GEOTECHNICAL INVESTIGATION FOR A CEMENT FACTORY IN BOULDER DEPOSIT

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ABSTRACT

The paper discusses the geotechnical investigations planned and carried out for a cement factory in boulder deposit. A cement factory normally has very typical structures, e.g. heavily loaded structures as cement silo, tall structures as preheater tower. In addition to these, machine foundations are also required as for crusher and rotary kiln. Keeping in view the nature of the loads, type of structures and the soil strata special geotechnical testing program was planned. The various in-situ tests included (i) field density tests (ii) shear strength tests, (iii) cyclic vertical plate load tests, (iv) horizontal plate load tests, (v) passive earth pressure tests and (vi) block resonance tests. Also, the samples brought from the field were tested in the laboratory for classification purposes. Describing briefly the test procedure, the data obtained from various tests have been interpreted to estimate various parameters used for design of foundations. Based on test data suitable values of angle of internal friction, allowable soil pressure, horizontal subgrade modulus and dynamic soil constants have been recommended for foundation design.

Key words: boulder, foundation, field test, in-situ test, passive earth pressure, special shear test

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INTRODUCTION

A dry process cement plant with 3×10^5 kg per day capacity is being set up in Bhutan close to the India-Bhutan border. The soils at site were boulder deposit with the average size of boulder varying from 150 to 300 mm. The cement factory normally has heavily loaded structures e.g. cement silo, tall structures e.g. preheater tower and also machine foundations. Thus the nature of loads imposed by these structures are varying. Also, the soil being a boulder deposit a special geotechnical investigation programme is necessary.

The paper discusses the various tests planned and executed for the detailed geotechnical investigations of the site. The results are presented. Based on these results recommendations on various soil properties used for the analysis and design of foundations are made.

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SITE-TOPOGRAPHY, GEOLOGY AND SOILS

The proposed site lies on a gently sloping terrace on the left bank of river Sukti, north-east of Makrapara Tea Estate at the India-Bhutan border. The ground elevation in the factory area varies from 284 m to 276 m above datum with a general slope towards WSW (Fig.1). The topography of the area around the proposed site steeply rises towards east and north forming hills. Steep escarpments and ravines varying from 8 m to 10 m in depth are noted along the left bank of river Sukti at the edge of the terrace east of the factory site. Contoured terraces are utilized for cultivation at higher elevations near the hills, whereas wide level terraces have been developed in the factory site area.

The terrace deposit overly the dark grey and carbonaceous phyllites and quartzites of Baxa series which are exposed along the hill ranges east north east of the site. The deposits overlying Baxa series, consist of angular to subrounded boulders embedded into a matrix of buff coloured gravelly soil. Bulk of boulders as observed in the test pits were composed of quartzite and vary in size from few millimeters to upto one meter with average size from 150 mm to 300 mm. A small percentage of boulders of schists and phyllites (generally less than 100 mm in size) were also noted.

The boulder bed is overlain by buff coloured soil consisting of gravel and sand admixture with small fragments of clay and silt fractions. The thickness of this soil layer varies from 150 mm to 700 mm in the north western parts of the factory and the thickness considerably increases towards the south eastern borders of the factory and colony site. Angular and subrounded fragments of rock embedded in the soil are noted to occur.

The soils in general are cohesionless.

Flowing streams in the nala-section in the hill ranges towards north and north-east become almost dry as they reach the terrace deposits due to high permeability and lower

Fig. 1. Proposed cement factory and township sites

water table conditions. No observations of depth of water table in the boulder strata were available.

TEST RESULTS AND INTERPRETATION

Tests Performed

The examination of pits at different locations in the township and the factory site revealed that the soils mainly consisted of coarse grained ranging from boulder, gravel to sand. The township normally has two to three storeyed structures whereas the plant has heavily loaded structures e.g. cement silos, structures with vibratory loads e.g. raw mill and kiln. In view of the nature of soils and the different types of loads coming on foundations, the following field tests were performed:

Field density tests-seven different locations

Shear strength tests-two locations

Vertical cyclic plate load tests-six locations

Horizontal plate load tests-six locations

in the pits used for vertical plate load tests

Resonance tests-two locations

Passive pressure tests-two locations

In addition to these field tests, the samples brought from the field were also tested in the laboratory for classification purposes.

