BUILDING DIGEST

CENTRAL BUILDING RESEARCH INSTITUTE, INDIA

FOUNDATIONS FOR ELECTRIFICATION MASTS

Introduction

Foundations for masts and towers are subjected to high overturning moments which may be due to wind, wire tension etc. In case of isolated block foundations, which are commonly provided for masts, the overturning moment due to lateral loads is resisted by the passive earth pressure mobilised in the surrounding soil; the eccentric soil reaction below the base of the block and the frictional forces acting along the two sides of the block parallel to the plane of rotation (Fig. 1).

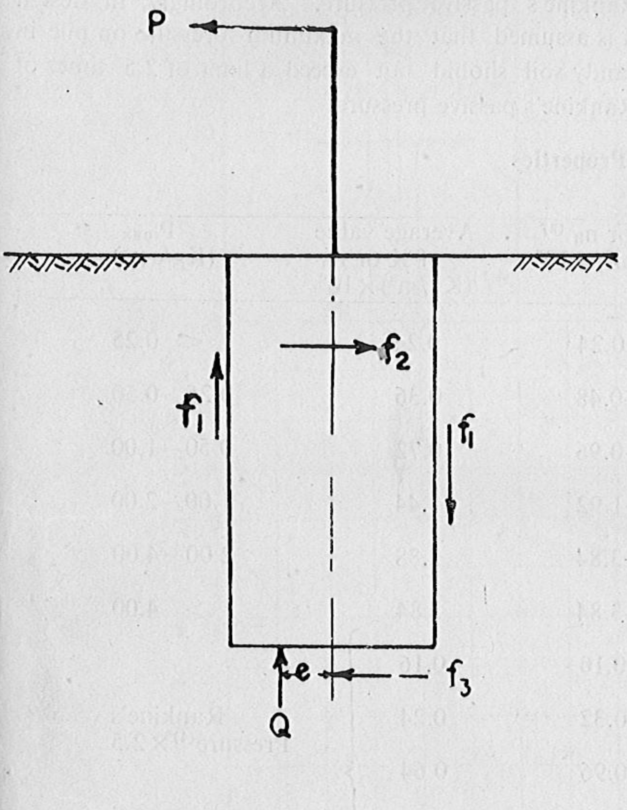

FIG. 1 FORCES ACTING ON THE BLOCK FOUNDATION

On the Indian Railways, the following four types of block foundations are provided :-

1. Gravity type
2. Semi-gravity type
3. Side-bearing type
4. Side-bearing with undercut at base

The gravity type of foundation is provided in loose embankments where the passive resistance offered by surrounding soil is completely neglected. The semi-gravity foundation is suitable for medium dense soil. The side-bearing block foundation is provided in soils with high bearing capacity. That with undercut is exclusively used for expansive soil.

The first two types of foundations are generally very uneconomical because the overturning moment in such foundations is resisted partly or wholly by the weight of the foundation. The side-bearing foundation, though cheaper, is suitable only in good soils. The side-bearing foundation with undercut is also very expensive as it is taken down to the depth where there is no significant volume change of the expansive soil. In our country, this depth is about 3.0 metre. There is another serious disadvantage of the block type of foundation. It is constructed in open pit and special measures to protect the track are required. The trains are invariably slowed down during the construction.

The shortcomings of block type of foundation can be effectively overcome by pile foundations. A single short bored pile can economically replace huge block foundation. Moreover, it can be constructed without interfering with the track in significantly less time and hence the speed of trains need not be reduced during the construction of such foundations.

Design of Pile Foundation

The design criteria for a pile foundation are: (i) deflection at ground level should not be more than the permissible limit and (ii) maximum pressure developed along the length of the pile should be well within the ultimate lateral bearing capacity of soil. Based upon the rigid pole theory, the deflection ($\triangle$) at the ground level and maximum pressure (P_{max}) in soils are given by the following expressions :

(1) Cohesionless Soil

$$\triangle = \frac{6\,(4M_o + 3H_o\,L)}{n_h\,L^3} \quad \ldots(1)$$

$$P_{max} = \frac{3\,(4M_o + 3\,H_oL)^2}{4\,L^2d\,(3\,M_o + 2H_oL)} \quad \ldots(2)$$

(2) Cohesive Soil

$$\triangle = \frac{2\,(3M_o + 2\,H_oL)}{d\,KL^2} \quad \ldots(3)$$

$$P_{max} = \frac{2\,(3\,M_o + 2\,H_oL)}{L^2\,d.} \quad \ldots(4)$$

where,

M_o = Moment at ground level
H_o = Horizontal load at ground level
L = Length of pile
d = Diameter of pile
n_h = Constant of modulus of subgrade reaction in cohesionless soil in the relation $k_x = n_h\,x/d$
k_x = Modulus of subgrade reaction at a depth x
and K = Constant of modulus of subgrade reaction in cohesive soils.

