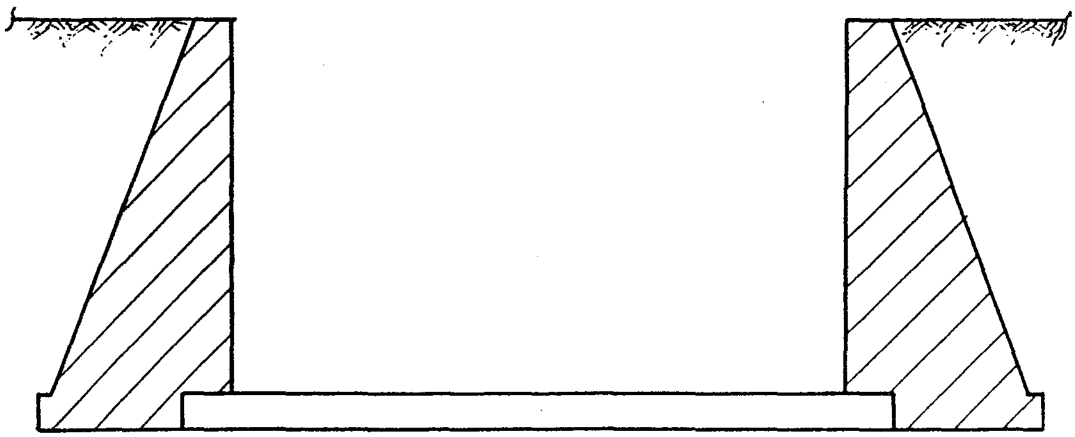
With Cantilever or Counterfort Wall

Separate Wing Wall and Apron

Plate No. 4

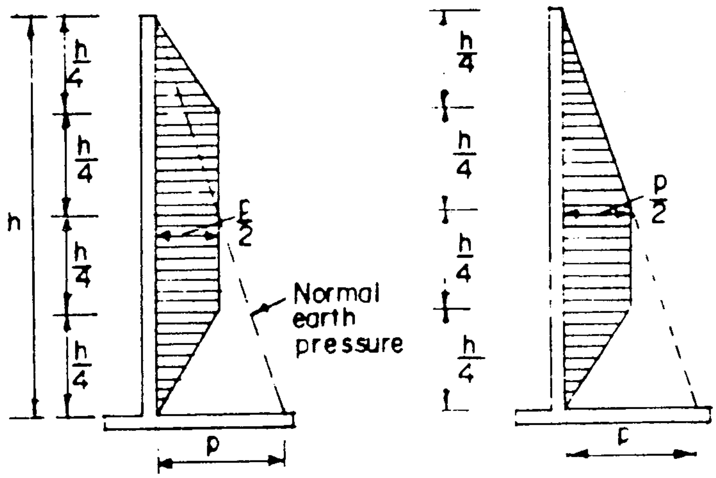


Wall Supported by Base Fully



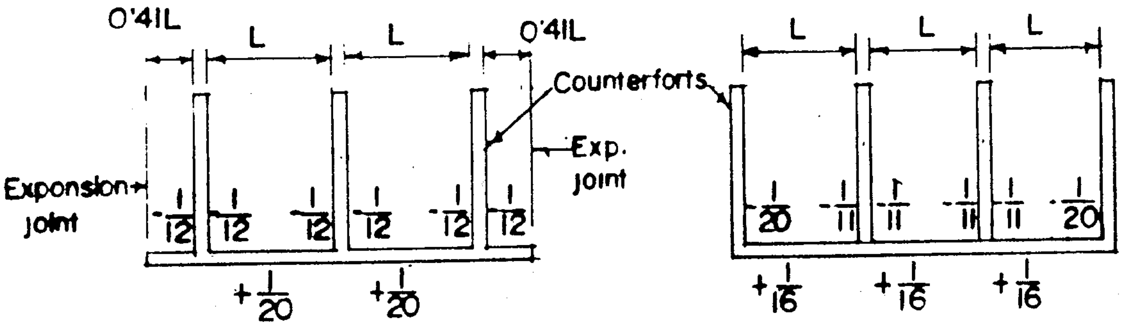
Wall Supported by Base Partly
Combined Wing Wall and Apron .

Plate No. 4



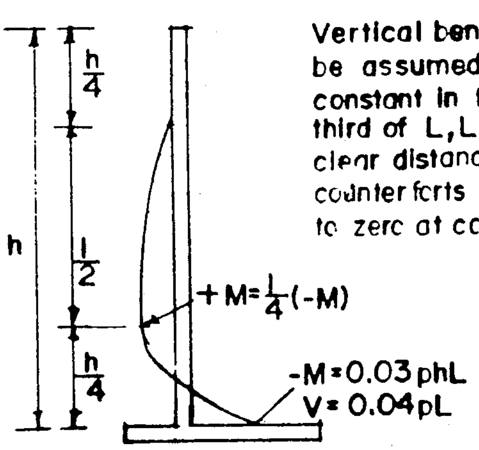
Pressure used for positive moment computation

Pressure used for negative moment computation



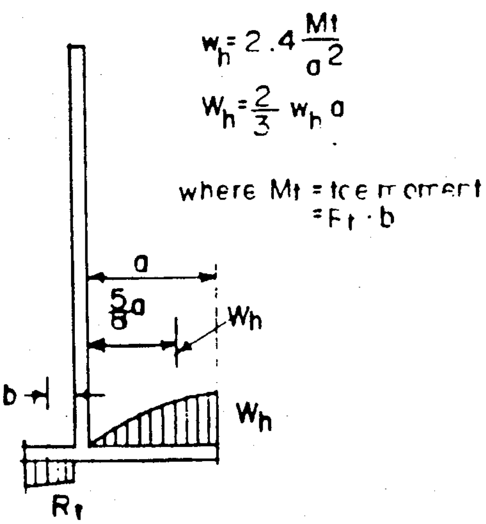
Moment coefficients used in conjunction with the above earth pressures

(a) Horizontal bending in the face slab



Vertical bending may be assumed to be constant in the middle third of L, L being the clear distance between counterforts and reduce to zero at counterforts

(b) Vertical bending in one foot strip of face slab



(c) Equivalent load (downward) on heel slab due to moment in the toe slab

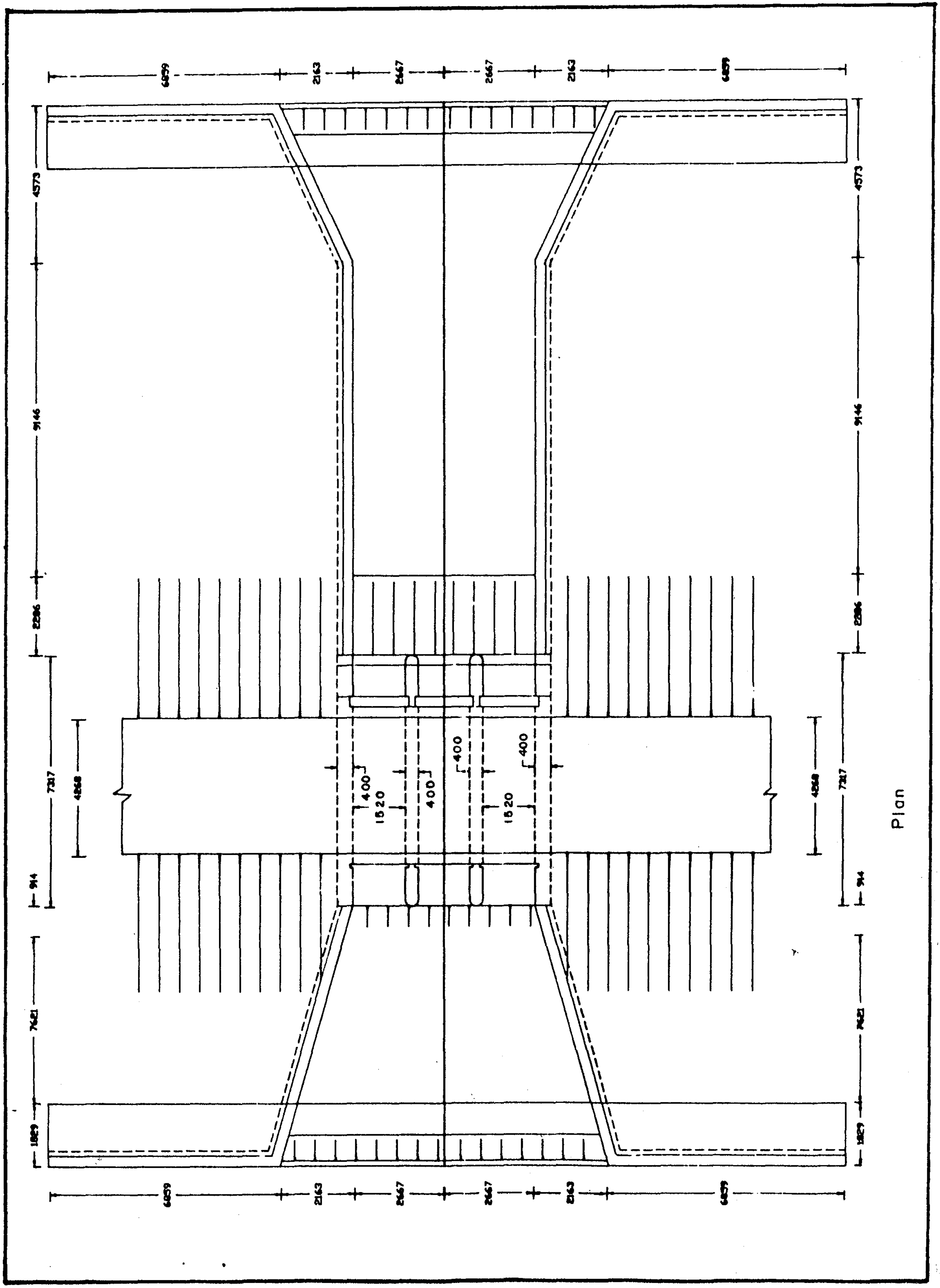
Plate no. 5 Huntington's Design Procedure for Counterfort Wall

STRUCTURAL DESIGN OF REGULATOR

C. DESIGN EXAMPLE

PART - C : DESIGN EXAMPLE

Table of Contents **Page**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400
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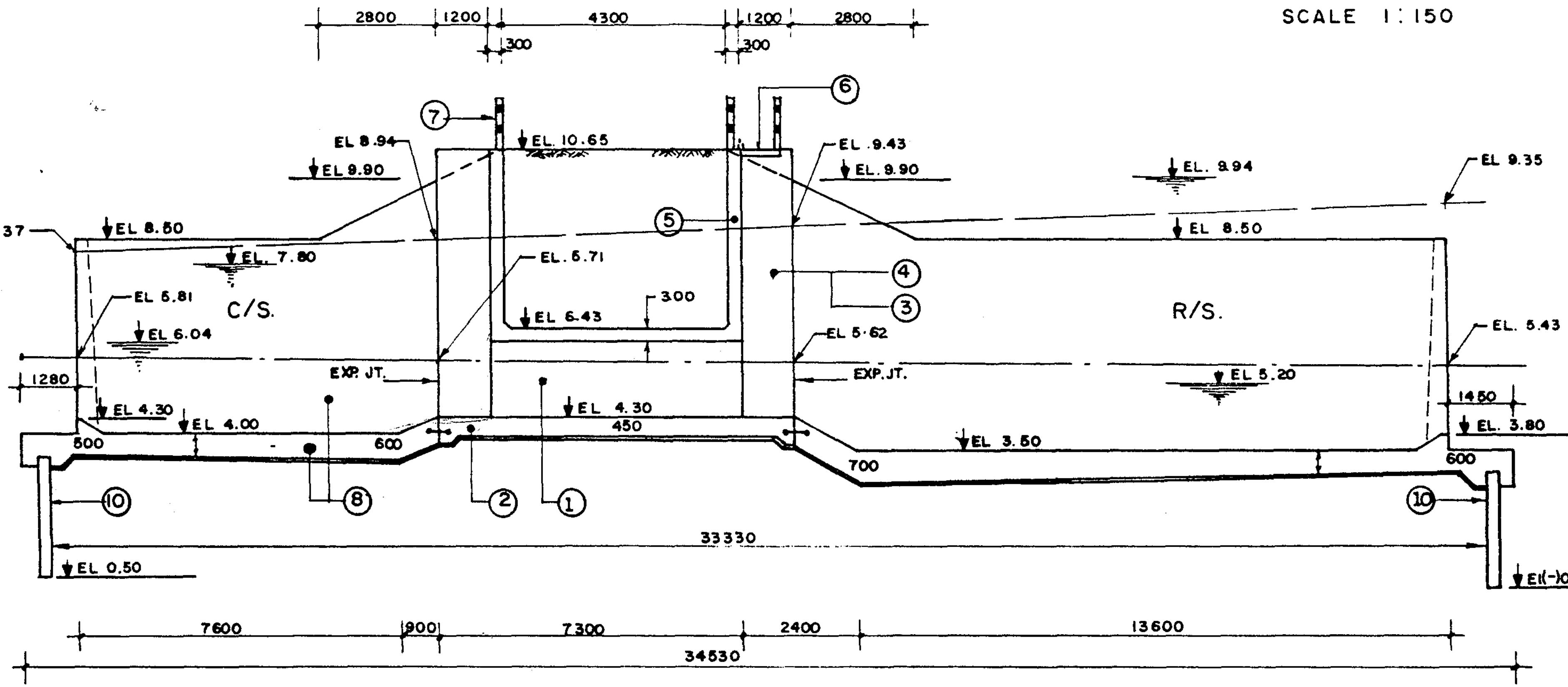
1520
1520
Plan

SCALE 1:150



Longitudinal Profile

LEGEND

1 Box Conduit	6 Operating Platform
2 Extended part of Conduit Base Slab	7 Railing
3 Extended part of Conduit Abutment	8 Wing wall and Apron (Monolithic)
4 Extended part of Conduit Pier	9 Return wall
5 Head wall	10 Sheet pile

1.0 STRUCTURAL LAYOUT

The structural layout of regulator in the form of plan and longitudinal profile has been shown in the previous pages.

2.0 DESIGN CRITERIA

2.1 Design Method

Working stress method has been followed in this text.

2.2 Loads

Reinforced Cement Concrete : 23.6 Kn/m^3
Moist Soil : 17.3 Kn/m^3
Saturated Soil : 18.90 Kn/m^3
Water : 9.80 Kn/m^3
Truck load : H10 or = 90 Kn Truck (On Regulator)
Imposed load : 7.20 Kn/m^2 (On operating deck)
Gate hoisting load + Accessories = 27 kn (On operating deck)
Depth of soil equivalent to LL Surcharge : 0.90 m

2.3 Backfill Soil Parameters

Sand of $\phi = 30^\circ$ (for backfill soil of Wingwall & Returnwall)
Sand of $\phi = 20^\circ$ (for conduit, abutments & headwalls)
Unit weight of soil = same as above

2.4 Allowable stresses and Design Constant

fc = 6.9N/mm^2 (f'c = 17.3 N/mm^2)

fs = 124 N/mm^2 (fy = 276 N/mm^2)

Vc = 0.377 N/mm^2

n = 10, K = 0.357, J = 0.881

R = 1.1 N/mm^2

2.5 Simplified Design formula

The simplified design formulas using the above mentioned design criteria have been derived as follows:

Effective depth from Moment, $dm = \left[\dfrac{M \times 10^6}{R \times b}\right]^{\frac{1}{2}} = \left[\dfrac{M \times 10^6}{1.1 \times b}\right]^{\frac{1}{2}}$

$= 30.15M^{\frac{1}{2}}$ for b = 1.00m

Effective depth from Shear, $ds = \dfrac{V \times 10^3}{v_c \times b} = \dfrac{V \times 10^3}{0.377xb}$

$= 2.65V$ for b = 1.00 m

Reinforcement Area, $A_s = \dfrac{M \times 10^6}{fs\ jd} = \dfrac{M \times 10^6}{124 \times 0.881 \times d}$

$= 9154\ M/d$

Where, M in Kn-m, V in Kn and b, d, d_m, d_s in mm and A_s in mm^2/m.

Shear & Moment due to Earth Pressure on Vertical Wall of level Backfill.

<u>For Fully Moist Soil (Refer fig. 2a of page 7-24)</u>

Total Pressure, $P = \frac{1}{2} \text{ Ca } \gamma_{\text{moist}} h^2$

$= \frac{1}{2} \times 0.33 \times 17.3h^2$

$= 2.85h^2$

Moment, $M = 1/6 \text{ Ca } \gamma_{\text{moist}} h^3$

$= 1/6 \times 0.33 \times 17.3 \times h^3$

$= 0.95h^3$

<u>For Fully Saturated Soil (Refer fig. 2c of page 7-24)</u>

Total Pressure, $P = \frac{1}{2}(\text{Ca } \gamma_{\text{sub}} + \gamma_w) h^2$

$= \frac{1}{2}(0.33x9.1+9.8)h^2$

$= 6.40 h^2$

Moment, $M = 1/6 (\text{Ca } \gamma_{\text{sub}} + \gamma_w) h^3$

$= 2.14 h^3$

<u>For Partly Saturated Soil (Refer fig. 2d of page 7-24)</u>

Total Pressure, $P = P_1 + P_2 + P_3$

$= 2.85h_1^2 + 0.33 \times 17.3 \ h_1 h_2 + 6.40 h_2^2$

$= 2.85h_1^2 + 5.71 h_1 h_2 + 6.40 h_2^2$

Where, $P_1 = \frac{1}{2}\ Ca\ \gamma_{moist}\ h_1^2 = \frac{1}{2} \times 0.33 \times 17.3h_1^2 = 2.85h_1^2$

$P_2 = Ca\ \gamma_{moist}\ h_1\ h_2 = 0.33 \times 17.3h_1\ h_2 = 5.71\ h_1h_2$

$P_3 = \frac{1}{2}(Ca\ \gamma_{sub} + \gamma_w)\ h_2^2 = \frac{1}{2}(0.33\text{x}9.1+9.8)h_2^2 = 6.40\ h_2^2$

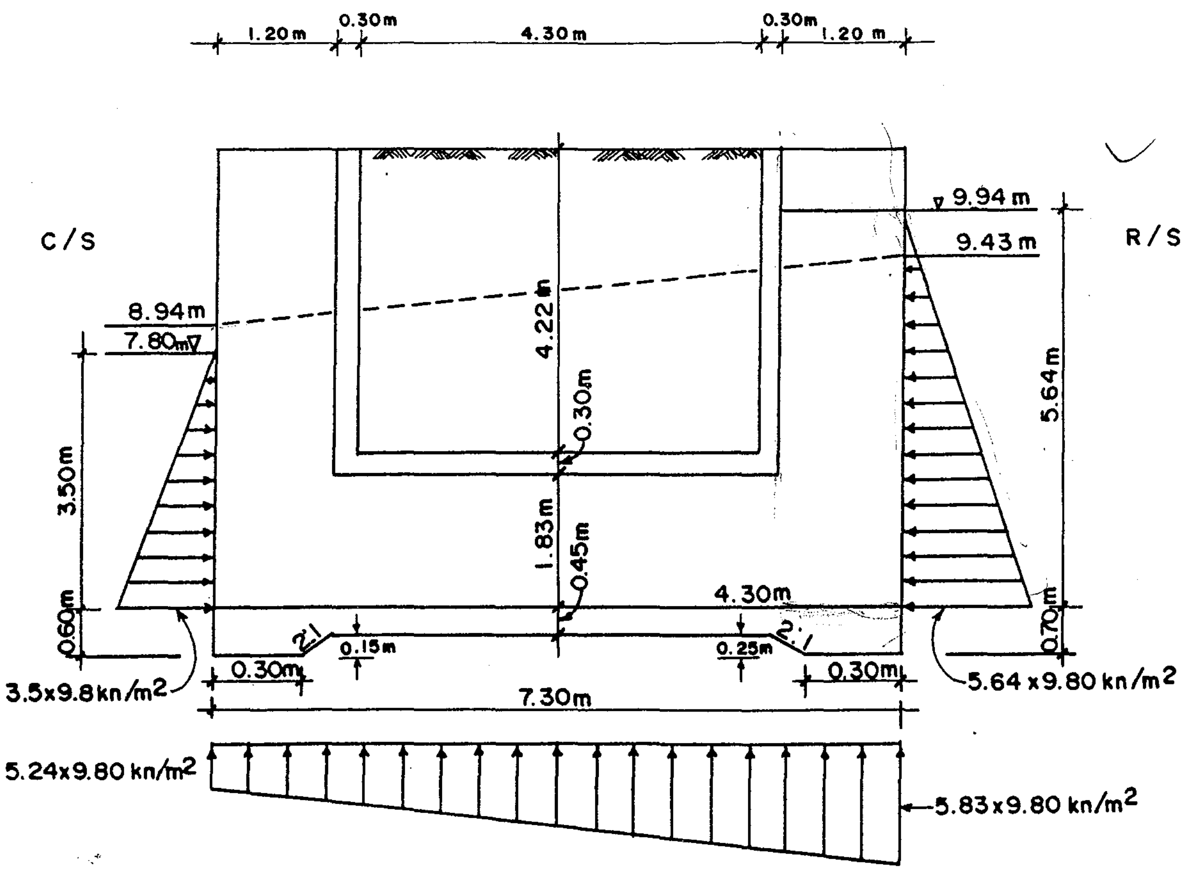
$\text{Moment} = P_1 \times (h_2 + h_1/3) + P_2 \times (h_2/2) + P_3 \times (h_2/3)$

2.6 Other relevant criteria

Other relevant criteria may be obtain from part A of this text.

3.0 STABILITY ANALYSIS

3.1 Central part of the structure



3.1.1 Detail loads & Pressures

(i) Concrete & Earthload

Base slab	:	0.45 x 7.30 x 6.16 x 23.6	= 477.56 kn
		0.15 x 0.45 x 6.16 x 23.6	= 9.81 ''
		0.25 x 0.55 x 6.16 x 23.6	= 19.99 ''
Top slab	:	0.30 x 4.9 x 6.16 x 23.6	= 213.70 ''
Head wall	:	2x0.30 x4.22 x 6.16 x 23.6	= 368.09 ''
Pier	:	2x0.40 x1.83 x 4.90 x 23.6	= 169.29 ''
		4x0.40 x1.20 x 6.35 x 23.6	= 287.73 ''
Abutment	:	2x0.40 x1.83 x 4.90 x 23.6	= 169.30 ''
		4x0.5(0.40+0.30)x4.22x1.20x23.6	= 167.31 ''
		4 x 2.13 x 1.20 x 23.60	= 241.29 ''
Embankment	:	6.16 x 4.30 x 4.22 x 17.3	=1933.78 ''
			= 4057.85 Kn

(Operating platform, Railing, Gate etc. not included).

(ii) Water load

Upstream part	:	3.5 x 1.20 x 4.56 x 9.80	= 187.69 Kn
Downstream part	:	5.64 x 1.20 x 4.56 x 9.80	= 302.45 ''
Central part	:	4.56 x 4.9 x 1.83 x 9.80	= 400.72 ''
			= 890.86 Kn

(iii) Uplift

Uplift Pressure = 0.5(5.83+5.24)x7.30x6.16x9.80 = 2439.20 Kn

(iv) Water pressure

From c/s to R/s: $0.50x(3.5)^2$ x 9.80 x 5.36 = 321.73 Kn

From R/s to c/s: $0.50x(5.64)^2$ x 9.80 x 5.36 = 835.45 Kn

3.1.2 Summary of loads and pressures

(i)	Concrete and soil load		= 4057.85 Kn
(ii)	Water load		= 890.86 Kn
		Total	= 4948.71 Kn
(iii)	Uplift		= 2439.20 Kn
(iv)	Net water pressure		= 835.45 - 321.73
			= 513.72 Kn

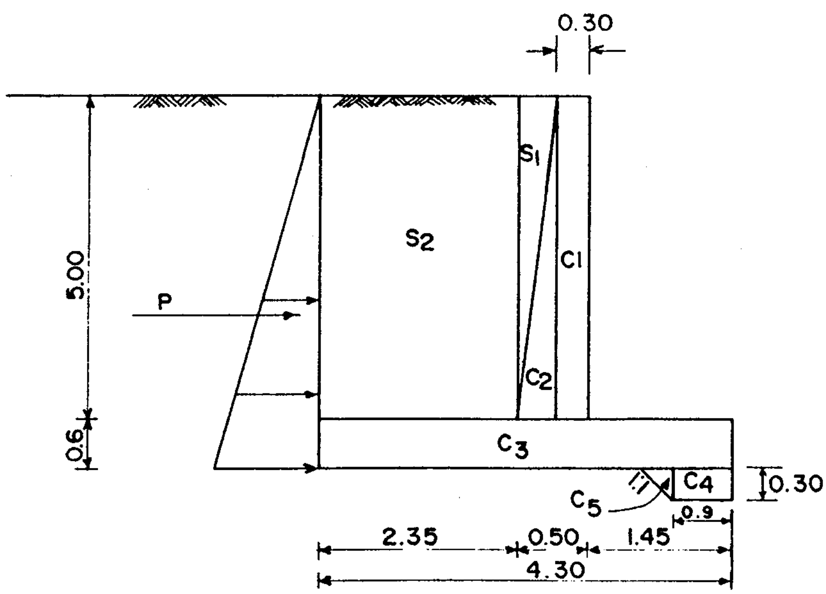
3.1.3 Factor of safety

(i) Uplift $= \dfrac{4948.71}{2439.20} = 2.03 > 1.10$ O.K.

(ii) Sliding $= \dfrac{0.36(4948.71 - 2439.20)}{513.72} = 1.76 > 1.5$ O.K.

3.2 Return wall (Typical computation shown for R/S Return wall)

Loading After Construction of wall:



Stability Analysis

Weight and Pressure (Kn)		Arm (m)	Mr (Kn.m)
C1 : 0.3 x 5 x 23.6	= 35.40	1.60	56.64
C2 : 0.5 x 0.20 x 5 x 23.6	= 11.80	1.82	21.48
C3 : 0.60 x 4.3 x 23.6	= 60.89	2.15	130.91
C4 : 0.90 x 0.30 x 23.6	= 6.37	0.45	2.87
C5 : 0.5 x 0.3 x 0.3 x 23.6	= 1.06	1.00	1.06
S1 : 0.5 x 0.20 x 5 x 17.3	= 8.65	1.88	16.26
S2 : 2.35 x 5 x 17.3	=203.27	3.12	634.20
	327.44		863.42
	=======		=======
P : 2.85 x 5.6 x 5.6	= 89.38	2.17	193.95

Factor of Safety

$$\text{Overturning} = \frac{863.42}{193.95} = 4.45 > 1.50 \text{ O.K.}$$

$$\text{Sliding} = \frac{327.44 \times 0.466}{89.38} = 1.71 > 1.50 \text{ O K.}$$

Imposed Pressure

$$\text{Eccentricity, } e = \frac{4.30}{2} - \frac{863.42 - 193.95}{327.44}$$

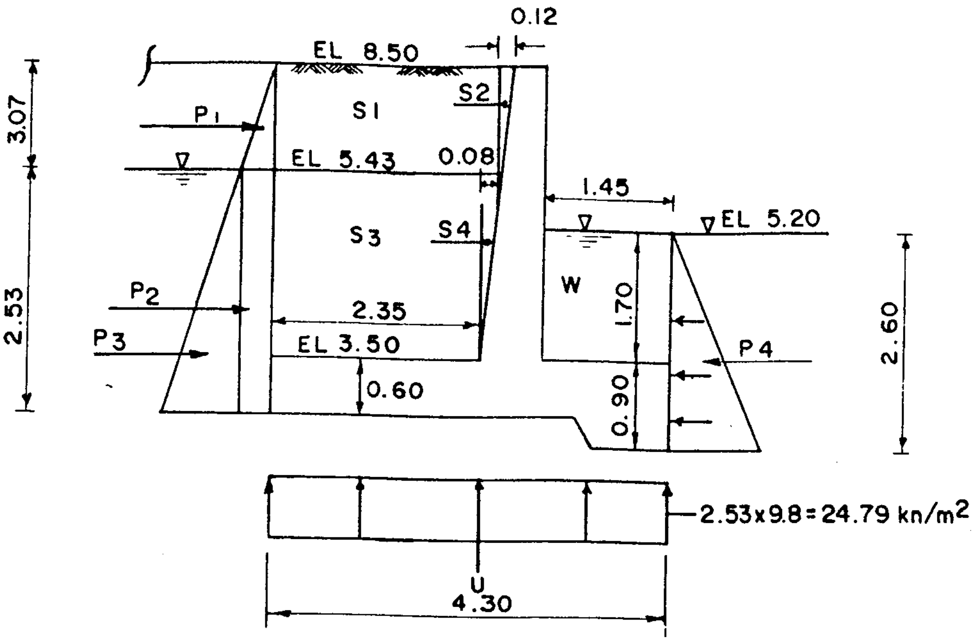
$$= 2.15 - 2.045 = 0.105 \text{ m}$$

$$\text{Pressure} = \frac{327.44}{4.30} \left(1 \pm \frac{6 \times 0.105}{4.30} \right)$$

$$= 87.27 \text{ Kn/m}^2 \text{ (toe)}$$

$$= 65.03 \text{ Kn/m}^2 \text{ (heel)}$$

Loading during operation of structure:



Weight and Pressure (Kn)		Arm (m)	Moment (Kn-m)
C1 to C5 : (As before)	= 115.52	-	212.96
S1 : 3.07 x 2.43 x 17.30	= 129.06	3.08	397.50
S2 : 0.5 x 0.12 x 3.07 x 17.30	= 3.19	1.83	5.84
S3 : 1.93 x 2.35 x 18.90	= 85.72	3.12	267.45
S4 : 0.5 x 1.93 x 0.08 x 18.90	= 1.46	1.92	2.80
W : 1.70 x 1.45 x 9.80	= 24.16	0.73	17.64
U : 24.79 x 4.30	=(-)106.60	2.15	(-)229.19
	ΣV= 252.51		ΣMv= 675.00
P1 : 2.85 x 3.07 x 3.07	= 26.86	3.85	103.41
P2 : 0.33 x 17.3 x 3.07 x 2.53	= 44.34	1.56	69.17
P3 : 6.40 x 2.53 x 2.53	= 40.96	1.14	46.69
P4 : 0.5 x 9.80 x 2.6 x 2.6	=(-)33.12	0.87	(-)28.81
	ΣP = 79.04		ΣMp=190.46

Factor of safety:

$$\text{Overturning} = \frac{675.00}{190.46} = 3.54 > 1.50 \text{ O.K.}$$

$$\text{Sliding} = \frac{0.466 \times 252.51}{79.04} = 1.49 \approx 1.50$$

$$\text{Uplift} = \frac{252.51 + 106.60}{106.60} = 3.37 > 1.10 \text{ O.K.}$$

Foundation Pressure

$$\text{Eccentricity, } e = \frac{4.30}{2} - \frac{675.00 - 190.46}{252.51}$$

$$= 2.15 - 1.91 = 0.24 \text{ m}$$

$$\text{Pressure} = \frac{252.51}{4.30} \left(1 \pm \frac{6 \times 0.24}{4.30} \right)$$

$$= 78.39 \text{ Kn/m}^2 \text{ (toe)}$$

$$= 39.06 \text{ Kn/m}^2 \text{ (heel)}$$

4.0 STRUCTURAL DESIGN

4.1 Central Part

4.1.1 Railing

Post Size	=	200mm x 200mm
Railing Size	=	150 mm x 150 mm
Spacing	=	(Length of headwall - 0.20)/ No of Span
	=	(5.96 - 0.20)/ 5 = 1.152m c/c of post

i) Live Load on rail members

At top = 1.46 kn/m (100 lb/ft)

At side = 1.46 kn/m

ii) Design of Rail member

DL = 0.15x0.15x23.6 = 0.53 kn/m

LL = @ 1.46 kn/m = 1.46 kn/m

$$\frac{}{1.99 \text{ kn/m}}$$

$\pm M = 1.99 \times 1.152^2/10$

$= 0.26$ kn-m

V = 1.15x1.99x1.152/2 = 1.32 kn

$$d_s = \frac{1.32 \times 10^3}{0.377 \times 150} = 23.34 \text{ mm}$$

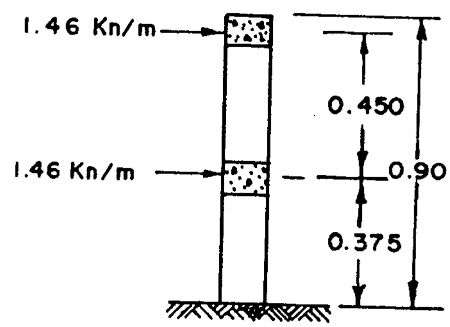
$$d_m = \left[\frac{0.26 \times 10^6}{1.1 \times 150}\right]^{\frac{1}{2}} = 39.70 \text{ mm}$$

$d_{provided} = 150 - (25+5) = 120$ mm

$$As = \frac{9154 \times 0.26}{120} = 19.83 \text{ mm}^2$$

Use 4-10mm ϕ one in each corner with 6mm ϕ stirrup @ 150mm c/c

iii) Design of Post



$M = (1.46 \times 0.375 + 1.46 \times 0.825) \times 1.152$

$= 2.02$ kn-m

$$d = \left[\frac{2.02 \times 10^6}{1.1 \times 200}\right]^{\frac{1}{2}} = 95.82 \text{ mm}$$

$d_{Provided} = 200-(25+6) = 169$mm

$$As = \frac{9154 \times 2.02}{169} = 109.41 \text{ mm}^2$$

use 4 - 12 mm ϕ with 6 mm ϕ stirrups @ 200 mm c/c

4.1.2 Headwall

h = El 10.65-El 6.43 = 4.22m

h_1 = 0.90m

$Ca = 0.49$ ($\phi = 20^0$), γ moist = 17.3 kn/m²

$Pa = Ca\ \gamma\ h = 0.49 \times 17.3 \times 4.22 = 35.77$ kn/m²

$Ps = Ca\ \gamma\ h = 0.49 \times 17.3 \times 0.90 = 7.63$ kn/m²

i) Design for horizontal reinforcement

$P_{design} = 0.5x35.77+7.63 = 25.51$ kn/m^2

Moment $= \pm 25.51x1.52^2/10 = 5.89$ kn-m

Shear $= 1.15x25.51x1.52/2 = 22.29$ kn

$d_{shear} = 22.29/.377 = 59.12$mm

$$d_{moment} = \left[\frac{5.89x1000}{1.1}\right]^{\frac{1}{2}} = 73.17 \text{ mm}$$

$d_{provided} = 300-(80+8) = 212$ mm

$$As = \frac{9154x5.89}{212} = 254.32 \text{ mm}^2/\text{m} \ (12\text{mm } \phi \ 444 \text{ mm c/c})$$

ii) Design for vertical reinforcement

$P_{design} = 35.77+7.63 = 43.4$ kn/m^2

Max moment at base of headwall

(-) M = 0.03 phl = 0.03x43.40x4.22x1.52

= 8.35 kn-m

$$As = \frac{9154x8.35}{212} = 360.55\text{mm}^2/\text{m} \ (12\text{mm}\phi \ @313\text{mm c/c})$$

Use following Reinforcement

Earth face 12 mm ϕ @ 300mm c/c (Both direction)

Exposed face 16 mm ϕ @ 300 mm c/c (")

4.1.3 Operation Deck

Design Span = 1.52m

Slab thickness = 0.25m

Width of slab = 1.20m

Design width of slab = 0.25m (assumed)

i) Loading

Uniform Dead load of slab = 0.25x0.25x23.60 = 1.48 kn/m

Uniform LL @ 7.2 kn/m^2 = 7.2x0.25 = 1.80 kn/m

Gates, hoist & hoisting load (Concentrated at midspan),

GHL = 27/2 = 13.5 kn

ii) Max. Moment & shear (M = cws, 3 span), w = total load

Load	+ M Coeff(c)	+ M	- M Coeff(c)	- M
DL = 1.48 Kn/m	0.080	0.27	0.100	0.34
LL = 1.80 Kn/m	0.101	0.42	0.117	0.49
GHL= 13.50 Kn	0.213	4.37	0.175	3.59
		5.06		4.42

Design Max. Moment = 5.06 Kn-m

$$d_{moment} = \left[\frac{5.06 \times 10^6}{1.1 \times 250} \right]^{\frac{1}{2}} = 135.65 \text{ mm}$$

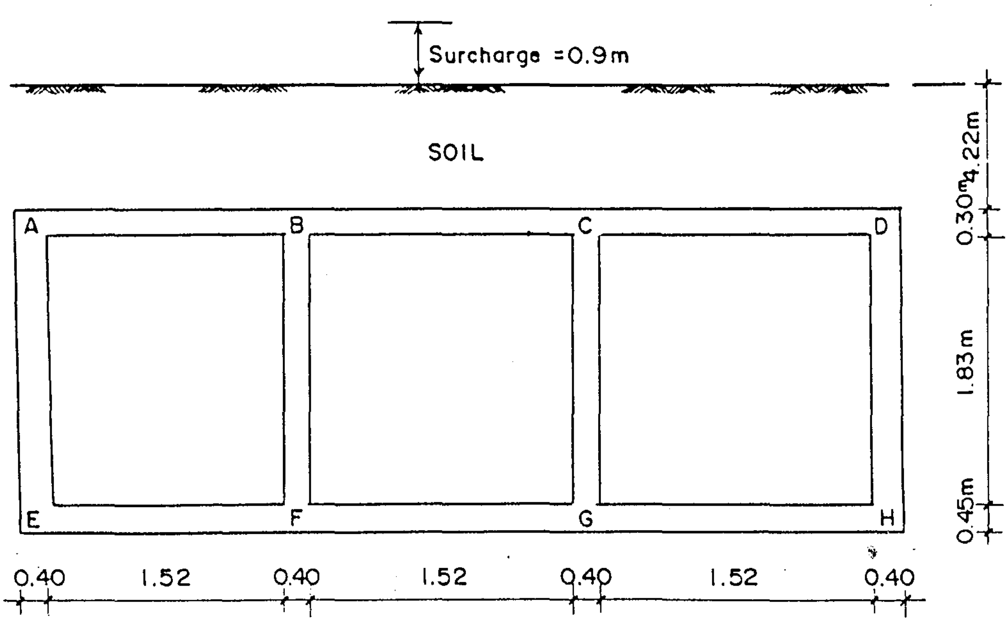
$d_{provided} = 250 - (40+6) = 204$ mm > d_{moment} O.K.

$$As = \frac{9154 \times 5.06}{204} = 227 \text{ mm}^2$$

use 2 - 12 mm ϕ bar within 250 mm width of slab
& 12 mm ϕ bar @ 150 mm c/c at other portion of slab.

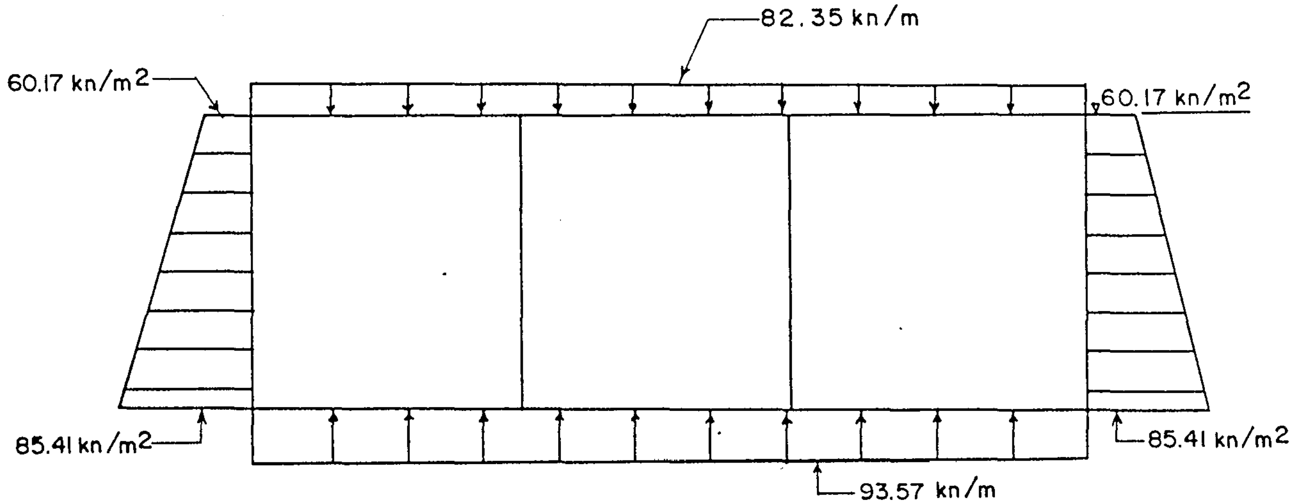
4.1.4 Conduit

i) **Frame dimensions and distribution factors:**



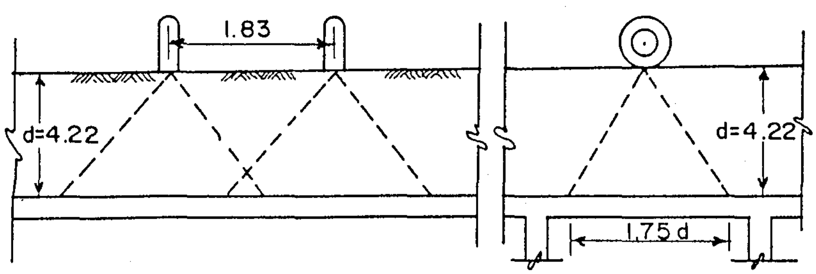
Joint	Member	Length(m)	t(m)	t^3/L	Dist. Factor
A/D	AB/DC	1.92	0.30	0.014	0.325
	AE/DH	2.20	0.40	0.029	0.675
B/C	BA/CD	1.92	0.30	0.014	0.246
	BC/CB	1.92	0.30	0.014	0.246
	BF/CG	2.20	0.40	0.029	0.508
E/H	EA/HD	2.20	0.40	0.029	0.381
	EF/HG	1.92	0.45	0.047	0.619
F/G	FE/GH	1.92	0.45	0.047	0.382
	FG/GF	1.92	0.45	0.047	0.382
	FB/GC	2.20	0.40	0.029	0.236

ii) Loadings on frame



iii) Loading on top slab

Wheel load



Depth of Earth = 4.22 m

Length within which wheel loads distribute:

Along the embankment = 1.75d = 1.75 x 4.22 = 7.38 m

Transverse = 1.75d+1.83 = 1.75 x 4.22 + 1.83

= 9.21 m> available width of 4.30 m, so use 4.30 m

Distributed area = 7.38 x 4.30 = 31.73 m^2

Load of rear wheels = 0.80 x 90 = 72 Kn

Unit load = 72/31.73 = 2.27 Kn/m^2

Load for H10 truck	= from above	= 2.27 Kn/m^2
Embankment earth	= 4.22 x 17.3	= 73.0 Kn/m^2
Self weight of top slab	= 0.30 x 23.6	= 7.08 Kn/m^2
		= 82.35 Kn/m^2

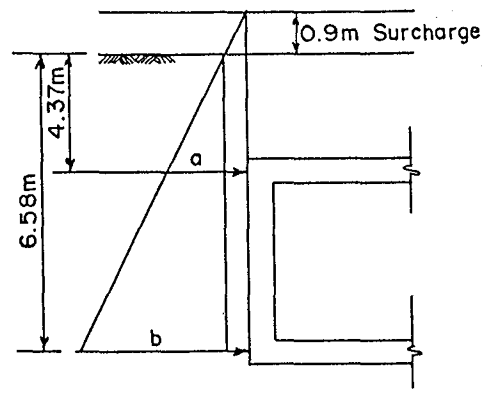
iv) Load on bottom slab

Pier & Abutment = (4 x 0.40 x 1.83 x 23.6)/6.160	= 11.22 Kn/m^2
Load from top slab = (as above) (Bottom slab not included for structural design).	= 82.35 Kn/m^2
	93.57 Kn/m^2

v) Load on Abutment slab

Earth pressure on side wall of box conduit

Loading : After construction.
Earth pressure coefficient at rest = 1-Sinϕ = 1-Sin 20° = 0.66.
At any depth, earth pressure with h' of soil surcharge, P = Ca γ (h+h').



Earth pressure at centre of top slab = 0.66x17.3x(4.37+0.9)= 60.17 Kn/m^2

Earth pressure at centre of bottom slab = 0.66x17.3x(6.58+0.9)= 85.41 Kn/m^2

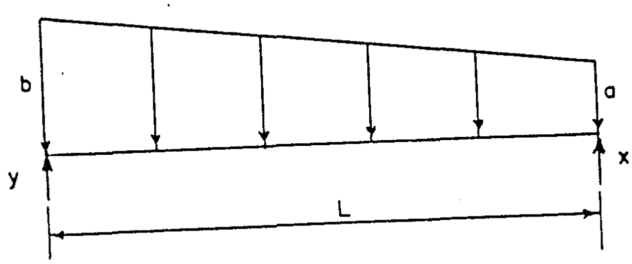
Fixed end moments

$$\text{Top slab} = \frac{82.35 \times 1.92^2}{12} = 25.30 \text{ Kn-m}$$

$$\text{Bottom slab} = \frac{93.57 \times 1.92^2}{12} = 28.74 \text{ Kn-m}$$

$$\text{Side slab (top)} = \frac{2.20^2}{60} (3 \times 60.17 + 2 \times 85.41) = 28.34 \text{ Kn-m}$$

$$\text{Side slab (Bottom)} = \frac{2.20^2}{60} (3 \times 85.41 + 2 \times 60.17) = 30.38 \text{ Kn-m}$$



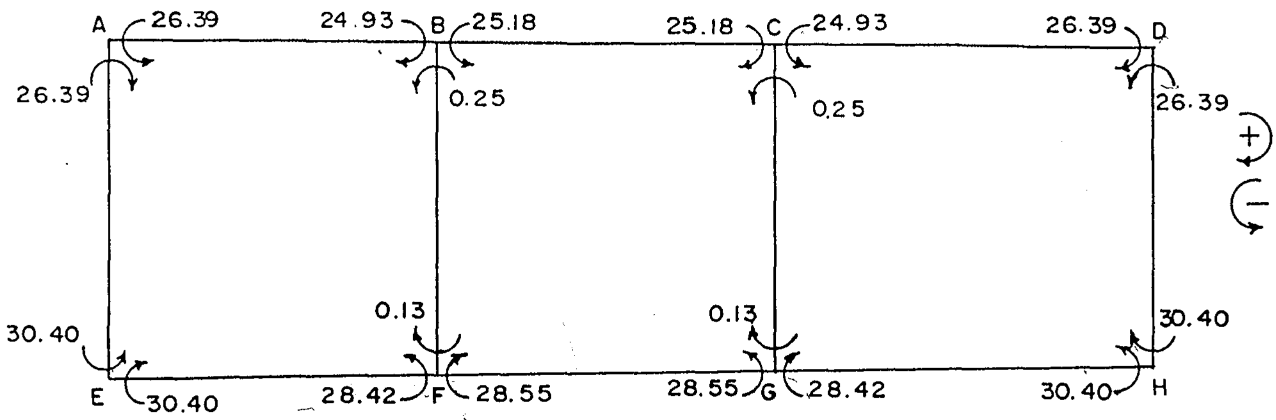
$Mx = L^2/60\ (3a + 2b)$

$My = L^2/60\ (3b + 2a)$

Calculations for moment distribution are shown in Table 8.

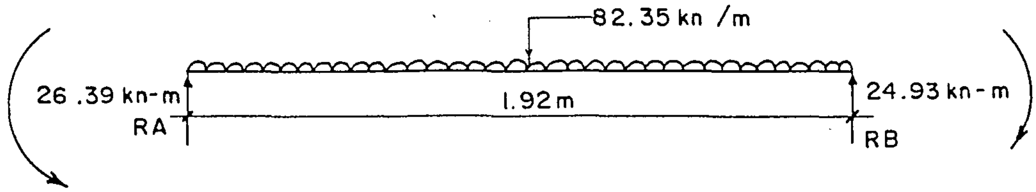
Table 8. Moment distribution

Joint	A and D		B and C			E and H		F and G		
Member	AB/DC	AE/DH	BA/CD	BC/CB	BF/CG	EA/HD	EF/HG	FE/GH	FB/GC	FG/GF
Distri. factor	0.325	0.675	0.246	0.246	0.508	0.381	0.619	0.382	0.236	0.382
F.E.M.	-25.30	+28.34	+25.30	-25.30	0	-30.38	+28.74	-28.74	0	+28.74
D.M.	-0.99	-2.05	0	0	0	+0.62	+1.02	0	0	0
C.O.M.	0	+0.31	-0.49	0	0	-1.03	0	+0.51	0	0
D.M.	-0.10	-0.21	+0.12	+0.12	+0.25	+0.39	+0.64	-0.19	-0.13	-0.19
Total	-26.39	+26.39	+24.93	-25.18	+0.25	-30.40	+30.40	-28.42	-0.13	+28.55



vi) Frame Analysis for shear and Moment

a) Top slab



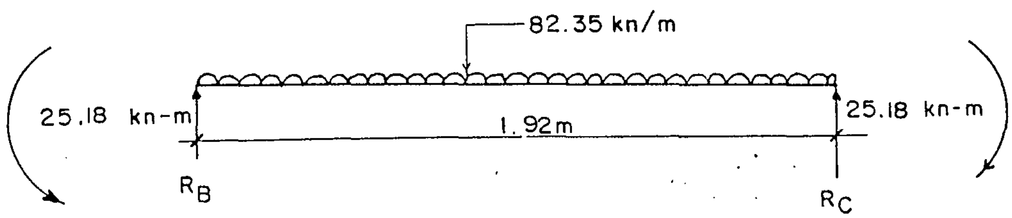
$$\frac{82.35 \times 1.92^2}{2} + 1.46 = R_A \times 1.92$$

or

R_A = 79.82 Kn, R_B = 82.35 x 1.92 - 79.82 = 78.29 Kn

$$\text{Location of zero Shear} = \frac{79.82}{82.35} = 0.97 \text{ m (from } R_A\text{)}$$

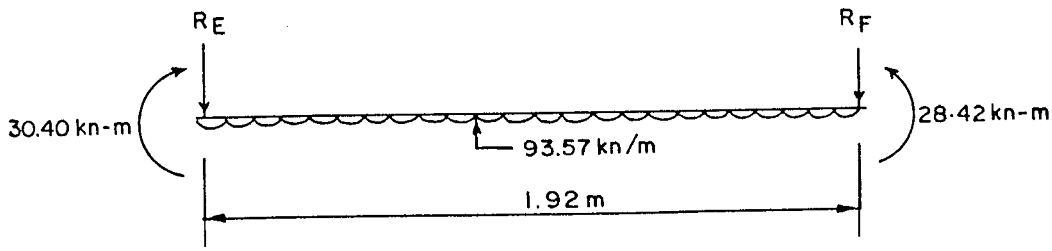
$$+ M_{max} = 79.82 \times 0.97 - 26.39 - 82.35 \times \frac{0.97^2}{2} = 12.29 \text{ Kn-m (Bott.ten.)}$$



$$R_B = R_C = \frac{82.35 \times 1.92}{2} = 79.05 \text{ Kn}$$

$$+M_{max} = \frac{82.35 \times 1.92^2}{8} - 25.18 = 12.77 \text{ Kn-m (Bottom tension)}$$

b) **Bottom slab**



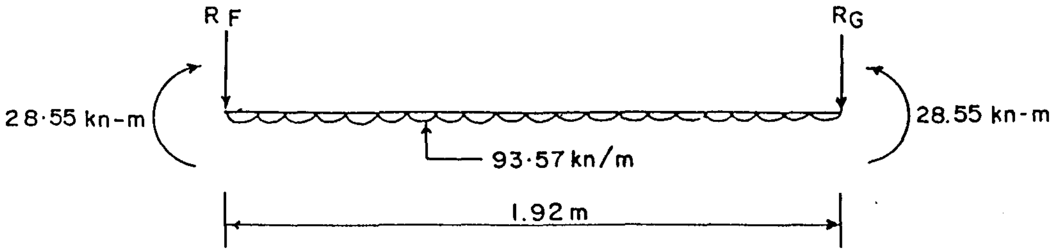
$$93.57 \times \frac{1.92^2}{2} + 1.98 = R_E \times 1.92, \quad R_E = 88.79 \text{ Kn}$$

$$R_F = 93.57 \times 1.92 - 90.86 = 88.79 \text{ Kn}$$

$$\text{Location of zero Shear} = \frac{90.86}{93.57} = 0.97 \text{ m (from } R_E\text{)}$$

$$+M_{max} = 90.86 \times 0.97 - 30.40 - 93.57 \times \frac{0.97^2}{2}$$

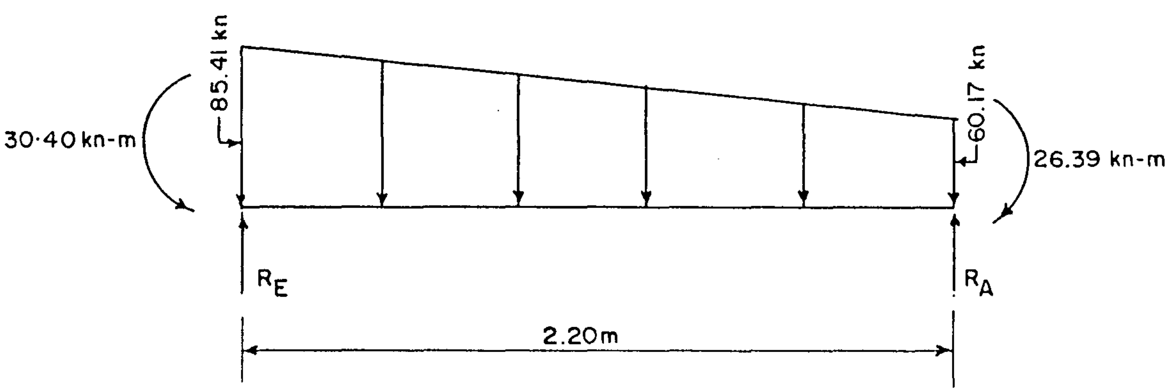
$$= 13.71 \text{ Kn-m (top tension)}$$



$$R_F = R_G = \frac{93.57 \times 1.92}{2} = 89.83 \text{ Kn}$$

$$+ M_{max} = \frac{93.57 \times 1.92^2}{8} - 28.55 = 14.57 \text{ Kn-m (top tension)}$$

c) **Abutment**



$$60.17 \times \frac{2.20^2}{2} + 0.5 \times 25.24 \times \frac{2.20^2}{3} - 4.01 = R_A \times 2.20$$

$R_A = 73.62$ Kn, $R_B = \{(60.17 + 85.41)/2\} \times 2.20 - 73.62 = 86.52$ Kn

Location of zero shear from right support is 'x'

$$60.17\, x + 0.5\, (x \cdot \frac{25.24}{2.20})\, x = 73.62$$

or $5.73\, x^2 + 60.17x - 73.62 = 0$

By trial $x = 1.11$ m

$$+ M_{max} = 26.39 + 60.17 \times \frac{1.11^2}{2} + \frac{1}{2} \times \frac{25.24}{2.20} \times \frac{1.11^3}{3} - 73.62 \times 1.11$$

$= - 15.65$ (Exposed face tension)

Table 9 shows calculation for checking thickness and reinforcement for R.C.C. members.

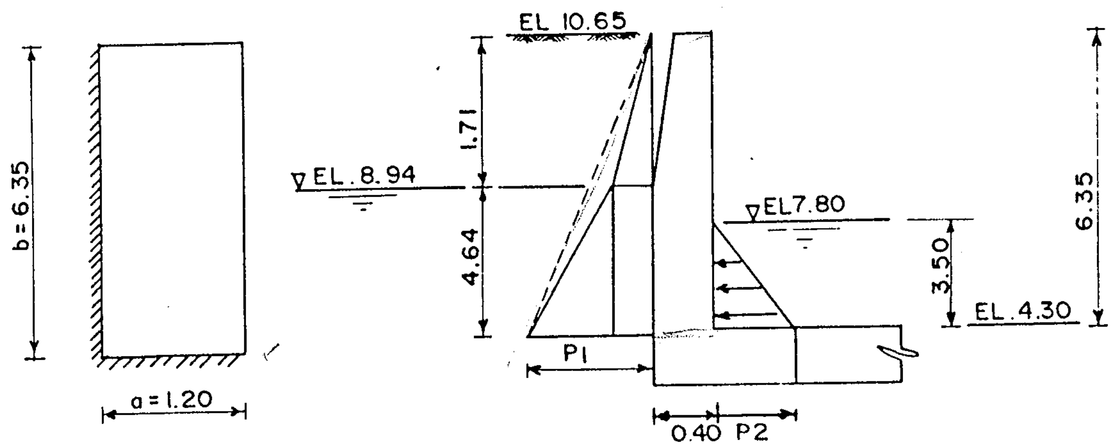
Table 9. Checking of Thickness and Reinforcement Calculation

Items	Top Slab		Bottom Slab		Side slab		Pier
	Support	Centre	Support	Centre	Support	Centre	
Moment (Kn-m)	26.39	12.77	30.40	14.57	30.40	15.65	0.25
Shear (Kn)	79.82	0	90.86	0	86.52	0	-
d provided (mm)	211		361		311		-
d shear $= \frac{V}{0.377}$ (mm)	212		241		230		-
d mom. $= \left[\frac{M\text{x}1000}{1.1}\right]^{\frac{1}{2}}$ (mm)	155		166.72		166		-
As $= \frac{9154M}{d}$ (mm^2/m)	1145	554	791	370	895	461	-
Spacing (mm c/c.)	16mmΦ @ 175	12mmΦ @ 204	16mmΦ @ 260	16mmΦ @ 543	16mmΦ @ 225	16mmΦ @ 436	-
Spacing using min. reinf. (mm c/c)	12 mmΦ @ 250	12 mmΦ @ 180	12 mmΦ @ 200	16 mmΦ @ 200	12 mmΦ @ 200	16 mmΦ @ 200	16 mmΦ @ 250
Use spacing (mm c/c)	16mmΦ @ 175	12mmΦ @ 180	16mmΦ @ 250	16mmΦ @ 200	16mmΦ @ 200	16 mmΦ @ 200	16 mmΦ @ 250

4.1.5 Extended part of Conduit (c/s)

i) Abutment

<u>Design as wall fixed at two ends</u>



To simplify the design neglect water pressure in the bay for horizontal bending and assume triangular pressure diagram (dotted line) in the earth face. As this simplification is on the conservative side, it will not affect the safety of the structure.

$P_1 = Ca\ \gamma\ moist \times h_1 + (Ca\ \gamma_{sub} + \gamma\ w)h_2$

$= 0.49 \times 17.3 \times 1.71 + (0.49 \times 9.10 + 9.80) \times 4.64$

$= 14.49 + 66.16 = 80.65\ Kn/m^2$

$P_2 = \gamma\ wh_3 = 9.8 \times 3.50 = 34.30\ Kn/m^2$

$a/b = 1.20/6.35 = 0.19 = 1/5.29$

using $a/b = \frac{1}{4}$ from plate 3, Max. Coefficients are:

$Cx = 0.0150, \quad Cy = 0.0221$

$a/b = 1.20/3.50 = 0.34 = 1/2.91$

using $a/b = 3/8$, $Cx = 0.0246$, $Cy = 0.0354$

$M = Cpb^2$

for $P_1 = 82.27\ Kn/m^2$,

$Mx = 0.0150 \times 80.65 \times 6.35^2 = 48.78$ Kn-m

$My = 0.0221 \times 80.65 \times 6.35^2 = 71.87$ Kn-m

for $P_2 = 34.30\ Kn/m^2$,

$Mx = 0.0246 \times 34.30 \times 3.50^2 = 10.34$ Kn-m

$My = 0.0354 \times 34.30 \times 3.50^2 = 14.87$ Kn-m

Net vertical B.M = 71.87 - 14.87 = 57.00 Kn-m

$$d_{moment} = \left[\frac{57.00 \times 1000}{1.1}\right]^{\frac{1}{2}} = 227.63 \text{ mm}$$

$d_{provided} = 400 - (80+8) = 312.00$ mm

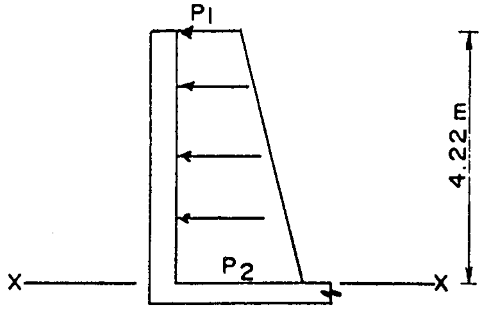
$$As = \frac{9154 \times 57.00}{312} = 1672 \text{ mm}^2/\text{m}$$

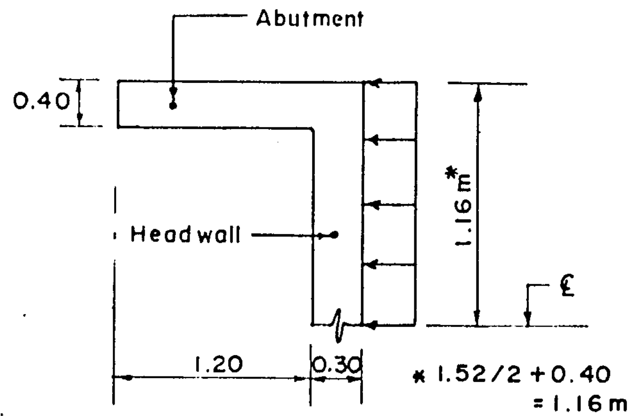
use 16 mm ϕ @ 100 mm c/c (vertically)

$$As = \frac{9154 \times 48.78}{312.0} = 1431 \text{ mm}^2/\text{m}$$

use 16 mm ϕ @ 130 mm c/c (horizontally)

Design as cantilever beam against earth pressure behind headwall





Depth of LL surcharge = 0.90m

P_1 = 0.49 x 17.3 x 0.90 = 7.63 Kn/m^2

P_2 = 7.63 + 0.49 x 17.30 x 4.22 = 43.40 Kn/m^2

P_1 - P_2 = 35.77 Kn/m^2

Vxx = 0.5 (7.63 + 43.40) x 4.22 = 107.67 Kn/m

= 107.67 x 1.16 = 124.90 Kn

Mxx = 7.63 x 4.22^2/2 + 0.5 x 35.77 x 4.22 x 1/3 x 4.22 = 174.10 Kn/m

= 174.10 x 1.16 = 201.96 Kn-m

$$d_{shear} = \frac{124.90 \times 1000}{0.377 \times 400} = 828 \text{ mm}$$

$$d_{moment} = \left[\frac{201.96 \times 10^6}{1.1 \times 400} \right]^{\frac{1}{2}} = 677 \text{ mm}$$

$d_{provided}$ = 1500 - 150 = 1350 mm, O.K.

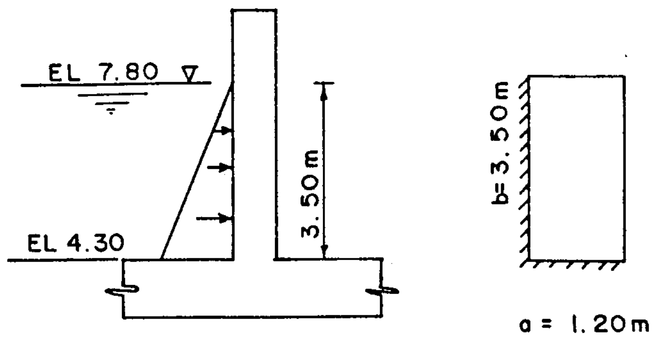
$$As = \frac{9154 \times 201.96}{1350} = 1369 \text{ mm}^2$$

use 4 nos - 19 mm ϕ

ii) **Pier**

<u>Design as wall fixed at two ends</u>

Consider water level at El. 7.80 m on one side of pier with other side dry.



P_1 = 9.80 x 3.50 = 34.3 Kn/m^2

a/b= 1.20/3.50 = 0.34 = 1/2.92

using a/b = 3/8, Coefficients are:

Cx = 0.0246, Cy = 0.0354

Mx = 0.0246 x 34.3 x 3.5^2 = 10.34 Kn-m

My = 0.0354 x 34.3 x 3.5^2 = 14.87 Kn-m

$$d_{moment} = \left[\frac{14.87 \times 1000}{1.1}\right]^{\frac{1}{2}} = 116.27 \text{ mm}$$

$d_{provided}$ = 400 -(80 + 8) = 312 mm, O.K.