In-situ Density and Relative Density

The in-situ density is determined at six different locations in pits 1 to 6 (Fig.1). It is determined by core cutter method on fraction excluding boulder. The average in-situ density was noted to be 18 kN/m^3 . At the location of pit-2, where the size and percentage of boulders was more, a pit $1 \text{ m} \times 1 \text{ m} \times 1 \text{ m}$ deep was excavated. The in-situ density as determined by weighing the excavated soil and noting the volume of the pit was noted to be 20 kN/m^3 . The average relative density of deposit was noted to be 53 percent (Prakash and Ranjan et al., 1976). This indicates that in general the strata is in a medium state of packing.

Shear Strength Tests

Shear parameters of the deposit are essential for estimation of bearing capacity. These parameters are usually determined through carefully conducted tests in the laboratory on samples from field or through in-situ tests. Reliable estimation of shear parameters through tests in the laboratory demands the simulation of in-situ structure and compactness, which poses a major problem particularly in the boulder deposit. Under these situations, in-situ tests are likely to yield satisfactory results. The conventional size samples normally used for shear tests are so small that particles more than a certain size cannot be tested accurately (Prakash and Ranjan, 1975 b). In view of size of particles and nature of deposit, the in-situ direct shear tests on large size sample are used. The shear parameters are evaluated through the following tests:

Large scale direct shear test:

A field shear box of sample size $0.706 \text{ m} \times 0.706 \text{ m}$ (area 0.5 m^2) was planned. The sides of the box were suitably designed (Ranjan, 1981) which were connected through bolts and nuts to form the box.

The shear box tests were carried out at two locations marked SB_1 and SB_2 (Fig.1). Tests at two normal loads was carried out at each location. The excavation required for the box was made leaving two blocks $0.80 \text{ m} \times 0.80 \text{ m}$ in size at suitable position. After

Fig. 2. Test set up for shear box test

the bed plate. A train of rollers was put below the box frame and the bed plate. After the box reached the required position the excess soil was trimmed and the top plate was put. Next the normal load was applied through a platform made of R.S.J. and wooden sleepers and loaded with sand bags. The shear force on the sample was applied through a remote controlled hydraulic jack and proving ring taking reaction from the other box. Fig. 2 shows the test set-up. The readings of shear displacement were taken through suitably mounted dial gauges.

After the required normal load on the box was put, the dial gauges were set, the desired shear force was applied by operating the jack. The shear force resulted in the shear displacement. The shear force was maintained constant till the shear displacement was complete. The readings of dial gauge were taken and the next increment of shear force applied. The process was repeated till one of the samples (carrying smaller intensity of normal load) failed. The test on the second sample was continued by blocking the space between the failed sample box and trench through braces.

The same procedure was adopted for carrying out the test at other location. The tests are carried out at four normal stresses of 100, 200, 300 and 400 kN/m². The test on 400 kN/m² normal stress could not be completed as the platform slipped on to a side, damaging the sample. Shear displacement versus ratio of shear stress to normal stress plots are shown in Fig. 3.

Medium scale direct shear test:

Direct shear tests on 0.30 m × 0.30 m sample sizes were also carried. The procedure followed was similar to the one adopted for the large scale direct shear test.

Fig. 3. Shear displacement vs ratio of shear stress and normal stress (Sample size 0.5 m²)

the excavation was complete, the assembled shear box frame was put on the trimmed blocks, and soil below the frame was gradually removed making the frame slide down. Two rolled steel joists (R.S.J.) were embedded along the undisturbed blocks to form

Four tests at two different locations close to large scale shear box tests were carried out at normal stresses of 50, 100, 125 and 200 kN/m². Shear displacement versus ratio of shear stress to normal stress plot is shown in Fig. 4.

Taking the peak shear stress values from Fig. 3 and 4 the Mohr's envelope is plotted in Fig. 5. A perusal of Fig. 5 indicates that the results from 0.5 m² sample size test are consistently lower than those obtained from 0.09 m² sample size.

Fig. 4. Shear displacement vs ratio of shear stress and normal stress (Sample size 0.09 m²)

Fig. 5. Normal stress vs shear stress plot from direct shear test

Thus indicating a value of 26.6° for the angle of internal friction from 0.5 m² sample size and indicating a value of 35° from 0.09 m² sample size box.