The length of pile can therefore be determined if allowable deflection, maximum pressure and modulus of subgrade reaction of soil are known. It has been found that in cohesionless soil, deflection (Eq. 1) and in cohesive soil, the maximum pressure (Eq. 4) govern the design.

Based on field tests carried out in clayey and sandy soils, the following design criteria have been laid down :

(1) Deflection at ground level (at ultimate load) should not exceed 12 mm. In case of sandy soil, tests showed that the computed deflections were more than 3 times the observed ones. Hence in this case, the predicted load corresponding to 12 mm deflection can be taken as safe load. In case of clayey soils, the tests showed that the observed and the predicted deflections were almost the same at ultimate load. Hence, a factor of safety of 1 5 is used to get the allowable loads.

(2) The maximum pressure on pile [Eqs. (2) and (3)] should not exceed the maximum passive pressure at ultimate load. A factor of safety of 1.5 is used. The maximum passive pressure can be determined from the Rankine equation in cohesive soil. However, it was observed in field tests that in case of sandy soil, the maximum pressure was more than 2.5 times the Rankine's passive pressure. Accordingly, in design it is assumed that the maximum pressure on pile in sandy soil should not exceed a limit of 2.5 times of Rankine's passive pressure.

TABLE 1 **Soil Properties**

Soil	Description	N(1)	Nc(2)	K(3) or n_h (3) $(Kg/m^3)\times10^6$	Average value of K or n_h $(Kg/m^3)\times10^6$	P_{max} (Kg/cm^2)
CLAY	Very soft	<2	<3	< 0.24	0.24	< 0.25
,,	Soft	2-4	3-6	0.24—0.48	0.36	0.25—0.50
,,	Medium	4-8	6-12	0.48—0.96	0.72	0.50—1.00
,,	Stiff	8-15	12-22	0.96—1.92	1.44	1.00—2.00
,,	Very stiff	15-30	22-45	1.92—3.84	2.88	2.00—4.00
,,	Hard	> 30	> 45	>3.84	3.84	> 4.00
SAND	Very loose	< 5	< 8	<0.16	0.16	Rankine's Pressure(4) × 2.5
,,	loose	5-10	8-15	0.16—0.32	0.24	
,,	Medium	10-30	15-45	0.32—0.96	0.64	
,,	Dense	30-50	45-75	0.96—1.92	1.44	
,,	Very Dense	>50	>75	>1.92	1.92	

1. N = No. of blows of Standard Penetration Test.
2. Nc = No. of blows of dynamic cone test with 6.25 cm dia. cone.
3. K = constant of modulus of horizontal subgrade reaction in case of clays
   n_h = coonstant of modulus of horizontal subgrade reaction in case of sands
4. Rankine's pressure $P = \gamma H \tan^2 (45° + \phi/2)$

Based on these two criteria, design curves are plotted (Fig. 2 and 3). The lengths corresponding to different moments are also given in Tables 2 and 3. Knowing the type of soil and the moment, the design length can be determined.

Determination of Soil Properties

The soil properties like modulus of subgrade reaction, allowable passive pressure etc. can be determined either by plate load tests or penetration tests. There are two types of dynamic penetration tests, namely, Standard Penetration Test and Dynamic Cone Test. The former is well known. In the Dynamic Cone Test, a cone of 6.25 cm dia., fitted at the end of drill rods, is driven into the ground (Fig. 4) continuously with the help of a 64 Kg hammer, falling freely from a height of 75 cm. The number of blows are recorded for every 30 cm penetration. For places where sufficient space is not available, such as between two railway tracks, the tripod shown in Fig. 4 can be avoided and the hammer shown in Fig. 4 can be replaced with the modified hammer (Fig. 5) which is also of the same weight. Since the behaviour of piles under lateral load is mainly governed by top soil, the average properties of soil upto one metre depth should be considered in the design. Thus, knowing the average value of standard penetration resistance (N), or dynamic cone resistance (N_c) or the bearing capacity from plate load test at one metre depth, the coefficient of horizontal subgrade reaction (n_h or k) and P_{max} can be determined from Table 1. From these values of n_h (or k) and P_{max}, the length of pile