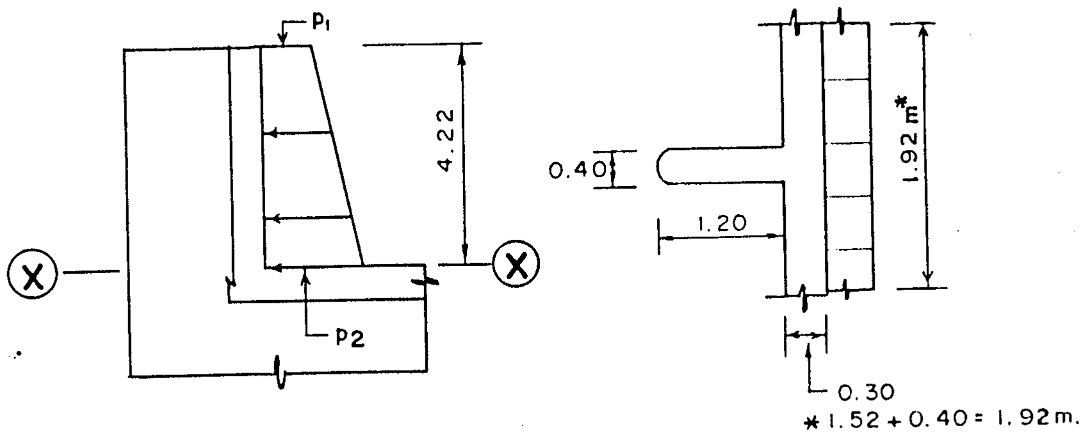
$$As = \frac{9154 \times 14.87}{312} = 436 \text{ mm}^2/\text{m}$$

use 12 mm ϕ @ 250 mm c/c vertically on both side.

$$As = \frac{9154 \times 10.34}{436} = 217 \ mm^2/m$$

use 12 mm ϕ @ 300 mm c/c horizontally on both sides

Design as cantilever beam



V_{xx} = 107.67 x 1.92 = 206.73 Kn

M_{xx} = 174.10 x 1.92 = 334.27 Kn-m

$$d_{shear} = \frac{206.73 \times 1000}{0.377 \times 400} = 1371 \ mm$$

$$d_{moment} = \left[\frac{334.27 \times 10^6}{1.1 \times 400} \right]^{\frac{1}{2}} = 871.61 \ mm$$

$d_{provided}$ = 1500 - 150 = 1350 mm say O.K.

$$As = \frac{.9154 \times 334.27}{1350} = 2267 \ mm^2$$

use 8 - 19 mm ϕ bar

iii) **Base slab**

<u>Load through abutment</u>

Abutment = 0.4 x 2.13 x 1.20 x 23.6 + 0.5(0.3+0.4) x 4.22 x 1.20 x 23.6

= 65.96 Kn

Soil = 0.5 x 4.22 x 0.10 x 1.20 x 17.30 = 4.38 Kn

Deck = 0.76 x 1.20 x 0.20 x 23.6 = 4.30 Kn

Gates & accessories = 20/4 (@ 20 Kn) = 5.00 Kn

79.64 Kn/1.20 = 66.37 Kn/m

<u>Load through Pier</u>

Pier = 0.4 x 6.35 x 1.20 x 23.6 = 71.93 Kn

Deck = 1.52 x 0.20 x 1.20 x 23.6 = 8.61 Kn

Gates & Accessories = 20/2 (@ 20 Kn) = 10.00 Kn

90.54 /1.20 = 75.45 Kn/m

$$\text{Base pressure} = \frac{2 \times 66.37 + 2 \times 75.45}{5.76} = 49.24 \ Kn/m$$

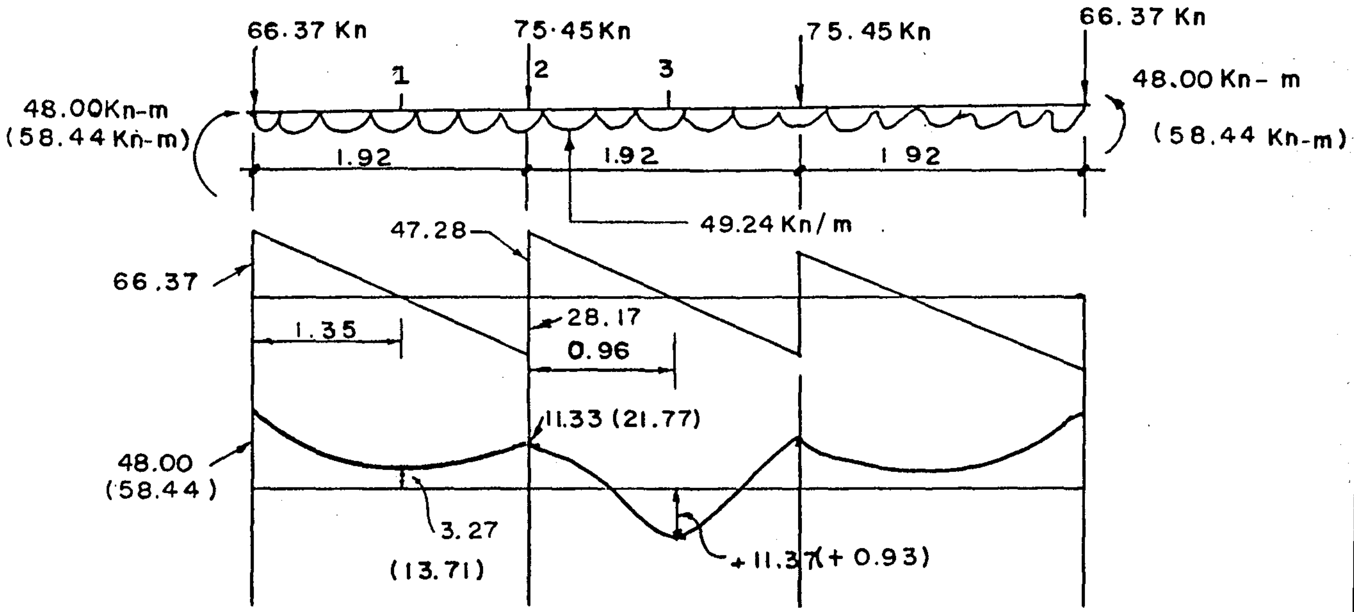
End Moment

After construction condition:

$0.0221 \times 0.49 \times 17.3 \times 6.35 \times 6.35^2 = 48$ Kn-m

During operation condition : 58.44 Kn-m (Refer section 4.1.5.i)

Shear & Moment



$M_1 = 66.37 \times 1.35 - 49.24 \times 1.35^2/2 - 48.00\ (58.44)$

$= 89.60 - 44.87 - 48.00\ (58.44)$

$= -\ 3.27\ (-13.71)$

$M_2 = 66.37 \times 1.92 - 49.24 \times 1.92^2/2 - 48.00\ (58.44)$

$= 127.43 - 90.76 - 48(58.44)$

$= -\ 11.33\ (-21.77)$

$M_3 = 66.37 \times 2.88 + 75.45 \times 0.96 - 49.24 \times 2.88^2/2 - 48.00(58.44)$

$= 191.15 + 72.43 - 204.21 - 48(58.44)$

$= + 11.37\ (+0.93)$

Max. top tension moment = 11.37 Kn-m

Max. bottom tension moment = 58.44 Kn-m

Max. shear = 66.37 Kn

4.2 Stilling basin/Transition Part

Design procedure for wingwall and apron of R/s stilling basin, as shown in next page, is presented below:

(i) Wingwall

The wingwall has been designed for two heights i.e 5.60 m and 5.00 m at the start and at the end of glacis respectively.

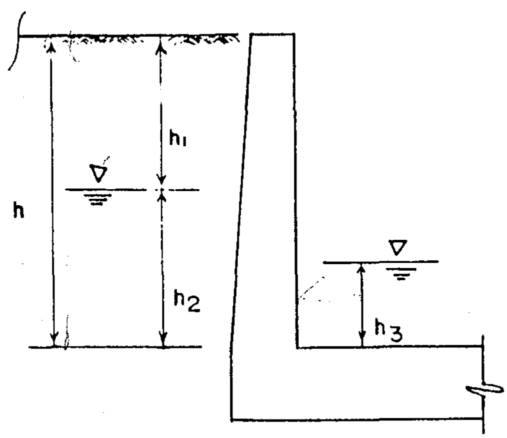
Section 1-1

$h = 5.60$ m

h_1 = El 9.90 - El 5.62 = 4.28 m

h_2 = El 5.62 - El 4.30 = 1.32 m

h_3 = El 5.20 - El 4.30 = 0.90 m

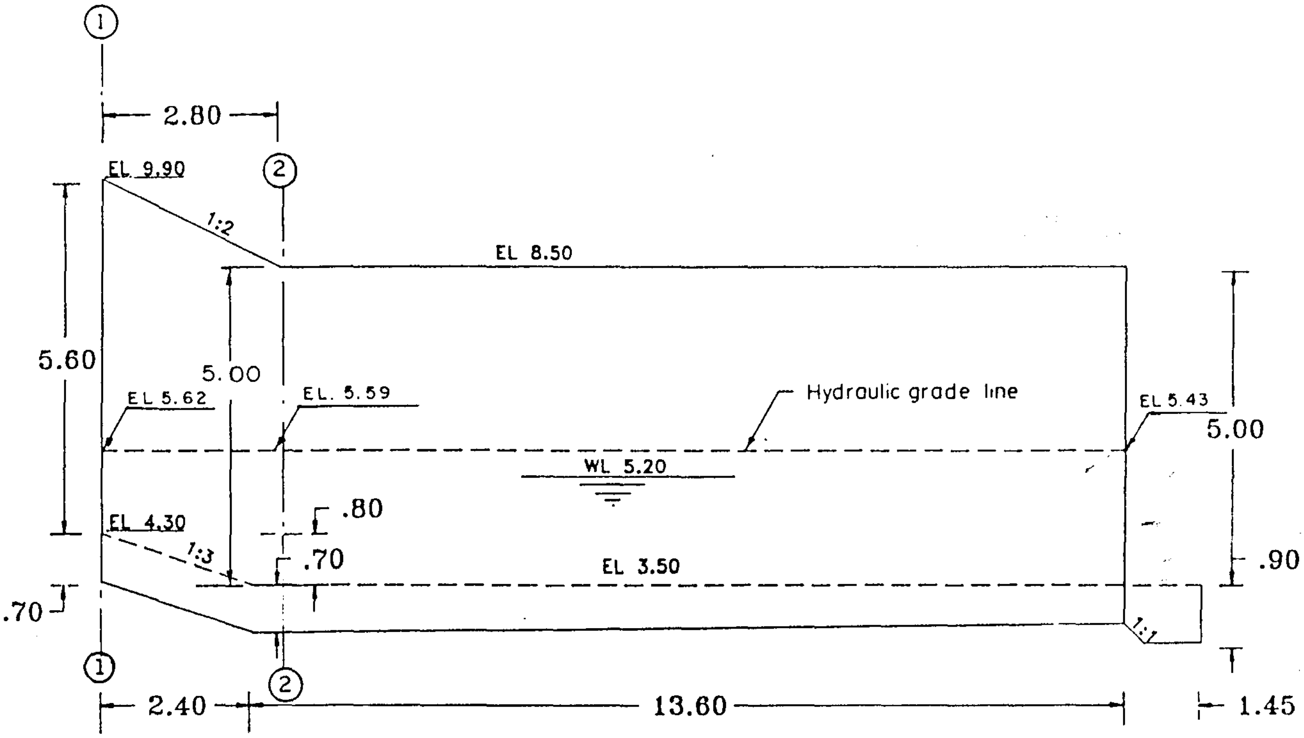


After construction

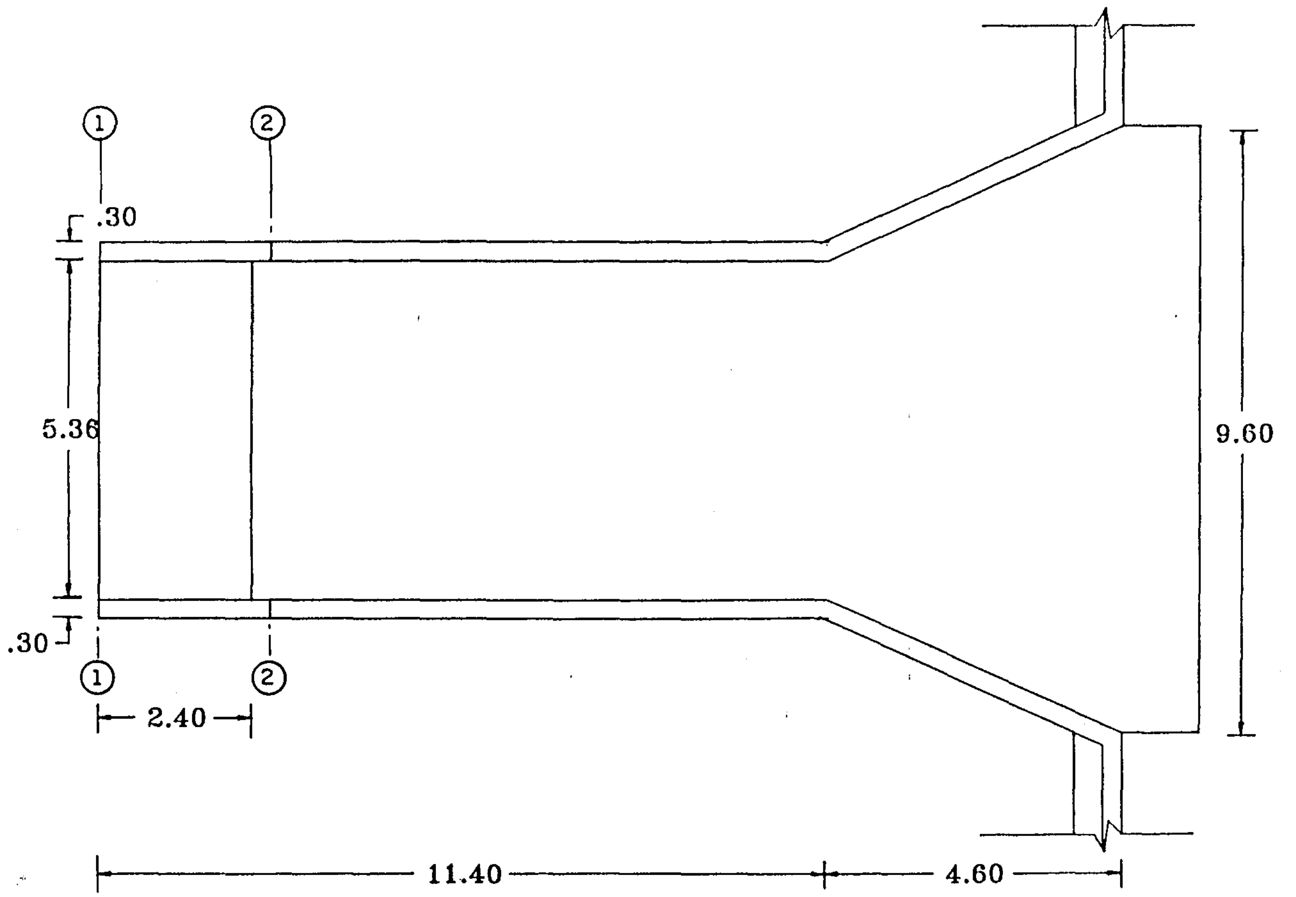
$P = 2.85\ h^2$

$= 2.85 \times 5.60^2$

$= 89.38$ Kn



R/S WING WALL



R/S APRON

$M = 0.95 h^3$

$= 0.95 \times 5.60^3$

$= 166.83$ Kn-m

<u>During operation</u>

$P = P_1 + P_2 + P_3 - P_4$

$= 2.85\ h_1^2 + 5.71\ h_1 h_2 + 6.40\ h_2^2 - \frac{1}{2} \times 9.8 \times h_3^2$

$= 2.85 \times 4.28^2 + 5.71 \times 4.28 \times 1.32 + 6.40 \times 1.32^2 - 4.9 \times 0.9^2$

$= 52.21 + 32.26 + 11.15 - 3.97 = 91.65$ Kn

$M = P_1 \times (h_2 + 1/3\ h_1) + P_2 \times h_2/2 + P_3 h_2/3 - P_4 h_3/3$

$= 52.21 \times (1.32 + 1/3 \times 4.28) + 32.26 \times 1.32/2$

$+ 11.15 \times 1.32/3 - 3.97 \times 0.90/3$

$= 143.40 + 21.29 + 4.91 - 1.19 = 168.41$ Kn-m

Design shear = 91.65 Kn

Design Moment = 168.41 Kn-m

$d_{shear} = 91.65/0.377 = 243.10$ mm

$$d_{moment} = \left[\frac{168.41 \times 1000}{1.1}\right]^{\frac{1}{2}} = 391.28 \text{ mm}$$

$d_{provided} = 500 - (80 + 12.5) = 407.50$ mm

$$As = \frac{9154 \times 168.41}{407.50} = 3783 \text{ mm}^2/\text{m}$$

use 22 mm ϕ @ 100 mm c/c.

Section 2-2

$h = 5.00$ m

h_1 = El 8.50 - El 5.59 = 2.91 m

h_2 = El 5.59 - El 3.50 = 2.09 m

h_3 = El 5.20 - El 3.50 = 1.70 m

After construction

$P = 2.85\ h^2 = 2.85 \times 5^2 = 71.25$ Kn

$M = 0.95\ h^3 = 0.95 \times 5.00^3 = 118.75$ Kn-m

During operation

$P = 2.85 \times 2.91^2 + 5.71 \times 2.91 \times 2.09 + 6.40 \times 2.09^2 - 4.90 \times 1.70^2$

$= 24.13 + 34.73 + 27.95 - 14.16$

$= 72.65$ Kn

$M = 24.13 \times (2.09 + 1/3 \times 2.91) + 34.73 \times 2.09/2 + 27.95 \times 2.09/3 -$
$14.16 \times 1.70/3$

$= 73.84 + 36.29 + 19.47 - 8.02 = 121.58$ Kn-m

Design shear = 72.65 kn, Design Moment = 121.58 Kn-m

d_{shear} = 72.65/0.377 = 192.70 mm

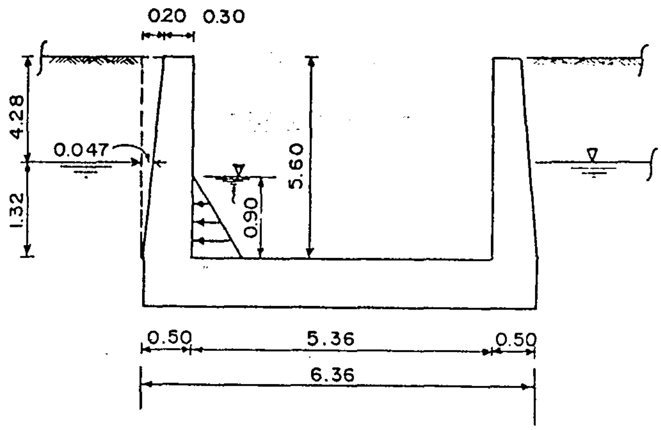
$$d_{moment} = \left[\frac{121.58 \times 1000}{1.1}\right]^{\frac{1}{2}} = 332.46 \text{ mm}$$

$d_{provided}$ = 500 - 92.5 = 407.5 mm

$$As = \frac{9154 \times 121.58}{407.50} = 2731 \text{ mm}^2/\text{m}$$

use 19 mm ϕ @ 100 mm c/c.

(ii) Apron or Floor



Section 1-1

$B = 6.36$ m, $B/2 - t_2 = 2.68$ m

Loading After Construction

$WW = 0.5\ (0.30 + 0.50) \times 5.60 \times 23.60 = 52.86$ Kn

$S = 0.5 \times 0.20 \times 5.60 \times 17.30 = 9.69$ Kn

$WW + S = 62.55$ Kn

$M_{end} = 0.95 \times 5.60^3 = 166.83$ Kn-m

$M_{centre} = 0.5\ (WW+S)(B/2 - t_2) - M_{end}$

$= 0.5 \times 62.55 \times 2.68 - 166.83$

$= 83.82 - 166.83 = -83.01$ Kn-m (bottom tension)

Loading During Operation

$WW = 52.86$ Kn,

$S = 0.5(0.20 + 0.047) \times 4.28 \times 17.30 + 0.5 \times 0.047 \times 1.32 \times 18.90$

$= 9.14 + 0.59 = 9.73$ Kn

$WW + S = 62.59$ Kn, $W = 0.5 \times 5.36 \times 0.90 \times 9.80 = 23.64$ Kn

$M_{end} = 168.41$ Kn-m (From page 7-90)

$M_{centre} = 0.5\ (WW+S)(B/2 - t_2) - \frac{1}{2}Wt_2 - M_{end}$

$= 0.5 \times 62.59 \times 2.68 - \frac{1}{2} \times 23.64 \times 0.5 - 168.41$

$= -90.45$ Kn-m (bottom tension)

Design Moment

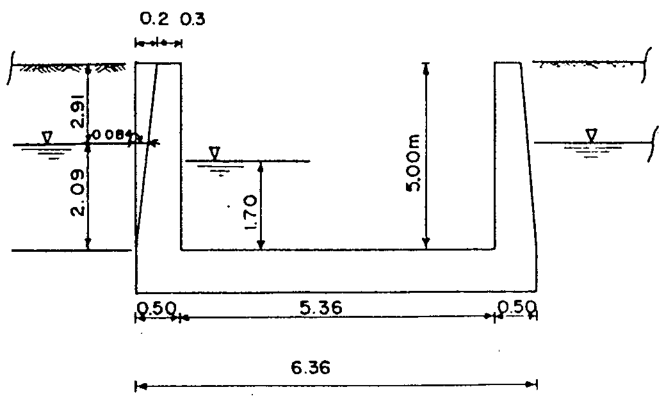
At end : 168.41 Kn-m
At centre: 90.45 Kn-m
Bottom tension

$d = 700 - (80 + 10) = 610.0$ mm

$$As = \frac{9154 \times 168.41}{610.00} = 2527 \text{ mm}^2/\text{m}$$

use 19 mm ϕ @ 120 mm c/c although

Section 2-2



$B = 6.36$ m, $B/2 - t_2 = 2.68$ m

Loading After Construction

$WW = 0.5\ (0.3 + 0.5) \times 5.0 \times 23.60 = 47.2$ Kn

$S = 0.5 \times 0.20 \times 5.00 \times 17.30 = 8.65$ Kn

$WW + S = 55.85$ Kn

$M_{end} = 0.95 \times 5.00^3 = 118.75$ Kn-m

$M_{centre} = 0.5 \times 55.85 \times 2.68 - 118.75 = - 43.91$ kn-m (bottom tension)

Loading During Operation

$WW = 47.2$ Kn

$S = 0.5(0.2 + 0.084) \times 2.91 \times 17.30 + 0.5 \times 0.084 \times 2.09 \times 18.90$

$= 8.81$ Kn

WW + S = 56.01 Kn

W = 0.5 x 5.36 x 1.70 x 9.80 = 44.65 Kn

M_{end} = 121.58 Kn-m (From page 7-91)

M_{centre} = 0.5 x 56.01 x 2.68 - 0.5 x 44.65 x 0.5 - 121.58

= - 57.69 Kn-m (bottom tension)

Design Moment

$$\left.\begin{array}{l}\text{At end : 121.58 Kn-m} \\ \text{At centre: 57.69 Kn-m}\end{array}\right]\ \text{Bottom tension}$$

d = 610.00 mm

$$As = \frac{9154 \times 121.58}{610.00} = 1824\ mm^2/m$$

use 19 mm ϕ @ 170 mm c/c

4.3 **Returnwall (Refer to figure in stability analysis)**

(i) Stem

h = 5.00 m (R/S)

During Construction

P = 2.85 h^2 = 2.85 x 5.0^2 = 71.25 Kn

M = 0.95 h^3 = 0.95 x 5.00^3 = 118.75 Kn-m

During Operation

$h_1 = 3.07$ m, $h_2 = 1.93$ m, $h_3 = 1.70$ m (Refer fig. in page 7-89)

$$P = 2.85\ h_1^2 + 5.71\ h_1 h_2 + 6.40\ h_2^2 - 0.5 \times 9.80 \times h_3^2$$

$$= 2.85 \times 3.07^2 + 5.71 \times 3.07 \times 1.93 + 6.40 \times 1.93^2 - 0.5 \times 9.8 \times 1.7^2$$

$$= 26.86 + 33.83 + 23.84 - 14.16 = 70.37 \text{ kn}$$

$$M = 26.86\ (1.93 + 1/3 \times 3.07) + 33.83 \times 1.93/2 + 23.84 \times 1.93/3 - 14.16 \times 1.70/3$$

$$= 119.29 \text{ Kn-m}$$

Design shear = 71.25 Kn

Design Moment = 119.29 Kn-m

$$d_{shear} = 71.25/0.377 = 189.00 \text{ mm}$$

$$d_{moment} = \left[\frac{119.29 \times 1000}{1.1}\right]^{\frac{1}{2}} = 329.31 \text{ mm}$$

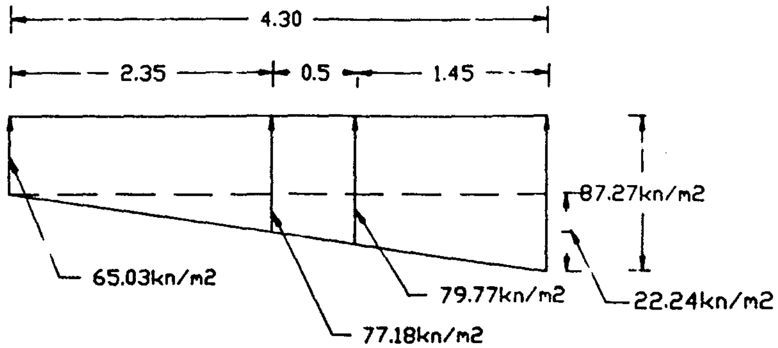
$d_{provided} = 500 - (80 + 10) = 410.00$ mm, O.K.

$$As = \frac{9154 \times 119.29}{410.00} = 2663 \text{ mm}^2/\text{m}$$

use 19 mm ϕ @ 100 mm c/c

(ii) **Base**

<u>During Construction (Refer page 7-62 & 7-63)</u>



Toe:

V = 0.5(79.77 + 87.27)x 1.45 - 0.60 x 1.45 x 23.6 - 0.3 x 1.05 x 23.6

= 121.10 - 20.53 - 7.43

= 121.10 - 27.96 = 93.14 Kn

M = 79.77 x $1.45^2/2$ + $\frac{1}{2}$ x 1.45 x 7.5 x 2/3 x 1.45 - 0.60 x 23.6 x $1.45^2/2$ - 0.3 x 0.90 x 23.6 x 1.0 - $\frac{1}{2}$ x 0.3 x 0.3 x 23.6 x 0.45

= 83.86 + 5.26 - 14.88 - 6.37 - 0.48

= 89.12 - 21.73 = 67.39 Kn-m

heel:

V = 0.5 (65.03 + 77.18) x 2.35 - 0.60 x 2.35 x 23.6 - 5.00 x 2.35 x 17.3

= 167.10 - 33.28 - 203.28

= 167.10 - 236.56 = - 69.46 Kn

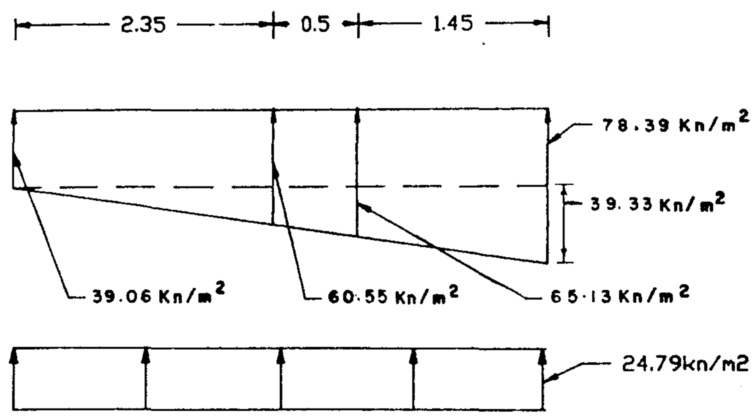
$M = 65.03 \times 2.35^2/2 + 0.5 \times 12.15 \times 2.35 \times 1/3 \times 2.35$

$- 0.60 \times 23.60 \times 2.35^2/2 - 5 \times 17.3 \times 2.35^2/2$

$= 179.56 + 11.18 - 39.10 - 238.85$

$= - 87.21$ kn-m

<u>During Operation (Refer page 7-64 & 7-65)</u>



Toe :

$V = 0.5(65.13 + 78.39) \times 1.45 + 24.79 \times 1.45 - 20.53 - 7.43 - 1.70 \times 1.45 \times 9.80$

$= 104.05 + 35.94 - 27.96 - 24.16 = 87.87$ kn

$M = (65.13 + 24.79) \times 1.45^2/2 + \frac{1}{2} \times 13.26 \times 1.45 \times 2/3 \times 1.45 - 21.73$

$- 1.70 \times 9.80 \times 1.45^2/2$

$= 94.53 + 9.29 - 21.73 - 17.51 = 64.58$ Kn-m

Heel:

$V = 0.5(39.06 + 60.55) \times 2.35 + 24.79 \times 2.35 - 0.6 \times 2.35 \times 23.6 - 3.07 \times 17.30$

$\times 2.35 - 1.93 \times 18.90 \times 2.35$

$= 117.04 + 58.26 - 33.28 - 124.81 - 85.72$

$= - 68.51$ kn

$M = (39.06 + 24.79) \times 2.35^2/2 + \frac{1}{2} \times 21.49 \times 2.35 \times 1/3 \times 2.35 - (3.07$

$\times 17.3 + 1.93 \times 18.90 + 0.6 \times 23.60) \times 2.35^2/2$

$= 176.30 + 19.78 - 286.48 = - 90.40$ Kn-m

Design Moment & Shear

Maximum Shear = 93.14 Kn

Maximum Toe moment = 67.39 Kn-m

Maximum Heel moment= 90.40 Kn-m

$d_{shear} = 93.14/0.377 = 247$ mm

$$d_{moment} = \left[\frac{90.40 \times 1000}{1.1}\right]^{\frac{1}{2}} = 286.67 \text{ mm}$$

$d_{provided} = 600 - (80 + 10) = 510.00$ mm

$$As\ (toe) = \frac{9154 \times 67.39}{510.00} = 1210 \text{ mm}^2/\text{m}$$

use 19 mm ϕ @ 250 mm c/c

$$As\ (heel) = \frac{9154 \times 90.40}{510.00} = 1622 \text{ mm}^2/\text{m}$$

use 19 mm ϕ @ 190 mm c/c

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correction made on 27/4/93
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CHAPTER EIGHT

DEWATERING OF CONSTRUCTION SITES

Mofazzal Ahmed

LEARNING OBJECTIVE:

1. After completion of this topic we shall have a clear understanding of

   a) the physical meaning of dewatering and its objectives,

   b) soil and aquifer characteristics that influence dewatering,

   c) potential sources of seepage that might affect the volume of dewatering,

   d) basic principles of groundwater flow that can be applied to dewatering design,

   e) different dewatering methods and their merits & demerits.

   f) the procedure of calculating dewatering load and selecting dewatering method for a particular site,

   g) principles and step by step procedures of dewatering system design,

   h) principles of pump design and pump selection,

   i) potential hazards of dewatering and precautionary measures, and

   j) a complete case study.

2. After completion of this topic we shall be able to

   a) define objectives for a dewatering system,

   b) identify and analyze soil and aquifer parameters and potential seepage source that will influence the dewatering design,

   c) analyze and apply related principles of ground water flow,

   d) analyze available methods of dewatering and assess their suitability for a particular site,

   e) select a particular dewatering method and design the system,

   f) design and select pumps, and,

   g) plan and design disposal system.

DEWATERING OF CONSTRUCTION SITES

DEWATERING OF CONSTRUCTION SITES

1.0.0 INTRODUCTION

The word dewatering means artificial removal of water from a construction site or lowering of ground water table to a desired level. The extent of such dewatering depends on the type of structure and characteristics of underlying formation. This is very important for civil engineering structures, especially for hydraulic structures, where the bottoms of foundations are required to be at levels below ground water table. Besides, in many cases it may happen that natural water bodies like lakes, ponds, canals, rivers, sea or swampy areas exist very close to the construction site. As the soil is a porous media, sometimes with significantly high permeability, it permits smooth flow of water from the source to the excavated area. Such flow of water to the excavated area creates lot of problems during construction. As it is very difficult to carry out excavation and construction of structure in water or in slashes formed of saturated soil, the site must be kept clean & dry and the flow of water to the excavated site must be stopped. This can be accomplished by sump pumping or pre-drainage methods that include wells & well points. However,design and installation of such dewatering methods are not so easy. For this a thorough knowledge of soil-aquifer subsystem, ground water hydraulics and familiarity with different dewatering methods are required. The following sections deal with different aspects of dewatering system.

2.0.0 OBJECTIVES OF DEWATERING

Basic objectives of dewatering a construction site are as follows:

a) Intercept seepage and lower ground water table.

b) Improve stability of slopes, prevent sloughing or slope failure and provide a neat & tidy working site.

c) Improve density and compaction characteristics of foundation soil.

d) Reduce water content of borrow pit soils so that they can be used as compacted fill without much waiting for drying.

e) Prevent bottoms of excavation from heaving or becoming 'quick' under the influence of ground water flow.

f) Reduce earth pressure on temporary supports and sheeting.

3.0.0 FACTORS INFLUENCING DEWATERING DESIGN

A dewatering system can not be designed unless we have sufficient knowledge about soil & aquifer characteristics, recharge potentials and ground water hydraulics. These topics are very briefly discussed in the following sections.

3.1.0 SOIL CHARACTERISTICS

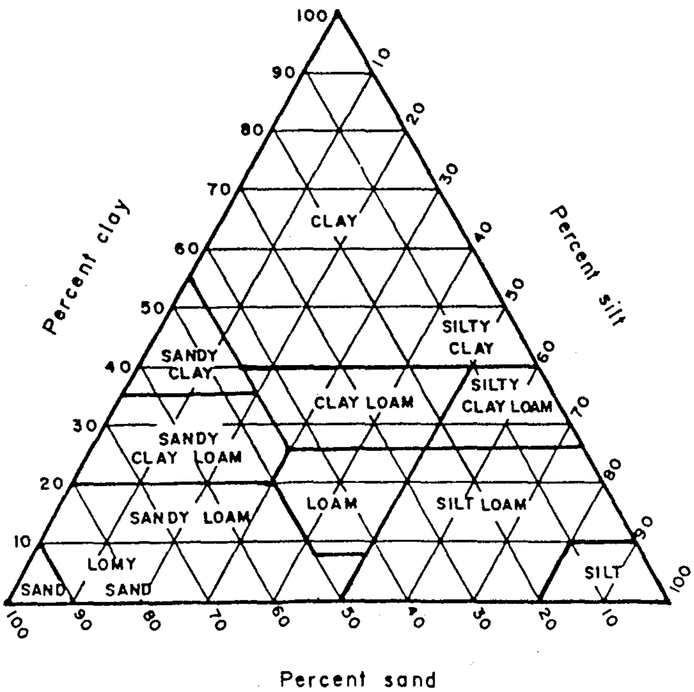
This is a very important factor for analyzing the media properties and estimating the volume of flow to be dewatered. The flow through a porous media is directly related to the soil type, size & distribution of soil grains, porosity & continuity of pore spaces, stratification anisotropy and soil permeability.

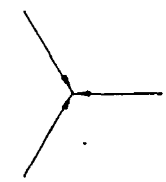
3.1.1 Soil Type

Different soil formations are composed of different types of soils. Even a single formation may composed of different types of soils at different depths and locations. Major types of soils usually encountered are gravel, sand, silt and clay or mixtures of one with the other. Fig. 1 shows a soil classification chart based on grain size. Type of soil greatly influences a dewatering scheme. For example gravel & sand will yield large volume of water and will need gigantic dewatering arrangements, whereas silt and clay will yield very little water, demanding smaller dewatering installations. The textural classification of soil can be evaluated from a soil-textural triangle (Fig. 2) prepared by the U.S. soil survey staff (1951). Interpretation of the diagram is simple. For example if a soil consists of 70% sand, 20% silt and 10% clay, it is a sandy loam.

Classification System	Grain Size (mm): 100 — 10 — 1 — 0.1 — 0.01 — 0.001 — 0.0001
Unified	Cobb. | Gravel | Sand | Fines (silt and clay)
	75 — 4.75 — 0.075
AASHTO	Bould. | Gravel | Sand | Silt | Clay
	75 — 2 — 0.05 — 0.002
MIT	Gravel | Sand | Silt | Clay
	2 — 0.06 — 0.002
ASTM	Gravel | Sand | Silt | Clay
	2 — 0.075 — 0.005
USDA	Cobb. | Gravel | Sand | Silt | Clay
	2 — 0.05 — 0.002

Fig. 1. Soil Classification by Size





KEY

Fig. 2. Soil Texture Triangle
(US Department of Agriculture)

3.1.2 Grain Size Distribution

The particle size or grain size distribution of a soil has major effects on hydraulic properties of the soil. These characteristics are usually expressed in the form of a grain size distribution curve or a gradation curve (Fig. 3), which is obtained through mechanical analysis (sieve analysis) of the soil samples. An oven dried sample of the soil is shaken through a series of standard sieves. The results of such sieve analysis are then plotted on a semi-logarithmic paper with particle sizes in the logarithmic scale and percent passing on the normal scale. Particles finer than 200 mesh sieve are usually classified as silt or clay, the gradation properties of which are determined by hydrometer analysis.

Depending on its particle size distribution a soil sample is usually classified as uniform, well graded or gap graded. Uniform soils contain particles of more or less of same size (Fig.3.a). This kind of soil is highly porous and allows water to move more freely. On the other hand, a well graded soil may be composed of wide range of particle sizes such as gravel, sand, silt and clay (Fig.3.b). Well graded sample are usually dense and compact and do not allow easy movement of water.

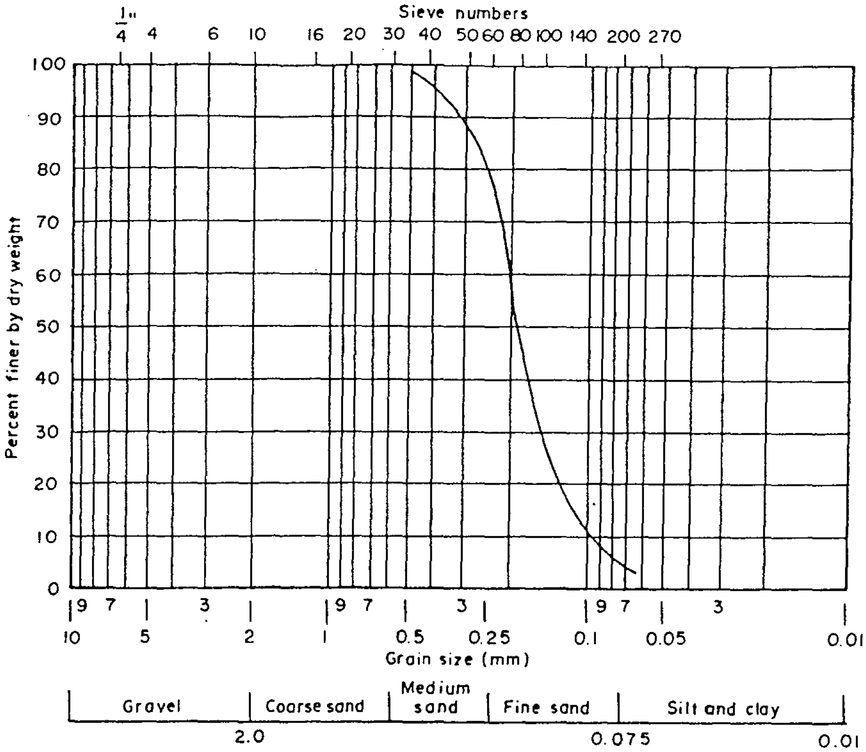
The uniformity of gradation is usually measured by a coefficient, C_u, called uniformity coefficient. It is defined as the ratio of D_{60} size of soil to its D_{10} size:

$$C_u = \frac{D_{60}}{D_{10}} \qquad \text{.......................................(1)}$$

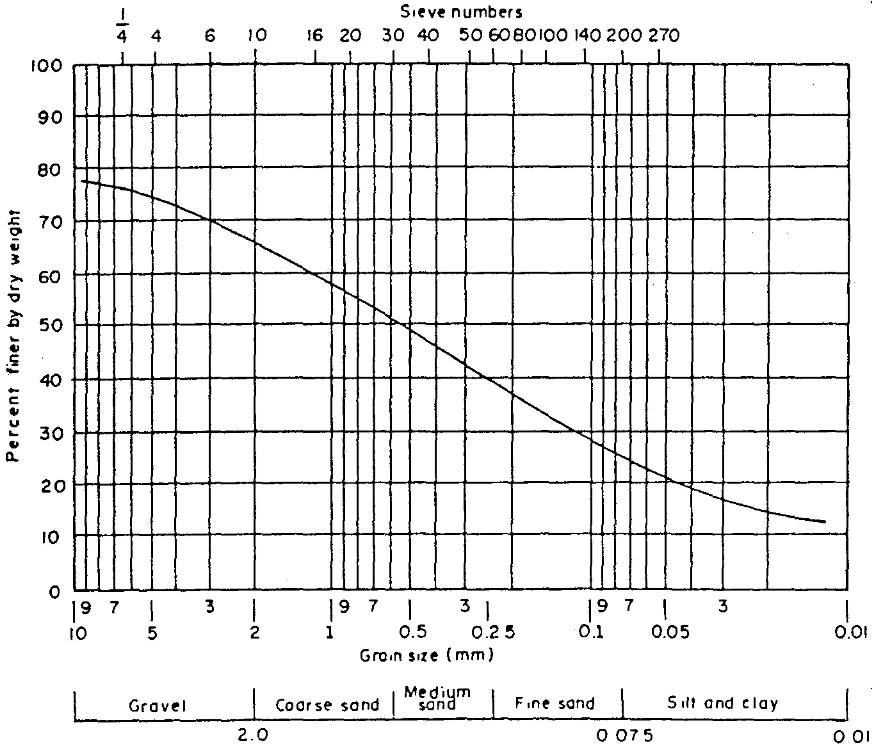
Where,

D_{60} = Particle size which is larger than 60% of the particles i.e 60% of the particles are finer than this size.

D_{10} = Particle size which is larger than 10% of the particles i.e. 10% of the particles are finer than this size.



a) Uniform Soil



b) Well Graded Soil

Fig. 3. Gradation Curve

Minimum value of this coefficient is 1, indicating that all particles are of the same size. A higher uniformity coefficient means a more well graded soil. There are two other important parameters of gradation viz average grain size (D_{50}) and effective grain size (D_{10}). The average grain size gives a feeling of the average condition of the soil. The effective grain size is frequently used to estimate the field permeability of soil.

3.1.3 Porosity, Void Ratio And Water Content

These are very important parameters for dewatering as they help to estimate the permeability of soil that determines dewatering volumes and the compressibility of soil that predicts the probable ground subsidence due to dewatering. Both porosity and void ratio indicate the magnitude of open space in the soil sample. A loose soil sample has higher pore space than a denser one (Fig. 4). Porosity is defined as the ratio of volume of voids (V_v) to volume of solids (V_s).

Water content of a soil is defined as the amount of water contained in the pore spaces. It can be 100% for a highly porous saturated soil sample. This parameter is very important for dewatering of silts and clays because it indicates the volume of storage depletion that might have to be handled even after sufficient decline of the water table has taken place.

High porosity of soil does not always mean higher freedom for water movement. The easiness of water movement actually depends on the size and continuity of the pore spaces.

3.1.4 Stratification And Soil Anisotropy

Anisotropy and stratification of a soil formation are vital considerations for designing a dewatering system. A soil is said to be isotropic when it is homogeneous or has identical properties in all directions. In reality isotropy is an absurd condition as most of the soils are either stratified or have different

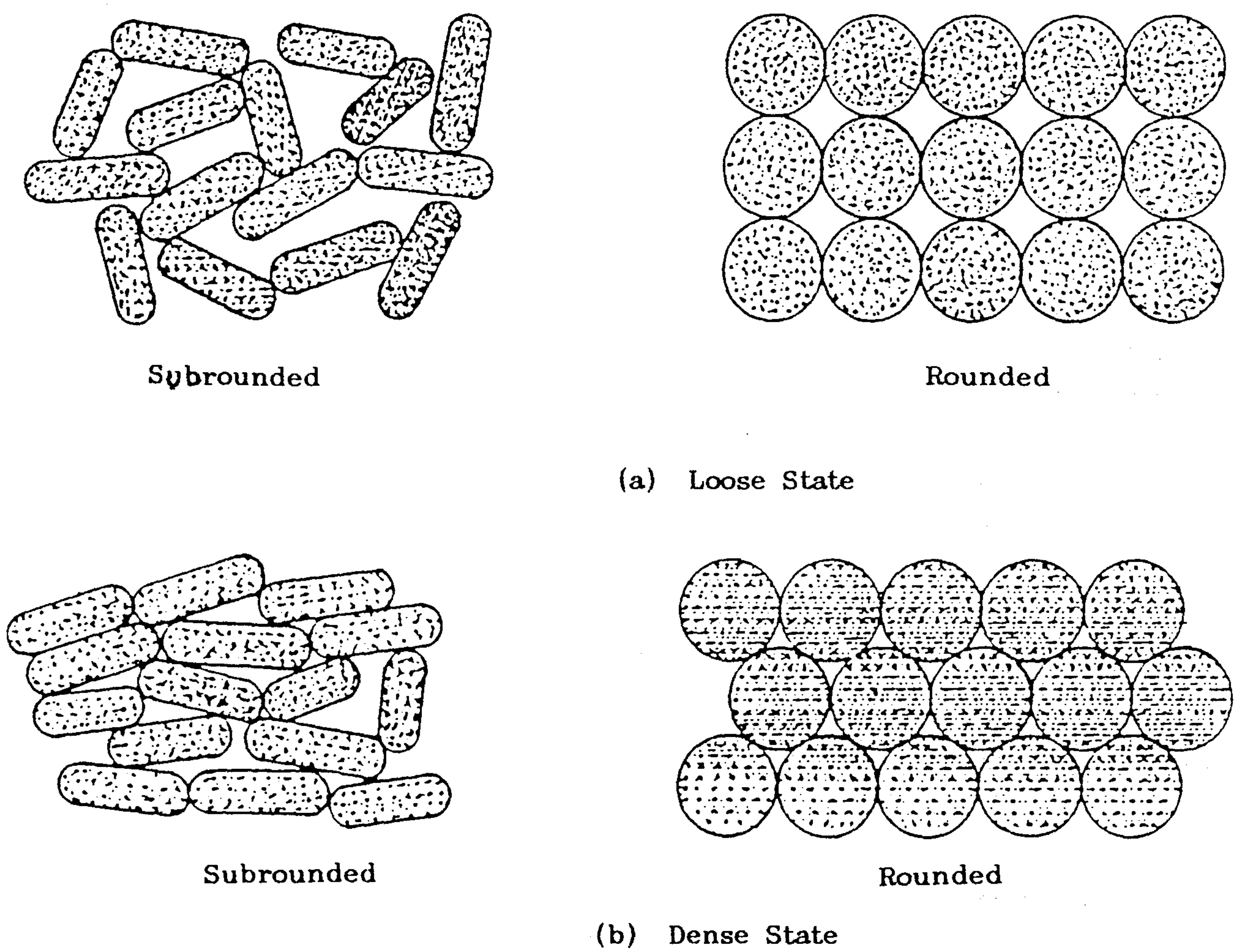


Fig. 4. Pore Spaces in Soil

properties in different direction. A stratified soil means a formation composed of different layers of soil having different constituents. A layered system has much less freedom of water movement in the vertical direction than in the horizontal direction. If the anisotropic properties of a soil in all directions are known, it can be transferred into a theoretically equivalent isotropic medium for practical purposes. Otherwise the soil can be treated as an isotropic medium and approximate results can be obtained on the basis of simplifying assumptions.

3.2.0 AQUIFER CHARACTERISTICS

An aquifer is a saturated bed or formation that yields water in sufficient quantity. Most of the aquifers are large and can be visualized as underground storage reservoirs. Water enters a reservoir from natural or artificial recharge, flows out under the action of gravity or is extracted by pumped wells. The following sections deal with different characteristics of an aquifer.

3.2.1 Aquifer Types

There are three types of aquifers, i) unconfined aquifer, ii) confined aquifer and iii) leaky aquifer.

i) Unconfined aquifer

Unconfined aquifers are usually called water table aquifers where the water table or top of water surface is under atmospheric pressures (Fig. 5). Some times, clay lenses or other impermeable strata above the actual water table intercept the percolating water and form mini reservoirs. Water tables of such shallow water bodies overlying the actual aquifer are called perched water table (Fig. 6). Wells tapping the perched reservoirs yield very small quantities of water.

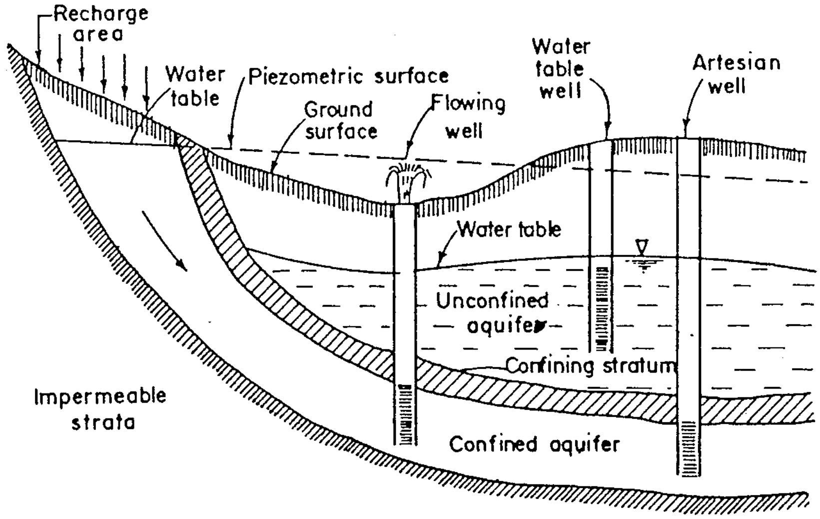


Fig. 5. Confined and Unconfined Aquifer

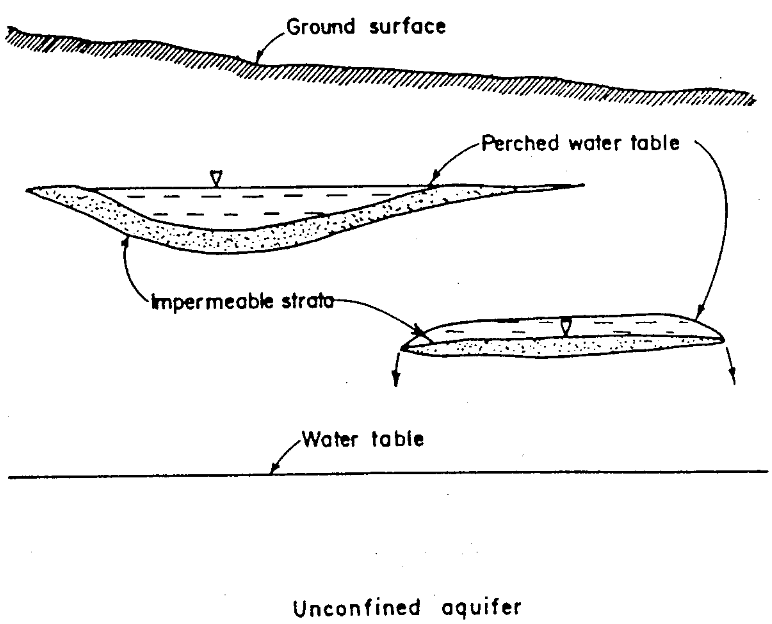


Fig. 6. Perched Aquifer

ii) **Confined aquifer**

Confined aquifers are usually called artesian aquifers or pressure aquifers. These aquifers occur where ground water is confined under a pressure greater than atmospheric pressure by overlying impermeable strata (Fig. 5). Water enters the confined aquifer in an area where the confining bed rises to the surface or is connected to an overlying stream. When wells penetrate such aquifers water level rises above the bottom of confining bed and some times flows out freely. Pressure heads in the confined aquifer or level of water in piezometers punching the confining bed is called piezometric surface.

iii) **Leaky aquifers**

In practical cases completely confined or un-confined aquifers are less frequent. Most of the aquifers are either leaky or semi-confined by an overlying semi-pervious layer (Fig. 7). Pumping from a leaky aquifer removes water both by horizontal flow within the aquifer and vertical flow through the semiconfining layer. However, behaviours of such aquifers are not further elaborated here as their mathematical treatment is very complex. For simplicity all aquifers will be treated either as confined aquifer or as unconfined aquifers.

3.2.2 Location Of Water Table Or Piezometric Surface

Location of ground water table or piezometric surface and its fluctuation with the seasons of the year greatly influence the required lowering of water table/ piezometric surface and hence the volume of dewatering. For confined aquifer the piezometric surface can be established by installing piezometers or wells in the aquifer and recording the water level in the piezometer or well. As water in the confined aquifer is under pressure it will easily flow to a well or piezometer.

For unconfined aquifer, establishing water table is not so easy as water may take several hours to weeks to come to a final position in the well or bore hole. The following methods are usually adopted to ascertain the level of ground water table.

i) For permeable soils like sand and gravel, a bore hole is drilled into the formation. Due to high permeability water readily seeps into the bore hole and the water comes to its final position within matter of moments. The hole is then cleaned with a horizontal jet and the water level is measured by lowering a chalk coated steel tape or some other electronic device. Normal practice is to measure water level 24 hours after completion of the bore hole.

ii) For silty and sandy silty soils ground water may take several days to came to its final position. In such case if the soil permeability is more or less homogeneous, approximate water level can be predicated as follows using Hvorslev method.

a) Water in the bore hole is bailed out upto certain level in the bore hole and then allowed to rise slowly. Water levels are then measured at two or more equal consecutive time intervals (Fig. 8.a).

b) The final water level is then estimated from:

$$H_o = \frac{h_1^2}{h_1-h_2} \quad \text{..............................(2)}$$

$$H_2 = \frac{h_2^2}{h_1-h_2} \quad \text{..............................(3)}$$

$$H_3 = \frac{h_3^2}{h_2-h_3} \quad \text{..............................(4)}$$

Where,

H_o, H_1, H_3, h_1, h_2, h_3
etc. are as shown in Fig. 8.a

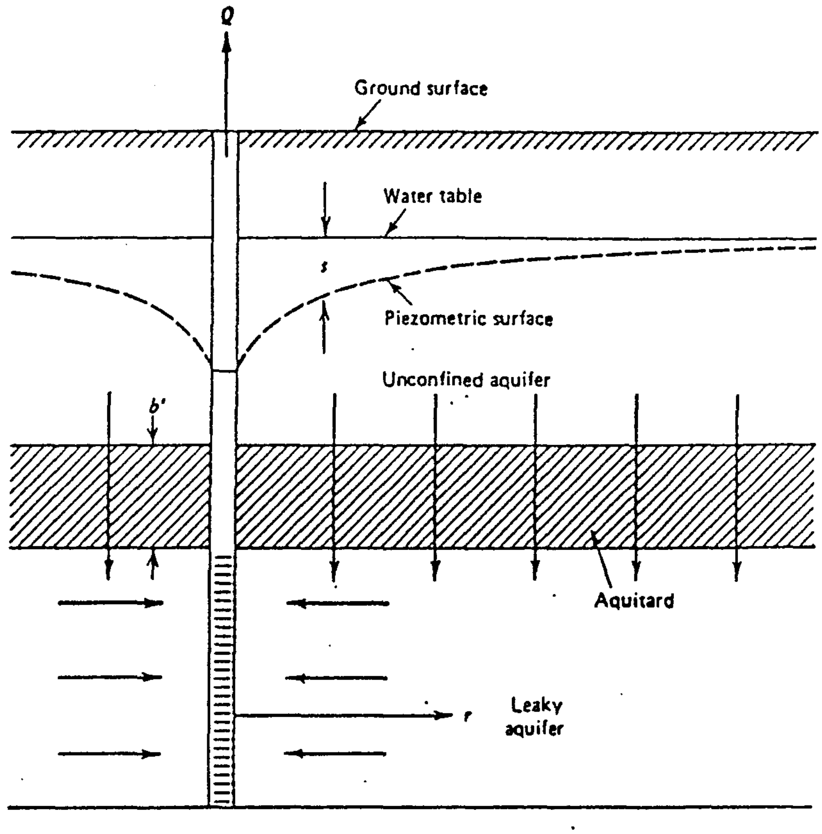


Fig. 7. Leaky Aquifer

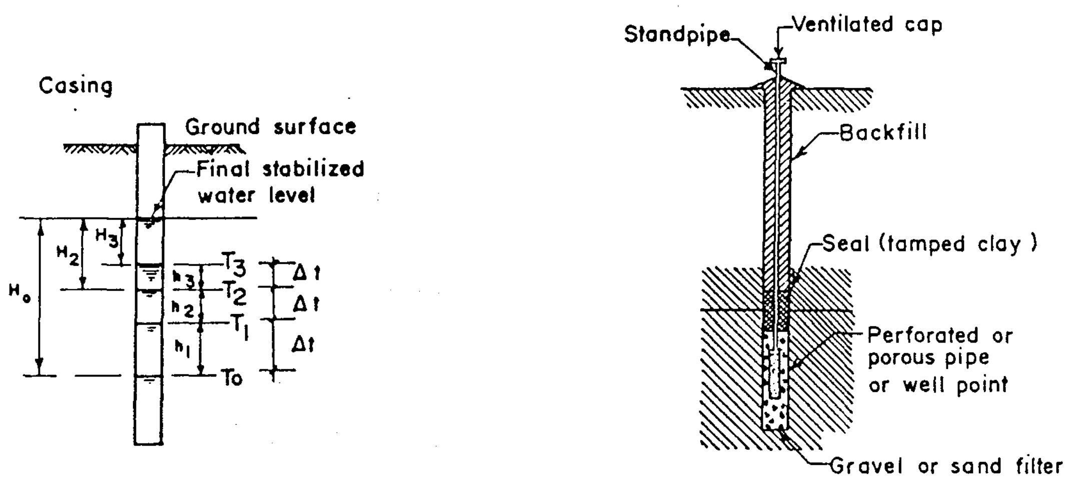


Fig. 8. Estimating Water Table

For soils which are much more impermeable, water table can not be estimated from bore hole as it might take even weeks to come to an equilibrium position. In such case water levels are measured from observation wells or piezometers with a ventilated cap (Fig. 8.b). However, water levels in adjacent ponds or lakes may serve as a good basis for establishing the ground water level.

3.2.3 Aquifer Constants

The potential yield of an aquifer can be more or less completely defined by some parameters like specific yield, storage coefficient and hydraulic conductivity or transmissibility. These parameters are usually termed as aquifer constants, which are illustrated below:

i) Specific yield

The term specific yield or capacity is used to describe the discharge capacity of an aquifer, specially the unconfined aquifer, per unit drawdown of the water table. It is expressed as:

$$q_s = \frac{Q}{S} \quad .... \quad (5)$$

Where, q_s = Specific capacity
Q = total discharge
S = Total drawdown.

This is a very important parameter for determining the volume of dewatering for each foot of drawdown to be achieved.

ii) **Storage coefficient**

The term storage coefficient is defined as the volume of water that an aquifer releases from or takes into storage per unit surface area of the aquifer per unit change in the head (Fig. 9). It is a convenient parameter for describing the yield of a confined aquifer. The coefficient is a dimensionless parameter, involving a volume of water per unit volume of aquifer. For a vertical column of unit area extending through a confined aquifer the storage coefficient, C_s, equals the volume of water released from the aquifer when the piezometric surface declines a unit distance. For most confined aquifers, the values of C_s fall in the range $0.00005 < C_s < 0.005$. This coefficient is best determined from pumping test. For unconfined aquifer storage coefficient may be upto 0.2 and the water drains by gravity from the pores.

iii) **Permeability**

The volume of water that an aquifer will yield, depends on its permeability which is expressed in terms of hydraulic conductivity, K. This may be defined as the ease with which water can move through the soil. There are many different units of permeability in use. Among these, the Meinzer unit is perhaps the easiest to conceive. It is defined as the flow of water in gallons per day per square foot section of the aquifer under a unit hydraulic gradient (1 foot vertical head lost per foot of horizontal distance). The unit of permeability that is most commonly used in soil mechanics is micron per second, μ/sec. One micron equals 10^{-4} cm or 10^{-6} meter. It can be easily converted to other units like ft/sec, m/sec and ft/day etc. **Table 1** gives a relationship between μ/sec and other units of permeability.

Different soils have different permeabilities. It is a function of soil type, grain size & its distribution, porosity, size & continuity of pore spaces, isotropy and density of soils. Approximate hydraulic conductivities (μ/sec) of soils that are most commonly encountered in natural deposits are given in **Table 2.** In absence of test results these values can be used as a rough guide for estimating or making preliminary design of a dewatering system.

Table 1. Conversion factors for units of permeability

Unit	Multiply by	To convert to	Multiply by	To Convert to
µ/sec	10^{-4}	cm/sec	10000	µ/sec
	2.13	Gallon/day/ft²	0.47	
	1.97×10^{-4}	Feet/min.	5081	
	0.283	Feet/day	3.53	
	3.40	Inches/day	0.294	
	8.64×10^{-2}	Meters/day	11.57	

Different soils have different permeabilities. It is a function of soil type, grain size & its distribution, porosity, size & continuity of pore spaces, isotropy and density of soils. Approximate permeabilities (µ/sec) of soils that are most commonly encountered in natural deposits are given in **Table 2.** In absence of test results these values can be used as a rough guide for estimating or making preliminary design of a dewatering system.

Table 2. Representative values of Coefficients of permeabilities

Soil type	Coefficient of permeability	
	Micron/sec (µ/sec).	Meter/day (m/day).
Clay	0.01 and smaller	0.00086 and smaller
Silt	5.00 to 1.00	0.43 to 0.086
Silty Sand	50.00 to 10.00	4.32 to 0.864
Fine Sand	500.00 to 10.00	43.20 to 0.86
Sand(mixture)	100.00 to 50.00	8.64 to 4.32
Coarse Sand	1000.00 to 100.00	86.4 to 8.64
Clean Gravel	10,000.00 and over	864.00 and over

Sources: S.K. Garg, M. Asaduzzaman and J. Patrick Powers

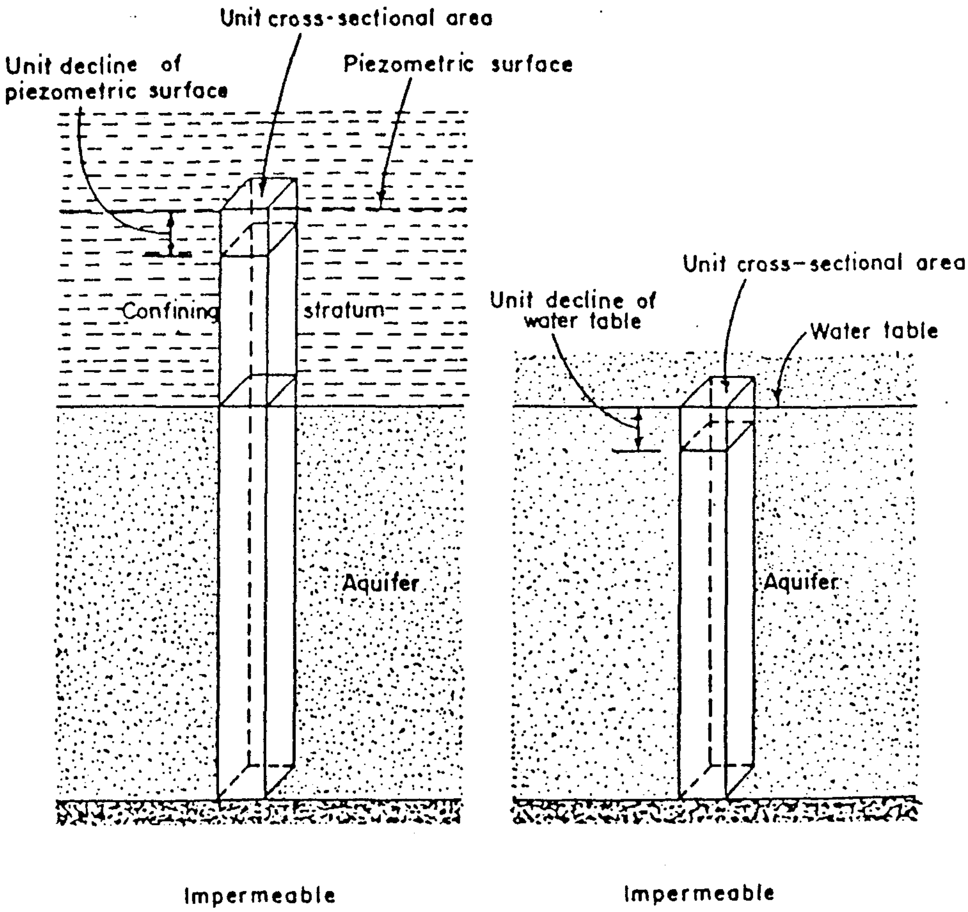


Fig. 9. Definition Sketch of Storage Coefficient

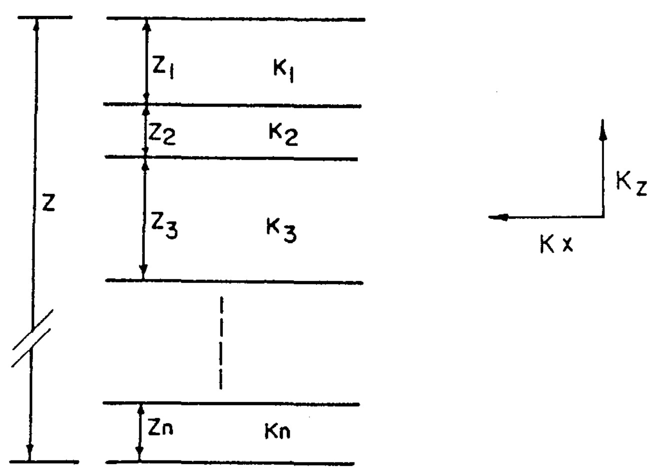


Fig. 10. Soil Anisotropy

In an anisotropic soil, permeabilities are different in horizontal and vertical direction. Usually the horizontal permeability, k_x is greater than the vertical permeability k_z. Besides, in the case of a stratified formation, each layer may have different horizontal and vertical permeabilities, k_{xi} and K_{zi} respectively (Fig. 10). The equivalent horizontal, vertical and media permeabilities are calculated as follows:

Equivalent horizontal permeability

$$K_x = \frac{K_{x1} Z_1 + K_{x2} Z_2 + \cdots + K_{xn} Z_n}{Z_1 + Z_2 + \cdots + Z_n} \qquad \text{.................. (6)}$$

When, $Z_1 = Z_2 = Z_3 = \cdots = Z_n$,

$$K_x = \frac{K_{x1} + K_{x2} + K_{x3} + \cdots + K_{xn}}{n} \qquad \text{................ (7)}$$

Equivalent vertical permeability

$$K_z = \frac{Z_1 + Z_2 + \cdots + Z_n}{\dfrac{Z_1}{K_{z1}} + \dfrac{Z_2}{K_{z2}} + \dfrac{Z_n}{K_{zn}}} \qquad \text{..................(8)}$$

When, $Z_1 = Z_2 = \cdots = Z_n$,

$$K_z = \frac{n}{\dfrac{1}{K_{z1}} + \dfrac{1}{K_{z2}} + \cdots + \dfrac{1}{K_{zn}}} \qquad \text{..................(9)}$$

Equivalent media permeability

$$K = \sqrt{(K_x K_z)} \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (10)$$

Where, K_{xi} = horizontal permeability of ith layer

K_{zi} = vertical permeability of ith layer

z_i = thickness of ith layer

i = 1 - n, number of layers.