Retaining wall test:

The shear parameter under static loading are further evaluated from retaining wall tests. Also, the retaining wall test under vibratory loads is performed successfully to obtain the data regarding the behaviour and strength of the boulder deposit under vibratory loads.

The tests were carried at the site at two different locations marked and identified as PP₁ and PP₃ on Fig.1. For each test, two reinforced concrete walls 1.5 m high 1.0 m wide and 0.30 m thick suitably designed were cast in a 2.0 m wide pit at the opposite faces (Fig.6). Foundation bolts for fixing of a mechanical oscillator were embedded in one of the walls and a loading frame. Also, on either side of the test wall two side walls of brick in cement mortar were built for a length of 0.5 m. This was done to isolate the test wall from the opposite side effects.

The mechanical oscillator with a D.C. motor was fixed on the wall in such a way that vibratory forces could be imparted in a horizontal direction at a depth of 1.0 m (i.e. at two thirds the height of the wall) from the top was utilised to apply static load on the wall at its two thirds point from the top without transferring the loads on to the oscillator assembly. Static load was applied on the test wall through a hydraulic jack taking reactions from the wall on the opposite face of the pit.

Three of the dial gauges were fixed on the test wall to check any rotation or tilting and the fourth dial gauge fixed on the reaction wall. Photo.1 shows the test set-up.

In the first test conducted at location PP₁, (Fig.1) first a static load of 20 kN was applied. This load was maintained constant till the deflections under load became constant. The readings of the four dial gauges were then recorded. The load was then increased to 40 kN and then to 80 kN. Under each of the applied load the readings of dial gauges were taken after these became constant. At 80 kN load an eccentricity for the mechanical oscillator was selected and after pulling back the travelling rods of the dial gauges without disturbing the fixing positions the motor was started and run at a low frequency for 30 seconds. The frequency of vibration required for calculating

Fig. 6. Set up for retaining wall test

Photo. 1. Set up for dynamic passive pressure test

dynamic load was measured using tacheometer. After applying the load for 30 seconds the motor was stopped, dial gauges were released and readings taken. The dial gauges were arrested again for the next increment of vibratory load. A higher frequency was then chosen and the procedure repeated. The tests were performed upto a frequency of 28.6 Hz.

The eccentricity of the oscillator was then changed and the test repeated. The hydraulic jack was operated to the necessary level during the vibrations also to maintain the static loads.

After the dynamic test was completed the hydraulic jack was operated to increase the static load to the next value of 120 kN and 160 kN and dial gauge readings were taken under each load as discussed above.

At 160 kN static load, dynamic tests was again carried out. After this the wall was tested by increasing the static load till the soil behind the wall failed. The zone of failure and crack pattern was observed.

Fig. 7. Load-displacement plot from retaining wall test at kiln site

Full line curve of Fig.7 shows the static load versus displacement plot for the wall at kiln site. The effect of dynamic load application with a static load is shown in the figure as the points with number besides them. These points correspond to a load equal to the sum of static load and the dynamic load caused by the oscillator and a deflection at the static load plus the additional deflection caused by the

particular dynamic load as applied.

It can be seen from Fig.7 that the points from dynamic loads are lying in a very narrow band. It indicates that the strength of cohesionless soils is only slightly affected by the dynamic loads. These findings are in agreement with those of previous investigations (Prakash et al., 1973).

The failure load is determined by intersection of tangents to full line curve (Fig.7). It gives a load of 220 kN. The corresponding failure load from other location was 240 kN. Thus taking an average value of failure load 230 kN computations are made.

The expression (Coulomb, 1773) for computation passive earth pressure in vertical retaining walls is as follows:

$$P_p = \left(\frac{1}{2} \gamma H^2 \right) \frac{1}{\cos \delta} \left[\frac{1}{\frac{1}{\cos \phi} - \sqrt{\tan^2 \phi + \tan \phi \cdot \tan \delta}} \right] \quad (1)$$

where γ = density of the back fill material

H = height of retaining wall

ϕ = angle of internal friction

δ = angle of wall friction

In Eq. (1), substituting $P_p = 23.0$ tons, $\gamma = 18.6$ kN/m³, $H = 1.5$ m, and $\delta = 2/3 \phi$ and solving the equation by trial and error, ϕ value of 35.5° is obtained.