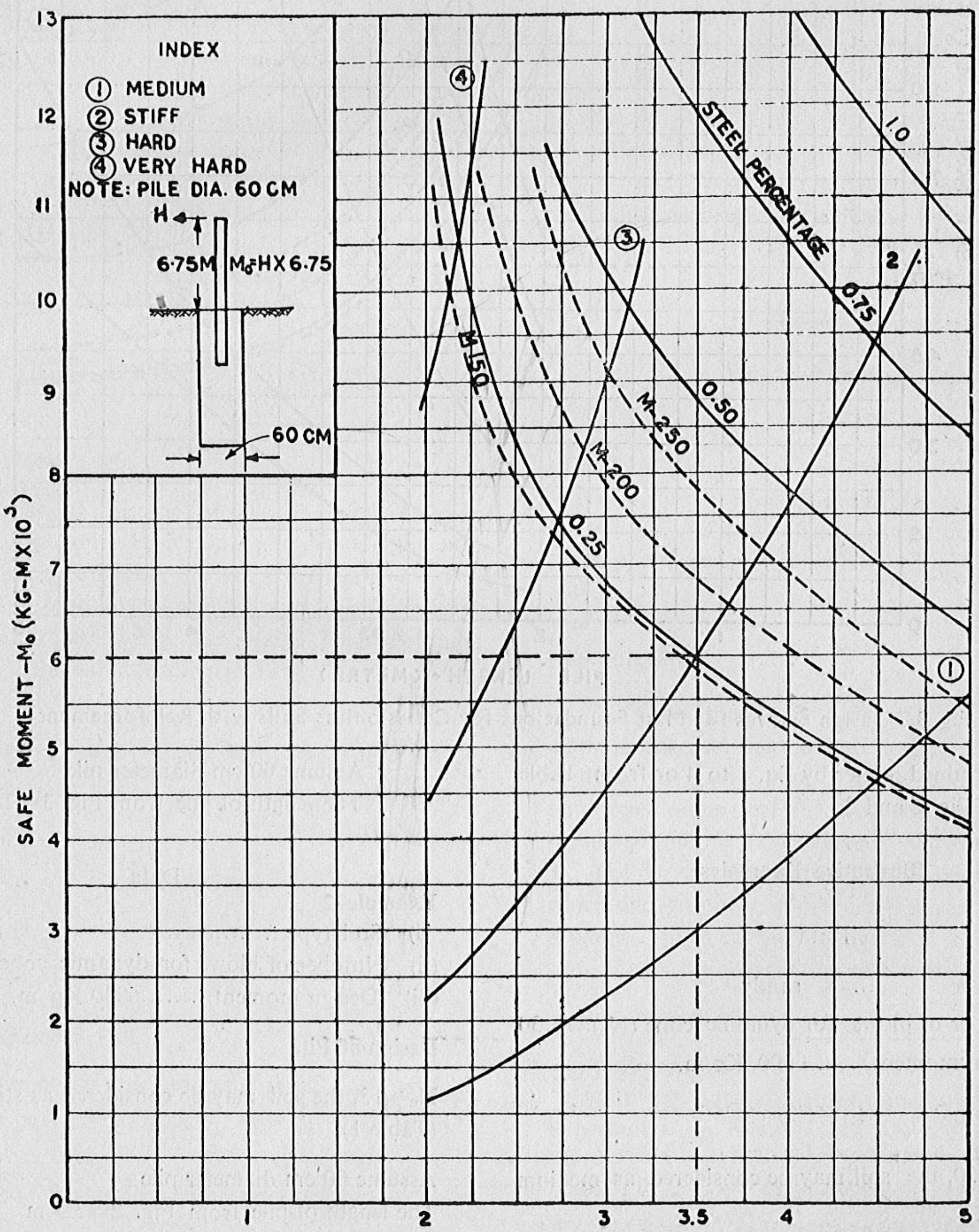

Fig. 2 Design Curves for Clayey Soils with Reinforcement.

