Foundation Treatment of Ore-Handling Yard by Preloading and Vertical Sand Drains

Traitement d'une fondation pour plate-forme de manutention des minerais au moyen d'un chargement préliminaire sur drains verticaux en sable

S. R. MEHRA, PROF., C.E., M.I.E. (IND.), M.I.C.E., F.N.I., *Director, Central Road Research Institute, New Delhi, India*
T. K. NATARAJAN, B.E., M.S. (ILLINOIS), *Assistant Director, Central Road Research Institute, New Delhi, India*

SUMMARY

This paper sets forth experiences concerning the foundation treatment for an ore-handling yard in Vizag Port, India. A 70-ft deep deposit of very soft clay was treated to carry 30-ft high dumps of iron ore weighing 160 lb/cu.ft. The technique adopted was to preload in stages in conjunction with the use of vertical sand drains. This preloading project represents one of the more extensive applications of partially penetrating vertical sand drains used expressly for purposes of strength gain.

SOMMAIRE

On decrit des expériences concernant le traitement de la fondation d'un plan pour la manutention des minerais à Port Vizag en Inde. Un gisement d'argile ayant 70 pieds de profondeur a été traité pour supporter des minerais de fer ayant une hauteur de 30 pieds et pesant 160 livres/pied cube. La technique adoptée était d'effectuer le chargement d'avance en temps successifs, de concert avec l'utilisation de drains verticaux en sable. Ce travail de chargement d'avance représente une des plus importantes applications de drains verticaux en sable penétrant partiellement, utilisés expressément dans le but d'un accroissement de résistance.

THE ORE-HANDLING SCHEME of Vizag Port involves the transportation of iron ore from the mines to the port by rail, and the stacking and loading of it on to ships for export to Japan. The capacity of the ore-handling scheme is designed to be 2,000 tons per hour. Fig. 1 gives a general plan showing the rail tracks and the ore dumps in the port area which require treatment. A stacker track, running between the two rows of ore dumps (Fig. 1), was already under construction, rendering it impracticable to treat the depth of foundation soil just underneath the stacker track by vertical sand drains. The need for some kind of foundation treatment was recognised only after the project was under way and after the railway embankment had begun to fail when it reached a certain height. Since time was important in this project, the preliminary investigations on strength characteristics were done very quickly. The average shear strength at the time of failure was estimated by analysing the stability of the fill which had failed. The reliability of this estimate was subsequently borne out by detailed subsoil testing at a later stage.

In brief, the problem was that to ensure the stability of ore dumps the shearing strength of the subsoil clay had to be increased from an original value of $\tau = 250$ psf to an ultimate value of $\tau = 950$ psf (average). Furthermore, the proposed technique had to be tailored to the following requirements:

1. The use of permanent counterweight or loading berms was ruled out for lack of room in the layout of the ore yard.
2. The ore-handling machinery must travel at a specified grade.
3. The design height of the iron ore dumps (unit weight of ore = 160 lb/cu.ft.) was 30 ft.
4. The construction of the stacker track between the ore dumps was already on the verge of completion and accordingly the manner of proposed foundation treatment could not involve any disturbance of the stability of the track as already placed. Therefore, the depth of clay just underneath the stacker track had perforce to be left untreated.
5. The design aimed at a minimum factor of safety of 1.3 with respect to bearing capacity failure.
6. The requisite degree of stabilization of the subsoil was to be accomplished within 6 to 9 months.

The answer to this problem was determined to be preloading in conjunction with the use of vertical sand drains. The preloading itself was to be done in stages to prevent heaving of the stacker track. The observations made during subsoil exploration and analysis of foundation studies and the important features in the design of sand drains are set out below.

DESIGN DATA AND INDEX PROPERTIES OF SUBSOIL

Very soft clay, $c_u = 250$ psf
Depth of soft clay = 65 ft
Water table = 1 ft below original ground level (RL = +4)
Liquid limit = 80–110
Plasticity index = 55–75
Natural water content = 70–90 per cent
Initial void ratio = 1.9–2.3
Dry bulk density = 50–60 lb/cu.ft.
Length of stretch treated = 2,420 ft
Width of stretch treated = 520 ft
Weight of iron ore = 160 lb/cu.ft.
Design height of ore stacks = 30 ft
Drainage face = one only
Sensitivity of clay = 2–4 (average)
No. of sand drains = 6,500 (75 ft deep)