Eq. (1) holds good for two dimensional case. The value of ϕ of 35.5° obtained using Eq. (1) is thus for plane strain case. Cornforth (1964) reported ϕ -values higher in plane strain case as compared to triaxial compression test. Assuming 10% lower value in triaxial compression test a value of 32° be adopted for the deposit.

Vertical plate load test:

Tests at 3.0 m depth at six different locations were carried out in pits marked pit 1 to pit 6 (Fig.1). The tests were carried out using 0.45 m × 0.45 m size plate. Cyclic plate load tests were carried to obtain the coefficient of elastic uniform compression of the deposit also in addition to the allowable soil pressure.

A test pit 3 m × 3 m in plan with 3 m depth was made at the location of the test. A platform was then made on the pit using R.S.J. and wooden sleepers. The platform was then loaded. The load on the plate applied taking reaction from the loaded platform through a remote controlled hydraulic jack. Four dial gauges set-up on independent datum bars mounted at four corners of the test plate were used for recording settlement under a given load.

The load on the plate was maintained constant till the settlement under load was practically constant. The readings of dial gauges were recorded. The load on the plate was then reduced to zero and after the dial gauge readings have become constant the dial readings are recorded. Next the load on the plate is increased to a higher intensity, the dial readings taken after they are constant and the load on the plate reduced again. These cycles of loading and unloading continued and reading at each stage recorded. From the observations of dial gauges at each loading average settlement was computed.

The load settlement data from each test was plotted. Fig.8 shows a typical plot from vertical plate load test at location VPL-3 (Fig.1).

The bearing capacity of a square footing in a cohesionless deposit (having unit cohesion, $c=0$) is given by Eq. (2).

$$q_u = \gamma D_f N_q + 0.4 \gamma B N_\gamma \quad (2)$$

Fig. 8. Load-settlement curve from vertical cyclic plate load test (VPL-3) at raw mill site

Analysis of the data will give even higher values of angle of internal friction.

where γ = unit weight of soil

D_f = depth of footing

B = breadth of footing

N_r, N_q = bearing capacity factor

For a surface footing, $D_f = 0$ and hence Eq. (2) reduced to Eq. (3).

$$q_u = 0.4 \gamma B N_r \quad (3)$$

Table 1 gives the ultimate bearing capacity of square plates at various locations. A glance of this table indicates that the minimum value of ultimate load for $0.45 \text{ m} \times 0.45 \text{ m}$ size plate is 200 kN/m^2 . Substituting for $\gamma = 18.6 \text{ kN/m}^3$. $B = 0.45 \text{ m}$.

$$N_r = \frac{200 \times 10^3}{0.4 \times 1860 \times 10 \times 0.45} = 59.8$$

For N_r value of 59.8, ϕ value is 37.0 (Terzaghi and Peck, 1967).

As indicated in Table 1, the values of ultimate load are higher at other sites.

Table 1. Ultimate load* from vertical plate load test

S. No.	Location of various structures	Size of plate (m $\times$ m)	Depth of plate (m)	Ultimate load (kN/m ²)	Remarks
1	2	3	4	5	6
1.	Kiln site	0.45 $\times$ 0.45	3.0	200	Lowest value adopted for analysis
2.	Cement Silo site	0.45 $\times$ 0.45	3.0	275	
3.	Raw Mill site	0.45 $\times$ 0.45	3.0	200	
4.	Hostel site	0.45 $\times$ 0.45	3.0	300	
5.	Labour Colony site	0.45 $\times$ 0.45	3.0	555	
6.	Unloading Point	0.45 $\times$ 0.45	3.0	362	

* Ultimate load of an actual footing shall be many times more than the ultimate load of the plate on account of the size and surcharge effects.

Angle of internal friction values:

Table 2 gives the average ϕ value obtained for the site through different tests.

A glance of Table 2 indicates that the ϕ value for the deposit from various tests varies between 26.6° to 38.6° . The ϕ value from 0.09 m^2 size direct shear test is higher

Table 2. Angle of internal friction values

S. No.	Test	Average ϕ value	Remarks
1.	Direct shear test		
	0.5 m ² size	26.6°	
	0.09 m ² size	38.6°	
2.	Retaining wall test	32.0°	
3.	Plate load test	37.0°	

than that obtained by 0.50 m² sample size. This is in order. As discussed earlier since the deposit consists mainly of coarse grained soils with average size 150 to 300 mm the results from large size direct shear test would yield a more realistic value. The ϕ value obtained indirectly from other tests indicates higher ϕ value i.e. 32.0° and 37.0°. Also the relative density of deposit, 53 percent suggests compact to dense deposit. The lower ϕ -value from 0.50 m² sample size direct shear test may be due to some disturbance to the sample in trimming. In view of this an average value of ϕ of 32° may be adopted for the deposit.