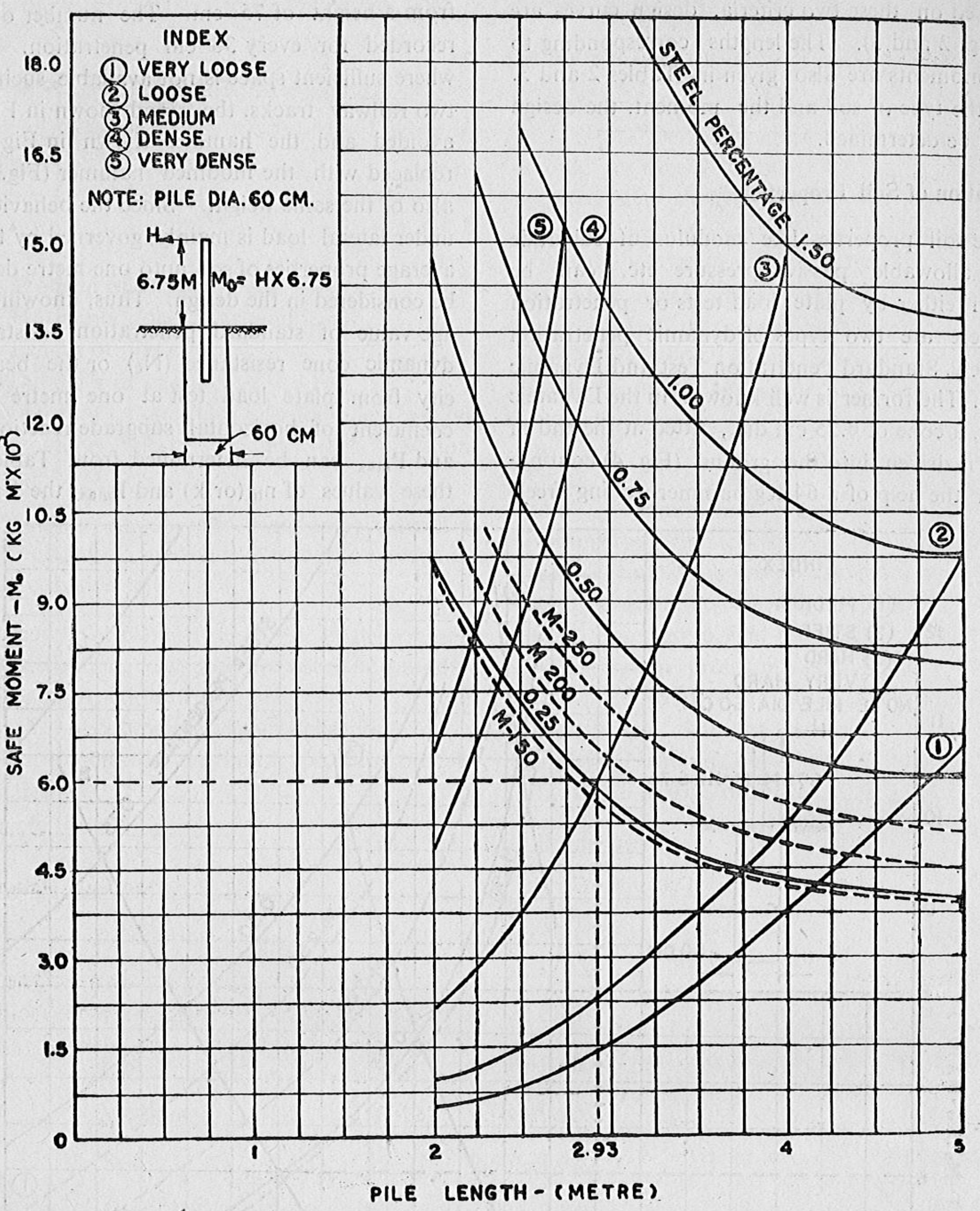

Fig. 3 Design Curves for Mast Foundations for Cohesionless Soils with Reinforcement

can be determined either by Eq. 1 to 4 or from Table 2 and 3 or Fig. 2 and 3.