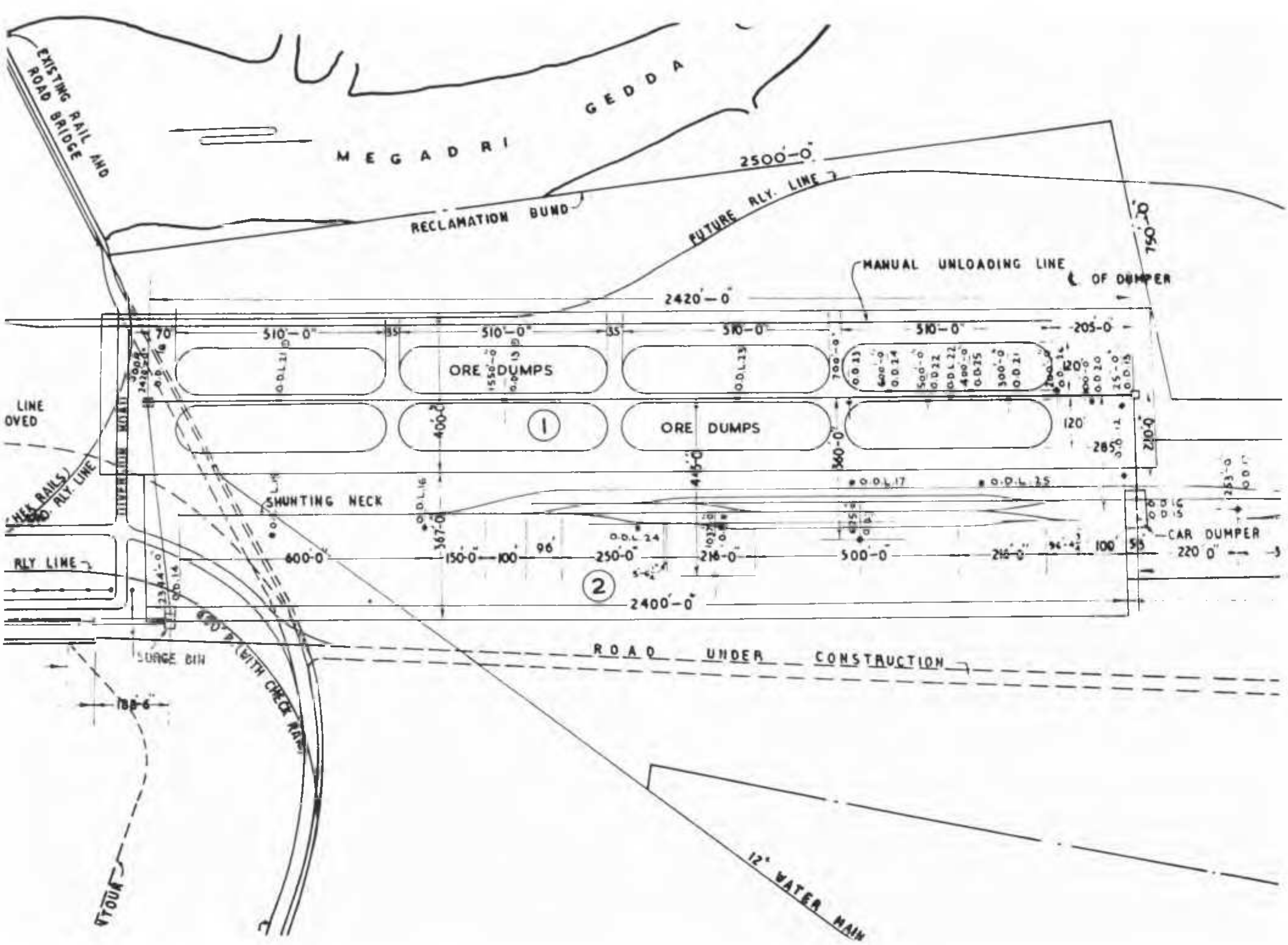


FIG. 1. General layout of ore dumps and rail tracks.