Soil Pressure

Allowable soil pressure :

The failure load from each of the load-settlement curve from vertical plate load test is obtained by drawing the two tangents i.e. at the initial portion of the curve and extending the straight line portion of the lower part of the curve and taking point of intersection. The results have been tabulated in Table 1.

The net safe bearing capacity, q_s of a square footing on a cohesionless soil may be computed by Eq. (4)

$$q_s = \frac{1}{F} [\gamma D(N_q - 1) + 0.4 \gamma B N_\gamma] \quad (4)$$

where γ = unit weight of soil

F = factor of safety

D = depth of footing

B = breadth of footing

N_q, N_γ = bearing capacity factors

As discussed earlier, the average value of angle of internal friction for the deposit may be taken as 32°. For $\phi = 32^\circ$ corresponding values of bearing capacity factors are: (Terzaghi and Peck, 1967)

$$N_\gamma = 26$$

$$N_q = 27$$

For a 2.0 m square footing located at a depth of 2.0 m the net safe bearing capacity for a factor of safety of 3.0 shall be

$$q_s = \frac{1860 \times 10}{3} [2.2(27 - 1) + 0.4 \times 2.0 \times 26] = 441.5 \text{ kN/m}^2$$

For a column load of 1000 kN assume a footing of 2.0 m $\times$ 2.0 m located at 2.0 m depth. Though the Indian Standard Code of practice (IS: 1904-1966) specifies a settlement of 40 mm for isolated footing on cohesionless soil, yet assuming a permissible settlement of 20 mm only in view of the importance of structures, the settlement of the plate is given by Eq. (5)

$$S_p = S_f \times \left[\frac{B_p(B_f + 0.30)}{B_f(B_p + 0.30)} \right]^2 \quad (5)$$

where S_p = settlement of plate in mm

S_f = allowable settlement of foundation in mm

B_p = width of the test plate in m

B_f = width of the footing in m

From the data

$$S_p = 20 \left[\frac{0.45(2.00 + 0.30)}{2.00(0.45 + 0.30)} \right]^2 = 9.5 \text{ mm}$$

Considering the load-settlement curve for Kiln site (Fig. 8) for a settlement of 9.5 mm

the load is 250 kN/m².

From the above two considerations, the smallest value is 250 kN/m² for a 2.0 m × 2.0 m square footing located at a depth of 2.0 m. Thus for a 1000 kN column load, a 2.0 m × 2.0 m footing at 2.0 m depth is in order. However for other column loads and different permissible settlement values the footing size and allowable pressure could be worked as indicated above.

Effect of water table:

No account of water table is made in the computations of allowable soil pressure (Section, *Allowable soil pressure*) as the water table is assumed to be at a depth greater than the width of foundation below the base of footing.

However, the influence of the position of water table may be taken into account by multiplying the values of allowable soil pressure by a correction factor C_w given by Eq. (6)

$$C_w = 0.5 + 0.5 \left(\frac{D_w}{D_f + B} \right) \quad (6)$$

where D_w = depth of water table below ground surface (m)

D_f = depth of the base of foundation below ground surface (m)

B = width of foundation (m)

The correction factor (Eq. (6)) is applicable only when $D_w \leq (D_f + B)$. In case where $D_w \geq (D_f + B)$ no effect of water table is required in analysis.

Modulus of horizontal subgrade reaction:

The modulus of horizontal subgrade reaction for the deposit is obtained from horizontal plate load test. The test was carried out at six locations in the pits used for vertical plate load test. 0.6 m × 0.6 m size plates at 2.70 m depth below ground surface were used. The settlement under a load was noted through dial gauges.

Fig. 9 shows the typical load versus displacement plot from lateral plate load test. Similar curves were obtained for other sites.