These equations are very much theoretical but are helpful for understanding the effect of anisotropy on media permeability. Practical use of these equations are rather limited as it is difficult to measure the vertical permeability. The average permeability or equivalent media permeability is usually determined by direct or indirect tests.

As permeability is a major factor for dewatering problems, no dewatering system can be designed successfully without a sound knowledge of soil permeability. A soil with high permeability will yield large volumes of water to be dewatered while a soil with less permeability may need insignificant dewatering. So, it becomes obvious that the volume of water to be dewatered and hence the cost of a dewatering system is directly dependent on the permeability of soil. Therefore, for all major projects needing significant lowering of ground water table, permeability of soil should be determined before designing the dewatering system.

METHODS OF DETERMINING PERMEABILITY

There are three basic modes of tests for determining permeability of a soil. These are laboratory test, field test and combined field laboratory-test.

A) **Laboratory test**

The tests are done in the laboratory on samples collected from the soil strata. Such tests provide sound basis for academic purposes but are not dependable for practical use because laboratory tests predict the permeability of the samples only and do not consider the soil stratification and directional anisotropy. Besides the samples do not remain true representatives of the field condition in respect of density, particle size distribution & orientation, void ratio & pore spaces as they are more or less disturbed during sampling and transportation. As such details of laboratory procedures are not illustrated here.

B) **Field tests**

Field tests are very important for determining the permeability of soil because the results of such tests are more or less true representative of the field condition. There are two types of field tests, Seepage test and Pumping test.

1) **Seepage test**

Seepage tests constitute observation of seepage flow in the bore holes (Fig. 11). Holes in which seepage tests are done should be drilled using only clear water as drilling fluid. Otherwise mud cakes will form on the sides and bottom of the bore hole which will impede seepage of water. The tests are performed intermittently as the bore hole is advanced, when the hole reaches the level at which the test is desired, the hole is cleaned and flushed using clear water until a clean surface of undisturbed material exists at the bottom of the hole. Permeability tests are then performed by either of the following three ways:

a) **Variable head method**

This method is used where permeability of soil is not too large. There are two types of variable head methods, rising head method and falling head method.

Rising head method is suitable for comparatively coarser particles. In this method water in the bore hole is pumped to a depth below the original ground water table and the natural rate of rise of water in the bore hole is observed. As water flows from the soil to the bore hole, there is a danger of the soil in the bottom of bore hole becoming quick or unstable in rising head method.

Falling head method is suitable for comparatively fine grained soils or when there is a possibility of quick condition at the bottom of the bore hole during rising head method. In this method water level in the bore hole is raised by adding water and the permeability is calculated from rate of fall of water level in the hole. As water from the hole flows to soil, it should be clear and free of suspended materials otherwise it will clog the pores around the hole and give erroneous results.

These tests are done usually 2/3 days after drilling the bore holes for soil investigation. In either of the above cases, for a completely cased bore hole with its bottom flush in a uniform soil, the permeability of soil at shallow depth can be estimated from the following equation:

$$K = \frac{\pi D}{11(t_2-t_1)} \ln \frac{h_1}{h_2} \quad \ldots\ldots\ldots\ldots\ldots(11)$$

Where, K = permeability of soil, cm/sec

D = diameter of casing, cm, $15 \le D \le 150$ cm

t_1 & t_2 = initial and final time, sec

h_1 & h_2 = difference in elevation between water level in the bore hole and the original water table at times t_1 & t_2, cm.

b) Constant head method

Constant head method is used where permeability of soil is too high and variable head method does not allow enough time to record the time & level of water in the bore hole. In this method water is added to the casing at a rate sufficient to maintain a constant water level at or near the top of the casing for a minimum period of 10 minutes. Water is usually added from a calibrated container. Permeability of soil with a completely cased bore hole having its bottom in an uniform soil is calculated from the following formula:

$$K = \frac{q}{2.75\ D\ H_c} \qquad \text{..............................(12)}$$

Where, K = permeability, cm/sec.

q = rate of flow, cm^3/sec.

D = dia. of casing, cm.

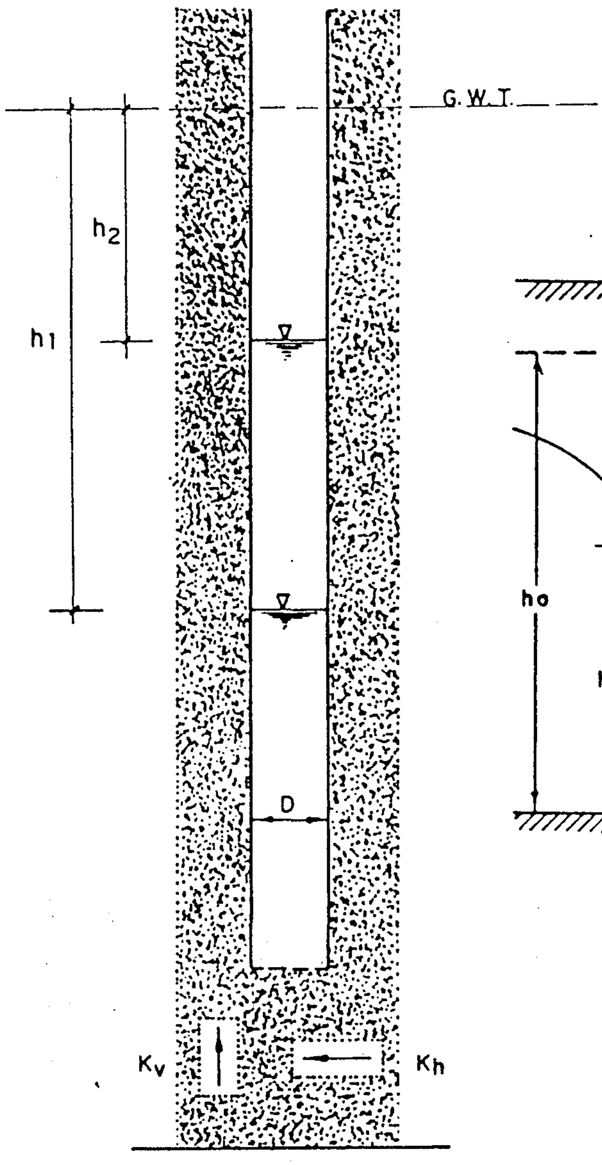
H_c = constant head above ground water level, cm.

However, none of the above formulas are dependable for stratified soils.

2) Pumping test

Pumping test is the most reliable test for estimating in-situ permeability of soil. However, this test is rather costly and time consuming and may not be justified for small projects. But for major projects where large volumes of dewatering is anticipated pumping test should be carried out for estimating field permeability as accurately as possible. Otherwise construction of the project may be unnecessarily delayed or hampered due to dewatering problems, which will eventually increase the project cost that may outweigh the cost of pumping test.

In pumping test, hydraulic conductivity of an aquifer is determined by pumping a well at constant rate and observing drawdown of piezometric surface or water table in observation wells installed at some distances from the well (Fig. 12). There are two types of pumping tests, i) steady state test and ii) non-steady state or transient state test.



FLUSH BOTTOM IN
UNIFORM
SOIL

Fig. 11. Seepage Test

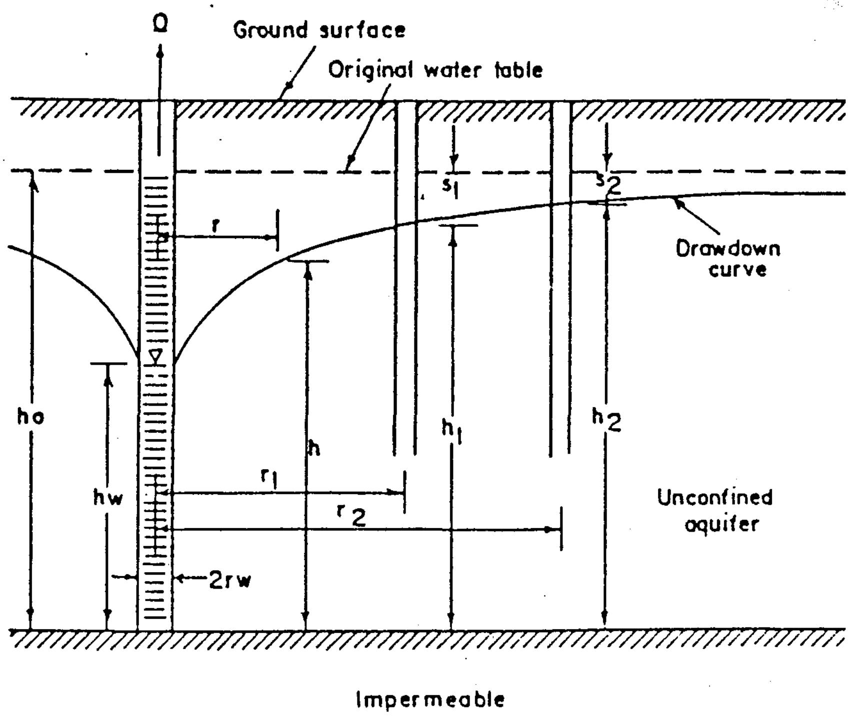


Fig. 12. Pumping Test

In steady state test pumping is continued for a sufficiently long time so that water level in the observation wells approach a equilibrium position (actually a near equilibrium condition is reached). At equilibrium, the permeability of the aquifer is given by:

For confined aquifer

$$K = \frac{Q \ln (r_2/r_1)}{2\pi D \, (h_2 - h_1)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(13)$$

For unconfined aquifer

$$K = \frac{Q \ln (r_2/r_1)}{\pi (h_2^2 - h_1^2)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(14)$$

Where, K = permeability of aquifer

Q = Steady discharge from well

r_1, r_2 = radial distances from the well

h_1, h_2 = height of piezometric surface or water level at distances r_1 & r_2

D = Depth of aquifer

In case of unsteady or transient state test water level drops in observation wells are measured in relation to time and the discharge-draw-down-time records are used for predicting the Transmissibility (T) and storage coefficient (C_s) of the aquifer. The transmissibility of the aquifer is a measure of the flow through unit width of the total aquifer. Permeability factor K is found out by dividing the transmissibility T by depth (D) of the aquifer in case of confined aquifer and by average depth of water table, $(h_1+h_2)/2$, in the case of unconfined aquifer.

The unsteady tests are more suitable for confined aquifers. However, with certain simplifying assumptions it can be used for unconfined aquifers too. For calculating the aquifer constants. Thies (1935) has developed the following two equations by solving the unsteady flow equations.

$$T = \frac{Q}{4\pi S} W(u) \qquad \text{.........................(15)}$$

$$C_s = \frac{4Tu}{r^2/t} \qquad \text{.........................(16)}$$

Where, T = Transmissibility (m^2/day)

C_s = Storage coefficient

r = distance from test well (m)

t = time from start of pumping (day)

S = draw down (m/day)

Q = discharge (m^3/day)

u & W(u) = Well functions.

Since, u, W(u) are both functions of T, & C_s, the above equations can not be solved directly. Thies, Chow, Jacob, Waltom and others have developed special techniques for solving these equations. The following sections deal with the methods of Thies, Chow and Jacob only.

a) Thies method

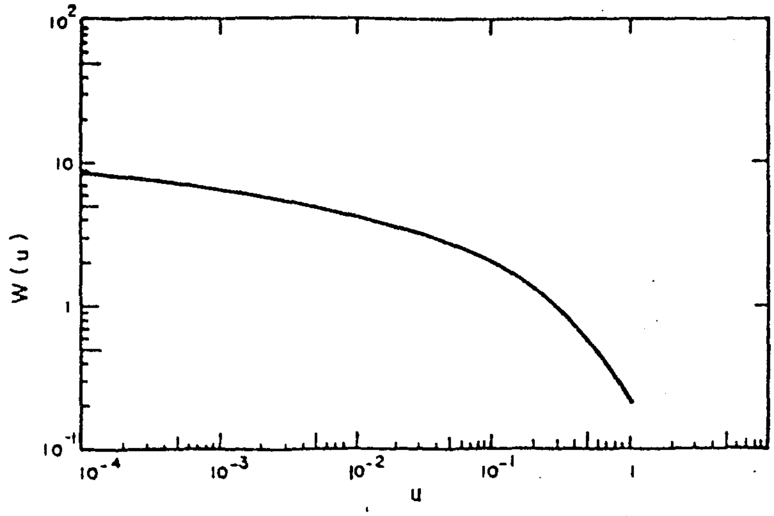
This is a graphical method for evaluating T and C_s. The step by step procedures are as follows:

- Plot a type curve of u Vs. W(u) in double logarithmic paper using values from **Table 3 (Fig.13.a).**
- Plot the values of r, t and S as S Vs r^2/t in another double log paper (Fig. 13.b).

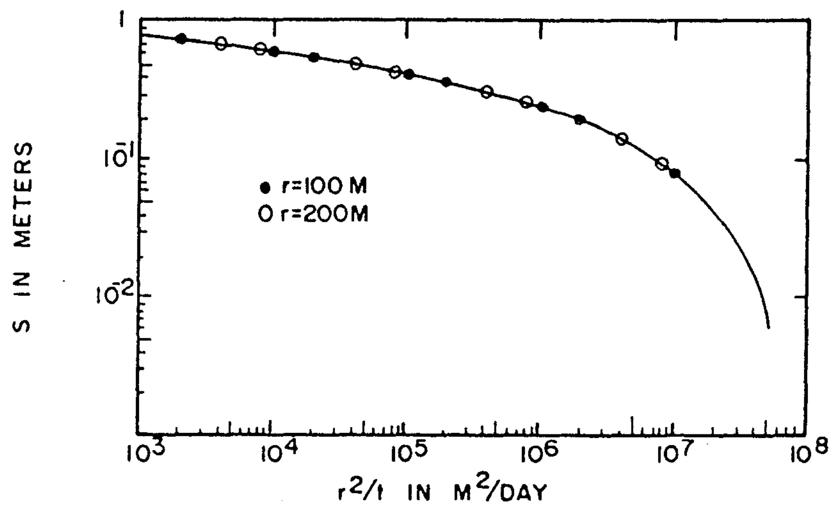
- Place the plot of r^2/t vs. S on the type curve and move, with the axes parallel, until the whole or part of the two curves coincide (Fig. 13.c).
- Arbitrarily select a match point on the two curves and read the values of S, r^2/t, W(u) and u.
- Substitute the values of S, r^2/t, W (u) and u in equations 15 and 16 and calculate the values of T & C_s.
- Calculate the value of K by dividing T with aquifer depth, D, for confined aquifer or average depth, $(h_1 + h_2)/2$, of water table for unconfined aquifer.

Table 3. Values of W(u) for Values of u

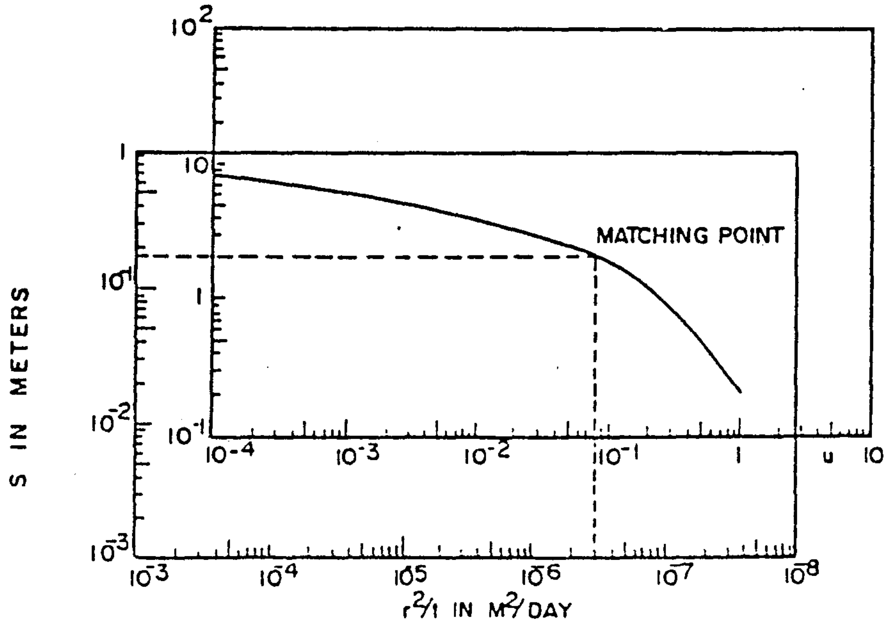
u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
X 1	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
X 10^{-1}	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
X 10^{-2}	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
X 10^{-3}	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
X 10^{-4}	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
X 10^{-5}	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
X 10^{-6}	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	1.04
X 10^{-7}	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
X 10-8	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
X 10^{-9}	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
X 10^{-10}	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
X 10^{-11}	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
X 10^{-12}	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.96	24.86
X 10^{-13}	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
X 10^{-14}	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
X 10^{-15}	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76



a) W (u) VS. u Curve



b) S VS. r^2/t Curve



c) Matching

Fig. 13. Thies Method of Estimating Transmissibility

b) <u>Chow method</u>

This method, developed by Chow in 1952, gives a direct solution of the equations 15 & 16 by introducing a function:

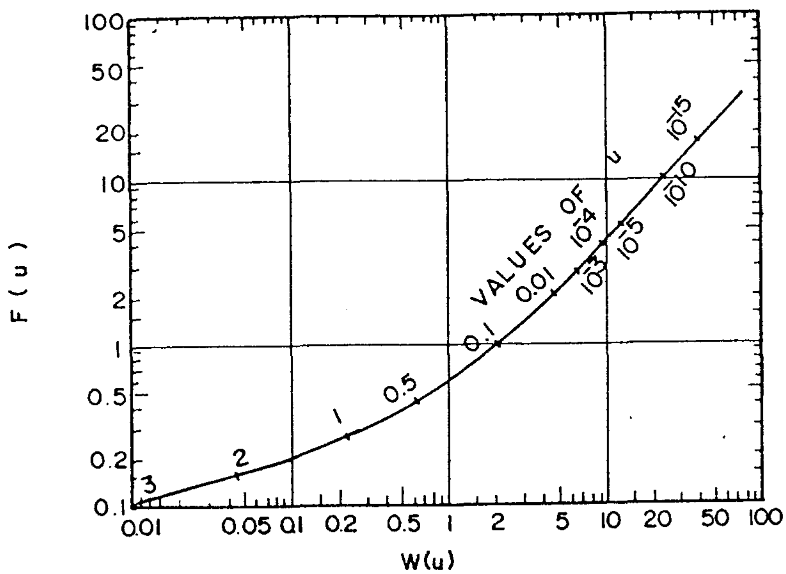
$$F(u) = \frac{W(u)\ e^{u}}{2.3} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

The step by step procedures are as follows:

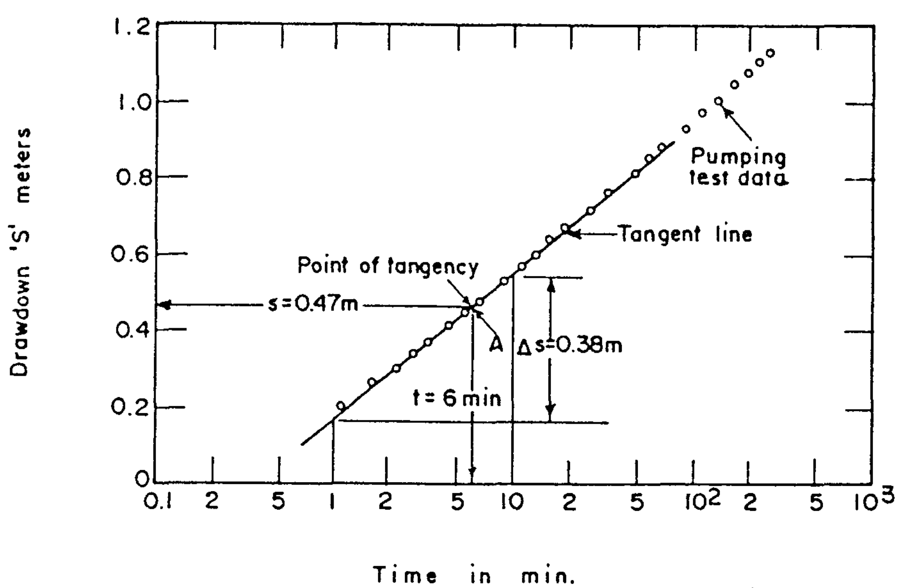
- Take values of u & W(u) from **Table 3** and calculate the value of F(u).
- Plot a type curve of F(u), W(u) & u in double log paper with W(u) as abscissa and u on the curve (Fig. 14.a)
- Plot the values of drawdown S. vs. time t, of the observation well at a distance r, on a semilog paper with log t as abscissa and S as ordinate (Fig. 14.b).
- Select a point 'A' on the S log t curve and draw a tangent at this point. The slope of the tangent at 'A' gives the drawdown difference dS_A for one log cycle of time.
- Calculate the value of F(u) for S_A and dS_A from:

$$F(u) = \frac{S_A}{dS_A} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

- Put this value of F(u) on the type curve and find the values of u, & W(u).
- Calculate the value of T & C_s by putting the values of u & W(u) in equations 15 & 16.
- Calculate the value of K in the same way as in thies method.



a) F (u) VS. W (u) Curve



b) S VS. t Curve

Fig. 14. Chow Method of Estimating Transmissibility

c) <u>Jacob method</u>

Jacob method (Jecob & Cooper, 1946) gives a direct solution of equations 15 & 16. The step by step procedures are:-

- Tabulate the values of drawdown, S and time, t for an observation well at a distance, r from the well.

- Plot S Vs t on a semilog paper with log t as abscissa (Fig. 15). Extend the straight line portion to the time axis.

- Select a point on the S log t curve. Draw a tangent at this point and find the slope dS. Calculate the value of transmissibility, T from:

$$T = \frac{2.3Q}{4\pi dS} \qquad \text{.................................(19)}$$

- Find the value of to at the meeting point of the extended straight line with the time axis. Calculate storage coefficient C_s from:

$$C_s = \frac{2.25\ T\ t_o}{r^2} \qquad \text{........................ (20)}$$

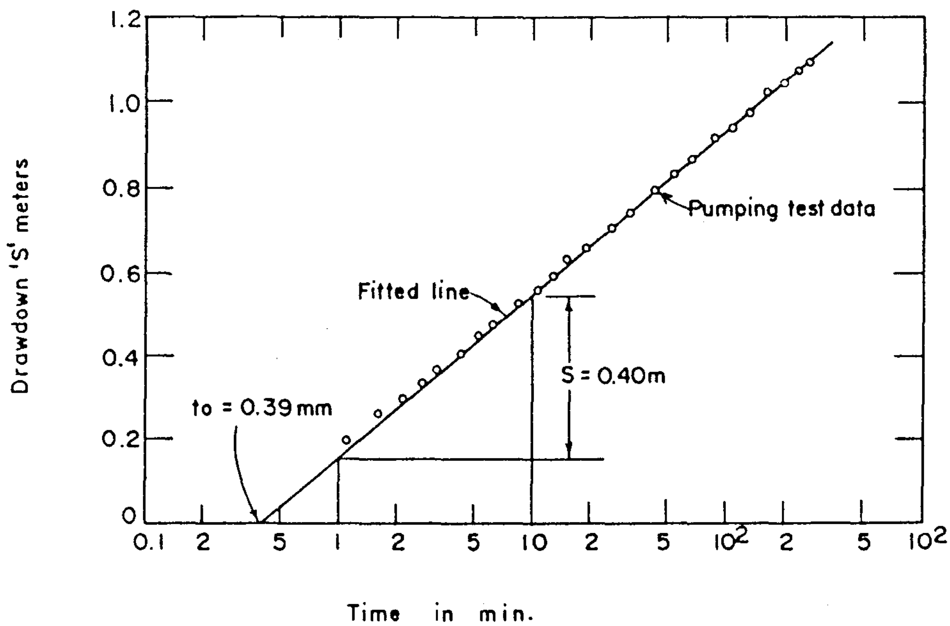


Fig. 15. Jacob Method of Estimating Transmissibility

3) Combined field - laboratory test

This method combines some of the direct field tests with laboratory tests to predict the permeability of soil. Byron Prugh developed this method based on mechanical analysis and in-situ density tests. The method gives very approximate result and is suitable for granular soils only.

The D_{50} size and uniformity coefficient, C_u of the soil are determined from the mechanical analysis (gradation curve). The in-situ density is obtained from the blow counts of standard penetration test, SPT (Table 4).

Depending on the relative density of the soil, one of the three charts shown in Fig. 16, can be selected (Source: J.P. Powers). The chart gives a direct value of permeability corresponding to the value of uniformity coefficient Cu and D_{50} size of the soil sample. Since this is a very approximate method, several values of K should be determined and averaged to get a representative value of the same.

Table 4. Relative density of granular soils

Compactness	Very loose	Loose	Medium	Dense	Very dense
Standard penetration resistance, N of SPT	0 – 4	4 – 10	10 – 30	30 – 50	> 50
Relative density, Dr,	0 – 15%	15% – 35%	35% – 65%	65% – 85%	85% – 100%
Angle of internal friction, ϕ, degrees	– 28	28 – 30	30 – 36	36 – 41	41 –

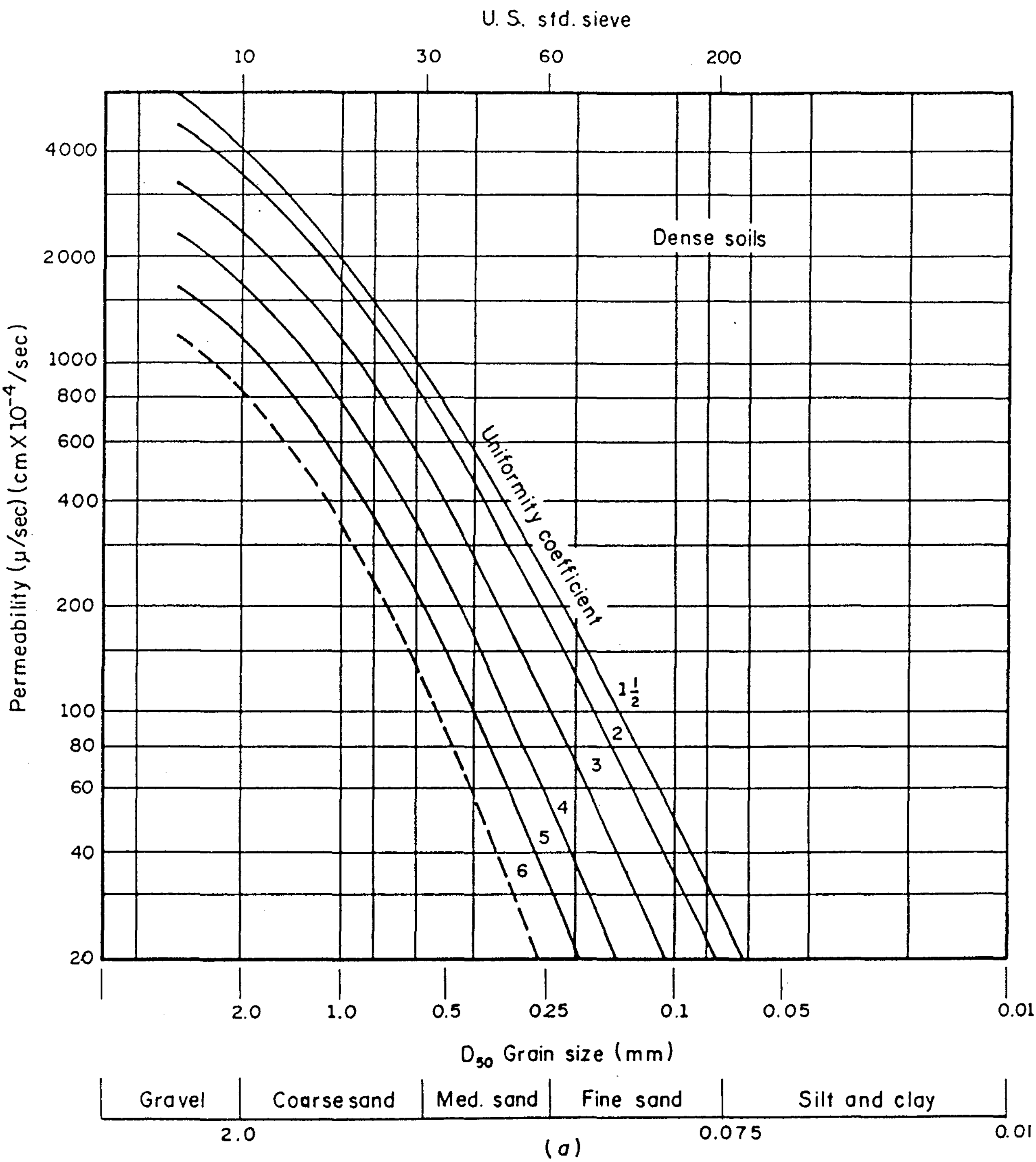


Fig. 16. Prugh Method of Estimating Permeability a) Dense Soil

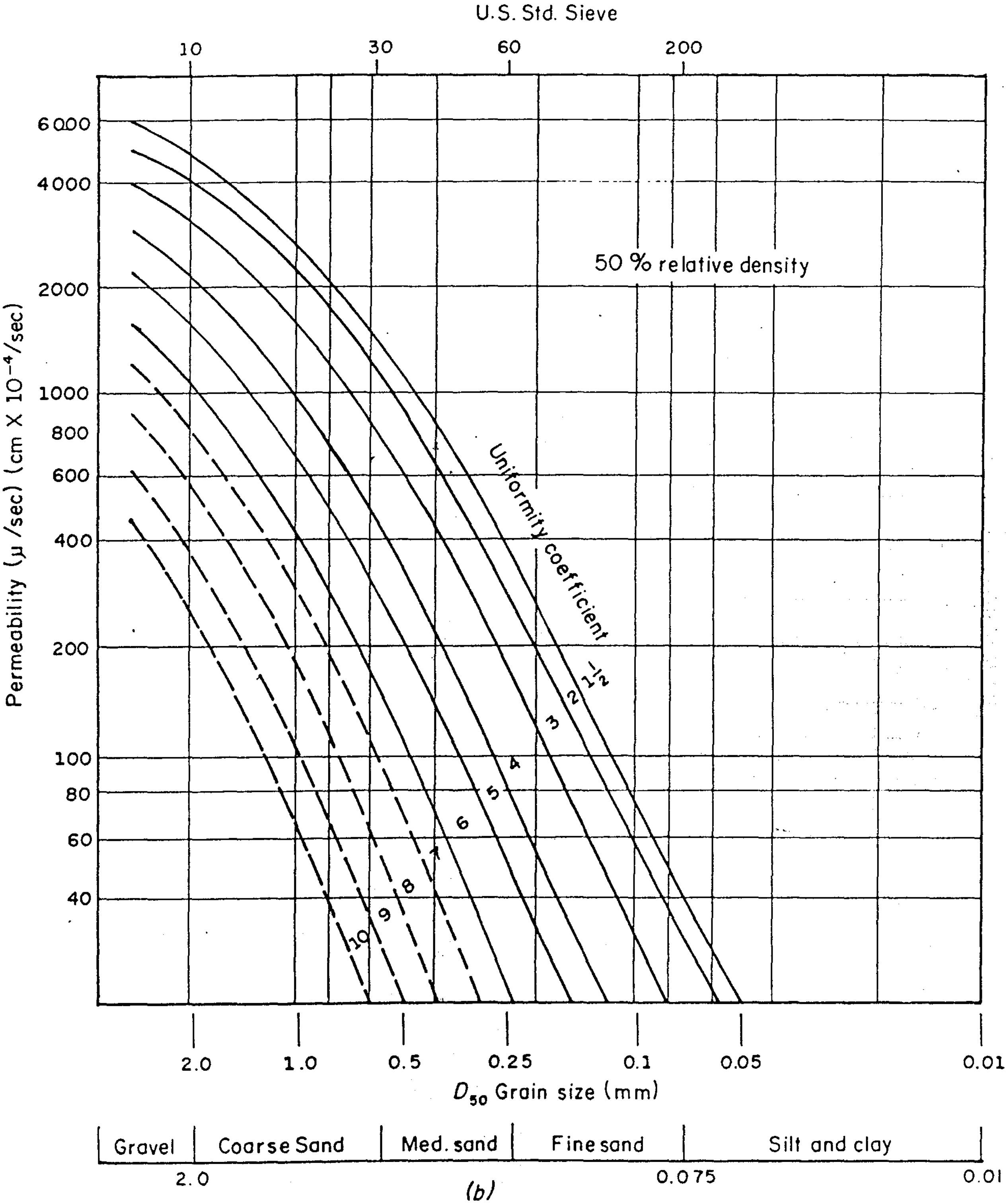


Fig. 16. Prugh method of estimating permeability b) Medium dense Soil

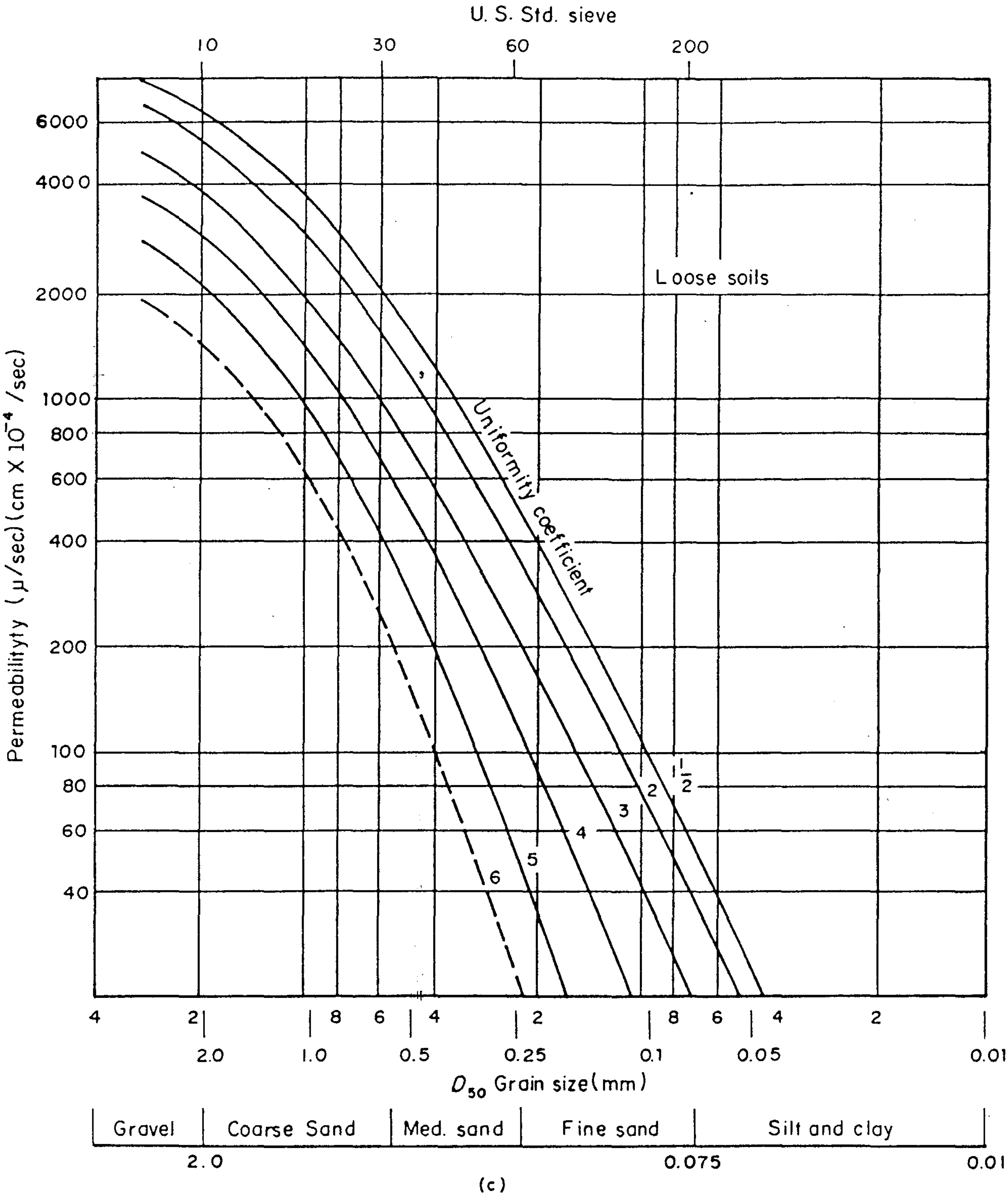


Fig. 16. Prugh Method Estimating Permeability c) Loose Soils

3.3.0 HYDRAULICS OF GROUND WATER FLOW

Underground water is always in motion and the velocity of flow is usually very small. In analyzing the ground water motion, the actual tortuous flow paths of water molecules through the soil pores are assumed to be continuous smooth paths. The resulting smooth flow lines of water molecules are called stream lines. When the stream lines are straight and parallel and the flow does not change with distance, it is called uniform flow. On the other hand, when the stream lines curve, diverge or converge and the flow changes with distance it is called non uniform flow. Besides, the flow is termed as steady if, it does not change with time and non steady or transient when it changes with time.

The flow of water through a saturated porous media is governed by Darcy's law which dictates that the velocity of flow is directly proportional to the hydraulic gradient i.e the rate of change of potential head or total head. Neglecting the velocity head, which is usually very small for ground water flow, a potential head of a given point is defined as the sum of pressure head and elevation head at the point (Fig. 17). The velocity of flow is then given by:

$$V = K \frac{(h_1 + z_1) - (h_2 + z_2)}{L} \qquad \ldots\ldots\ldots\ldots(21)$$

$$= K \frac{h_e}{L} \qquad \ldots\ldots\ldots\ldots(22)$$

$$= K\, i \qquad \ldots\ldots\ldots\ldots(23)$$

Where,

- v = velocity of flow
- $(h_1 + z_1)$ = potential head at point 1
- $(h_2 + z_2)$ = potential head at point 2
- L = distance between points 1 & 2 along the streams line.
- K = constant of proportionality usually defined as soil permeability.
- h_e = $(h_1 + z_1) - (h_2 + z_2)$ = head lost between points 1 & 2.
- i = $\frac{h_e}{L}$ = hydraulic gradient.

However, Darcy's law is valid only for laminar flow where viscous forces are dominant. For turbulent flow where the inertial forces are dominant, the linear relationship expressed by Darcy's law is no longer valid. The laminar and turbulent flow is usually distinguished by a non-dimensional parameter, N_R, called Rynolds number, which is expressed as:

$$N_R = \frac{\rho V D}{\mu} \qquad \text{...(24)}$$

Where,

- ρ = fluid density
- V = velocity of flow
- D = effective grain size D_{10} for soil and diameter for pipe.
- μ = Viscosity of fluid (P or gm/cm.s).

Experiments show that Darcy's law is valid for $N_R \leq 1$ and does not defer seriously upto $N_R = 10$.

Fortunately, except few rare situations like flow near pumped wells, most natural ground water flow occurs with $N_R \leq 1$.

The velocity V, referred in the above equation is termed as Darcy velocity because it assumes that the flow occurs through the entire cross section of the material irrespective of the solids and pores. In actual case the flow is limited to the pore spaces only, which can be idealised as parallel capillary tubes. Fig. 18 shows such a bundle of capillary tubes that represent a soil cross section. The actual velocity through this cross section is equal to the Darcy velocity divided by porosity of the section. However, for practical purposes, the term velocity in the following sections will refer to the Darcy velocity, v, only.

3.3.1 Well hydraulics

Well hydraulics explain the characteristics of ground water flow to wells penetrating confined or unconfined aquifers with or without vertical recharge. When the well is at rest i.e when there is no flow in the well, water pressure in the well is the same as that in the aquifer formation. In this condition the level at which water stands in the well is called static water level. For unconfined aquifer this level coincides with the water table and for confined aquifer this coincides with the piezometric surface. When water is pumped, water level in the well drops below the static water level and a hydraulic gradient is set up between the water level/piezometric surface in the well and in the formation outside the well, which causes water in the aquifer to flow in the well from all direction. As the flow continues, water table or piezometric surface develops an increasingly steeper slope towards the well.

The difference in the level between the static water level/piezometric surface and the surface of the cone of depression is called draw down, which is a function of distance from the well. The line defining the points of draw down as function of distance is called draw down curve. In three dimensional form, rotation of the draw down curve forms a cone shaped surface which is called cone of depression. The outer limit of this cone of depression where it almost coincides with the static water or piezometric level, describes the zone of influence.

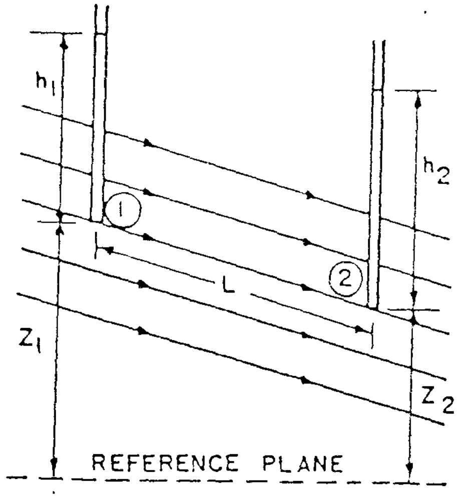


Fig. 17. Definition Sketch of Darcy's Law

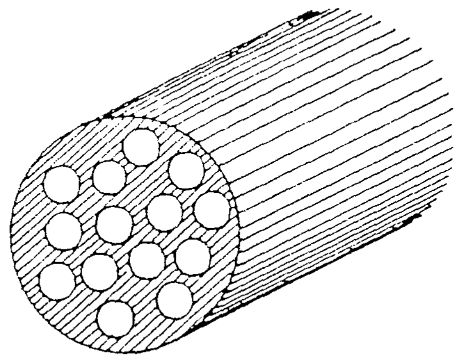


Fig. 18. Bundle of Capillary Tubes

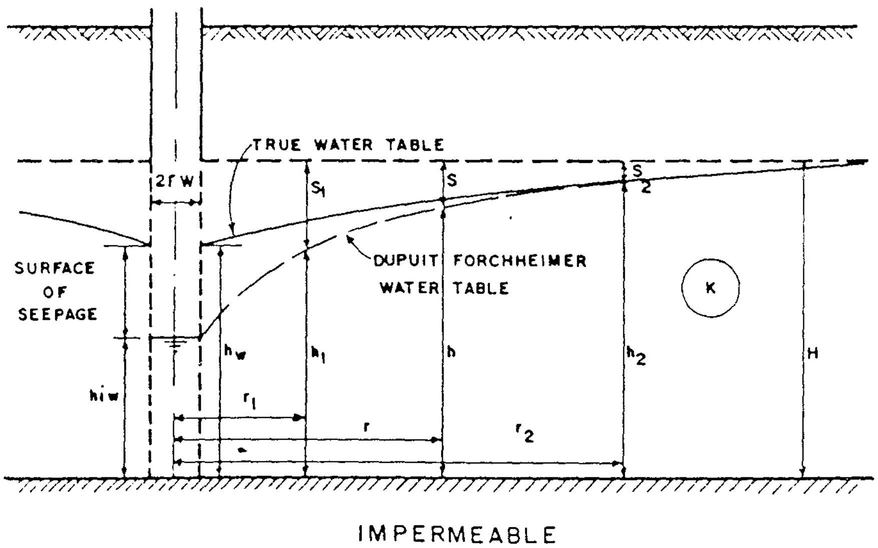
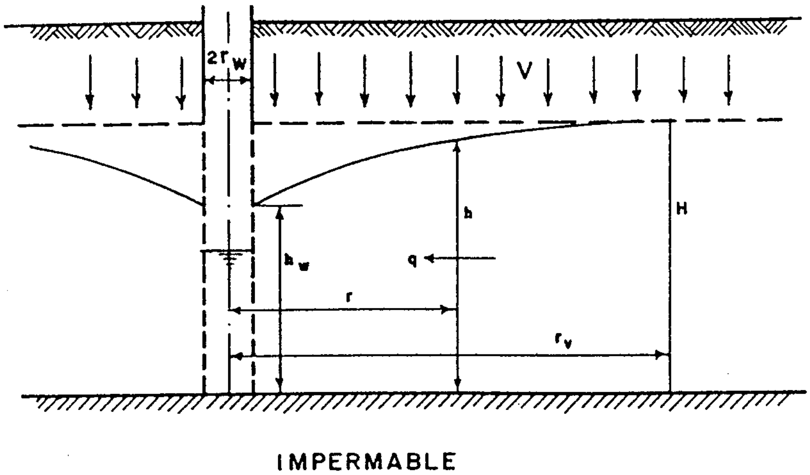
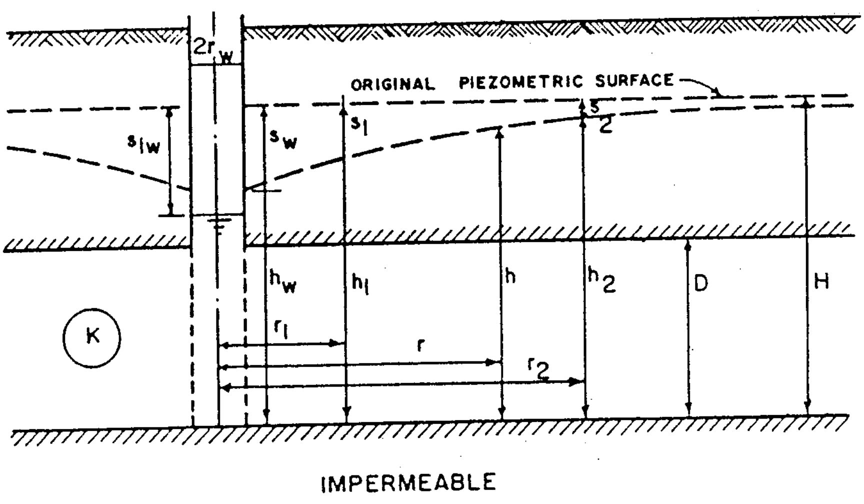


Fig. 19. Geometry of Flow to Wells a) Unconfined Aquifer without Vertical Recharge



b) Unconfined Aquifer with Vertical Recharge.



c) Confined Aquifer.

Fig. 19. Geometry of Flow to Wells

Figs. 19, a, b, & c describe the geometry of flow to wells in unconfined and confined aquifers with or without vertical recharge. The vertical recharge mainly comes from precipitation, irrigation and percolation losses from canals or other water bodies.

i) <u>Flow to wells</u>

In determining the flow to wells, it is assumed that the aquifer is infinite in areal extent and contributes water to the well from all direction.

Groundwater flow to wells fully penetrating an aquifer is given by the following equations.

a) <u>Unconfined aquifer</u>

<u>Without vertical recharge</u>

$$Q = \pi K \frac{h_o^2 - h_w^2}{\ln(r_o/r_w)} \quad \ldots\ldots(25)$$

$$= \pi K \frac{h_2^2 - h_1^2}{\ln(r_2/r_1)} \quad \ldots\ldots(26)$$

<u>With vertical recharge</u>

$$Q = \pi K \frac{h_o^2 - h_w^2}{\ln(r_o/r_w)} + \frac{\pi W}{2} \frac{r_o^2 - r_w^2}{\ln(r_o/r_w)} \quad \ldots\ldots(27)$$

Where, Q = well discharge

K = Aquifer permeability

W = Recharge rate

And h_o, h_w, h_1, h_2, h, r_o, r_w, r etc. are as in Fig. 19, a, b, & c.

b) <u>Confined aquifer</u>

As vertical recharge for confined aquifers are not usual, the equation has been developed for horizontal recharge only.

$$Q = 2\pi KD \frac{h_o - h_w}{\ln(r_o/r_w)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

$$= 2\pi KD \frac{h_2 - h_1}{\ln(r_2/r_1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Where, Q = well discharge

K = Aquifer permeability

D = Aquifer thickness

And h_o, h_w, h_1, h_2, h, r_o, r_w, r_1 & r_2 etc. are as in Fig. 19, a,b, & c.

The radius of influence r_o describes the extent of the zone that contributes to the well. It is the last limit of the cone of depression produced around the well. The value of r_o depends on the nature of soil, permeability of aquifer and the draw down and should be determined from field test. However, for small jobs where field test results may not be available, it can be estimated from Sichardt's empirical formula as:

$$r_o = 3\ S_w\sqrt{K} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

Where, r_o = radius of influence, ft.

S_w = draw down in the well, ft.

K = permeability, microns/sec

ii) <u>Well losses</u>

The above equations of flow to wells are very theoretical which assume idealized aquifer and simplified flow conditions. However, as the approximations involved in these assumptions are not much out of place, the results can be used for practical purposes where rough estimates provide useful information. In actual situation water level/piezometric surface just out side the well is much above the water level inside the well (Fig. 20.a). The theoretical water table, based on Dupuit-Forchheimer assumptions, is much below the actual water table. The difference is maximum near the well and diminishes gradually with distance from the well. However, at a distance greater than one and half times the aquifer thickness, the difference is negligible. This difference of elevation between the water levels inside and outside of the well is called well loss. Well loss has two components, seepage loss and entrance loss, which are functions of seepage head and velocity head.

Seepage head or seepage face that develops due to turbulent flow situations around the well is the distance through which water seeps down the vertical wall of the well screen. Velocity head is a function of velocity of flow through the gravel pack, screen and pumping units.

Therefore, it appears that the total draw down in the well at steady state condition is the sum of actual drawdown due to loss of head in the aquifer (aquifer loss) and the well loss. The aquifer loss is the actual drawdown outside the well. Well loss is expressed as a function of well discharge. Total drawdown is then given by:

$$S_w = BQ + CQ^n \qquad \text{................................(31)}$$

Where, S_w = total draw down in the well.

BQ = aquifer loss

CQ^n = well loss

n = a constant greater than one and should be determined from experiment (Jacob suggested a value of $n = 2$).

C = well loss constant

The well loss constant C depends on the condition of the well screen and its surrounding. Walton, based on his experience, suggested certain values for the constant C (**Table 5**), which are very helpful for estimating the well efficiency.

Table 5. Relation of Well Loss coefficient to Well Condition (after Walton)

Well Loss Coefficient C, min^2/m^5	Well Condition
<0.5	Properly designed and developed
0.5 to 1.0	Mild deterioration or clogging
1.0 to 4.0	Severe deterioration or clogging
> 4.0	Difficult to restore well to original capacity.

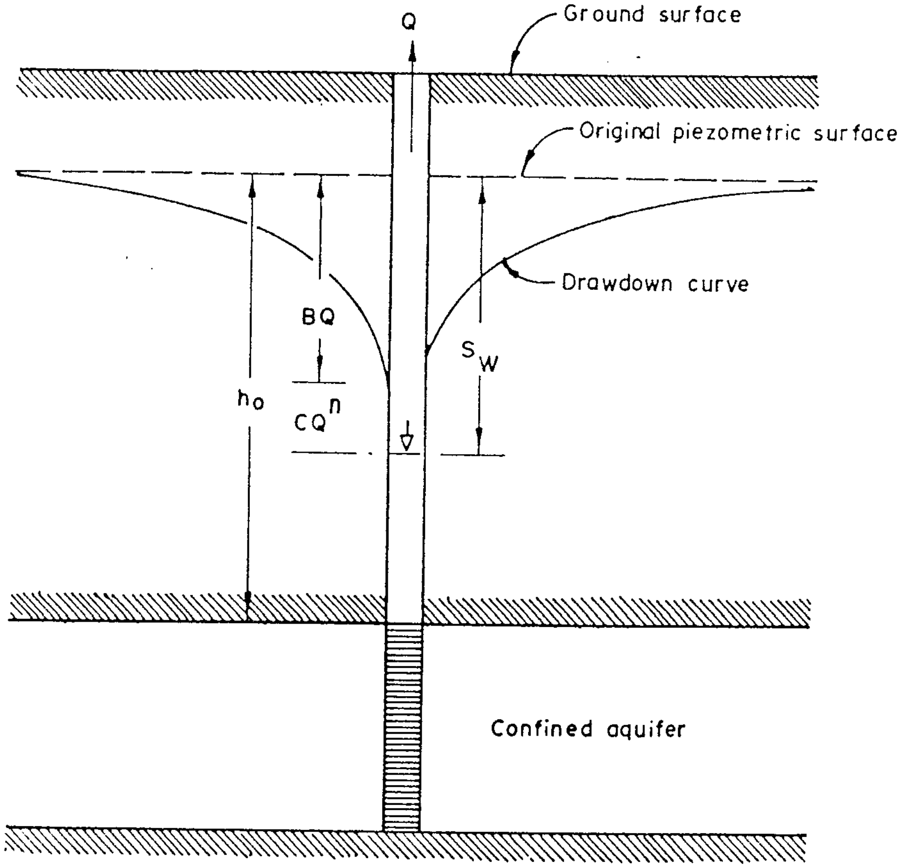
For important jobs, well loss constant C and formation loss constant B should be determined from field test. In this test the well is initially pumped at a slow rate until the draw down within the well stabilizes. The well is then pumped continuously and drawdown at the well is recorded at certain interval of time. The constants B & C are determined by rearranging the well loss equation as follows:

$$S_w = BQ + CQ^n$$

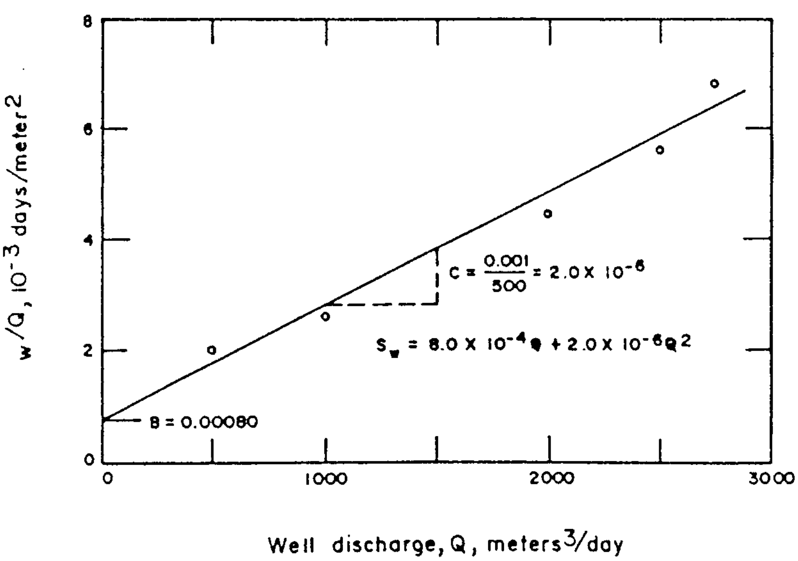
$$= BQ + CQ^2 \quad \text{(letting } n = 2) \quad \ldots\ldots (32)$$

$$\frac{S_w}{Q} = B + CQ \quad \text{(dividing by Q)} \quad \ldots\ldots (33)$$

Which is the equation of a straight line.



a) Definition Sketch



b) SW/Q VS. Q

Fig. 20. Well Loss

The values of S_w/Q are plotted against Q, and a straight line is fitted through these points. The slope of the straight line gives the value of C and the intercept on the S_w/Q axis gives the value of B (Fig. 20.b).

Once the values of B & C are determined, the magnitude of well loss can be estimated. This value when deducted from the draw down inside the well will give the actual draw down outside the well. This is very important for estimating the expected lowering of water table by a dewatering installation. As a very crude approximation well loss can be taken as 1/3 of the aquifer loss.

iii) **Maximum well yield**

It was already mentioned that the equations for flow to wells have been derived on the basis of some simplifying assumptions. These equations give good results within certain limits and beyond these limits the results are no more reliable. Let's consider the thiem's equations (eq. no. 25 & 28) for flow to wells:

$$Q = \pi K \frac{h_o^2 - h_w^2}{\ln(r_o/r_w)} \quad \text{------(unconfined aquifer).........(25)}$$

$$Q = 2\pi KD \frac{h_o - h_w}{\ln(r_o/r_w)} \quad \text{------(confined aquifer)..........(28)}$$

From these equations it appears that:

$$Q = 0, \quad \text{when} \quad h_w = h_o$$

$$Q = Q_{max}, \quad \text{when} \quad h_w = 0$$

The plot of these equations as a function of h is given in Fig. 21. The result of maximum discharge is just absurd because hw = o means there is no water in the well and the entire Q enters through a point. This can only happen when the velocity is infinite which is impossible. This shows that these equations are theoretically unable to predict the maximum yield.

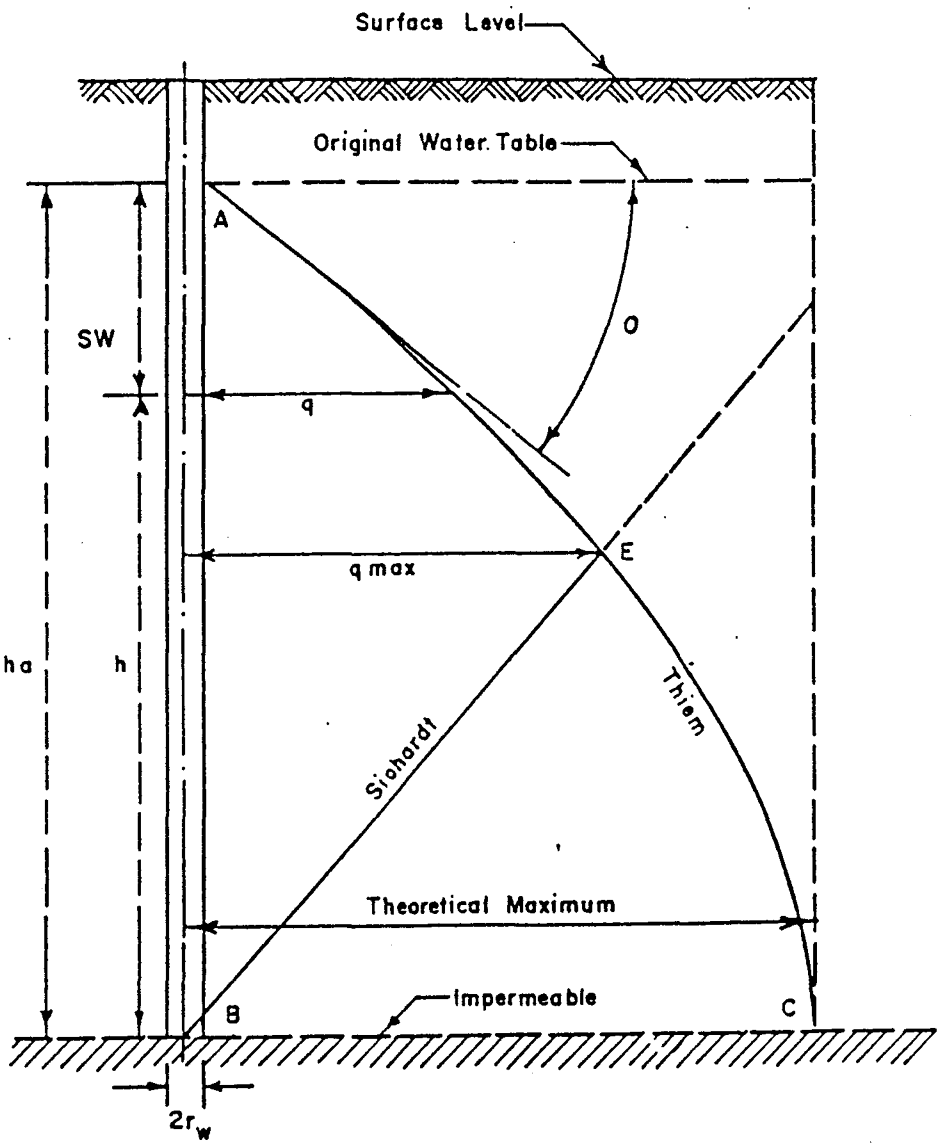


Fig. 21. Maximum Well Yield

However, it is obvious that the true maximum lies somewhere in between the extreme points. This can be solved by using the sichardt's equation of maximum velocity through the well screen. The max. velocity is given by:

$$V_r = \frac{\sqrt{K}}{15} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

Where, V_r = velocity, m/sec

K = Permeability, m/sec

From equation 34 it appears that:

$$Q = 2\pi r_w h \; V_r$$

$$= 2\pi r_w h \; \frac{\sqrt{K}}{15} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

The plot of this equation, when superimposed in Fig. 21, gives a straight line that meets the thiems plot at point E. The value of Q corresponding to the point E is then the maximum obtainable discharge.

3.3.2 Storage depletion

Storage depletion is of great importance during the initial stages of pumping specially when the extent of dewatering is large. Theoretically we assume that water is released from storage immediately after the water table is lowered. In practice it does not happen. There is always some time lag between lowering of water table and release of stored water, which is more prominent for water table aquifers or unconfined aquifers.

So, before equilibrium is reached, there is always some additional quantity of water that drains form storage (Fig. 22). When the aquifer permeability is high and the drawdown is considerable, the volume of storage depletion can be as high as or even more than the steady state flow. If the aquifer characteristics are determined from pumping test, storage depletion may not be a serious problem as it has already been taken care of, at least partly. In other cases, depletion of this stored water should be properly taken care of.

3.3.3 Multiple well system

So far we have developed equations and formulas for single wells. In reality there may be situations like construction dewatering where several wells have to work simultaneously. In such cases the cone of depression of one well overlaps the cone of depression of the near by wells. This overlapping of the cones of depressions is called interfacing.

For a group of wells forming a well field, the resultant draw down at any point can be determined if the well discharges are known. Fig. 23, shows a group of wells with discharges Q_1, Q_2 & Q_3 and the resulting draw down curves. From the principal of superposition, draw down at any point of the well field is equal to the summation of draw downs caused by individual wells. For example if S_t is the net draw down at any point of the well field and S_a, S_b ---- S_n etc. are the draw downs at the same point caused by the wells a, b, c, --- & n respectively, then:

$$St = S_a + S_b + S_c + \text{-----} + S_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

The degree of allowable interference of the wells in a well field depends on the object of the project. For water supply project, the interference should be minimum so that the individual well yields are not significantly reduced. On the other hand for drainage wells interference may be desired for getting more draw down. In any case there should be some limit that will give the optimum result.

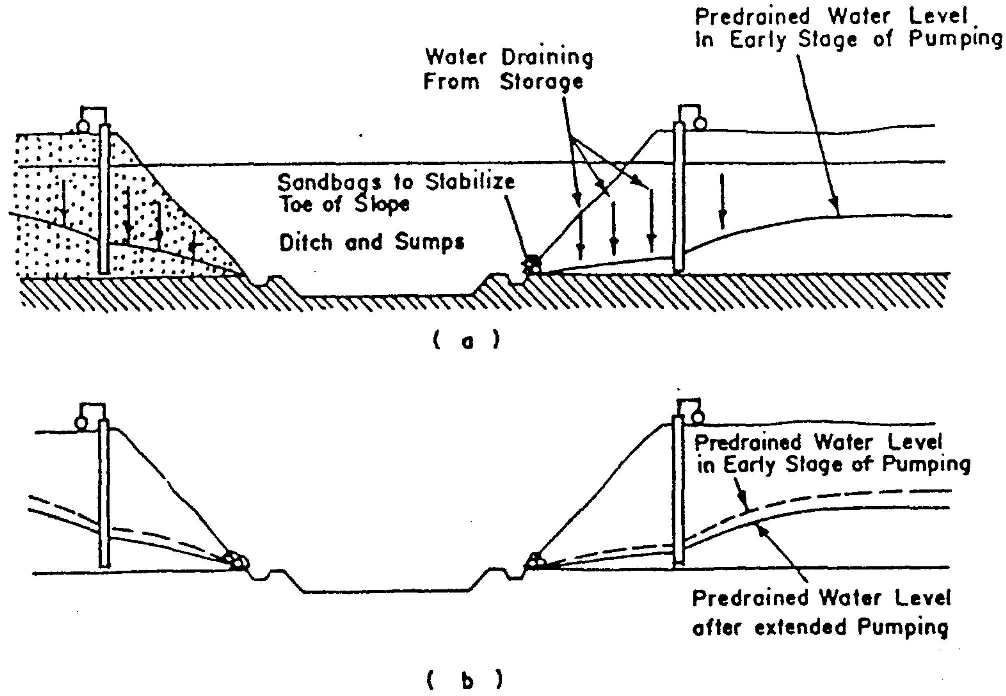


Fig. 22. Storage depletion a) Storage Depletion during early Pumping
b) Late Pumping after Storage Depletion

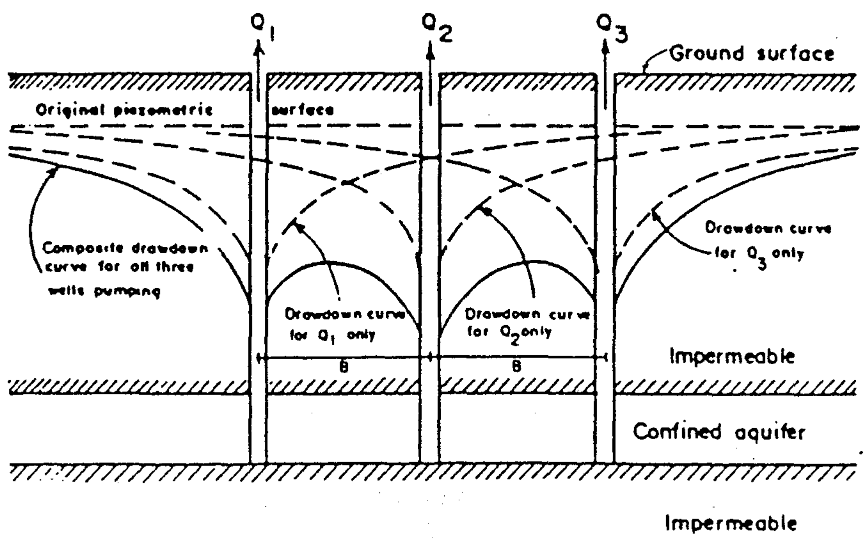


Fig. 23. Multiwell Pumping

According to sichardt minimum distance between the centres of two wells should not be less than 15 times the well diameter. He set the following limit:

$$L_{min} = 5 \times 2\pi r_w$$
$$= 31.50\ r_w \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(37)$$

where, L_{min} = min. distance between two wells

r_w = radius of the well.