Illustrative Examples

Example–1

Data

(i) Soil type ...... sandy

(ii) Number of blows for dynamic cone (N_c)......30

(iii) Design moment ...... 6000 Kg m.

Design of Pile

For N_c = 30, the soil may be considered as medium (Table 1).

Assume 60 cm diameter pile

The length of pile from Fig. 3=2.9 m

Data

Example–2

(i) Soil type.........clayey

(ii) Number of blows for dynamic cone (N_c) 13

(iii) Design moment.........6000 Kg m.

Design of Pile

N_c=13, the soil may be considered as stiff clay (Table 1).

Assume 60 cm diameter pile

The length of pile from Fig. 2=3.5 m

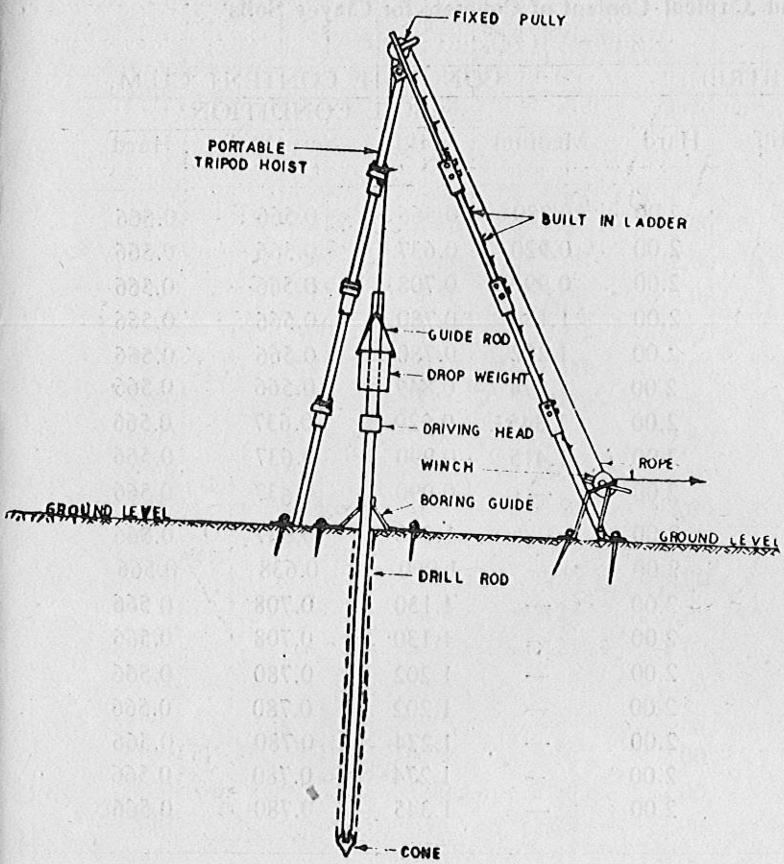

Fig. 4 Test Set-up for Dynamic Cone Test

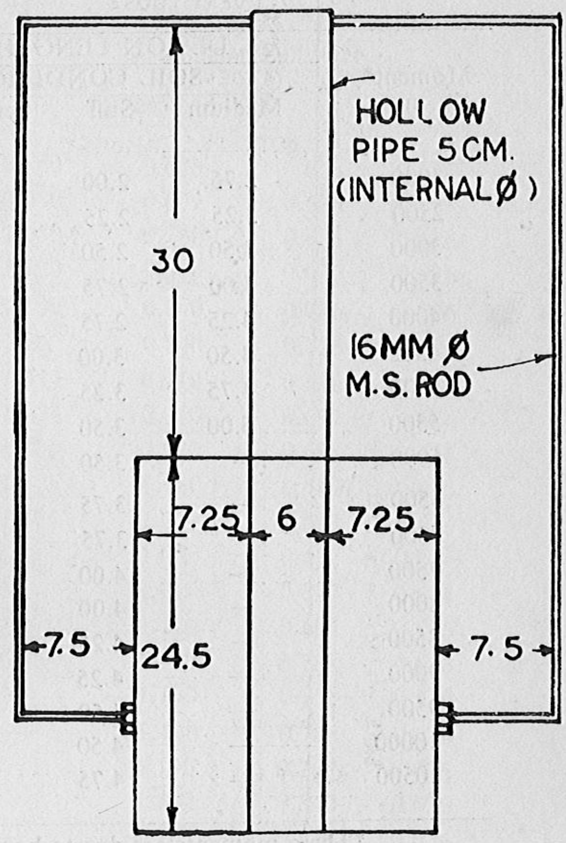

Fig. 5 Modified Hammer

Effect of Water-Table

The lateral resistance of piles is mainly governed by the top soil. Hence, if the water-table is not within one metre depth, the lateral resistance of soil may not substantially change. However, to take into account any uncertainty, the design length of pile should be increased according to Table 5.

Table 5

Increase in Length of Pile for Water-Table

Depth of water below G.L. (metre)	Increase in length (Percent)
0—1	50
1—4	25
4— below	Nil

Piles in Expansive Soil

An under-reamed pile should be used instead of straight bore pile in case of expansive soil or soil with loose filling upto one metre. The length of such piles for different design moments should be the same as given in Table 2. The construction technique of these pile foundations is described in Building Digest 56*.

Reinforcement in Pile