Modulus of horizontal subgrade reaction in kN/m³ (Terzaghi, 1955) is defined as the

Fig. 9. Load-displacement curve from lateral plate load test (HPL-5) at labour colony site

Fig. 10. Load-elastic settlement plot from vertical cyclic plate load test (VPL-1) at kiln site

Table 3. Modulus of horizontal subgrade reaction

S. No.	Location of various structures	Size of plate (m × m)	Depth of plate (m)	K (kN/m ³)
1	2	3	4	5
1.	Kiln site	0.60 × 0.60	2.7	2.4 × 10 ⁴
2.	Cement silo site	0.60 × 0.60	2.7	1.5 × 10 ⁴
3.	Raw mill site	0.60 × 0.60	2.7	1.75 × 10 ⁴
4.	Hostel site	0.60 × 0.60	2.7	2.0 × 10 ⁴
5.	Labour colony site	0.60 × 0.60	2.7	4.0 × 10 ⁴
6.	Unloading point site	0.60 × 0.60	2.7	2.18 × 10 ⁴

ratio between the intensity of load and the settlement. Symbolically it is represented by Eq. (7).

$$K = \frac{q}{y} \quad (7)$$

where K = modulus of subgrade reaction

q = intensity of load

y = settlement

The modulus of subgrade reaction is dependent upon several factors. The important of them being, type of soil, size and shape of loaded area, and depth (Terzaghi, 1955). As the value of horizontal subgrade modulus depends upon the displacement considered, it is computed for a horizontal displacement of 1.25 mm as for the vertical case (IS; 2950 Part 1, 1973). The values of horizontal subgrade modulus for various sites are tabulated in Table 3.

Table 3 indicates that the value of horizontal subgrade reaction varies from $1.50 \times 10^4 \text{ kN/m}^3$ to $4.0 \times 10^4 \text{ kN/m}^3$. The lowest value of $1.5 \times 10^4 \text{ kN/m}^3$ is recommended for adoption.

Coefficient of Elastic Uniform Compression

The coefficient of elastic uniform compression for the deposit is determined from the cyclic plate load test and also the block resonance test. The results are presented and discussed in the following sections.

Vertical cyclic plate load test:

The cyclic plate load test data at a site is utilised to obtain the elastic settlement of the plate under a given intensity of load. Fig. 10 shows a plot between load per unit area (kN/m^2) the elastic settlement (mm) for Kiln site. From this plot the coefficient of elastic uniform compression is taken as the slope of the line. Similar curves for other sites are also plotted. The values of coefficient of elastic uniform compression are tabulated in column 3 of Table 4. The coefficient of elastic uniform compression, C_u is

Table 4. Coefficient of elastic uniform compression from vertical cyclic plate load test

S. No.	Location of various structures	C_u (for an area of 0.2025 m ²) (kN/m ³)	C_u (for 10 m ² area) (kN/m ³)
1.	Kiln site	30.0×10^4	4.27×10^4
2.	Cement silo site	11.40×10^4	1.72×10^4
3.	Raw mill site	12.75×10^4	1.81×10^4
4.	Hostel site	13.33×10^4	1.90×10^4
5.	Labour colony site	32.00×10^4	4.55×10^4
6.	Unloading point site	13.00×10^4	1.85×10^4

a function of the loaded area (Eq. (8)).

$$\frac{C_{u1}}{C_{u2}} = \sqrt{\frac{A_2}{A_1}} \quad (8)$$

where C_{u1} = coefficient of elastic uniform compression for an area, A_1 .
 C_{u2} = coefficient of elastic uniform compression for an area, A_2 .

Block resonance test:

Elastic soil constants as per Indian Standard (IS: 5249-1969), can be conveniently estimated through resonance tests on concrete block $1.5 \text{ m} \times 0.75 \text{ m} \times 0.7 \text{ m}$ high in size cast at the proposed foundation level. These tests were performed at two different locations marked as RB_1 and RB_2 (Fig. 1) in pits of size $3.0 \text{ m} \times 3.0 \text{ m} \times 3.0 \text{ m}$ deep.

The block resonance tests were carried out by mounting an oscillator through suitable foundation bolts on the top of the block. The oscillator could be suitably set to produce either vertical or horizontal vibrations. The oscillator was driven with a motor, the speed of which was varied with an independent speed control unit. The oscillator was excited in steps starting from at rest. The vibrations of the concrete blocks were picked up by acceleration pick ups fixed on the blocks at suitable places and amplified by means of an amplifier. The amplitudes were recorded through an oscilloscope. Amplitudes at different frequencies and three dynamic force levels ($\theta = 70^\circ$, 105° and 140°) were recorded. Photo. 2 shows the concrete block with oscillator and motor mounted on it.