However, in reality spacings of shallow and deep tubewells are always larger than the spacing given by above equation and therefore, this checking may not be necessary.

3.3.4 Influence of boundary conditions on flow to wells

Boundary condition greatly effects the flow patterns to wells if it falls within the zone of influence of the well. However, it has little effect if it falls beyond 1.5 to 2 times the zone of influence. There are two common types of boundaries i) barrier boundary and ii) Recharge boundary. A barrier boundary is a kind of impervious barrier or rock barrier that does not contribute water to the pumped well. It actually defines the limit of the aquifer at the barrier and contradicts the assumption of infinite hydraulic system.

A recharges boundary is kind of source or line source within the zone of influence that contributes significantly to the flow of well but does not allow the drawdown curve to extend beyond the boundary. Water level in the boundary remains constant at its original level.

Fig. 24 shows a barrier boundary. The problem is solved by creating hypothetical infinite hydraulic system that satisfies the boundary conditions of the finite system. The no flow condition across the boundary is fulfilled by placing an imaginary discharging well of equal strength at equal distance and on the other side of the boundary along the axis of the real well. Interfacing of the cone

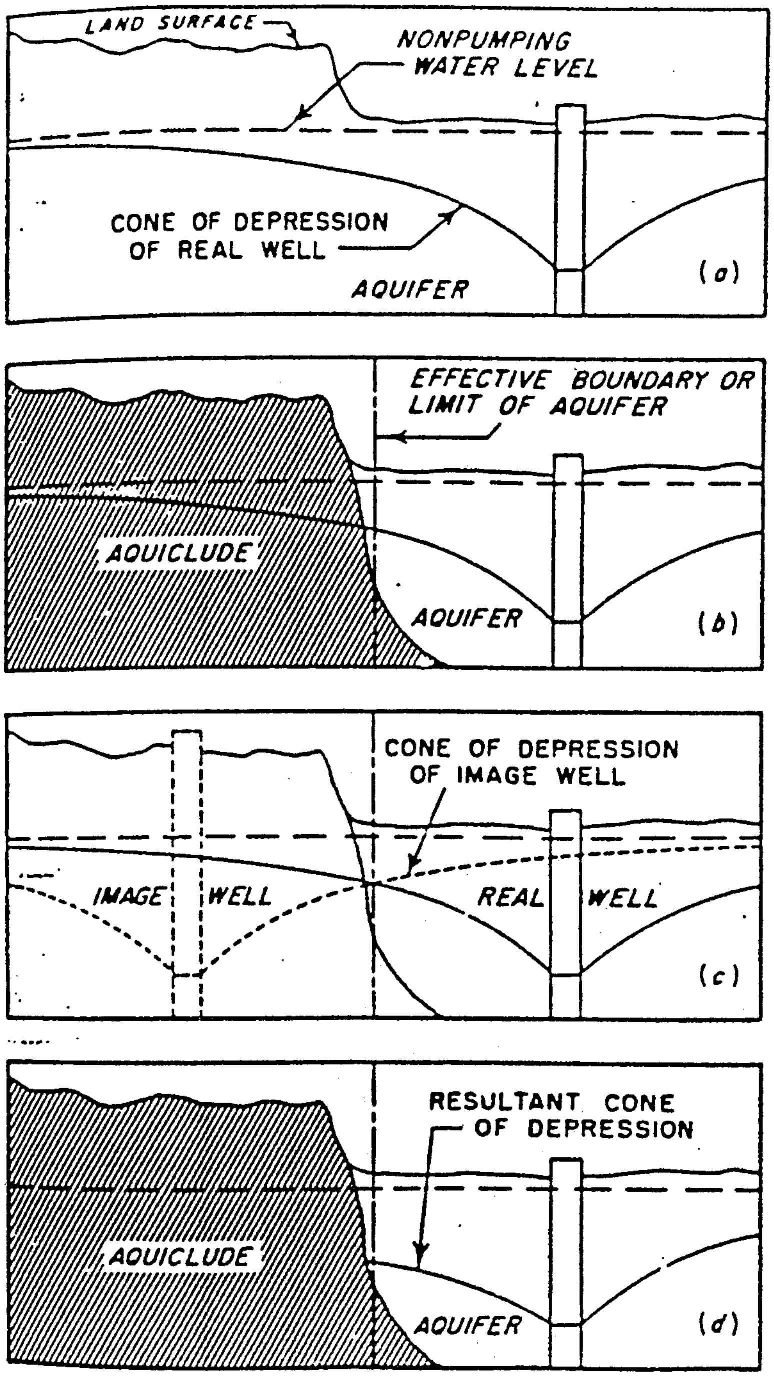


Fig. 24. Influence of Barrier Boundary.

of depressions of the real well and the imaginary well produces a ground water divide at the barrier line, that satisfies the no flow condition. Actual draw down in the barrier is obtained by summing up the drawdowns caused by the real well and the image well. Fig. 24.d shows the resultant drawdown for such a barrier condition

Fig. 25 shows a recharge boundary. The recharge boundary is formed by an infinite perennial stream. As the cone of depression can not extend beyond the stream, there is no drawdown along the effective line of recharge, somewhere away from the bank. The system is simulated by placing a recharging image well at equal distance and on the other side of the boundary line along the axis of the real wall. The recharge well produces an inverted cone of impression and the real well produces a cone of depression along the boundary.

In other words, the imaginary well produces a build-up and the real well produces a drew down at the boundary line. The resultant real draw down at the boundary is obtained by summing up the build-up due to the image well and the drawdown due to the real well. The resultant drawdown of the real well is steeper on the river side and flatter on the country side than it would be if no such boundary was present. For equal discharge, the with-boundary drawdown in the well is less than no-boundary drawdown in the well. Since discharge is directly proportional to the drawdown, the no-boundary drawdown in the well can be maintained by increasing the discharge of the wells near the recharge boundary by approximately the ratio of no-boundary drawdown S_w and the with-boundary drawdown S_{wb}. In other words,

$$\bar{Q} = Q \frac{S_w}{S_{wb}} \qquad \text{.......................................(38)}$$

$\bar{Q}$ = design discharge

Q = initial discharge

The discharges can also be estimated from flow net analysis as in Fig. 26.

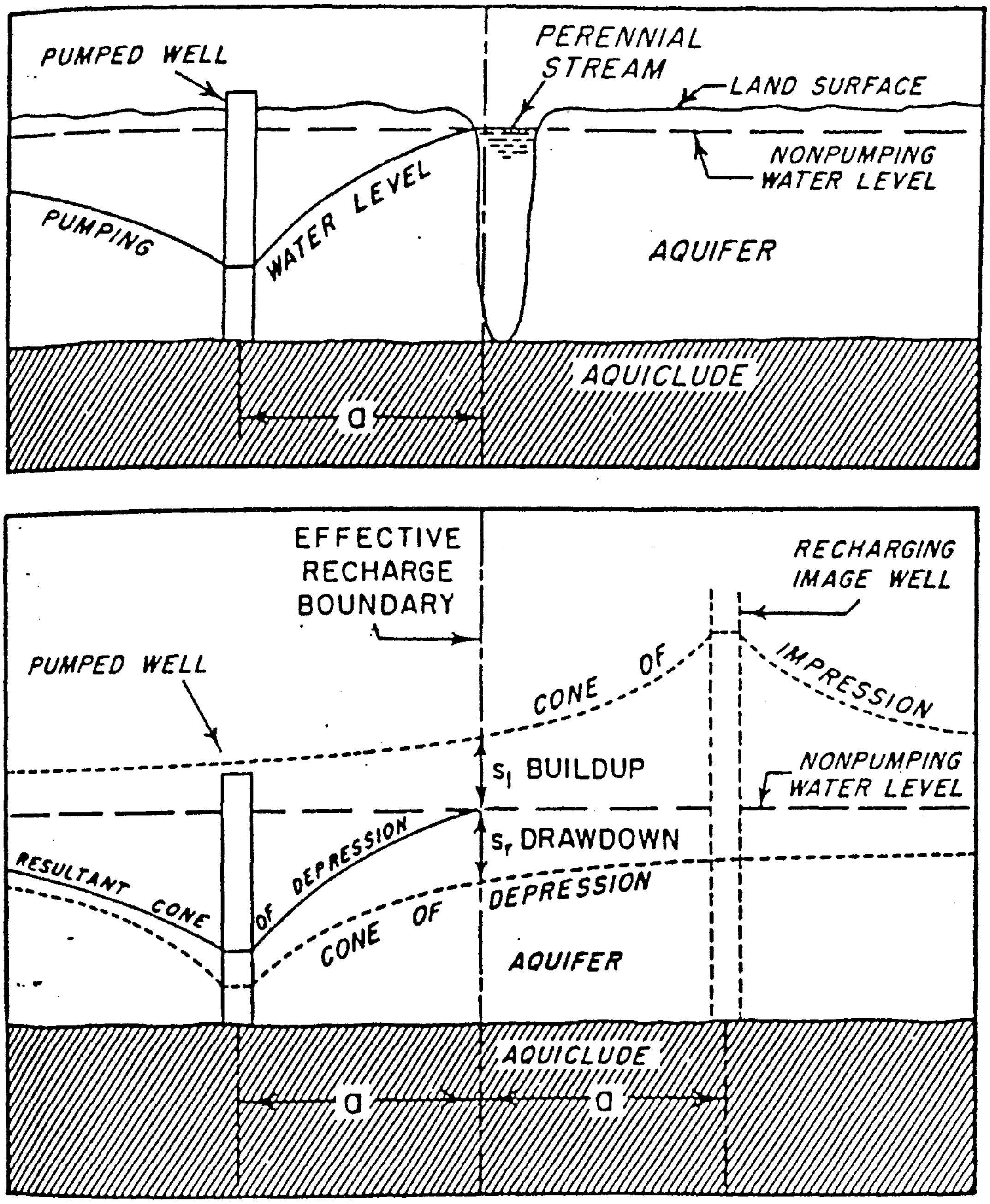


Fig. 25. Influence of Recharge Boundary.

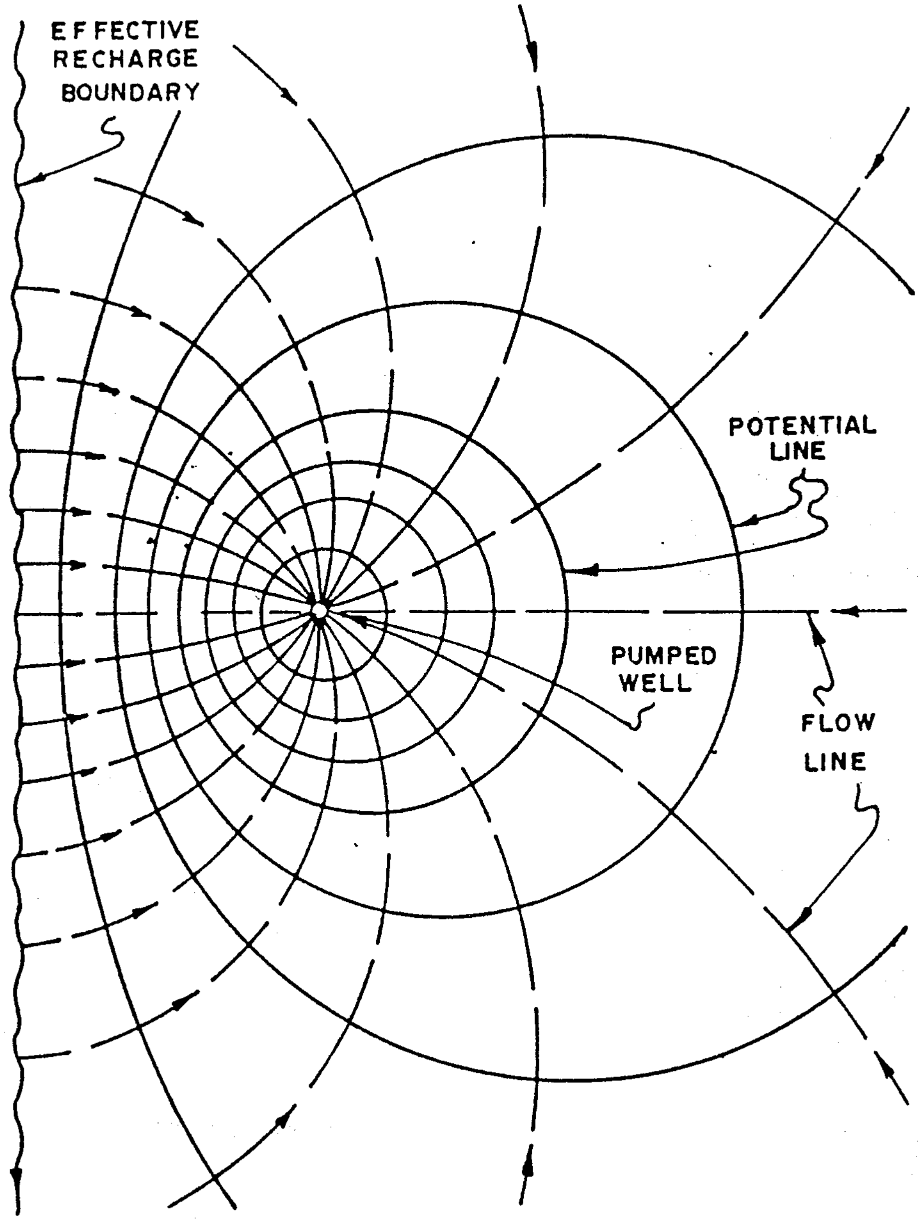


Fig. 26. Recharge Boundary - Discharge by Flownet Theory.

Flow net is a graphical solution of Lapplaces equation for steady state flow in uniform isotropic soils i.e.

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta z^2} = 0$$

The equation states that gradient change in x direction $\delta^2 h / \delta x^2$ plus gradient change is z direction $\delta^2 h / \delta z^2$ is equal to zero. The equation represents a family of two groups of curves intersecting in x-z plane. One group represents flow lines or stream lines and second group represents constant head lines or equipotential lines. The two groups intersect at right angles and form a set of squares or nearly squares (curvilinear or straight). Right angle intersection is necessary because in this situation the flow length is minimum and the hydraulic gradient is maximum. The flow has a tendency to move along the maximum gradient which is always perpendicular to the equipotential lines. The equipotential lines at any time represent the sum of pressure head and elevation head. Velocity head is usually neglected.

Flow nets can be drawn experimentally through model studies or by graphical method. The graphical method is a trial and error procedure and is most commonly used.

A brief outline of constructing flownets by trial and error method is given below:

1. Draw the hydraulic system and if needed, the soil profiles, to a convenient scale.
2. Establish the boundary conditions, usually two boundary flow lines (e.g. two perpendicular flow lines along the well axis) and two boundary equipotential lines (e.g. one along the recharge line and the other around the well casing).
3. Sketch one flow line or one equipotential line adjacent to a boundary flow line or a boundary equipotential line. The lines must intersect at right angles.

4. Continue sketching equipotential lines and flow lines, keeping in mind that roughly square figures are developed in this process. It may happen that many of the figures are far from squares, even then it will give sufficiently accurate result for practical purposes.
5. The first trial may not look acceptable. Give two or more trials correcting angles and shapes of figures. Usually three to ten flow lines are sufficient for many cases.
6. If the soil mass is not isotropic, transform the horizontal scale multiplying by a ratio $\sqrt{(K_z/K_x)}$, where K_z and K_x are vertical and horizontal permeabilities respectively.

After constructing the flow net the discharge can be estimated as follows:

1. Let us consider Fig. 26. Total flow may be thought of being equal to the product of number of flow paths n_f and the quantity of flow dq per flow path. Thus,

$$q = n_f . dq \qquad \text{.......................} \quad (39)$$

2. Total head h, is equal to the product of all equipotential drops n_d and the incremental head loss dh. Thus,

$$h = n_d . dh \qquad \text{.......................} \quad (40)$$

3. According to Darcy's law, flow through any square (perpendicular to equipotential lines) is :

$$dq = K\ i\ b = K . \frac{dh}{l} . b$$

$$= K.dh. \frac{b}{l}, \qquad \frac{b}{l} = 1 \text{ (as the length \& breadth of squares are assumed to be equal)}$$

$$= K.dh.$$

From equation 39 & 40

$$
\begin{aligned}
q &= n_f \cdot dq \\
&= n_f \cdot k \cdot dh \\
&= n_f \cdot K \cdot \frac{h}{n_d} \\
&= Kh \left(\frac{n_f}{n_d}\right)
\end{aligned}
$$

Above equation shows that the flow depends on the ratio of n_f & n_d, rather than their actual numbers. If properly drawn, irrespective of the number of flow paths & equipotential drops, the ratio of n_f/n_d should remain constant.

3.3.5 Flow to an infinite trench

For wells we assume that the flow comes from a cylindrical source at a distance equal to the radius of influence r_o, from the well. But for many dewatering problems, where the length of the excavation pit is several times the breadth, it is convenient to transfer the pit into an equivalent trench rather than an equivalent well. From Darcy's law, the flow from one side of an infinite trench for unit length is given by:

$$\frac{Q}{x} = K.A. \frac{dh}{dl} \quad \text{.................... (41)}$$

For confined aquifer with depth B, Fig. 27.a.

$$
\begin{aligned}
\frac{Q}{x} &= K.B.1. \frac{H-h}{L} \\
&= \frac{KB\ (H-h)}{L} \quad \text{.................... (42)}
\end{aligned}
$$

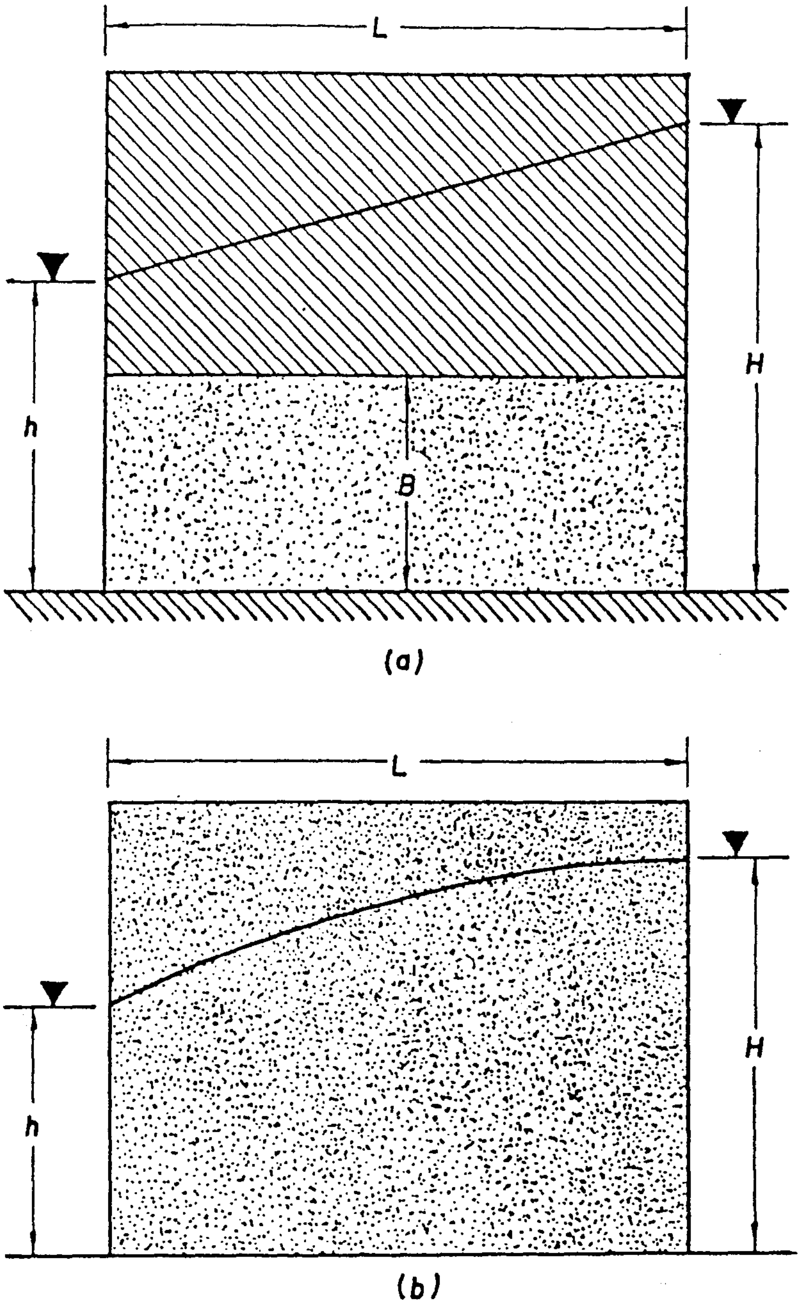


Fig. 27. Flow to Infinite Trench a) Confined Aquifer
b) Unconfined Aquifer

For unconfined aquifer, Fig. 27.b.

$$\frac{Q}{x} = K.\left(\frac{H+h}{2}\right).\,1\,.\left(\frac{H-h}{L}\right) \qquad \ldots\ldots\ldots\ldots\ldots\ldots (43)$$

$$= K.\left(\frac{H^2 - h^2}{2L}\right) \qquad \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

Where, K = permeability of soil

X = length of trench

Q = flow to trench from one side

L = distance of assumed line source from the trench

= $r_o/2$,

It has been observed that a line source at a distance $L = r_o/2$ produces the same effect as a cylindrical source at a distance of r_o, the radius of influence.

3.4.0 POTENTIAL SOURCES OF SEEPAGE

Seepage from surface water sources or any other sources may significantly affect the volume of dewatering. However, the magnitude of this seepage will depend on the type & proximity of the surface water body and characteristics of the interconnecting media. Since, the hydraulic structures are usually situated near natural water bodies like, ocean, lakes, rivers and canals, influence of seepage from such water bodies on the volume of dewatering must be evaluated prior to installation of the system. Most of the seepage sources can be grouped into the following categories:

3.4.1 Seepage from Surface water Bodies

These water bodies include:

- lakes and reservoirs
- bays and oceans
- rivers and canals
- precipitation and,
- seepage from dewatering disposal.

In case of lakes & reservoirs, and bays and oceans tidal effects on the dewatering volume may be prominent. But there is no simple method to calculate tidal effects on dewatering volume. However, as an approximation, average maximum and minimum tide levels above and below the mean level, may be considered as recharging and discharging line sources and the corresponding dewatering volume can be estimated by image well theories or flownet analysis. If accurate results are desired to schedule the pumping arrangements during high and low tides, the characteristics of tidal waves (period, amplitude and time to peak) in the surface water body should be observed and related to those of the induced wave in the aquifer.

Characteristics of tide waves and induced waves can be observed by installing gage stations and piezometers respectively and taking the time-level records at close time intervals. Usually there is a time lag between the peak of tide wave and that of the induced wave and the magnitude of the latter is much less than that of the former. This time lag and attenuation characteristics depend on the type of soil and its permeability and should be determined experimentally.

Similarly, for rivers and canals the increase in dewatering volume can be estimated by image well theory or flownet analysis. For practical purposes a 10 - 20% increase in dewatering volume will be sufficient to handle the above situations.

If the excavation pit is hydraulically connected to the surface water body by a confined bed of sand, neither sump pumping nor well pumping can handle the flow to pit (Fig. 28). The confined sand bed should be punched by sand drains so that tubewells installed in the lower formation can tap the flow through this confined bed.

However, if the media between the surface water body and the excavation pit is impermeable or the surface water body is beyond the zone of influence of the excavation pit, the effect of the surface water body can be neglected.

3.4.2 Seepage from Existing Structures

This includes leaks from water mains, sanitary sewers and storm sewers and can be of significant importance in built-up areas. The sources can some times be identified by analyzing leaking water. Presence of residual chlorine will indicate a water main while presence of coliform bacteria or very high temperature may indicate a leaking sanitary sewer. Such seepage can usually be dealt with by repairing the utilities or by providing necessary plugs or cutoffs in or around the sources of leakage.

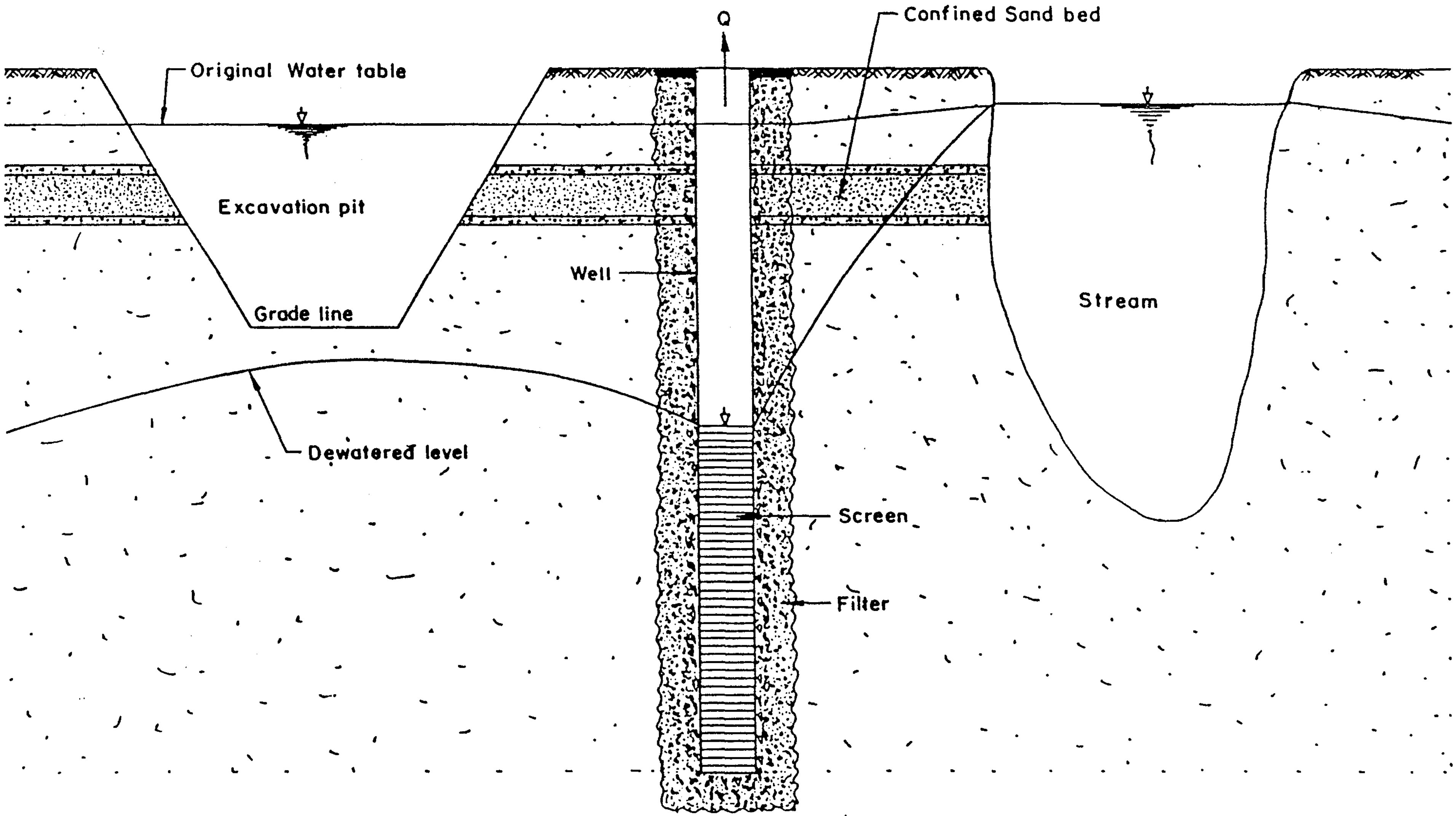


Fig. 28. Stream Hydraulically Connected to Excavation.

4.0.0 DEWATERING SYSTEM PLANNING

This section deals with review of available dewatering methods and their merits & demerits, analysis of sites & determination of dewatering needs, selection of dewatering methods, installation of dewatering system and effects of dewatering etc.

4.1.0 Review of Available Dewatering Methods

The most commonly used dewatering methods are :

a) Sump pumping or open pumping
b) Predrainage or well pumping
   - Shallow tube well method
   - Deep tube well method
   - Well point method
c) Vacuum method
d) Electro-osmosis method

4.1.1 Sump pumping or open pumping

i) Salient features

The sump pumping or open pumping is a very simple method of dewatering that utilizes shallow trenches dug along the outer edges of an excavation pit. The trenches collect and conduct the seepage water to pits or sumps at specific locations, from which the water is pumped out of the excavation site (**Fig. 29a**). This method is suitable where excavation is done in relatively stable & cemented soils, size of the excavation is small and the depth of dewatering is shallow.

Every excavation requires some open pumping to remove rain water. Besides, in many cases where excessive seepage becomes a problem, sump pumping is used to supplement other modes of dewatering like shallow tube wells, deep tube wells or well points, to take out the extra water.

Usually, during excavation, the perimeter ditches are excavated first. This helps control of lateral seepage in the ditch around the perimeter. The central soil mass is then excavated more efficiently and comfortably.

If storage depletion is significant, the perimeter ditches should be excavated well in advance and pumping should be carried out to give the stored water a chance to bleed for some times. To prevent sloughing of sides due to storage depletion, the preliminary slopes are usually kept much flatter and then trimmed back to the final slope after the seepage flow has stopped (**Fig. 29b**).

The ditches should be sufficiently deep and wide to handle the anticipated discharge from dewatering plus incidental overloads. The sides and the beds of the ditches should be lined with gravel filter to prevent erosion which would increase sediment load in the water reaching the sumps. When the sides of the ditches are subject to sloughing (except in the case of clayey soils), it may be necessary to fill the ditch with gravel. The water is then made to flow through the pore spaces of the gravel media (**Fig. 29c**).

If the flow to be handled is larger than the capacity of the gravel filled ditch, its gradient must be increased or it must be equipped with an embedded pipe, perforated or other wise permeable (**Fig. 29c**). In case of a perforated pipe care must be taken so that the fines do not enter the pipe and overload the sump.

The final sump or sumps from which water is pumped out, should be deep enough to handle the drainage from the entire excavation. The approaches to the sump should be so designed that they arrest much of the sediments or fines by sedimentation or filtration. Otherwise, fines carried by water to the sumps will damage the pumping equipments.

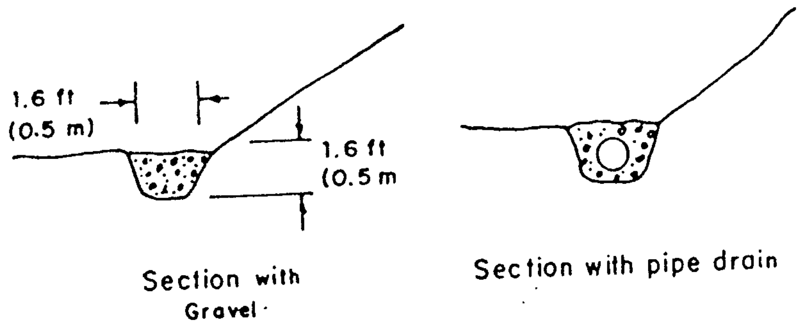


Fig. 29c. Gravel Filled Trench Section

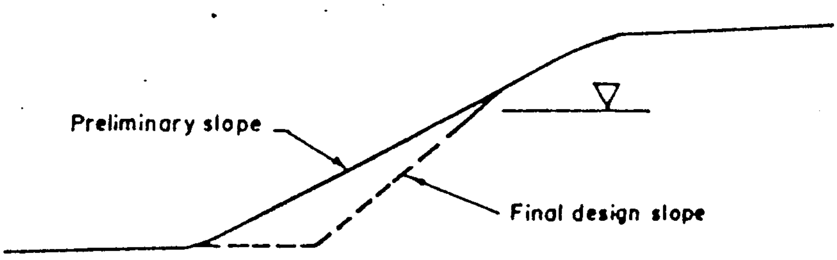


Fig. 29b. Preliminary Slope of Trenches.

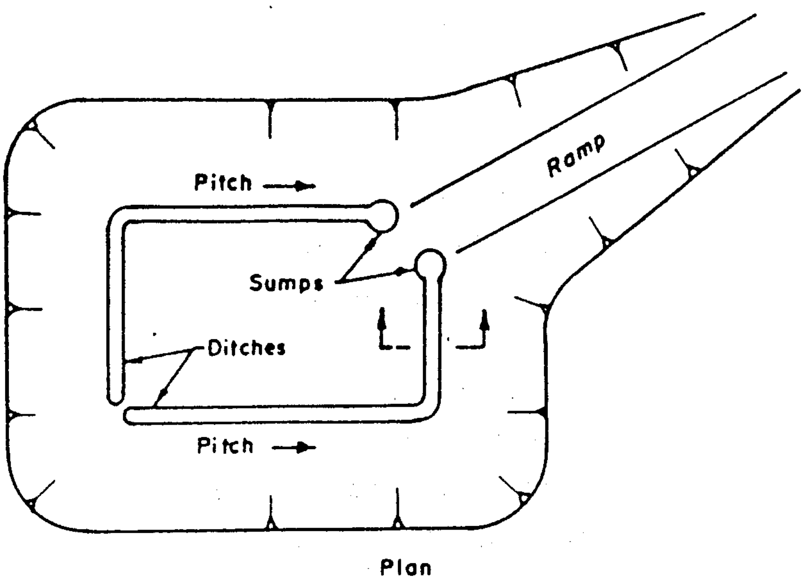


Fig. 29a. Layout Plan of Sump and Dewatering Trenches.

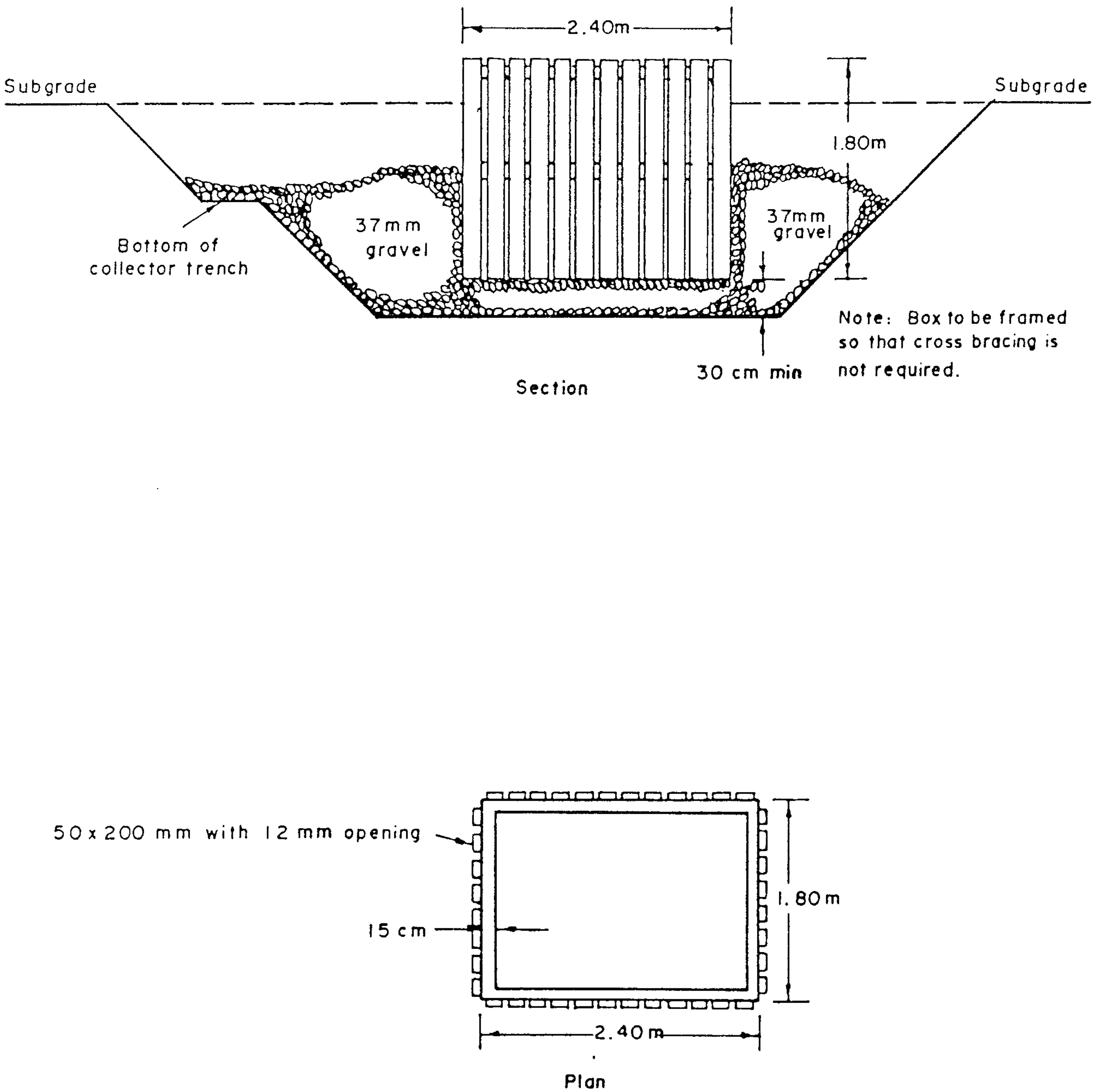


Fig. 30. Detail of Typical Sump. (Dimensions Variable).

The sump should be of ample size so that the velocity of water entering the sump is small and the sediments in suspension settle down. A large sump will provide space for storage of sediments between successive cleaning. **Fig. 30** shows a typical sump with perforated barriers and gravel filter. For small works, a perforated oil drum, filled with brick bats or gravels will also serve the purpose.

As the fines are gradually accumulated in the filters, after a period of time the filters get clogged and stop the flow of water. Therefore, the filters should be periodically removed, cleaned or replaced.

ii) Flow through trenches

The flow through the open trenches or trenches with embedded pipes are approximated by Mannings formula and that through gravel filled trenches is estimated by Darcy's formula:

Trench with embedded pipe

$$Q_t = \frac{1.49}{n} A R^{2/3} S^{\frac{1}{2}} \text{ ; (Fps)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

$$= \frac{1}{n} A R^{2/3} S^{\frac{1}{2}} \text{ ; (MKS)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

Trench filled with gravel

$$Q_t = K.A.I \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

Where, n = roughness coefficient of trench or pipe
A = cross sectional area of trench or pipe
R = hydraulic radius of trench/pipe
S = slope of trench/pipe
K = permeability of gravel filled trench.
Q_t = capacity of trench

The load to each trench is calculated from :

$$Q_t = \frac{Q}{n_t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

Where, Q_t = drainage load to each trench/time

Q = total volume to be dewatered/time.

n_t = number of channels or trenches

The size of the trenches may increase towards the down stream direction or they may be of uniform cross section, that will handle the largest flow.

iii) Problems associated with sump pumping

The following problems are usually associated with sump pumping:

a) Sand boils

If the soil in the subgrade contains pockets of fine sand and the gradient of seepage flow exceeds the safe exit gradient the soil particles lose all their weight and become quick. Under this situation mixture of soil and water blows upward just like a mini volcano. The boils are dangerous as because they lead to undermining of the subsurface soils and subsequent formation of tunnels or soil pipes. These tunnels or soil pipes may in turn cause subsidence of the surrounding soil or the structures.

Once the boils have been observed in an excavation, pumping should be stopped and a dyke of earth or sand bags should be constructed around the boil. The area so enclosed by dykes should then be flooded to a level, high enough to balance the seepage head. Alternatively a drum, with perforated bottom and filled with brick bats as filter, should be placed over the boils (**Fig. 31a**). Clear water should than be pumped from the drum. After the boil has been temporarily stopped, the hydraulic head should be released by predrainage with wells or well points. The boils should be finally sealed with concrete casting.

b) <u>Ground heaving</u>

If the excavation site consists of a layer of sand or other coarse grained soils separated from an underlying aquifer by a thin layer of silt or clay, lowering of water table in the upper water bearing strata will not relieve the pressure in the confined underlying aquifer (**Fig.31b**). A piezometer installed in the underlying aquifer will show the original pressure head. If h_2 is the depth of water in the piezometer, the confining layer is subjected to a hydrostatic pressure equal to $\gamma_w h_2$. The critical uplift pressure on the soil above line a--a will be equal to the weight of overlying soil i.e $\gamma_s h_1$. If $\gamma_s h_1 \leq \gamma_w h_2$, then the soil in the bottom of the excavation becomes unstable and heaves upward.

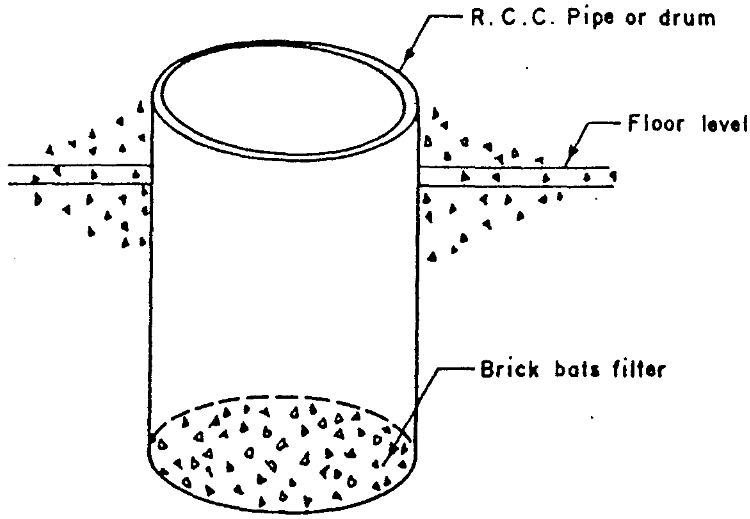
If the clay layer has uniform thickness, then the bottom heaves bodily. Otherwise it causes local heaves or boils at points of minimum h_1.

The critical depth of excavation, h_c, below the piezometric head of underlying aquifer is given by:

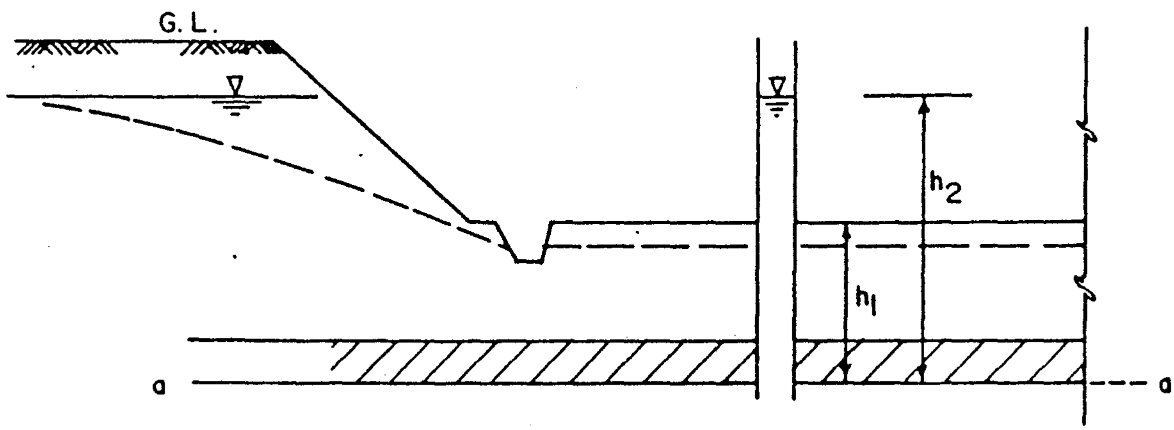
$$h_c = h_2 - h_1 = \left(\frac{\gamma_s}{\gamma_w} - 1\right) h_1 \qquad (49)$$

Where,
- h_1 = height of grade line above the bottom of clay layer.
- h_2 = pressure head above the bottom of clay layer.
- h = depth of excavation below piezometric level
- γ_w = unit weight of water
- γ_s = unit weight of soil (moist)

To avoid ground heaving, the depth of excavation below the piezometric level should always be less than the critical depth, h_c. If this condition can not be satisfied, the underlying aquifer should also be drained by vertical sand drains, well points or deep wells, so that the pressure head is lowered below the critical condition.



a) Perporated Drum over Boils.



Confined Aquifer

b) Mechanism of Ground Heaving.

Fig. 31. Boils and Ground Heaving.

4.1.2 Predrainage or well pumping

In predrainage method the water table is lowered before excavation of the soil while in sump pumping water is removed during excavation. In case of predrainage sufficient number of piezometers should be installed to ascertain the desired lowering of water table before advancing excavation to the final grade. There are two principal types of predrainage techniques, Tube well pumping and Well point pumping, which are briefly discussed in the following sections.

i) Tube well pumping

a) Salient features

The wells are installed in or around the periphery of the excavation. As the water is pumped, water level in and around the wells drops down and the seepage of ground water into the excavation pit is cutoff.

There are two types of tube wells, shallow tube wells and deep tube wells. The definition of a shallow and a deep tube well is rather conventional. On a regional basis, three aquifers have been identified in Bangladesh, namely,

- Upper aquifer, usually called the composite aquifer
- Main aquifer below the upper aquifer and,
- Deep aquifer below the main aquifer.

The deep aquifer is separated from the main aquifer by clay layers of varied thickness. The main aquifer occurs at a depth ranging from 6m (17 ft) in the northwest to about 83m (250 ft) in the south. Tube wells that draw water from the composite aquifer are usually termed as shallow tube well and those pumping water from the main aquifer are termed as deep tube well.

Shallow tube wells normally vary from 100 mm to 150 mm in diameter and use suction mode centrifugal pumps. Deep tube wells are usually more than 6" in diameter and use submersible pumps or turbine pumps for pumping. As shown in **Fig. 32**, the main components of a tube well are :

- sand plug
- screen
- shaft or column and,
- a housing pipe

The housing pipe accommodates the submersible pump or turbine pump and therefore, is larger than the main shaft or column pipe. A shallow tube well does not need a housing pipe.

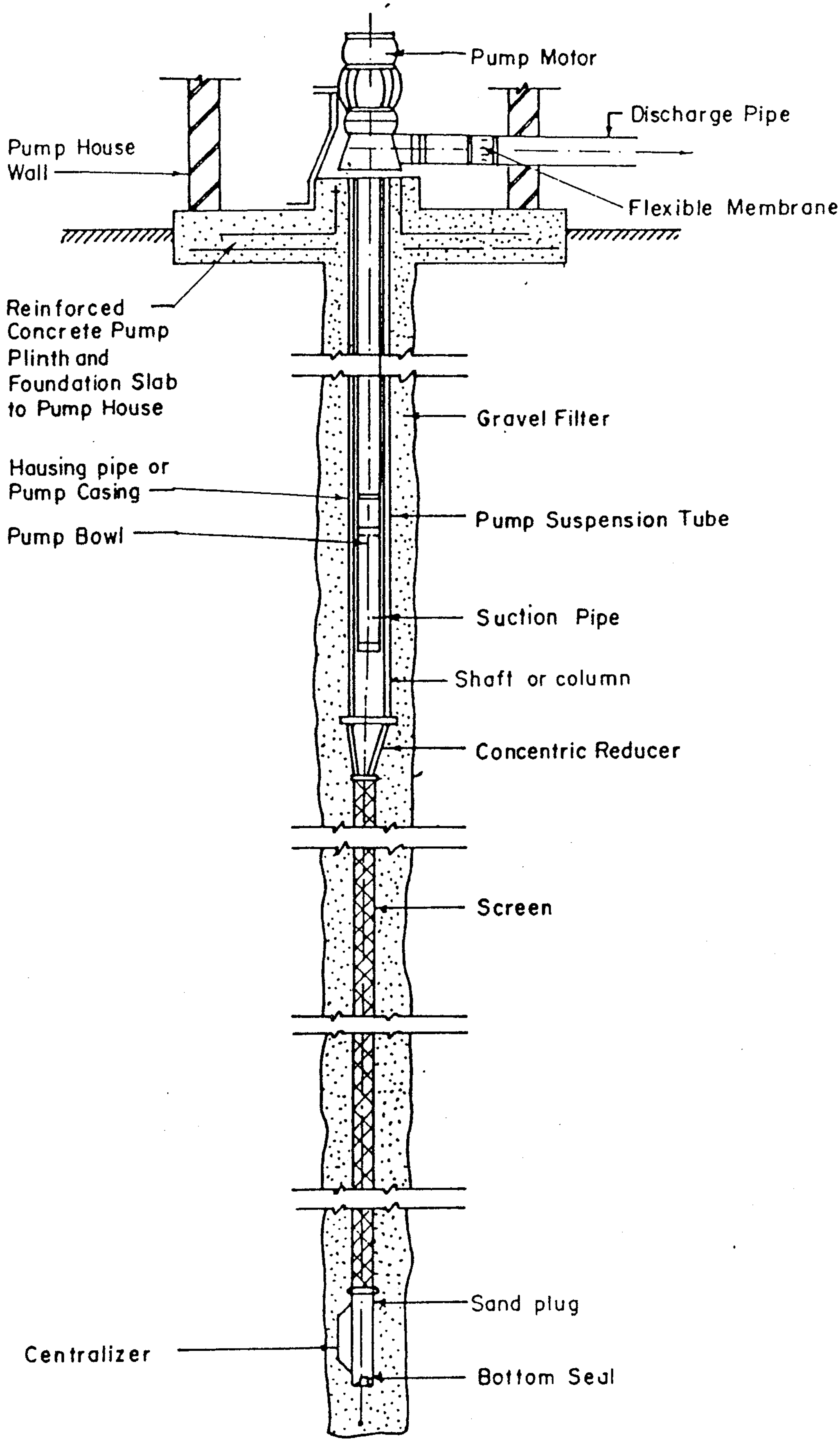
As shallow tube wells use centrifugal pumps, which work in suction mode, their maximum theoretical lifting capacity is limited to 10.36 m of water (34 feet). In practice, this suction limit is reduced to about 7.5 m of water because of losses in the system. However, this difficulty can be overcome if either the pump centre line is lowered or a submersible pump is used.

Deep tube wells can pump water from very large depths. These are suitable if large volumes of water must be withdrawn with significant lowering of water table. Dewatering of a construction site by tube wells mainly constitute the following elements:

b) Well design

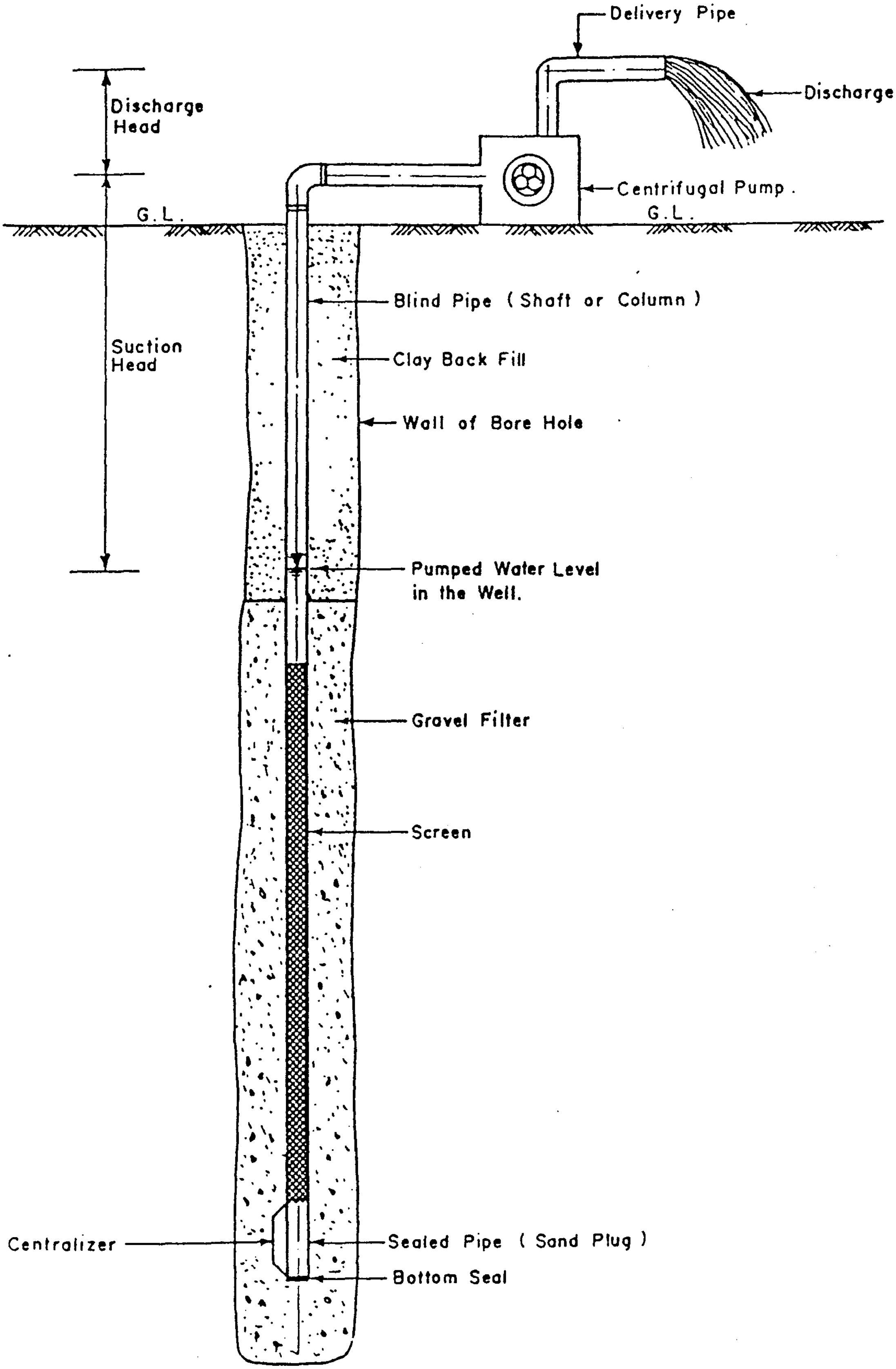
The design of a well depends on the depth & characteristics of the aquifer and the volume of water to be pumped. Selection of a shallow or a deep tube well is a matter of project requirements, cost factors and installation difficulties. The design procedures are similar both for shallow and deep tube wells.

Well design mainly involves design of its screen, through which water enters the well. For design of the screen, geological information of the underlying strata and the grain size analysis results of the aquifer are necessary.



a) Deep Tubewell

Fig. 32. Components of Tubewell.



b) Shallow Tubewell

Fig. 32. (Contd.) Components of a Tubewell

The grain size distribution curves provide effective grain size and uniformity coefficient of particles. These results are very important for calculating slot openings of the screen. The slot openings and the percentage of total open area is usually provided by the manufacturer. The main thing is to select the right size for a particular aquifer condition. Recommended minimum screen diameters are given in **Table 6.** For average conditions with naturally developed filters, guide lines as in **Table 7,** may be followed for selecting screen slot sizes.

However, if gravel pack is used the screen can be designed to retain 90% of the gravel pack material.

For a multilayered aquifer system, the finest and the coarsest layers should be analyzed separately. If D_{50} of the coarsest layer is less than 4 times the D_{50} of the finest layer, a uniform slot or gravel pack size should be used based on the finest material. If the difference between the D_{50} sizes of the finest and the coarsest materials is more than fourfold, slot sizes should be designed for individual layers. Besides, if fine material overlies the coarse material, then the screen for fine material should penetrate minimum 1 meter in to the coarse material and the slot size for the coarse material should not be more the 2 times the slot size for fine material.

Table 6. Recommended Minimum Diameters for Well Casings and Screens (after U.S. Bureau of Reclamation)

Well Yield m^3/day	Nominal Pump Chamber Casing (cm)	Surface Casing Diameter, cm		Nominal Screen Diameter, cm
		Naturally Developed Wells	Gravel-Packed Wells	
<270	15	25	45	5
270-680	20	30	50	10
680-1,900	25	35	55	15
1,900-4,400	30	40	60	20
4,400-7,600	35	45	65	25
7,600-14,000	40	50	70	30
14,000-19,000	50	60	80	35
19,000-27,000	60	70	90	40

To keep the screens free from encrustation & corrosion and to minimize well losses, entrance velocity of water through the screens is usually kept within specified limits. According to Hunter (1970), this limit should not exceed 3 cm/sec or o.1 ft/sec. On the other hand, as the fine grained soils tend to clog more easily than the coarser ones, field experience shows that higher permeability of aquifer.

Table 7. Selection of screen slot size

Aquifer condition	Slot size equal to sieve size that retains
a) Fine uniform soil uniformity coefficient, Cu <3.	40% of material (D_{40}) for non corrosive ground water and 50% of material (D_{50}) for corrosive ground water.
b) Coarse sand and gravel	D_{30} to D_{50} of the sand fraction
c) Non uniform soils (Cu>6)	D_{30} if overlying soil is stable and D_{60} if overlying soil is unstable.

Source : Herman Bouwer.

Materials can allow higher entrance velocity. Based on this, walton (1962) suggested some representative values of entrance velocity as a function of hydraulic conductivity (**Table 8**).

These velocities suggested by Walton are on the basis of permeability of the material in contact with the well screen. For naturally developed wells,without gravel filter, K, should be the permeability of the aquifer materials.

Table 8. Optimum entrance velocity through well screen

Hydraulic conductivity K, of aquifer, m/day	Optimum entrance velocity through the screen, m/min.
> 250	3.7
250	3.4
200	3.00
160	2.70
120	2.40
100	2.10
80	1.80
60	1.50
40	1.20
20	0.90
< 20	0.60

Source : D. K. Todd.

After selecting the entrance velocity and the slot size the screen length is calculated from the following formula:

$$L_s = \frac{q_w}{C \cdot A_o \cdot V_s} \quad \text{.............................(50)}$$

Where, L_s = length of screen, m

V_s = entrance velocity, m/min

q = well discharge m^3/min.

C = clogging coefficient

= 0.50 (estimated on the assumption of 50% blockage of screen area).

A_o = open area/unit length of commercially available screens, Typical values are given in **Table 9**.

Table 9. Typical Open Areas of Commercially Available Well screens

Nominal Diameter (mm)	Slot Size (mm)	Approximate Open Area (cm^2/m)			
		Continuous Wire	Double Louvre	Slotted PVC	Wire Mesh
100	0.375	584.05	-	188.34	1614.60
	0.75	986.11	124.85	304.72	-
	1.50	1506.68	-	573.46	-
	2.25	1828.33	-	918.40	-
	3.00	2046.29	-	1218.88	-
150	0.375	871.84	-	213.73	2378.52
	0.75	1269.68	146.01	427.45	-
	1.50	1794.48	294.15	852.80	-
	2.25	2262.14	438.03	1280.25	-
	3.00	2607.06	607.33	1705.60	-
200	0.375	831.63	-	275.09	3095.87
	0.75	1466.47	192.57	548.07	-
	1.50	2391.22	393.60	1096.15	-
	2.25	3017.60	584.05	1646.35	-
	3.00	3476.79	810.48	2194.42	-
300	0.375	1252.70	-	427.46	4577.18
	0.75	1724.64	289.91	852.79	-
	1.50	2484.33	588.28	1703.48	-
	2.25	3299.04	873.96	2560.51	-
	3.00	3946.57	1214.05	3413.31	-
450	0.375	1132.13	-	609.44	6462.65
	0.75	2099.20	418.99	1218.89	-
	1.50	3588.95	850.68	2452.59	-
	2.25	4826.88	1263.32	3656.66	-
	3.00	5741.05	1754.27	4602.57	-
600	0.375	1760.62	-	-	8616.86
	0.75	2486.45	548.07	-	-
	1.50	3646.08	1113.08	-	-
	2.25	4975.00	1252.69	-	-
	3.00	6124.06	2293.88	-	-

The screens are usually made of variety of metals & metal alloys, plastics, asbestos-cement, fibre glass reinforced epoxy and coated base metals etc. However, as well screens are susceptible to corrosion and encrustation, nonferrous metals, alloys and plastics are mostly preferred.

The common types of commercially available screens are as follows:

Slotted PVC screen:

Fig. 33.a shows a kind of slotted PVC well screen. Usual screen sizes vary from 100 - 300 mm in diameter and the slot sizes range from 0.010" to 0.100 inch (0.25 mm to 2.5 mm). Smaller sizes are available for piezometers and observation wells. PVC screen is inexpensive and can be conveniently installed with solvent welded couplings. It is resistant to corrosion and can be recommended in encrusting water where acidization may be needed. For normal situation, ASTM schedule 40 wall thickness is widely used. However, for severely loaded deep wells ASTM schedule 80 wall thickness is preferable. As PVC screens are deeply slotted, and may be partly clogged by sand particles, the value of clogging factor C should be carefully selected in calculating screen area, so that the losses can be kept to a reasonable limit.

Continuous slot well screen:

Fig. 33.b shows a continuous slot well screen, with diameters varying from 4" to 36" (100-900mm) and the size of openings ranging from .003 to 0.250 in (.08 to 6 mm). Usual materials of construction are galvanised steel or stainless steel and other suitable alloys. The screen has a high open area with precisely controlled slot sizes. The continuous slot makes development more effective, particularly in the case of shaped wire design (**Fig. 33.b**).

The strength of the screens are such that if handled carefully, shallow screens on temporary jobs can be removed and re-used. Screens are usually assembled by arc welding. Galvanised pipes are moderately priced and are usually used for dewatering services. However, in highly corrosive water or where repeated acidization is needed to remove encrustation, galvanized screens may not be suitable. Stainless steel screens are very costly and may not be always justified.

Louvred screen:

Fig. 33.c shows a louvred well screen, which is formed by piercing and deforming sheet metal. Diameters of such screens vary form 15 - 120 cm with openings ranging from 0.032" to 0.250" (0.75 to 6 mm). Mild steel construction is quite reasonable both cost wise and strength wise. Alloys are rather costly. Steel louvred screens are not suitable in corrosive waters or under repeated acidization.

The slot dimensions of the louvred screen are large and can not be controlled precisely. As such, they are suitable for gravel packed wells where large openings and high velocities are desirable. However, as the slot size can not be smaller than .032" (.75mm), they are not suitable for naturally developed wells.

Wire mesh screen:

Fig. 33.d shows a typical wire mesh screen. The screen is woven with wire mesh and then mounted on a perforated pipe body. This screen is very effective for jetted wells, particularly in finer soils where openings smaller than 0.020 inch (0.5 mm) are needed. However, because of small openings the wire mesh well screen is less effective in drilled wells where extensive development is required.

Miscellaneous screens:

Beside the screens mentioned above, there are many other types that are used for specific jobs or purposes. Among these, slotted fibre glass and continuously slotted plastic screens are occasionally used for dewatering purposes.

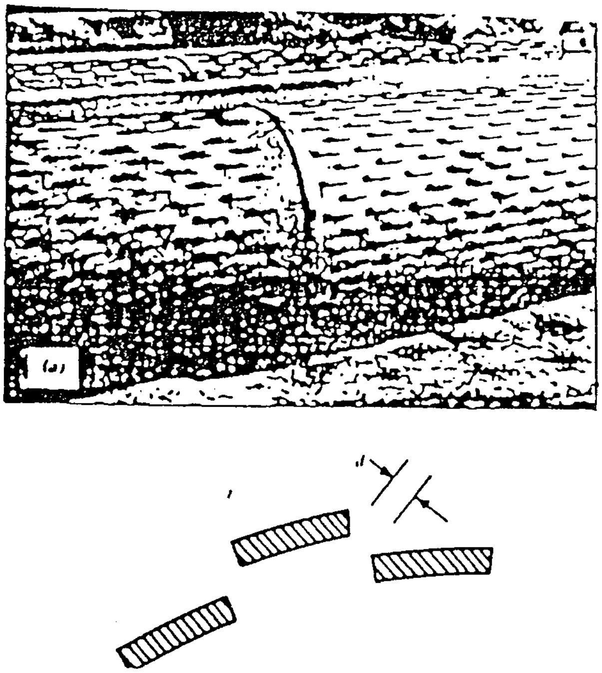


Fig. 33a. Louvered Screen

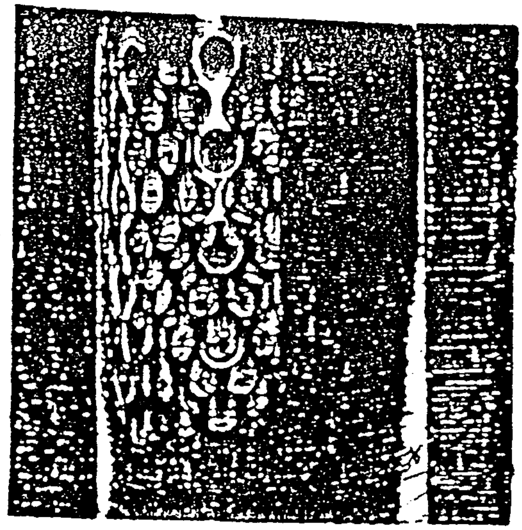


Fig. 33d. Wiremesh Screen.