When the pile is sufficiently long, the tensile stress in concrete may be more than the modulus of rupture. In such a case, the pile should be properly reinforced. The steel in percentage of area of cross section of pile to be provided is shown in Fig. 2 and 3. The curves with dotted line show the limits upto which pile with no reinforcement can be made. In the examples given earlier, the required steel is 0.25 percent of area of cross section of the pile.

Cost Economics

Comparison for different types of soils has shown that the overall cost of pile foundation is 40 to 60 percent of the cost of block foundation.

*Avilable from Director, CBRI, Roorkee at Rs. 2 00 per copy.

TABLE 2 Showing Lengths and Cubical Content of Concrete for Clayey Soils

Moment* Kg. m.	DESIGN LENGTH (METRE) SOIL CONDITION				CONCRETE CONTENT CU.M. SOIL CONDITION			
	Medium	Stiff	Very Stiff	Hard	Medium	Stiff	Very Stiff	Hard
2000	2.75	2.00	2.00	2.00	0.780	0.566	0.566	0.566
2500	3.25	2.25	2.00	2.00	0.920	0.637	0.566	0.566
3000	3.50	2.50	2.00	2.00	0.990	0.708	0.566	0.566
3500	4.00	2.75	2.00	2.00	1.130	0.780	0.566	0.566
4000	4.25	2.75	2.00	2.00	1.202	0.780	0.566	0.566
4500	4.50	3.00	2.00	2.00	1.274	0.849	0.566	0.566
5000	4.75	3.25	2.25	2.00	1.345	0.920	0.637	0.566
5500	5.00	3.50	2.25	2.00	1.415	0.990	0.637	0.566
6000	—	3.50	2.25	2.00	—	0.990	0.637	0.566
6500	—	3.75	2.25	2.00	—	1.060	0.637	0.566
7000	—	3.75	2.25	2.00	—	1.060	0.638	0.566
7500	—	4.00	2.50	2.00	—	1.130	0.708	0.566
8000	—	4.00	2.50	2.00	—	1.130	0.708	0.566
8500	—	4.25	2.75	2.00	—	1.202	0.780	0.566
9000	—	4.25	2.75	2.00	—	1.202	0.780	0.566
9500	—	4.50	2.75	2.00	—	1.274	0.780	0.566
10000	—	4.50	2.75	2.00	—	1.274	0.780	0.566
10500	—	4.75	2.75	2.00	—	1.345	0.780	0.566

*These moments are due to horizontal force applied at a height of 6.75 m above G.L.