Both the vertical and horizontal resonance tests were carried out.

Fig. 11 shows the frequency versus amplitude plots for vertical block resonance at Kiln location (RB_1 -Fig. 1) at angle of eccentricity setting of the oscillator for $\theta = 105^\circ$ similar curves were obtained for other eccentricities. The natural frequencies obtained from these curves are tabulated in column 3 of Table 5.

The coefficient of elastic uniform compression (C_u) is obtained using the Eq. (9).

Photo. 2. Concrete block with motor and oscillator for block resonance test

Fig. 11. Frequency-amplitude plot from vertical resonance test at kiln site

Table 5. Vertical resonance test results of Kiln site

S. No.	θ	f_n (Hz)	C_u for area 1.125 m^2 (kN/m^3)	C_u for area 10 m^2 (kN/m^3)
1	2	3	4	5
1.	70°	—	—	—
2.	105°	30.0	6.0×10^4	2.02×10^4
3.	140°	28.7	5.32×10^4	1.80×10^4

$$4 \pi f_n^2 = \left(\frac{C_u \cdot A}{m} \right) \quad (9)$$

$$C_u = \left(\frac{4 \pi m}{A} \right) f_n^2$$

where m = mass of the block including the weight of mountings

A = area of foundation block

f_n = natural frequency

Values of C_u obtained using Eq. (9) are tabulated in column 4 of Table 5. The values of C_u for 10 m^2 area were obtained using Eq. (8) and are tabulated in column 5 of Table 5. The results for Raw mill site are tabulated in Table 6.

Fig. 12 shows the amplitude versus frequency plots for horizontal block resonance test for eccentricity setting of 70° and respectively for test conducted at Kiln site. In each of these figures two curves corresponding to two acceleration pickups fixed at top and bottom of block have been plotted. The natural frequencies obtained from these figures are tabulated in column 3 of Table 7.

When oscillator is mounted on the top of the block, the horizontal sinusoidal force produced by the oscillator caused the block to translate in the longitudinal direction as well as pitching about transverse axis of the blocks. The system thus constituted a two degrees of freedom system. To determine whether the natural frequencies obtained from figures correspond to first or second natural frequency phase was checked each time. It was seen from the phase that the natural frequency obtained is the first natural frequency (Barkan, 1962). After determining the mode of vibration the coefficient of elastic uniform shear (C_r) is obtained using the following relation.

$$C_r = \frac{8 \pi^2 \gamma^2 f_{nx}^2}{A_0 + I_0 + \sqrt{(A_0 + I_0)^2 - A_0 I_0}} \quad (10)$$

where $\gamma = \frac{M_m}{M_{m0}}$

f_{nx} = horizontal resonant frequency of model and block system

$I_0 = 3.46 I / M_{m0}$

M_m = mass moment of Inertia of block about horizontal axis passing through the centre of contact area of block with soil and perpendicular to the direc-

Fig. 12. Frequency-amplitude plot from horizontal resonance test at kiln site

Table 6. Vertical resonance test results of raw mill site

S. No.	θ	f_n (Hz)	C_u for area 1.125 m^2 (kN/m^3)	C_u for area 10 m^2 (kN/m^3)
1	2	3	4	5
1.	70°	30.25	5.9×10^4	1.98×10^4
2.	105°	29.7	5.7×10^4	1.92×10^4
3.	140°	27.8	5.0×10^4	1.67×10^4

Table 7. Horizontal resonance test results of Kiln site

S. No.	θ	f_n (Hz)	C_r for area 1.125 m ² (kN/m ³)	C_r for area 10 m ² (kN/m ³)
1	2	3	4	5
1.	70°	22.0	3.87×10^4	1.3×10^4
2.	105°	21.4	3.66×10^4	1.23×10^4
3.	140°	21.4	3.53×10^4	1.19×10^4

tion of vibration.

I = Moment of inertia of the foundation contact area about the horizontal axis passing through c.g. of area and perpendicular to the direction of vibration.

M_{m0} = Moment of inertia of block about the horizontal axis passing through the centre of contact area of block with soil and perpendicular to the direction of vibration.