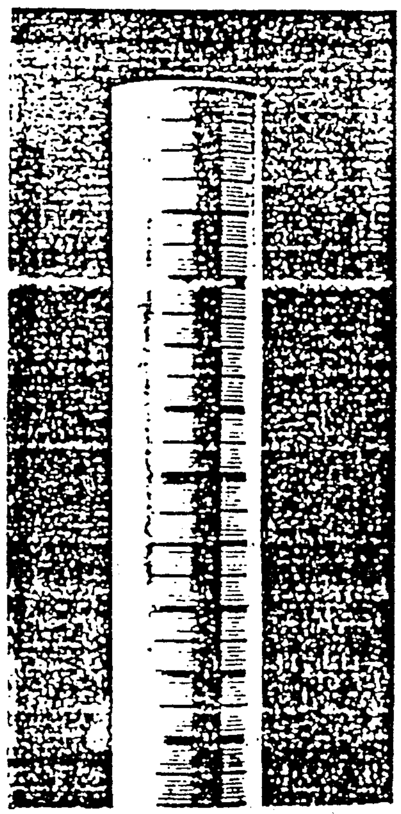


Fig. 33a. Slotted Screen

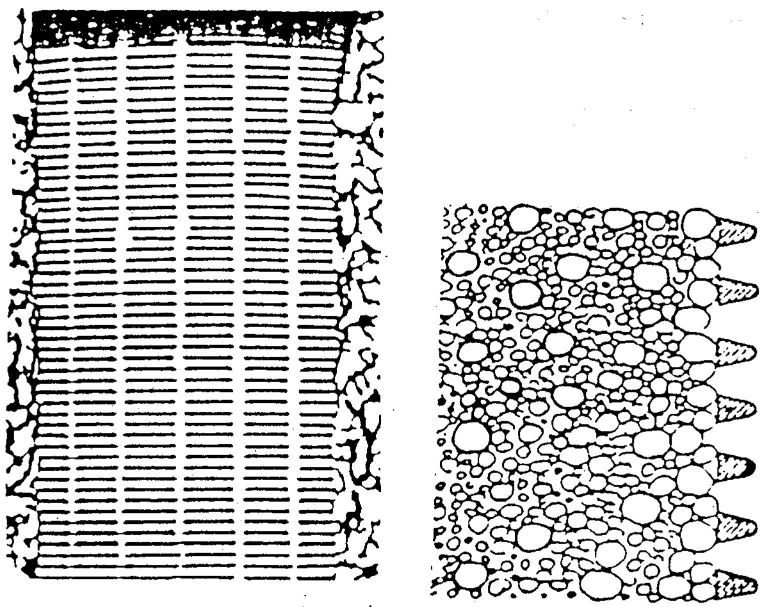


Fig. 33b. Continuous Slot Screen.

c) <u>Gravel pack design</u>

Gravel packing is a kind of gravel envelop formed around the well screen, specially in the case of fine textured aquifers. Besides, the drill hole is always some what larger than the well screen & the shaft pipes and it becomes necessary to fill up the annular gap between the walls of the hole and the outside face of the well pipe with gravel packs. The functions of gravel packs are:

- to stabilize the aquifer
- to minimise sand pumping
- to permit use of large screen slot with maximum open area and
- to provide an annular zone with high permeability which increases the effective well radius and its yield

To perform the above functions gravel pack material should be as uniform as possible so that it has high permeability and can be placed without segregation. For a good filter an uniformity coefficient, C_u, of 3 or less is recommended.

Rounded grains are preferable because they produce high porosity and have little tendency to bridge during placement. Thickness of the gravel pack should vary from 75 mm to 200 mm. If it is less than 75 mm it will not be possible with normal construction procedure to ensure uniform filter thickness around the screen. Again filter thickness greater than 200 mm may create difficulty in developing the walls of drilled hole. Centralizers should be used to keep the filter thickness as even as possible around the hole.

Placement of gravels may be as critical as its selection. Uniform gravels can be directly poured into the annular space between the well and the bore hole, as long as bridging between the screen and the bore hole wall is avoided. Graded gravels can not be poured from the surface because differently sized particles will segregate and form bands of predominantly fine and coarse materials in the envelope, which greatly reduce the effectiveness of the gravel pack. Such problems can be avoided by pouring the gravel pack material through tremie or conductor pipes.

Gravel packs are usually placed upto 3 meter above the well screen. The rest of the bore hole is filled with either clay or cement grouting so that surface water can not seep into the well. To avoid penetration of the grout in the gravel pack, the top of the pack should be covered with a layer of sand or other sealing materials before grouting.

Optimum size of filter particles is a matter of compromise. Coarse filters will allow use of screens with large opening where development will be quick and losses will be minimum. But too coarse filters will permit continuous movement of fines, which is undesirable. Various investigators have developed a number of criteria for selection of suitable filter material. The following criteria are usually followed for selection of gravel pack material (Source J.P. Powers):

- For uniform soil ($C_u \leq 3$) D_{50} size of filter should be 4 to 5 times the D_{50} size of soil.

- For well graded soils (C_u = 4 to 6), D_{50} size of filter should be 5 to 6 times the D_{50} size of soil.

- For very well graded soils ($C_u \geq 7$), where development of some fines are desirable to increase well yield, D_{50} size of filter may be upto 8 times the D_{50} size of soil.

Design of filter packs needs careful judgement. Some of the common methods used for filter and screen design are described below:

1) <u>General method</u>

Fig. 34 illustrates the general method of designing gravel pack. It is a very simple but useful method of gravel pack design. The step by step procedures are:

a) Take a sample of soil from the selected strata.
b) Plot the results of mechanical analysis or sieve analysis as a gradation curve.

c) From this curve find the D_{50}, D_{60} & D_{10} particle sizes and calculate the uniformity coefficient by $C_u = D_{60}/D_{10}$.

$$\text{In this case : } C_u = \frac{0.5}{0.1} = 5, \text{ and } D_{50} = 0.4 \text{ mm.}$$

d) Multiply the D_{50} size of this soil by a factor as per the criteria set above to get the D_{50} size of the filter.

In this case : D_{50} filter = 5 x 0.4 = 2 mm.

e) Plot a new curve through the calculated D_{50} size (in this case 2.0 mm) by trial and error so that the uniformity coefficient of the filter is ≤ 3.0. This will provide the basic curve for filter design. However, since it is difficult to specify filters with a single curve, two more curves on either side of the basic curve and parallel to the basic curve are drawn to specify a range which is usually taken as ± 20% from the basic curve.

In this example the range of particle sizes are specified as:

U.S. Sieve	Percent passing
# 4	92 - 100%
# 10	42 - 64%
# 16	14 - 32%
# 30	0 - 8%
# 50	0 - 2%

The slot size of the screen is designed to pass 10% of the fine limit and 0% of the coarse limit of filter material, in this case 0.63 mm or 0.025 inch.

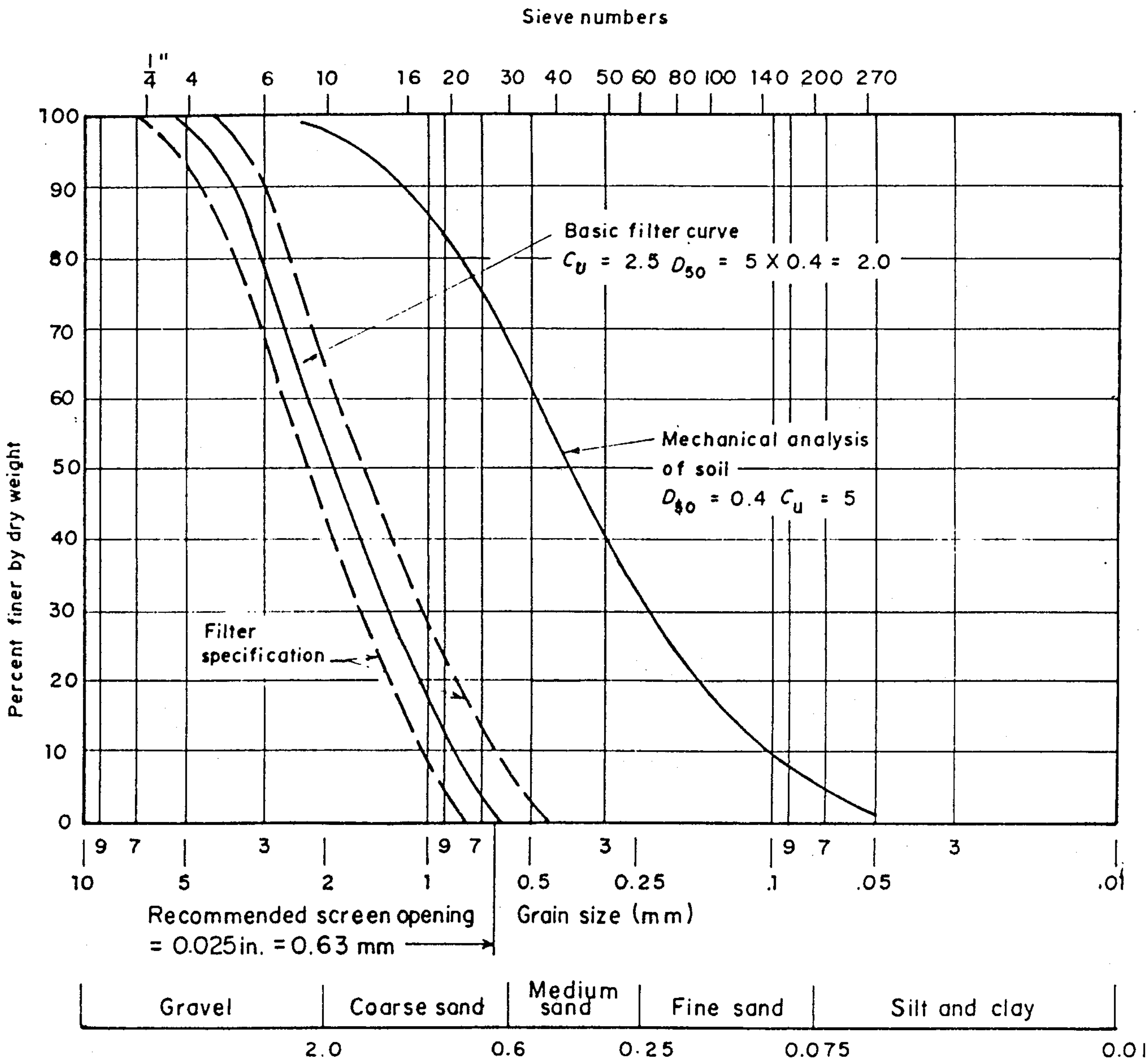


Fig. 34. General Method of Filter Design.

2) **Prugh method**

This is a more precise method of filter selection. Prugh based his criteria on the works of Terzaghi, Smith, leather wood and karpoff Fig. 35. The selection criteria are:

a) D_{50} size of filter should fall between 4 and 5.5 times the D_{50} size of aquifer material.

b) D_{15} size of filter should fall between five times the D_{85} size and four times the D_{15} size of the aquifer.

Maximum size of the filter should guard against continuous movement of fines and the minimum size should ensure free movement of water so that the yield of the well is not reduced. The filter should be selected within the range of (a) & (b) and the screen should be designed to pass 10% of the fine limit.

d) **Boring of wells**

There are several methods of drilling water wells. Shallow wells less than 15 m deep are usually constructed by boring, digging or jetting. Deeper tubewells are constructed by percussion drillings, wash boring and rotary drilling. Some of these methods are briefly discussed below:

Augur boring

In this method the bore hole is advanced by hand or power operated augers with periodic or continuous removal of material. Hand augers are used in soft to stiff cohesive soils & in sandy silty soils above water table. The depth is usually limited to 6 m. Power driven augers can be driven to greater depths, even upto 30 m and can be used for almost all types of soils.

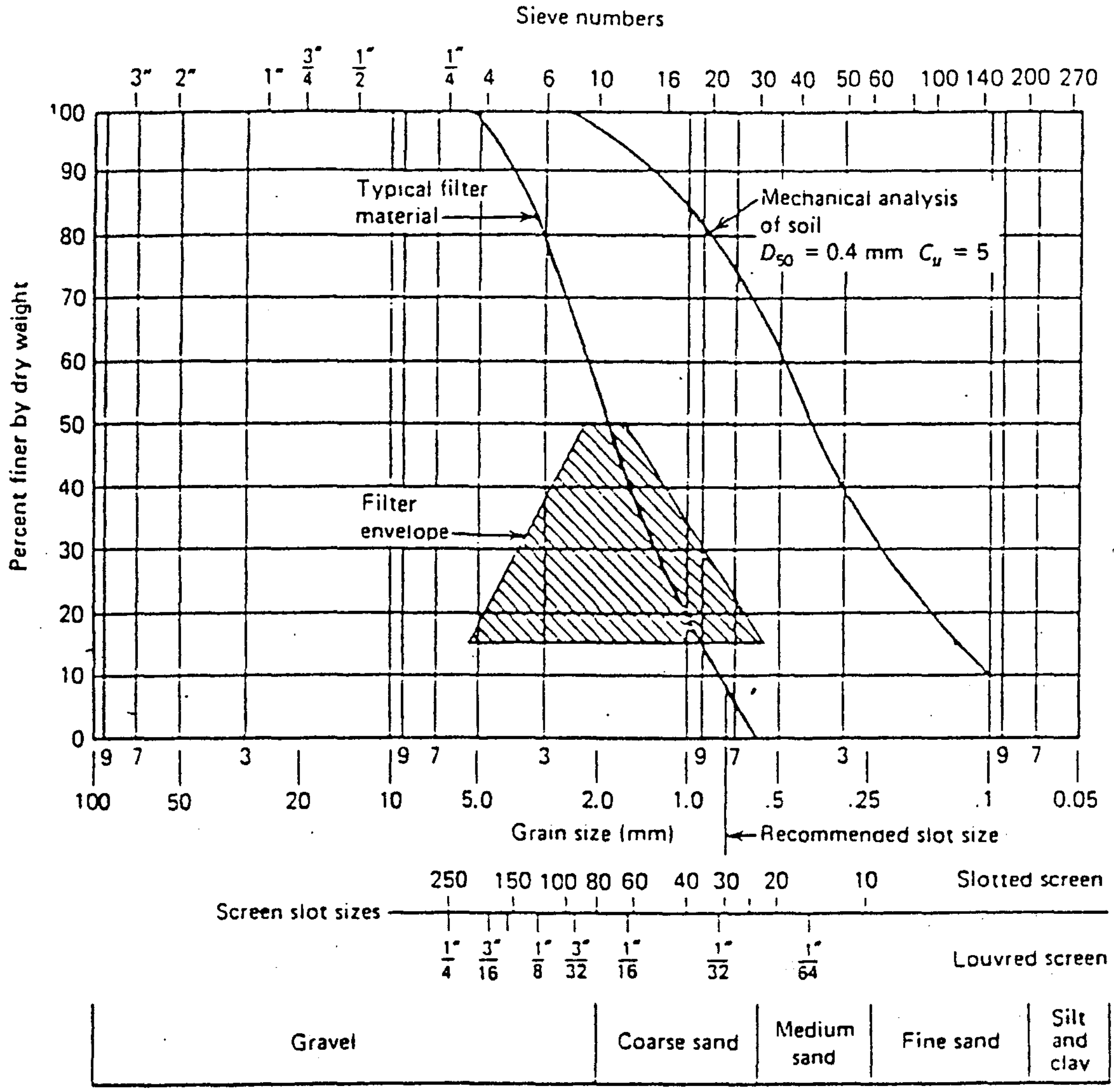


Fig. 35. Prugh Method of Filter Design.

Wash boring

Wash boring can be used for almost all types of soils except those containing boulders. Bore hole is advanced by chopping or twisting action of a light chopping bit and jetting action of a drilling fluid, usually water under pressure. Water stored in a sump is pumped under pressure into the drilling pipe, which jets out of the drilling bit, comes up with the drilled materials through the annular space around the drilled pipe and is then allowed to pass through a settling tank, where it deposits the suspended sediments. From the settling tank water is then allowed to over flow to a clear tank from which it is again recirculated into the drilling pipe. Where recirculation becomes difficult fresh water from a source is directly pumped into the drilling pipe. Casing is usually needed to prevent cave-ins in unconsolidated formation.

Percussion drilling

This method is suitable for almost any type of soil including soils containing boulders. In this method the bore hole is advanced by chopping action of a heavy drill bit driven by power. Water is added at the bottom of the bore hole during chopping action if the water table is not already struck. Slurry formed at the bottom of bore hole is removed by a bailer. Casing is generally needed to prevent cave-ins in unconsolidated formation.

Rotary drilling

This is a very fast method of drilling and can be used in all types of soils including rocks. The bore hole is advanced by power rotation of a drill bit and removal of cuttings by circulating fluids which may be water or bentonite slurry or mud slurry. Diamond or tungsten bits are used when rocks are encountered. In direct rotary method drilling fluid enters through the drilling pipe and comes out through the annular gap and forms a mud cake on the wall of drill hole by filtration. This seals the wall and prevents caving and loss of drilling liquid. In reverse rotary method, circulation of drilling fluid is in the reverse order. This method is suitable for unconsolidated formation. Casing may or may not be needed.

e) <u>Installation of wells</u>

After completion of the bore hole the blind pipes and strainers are lowered in the hole and fixed in position. The whole finished length of the well is then kept suspended, and aligned to the vertical before filling the annular spaces by shrouding materials or gravel packs.

Verticality of the bore hole and the well pipes can checked as per the procedures in **Fig. 36.** The plumb line is suspended from a pulley h meter (3 - 5 m) above the ground level and is made to pass through the centroid of the pipe or bore hole. The plumb thread is marked every h meter. The plumb ring is lowered into the hole or pipe and the deviation Ds (mm) from the pipe centroid on the surface is measured. Then total deviation Dt (mm) at a depth D from the pulley is calculated from:

$$\text{Dt mm} = \frac{\text{Ds (mm)} \times \text{D (m)}}{\text{h (m)}} \qquad \text{.................. (51)}$$

For convenience of rectification verticality checks should be done before placement of gravel packs. According to Sir M. MacDonald and Partners allowable limit for vertical deviation should not exceed 1 in 700 and according to Garg, as he mentioned in his book "Groundwater and Tubewells", the deviation should not exceed 1 in 300 for casings less than 35 cm in diameter. However, irrespective of the limitation set by the experts allowable deviation should be confirm to the maximum tolerance of pumps installed .

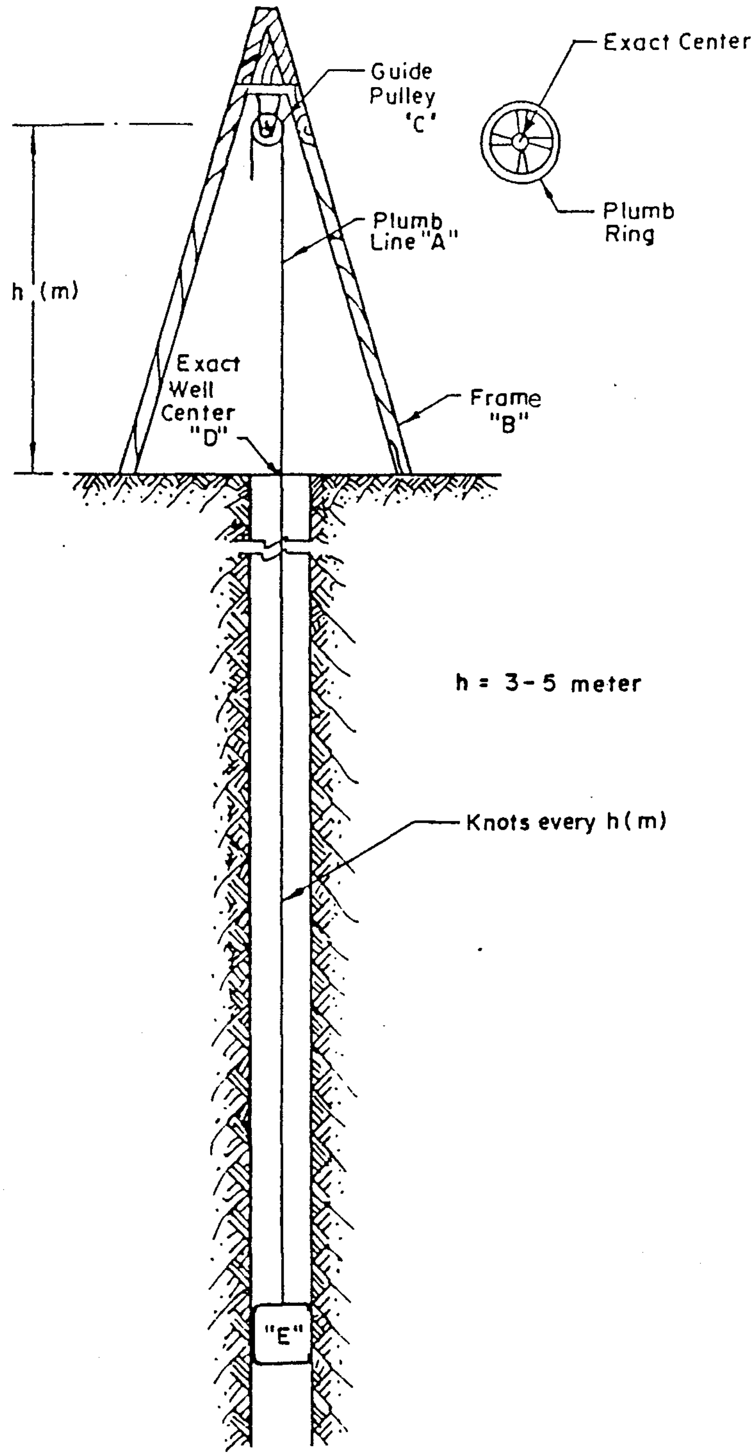


Fig. 36. Method of Plumbing Well.

f) Well development

After installation of a well is completed, it is developed to increase its specific capacity, prevent sanding and obtain maximum economic well life. The process is carried out by removing the finer materials from the natural formation surrounding the screen. Although gravel packing serves much of the above purposes, development is still necessary for better performance. There are several methods of well development. Some of the common methods are discussed in the following sections.

Pumping from well :

Pumping is carried out either singly or in combination with other methods. In this method the well is pumped in steps, initially at very low discharge (high initial discharge will tend to apply sudden pull on the sand particles and can thus cause bridging of particles, which will prevent movement of fines) and then at a higher discharge up to a limit $\geq$ 1.5 times the design discharge. At each step the well is pumped until clear water appears. After this pumping is stopped and water in the well is allowed to surge back into the formation. This irregular and non-continuous pumping agitates the fine materials surrounding the well so that they can be carried into the well and pumped out easily. The coarser fraction entering the well is removed by bailer or sand pump from the bottom.

Surging with block :

This process is carried out by up and down movement of a surge block attached to the bottom of a drill stem (particularly applicable with percussion method). The surge block is 2-5 cm smaller than the well screen and is padded with rubber or leather belts so that it does not damage the screen. As the block is moved up & down a surging action is imported to the water. The down ward stroke causes back washing and dislodging of bridges, while the upstroke pulls the dislodged particles into the well. The initial strokes are confined to the bottom of the screen and then progressively carried up to the top of the screen. This procedure is continued until the materials collected at the bottom of well are negligible.

Surging with air :

In this method the well is fitted with an air pipe and a discharge pipe (**Fig.37**). The air pipe is connected to the air compressor. Initially the air pipe is kept closed and the air pressure is allowed to build up to certain limit. The air is then suddenly released into the well by quick opening of the valve. Sudden surging of air loosens the fine materials surrounding the screen. The loose materials are then brought into the well by continuous injection of air that creates an air lift pump. This operation is repeated until accumulation of fines in the well is negligible.

Hydraulic jetting :

Jetting with a high velocity stream of water is a very effective method of well development (**Fig. 38**). The jet nozzle, mounted horizontally, is attached to a string of pipe, which is connected through a swivel and hose attachment to a high pressure high capacity pump. The jet head is slowly rotated to successively higher levels. It causes movement of fine grained particles from the unconsolidated aquifer to the well by turbulent action. This method is particularly suitable for gravel packed wells.

ii) Well point system

a) Salient features

A well point is a 37 mm to 75 mm diameter pipe usually of short length, varying from 60 cm to 120 cm. The lower end of the pipe has a driving head with water holes for jetting (**Fig. 39**). The well point pipe is perforated and is covered with a screen that allows entry of water with min. possible head loss and at the same time prevents entrance of fine particles from the surrounding formation. The well points are usually connected to 50 to 75 mm diameter pipes known as riser pipes. The upper ends of the riser pipes are connected to a header pipe or suction header pipe through a swing joint. The header pipes, which are usually 15 - 25 cm in diameter, are then connected to pumps.

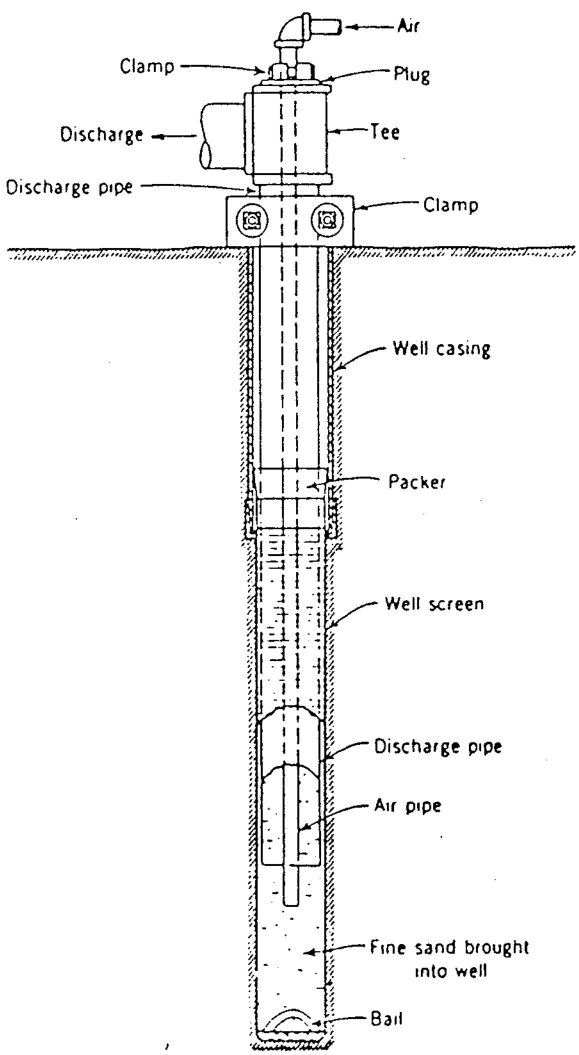


Fig. 37. Surging with Air.

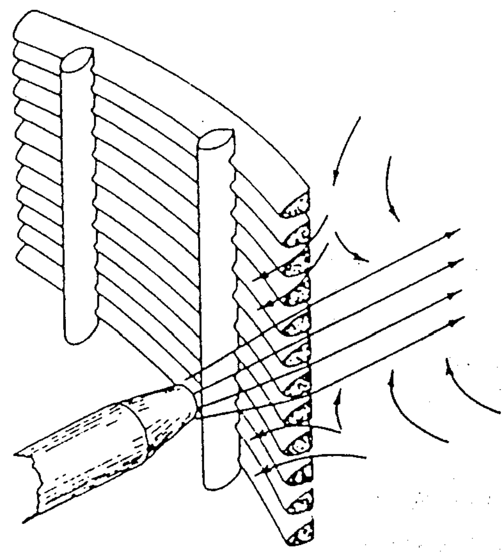


Fig. 38. Hydraulic jetting.

The swing joint is very important for effective functioning of the well points (**Fig. 40**). It consists of a control valve, flexible hose, a disconnecting device, elbows and nipples as required. A control valve is necessary to shut off defective well points and for tuning of well points that are drawing excessive amount of air. The valve must provide a tight shut off, be suitable for throttling and have enough open area to pass the design discharge without much friction. Well points are most suitable for shallow aquifer where lowering of water table does not exceed 4.57 to 6.10 meter.

However, if the depth of water table lowering is more than this limit multistage well points may be installed (**Fig. 41**) as single stage pumping becomes ineffective due to suction limitation.

This system is very good for sandy soils. For highly pervious soil like gravels, the well point spacing required to handle the volume of water may be so close that the system becomes impractical. It is not so suitable for draining clayey soils because of very slow seepage rate. In stratified soils, well points do not draw water from all the strata above. To facilitate drainage of all the strata vertical sand drains ranging from 30 -60 cm in dia. are installed at a spacing of 3.66 m to 4.50 m c/c within the zone of influence (**Fig. 42**).

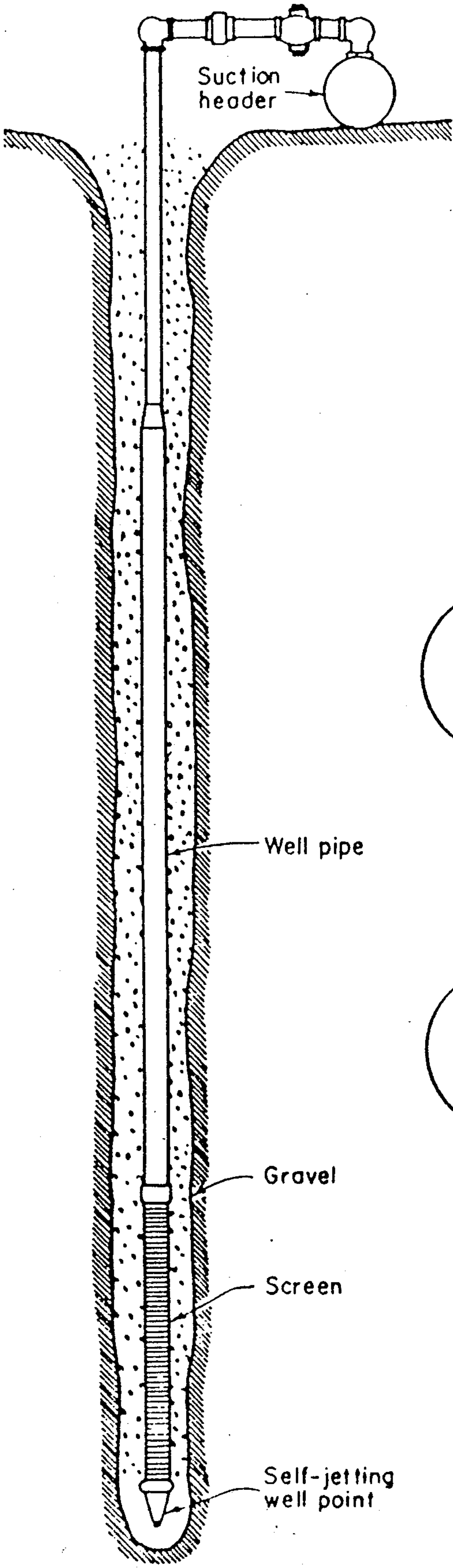


Fig. 39. Self Jetting Well Point.

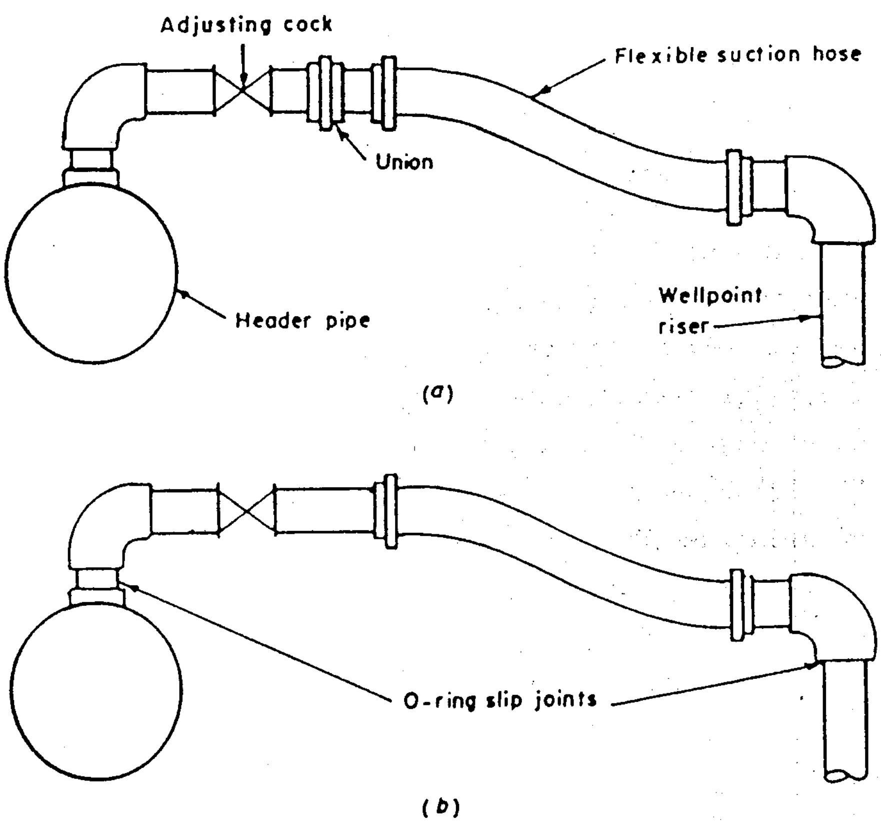


Fig. 40. Swing Joints.

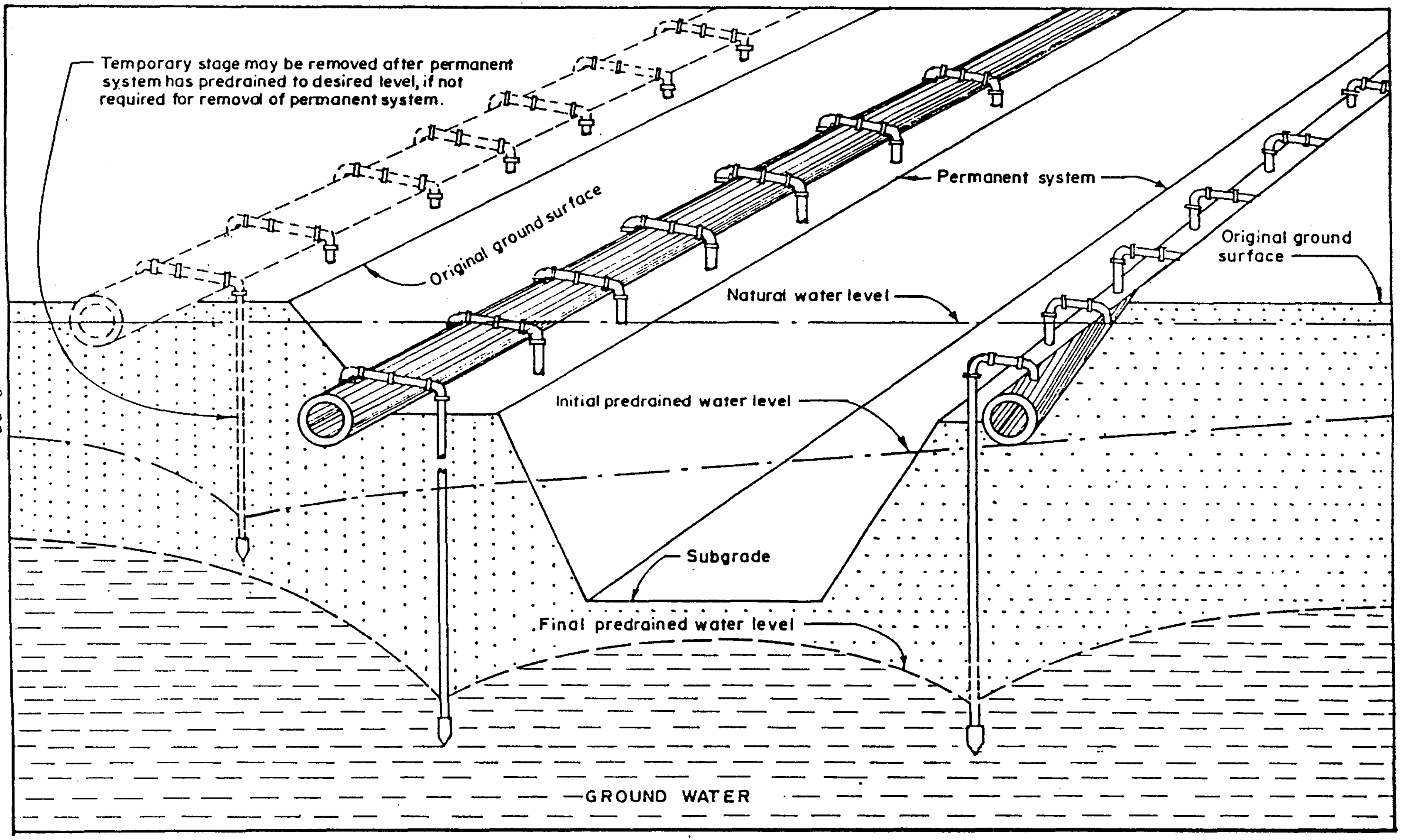


Fig. 41. Multistage Well Point.

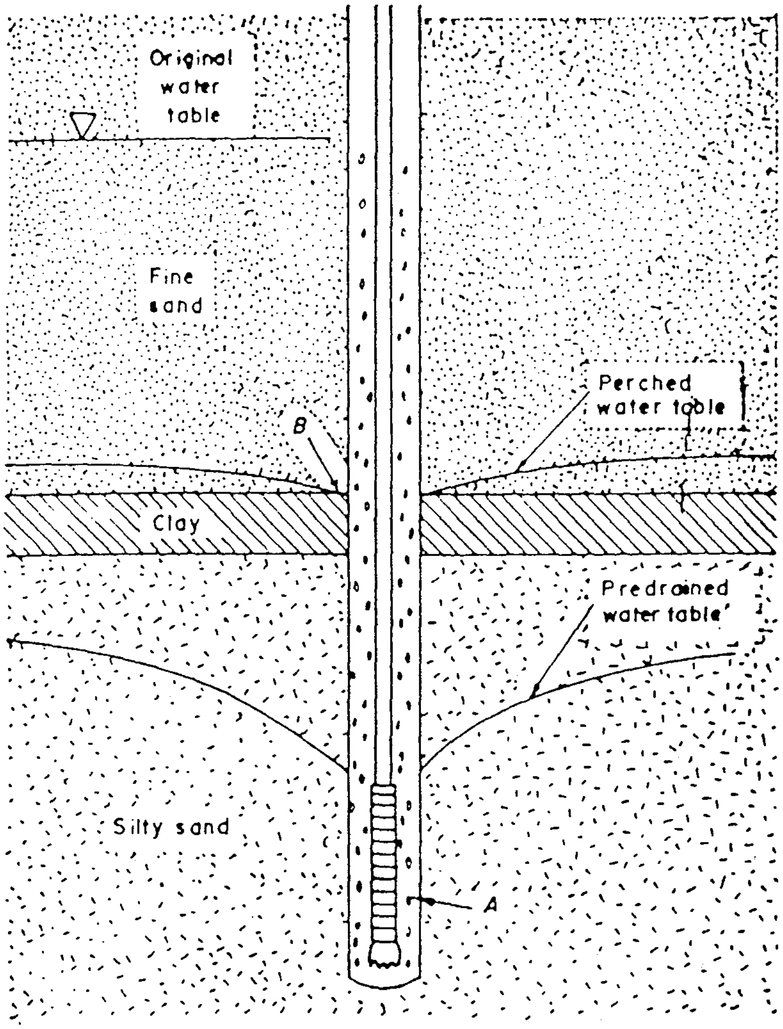


Fig. 42. a. Filter Column Through Clay Layer.

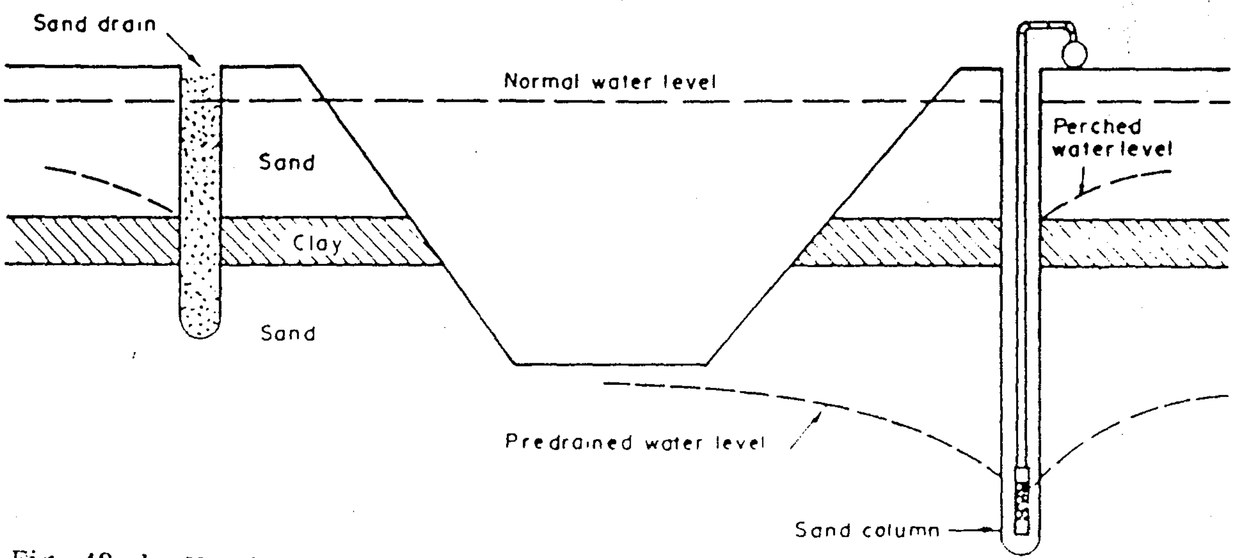


Fig. 42. b. Vertical Sand Drain.

The spacing of well points usually varies from 90 cm to 3.05 m, which depends on the type of soil. Nominal capacities of well points are given in **Table 10.**

Table 10. Nominal capacities of well points

Diameter (mm)	Capacity (gpm/per point)	Remarks
38 mm ϕ self jetting type	10-15	Screens of self jetted well points are made of heavy wire mesh, slotted plastic, perforated plates or continuous wire wrapping with 10-40% open areas.
50 mm ϕ ,,	up to 25	- do -
50 mm ϕ high capacity points	up to 35	- do -
75 mm ϕ points	≥ 35	- do -

Head loss through well points depends on the size of points, discharge and entrance resistance through the screen. Approximate friction loss for flow through various size well points can be obtained from **Table 11.**

Table 11. Friction loss in Well points in Feet of H_2O

Yield per Well point(gpm)							
Well point Type	5	10	20	30	40	50	60
37 mm. Self Jetting Wellpoint	0.36	0.9	3.12	7.48			
50 mm. Self Jetting Wellpoint		0.7	2.35	4.87	9.57		
50 mm. Moreflow,Drawdown Type			2.31	2.88	4.57	5.70	
50 mm. Moreflow, Open Type				1.58	2.71	3.28	4.40
100 mm. Suction Well					2.08		3.77

Vertical location of the well points should be based on adequate information on soil conditions. Well points as deep as 35 m are not uncommon in deep pressure relief installations. **Fig. 43** shows recommended well point depths under various conditions. For uniform soils the top of the screen is placed min. 1.25 m to 1.5 m below the subgrade or level of excavation. If the formation contains a clay layer at or above the subgrade level, the top of the screen should be placed ± 15 cm above the top of clay. On the other hand if deep coarse layer is encountered below the subgrade, the screen of the well point should be placed at the coarse layer.

Suction limit of well points depends on the vacuum that can be produced in the system by a pump. Theoretically, under perfect conditions a vacuum equivalent to 75 cm of Hg can be produced (at sea level). But in practice this limit of vacuum can not be attained due to many reasons like pump imperfection, cavitation, leakage in the system, friction losses, well losses & vapour pressure etc. Besides, if the water table is lowered to near the tip of well points, air enters through the well point screens and breaks the vacuum. Standard well point pumps can not lower the absolute pressure below 12.5 cm of Hg or 1.70 m of water. Therefore, the vacuum that they can produced is equal to the atmospheric pressure less the absolute pressure & other losses in the system.

$$h_{vac} = h_{atmos} - h_{abs} - h_{loss} \quad \text{..........................(52)}$$

Where, h_{atmos} = atmospheric pressure (34 ft or 10.4 m of water)

h_{abs} = absolute pressure in the system

h_{loss} = losses in the system

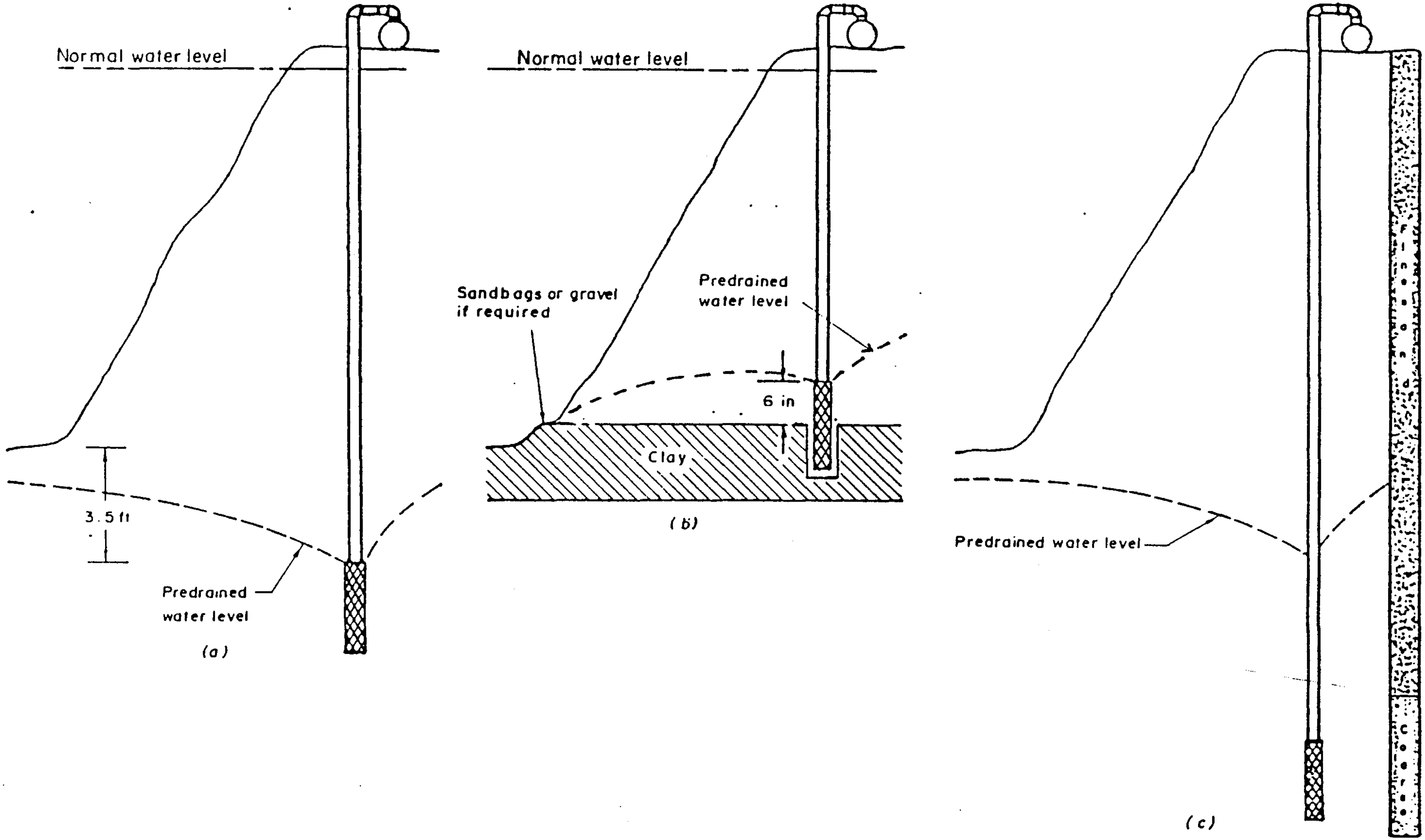


Fig. 43. Recommended Well Point Depths under various condition.

Example:-

Let the abs. pressure (h_{abs}) needed for a pump be = 150 cm (5 ft) of water. Total system loss including vapour pressure etc. = 150 cm (5 ft) of water and atmospheric pressure at sea level = 10.4 m (34 ft) of water.

Then,

the vacuum that can be produced by the pump is :-

$$
\begin{aligned}
h_{vac} &= h_{atmos} - h_{abs} - h_{loss} \\
&= 10.4 - 1.5 - 1.5 \\
&= 7.4 \text{ m (24 ft) of water.}
\end{aligned}
$$

In reality well points can hardly function efficiently above suction limits of 4.5 m (15 feet). Since a well point system usually consists of long lengths of piping like riser pipes and header pipes, the system should be properly checked for leakage.

Well points are usually provided with sand packs rather than gravel packs around the screen. The sand increases the effective diameter of the well point, reduces the entrance loss, and prevents clogging. Most commercial well point screens are designed to operate in contact with concrete quality sand (FM 1.5 to 2.5). If the aquifer material is finer than concrete sand, concrete quality sand will improve the situation. **Fig. 44** shows the characteristics of concrete quality sand. However, in some cases the formation material may be so fine that they might migrate into the concrete sand filter and clog it. In this case performance of well points may be improved by using mortar quality sand filter. **Fig. 45** shows characteristics of typical mortar sand filter. The screen is designed to retain 90% of this filter.

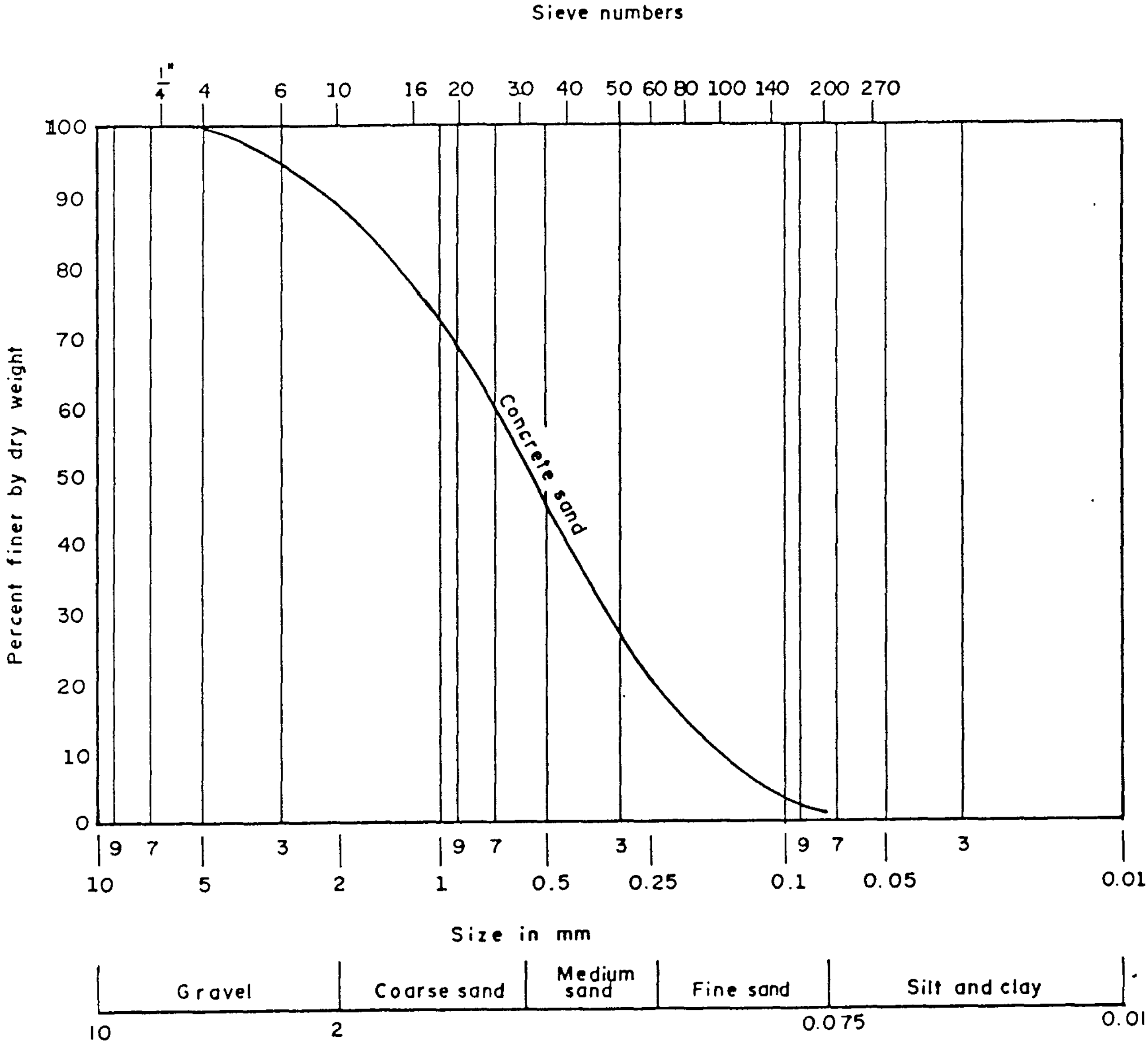


Fig. 44. Typical Filter of Concrete Sand for Well Points.

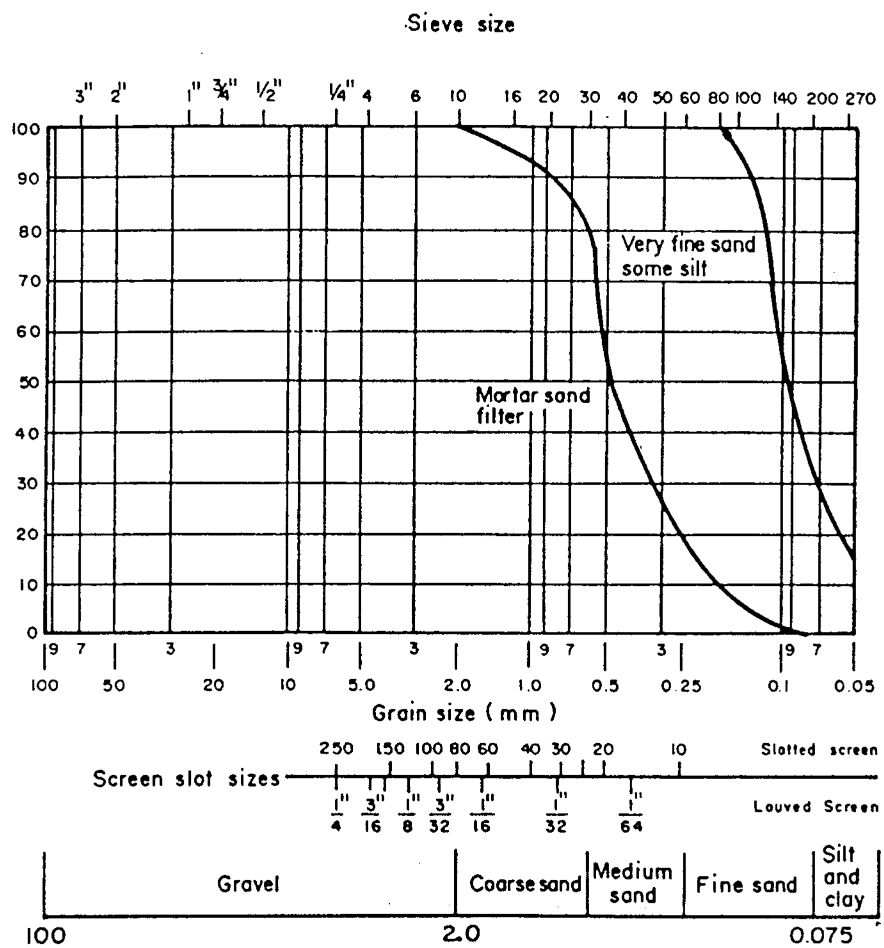


Fig. 45. Typical Filter of Mortar Sand for Well Points.

Well points are mostly installed by self jetting (**Fig. 46**). It can also be installed in a predrilled hole made by a hole puncher with or without casing. The jetting method produces a clean hole of superior quality. Self jetting well points can be installed quite easily in clean medium sands. After attaching the jetting hose, the well point and the riser pipe is kept suspended in a vertical position and then the jet water is turned on. A high pressure jet emerges out of the tip of the well point, dislodges the soil particles, advances the hole and removes the dislodged materials. For normal sandy soils a jet pressure of 345-414 kn/m^2 (50-60 psi) is adequate to install a well point. For coarse sand and gravel, jetting pressure up to 862 kn/m^2 (125 psi) may be required in order to carry out the heavier particles. During jetting it must be ensured that the jet water returns to the surface. The jet water is supplied by a high pressure jet pump (517-862 kn/m^2), with capacities ranging from 500 to 1000 gpm.

After the well point has been jetted to grade, the jet water should be continued until the return water is reasonably clear. The jet flow is then reduced so that there is only a small flow returning to the surface. After this filter sand is slowly tremied or poured into to the hole up to the water table. The remainder of the hole is filled with silt or clay to minimise entry of air in to the well point through the sand filter.

b) <u>Problems associated with well points:</u>

The following problems are usually encountered during installation and execution of a well point system :

<u>Loss of vacuum in the pump</u> :

Standard pumps should create a steady vacuum of 62.5-67.5 cm of Hg. Many pumps can not produce this vacuum during operation. Checking is normally done by closing the valve between the header and the pump suction. If the pump records a vacuum lower than the above limit, the pump must be leaking or the moving parts are worn. In such case, the pump should be properly sealed or repaired.

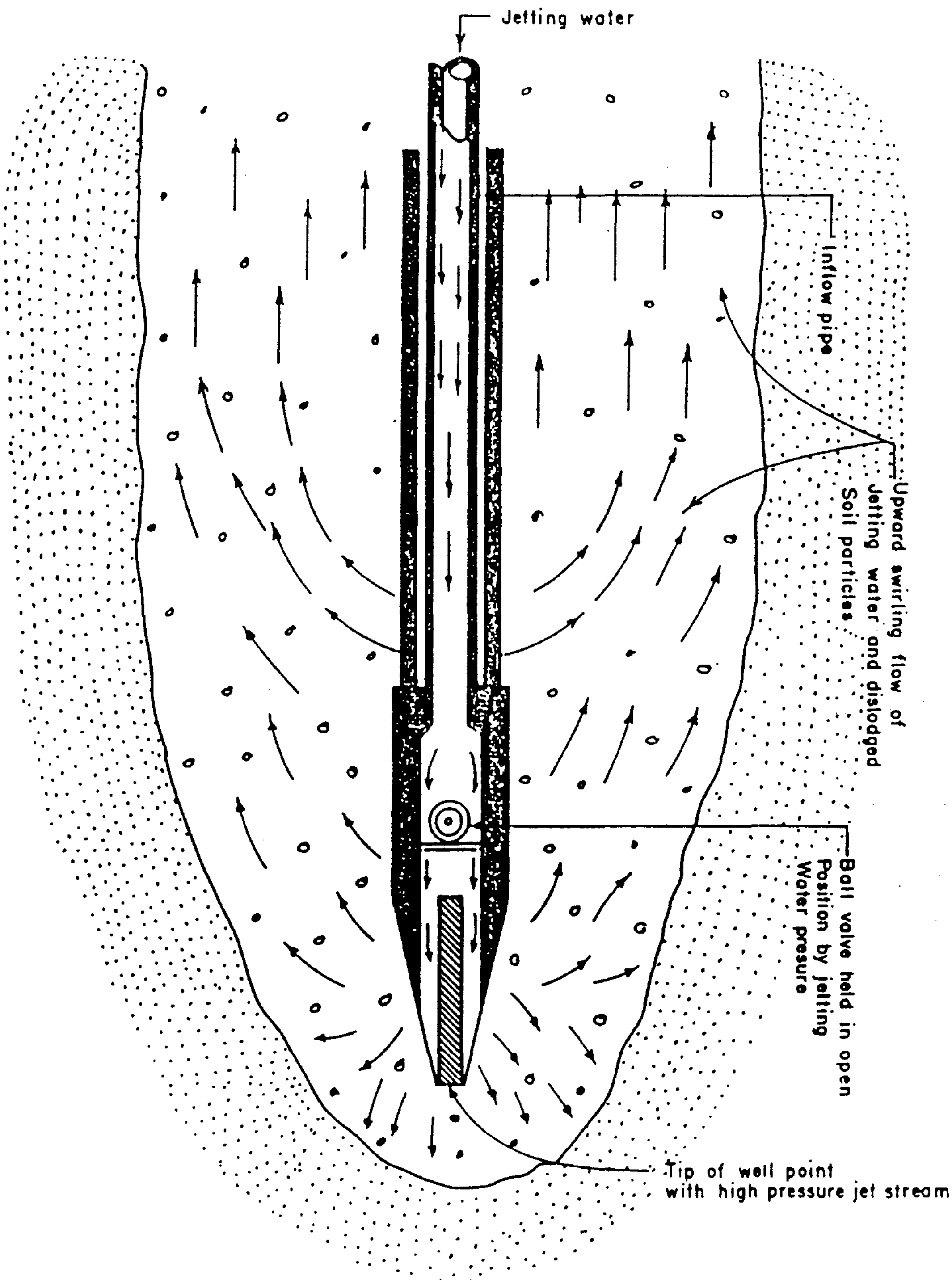


Fig. 46. Self Jetting Action of Well Points.

Leakage in the header pipe :

Some times it becomes difficult to achieve the desired vacuum due to leakage in the header pipe. To avoid this all joints and connections in the header pipe should be properly checked and sealed. For checking tightness of the header, all valves in the pump suction line and swing joints should be closed. The header is then filled with water under a pressure of 67-103 kn/m^2 (10-15 psi) and checked for leaks (**Fig. 47**).

Entry of air through well point screen :

This can create a major problem during execution of the dewatering system. It will happen when the water table has been lowered below the top of the well point (**Fig. 48**). Because of variation in soil properties and/or installation defects some well points may draw air. When a well point draws excessive air, it enters in surges instead of smooth flow of bubbles as is desired. A well point in this situation can be identified by unusual sounds or by creation of low unsteady vacuum in the system. One or more well points drawing excessive air can severely overload the entire system, causing reduction of vacuum and subsequent failure to achieve the required draw down. This can be minimised by :

- properly tuning the well points. Tuning is the procedure of balancing the discharge of well points, so that each draws its maximum potential yield without an excessive amount of air.
- Controlling the shut-off valves in the swing joints so that water is not pulled out of the well point faster than it is entering.
- Closing the valves on the badly disturbed wells.

Interference of header pipe:

Another problem with the well point system is the interference of header pipes with the labour & equipments employed for excavation, which badly hampers the progress of work. So, placement of header pipes should be so planned that there is least obstruction to the process of excavation & construction.

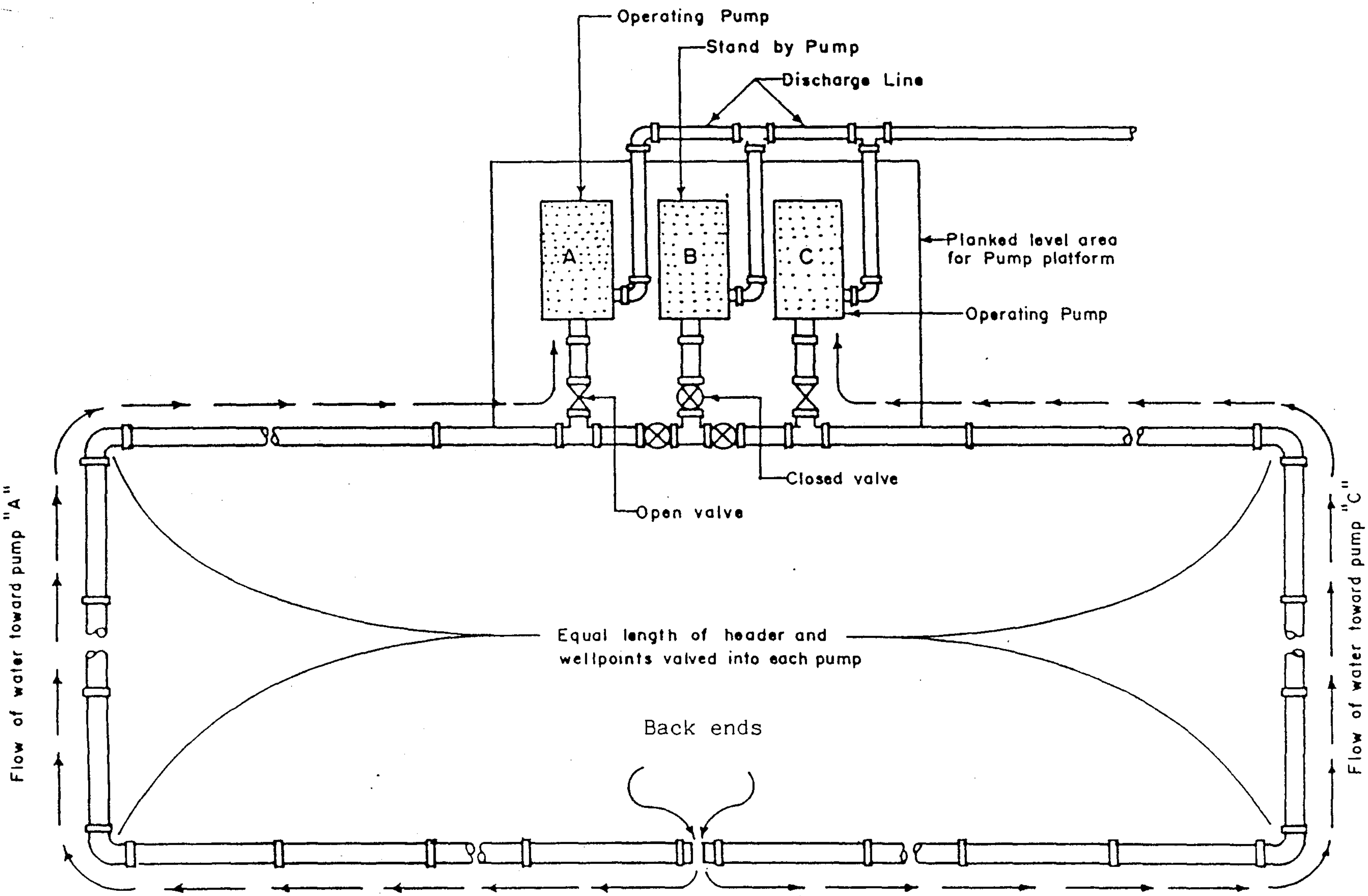


Fig. 47. Header Layout Plan.