TABLE 3 Showing Length of Pile and Cubical Content of Concrete for Sandy Soils

Moment Kg.m.	DESIGN LENGTH (METRE) SOIL CONDITION					CONCRETE CONTENT CU.M. SOIL CONDITION				
	V. Loose	Loose	Medium	Dense	V. Dense	V. Loose	Loose	Medium	Dense	V. Dense
2000	3.25	2.75	2.00	2.00	2.00	0.920	0.780	0.566	0.566	0.566
2500	3.50	3.00	2.25	2.00	2.00	0.990	0.849	0.637	0.566	0.566
3000	3.75	3.25	2.25	2.00	2.00	1.060	0.920	0.637	0.566	0.566
3500	4.00	3.50	2.50	2.00	2.00	1.130	0.990	0.708	0.566	0.566
4000	4.25	3.75	2.50	2.00	2.00	1.202	1.060	0.708	0.566	0.566
4500	4.25	3.75	2.75	2.00	2.00	1.202	1.060	0.780	0.566	0.566
5000	4.50	4.00	2.75	2.00	2.00	1.274	1.130	0.780	0.566	0.566
5500	4.75	4.00	3.00	2.25	2.00	1.345	1.130	0.849	0.637	0.566
6000	5.00	4.25	3.00	2.25	2.00	1.415	1.202	0.849	0.637	0.566
6500	—	4.50	3.00	2.25	2.25	—	1.274	0.849	0.637	0.637
7000	—	4.50	3.00	2.25	2.25	—	1.274	0.849	0.637	0.637
7500	—	4.50	3.25	2.50	2.25	—	1.274	0.920	0.708	0.637
8500	—	4.75	3.25	2.50	2.25	—	1.345	0.920	0.708	0.637
9000	—	5.00	3.50	2.50	2.50	—	1.415	0.990	0.708	0.708
9500	—	5.00	3.50	2.75	2.50	—	1.415	0.990	0.780	0.708
10000	—	—	3.50	2.75	2.50	—	—	0.990	0.780	0.708
10500	—	—	3.75	2.75	2.50	—	—	1.060	0.780	0.708

TABLE 4 **Showing Length of Under-Reamed Piles and Cubical Content of Concrete for Expansive Clayey Soils**

Moment Kg.m.	DESIGN LENGTH (METRE) SOIL CONDITION				CONCRETE CONTENT CU. M. SOIL CONDITION			
	Medium	Stiff	V. Stiff	Hard	Medium	Stiff	V. Stiff	Hard
2000	2.75	2.00	2.00	2.00	0.840	0.626	0.626	0.626
2500	3.25	2.25	2.00	2.00	0.980	0.697	0.626	0.626
3000	3.50	2.50	2.00	2.00	1.050	0.768	0.626	0.626
3400	3.75	2.50	2.00	2.00	1.120	0.768	0.626	0.626
3500	4.00	2.75	2.00	2.00	1.190	0.840	0.626	0.626
4000	4.25	2.75	2.00	2.00	1.262	0.840	0.626	0.626
4500	4.50	3.00	2.00	2.00	1.334	0.910	0.626	0.626
4700	4.50	3.00	2.25	2.00	1.334	0.910	0.697	0.626
5000	4.75	3.25	2.25	2.00	1.400	0.980	0.697	0.626
5400	5.00	3.25	2.25	2.00	1.475	0.980	0.697	0.626
5500	5.00	3.50	2.25	2.00	1.475	1.050	0.697	0.626
6000	—	3.50	2.25	2.00	—	1.050	0.697	0.626
6100	—	3.50	2.25	2.00	—	1.050	0.697	0.626
6300	—	3.75	2.25	2.00	—	1.120	0.697	0.626
6800	—	3.75	2.50	2.00	—	1.120	0.768	0.626
7300	—	4.00	2.50	2.00	—	1.190	0.768	0.626
7400	—	4.00	2.50	2.00	—	1.190	0.768	0.626
7800	—	4.00	2.50	2.00	—	1.190	0.768	0.626
8500	—	4.25	2.75	2.00	—	1.262	0.840	0.626
8900	—	4.25	2.75	2.00	—	1.262	0.840	0.626
9300	—	4.25	2.75	2.00	—	1.262	0.840	0.626
9500	—	4.50	2.75	2.00	—	1.334	0.840	0.626
10100	—	4.50	2.75	2.00	—	1.334	0.840	0.626
10500	—	4.75	2.75	2.00	—	1.405	0.840	0.626

There is a demand for short notes summarising available information on selected building topics for the use of Engineers and Architects in India. To meet the need, this Institute is bringing out a series of Building Digests from time to time and the present one is the 95th in the series. Readers are requested to send to the Institute their experience of adopting the suggestion given in this Digest.

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Prepared by : *Devendra Sharma, S.P. Srivastava, K G. Garg and M.R. Soneja,*

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