Positive sign is taken when f_{nx} is second natural frequency and negative sign when f_{nx} is first natural frequency. For the size of block used in the test, the Eq. (11) reduces to for first natural frequency:

$$C_r = 0.008 f_{n1}^2 \quad (11)$$

Using the above relation, value of C_r for the area of block was computed and these values are listed in column No. 4 of Table 7.

The value corresponding to 10.0 m² were computed using Eq. (12)

$$\frac{C_{r1}}{C_{r2}} = \sqrt{\frac{A_2}{A_1}} \quad (12)$$

where C_{r1} and C_{r2} are the values of coefficient of elastic uniform shear for areas A_1 and A_2 respectively. The values of C_r for 10 m² area are listed in column 5 of Table 7.

An examination of Table 5 and 6 reveals that the value of C_u decreases with increase in value of angle of eccentricity setting θ , of the oscillator. As θ increases, dynamic force increases, therefore the values of C_u decreases with increase in dynamic force. Further it may be noted that the value of C_u at Kiln site and Raw mill site are in close agreement. To be on the safer side a value of 1.70×10^4 kN/m³ for $A=10$ m² may be adopted for design purposes. Suitable modification should be made in this value according to the area of actual foundation.

A glance of Table 4, 5 and 6 indicates that even for 10 m² area the coefficient of elastic uniform compression, C_u values are different. This difference is primarily due to the fact that the C_u values are affected mainly by two important factors namely confining pressure and strain level (Prakash and Puri, 1977). Thus the observed C_u values are corrected for a confining pressure of 100 kN/m². The corresponding values of the strain levels are also computed. The results are tabulated in Table 8.

It is evident from Table 8 that the strain levels in cyclic plate load test and resonance test are of the order of 10^{-2} and 10^{-4} respectively. Though with the limited data no definite trend can be indicated. However ignoring the value of C_u obtained at kiln site from cyclic plate load test, the observations indicate that the C_u value increases as the strain level decreases.

On the basis of the values of C_r listed in Table 7 an average value of C_r of 1.2×10^4 kN/m³ for $A=10$ m² is recommended for the design. This value also needs area correction as the C_u value.

Correction for water table:

No account of water table is made in the computations of dynamic elastic constants

Table 8. Corrected C_u -values for confining pressure of 100 kN/m² and area 10 m²

S. No.	Location of various structures	Strain level	C_u -value (kN/m ³)
CYCLIC PLATE LOAD TEST			
1.	Kiln site	1.3×10^{-2}	3.675×10^4
2.	Cement silo site	1.1×10^{-2}	1.30×10^4
3.	Raw mill site	1.0×10^{-2}	1.55×10^4
4.	Hostel site	0.44×10^{-2}	1.44×10^4
5.	Labour colony site	0.55×10^{-2}	2.78×10^4
6.	Unloading point site	1.5×10^{-2}	1.259×10^4
RESONANCE TEST			
1.	Kiln site		
	$\theta=105^\circ$	1.2×10^{-4}	2.93×10^4
	$\theta=140^\circ$	1.33×10^{-4}	2.6×10^4
2.	Raw mill		
	$\theta=70^\circ$	1.6×10^{-4}	2.84×10^4
	$\theta=105^\circ$	2.4×10^{-4}	2.745×10^4
	$\theta=140^\circ$	2.8×10^{-4}	2.388×10^4

(section, *Block resonance test*) considering the position of water table at a great depth below ground level. However, the influence of position of water table should be taken into account by multiplying the dynamic elastic constants by a correction factor, C_w given by the Eq. (7) (section, *Effect of water table*).

CONCLUSIONS

The heterogeneous nature of deposit and the varied type of loading imposed by the structures in a cement factory indicate that a special geotechnical investigation programme is necessary to estimate parameters for foundation design. The large sized in-situ tests were necessary in view of size of particles present and the nature of deposit. Large size in-situ direct shear tests are useful in such boulder deposits to estimate shear parameters. The plate load tests both horizontal and vertical were useful for estimating soil properties such as allowable soil pressure and modulus of subgrade reaction. A cyclic plate load test also helped in estimating the coefficient of elastic uniform compression. Elastic constants required for machine foundation design were obtained by both cyclic plate load and block resonance tests. The results obtained from different tests differed widely and need correction on account of strain level, ground water and area of the foundation. Thus the efficacy of proposed in-situ tests in a boulder deposit has been demonstrated inspite of several uncontrolled factors inherent in a field testing programme.

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