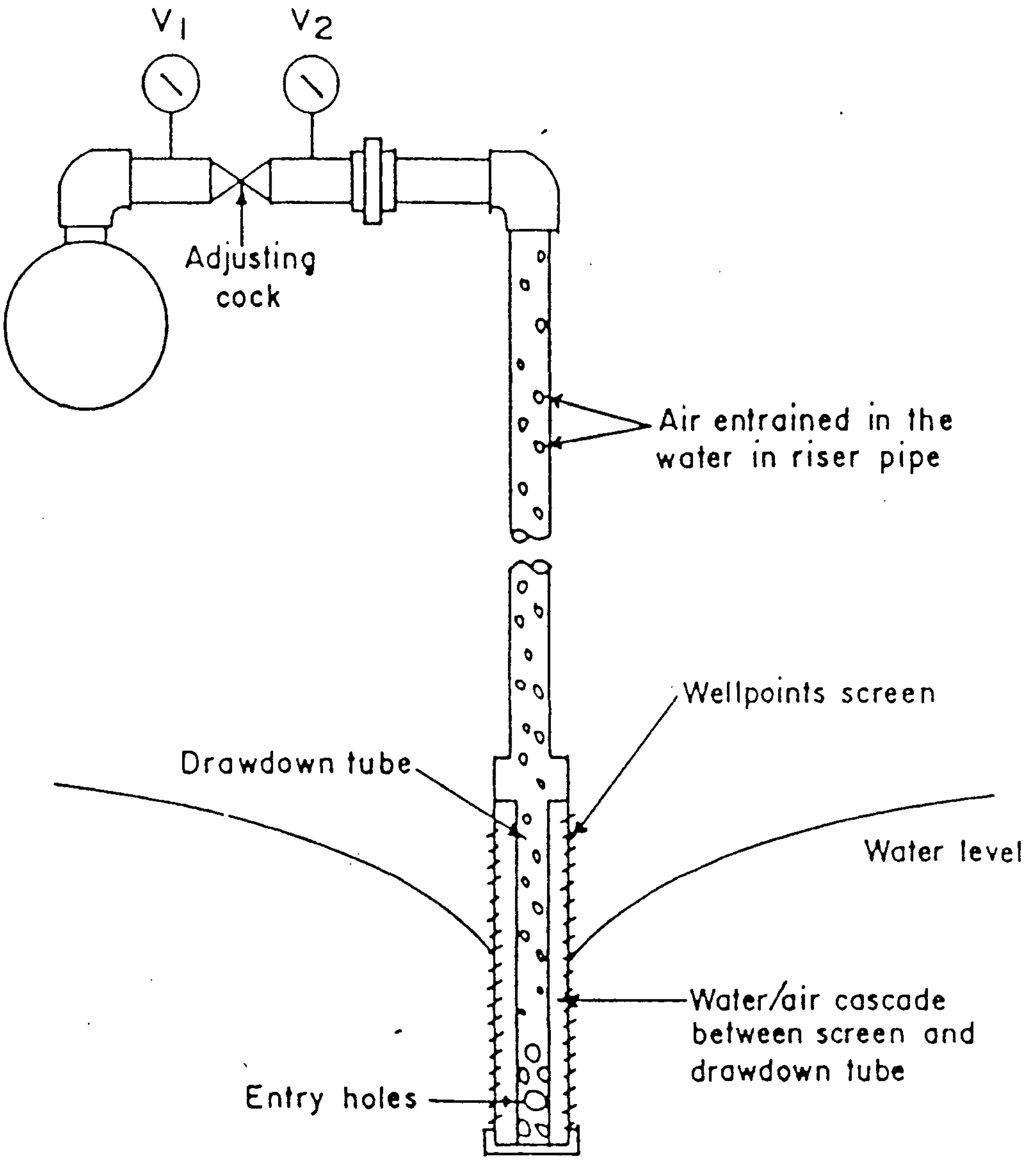


Fig. 48. Tuning of Well Points that Draw Air.

Slope instability

This problem is encountered when excavation is carried through a water table aquifer to an impermeable bed of clay or rocks. In this case the lowered water table will always remain above the top of impermeable layer and will cause seepage at the interface. If the overlying sand is uniform and does not contain plastic fines, even small seepage will cause severe ravelling of the slope.

In such case it becomes necessary to develop a condition where seepage water comes through certain filter or barrier. Excavation is carried to the predrained water level and a sand bag dike is constructed in the ditch (**Fig. 49**). The bags should be porous and should contain free draining material. When the predrained water level is very close to the clay layer and seepage is minor, a similar technic is employed using gravels instead of sand bags.

4.1.3 Vacuum method of dewatering

This is suitable for draining very fine silty sands, clay loams and organic soils, specially when the permeability of soil is less then 10^{-4} cm/sec. and effective grain size D_{10} is less than 0.05 mm. **Fig. 50** shows a typical arrangement of vacuum system. The well points are set in the hole and annular space is filled with filter sand up to 90 cm to 120 cm from the top. The topmost 90 to 120 cm is then sealed with tamped clay seal. When vacuum is applied to the well, it extends to the formation soil around the well and accelerates movement of pore water into the well point. Pumping relatively small quantities of water for few weeks will remarkably improve the strength of weak & unstable soils.

4.1.4 Electro-osmotic method of dewatering

This is a less common method but is very effective in stabilizing soft silts and clays by accelerating movement of pore water held under capillary forces, which never moves under gravity. In this method a line of metal well points are installed to serve as negative electrodes and a line of metal rods are installed to serve as anodes. When a electric current passes between the anodes and the cathodes, water flows out of the surrounding soil towards the cathode well from which it can be pumped by suction pumps or deep well pumps.

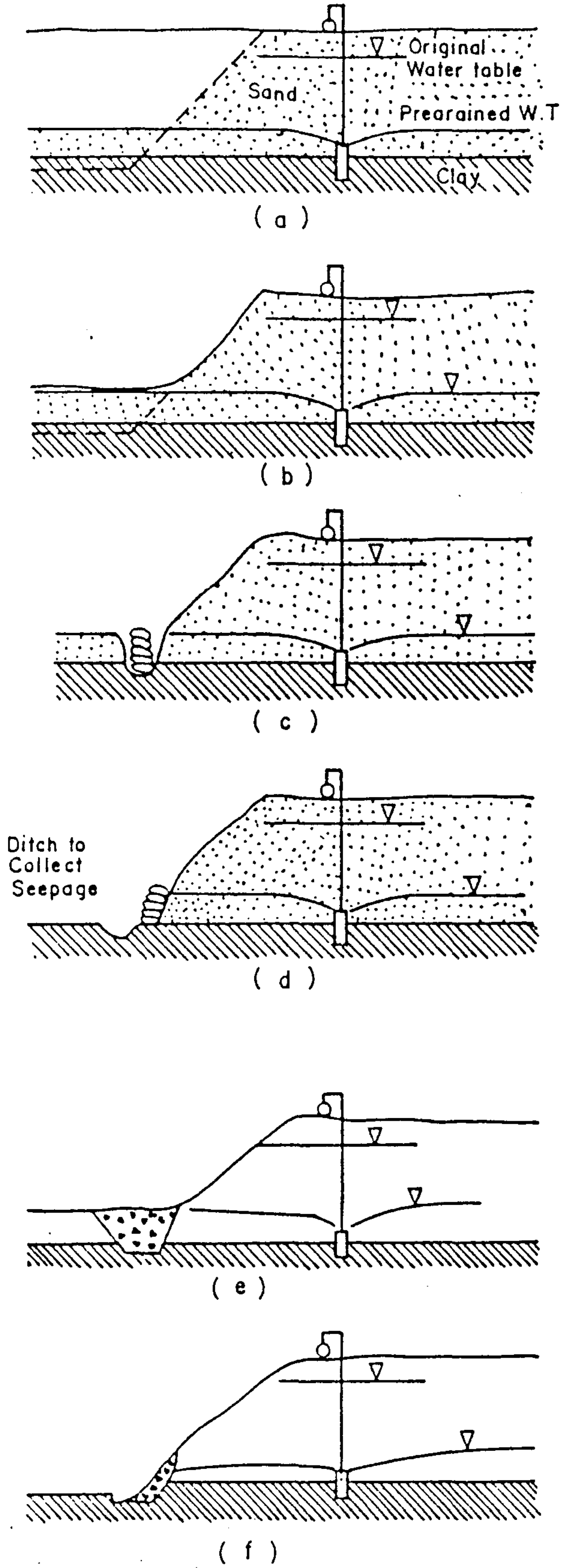


Fig. 49. Protection against Slope Failure.

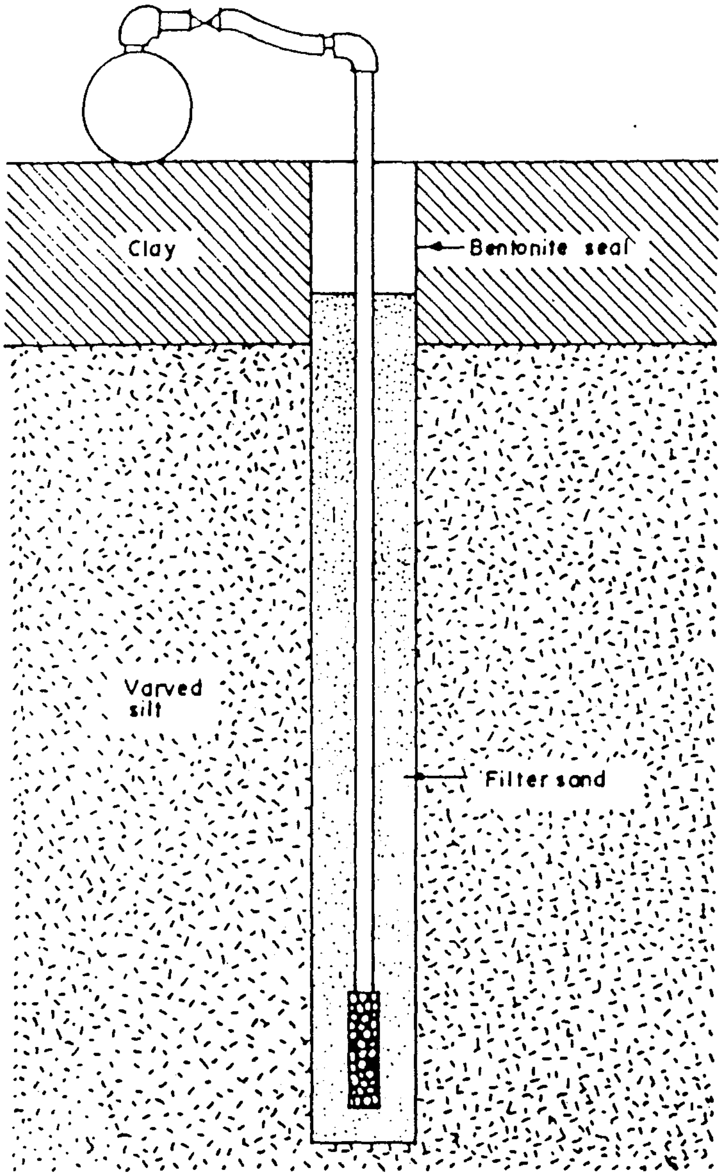


Fig. 50. Vacuum Method of Dewatering.

4.2.0 SELECTION OF DEWATERING METHODS

There are three common methods of dealing with ground water in a construction site i) seepage cutoff ii) gravity drainage iii) non gravity drainage. The decision to select a particular method or combination of methods depends mainly on the following factors:

a) Project type & its importance.

b) Type of structure and Nature of the soil

c) Size & depth of excavation

d) Proposed method of excavation & ground support.

e) Nature of existing neighbouring structures.

f) Ground water hydrology

g) Construction safety

h) Construction schedule & dewatering cost.

Depending on the above factors, the first decision has to be made as regards the broad type of drainage to be employed i.e. seepage cutoff, gravity drainage and non gravity drainage (vacuum drainage or electro-osmotic drainage).

Prugh, based on his practical experience, developed a very simple but approximate method to establish the limits of various drainage methods (**Fig. 51**). He analyzed grain size distribution curves for various soil samples collected from projects dewatered by different methods and then set limits for different drainage methods in a more or less arbitrary fashion. The limits, though very approximate, provide a very useful tool for making a decision.

If seepage cutoff method is selected the designer has to further decide whether to use sheet piles or slurry walls or grout curtains for enclosing the area and then to drain the enclosed soil mass. The decision will depend on relative cost and availability of technical know-how.

If gravity drainage is selected, he has to further decide whether to go for open pumping or well pumping. If non-gravity drainage is selected the designer has the option to go for vacuum drainage or electro-osmotic drainage.

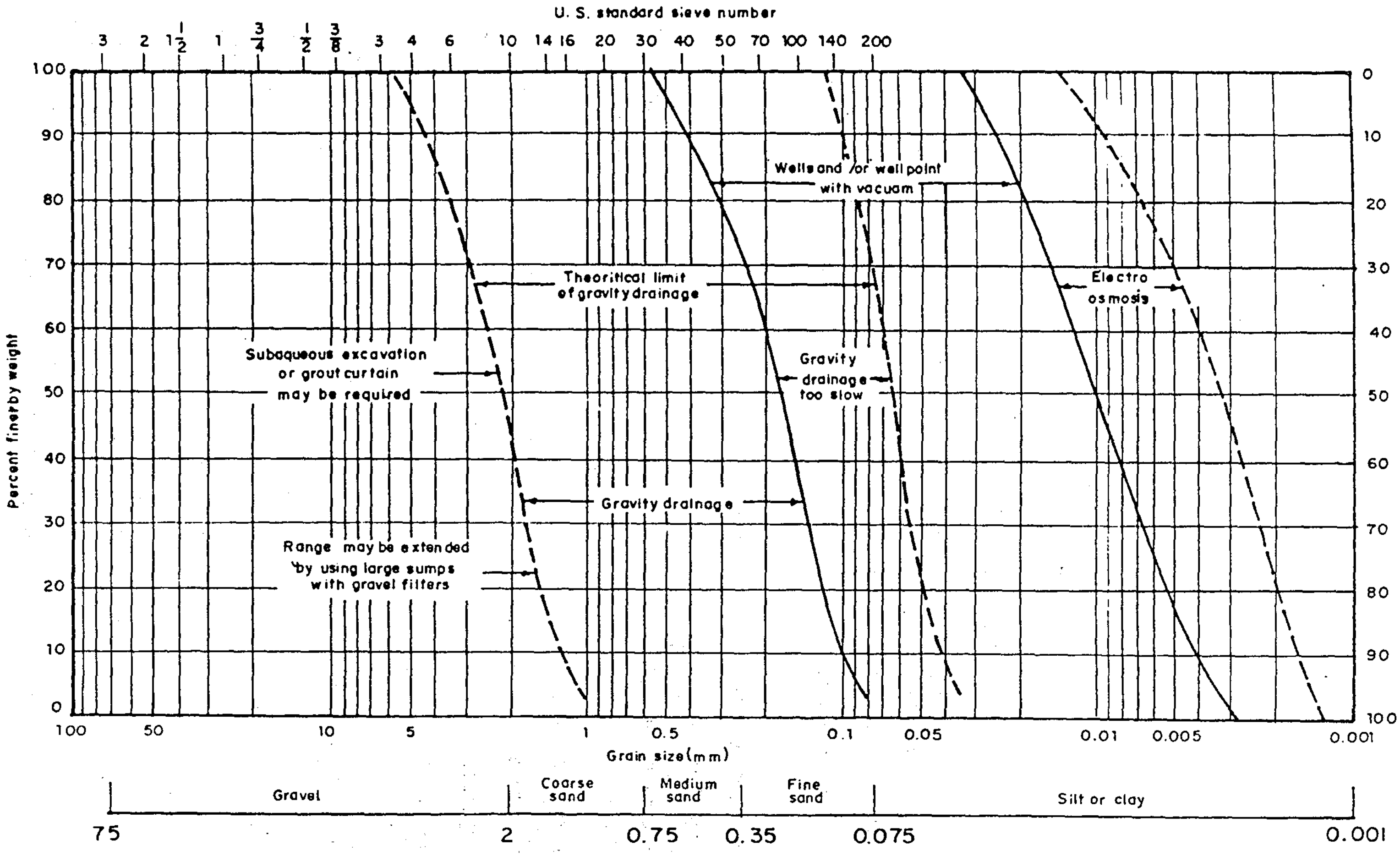


Fig. 51. Limits of Different Drainage Methods.

Open pumping or sump pumping from ditches are usually very simple and least expensive under favourable conditions. But in adverse situations it may cause unusual delay, cost overrun and occasional catastrophic failures. Tables 12 and 13, present some of the conditions that will help in deciding whether to go for open pumping or not.

<u>Table 12. Conditions Favourable to Sump Pumping</u>

Condition	Explanation
<u>Soil Characteristics</u>	
Dense, well graded granular soils, especially those with some degree of cementation or cohesive binder.	Such soils are low in permeability and seepage is likely to be low to moderate in volume. Slopes can bleed reasonable quantities of water without becoming unstable. Lateral seepage and boils in the bottom of an excavation will often become clear in a short time. So that foundation properties are not impaired.
Firm clays with no more than a few lenses of sand which are not connected to a significant water source.	Only small quantities of water can be expected from the sand lenses and it should diminish quickly to negligible value. No water is expected from the clay.
Hard fissured rock.	If the rock is hard even moderate to large quantities of water can be controlled by open pumping, as in typical quarry operations.
<u>Hydrologic Characteristics</u>	
Low to moderate dewatering head, remote source of recharge, low to moderate permeability and minor storage depletion.	These characteristics indicate that the quantity of water to be pumped will be low, minimizing problems with slope stability and subgrade deterioration and facilitating the construction and maintenance of sumps and ditches.
<u>Excavation Methods</u>	
Dragline, backhoe, clamshell.	These methods do not depend on traction within the excavation, and the unavoidable temporarily wet condition due to open pumping does not hamper progress.

Source : J. P. Powers

Table 12. (contd.) Conditions Favourable to Sump Pumping

Conditions	Explanation
Excavation Support	
Relatively flat slopes	Flat slopes, appropriate to the soils involved, can support moderate seepage without becoming unstable
Steel sheeting, slurry concrete walls	These methods cut off lateral flow, and assuming there are no problems at the subgrade open pumping is satisfactory
Miscellaneous	
Open unobstructed site	If there are no existing structures nearby, so that minor slides are only a nuisance, some degree of risk can be taken
Large excavations	In a large excavation the time necessary to move the earth is sometimes such that the slow process of lowering water with sumps and ditches does not seriously affect the schedule
Light foundation loads	When the structure being built puts little or no load on the foundation soils (for example, a sewage lift station) slight disturbance of the subsoil may not be harmful

Table 13. Conditions Unfavourable to Sump Pumping (Predrainage Usually advisable).

Conditions	Explanation
Soil Characteristics	
Loose, uniform granular soils without plastic fines	Such soils have moderate to high permeability and are very sensitive to seepage pressures. Slope instability, and loss of strength in the bottom and are likely to create problems when open pumping is carried out.
Soft granular silts and clays with moisture contents near or above the liquid limit	Such soils are inherently unstable, and slight seepage pressures in permeable lenses can trigger massive slides
Soft rock: rock with large fissures field with soft materials or soluble precipitates: sandstones with uncemented sand layers	If substantial quantities of water are open pumped soft rock may erode. Soft materials in the fissures of hard rock may be leached out. Uncemented sand layers can wash away The quantity of water may progressively increase and blocks of massive rock may shift.
Hydrologic Characteristics	
Moderate to high dewatering head Proximate source of recharge Moderate to high permeability	These characteristics indicate the potential for high water quantities. Even well graded gravels can become quick if the seepage gradient is high enough. Problems with construction and maintenance of ditches and pumps are aggravated
Large quantity of storage water	If the aquifer is such that large quantities of water must be depleted from storage then predrainage well in advance of excavation is advisable.
Artesian pressure below subgrade	Open pumping cannot cope with pressure below subgrade. Relief wells are advised

Source: J. P. Powers

Table 13. (contd.) Conditions Unfavourable to Sump Pumping

(Predrainage Usually advisable)

Conditions	Explanation
Excavation Methods	
Scrapers: loaders and trucks	These methods require good traction for efficient operation. Unavoidable temporarily wet conditions due to open pumping can seriously hamper progress. (If ditches and sumps are prepared in advance by dragline or backhoe, scraper operation may be feasible)
Excavation Support	
Steep Slopes	Steep slopes are sensitive to erosion and sloughing from seepage, and can also suffer rotary slides unless the water table is lowered sufficiently in advance of excavation
Miscellaneous	
Adjacent Structures	When existing structures would be endangered by slides, or loss of fines from the slopes, open pumping cannot be tolerated
Small excavation	In small excavations, delays due to open pumping can seriously delay the work
Heavy foundation load	When the structure being built bears heavily on the subsoil even minor disturbances must be avoided.

If open pumping does not fulfil the requirement, predrainage methods like shallow wells, deep wells and well points become inevitable. Conditions for selection of different predrainage methods are presented in **Table 14.**

Table 14. Checklist for Selection of Predrainage Methods

Conditions	Well point Systems	Shallow wells/ Suction wells	Deep Wells
Soil			
Silty and clayey sands	Good	Poor	Poor to fair
Clean sands and gravels	Good	Good	Good
Stratified soils	Good	Poor	Poor to fair
Clay or rock at subgrade	Fair to good	Poor	Poor
Hydrology			
High permeability	Good	Good	Good
Low permeability	Good	Poor	Poor to fair
Proximate recharge	Good	Poor	Poor
Remote recharge	Good	Good	Good
Schedule			
Rapid draw down required	Good	Good	Unsatisfactory
Slow draw down permissible.	Good	Good	Good
Excavation			
Shallow (<6 m)	Good	Good	Good
Deep (>6 m)	Multiple stage required	Multiple stage required	Good
Cramped	Interferences	Interferences	Good
Characteristics			
Normal spacing	1.5-3 m (5-10 ft)	6-12 m (20-40 ft)	>15 m (50 ft)
Normal Range of Capacity			
Per unit	0.1-25 gpm	50-400 gpm	25-3000 gpm
Total system	Low-5000 gpm	2000-25,000gpm	200-60,000 gpm
Efficiency with accurate design	Good	Good	Fair

Source : J.P. Powers

4.3.0 DESIGN OF DEWATERING SYSTEM

4.3.1 Determination of Dewatering volume

The dewatering volume Q, is a function of the area & depth of excavation, magnitude of water table lowering,permeability of soil and existence of nearby recharge bodies. In calculating the dewatered volume, the excavation area is converted into an equivalent sink or trench and the flow is calculated either by well formulas or by trench formulas or by both. The conventional methods of transforming the excavation pits into equivalent sinks or trenches and calculation of dewatering volumes are as follows:

i) Transformation of excavation pit.

Circular excavation pit :

In case of circular excavation pit (**Fig. 52.a**), the entire system is considered to be a large well with an equivalent radius, r_s, equal to the radius of the excavation.

Rectangular excavation pit:

A rectangular system as in **Fig. 52.b**, is assumed to act as a circular system having the same enclosed area or perimeter.

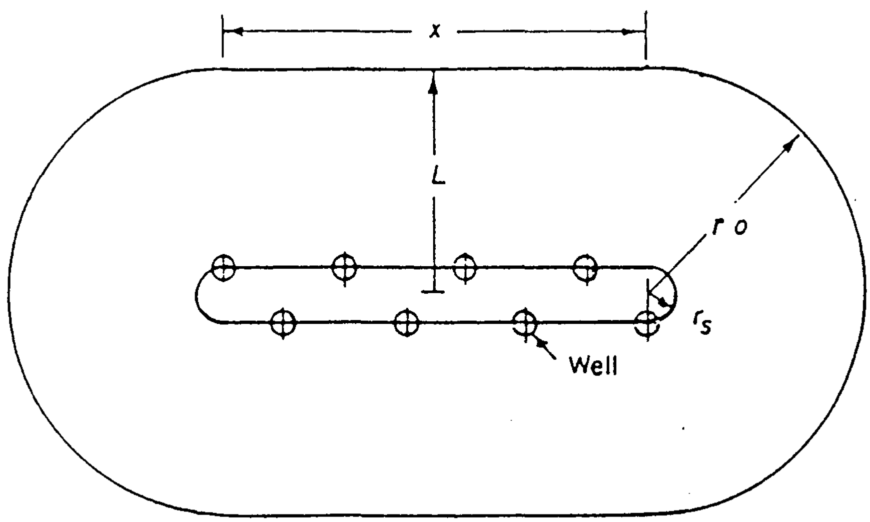
$$r_s = \sqrt{\left(\frac{ab}{\pi}\right)} \quad \text{(Same enclosed area)} \quad \ldots\ldots\ldots\ldots\ldots\ldots(53)$$

$$r_s = \left(\frac{a+b}{\pi}\right) \quad \text{(Same perimeter)} \quad \ldots\ldots\ldots\ldots\ldots\ldots(54)$$

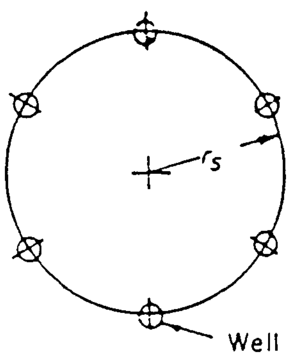
Where, r_s = radius of equivalent well or sink.

a = length of the excavation pit.

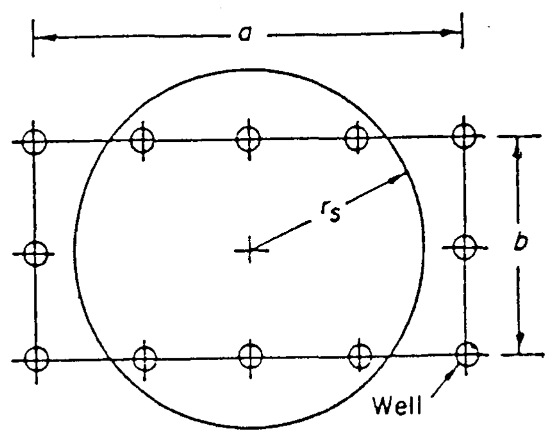
b = breadth of the excavation pit.



(c) Narrow excavation pit



(a) Circular pit



(b) Rectangular pit.

Fig. 52. Equivalent Radius of Excavation Pits.

Either of the equations will give satisfactory results when the wells or well points are closely spaced, radius of influence ro, is great in relation to r_s and ratio of a/b is less than 1.5. For wide or irregular spacing of wells, approximate dewatering volume can be estimated by equivalent well or sink method but in this case the desired lowering of water table should be checked by the method of cumulative draw down of interfacing wells and Q, should be revised accordingly.

<u>Irregular excavation pit</u>:

If the excavation pit is irregular then the equivalent radius can be calculated as follows:

$$r_s = \sqrt{\frac{A}{\pi}} \quad \text{(Same enclosed area)} \ldots\ldots\ldots\ldots\ldots\ldots (55)$$

$$r_s = \frac{P}{2\pi} \quad \text{(Same perimeter)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (56)$$

Where, A = area of excavation.

P = perimeter of excavation.

<u>Long narrow excavation</u>:

When the excavation pit is long and narrow where the ratio of a/b is large (**Fig. 52.c**), equivalent radius, r_s, is taken as half the width of the trench, and the volume is calculated by well formula at the ends and trench formula on the sides.

Table 15 and **Table 16,** show the sizes and equivalent radii of the excavation pits for different types of structures.

ii) <u>Calculation of dewatering volume</u>

After transforming the excavation pit into an equivalent sink or trench, the dewatering volume may be calculated by using the following formulas.

For an equivalent circular system:

$$Q = \pi K \left[\frac{h_o^2 - h_w^2}{\ln(r_o/r_s)} \right] \text{ (unconfined aquifer)} \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

$$Q = 2\pi K b \left[\frac{h_o - h_w}{\ln(r_o/r_s)} \right] \text{ (confined aquifer)} \ldots\ldots\ldots\ldots\ldots\ldots (28)$$

For long narrow trenches:

$$Q = \left[\frac{\pi K (ho^2 - hw^2)}{\ln (r_o/r_s)} \right] + \left[\frac{xK (ho^2 - hw^2)}{L} \right] \text{ (unconfined)} \ldots\ldots (57)$$

$$Q = 2\pi K D \left[\frac{h_o - h_w}{\ln(r_o/r_s)} \right] + 2 \left[\frac{xKD(h_o - h_w)}{L} \right] \text{ (Confined)} \ldots\ldots\ldots (58)$$

Where, Q = Volume to be dewatered

h_o = initial level of water table/piezometer

h_w = dewatered level of water table/piezometer

r_o = radius of influence

r_s = equivalent radius of well/sink

K = permeability

D = Thickness of confined aquifer

x = length of sides of trench

L = $r_o/2$

When perennial streams pass within the zone of influence of the system, the magnitude of flow can be estimated by image well theory or flownet analysis as in section 3.3.4. But in no case the flow should be less than that given by thiems equations developed for infinite aquifers.

The dewatered level hw should be taken 60 to 90 cm lower than the desired level of excavation and the value of r_o should be taken from pumping test. In absence of pumping test it can be estimated from the following equation, which is of course very approximate.

$$r_o = 3\ (h_o - h_w)\ \sqrt{K} \qquad (30)$$

Where, $(h_o - h_w)$ = drawdown in feet.

K = permeability, microns/sec.

It is obvious from the above equations that dewatering volume Q, is not a direct function of the size of the excavation. As the size factor r_s enters a logarithmic term in the above equations, Q becomes much less sensitive to the increase of r_s and hence the size of excavation pit. For example, a 100% increase in the size of the excavation pit may cause only 20% increase in the dewatering volume, Q. **Table 17**, shows approximate dewatering volumes for different structures in different soils. However, these figures are very crude and should be used carefully.

Table 15. Approximate size of excavation pit for different type of regulators (Fig. 53)

Regulator type	L, Length in meter							B, Breadth in meter					Remarks
	a	b	c	d	e	Ls	Le*	$\bar{a}$	$\bar{b}$	$\bar{c}$	B_{ave}	Be*	
1-1.50x1.80	9.20	8.50	7.00	16.10	11.40	52.20	68.20	5.00	2.32	5.50	4.27	14.27	Ls = Length of structure. Le = length of excavation B_{ave}= ave width of structure Be = ave width of excavation/sink.
2-1.5x1.80	9.25	10.00	7.00	11.00	11.25	48.50	58.50	10.00	4.29	11.00	8.43	18.43	
3-1.5x1.80	9.20	8.50	7.00	17.09	11.40	54.00	64.00	10.00	6.16	11.00	9.05	19.03	

Source: Typical drawings available at NHCL office.

*Wells are assumed to be at least 5 meter away from the edges of the structure and the perimeter of the pit/sink is assumed to pass through the centre line of the wells. The equivalent lengths and breadths of excavation sink are therefore, larger than the lengths and breadths of the structure by (5+5) = 10 metre.

Table 16. Equivalent radius, r_s, for different size of excavation pits (from Table 15)

Regulator type	Le (m)	Be,(m)	Le/Be	Remarks	rs, circular method (m)			Trench method	
					area basis	perimeter basis	average	rs (m)	x(m)
1-1.50x1.80	68.20	14.27	4.77	use trench method	-	-	-	7.135	53.93
2-1.5x1.80	58.50	18.43	3.17	Use circular method	18.53	24.50	21.51	-	-
3-1.5x1.80	64.00	19.03	3.36	,,	19.67	26.43	23.05	-	-

Note : trench method is preferable when Le/Be ≥ 4.

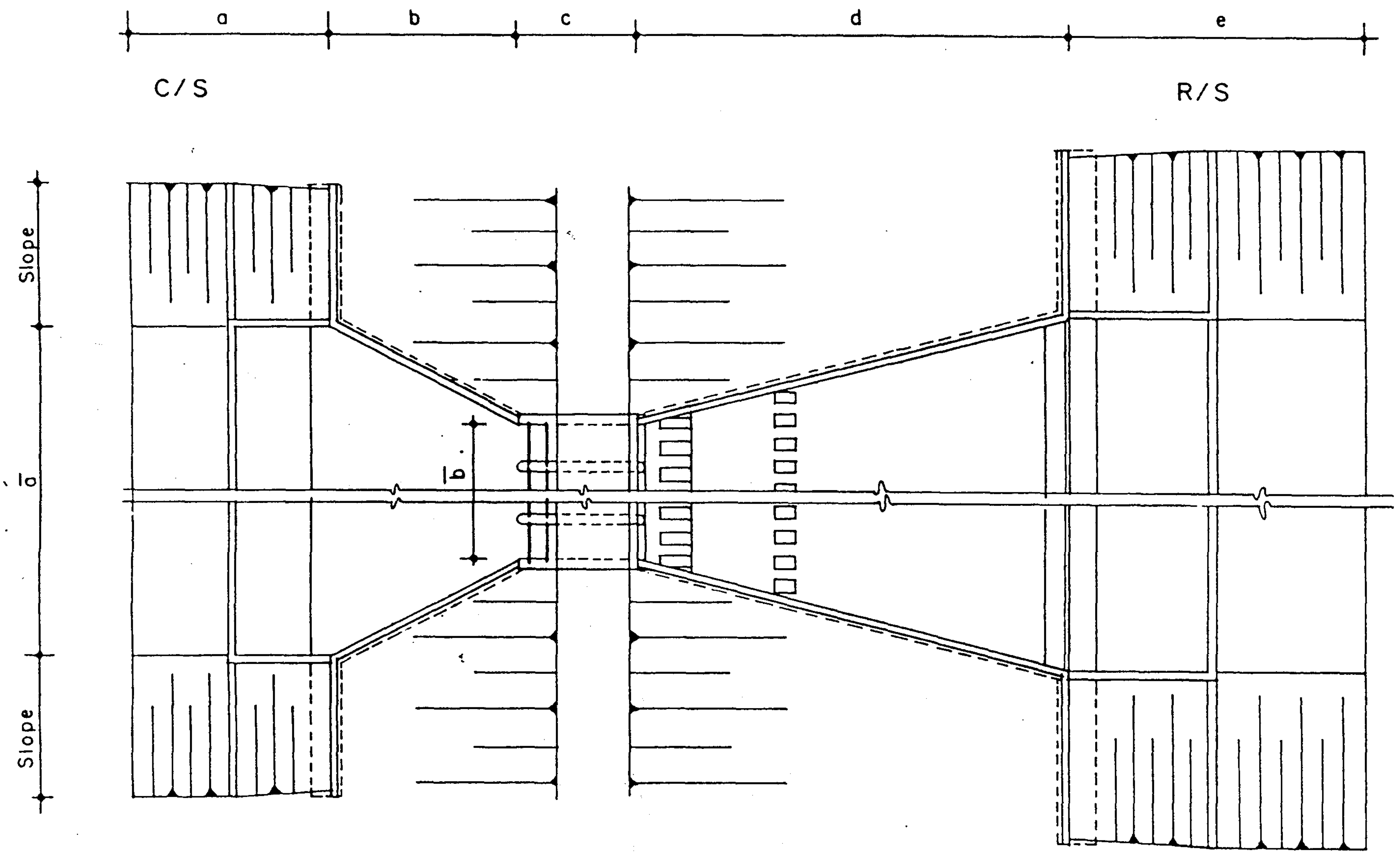


Fig. 53. Typical Plan of Regulator

Table 17. Approximate Dewatering volume for Different Regulators

a) For 1-1.50 m x 1.80 m regulator.

Soil type	* k m/day	h_o m	S_w m	h_w m	r_s (x) m	■ r_o m	Factor of safety	Q	
								m^3/day	cusec
Silty Sand	4.25	25	2	23	7.13	42.00	1.25	2213	0.90
			3	22	(53.90)	63.00		2365	0.97
			4	21		84.00		2500	1.02
Fine Sand	22.00	25	2	23	-do-	95.76	-do-	6160	2.50
			3	22		134.46		6970	2.84
			4	21		191.53		7677	3.12
Medium Sand	34.00	25	2	23	-do-	120.36	-do-	8361	3.40
			3	22		180.51		9611	3.91
			4	21		240.72		10708	4.36
Coarse Sand	47.50	25	2	23	-do-	140.71	-do-	10373	4.22
			3	22		211.08		12029	4.90
			4	21		281.43		13531	5.51
Sand (mixture)	6.50	25	2	23	-do-	52.05	-do-	2852	1.16
			3	22		78.08		3086	1.20
			4	21		104.10		3230	1.34

* K values in Table-13 has been obtained by averaging the values of K in Table 2. However, for silty sand an above average value of K has been selected because, in absence of test, there is always a probability of confusing fine sand with silty sand, which might result in disastrous consequences.

Table 17 (contd.) Approximate Dewatering volume for Different Regulators

b) For 2-1.50 m x 1.80 m regulator.

Soil type	* k m/day	h_o m	S_w m	h_w m	r_s m	■ r_o m	Factor of safety	Q	
								m^3/day	cusec
Silty Sand	4.25	25	2	23	21.51	42.00	1.25	± 2279	± 0.93 *:
			3	22		63.00		± 2279	± 0.93 *:
			4	21		84.00		± 2279	± 0.93 *:
Fine Sand	22.00	25	2	23	-do-	95.76	-do-	5563	2.26
			3	22		134.46		6408	2.61
			4	21		191.53		7288	2.96
Medium Sand	34.00	25	2	23	-do-	120.36	-do-	7613	3.10
			3	22		180.51		9029	3.68
			4	21		240.72		11782	4.80
Coarse Sand	47.50	25	2	23	-do-	140.71	-do-	9520	3.88
			3	22		211.08		11523	4.70
			4	21		281.43		13348	5.44
Sand (mixture)	6.50		2	23	-do-	52.05	-do-	2783	1.13
			3	22		78.08		2789	1.14
			4	21		104.10		2971	1.21

■ The values of r_o has been calculated from Sichardt's empirical equation which is function of drawdown and soil permeability. As this equation is very approximate, value of r_o should be checked by test pumping one or more wells during installation.

Table 17. (contd.) Approximate Dewatering volume for Different Regulators

c) For 3-1.50 m x 1.80 m regulator.

Soil type	* k m/day	h_o m	S_w m	h_w m	r_s m	■ r_o m	Factor of safety	Q m³/day	Q cusec	
Silty Sand	4.25	25	2	23	23.05	42.00	1.25	± 2670	± 1.08	*
			3	22		63.00		± 2670	± 1.08	*
			4	21		84.00		± 2670	± 1.08	*
Fine Sand	22.00	25	2	23	-do-	95.76	-do-	5838	2.38	
			3	22		134.46		6653	2.71	
			4	21		191.53		7495	3.05	
Medium Sand	34.75	25	2	23	-do-	120.36	-do-	7936	3.23	
			3	22		180.51		9382	3.82	
			4	21		240.72		10680	4.35	
Coarse Sand	47.50	25	2	23	-do-	140.71	-do-	9943	4.05	
			3	22		211.08		11895	4.84	
			4	21		281.43		13722	5.59	
Sand (mixture)	6.50		2	23	-do-	52.05	-do-	3023	1.23	
			3	22		78.08		3023	1.23	
			4	21		104.10		3129	1.27	

** At very low values of the coefficient of permeability, as in the case of silty sand, the logarithmic terms in equations 5 and 23 do not give proportionate results. As such to avoid confusion and to be on the safe side the calculated values of Q for silty sand have been slightly manipulated.

4.3.2 Design of Sump pumping or open pumping

Following steps may be followed for designing a sump pumping or open pumping system.

a) Estimate the length of drainage trench and determine the number of sumps depending on the capacity of pumps and available slope of the trench (length of trench may approximately be taken as equal to the perimeter of the excavation pit).

$$N_s = \frac{Q}{Q_p E} \qquad \text{.......... (59)}$$

Where, N_s = number of sumps
Q_p = Capacity of pump
E = efficiency of pump

b) Calculate the flow to each segment of trench by dividing Q by the total length of trench and then multiplying it by the length of individual segment.

$$Q_{ti} = \frac{Q}{L_t} L_{ti} \qquad \text{.......... (60)}$$

Where, Q_{ti} = flow to trench i,
L_t = total trench length
l_{ti} = length of trench segment, i.

c) Design the trench section by using any of the following equations.

$$Q_t = \frac{1.49}{n} A R^{2/3} S^{\frac{1}{2}} \quad \text{(FPS)} \qquad \text{.......... (61)}$$

$$= \frac{1}{n} A R^{2/3} S^{\frac{1}{2}} \quad \text{(MKS)} \qquad \text{.......... (62)}$$

$$Q_t = K.A.I, \quad \text{(For gravel filled trench.)} \qquad (63)$$

Where, A = Sectional area of trench

R = Hydraulic radius of trench or pipe (area divided by wetted perimeter).

S = Bed slope

n = Roughness coefficient of trench/pipe

k = Permeability of gravel

I = Hydraulic gradient of gravel media

Q_t = flow to trench

For most cases in shallow excavation, structures up to 3-1.5 m x 1.8 m vents in silty sand and fine sand will need an open trench having a bed width = 0.5 m, water depth = 0.30 m and a side slope = 1:2.

4.3.3. Design of tube wells

The following steps may be followed for design of tube wells (both shallow and deep):

a) Determine the number of wells based on minimum well spacing.

$$L_{min.} = 31.5\ r_w \qquad (37)$$

$$N_w \leq \frac{P}{L_{min}} \qquad (64)$$

Where, $L_{min.}$ = min. well spacing

N_w = Assumed number of wells(for preliminary estimate)

P = perimeter of excavation.

If number of wells calculated on the basis of minimum well spacing becomes too large, select a spacing of 10-20 meters for shallow tubewells and 20-50 meters for deep tubewells. Then estimate the number of wells by dividing the perimeter of excavation with the selected spacing. This will provide a good guess for the first trial.

b) Calculate the required capacity of individual well from:

$$q_w = \frac{Q}{N_w} \quad \text{.............................} (65)$$

Where, q_w = Required well discharge $\geq$ 0.1 cusec (2.83 l/sec)
Q = Total volume to be dewatered, cusec (m^3/day)

c) With the above q_w and well spacing analyse the cumulative effect of wells (group action) on the draw down as follows:

- let us consider a group of wells as in Fig. 54. The wells are assumed to be of identical capacity.
- let r_w be the diameter and q_w be the discharge of each well.
- let us consider the cumulative effect of draw down at a point, m, within the system. If r_w is radii of the wells, r_o is the radius of influence and x_1, x_2, x_3 ... x_m be the distances of the point, m, from the wells and z_1, z_2.... z_m etc. be the water levels at m due to influence respective wells, then according to the principle of superposition the following equations will hold good at point, m.

$$h_o^2 - z_1^2 = \frac{q_{w1}}{\pi K} (\ln r_o - \ln x_1) \text{ .. due to well-1}$$

$$h_o^2 - z_2^2 = \frac{q_{w2}}{\pi K} (\ln r_o - \ln x_2) \text{ .. due to well-2}$$

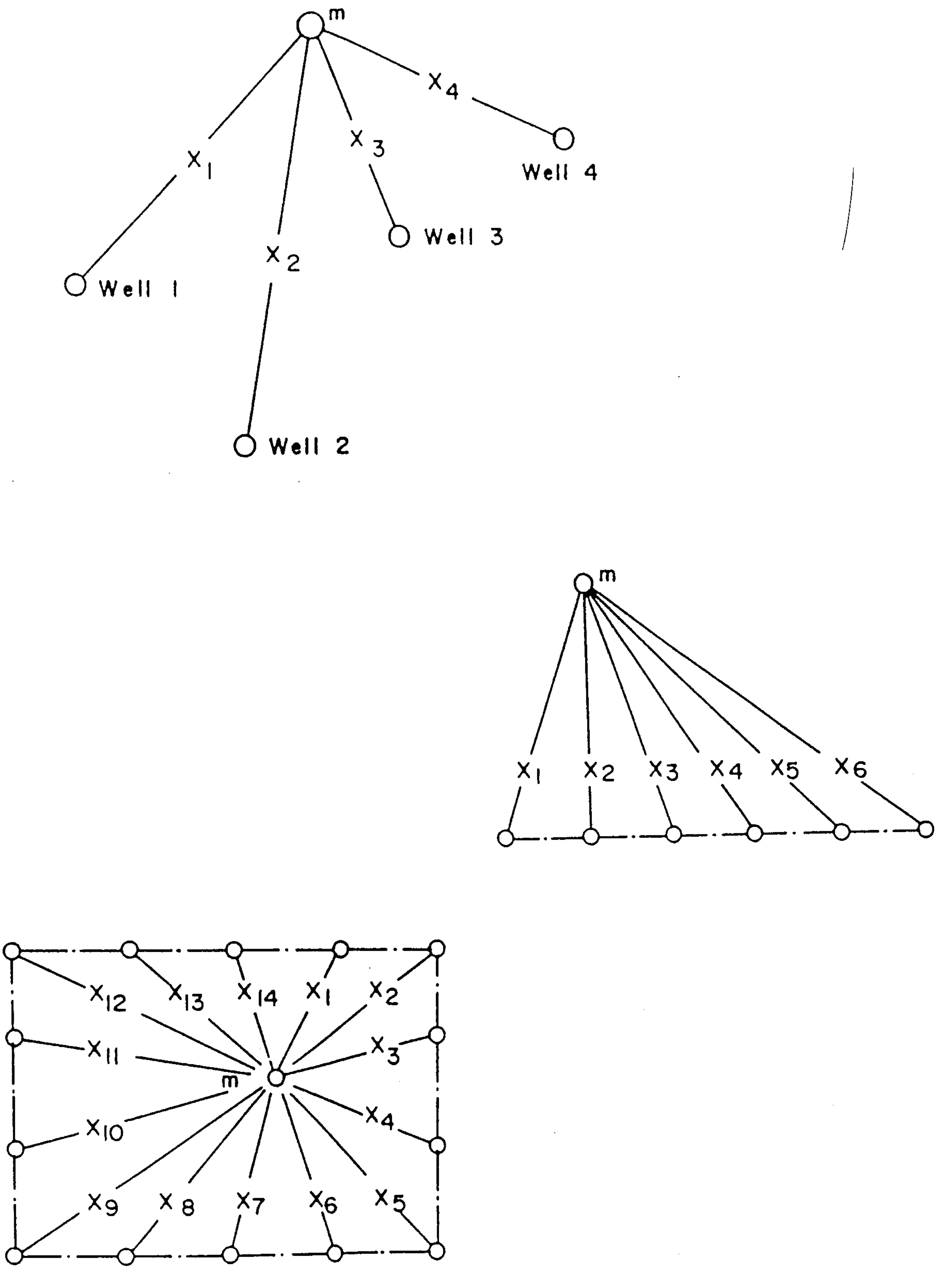


Fig. 54. Cumulative Effect of Well Groups.

And,

$$h_o^2 - z_n^2 = \frac{q_{wn}}{\pi K} (\ln r_o - \ln x_n) \text{ .. due to well - n}$$

Adding :

$$\sum_{i=1}^{i=n} (h_o^2 - z_i^2) = \frac{1}{\pi K} \sum_{i=1}^{i=n} q_{wi} (\ln r_o - \ln x_i) \text{..due to group action.}$$

$$= \frac{nq_w}{\pi K} [\ln r_o - \frac{1}{n} \ln (x_1.x_1..x_n)] \text{ ... (66)}$$

(Since the wells are identical $q_{w1} = q_{w2} = ... = q_{wn} = q_w$)

Taking $\sum_{i=1}^{i=n} (h_o^2 - z_i^2) = (h_o^2 - z_m^2)$, we have,

$$(h_o^2 - z^2_m) = \frac{nq_w}{\pi K} [\ln r_o - \frac{1}{n} \ln (x_1.x_2.x_3..x_n)]$$

$$= \frac{nq_w}{\pi K} [\ln r_o - \frac{1}{n} \ln P(x_n)] \text{ ....(67)}$$

$$Z_m = \sqrt{[h_o^2 - \frac{nq_w}{\pi K} \{\ln r_o - \frac{1}{n} \ln P(x_n)\}]} \text{ .... (68)}$$

Where, Z_m = Water level at m, due to cumulative well effect.

$P(x_n) = (x_1.x_2.x_3 \text{ ..... } x_n)$

h_o = depth of original water table.

From the above equation drawdowns S_m at point m in the well field can be found as:

$$S_m = (h_o - Z_m)$$
$$= h_o - \sqrt{[h_o^2 - \frac{q_w}{\pi K} \{\ln r_o - \frac{1}{n} \ln P(x_n)\}]}, \text{ at point m,} \quad (69)$$

To calculate the draw down so at the face of any well, simply move point m to that location and find so as in the above equation.

In reality drawdown, S_o, just outside the well is about 66% of the drawdown S_w at the well, due to well & other losses.

- Check the value of S_m with the desired level of water table at m. If not acceptable repeat the above procedures with revised spacing capacity/or and layout of wells until Sm ≥ desired draw down.

d) Design the well with the revised (if any) capacity q_w, as per guide lines below:

- Select a tentative length and depth of well screen depending on the soil test result.
- Select a suitable screen diameter from **Table – 3.**
- Design the gravel filter appropriate for the formation material .
- Design the screen slot size appropriate to the gravel filter or the formation material.
- Select allowable entrance velocity, V_s, through the screen from **Table–5.**
- Design the screen length using the equation.

$$L_s = \frac{q_w}{C.V_s.A_o} \quad \text{..................................(70)}$$

- If L_s becomes unreasonably long, increase the screen diameter and revise the calculations.

e) Design the pump based on the draw down S_w at the well (may approximately be taken as 1.25 times the design draw down in the pit).

Table 18 shows approximate size and capacities of shallow tube wells installed in different soils. Well capacities in the table have been based on average values of permeabilities given in Table-2. The values of permeabilities in Table-2 are very approximate and vary widely, even for a particular type of soil, depending

Table 18. Approximate capacity of shallow tubewells in different soils. (Blind pipe should be continued at least 50% beyond anticipated drawdown*)

Soil Type	Well dia. (D) (mm)	Screen length Ls, (m)	Slot area Ao, (m^2/m)	Entrance velocity Vs, m/min	Clog factor	Yield q, m^3/day (cusec)
Silty Sand	100	8	0.058 0.098	0.60 0.60	0.5 0.5	200(.08) 331(.13)
		10	0.058 0.098	0.60 0.60	0.5 0.5	250(.10) 422(.17)
		12	0.058 0.098	0.60 0.60	0.5 0.5	300(.12) 506(.20)
Fine Sand	100	8	0.058 0.098	0.90 0.90	0.5 0.5	300(.12) 506(.20)
		10	0.058 0.098	0.90 0.90	0.5 0.5	376(.15) 635(.25)
		12	0.058 0.098	0.90 0.90	0.5 0.5	451(.18) 762(.30)
Medium Sand	100	8	0.058 0.098 0.15	1.10 1.10 1.10	0.5 0.5 0.5	368(.15) 621(.25) 950(.39)
		10	0.058 0.098 0.15	1.10 1.10 1.10	0.5 0.5 0.5	459(.19) 776(.32) 1188(.48)
		12	0.058 0.098 0.15	1.10 1.10 1.10	0.5 0.5 0.5	551(.22) 931(.38) 1425(.58)

* May not be applicable for deep tubewells

Table 18 (Cont). Approximate capacity of shallow tubewells in different soils. (Blind pipe should be continued at least 50% beyond anticipated drawdown)

Soil Type	Well dia. (D) (mm)	Screen length Ls, (m)	Slot area Ao, (m^2/m)	Entrance velocity Vs, m/min	Clog factor	Yield q, m^3/day (cusec)
Coarse Sand	100	8	0.058	1.2	0.5	401(.16)
			0.098	1.2	0.5	678(.28)
			0.15	1.2	0.5	1037(.42)
		10	0.058	1.2	0.5	501(.20)
			0.098	1.2	0.5	846(.34)
			0.15	1.2	0.5	1296(.53)
		12	0.058	1.2	0.5	601(.24)
			0.098	1.2	0.5	1016(.41)
			0.15	1.2	0.5	1555(.63)
Sand (mixture)	100	8	0.058	0.60	0.5	200(.08)
			0.098	0.60	0.5	338(.14)
		10	0.058	0.60	0.5	250(.10)
			0.098	0.60	0.5	423(.17)
		12	0.058	0.60	0.5	300(.12)
			0.098	0.60	0.5	208(.20)

on the particle size distribution, compactness and directional homogeneity of the soil. As such these values should be used with proper care and judgement. Table 17 and Table 18 as well as the design steps mentioned in the previous sections will provide only preliminary estimates as to the number, size, seepage load and capacity of the tubewells. Actual values should be ascertained by test pumping some of the wells installed as per the results of preliminary design. If the results of test pumping vary, the preliminary design should be modified accordingly. The tubewells should be installed as per the requirements of final design.

4.3.4. Design of well points

Procedures for well point design are more or less similar to those of tube well design.

4.3.5. Design of vacuum well and electro-osmotic method

As these methods are very rarely practised in Bangladesh, their design procedures are not included in this text.

4.3.6 Design of pumps

Most of the commonly used dewatering pumps are either suction lift type or force lift type. Suction lift type mostly includes centrifugal pumps while the force lift type includes turbine pumps and submergible pumps. Pumps are usually provided with definite performance curves or characteristic curves (Fig. 55), which should be properly and carefully analysed before selecting a pump for a particular system.

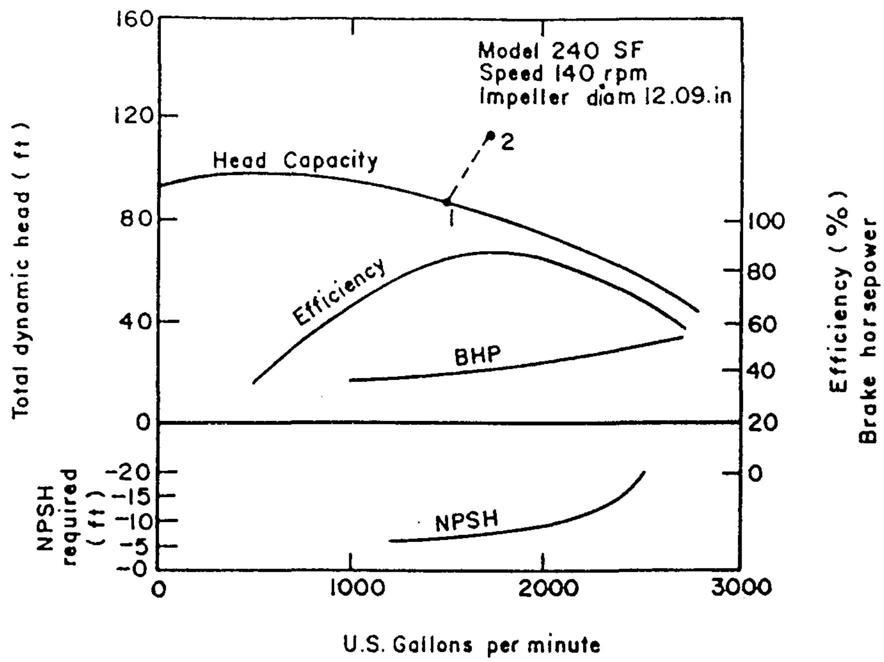
i) Pump characteristics

Some of the common pump characteristics are illustrated below:

a) Total pumping head or Total dynamic head (THD)

The total pumping head is usually called total dynamic head or TDH. This is the sum of all dynamic & potential heads like suction head, discharge head, velocity head and friction head. Fig. 56 illustrates the methods of calculating TDH for various pumping applications.

Fig. 56.a illustrates a deep well pump that has to pump water against the heads h_D, h_v and f_1. TDH = $h_D + f_1 + h_v$.



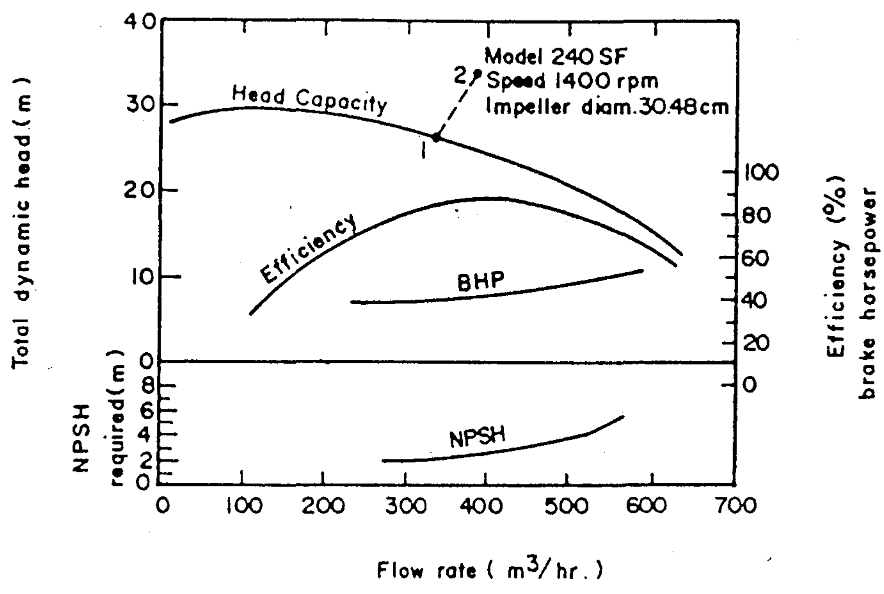
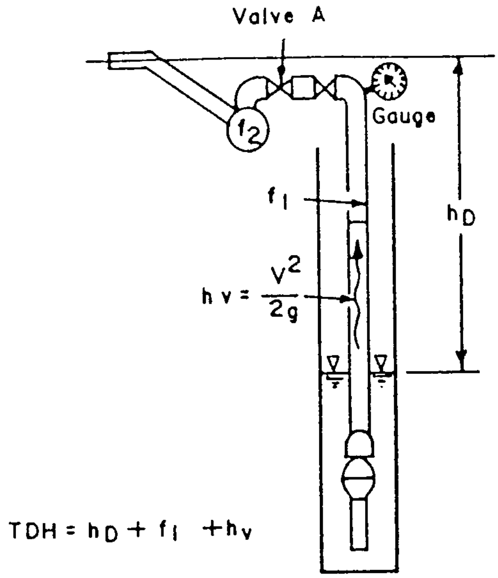
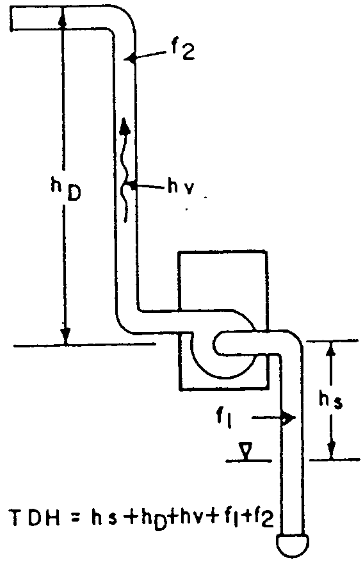


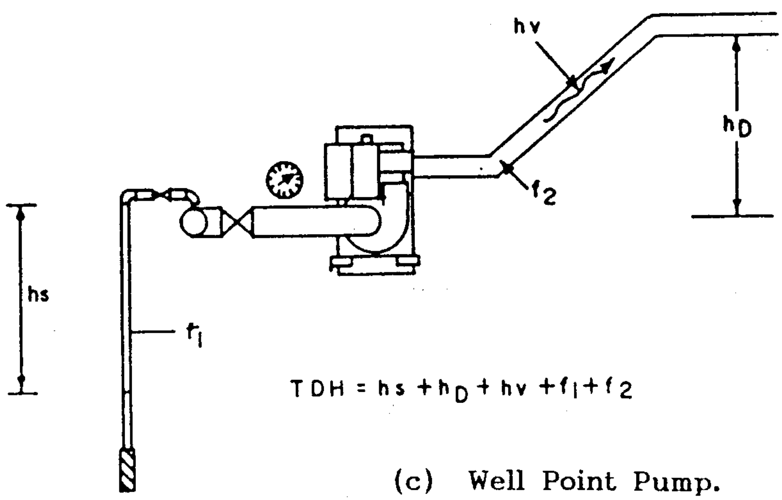
Fig. 55. Basic Pump Performance Curves a) U.S. Units b) Metric Units. Courtesy Moretrench American Corporation.



(a) Well Pump



(b) Sump Pump



(c) Well Point Pump.

Fig. 56. Calculating Total Dynamic Head (TDH).

Fig. 56.b represents a sump pump, which has to pump water against a discharge head, h_D, plus suction head h_s, plus velocity head h_v and friction heads f_1 and f_2. TDH = $h_s + h_D + f_1 + f_2 + h_v$.

Fig. 56.c shows calculation of heads for a well point pump, for which it is difficult to measure the actual suction head. However, an approximate value of h_s, can be estimated as equal to the operating vacuum of the well point pump, usually 64 cm of H_g or 8.5 m of water at see level.

Whatever is the type of pumps, the estimated TDH should always be increased by 10-15% to take care of pump wearing and unforeseen conditions.

b) **Head - capacity curve**

The head-capacity curve (Fig. 55) shows the capacity of pump (discharge volume Q) at various values of TDH. As TDH increase the capacity of pump decreases and vise-versa.

c) **Brake horse power (BHP)**

Brake horse power of a pump is the amount of power that must be applied to the pump. It may be called the input horse power and is greater than the water horse power (WHP) by the amount equal to the hydraulic and mechanical losses in the pump.

d) **Water horse power (WHP)**

It is the output horse power of a pump and is defined as the product of the head and the capacity (with proper conversion factors).

$$WHP = \frac{TDH\ (ft) \times Q\ (gpm)}{3960}, \quad (U.S) \quad \ldots\ldots\ldots\ldots\ldots\ldots(71)$$

$$WHP = \frac{TDH\ (m) \times Q\ (lpm)}{4569}, \quad \text{(metric)} \qquad \text{..............(72)}$$

Where,

gpm = gallon per minute

lpm = litre per minute

e) Efficiency

The efficiency E, of a pump is defined as the ratio of the output horse power or water horse power to the input horse power or Brake horse power. It is a measure of the effectiveness of the pump.

$$E = \frac{WHP}{BHP} \qquad \text{..............................(73)}$$

Fig. 55 shows the efficiency of a pump at various operating points. Required BHP of a pump can be determined by dividing the calculated WHP by the efficiency of the pump.

f) Net positive suction head (NPSH)

This a very important factor for selection of a pump. It can be defined as the absolute pressure in the eye of the impeller minus the vapour pressure of the fluid being pumped. It is a characteristic of pump design and depends on the diameter of the impeller, number & shape of vanes and the smoothness of the impeller surface. A rough surface is more subject to cavitations.

The pump will not operate satisfactorily if the available NPSH is less than the required NPSH. When the pump operates with lower absolute pressure or NPSH on the suction side, cavitation may occur. As the fluid enters the eye of the impeller, eddy currants develop.

If the absolute pressure in this region of high velocity drops below the vapour pressure of the fluid being pumped, the fluid will boil, and small bubbles of vapour will form. When these bubbles pass through the impeller, the pressure rises and the bubbles collapse with violent implosion. This phenomenon is termed as cavitation.

A cavitating pump makes rattling sound as if there is gravel in the impeller. Pumping capacity of a cavitating pump is much below the performance curve. If this cavitation is allowed to continue for long, the impeller surface will be hammered away, the bearings may be damaged and the shaft may be broken.

To avoid cavitation or pump failure, the available NPSH should always be greater than or equal to the required NPSH. The available NPSH is dependent on the job condition i.e. maximum vacuum or suction required for the job. Let us suppose that a pump in Fig. 55 has to apply a vacuum of 23 inch (58.5 cm) of Hg to a well point system. The barometer reading on the day is 30 inch (76 cm) of Hg and the water temperature is 60° F (15° c). Then the absolute pressure on the suction side of impeller will be:

$$P_a = (30 - 23) = 7 \text{ inch of Hg}$$
$$= 7.90 \text{ ft. of } H_2O$$

The vapour pressure of water at 60°F = 0.60 ft of H_2O

Then the available NPSH is given by:

$$(NPSH)_{avail} = 7.90 - 0.60$$
$$= 7.30 \text{ ft of } H_2O$$
$$= 2.23 \text{ m of } H_2O$$

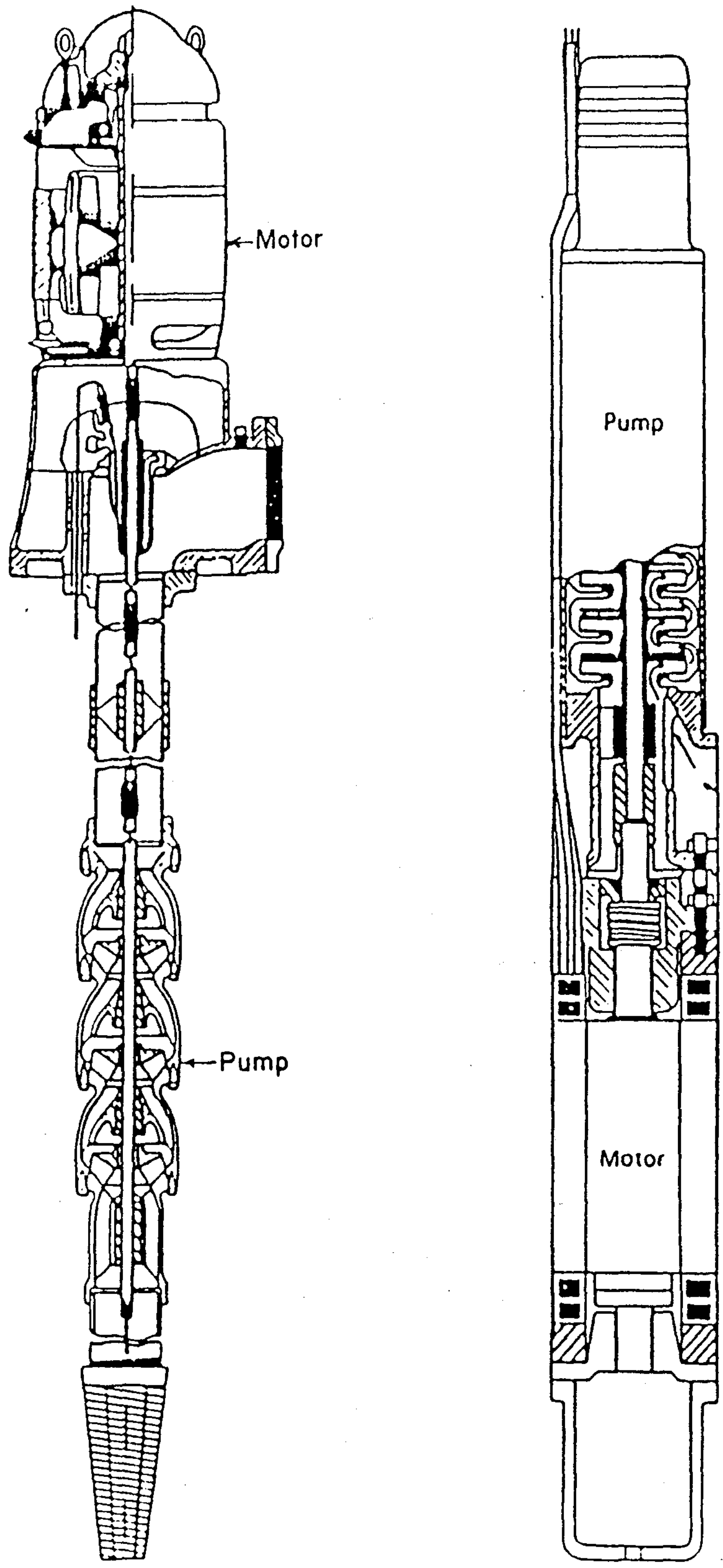
Fig. 55 shows that min. required NPSH for this pump is 1.80m of H_2O So the pump will perform effectively and may not cavitate.

ii) **Pump selection**

Considering all the above factors, a pump should be so selected that it can handle the load at maximum possible efficiency and at the same time can take care of the uncertainties that may occur due to side seepage, rain, and pump failure etc. Usually centrifugal pumps are used for sump pumping and shallow well pumping. The assembly may be mounted with a horizontal or vertical shaft. The horizontal design is more efficient and easy to install & maintain. The suction lifts of centrifugal pumps are very low and are commonly limited to 7.62 m (25 feet) of water. Because of this low suction head, the pumps are often placed at short distance (vertical) above the water level.

Deep wells require high lift large capacity pumps. Most commonly used pumps are deep well turbine and submersible pumps (Fig. 57). The turbine pump has its impeller suspended vertically on a long drive shaft within the discharge pipe. The bowl of the pump contains the impeller and the guide vanes. In a multistage pump, several bowls are connected in series to cope with higher heads. The pump is usually driven by an electric motor at the ground surface and connected by long vertical shaft positioned in the discharge pipe with the help of bearings. As the deep well pumps are submerged, they do not require any priming and are capable of operating under a wide range of water levels.

Submersible pumps are simply deep-well turbine pumps close-coupled to submersible electric motors. The pump efficiency is higher because of direct coupling and continuous cooling due to complete immersion of water. Pump sizes range from small units that fit inside an 8 cm casing to large units having numerous stages. Submersible pumps can lift water from deep wells where long shafts in crooked casing might prohibit installation of a deep-well turbine pump. Besides submersible pumps are easy to maintain and are free from noise. They do not need large over ground installation to provide protection against weathering & flooding.



(a) Turbine Pump (b) Submersible Pump.

Fig. 57. Deep Well Pumps.

5.0.0. DISPOSAL OF DEWATERED DISCHARGE

The discharge from the dewatering system should be disposed in such a way that it does not harm the existing structures or the environment and does not add to the load of the dewatering system by reinfiltration. It should be disposed through properly designed channels or pipe lines. The channels should be lined or unlined depending on whether they run over sandy soils or clayey soils. The outfall should be properly protected against erosion.

6.0.0. EFFECT OF DEWATERING

Dewatering for construction purposes may some times result in serious consequences like settlement of surrounding ground and subsequent damage to existing structures (Fig. 58). But, since most of the construction sites for hydraulic structures are remote from habitations, the effect of ground settlement may not be so serious. However, considering the potential dangers and implication of 3rd party claims, the matter should always receive careful attention.

The process of dewatering may cause settlement due to 1) removal of fines from excavation, 2) Boils, soil piping and slope failure, and 3) consolidation of compressible soils

6.1.0 SETTLEMENT DUE TO REMOVAL FINES

This happens when the wells or well points pump excessive amounts of fines from the surrounding soils due to improper design and construction practices. When the screen slots are not properly selected and gravel packs are not carefully installed, the fines in the soil formation moves into the well through the faulty gravel pack and screen. Removal of these fines from the formation creates voids and subsequent settlements. This can be avoided if the wells are properly designed and constructed.

6.2.0 SETTLEMENT DUE TO BOILS, SOIL PIPING & SLOPE FAILURE

This happens during open pumping or sump pumping where the method is unsuitable. Soil pumping and slope failure occur due to side seepage and boils occur due to critical exit gradient at the bottom of excavation. All these induce loss of formation soil and subsequent settlement of ground. Such problem can be avoided by installing predrainage methods instead of sump pumping.

6.3.0 SETTLEMENT DUE TO CONSOLIDATION

This type of settlement occurs due to consolidation of compressible materials like silts, clays and loose sands. Dewatering lower the water table and removes the buoyancy from the soil and therefore, increases the effective stress (Fig. 58). The effective stress σ at any point is the difference between total pressure (overburden) P, and the pore water pressure (buoancy) P_w, at that point, ie,

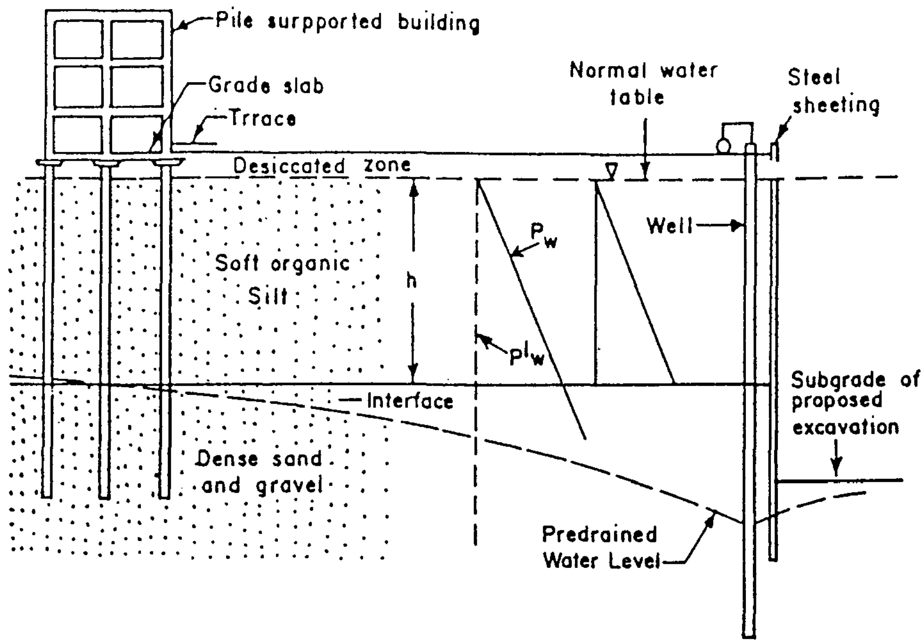
$$\bar{\sigma} = P - P_w \qquad \text{.......................} (74)$$

When pore water pressure is released, P_w tends to be zero and σ tends to be equal to P with a resultant increase in the effective stress, ($d\sigma$).

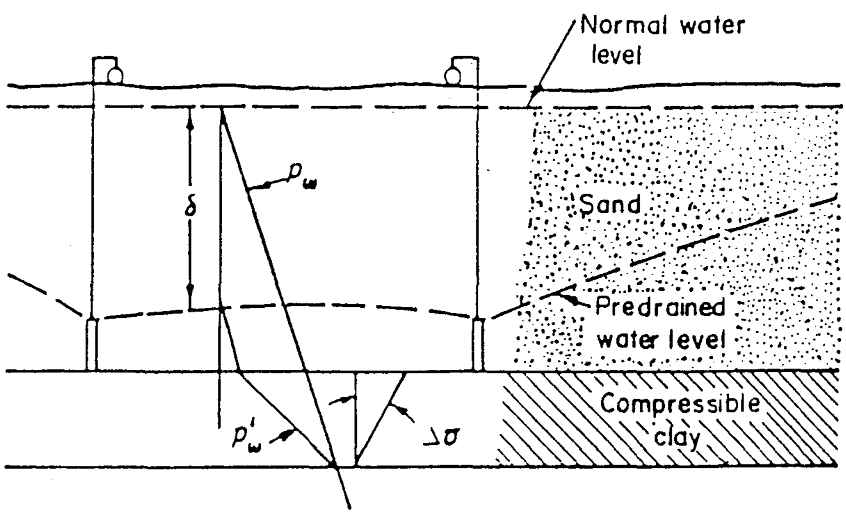
$$\overline{d\sigma} = P - (P - P_w)$$

$$= P_w. \qquad \text{.......................}(75)$$

If the magnitude of this increased loading is moderate, most of the ordinary soils will be capable of supporting this without significant consolidation. However, if the magnitude of the increased loading is very high or the soil in the nearby area is weak, the problem should be investigated prior to dewatering. Fig. 58.a & Fig. 58.b illustrate the effect of dewatering above and below a compressible layer. This type of problem can not be avoided even if dewatering is carried out properly.



b) Dewatering Under Compressible Layer.



a) Dewatering Over Compressible Layer.

Fig. 58. Effect of Dewatering.

DESIGN EXAMPLE

DESIGN EXAMPLE

Design of shallow tubewells (using tables):

1) **Design data**

Type of structure	: 3-1.5 m x 1.8 m sluice.
Ave. Ground level (GL)	: + 10.0 m
Ave. Ground water level (GWL)	: + 7.00 m
Bottom level of excavation	: + 4.50 m
Desired depth of W.T below the bottom of excavation	: 0.5 m
Desired drawdown	:(7.o m -4.5 m) + 0.5 m = 3.0 m
Soil type	: Silty sand.

2) **Design procedure**

a) From table - 2, selected permeability, $K = 4.25$ m/day.

b) From table - 15, length of excavation, $L_e = 64$ m.
Width of excavation, $B_e = 19$ m.
Perimeter of excavation, $P = 166$m.

c) From table - 16,

Equivalent radius, r_s = 19.67 m.(area basis)
= 26.43 m (perimeter basis).
= 23.05 m.(average)

d) From table - 17 (c), Dewatering volume, Q = 2670 m^3/day
= 1.08 cusec.

e) From the rule of thumb use pea gravel (0.5 mm to 6 mm) as filter. Assume (in absence of test results) that 10% size of the filter is 0.75 mm or 0.03 inch.

f) Select a screen slot size equal to 0.03 inch or 0.75 mm.

g) From table 9, for a 4 inch (100 mm) dia. screen, open area = 46.6 sq. inch/ft (0.098 m^2/m).

h) From table 18, q_w = 422 m^3/day (0.17 cusec) for 10 m screen
= 506 m^3/day (0.20 cusec) for 12 m screen

j) From equation 65, number of 100 mm dia. tube wells is given by:

$$N_w = \frac{Q}{q_w}$$

$$= \frac{2670}{422} = 6.35 \approx 7 \text{ for 10 m screen.}$$

$$= \frac{2670}{506} = 5.4 \approx 6 \text{ for 12 m screen.}$$

Select 6 nos. 100 mm ϕ wells with 12 m long screen having slot size = 0.03 inch or 0.75 mm and q_w = 506 m^3/day (0.20 cusec). For pure silt 4 nos. 100 mm ϕ tubewells with 10 m screen might have been sufficient.

k) Minimum spacing = $31.5 x r_w$ = 31.5x100 mm
= 1575 mm
= 1.57 m

Average spacing = 166 m /8 = 20.75 m $\gg$ 1.57 m

(For shallow tube well and deep tube well average spacing will always be greater than the specified minimum spacing and hence need not be checked).

i) Design of pump

a) Design drawdown in the pit = 3.0 m

Drawdown in the well (as per the rule of thumb)

= 1.25 x 3.0 m

= 3.75 m.

b) From table 18, desirable min. depth of blind pipe below G.W.L = 1.5x3.75 = 5.625 m ≈ 6 m.

c) let the pump be placed at 0.5 m above the ground water table.

Then:

length of suction pipe = 6m + 0.5 m = 6.5 m

length of discharge pipe = (10m - 7.5 m)+2 m ext.

= 4.5 meter.

Total length of blind pipe = 6.5 + 4.5 m = 11 meter

d) Assume total friction loss and velocity head as 15% of the combined suction and discharge head i.e.

$(f_1 + f_2 + h_v) = 0.15 \times (3.75 \text{ m} + 2.50 \text{ m}) = 0.94 \text{ m}$

e) From fig. 57, TDH = (3.75 m + 2.5 m + 0.94 m)

= 7.19 m ≈ 24 feet.

f) Select a pump that will operate satisfactorily within the range:

q_w = 368 to 762 m^3/day (0.15 to 0.30 cusec) and,

TDH = 6 to 10 m (20 to 30 feet.)

g) If necessary two or more wells can be connected to a single pump. Keep standby pumps to meet emergencies.

h) Sketch the tubewell as in **Fig. 59.**

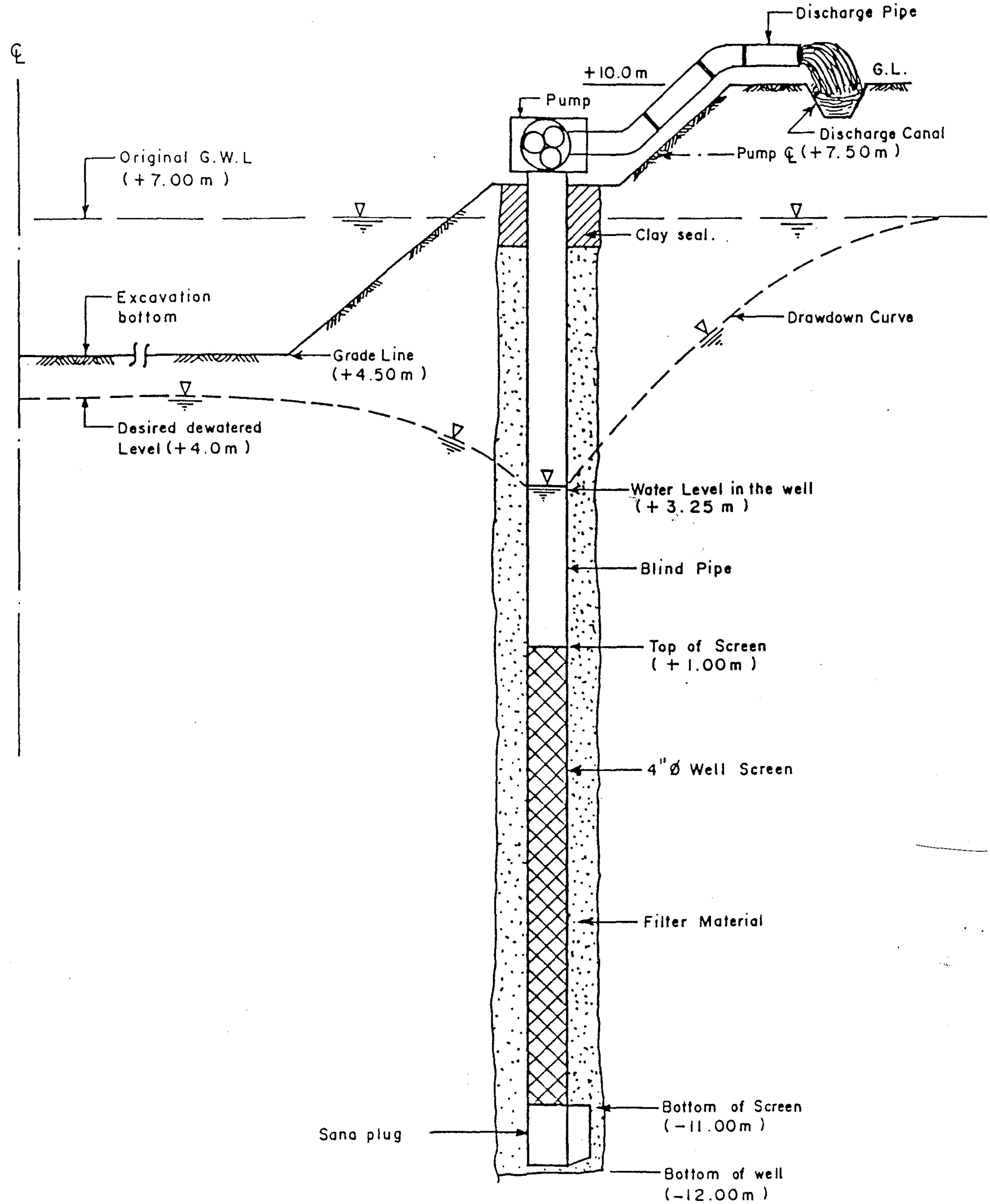


Fig. 59. Detail of Tubewell (as per design example).

CASE STUDY

In this case study F.P.S. units have been used for simplicity and ease of understanding as all original data, maps and other information were in F.P.S. units.

CASE STUDY

NAOTARA SYPHON JALDHAKA, DALIA

1. Location :- Nao Tara, Jaldhaka, Nilphamari.
2. Structure :- 8-vent Syphon under Teesta Canal.
3. Site Plan :- As in Fig. 60.
4. Excavation Lay out Plan - As in Fig. 61.
5. Average original ground level:- + 159 ft. (as reported by field office).
6. Average ground water level(as per soil report):- + 151.34 ft.
7. Maximum ground water level (during dewatering season) = + 151.00 ft.
8. Desired dewatered level:-

   D_w = + 133 ft. (river side)

   = + 143 ft. (country side)

9. Maximum design draw down:-

   S_{max} = + 151 - (+133) = 18 ft. (river side)

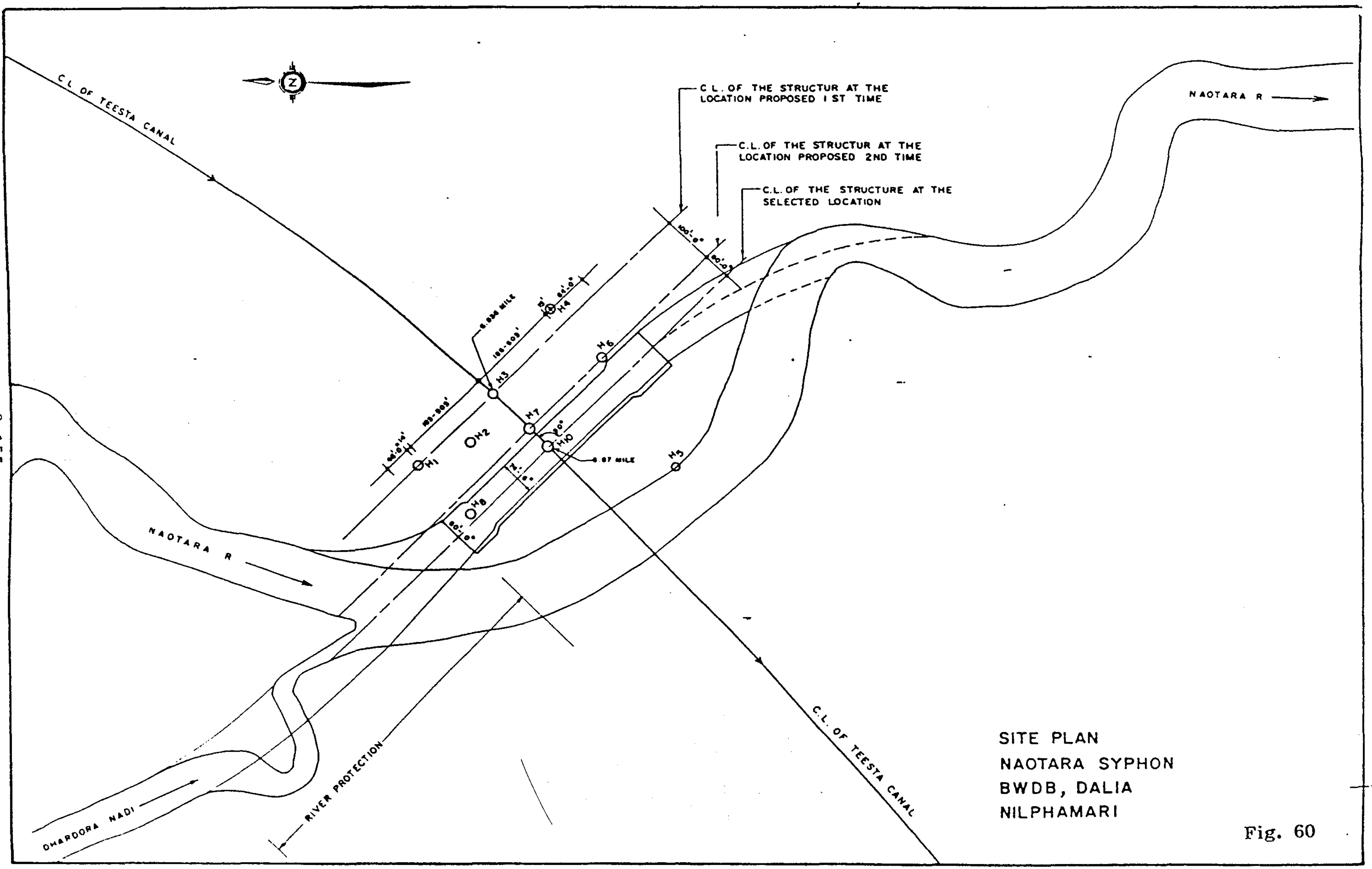
   = + 151 - (+143) = 8 ft. (country side).

10. **Subsoil characteristics:-**

    As indicated in the soil test report, sub-soil formation is somewhat stratified. Materials encountered are mostly loose to medium dense fine sand and medium coarse sand with little silt (Fig. 62). However, as the boring has been done upto 35 feet, actual nature of soil below this level is not known.

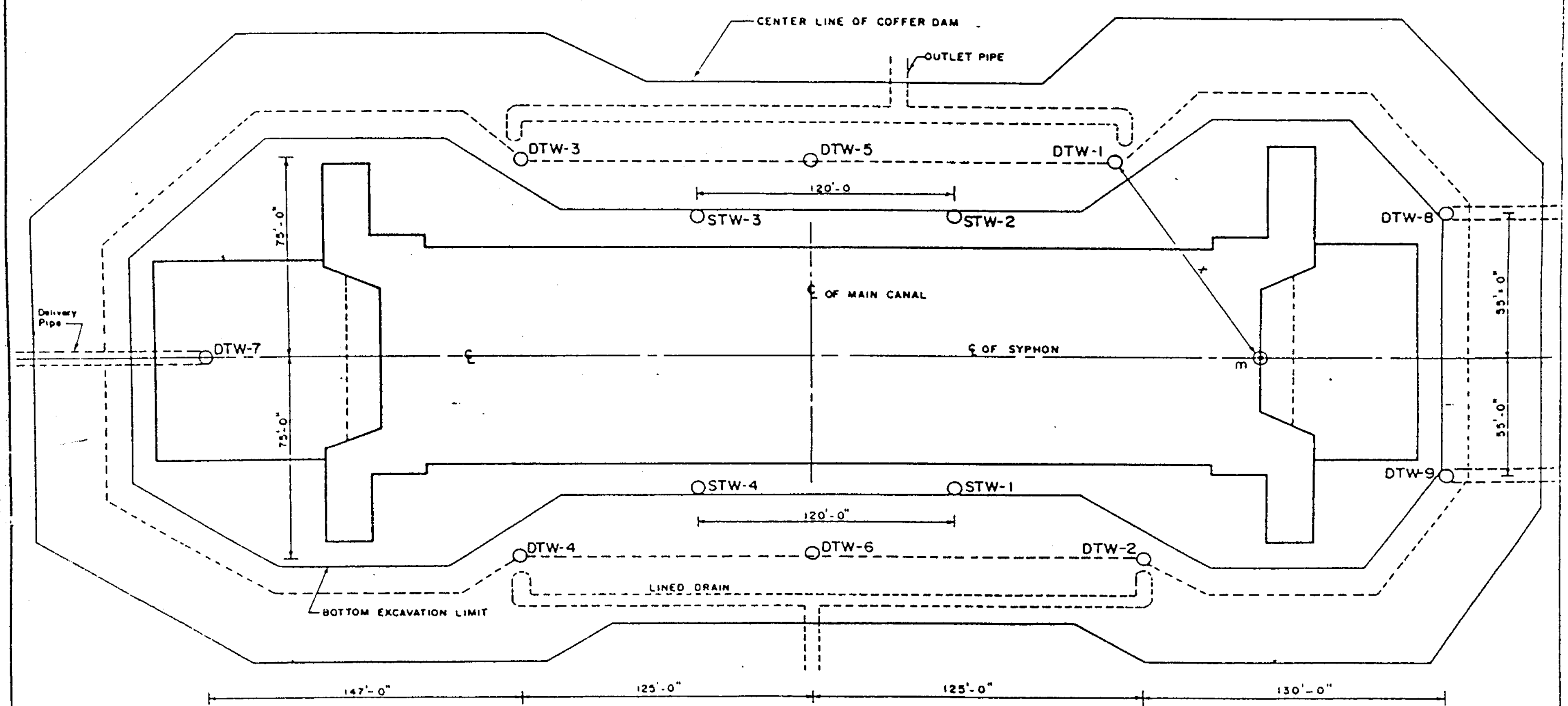
11. **Aquifer characteristics: -**

    Pumping test has been conducted to evaluate the aquifer characteristics at this site. As per results of pumping test, permeability K, of the aquifer is 78 ft/day = 278.88 μ/sec. There is no data as regards the actual depth of aquifer. However, as the well has penetrated upto the level of + 55 ft., it can be safely assumed that the depth h_o, of the aquifer is at least 96 ft. lets take h_o = 100 feet.



SITE PLAN
NAOTARA SYPHON
BWDB, DALIA
NILPHAMARI

Fig. 60



Distancs of point 'm' from different wells, x.

Well \ X	$1/x_1$	$2/x_2$	$3/x_3$	$4/x_4$	$5/x_5$	$6/x_6$	$7/x_7$	$8/x_8$	$9/x_9$
DTW	105'	105'	320'	320	205'	205'	440'	95'	95'
STW	140	140	240	240					
OW									

DTW = DEEP TUBE WELL
STW = SHALLOW TUBE WELL
OW = OBSERVATION WELL

DEWATERING TUBE - WELL LAYOUT
NAOTARA SYPHON
BWDB, DALIA
NILPHAMARI

Fig. 61

GEOTECHNICAL ENGINEERS & CONSULTANTS

PROJECT :- CONSTRUCTION OF NAOTARA SYPHON UNDER TESTA BARRAGE PROJECT

GROUND LEVEL./R L— 144.45

LOCATION :- NAOTARA

GROUND WATER LEVEL 7' ABOVE

BORE HOLE NO. 1/H-13

DATE: TIME:

DATE	NUMBER OF SAMPLE	TYPE OF SAM	DEPTH	THICKNESS	DESCRIPTION OF MATERIALS	LOG	DIA OF BORING	BLOWS ON SPON PER 6 INCH PENETRATION 6"	6"	6"	12"	STANDARD PENETRATION RESISTANCE BLOWS PER FOOT OF PENETRATION 5 10 15 20 25 30 35 40 45 50	INDEX / REMARKS
1-6-88				3'-0"	MEDIUM TO FINE SAND		4"φ	4	5	5	10		
	D 1		5'	5'-0"	FINE SAND SOME SILT			4	2	1	3		
	D 2		10'	22'-0"	GREY MEDIUM DENSE MEDIUM TO FINE SAND TR. GRAVEL			4	6	6	12		
	D 3		15'					6	8	10	18		
	D 4		20'					4	6	8	14		
	D 5		25'					4	6	8	14		
	D 6		30'					6	8	10	18		
	D 7		35'										
	D 8		40'										
	D 9		45'										
	D 10		50'										
	D 11		55'										
	D 12		60'										
	D 13		65'										
	D 14		70'										
	D 15		75'										

INDEX: DISTURBED; UN DISTURBED

CLAY

SILT

SAND

MICA

Fig. 62

12. **Size of the structure/excavation (Fig. 61):**

Since the wells are 25 to 30 ft. away from the edge of structure, the length and breadth of the excavation will be taken as 50 ft. (25+25 ft.) more than that of the structure. Approximate size of the equivalent sink or excavation pit is estimated to be = 600x180 ft.^{2}.

Equivalent radius r_s is given by:-

$$r_s = \sqrt{\left(\frac{600 \times 180}{\pi}\right)} = 185.45 \text{ ft (area basis).}$$

$$= \frac{600 \times 180}{\pi} = 248.40 \text{ ft. (perimeter. basis).}$$

$$\text{Average value of } r_s = \frac{185.45 + 248.40}{2} = 216.92 \text{ ft.}$$

14. Radius of influence, r_o

$$r_o = 3 \times S_w \times \sqrt{K}$$

$$= 3 \times 18 \times \sqrt{278.88}$$

$$= 901.78 \text{ ft.}$$

15. Dewatering volume, Q.

h_o = 100 ft.

h_w = 100-18 = 82 ft.

R_o = 901.78 ft.

r_s = 216.92 ft.

$$Q = \frac{\pi K(h_o^2 - h_w^2)}{\ln R_o/r_s}$$

$$= \frac{\pi \times 78(100^2 - 82^2)}{\ln \frac{901.78}{216.92}}$$

$$= \frac{\pi x78x3276}{1.42}$$

$= 565040.79\ ft^3/day \quad = 6.53$ cusec.

Add 20% for seepage from near by channel.

$Q = 6.53x1.20$

$= 7.83$ cusec.

Use factor of safety $F_s = 1.5$, against uncertainties.

$Q = 7.83x1.5 = 11.75$ cusec.

16. **Number of wells**

Since the dewatering volume is too high, it will require large number of shallow tube wells. Therefore, deep tube-wells are more desirable than shallow tube wells. In reality, there were 9 deep tube wells and 4 shallow tube wells. The ave. capacities of deep tube wells were measured to be 1.25 cusec and that of shallow tube wells were reported to be 0.5 cusec (shallow tube wells were not in operation during field visit). Therefore, total installed capacity is:-

Deep tube-well	- 9x1.25	=	11.25
Shallow tube-well	- 4x0.50	=	2.00
			13.25 cusec

But it was reported that the shallow tubewells were never in operation. As such the actual installed capacity was just equal to the capacity of 9 deep tube wells i.e. 11.25 cusec, which is very close to the design requirement.

17. **Design of wells**

A. **Deep tube-wells:**

q_w = 1.25 cusec

h_o = 100 ft

h_w = 82 ft.

G.L= + 143.98 ft.

pump centre line = + 160 ft (on the berm of cofferdam)

The size and length of tube wells actually installed were as follows:-

Housing pipe	= 14" dia & 50 ft. long.
Screen (filter)	= 6" dia & 60 ft. long.
Total length	= 50 + 60 = 110 ft.

The aquifer material was fine to medium coarse sand and the filter material was pea gravel, but no data as regards gradation, size & uniformity coefficient of the filter were available. However, as the size of pea gravel varies from 2 to 6 mm, and the screen should retain 90% of filter the slot size is assumed to be ± 2.5 mm. From the above information:-

Bottom level of screen = +160-(110)	=+ 50 ft.
Top level of screen =+ 50+(60)	= + 110 ft.
L_s (screen length)	= 60 ft.
A_o (open area/ft)	= 110 in^2/ft.(continuous slot).
	= 0.70 $ft.^2$/ft.
C (clog factor)	= 0.5
V_s (entrance velocity) =0.9m/min.	= 0.052 ft/sec.
q_w (well capacity)	= C. A_o. L_s. V_s
	= 0.5x0.76x60x0.052
	= 1.18 cusec.

Which is very close to the actual capacity measured i.e. 1.25 cusec. Therefore, the design is satisfactory.

B. **Shallow tube well**

q_w = 0.5 cusec.

h_o = 100 ft.

h_w = 82 ft.

The size and length of the tubewells actually installed was as follows:-

Housing pipe = 20 ft, 4" ϕ
Screen (Strainer)= 30 ft., 4" ϕ

The aquifer material were fine to medium coarse sand and the filter material was pea gravel. Based on similar considerations as for the deepwells it is assumed that the screen slot size is ± 2.5 mm (actual data could not be obtained).

From the above information:-

L_s = 30 ft.

A_o = 90 in/ft. = 0.66 ft.^{2}/ft.

C = 0.50

V_s = 0.9m/min. = 0.052 ft^2/ft.

q_w = C. A_o.L_s.V_s

= 0.5x.66x.30x.032

= 0.51 cusec,

which is almost equal to the reported actual capacity as observed during test pumping.

18. **Check for cumulative well effect**

let's select a point m, (fig.66) on the downstream side of the structure. Desired draw down at this point is 18 ft. There are 9 deep tube-wells and 4 shallow tube wells. Effects of these are investigated separately.

a) <u>Deep tube wells</u>

Distances of wells from the point, m

$= x_1, x_2, \ldots\ldots\ldots\ldots\ldots\ldots x_9$

= 105,105, 320, 320, 205, 205, 440, 95& 95 ft.

Value of the logarithmic product, $\ln p(x_n)$,

= ln (105x105x320x320x205x205x440x95x95)

= 46.68

Calculated dewatering volume, Q, = 6.53 cusec.

Design dewatering volume, Q_d, = 11.75 cusec.

Estimated permeability, K, = 78. ft/day.

$$\text{Adjusted permeability, } K_a, = \frac{11.75}{6.53} \times 78$$

$$= 140.35 \text{ ft/day.}$$

Cumulative well capacity, Q_w = 9x1.18 = 10.62 cusec.

= 917568 ft^3/ day

Then:

Cumulative drawdown at point, m, is given by:

$$S_m = h_o - \sqrt{[\{h_o^2 - \frac{Q_w}{\pi K_o}\{\ln r_o - \frac{1}{n} \ln P(x_n)\}]}$$

$$= 100 - \sqrt{[100^2 - \frac{917568}{\pi x 140.35}\{\ln(901.78) - \frac{1}{9} \times 46.68\}]}$$

$$= 100 - \sqrt{[10000 - 2082\,(6.80 - 5.18)]}$$

$$= 100 - \sqrt{[1000 - 3372.84]}$$

$$= 100 - \sqrt{[6627.16]}$$

$$= 100 - 81.40$$

$$= 18.59 \text{ ft.}$$

b) **Shallow tube well**

Since the shallow tubewells were never in operation their contribution to cumulative drawdown was nil. As such cumulative drawdown analysis for shallow tubewells was not carried out.

The calculation in (a) shows that the total cumulative drawdown at point, m, due to deep tubewells only is greater than the required drawdown, which is safe. Closeness of the magnitudes of the calculated and desired drawdowns indicates that the system was correctly designed.

20. **Observation and comments**

Dewatering system for the subproject consisted of 9 deep tube-wells with 6" dia, 60 ft. long screens and 4 shallow tube-wells with 4" dia, 30 ft. long screens. The shallow tubewells were never in operation. Design calculation shows that the system was quite adequate to handle the anticipated seepage volume. Of course, due to lack of proper data & information; the above calculation had to be based on many assumptions. However, since no major problems were encountered during actual execution of the system the assumptions and other design considerations seem to have been more or less correct.

During a field visit to the site, it was observed that there was some minor dewatering problems on the upstream side of the structure. This happened due to the existence of several local sand beds sandwiched between thin clay lenses, which could not be detected during sub-soil investigation. These sandbeds interconnected the excavation pit to the nearby river. Moreover, as the water in these sand beds were trapped by clay lenses, the tubewells could not tap this water. As such some additional sump pumps had to be installed to take care of this unforeseen seepage.

This problem might have been faced more efficiently by installing few vertical sand drains penetrating the confining lenses, if the existence of such lenses could have been identified during subsoil investigation.

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MA/lk/25/4/93

CHAPTER NINE

SPECIAL FOUNDATION METHOD

M. ZAMAN

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SPECIAL FOUNDATION METHOD

1.0 Introduction:

Instances arise where the soils at sites at shallow depths are inadequate for support of proposed structures. So in most of such cases deep foundations are used. The basic situation for a deep foundation is where suitable soil is not available near the usable portion of the structure.

It is a fact that deep foundations i.e piles and caissons, have been used more frequently than justified, because of the mistaken feeling that deep foundation present no construction problems and yield no settlements.

But deep foundation is not the only solution in such cases. Under certain condition sub-soil improvement by compaction, soil replacement, pre-compression, vibroflotation, stone columns, grouting etc can be carried out.

Often these techniques may be adopted as the most economical solution to a foundation design problem viz. providing shallow foundation on treated ground may be more economical than the deep foundation.

2.0 Types of special foundation:

Normally when a soft or very soft deposit is encountered, near the usable portion of the structure, one of the following practices may be followed:

I. Compacted fill below foundation grade.
II. Floating foundation.
III. Subsoil improvement and shallow foundation.
IV. Deep foundation viz. piles, caissons.

2.1 Compacted fill below foundation:

Compacted fill as shown in Fig.1, can be adopted when soft or very soft soil is encountered below the foundation grade. The depth of fill, Dfill, below the foundation grade is to be compacted and should be such that the stress from foundation on soft deposit is within the permissible limit. Permissible stress has to be calculated on the basis of shear strength of soft deposit.

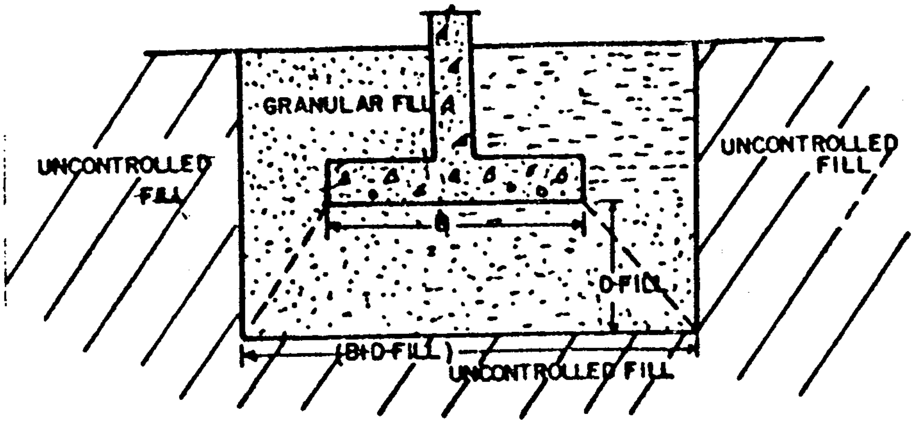


FIG. :1 FOUNDATION ON COMPACTED FILL

2.2 Floating foundation:

In floating foundation technique, a part or full weight of the structure is balanced by the excavated material. If the excavated material is as much as the weight of the structure to be supported, it is evident that the stress condition in the soil after the structure is constructed, is practically the same as before, hence the settlement of the structure will be negligible.

2.3 Sub soil improvement and shallow foundation:

Because many soils can be made into useful construction materials if properly treated, soil improvement has become a part of many present day civil engineering projects. Soil improvement methods reviewed here includes in-situ deep compaction of clayey and sandy soils, precompression with and without vertical drains, injection and grouting, admixture, stabilization, thermal method and soil reinforcement.

Most of man's construction is done on, in, or with soil. As the availability of suitable construction site decreases, the need to utilise poor soil for foundation support increases. In addition, it is becoming increasingly necessary to strengthen the ground under the existing structures to insure stability against adjacent excavation or tunnelling or to improve resistance against seismic or other special loadings. Furthermore, many hundreds of successful projects have shown that through the use of suitable reinforcement materials and systems, the use of nature's most abundant construction material soil can be greatly extended.

The basic concepts of soil improvement; namely, drainage, densification, cementation, reinforcement, drying and heating were developed hundreds or thousands of years ago and remain valid today. The coming of machines in 19th century resulted in vast increase in both the quantity and quality of work that could be done. Among the most significant development of past 60 years are the introduction of vibratory methods for densification of cohesionless soils, new injection and grouting materials and procedures, and new concepts of soil reinforcement.

2.3.1 Methods of Soil Improvement:

Methods of in-situ soil improvement, practised now a days are:

1. Deep Compaction.
2. Consolidation by Preloading and/ or Vertical drains.
3. Grouting, excluding ground water flow and seepage.
4. Soil Stabilization using admixtures and by ion exchange.
5. Thermal Stabilization.
6. Reinforcement of soil.

The report shows a tabular summary of the methods of soil improvement and considerations of factors governing the choice of a method for any given case. (Fig. 2)

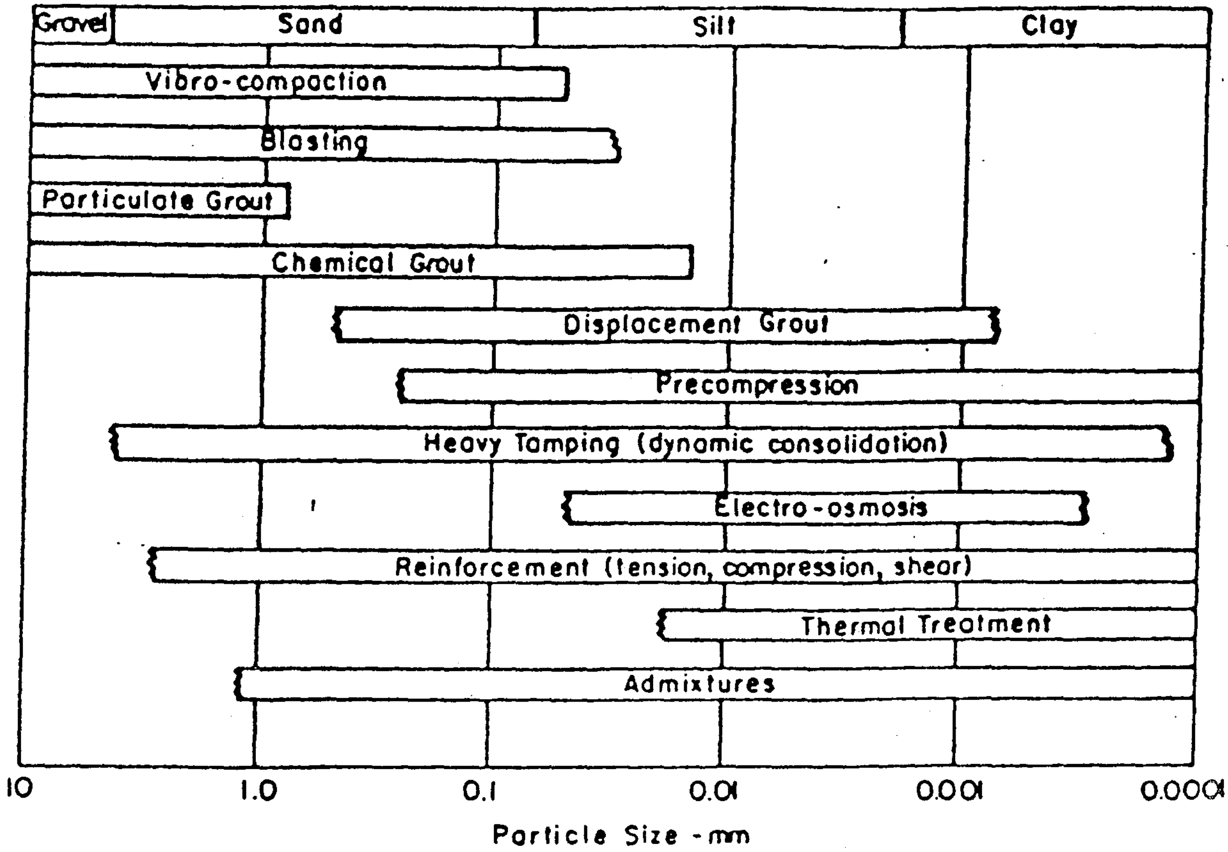


Fig. 2 - Applicable Grain Size Ranges for Different Stabilization Methods

2.3.1.1 Deep Compaction:

The methods of in-situ soil treatment can be divided conveniently into two groups viz (1) those applicable primarily for cohesionless soils and (2) those applicable to cohesive soils. Majority of these methods are indicated in fig.2 along with particle size range for which they are applicable.

Thick deposits of cohesionless soils may require improvements in order to eliminate the subsequent development of excessive settlements and to minimise the possibility for liquefaction under dynamic loading.

In situ densification of loose, cohesionless soil layers is usually done by dynamic methods. Methods that are used for in-situ densification of cohesionless soils are (a) Blasting, (b) Vibrocompaction, and (c) Heavy Tamping.

Vibrocompaction is used herein to refer collectively to all these methods involving insertion of vibrating probes into the ground with or without the addition of backfill material. Compaction piles are also considered in this category.

The ability of any of these methods to accomplish the needed improvement in properties depends on several factors, including:

(a) Soil type, especially its gradation and fines content.
(b) Degree of saturation and water table location.
(c) Initial relative density.
(d) Initial in-situ stresses.
(e) Initial soil structure.
(f) Characteristics of the method used.

2.3.1.(i) Mechanism of densification:

Densification of cohesionless soil layers with accompanying improvement in mechanical properties requires first that the initial soil structure be broken down so that the particles can be moved to new packing arrangements. In saturated cohesionless soils this is most readily accomplished by inducing liquefaction by means of dynamic and cyclic loadings.

In case of methods such as blasting and heavy tamping the compression wave generated by the sudden large energy release can give an immediate buildup in pore water pressure which reduces the shear strength. This wave is followed by a shear wave which is responsible for failure of the mass. After passage of these waves the soil particles settle into new and ultimately, more stable positions.

Vibrocompaction methods are effective in much the same way, except that the energy per event is many times smaller, the vibrations continue over a much longer period, and the effects are felt to distances from the energy of one to two meters instead of upto 10 meter or more as is the case with blasting and heavy tamping.

For partly saturated soils, including some containing fines and many waste fills, densification is mainly by collapse of the soil structure and escape of gas from the voids. The process is much the same as densification by impact compaction as commonly done in the laboratory.

Experience has indicated that it is often easy to densify to a specified high relative density from a loose initial condition than from an intermediate relative density.

2.3.1.(ii) Soil Type Consideration:

Vibrocompaction method is best suited for densification of clean, cohesionless soils. Experience has shown that they are generally ineffective when the percentage by weight of fines (particle finer than 200 mesh sieve or 0.074 mm diameter) exceeds 20 to 25. This is because the permeability of materials containing greater percentage of fines is too low to allow the rapid drainage of pore water that is required for densification following liquefaction under the action of vibratory forces.

Some soils containing greater amounts of fines e.g. some silty sands and leose, can be densified by blasting and heavy tamping, both of which impart large amounts of energy all at once and cause large ground emplacements.

For preliminary planning, it may be considered that the range of particle size distribution shown by Fig.3 will be best suited for densification by deep in-situ methods.

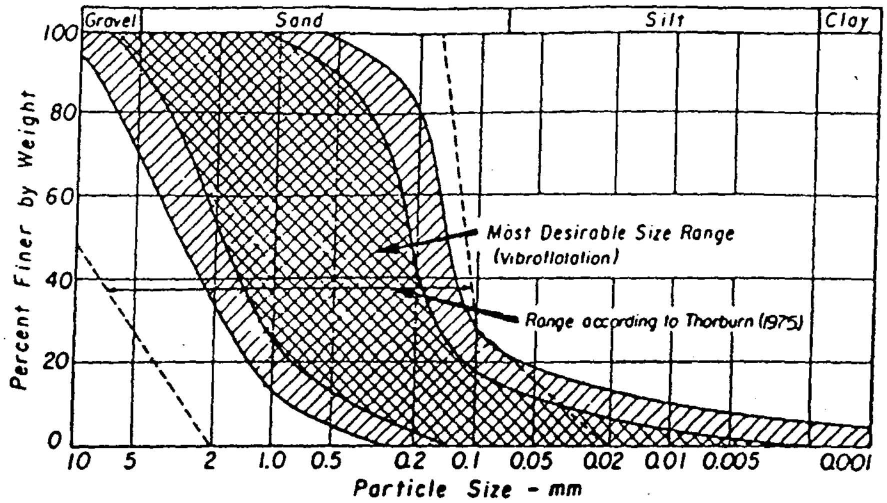


Fig.3 Range of Particle Size Distributions Suitable for Densification by Vibrocompaction

2.3.1.1.1. **Blasting:**

Deep compaction by detonation of buried explosives can provide a rapid, low cost means for soil improvement in some cases. The general procedure consists of:

1. Installation of pipe by jetting, vibration, or other means to desired depth of charge placement.
2. Placement of charge in pipe.
3. Backfilling the hole.
4. Destination of charges according to pre-established pattern.

In some cases the pipe is withdrawn prior to detonation of the charges. In others it is reclaimed after the blast, a new section is welded to the bottom and it can be used again. The explosives used include dynamite, TNT and ammoniate.

Saturated, clean sands are well suited for densification for blasting. Success in any case depends on the ability of the shock wave generated by the blast to break down the initial structure, and created a liquefaction condition for a sufficient period to enable particles to rearrange themselves in a denser packing. It follows, therefore, that the stronger the sand initially, the larger the charges will be required for effective densification. Thus, the greater the depth to which densification is needed and the higher the initial equivalent relative density, the greater the explosive charges required.

There appear to be no generally accepted theoretical design procedure for densification by blasting. Field trials are usually made prior to production blasting. A number of field cases have been summarised by Ivanov (1967) involving treatment depth upto 20 meter. From these experiences the following general guide lines emerge:

1. Charge size: 1.0 to 12.0 kg.
2. Depth of burial: 1/2 to 3/4 depth to bottom of layer to be treated.
3. Charge spacing in plan: 5.0 to 15.0 meter.
4. Number of coverage: 1 to 5, with 2 to 3 usual. Each coverage consists of a number of individual charges. Successive coverage are usually separated by hours or days.
5. Total explosive use: 8 to 150 gm/m^3.
6. Surface settlement: 2 to 10 % of layer thickness.

Significant surface settlements and improvements in the equivalent relative density of loose zones at greater than 30 meter have been achieved by this method. An interesting feature of this work is the observation that although the surface settlement is immediate which means that densification is also essentially immediate, results of cone penetration tests do not indicate an increase in equivalent relative density to the required value for several weeks, reflecting an aging or healing effect following disruption of the initial structure and formation of new one.

It has been possible by blasting to densify sands to equivalent relative density of 75 to 80 percent. In some cases, however, the results may be erratic, initially dense zones may be loosened, and the method is not likely to be effective in the upper one or two meters below the ground surface. Typical behaviour may be summarised as follows:

1. Almost immediate settlement of the ground surface, with little further settlement with time.

2. Initially loose zones show little immediate change in penetration resistance, increases slowly with time until after several weeks the material indicates a marked improvement in properties compared to its initial condition.

3. Zones which are initially very dense may be permanently loosened or weakened by the blast, however, the resultant condition is still likely to be satisfactory.

4. Ultimately, an effective blasting programme results in a deposit in which all the initially loose zones have been suitably improved.

2.3.1.1.2. Vibrocompaction and Compaction Piles:

These methods for deep compaction of cohesionless soils are characterized by the insertion of cylindrical or torpedo-shaped probe into the ground followed by compaction by vibration during withdrawal. In a number of methods a granular backfill is added so that a compacted gravel or sand column is left behind within a volume of sand compacted by vibration. Sinking of the probe to the desired treatment depth is usually accomplished using vibratory method, often supplemented by water jets at the tip. Injection of air at the same time has been found to facilitate penetration to large depths. Upward directed water jets along the sides has also been found helpful in some cases. Soil gradation suitable for densification by vibrocompaction are indicated in Fig.3.

Compaction piles of sand and gravel formed by these methods are also used in soft cohesive soils in which case they function as compression and shear reinforcement.

Ground treatment depths upto 20 m can be achieved by these methods. Depths in excess of 30 m can be attained in some cases.

In some instances penetration resistance are so high following densification by vibrocompaction that relative densities greater than 100 percent are indicated according to conventional correlations. In reality, however, this result is in most instances caused by the increased lateral pressure induced by the vibration process.

Vibrocompaction of large area is done in a grid pattern, either triangular or rectangular, with probe spacings usually in the range of 1.5 m to 3 m on centers. The actual spacings depend on the soil and backfill types, probe type and energy, and level of improvement required.

Although field tests are usually done to finalise designs, there are some guidelines that can be useful in preliminary studies. If it is desired to increase the average density of loose sand from an initial void ratio e_0 to a void ratio e, and if it is assumed that installation of a sand pile cause compaction only in lateral direction, the pile spacings may be determined using :

$$S = [\pi(1+e_0)/(e_0-e)]^{\frac{1}{2}} * a$$

for sand piles in square pattern, Fig.8(a) and

$$S = 1.08[\pi(1+e_0)/(e_0-e)]^{\frac{1}{2}} * a$$

for piles in triangular pattern, Fig.8(b), Where , a=diameter of pile.

Like blasting, vibrocompaction can loosen very dense sands and break up weakly cemented layers. Though the strength of such zones will therefore be decreased, they will usually still be sufficiently dense and strong for the project. Also as observed for sands densified by blasting and heavy tamping, the penetration resistance increases with time after treatment, even though densification and surface settlements are essentially complete at the end of treatment.

When coarse sands or gravels are used as backfill materials for vibrocompaction, the resulting piles can act as drains because of their high hydraulic conductivity relative to the surrounding sand. As such they may serve to dissipate pore pressures from potentially liquefiable deposits and thereby prevent liquefaction.

It is important to note that the use of drains to prevent liquefaction under seismic loading does not eliminate the potential for settlement.

Methods generally used are:

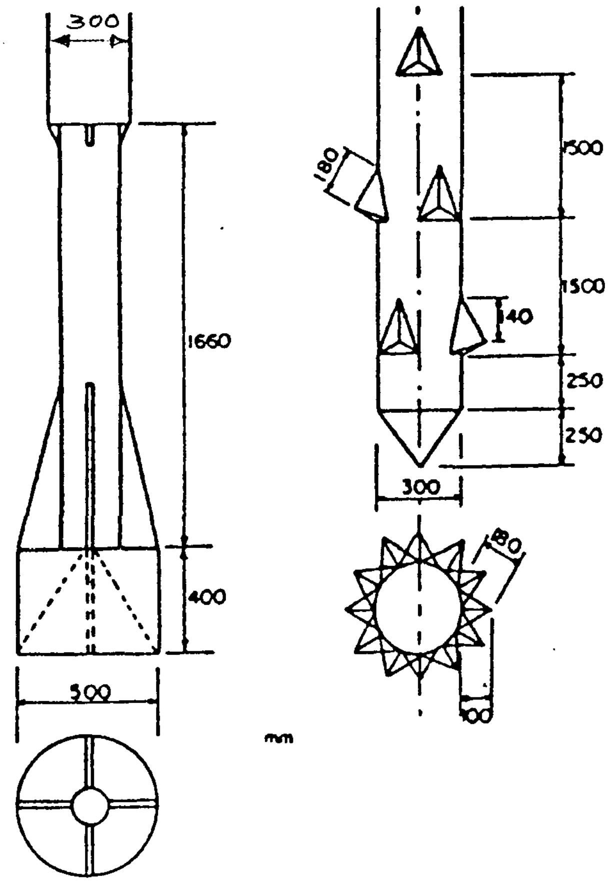
1. Terraprobe/ Vibrating probe
2. Vobroflotation
3. Vibrocomposer
4. Soil vibratory stabilizing system
5. Sand compaction pile

A brief description of some of the more extensively used vobrocompaction method is given below:

1. <u>Vibrating Probes:</u>

The Terraprobe method, developed in U.S.A (Anderson,1974) uses a foster vibro driver pile hammer on top of a 0.76 m dia open tubular probe (pipe pile) 3 to 5 m longer than the desired penetration depth.

Vibro rods developed by Saito (1977), Fig.4 are also driven using a vibratory pile driving hammer. Several cycles of insertion and withdrawal are used in the densification process.



(a) Double Tube Rod

(b) Rod With Projectives

FIG:4 VIBRO-RODS USED IN SAND DENSIFICATION

2. Vibroflotation:

This method was developed in Germany and its development has continued there and in the U.S.A where it was introduced in 1940's.The equipment consists of three main parts: the vibrator, the extension tubes and the supporting crane. A schematic diagram of the equipment and process is given in Fig.5. The vibrator is a hollow steel tube containing an eccentric weight mounted on a vertical axis in the lower part so as to give a horizontal vibration. Vibrator diameters are in the range of 350 to 450 mm and the length is about 5 m including a special flexible coupling.

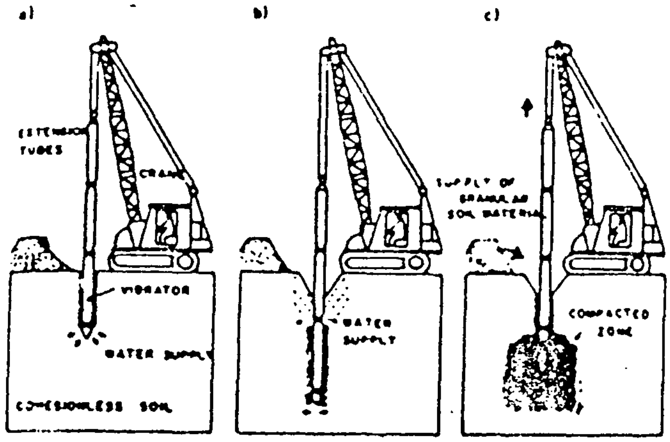


Fig. 5 Vibroflotation Equipment and Process

3. Vibro-Composer Method:

The sand compaction pile method was developed by Murayama in Japan in 1958. The apparatus and procedure used in the composer system are shown schematically in Fig.6. A casing pipe (400 - 500 mm dia) is driven to the desired depth by a vibrator at the top. A sand charge is then introduced into the pipe, the pipe is withdrawn part way while compressed air is blown down inside the casing to hold the sand in place. The pipe is vibrated down to compact the sand pile and enlarge the diameter. The process is repeated until the pipe reaches the ground surface. The resulting pile is usually 600 - 800 mm 1n diameter. The actual diameter can be estimated from the sand volume discharged into the ground.

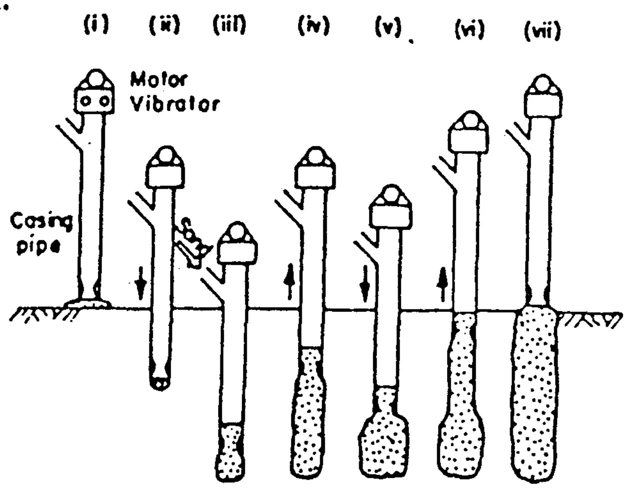


Fig. 6 Construction of Compaction Piles by the Compozer System

4. <u>Soil Vibratory Stabilizing System:</u>

This method, combines both the vertical vibration of a vibratory driving hammer and the horizontal vibration of a vilot depth compactor. The vilot is a special probe of about the same size as vibroflot unit. Sand backfill is used, but water is not used in either the sinking or compaction process.

The gradation of both at the in-situ soil and the backfill, which may or may not be the same material, influence the level of improvement that may be obtained. Coarse sand give better densification than fine sand, evidently the coarser material is better able to transmit vibrations.

Brown (1977) has defined a suitability number for vibroflotation backfills that is given by :

$$\text{Suitability number} = 1.7\ [3/(D_{50})^2 + 1/(D_{20})^2 + 1/(D_{10})^2]^{\frac{1}{2}}$$

In which D_{50}, D_{20} and D_{10} are 50, 20 and 10 percent size grain diameter in mm. Corresponding suitability numbers and backfill ratings are:

0 - 10	Excellent
10 - 20	Good
20 - 30	Fair
30 - 50	Poor
> 50	Unsuitable

The lower the suitability number the faster the vibroflot can be withdrawn while still achieving acceptable compaction. The influence of fines on the level of improvement that can be obtained by vibrocompaction is shown clearly by the data in Fig.7 (Saito, 1977). The data provide excellent support for the rule of thumb that vibrocompaction is ineffective in soils containing more than 20 percent fines.

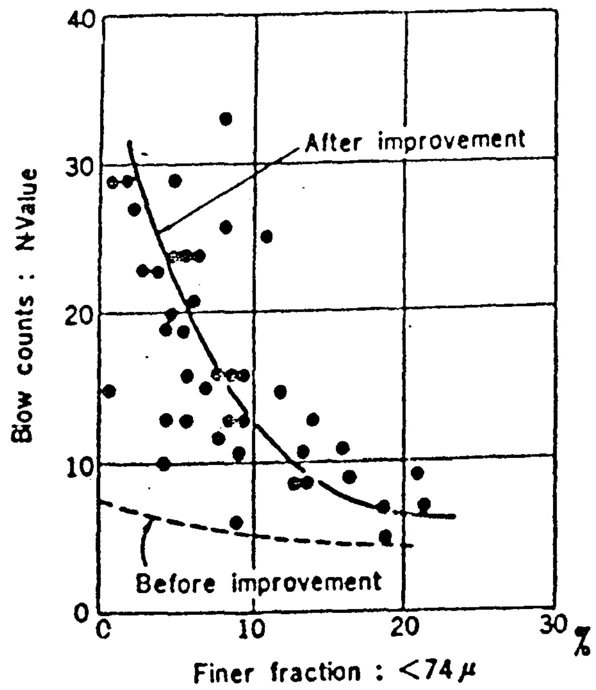


Fig. 7 Effect of Fines Content on Increase in Penetration Resistance by Vibro-compaction (from Saito, 1977)

5. Sand Compaction Pile Method:

The sand compaction pile is one of the most useful construction method for soft ground improvement. The method was established in 1960 by using the vibration type sand compaction pile.

Sand or similar material is injected and compacted into soft ground by vibrating or impacting load and a sand compaction pile of bigger diameter (bigger than the casing pipe) is formed in the soft ground. This method can be applied for improvement of sandy as well as clayey soil with the same equipment.

Principles and objectives for improvement of (a) Soft sandy soil and (b) soft clayey soil are:

(a) Soft Sandy Soil:

i. Compacting soft ground by injecting sand material with vibration load or percussion load.
ii. Increase stability of soil against liquefaction of soil due to earthquake or vibration.
iii. Increasing shearing strength of ground and resistance against horizontal load.
iv. Decreasing compressibility of soil.

(b) Soft Clayey Soil:

i. Increase shearing strength by forming a composite ground of clayey soil and compacted sand pile.
ii. Increasing bearing capacity and stability against sliding.
iii. Decrease consolidation settlement and non-uniform settlement by producing stress concentration on piles.
iv. Sand drains for accelerating initial settlement before application of final load.

This method can be applied to all soft ground site e.g. alluvial plain, marshy places, reclaimed land and marine land. For injecting and compacting sand there are two types of methods, these are vibration type and percussion type. The construction process and equipments used for vibration type of sand compaction pile is same as that of vibrocomposer method (Fig.6).

The equipments used for percussion type are, (1) Derrick/winch, (2) Casing pipe, (3) Hammer (Weighing 1.0 ton and above.The hammer is a solid cylinder, and sectional area is such that it can play within the casing pipe. Diameter of hammer section is about 6.0 to 8.0 cm lower than the opening in casing pipe).

The construction process is as follows:

1. A small hole of about 0.5 m depth is made on the ground with the hammer.

2. Casing pipe is installed at that point and kept vertical. The casing pipe is held in position with the wire rope connected to the winch i.e. downward movement of the casing pipe is restrained.

3. Coarse sand or sand gravel mix is placed within the casing pipe (300 to 400 mm dia) upto a depth of about 1.0 to 1.5 meter.

4. The coarse sand or sand gravel mix placed at the tip of casing pipe is then compacted to form a solid mass by the hammer, with a fall of 1.5m to 3.5m, holding the casing pipe in position. The sand packing forms a hard mass (shoe) at the tip of the casing.

5. The casing is then allowed to go downward with application of hammer blows on the shoe formed at the tip.

6. The casing pipe is penetrated to the desired depth by hammer blows on the artificial shoe.

7. As the casing pipe reaches the desired depth, the pipe is then pulled back by about 30 cm from its lowest position and then held firmly from the winch.

8. The artificial shoe is then detached from the casing pipe with the hammer blow.

9. The casing is drawn to a desired height, sand discharged into the casing pipe to depth of about 1.5 to 2.0 times the drawn up height of casing pipe.

10. The dicharged sand is compacted by the hammer.

The process described in (9) and (10) are repeated until the casing pipe reaches the ground surface. A sand pile of about 1.5 to 2.0 diameter of casing pipe is constructed to the ground surface. (Fig.8)

A detachable M.S or RCC shoe may be used at the tip of casing pipe during driving to the desired depth. The shoe is left at the deepest point during drawn up, sand filling and compaction process.

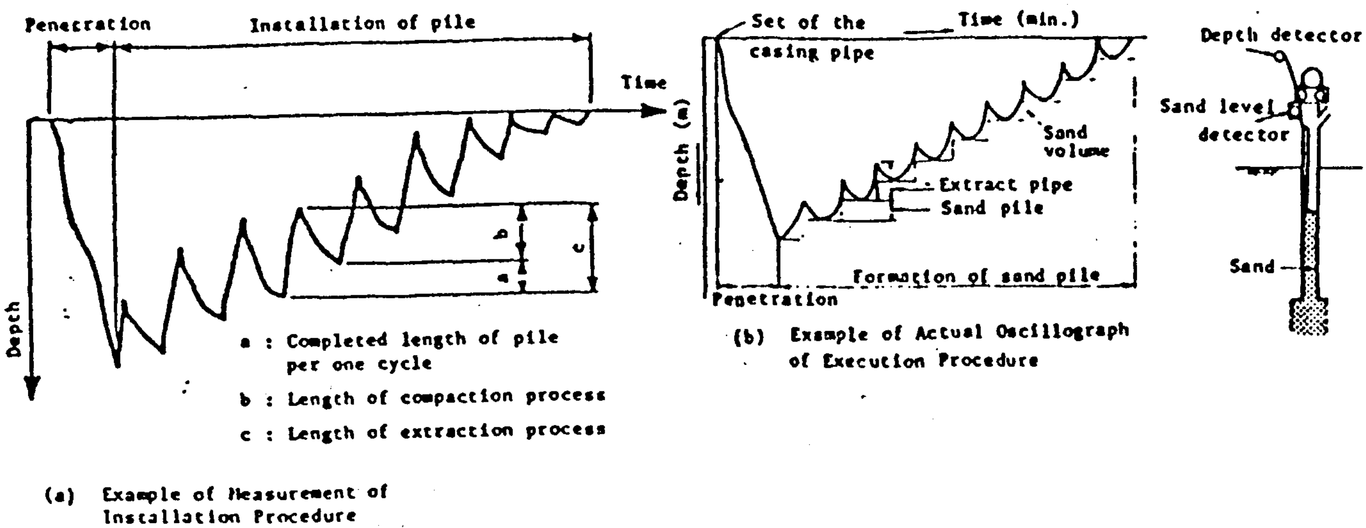


Fig. 8 Example of the Execution Procedure for the COMPOZER Method

A. SAND COMPACTION PILE FOR SANDY SOIL

In order to stabilize loose sandy ground, the spacing between sand piles 's' in Fig.9, is designed to densify the sandy ground from its initial void ratio e_0 to desired e, at which the ground is assured of allowable settlement and stability against failure. In practice the void ratio of the ground is usually estimated from the blow number of SPT, as per correlation chart given by Gibbs and Holtz (1957), Fig.10.

It is assumed that installation of compacted sand piles causes compaction of surrounding soils in lateral direction only, the required spacing 's' is given when sand piles are driven in square arrangement as shown in fig.9(a),

$$S = [\pi(1+e_0)/(e_0-e)]^{\frac{1}{2}} * a = [(1+e_0)/(e_0-e) * As]^{\frac{1}{2}}$$

Where, a = radius of sand pile

a_s = improvement replacement ratio

As = volume of sand per 1 meter depth of sand pile

A = volume of soil per 1 meter depth of ground assigned to sand pile.

Similarly when sand piles are driven to a regular triangular arrangement , as shown in Fig.9(b),

$$S = 1.08[\pi(1+e_0)/(e_0-e)]^{\frac{1}{2}} * a = 1.08[(1+e_0)As/(e_0-e)]^{\frac{1}{2}}$$

If one of the two factors 's' and 'a' is specified, the other can be determined by the equation corresponding to the pile arrangement.

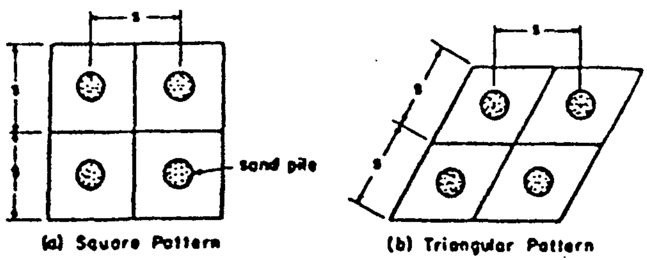


Fig. 9 Usual Vibrocompaction and Compaction Pile Patterns

A.1. The procedure of design may be elaborated as follows;

(1) Bearing capacity and the expected settlement is estimated from N-value = No, from SPT. Relative density and the angle of internal friction are also estimated or calcuted. Terzaghi's formula or Teng's formula may be used for estimating the bearing capacity.

(2) Settlement is estimated by the formula

$$S = [\ 1/(128N)]\ [\ B/(B + 0.3)\ ]^2 * P \quad (\text{Terzaghi-Peck})$$

Where, S = Settlement (m)

N = Average N-value

P = Load acting on foundation (t/m^2)

B = Width of foundation (m)

(3) Required N-value = (N_1)

Required N-value = (N_1) is estimated from allowable bearing capacity and allowable settlement by above formulas.

(4) Replacement ratio (a_s)

The relative density D_{R0}, D_{RI} and the void ratios e_0, e_1 are estimated from the SPT-values N_0, N_1 for before and after improvement respectively (Fig-10).

$$a_s = (e_0-e)/(1+e_0) = \Delta e/(1+e_0) = As/A$$

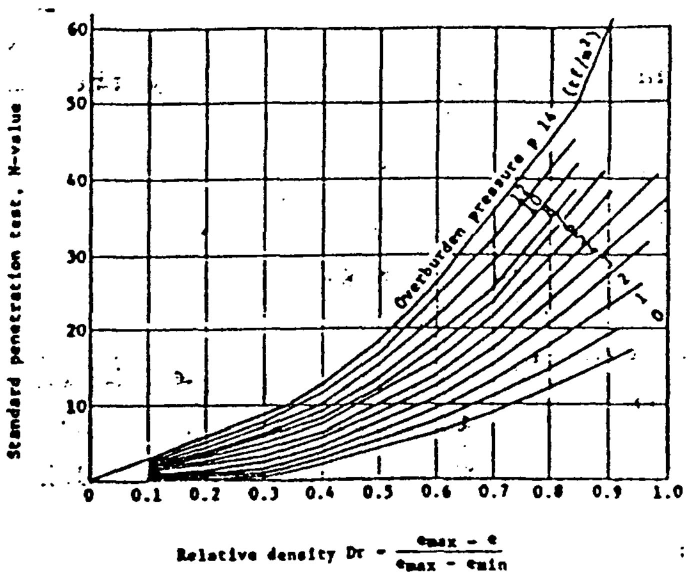
(5) Pile pitch (s) and sand force volume 'As'

Sand force volume As (m^3/m) for the area to be improved and the replacement ratio a_s are calculated as:

$As = a_s * A$

Where, $A = s^2$ [for square arrangement, Fig.9(a)] and $A = 3/2s^2$ [for triangular arrangement, Fig.9(b)].

Sand force volume 'As' is the volume of sand after compaction. So an allowance of 20-30 % is to be taken for measuring sand before injecting.



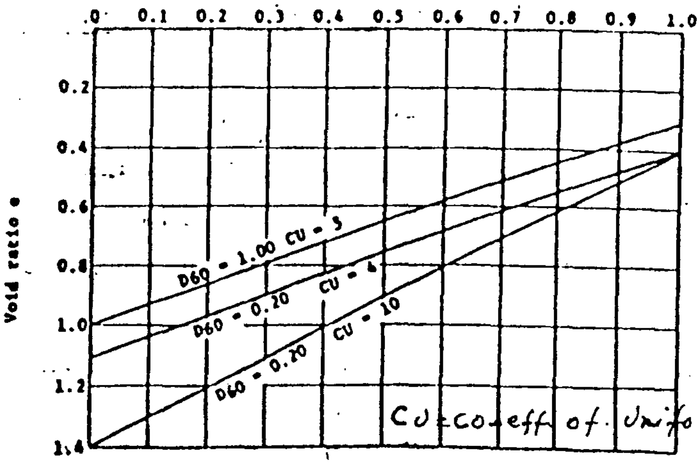


Fig.10. Relationship Between N-value and Relative Density (Gibbs and Holts, 1957)

(6) Selection of pile pitch 's' and sand force volume 'As':

Many combinations of 's' and 'As' are obtained from step (5). Selection of combination is made on consideration of cost, workability and construction period,(Fig.11 and 12).

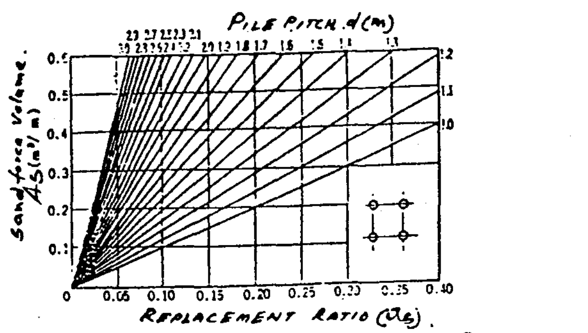


FIG. 11 :- REPLACEMENT RATIO, SAND FORCE VOLUME & PILE PITCH (SQ. ARRANGEMENT)

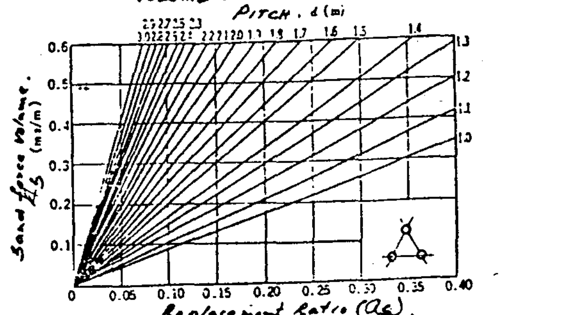


Fig.11

FIG.12 :- REPLACEMENT RATIO, SAND FORCE VOLUME AND PILE PITCH: (TR. ARRANGEMENT).

A.2. 'as' from past records:

Based on records of application of this method on soft sandy soil, curves have been prepared showing relations between a_s, N_0, N_1, and N_p.

N_0, N_1 and N_p being the SPT values before the improvement, at mid points between improved piles and at center of improved piles respectively. 'as' can also be estimated from these curves (Fig.13 and 14).

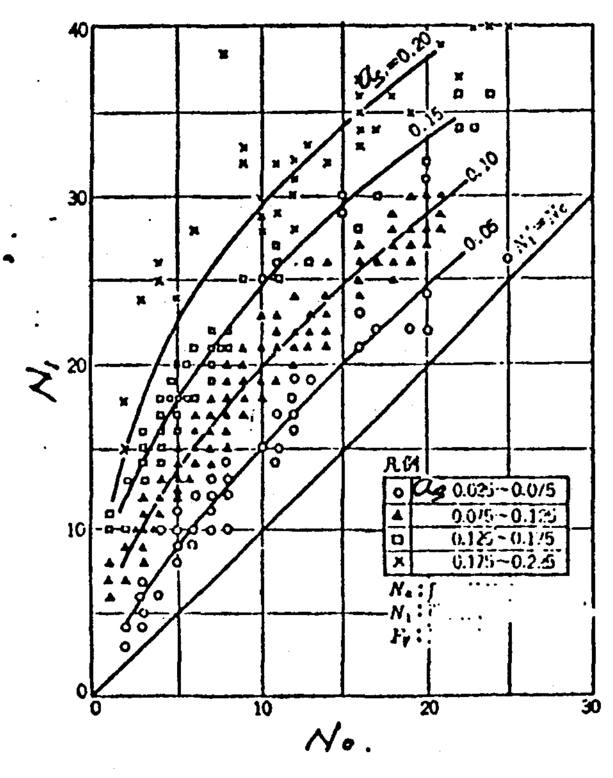


FIG. 13:- RELATION BETWEEN N_0, N_1 & a_s.

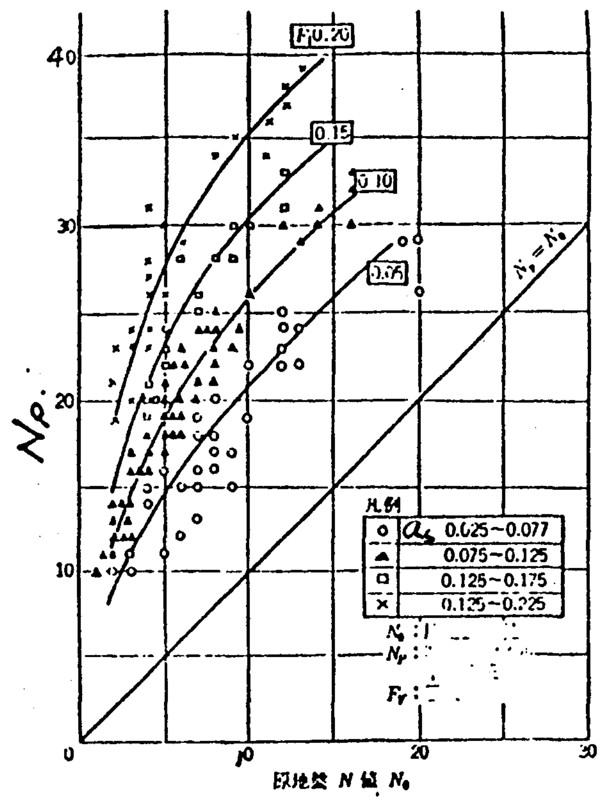


FIG. 14:- RELATION BETWEEN N_0 N_p & a_s.

The design of sand compaction pile for sandy soil can also be performed by use of nomographs. Fig. 15, shows the nomograph for the improvement of sandy soft ground.

EXAMPLE

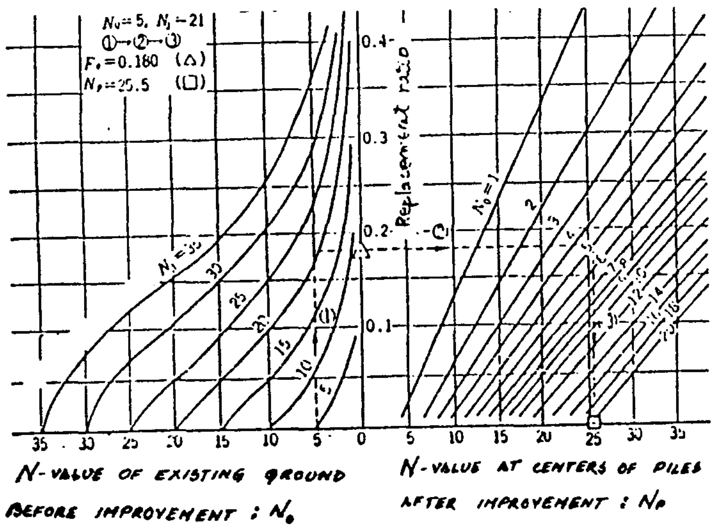


FIG-15:

A.3. The design example, using the nomograph, is as follows:

(1) SPT value of existing ground (No) is No=5, desired SPT value (N_1) among piles is N_1 = 21. No = 5 is marked on horizontal axis, vertical upward line is drawn from No = 5 to N_1 = 21 (arrow-1). Horizontal line (arrow-2) connects N_1 = 21 to No = 5, to right hand side. From the intersection of arrow-2 and No = 5, vertical line is drawn (arrow-3) to connect horizontal axis showing Np values. Np = 25.5 in this case.

(2) Arrow-2 shows 'a_s' = 0.18 (replacement ratio) on vertical axis.

(3) Pile pitch and sand force volume is then selected from Fig.11 and Fig.12, depending on square or triangular arrangement of piles.

B. SAND COMPACTION PILE FOR CLAYEY GROUND

When clayey ground with compacted sand piles (i.e., composite ground) is loaded, the bearing stresses on the sand piles and on the clayey ground become unequal due to the difference of rigidity of each material. The stress concentration on the sand piles increases with the increase of settlement of clayey ground.

Piles in soft clayey soil forms a composite ground of clayey soil and sand piles, depending on cohesion of clayey soil and compacted sand pile. On application of load different stress occur on clayey soil and on sand pile because of difference of index properties and engineering properties of soils (Fig. 16). When load is applied on composite ground, the stress σ, on composite ground is expressed by the equation:

$$\sigma A = \sigma_s A_s + \sigma_c (A-A_s) \quad \ldots\ldots\ldots \quad (1)$$

Where, A = Area covered by each pile

A_s = Area of sand pile

A_c = Area of clayey soil = $(A-A_s)$

σ_s = Stress in sand pile

σ_c = Stress in clayey soil

With distribution ratio, $n = \sigma_s/\sigma_c$ and replacement ratio $a_s = As/A$, the equation becomes

$$\mu_s = \sigma_s/\sigma = n/[1+(n-1)a_s] \quad \ldots\ldots\ldots \quad (2)$$

$$\mu_c = \sigma_c/\sigma = 1/[1+(n-1)a_s] \quad \ldots\ldots\ldots \quad (3)$$

Where, μ_s = Co-eff. of stress distribution in sand

μ_c = Co-eff. of stress distribution in clay

Piles and the clayey soil exhibit shearing strength under load at certain depth in case of a slip circle failure (Fig 17). In that case shearing strength τ_{sc} of composite ground is

$$\tau_{sc} = \tau_c + \tau_s$$

$$= (1-a_s)\ C_o + a_s(\mu_s * \sigma + \gamma_s * z) \tan \phi_s * \cos^2\theta \quad \ldots. \quad (4)$$

Where, τ_c = shearing strength of clayey soil

τ_s = shearing strength of sand pile

a_s = Replacement ratio of sand pile

C_o = Cohesion of original clayey soil

γ_s = Unit weight of sand in pile

ϕ_s = Angle of internal friction of pile sand

θ_s = Angle of slip surface against horizontal plane

Due to consolidation of clayey soil by the vertical forces, cohesion of soil 'C' increases as follows:

$$C = C_o + \mu_c * \bar{U} * m * \sigma \quad \text{..........} \quad (5)$$

Where, C = Cohesion of clayey soil after consolidation

$\bar{U}$ = Degree of consolidation

m = Rate of strength increase

From equation (4) & (5)

$$\tau_{sc} = (1-a_s)(C_o + \mu_s*\sigma*\bar{U}*m) + a_s(\mu_s*\sigma + \gamma_s*z)\tan\phi_s * \cos^2\theta \text{.....} \quad (6)$$

The distribution ratio n, between μ_s and μ_c varies with process of loading, improvement of soil etc. But past records on completed projects show that n varies from 3 to 5.

Sand pile in a composite ground accelerates consolidation of clayey ground composite foundation is characterized by its effect on reduction of consolidation settlement.

The settlement 'S_o' of a clayey subsoil sustaining load σ is estimated by the equation:

$$S_o = Mu * \sigma * z \quad \text{.........} \quad (7)$$

Where , Mu = Modulus of volume compressibility

z = thickness of the layer

Taking effect of reduction of stress on clay ($\sigma_c = \mu_c * \sigma$) as expressed by equation (3), settlement 'S' of the composite foundation is estimated by the equation :

$$S = Mu\ (\mu_c * \sigma) z \qquad \ldots\ldots\ldots\ldots \qquad (8)$$

Comparing (7) & (8) settlement reduction ratio 'ß' equals

$$\beta = S/S_o = \mu_c = 1/[1+(n-1)a_s] \qquad \ldots\ldots\ldots \qquad (9)$$

B.1. Design Procedure:

The Procedure of design of sand compaction pile for clayey soil is as follows:

i) N-value at centre of pile (Np) is obtained by assuming sand force volume As = 0.3 to 0.6 m^3/m, using Fig. 18.

ii) Angle of internal friction for sand pile (ϕ_s) is obtained from Fig.19, against Np from step (1)

iii) μ_s is computed from equation (2) by determining the distribution ratio of stress, ranging from n = 3 to 5

iv) Shearing strength of composite ground τ_{sc} is calculated from equation (4). a_s, C_o, μ_s, ϕ_s and stability analysis is carried out .

v) Different values of a_s is assumed for obtaining requisite safety factor.

vi) If designed safety factor is not obtained by varying a_s, preconsolidation by pre loading may be considered.

vii) Settlement 'S' of the composite ground is obtained by equation (9) after getting the settlement 'So' of the original ground.

viii) If 'S' is more than the allowable limit, design is to be reviewed with revised 'as'.

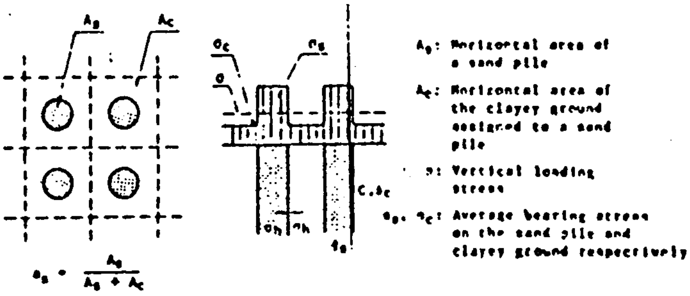


Fig. 16 Diagram of Composite Ground

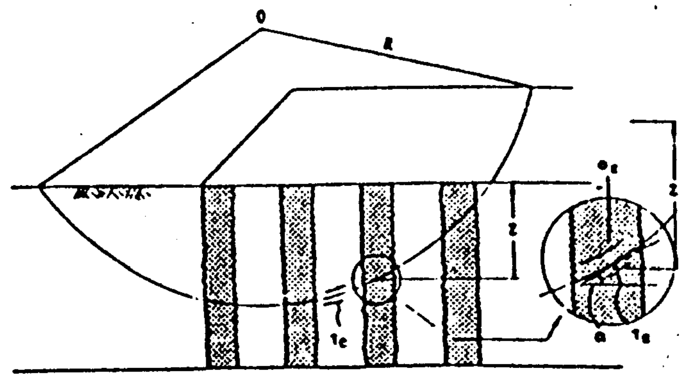


Fig. 17 Diagram of the Circular Sliding Surface Method Including Sand Piles

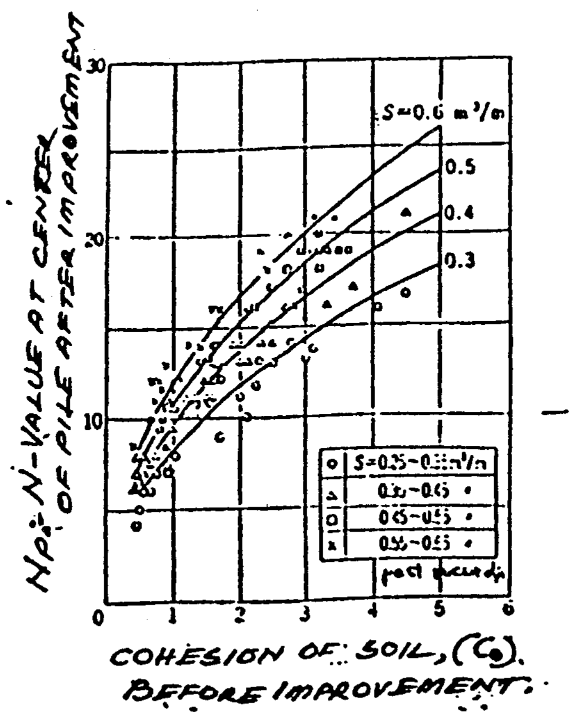


FIG.18:- Relation between Co & Np. (Clayey soil).

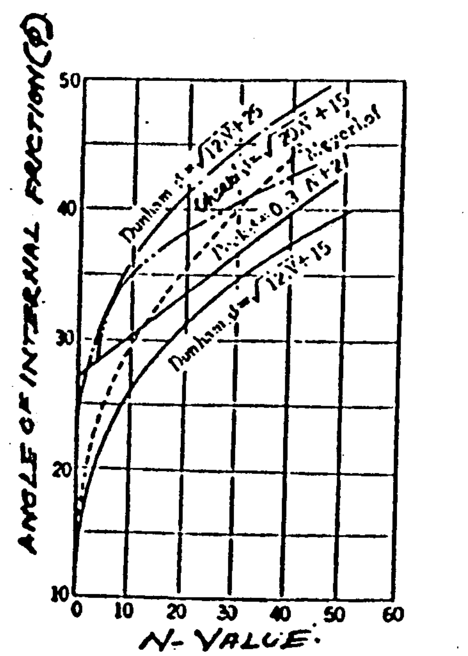


FIG. 19 . Relation between N-Value and φ.

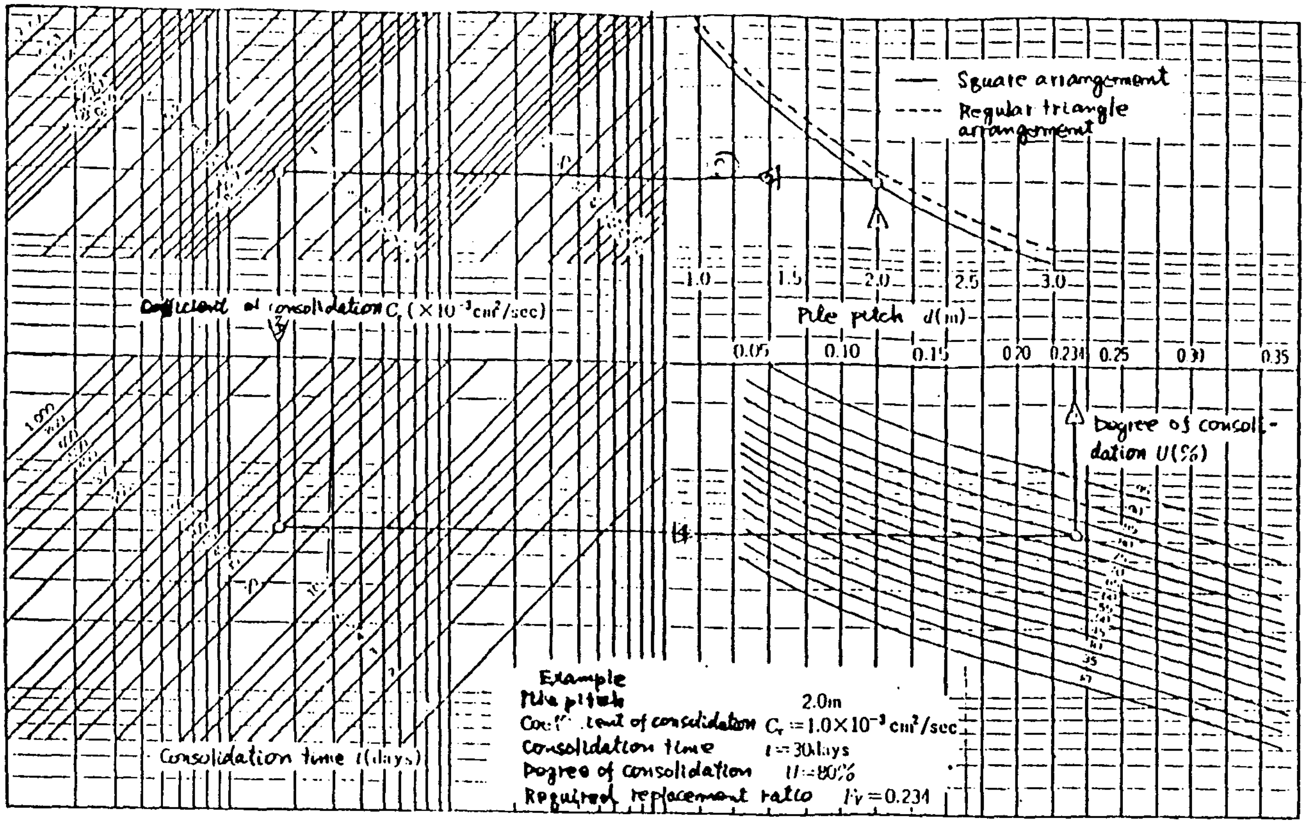


Fig. 20 Nomogram for obtaining Fv from degree of consolidation

B.2: 'as' from nomograph:

Replacement ratio as may also be obtained by using nomograph (Fig. 20), from the assumed degree of consolidation. The procedure is as follows :

(1) Assume pile pitch 's' on arrangement of piles (arrow-1).

(2) Horizontal line is drawn (arrow-2) from assumed pile pitch 's' to the point showing co-eff. of consolidation (Cv).

(3) Vertical line at right angles to arrow-2, is drawn to connect co-efficient of consolidation (Cv) to the line of assumed consolidation time (t), (arrow-3).

(4) Horizontal line at right angles to arrow-3, from selected time ~~(t)~~ of consolidation is drawn to connect degree of consolidation ($\bar{U}$), (arrow-4).

(5) The required replacement ratio (a_s) is obtained through a vertical line (arrow-5) drawn on horizontal axis from degree of consolidation.

C. SUPERVISION AND QUALITY CONTROL

I. Quality of pile sand including distribution of grain size and size of grains shall be checked to ensure the compaction as well as drainage capacity of sand pile. The grain size distribution should be as per distribution curve shown Fig. 27.

II. Effect of improvement in sandy soil ground is to be confirmed by executing Standard Penetration Test (SPT) or CPT among piles and centre of piles.

III. Effect of improvement of clayey soil ground is to be confirmed by SPT or CPT among piles and at centre of piles and also unconfined compression test of ground among piles.

IV. Diameter, length, and continuity of piles are checked by measuring sand force volume by per unit length of executed pile and total sand force volume for each executed pile.

V. Required volume of sand measured on the ground is 120% to 130% of the designed volume

D. Evaluation of Treated Ground:

Measurements of the effectiveness of the deep compaction can be made using one or more of following methods:

1. Surface settlements markers.
2. Volume of soil added to fill craters, to form compaction piles, or to carry out a vibrocompaction process.
3. Standard Penetration Tests (SPT).
4. Cone Penetration Tests (CPT).
5. Pressuremeter Tests (PMT).
6. Pile Driving Resistances.
7. Plate Load Tests.

Settlement measurements and SPT, CPT or PMT are the most commonly used methods. The CPT is particularly useful because it provides a continuous record of penetration resistance with depth, it is fast, and it is well suited for use in sands. Penetration tests for evaluation of improved grounds are usually done at locations intermediate between probe points in order to provide the most conservative estimate of improvement.

E. DESIGN EXAMPLE FOR SAND COMPACTION PILE IN SANDY SOIL (CASE STUDY)

Project : Pabna IRD Project

Structure : Kaitola Pumping Station

Data : Imposed load : 20.0 t/m^2
Allowable settlement : 25 to 30 mm
Average ground level : (+)10.00 to 10.40 m
Foundation level : (+)2.40 m (PWD)

Information on foundation soil :

Bore holes (Fig. 21) conducted at site, show that the soil upto El.(-) 3.60 m PWD is mainly very fine sand, with some deposits of silt and clay in some layer.

Allowable bearing capacity at foundation soil at the level of foundation calculated by Terzaghi's bearing capacity formula is 70.00 t/m^2, considering an average N value, of N_0 = 10, effect of water table has also been considered.

Settlement computed by the formula, $S = 1/(128N)[2B/(B+0.3)]^2 * P$,

is $S = 1/(128*10)[2*17/(17+0.3)]^2*20 = 0.06$ m
$= 60$ mm $>$ allowable

A firm bearing layer is found at El. (-) 3.65 m. So a 6.0 m thick layer from foundation level needs to be improved.

Estimated that SPT value of soil above El. (-) 3.65 will be 20 or more after improvement.

With improved N-value, N_1 = 20,

$S = 1/(128*20)[2*17/(17+0.3)]^2*20 = 0.03$ m = 30 mm, So OK

I) Length of pile :

Selected length of pile = 6.0 m

as the thickness of weak layer is 6.0 m.

II) Applied formula : (For Nomograph, Fig. 22)

$$N_1 = a_s \, N_p + (1-a_s)N_1$$

where, N_1 = N-value after improvement among sand piles

a_s = Replacement ratio

N_p = N-value at centre of pile

In designing sand piles for sandy subsoil by nomograph (Fig. 22), the N-value of original ground No. and the required N-value after stabilization N_1 are known, an improved N-value N_1' is found assuming an adequate value of replacement ratio a_s. The same procedure has only to be repeated with varied values of a_s until this N_1' value becomes equal to required value of N_1.

III) Replacement ratio :

With No = 10 (SPT value before improvement) and

N_1 = 20 (SPT value among piles after improvement),

From DESIGN CHART-1 (Fig. 22)

a_s =0.05	N_1 = 15
a_s =0.10	N_1 = 20
a_s =0.15	N_1 = 26

IV) Pile pitch; and sand force volume:

From Design Chart-II (Fig. 23), square arrangement,

pitch (m)	1.5	1.8	2.1
sand force volumes (m^3/m)	0.28	0.33	0.45
pile dia (cm)	59	65	76

Pile diameter is calculated from sand force volume.

V) Design recommendation :

a.	Improved N-value	N_1	= 20
b.	Length of pile	L	= 6.0 m
c.	Diameter of casing	D_1	= 30.0 cm
d.	Diameter of pile	D_2	= 65.0 cm
e.	Sand force volume	As	= 0.33 $m^3/m/no$
f.	Pile pitch	d	= 1.8 m
g.	Replacement ratio	a_s	= 0.10
h.	N-value at centre of piles	Np	= 26

The design computation can also be accomplished by using charts & formula derived for the purpose.

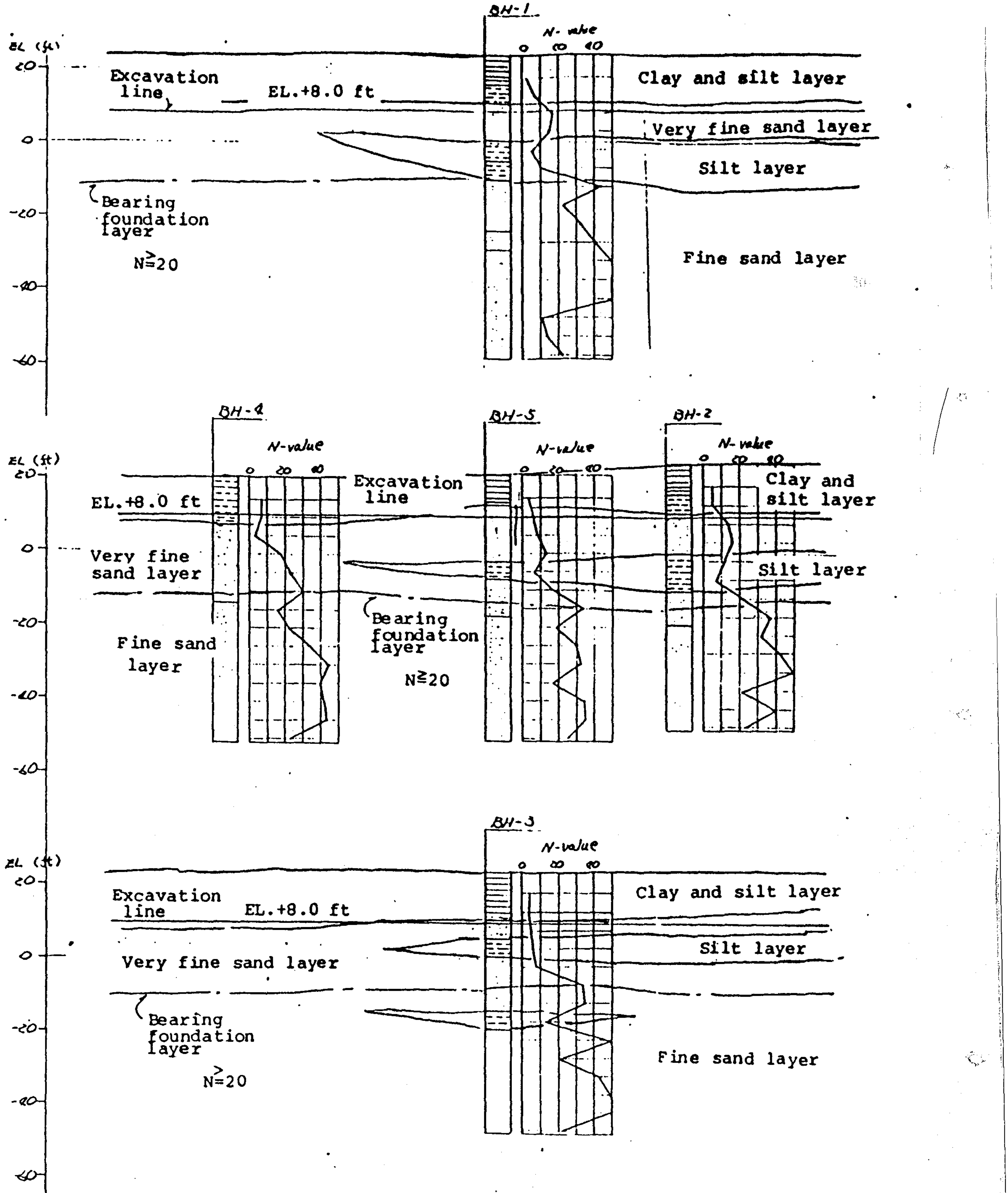


Fig. 21 Soil Profile at Kaitala Pumping Station

Design Chart-I, Relation between No-Ñi-as

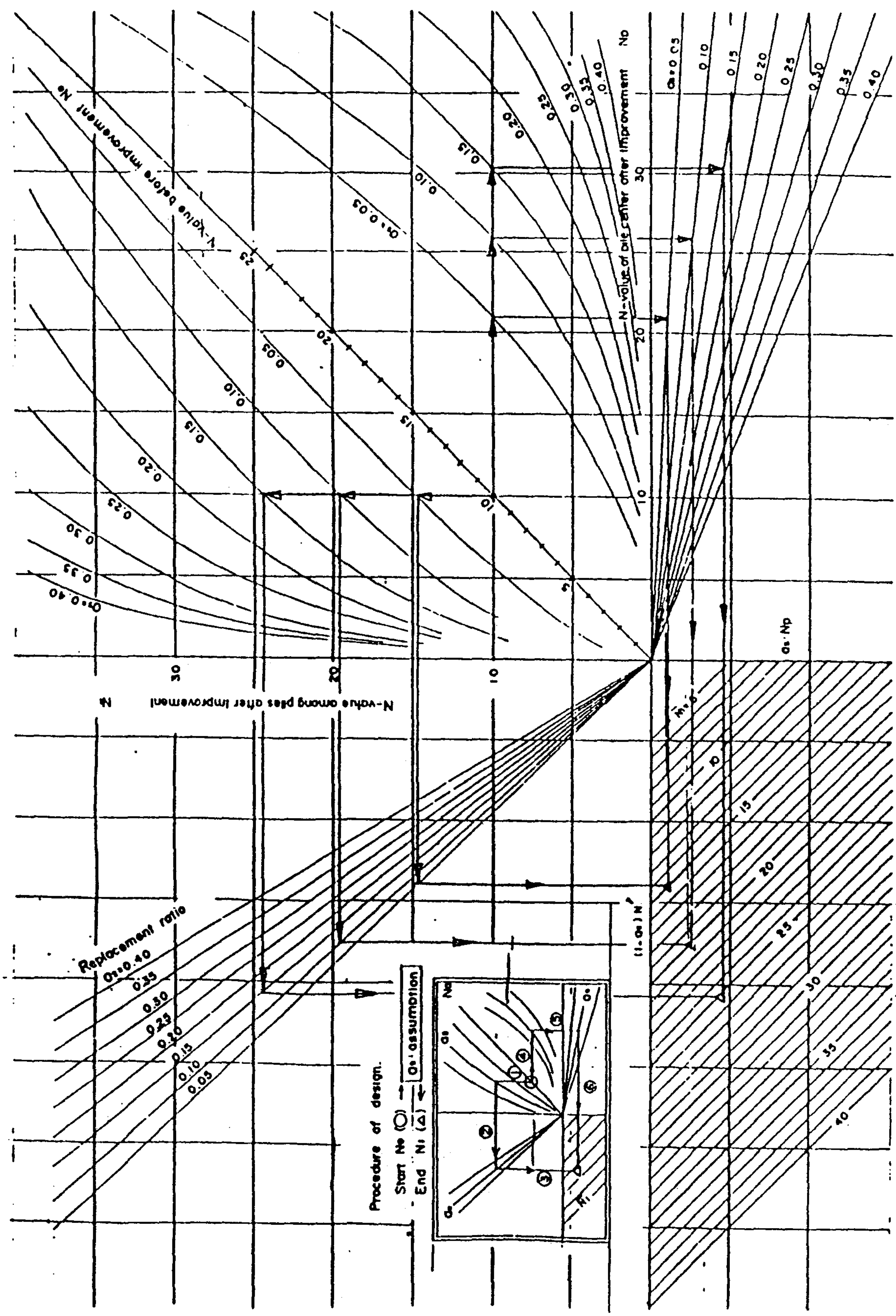


Fig: - 22

Design Chart-II Relation between As-d-as (Squire arrangement)

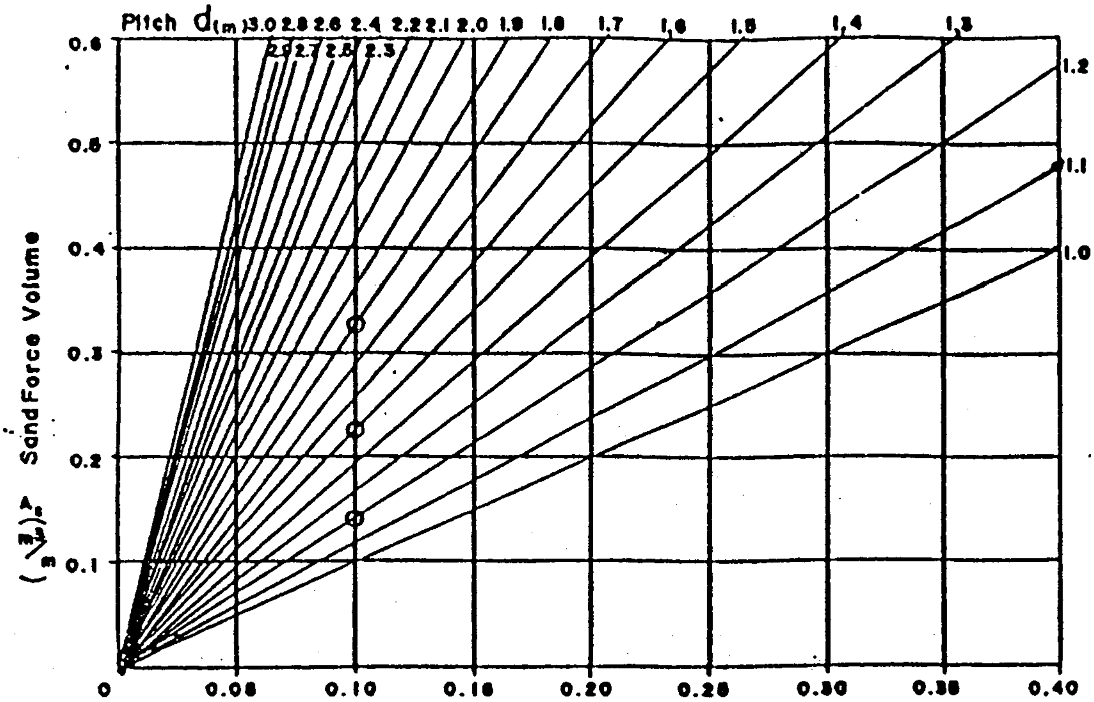


FIG. 23.

E.1. Design steps using charts & formula

1. Data :

   Foundation soil = Sand, uniformly graded
   Original N-value, N_0 = 10

2. Required N-value N_1 = 20

3. Replacement Ratio (a_s)
   Applied formula, $a_s = As/A = (e_0 - e)/(1 + e_0)$

From Fig. 10,

e_0 = 1.0 For N = 10, Overburden Pressure = 10 t/m^2

D_{RO} = 0.4 Uniformity co-efficient, Cu = 10

& e = 0.82

D_{RI} = 0.57

$\therefore a_s = (e_0 - e)/(1 + e_0) = (1.0-0.82)/(1.0+1.0) = 0.09$, say 0.10

4. Pile pitch & sand force volume

Assuming square arrangement of piles,

For, a_s = 0.10, pile pitch = 1.5 m

A_s = 0.23 m^3/m

$\therefore$ diameter of pile = 0.54 m = 54 cm

5. Required N_1 & N_p

From Fig. 13 & Fig. 14,

N_1 = 20, & N_p = 26, for N_0 = 10 and a_s = 0.10

2.3.1.1.3. Heavy Tamping:

Soil compaction by heavy tamping involves repeated dropping of heavy weights on to the ground surface. The method is also termed dynamic compaction, dynamic consolidation, or pounding. The technique as developed in its present form for improvement of large areas to depths upto 30m was pioneered by Menard (Menard, 1974, Menard and Broise,1975). When applied to partly saturated soils, the densification process is essentially the same as that for impact (Proctor) compaction in the laboratory.

In the case of saturated cohesionless soils liquefaction can be induced, and the densification process is similar to that accompanying blasting and vibrocompaction.

The effectiveness of the method in saturated, fine grained soils is uncertain, both successes and failures have been reported. It would appear that in such materials a breakdown in the soil structure, the generation of excess pore water pressures, and the formation of drainage channels by fissuring may be required. Heavy tamping has been especially effective for compaction of waste and rubble fills.

The pounders used for heavy tamping may be concrete blocks, steel plates or thick steel shells filled with concrete or sand and may range from one or two upto 200 tons in weight. Drop heights upto 40m have been used. The pounders are usually square or circular in plan and have dimensions of upto a few meters depending on weight required, material, and the dynamic bearing capacity at the surface of the ground to be treated. More Streamlined shapes have been used for underwater tamping.

For large area compaction several repetitions at points spaced several meters apart in a grid pattern are applied. A typical treatment will result in an average of 2-3 blows/m^2. An illustration of a typical grid pattern and representative equipment is shown in Fig.24. Two or three coverages of an area may be required, separated by time intervals dependent on the rate of dissipation of excess pore water pressure and strength regain. The general response of the ground as a function of time after a coverage is shown in Fig.25.

The time interval required between coverages may range from days for freely draining coarse sands to weaks for finer-grained soils. The ground surface is usually levelled between coverages.

To ensure uniformity and high density in the near surface zone, surface ironing is used. Small impacts by the pounder are made over to entire surface. Surface settlements may be from 2 to 5 % of the thickness of the zone being densified per coverage.

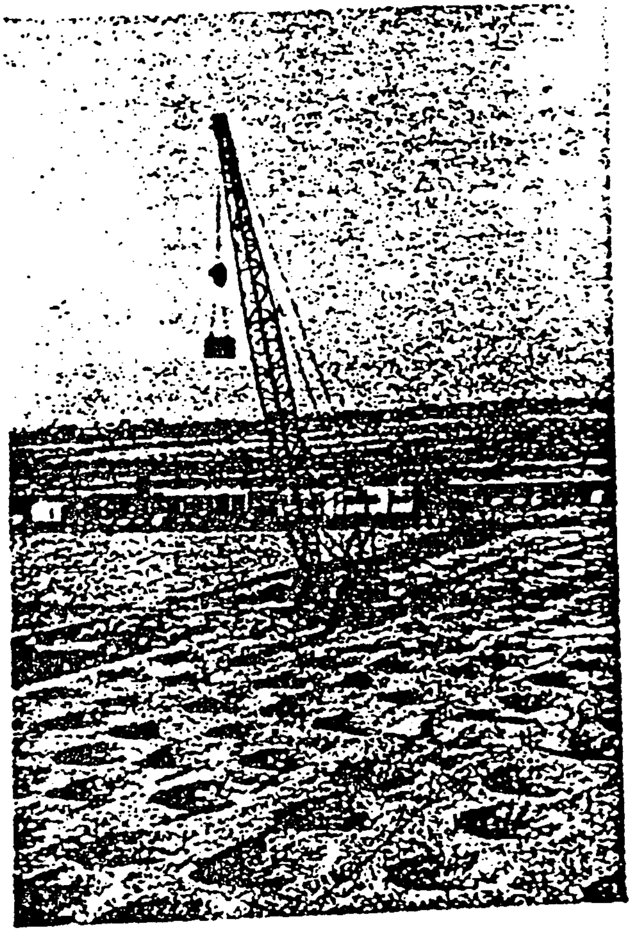
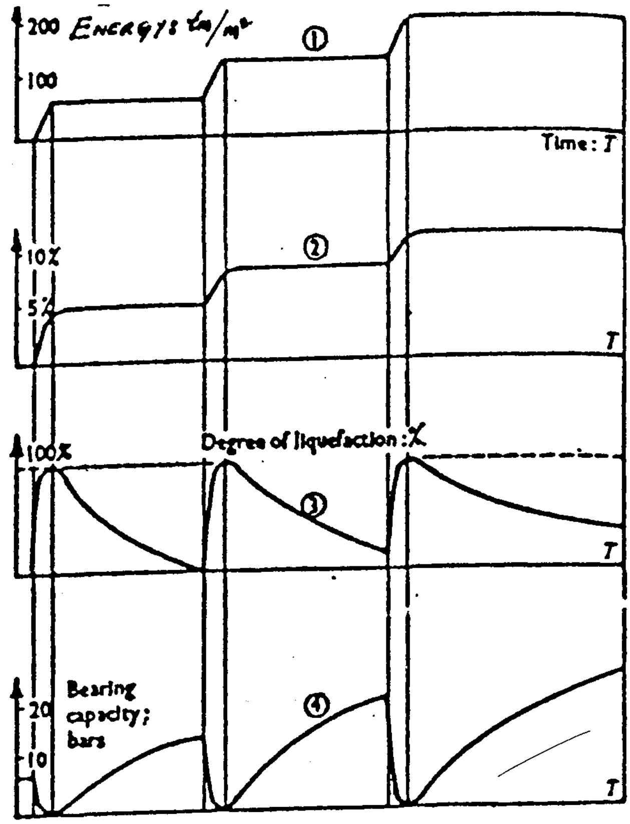


Fig. 24. Illustration of Heavy Tamping



① Applied energy in tm/m²
② Volume variation with time
③ Ratio of pore-pressure to initial effective stress
④ Variation of bearing capacity

Time between passes varies from one to four weeks according to the soil type.

Fig. 25 Ground Response with Time After Successive Coverages of Dynamic Consolidation

When heavy tamping is used to prepare ground for support of relatively light (low rise) structures on shallow foundations, treatment is sometimes made only at footing locations.

Of particular interest when this method is being considered are the depth of influence and the level of property improvement that may be achieved.

Menard and Broise (1975) proposed that

$$D = \sqrt{(WH)}$$

Where, D = depth of influence (m)

W = falling weight, metric tons

H = height of drop (m)

Leonards et al. (1980) analyzed seven cases and concluded

$$D = 1/2\ \sqrt{(WH)}$$

was more appropriate, and Lukas (1980) concluded that

$$D = (0.65 \text{ to } 0.80)\sqrt{(WH)}$$

was best suited for the eight cases studied by him.

The depth of influence should depend on factors in addition to the impact energy. Soil type might be expected to be the most import. A crane drop is less efficient than a free drop. The presence of soft layers has a damping influence on the dynamic forces. within a homogeneous soil layer the amount of ground improvement decreases with the depth as shown, by Fig.26.

The amount of soil improvement that develops in any case depends on soil type, water conditions, and input energy per unit area. Finer-grained soils cannot be strengthened to the same level as can coarser materials. Soft layers of clay and peat inhibit high compaction of adjacent cohesionless material because of their flexibility. A review of available cases suggests that there may be a definable maximum level of improvement. Leonards et al (1980) suggest that this may be a cone penetration resistance of about 150 kg/cm^2.

A study of the data associated with the cases plotted in Fig.27 shows maximum values of cone penetration resistance of 180 kg/cm^2 and standard penetration resistance of 45 blows/0.3 m. Finer grained more compressible soils may have maximum values that are less than half of these. Values decrease from a maximum near the ground surface to the original in-situ value at depth D.

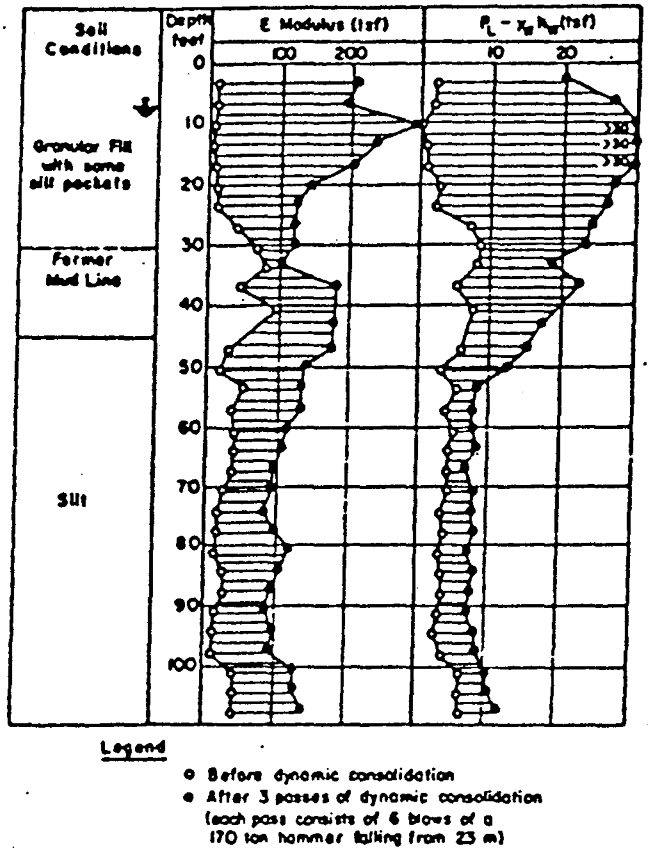


Fig. 26. Variation in Pressuremeter Modulus and Limit Pressure with Depth at Nice Airport

<u>Progressive Liquefaction:</u>

In many cases the volume of soil densified by deep compaction lies within a potentially liquefiable deposit of much larger areal extent. The question arises then concerning whether, if in an earthquake the surrounding soil liquefies, there will be the possibility of loss of stability in the densified zone. Conceivable, the development of high pore pressures in the liquefied zone could generate higher pore pressures in the densified zone with consequent loss strength. To guard against this possibility it should be sufficient to extend the zone of soil improvement laterally outward from the required foundation area a distance equal to the thickness of the layer being densified.

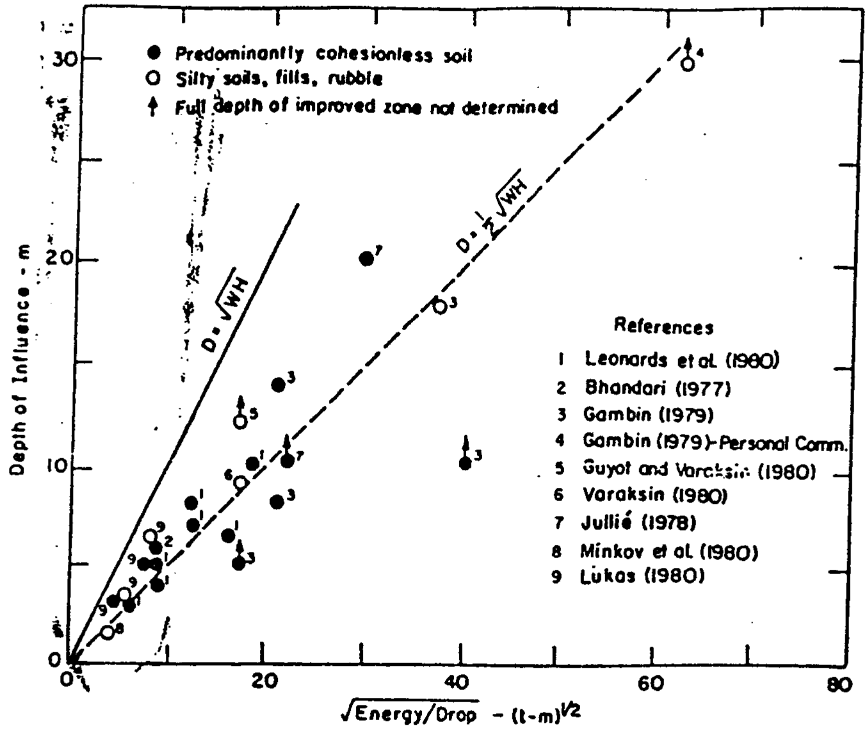


Fig. 27 Depth of Influence as a Function of Impact Energy for Heavy Tamping

2.3.1.2 SOIL IMPROVEMENT BY PRECOMPRESSION:

The strengthening and preconsolidation of weak and compressible soils by preloading prior to construction is one of the oldest and most widely used methods for soil improvement. It is particularly well-suited for use with soils that undergo large volume decreases and strength increases under sustained static loads and when there is sufficient time available for the required compression to occur.

Surcharge loads i.e. loads in excess of those to be applied by a permanent fill or structure can be used to accelerate the process. When the anticipated time for compression is excessive, vertical drains may be used to shorten the time required provided the impression is of the primary consolidation type.

The soil types best suited for improvement by precompression are saturated soft clays, compressible silts, organic clays, and peats. Vertical dains are of greatest effectiveness in inorganic clays and silts that exhibit little secondary compression.

Type of Preloads

Although earth fills are the most commonly used type preload, any system that leads to drainage of poor water and compression of the soil may be suitable. Water in tanks has been used to preload small areas, and water in lined ponds can be used to preload larger areas

Groundwater lowering provides an increase in consolidation pressure equal to the unit weight of water times the drawdown distance. Consolidation by electro-osmosis is the same in many respects as consolidation under externally applied stresses, except that the driving force for drainage is induced internally by an electrical field.

Preloading by vacuum, water table lowering, and electro-osmosis offer the advantages that there are no stability problems and large volumes of surcharge fill are not required. On the other hand, they are more complex in execution than the other methods.

Theoretical basis for design:

The usual objective of design of surcharge loads and duration of their application is to reduce the magnitude of settlement after construction.

Settlement at any time may be expressed as $S_t = S_i + \bar{U}S_{cons} + S_s$

In which, S_t = Settlement at time t,

S_i = Immediate settlement,

$\bar{U}$ = Average degree of consolidation,

S_{cons} = Final consolidation settlement'

S_s = Secondary settlement.

The objective may be either (1) to determine the magnitude of surcharge pressure (P) required to ensure that the total settlement anticipated under the final pressure will be complete in a given length of time, or (2) to determine the length of time required to achieve a given amount of settlement under a given surcharge load.

<u>Vertical Drains:</u>

In many cases the time required for surcharging is excessive, the surcharge required for the time is too great, or the rate of strength gain is too slow to permit rapid fill placement.

Vertical drains may be used to accelerate the rate of settlement, for soils whose compression is dominated by primary consolidation, because consolidation time vary as the square of the drainage path length.

A schematic diagram of a typical vertical drain installation is shown in Fig.28. Vertical drains are ineffective in peats, organic clays and other soils whose settlement behaviour is dominated by secondary compression.

Until a few years ago vertical drains of sand, typically 200 to 500 mm in diameter and spaced from 1.5 to 6.0 m on centers, were widely used. Installation was accomplished using a variety of techniques of both the displacement and non-displacement type. Displacement type drains, while generally less expensive and faster to install, can disturb the surrounding soils.

The conventional sand/gravel drains installed for the acceleration of consolidation are now things of the past in advanced countries. A variety of prefabricated drains has come into wide use. These drains can be rapidly installed to depths upto 50m by machines with special mandrels.

There are several types of prefabricated drains under names as Alidrain, Geodrain, Wickdrain, Colband, Medradrain, PVC drain etc.

The drain usually consist of a core of fibrous material. The cross section design provides for a system of vertical channels for water flow.

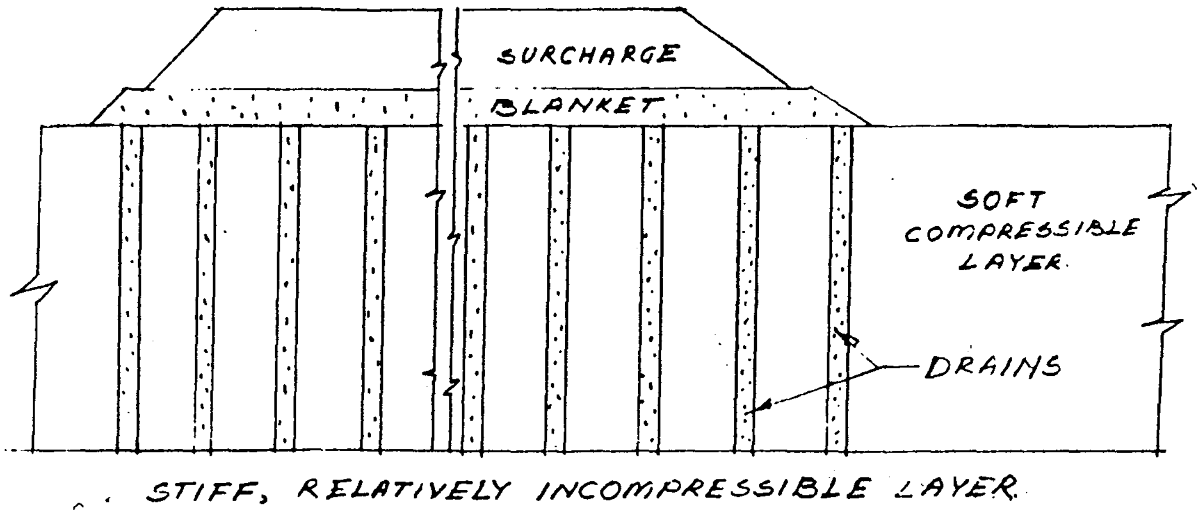


FIG. 28. TYPICAL PRECOMPRESSION TECHNIQUE WITH DRAINS.

2.3.1.3 Precompression by electro-osmosis:

When a direct electric current is passed through saturated soil, water moves toward the cathode. Because water can be made to flow through finer grained soils from anode to cathode in a direct current electrical field, i.e by electro-osmosis, consolidation will result if water is removed at the cathode but not replaced at the anode.

The decrease in volume of soil (consolidation) is equal to the volume of water removed.

2.3.1.4 Injection and Grouting:

In 1802 French Engineer Charles Beriguy repaired a scouring sluice at Dieppe by injecting grout of clay and hydraulic lime beneath it. Since then injection of materials into the ground has developed into a widely used method for soil stabilization and ground improvement.

Because of its high cost, grouting is usually limited to zones of relatively small volume and to special problems that cannot be solved by other methods. Most of the early applications of grouting were for ground water control or shut off, and these continue to be very important applications today. More recently injections have been used for ground strengthening and ground movement control.

2.3.1.5 Soil Reinforcement:

Sand and gravel compaction piles, discussed in this report is one form of soil reinforcement. Pile is also another form of soil reinforcement. Other types of soil reinforcements used are:

Stone Columns are compacted columns of gravel or crushed rock installed into soft soils. Diameters are usually in the range of 0.6 to 1.0 m. They provide vertical support for overlying structures and function as drain for the soft soil. They can be used also to resist shear in horizontal and inclined directions. Sand columns installed by the composer system are used for the same purpose.

Root Piles or Micro piles are small diameter piles (75 to 250 mm) of concrete cast in place, usually with a reinforcing bar at center. They are installed in groups with individual piles both vertical and inclined. They can be used both for support of structures and for stabilization of the soil against movement and loss of stability.

Soil nailing consists of a series of reinforcing bars grouted into the ground to be supported.

Reinforced earth is a constructed composite material consisting of alternating layers compacted backfill and tensile reinforcing material. Both metals and geotextiles are used as reinforcements.

A tabular summary of soil improvement methods discussed and consideration of factors governing the method of choice for a given case is shown in Fig.2.

2.4. DEEP FOUNDATIONS:

There are two types of deep foundations viz. pile foundation and caisson or well foundation.

2.4.1 Pile Foundation:

Piles are vertical or slightly slanting structural foundation members having relatively small cross-sectional dimensions with respect to their length.

A pile foundation even a single pile is statically indeterminate to a high degree. Empirical knowledge and results of load tests at actual sites are adopted for the solution to a pile foundation problem.

The capacity of a pile is based on

1. Structural capacity of the pile to support the load'

2. The support provided by the surrounding and underlying soil and/or rock.

The smaller of the two values from the above considerations is taken as the capacity of the pile.

2.4.2 Caisson Foundation:

The term caisson is derived from the French word, caisse, meaning a chest or box. Caisson has come to mean a box like structure, round or rectangular, which is sunk from the surface of either land or water to some desired depth.

The function of caisson foundation is to 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. Caisson foundations are also used in rivers and maritime constructions to enable foundations to be taken below zones of soil affected by scour. It fulfil a similar function of piled foundation, the main difference being in the method of construction.

Wherever considerations of scour or bearing capacity require foundations being taken to a depth of more than 5 to 7 meters, open excavations become costly and uneconomic. Because of greater earth work involved, the progress of the work in open excavation will be very slow. Another disadvantage in adopting the ordinary type of footing is that the excavated material refilled around the structure is loose and hence easily scourable as compared to natural ground.

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