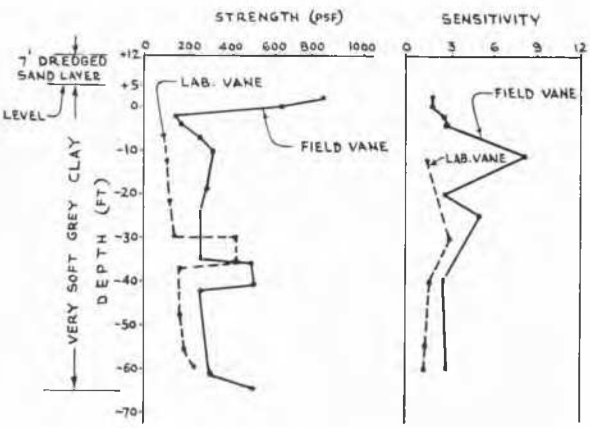


FIG. 2. Strength and sensitivity profile of subsoil.

Fig. 2 depicts the strength profile and sensitivity of the soil at different depths; Fig. 3, the cross-section of the railway embankment, iron ore dumps, and the stacker track; and Fig. 4, the railway embankment built up of the local clay, which failed when it reached a height of 11.5 ft.

The clay embankment that failed rested on a deposit of soft clay 25 ft in depth. The embankment was 70 ft wide at the top and 116 ft wide at the bottom, with a 2:1 side slope. The embankment material had a unit saturated weight of 90 lb/cu.ft. The borrow pits were 5 ft deep and 20 ft away from the toe of the embankment and appear to have added to the embankment's instability, as judged from an analysis of stability of the embankment at the time of failure. It is suggested that the stability analysis of fills on soft clay, when the depth of clay layer is less than 0.25 times the base width of fill, should be based on the theory of plasticity as applied to failure by plastic flow in a squeeze test (propounded by Jurgenson, 1940; and Sowers, 1962).

Another stretch of railway embankment built of "moorum" (gravelly soil), 14 ft high, was just stable. This 14-ft embankment with a base width of 120 ft was resting on a recent layer of dredged sand, 7 ft thick, overlying the soft clay deposit 70 ft deep. The stability analysis was performed using a slip-circle method involving the conventional assumption of a circular surface of sliding.

Yet another test fill, rectangular in plan, was built to failure. At a height of 15 ft signs of failure (tension cracks) became discernible. The stability for this case was analysed as for the bearing capacity of a rectangular footing.

These three specific cases of failure were regarded as constituting large-scale shear tests in the field and on this basis, it was established with reasonable certainty that the average shearing resistance of the subsoil was of the order of 250 psf. It is noteworthy that this estimated value from the stability analysis of embankments on the point of failure

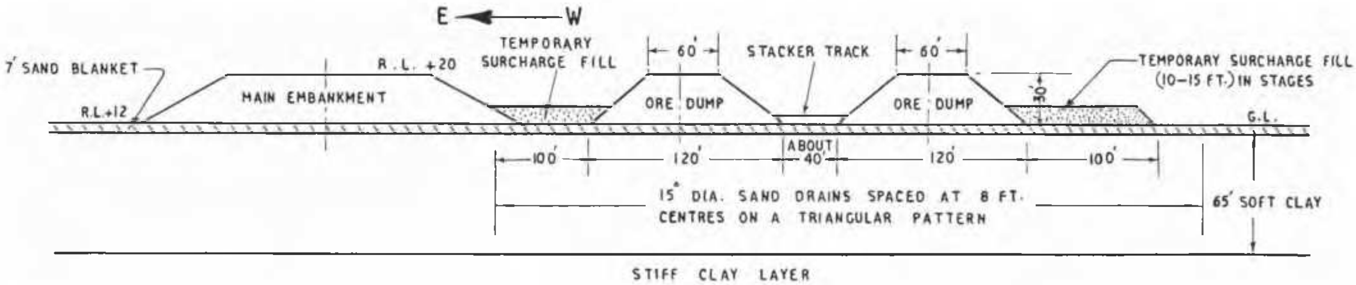


FIG. 3. Cross-section of ore dump, railway embankment, and stacker track. (Not to scale.)

FIG. 4. Instability of the base of fill already built to partial height without treatment.

agreed quite well with the results of field vane tests conducted subsequently, whereas carefully conducted triaxial tests and unconfined compression tests gave disquietingly lower strength values viz., 150–180 psf. The discrepancy is attributable to the fact that the samplers used in the preliminary exploration had a high area ratio so that sample disturbance accounted in a large measure for the low values of strength recorded by triaxial and unconfined compression tests. However, it was found that even with such partially disturbed samples the laboratory vane test bids fair to be more reliable.

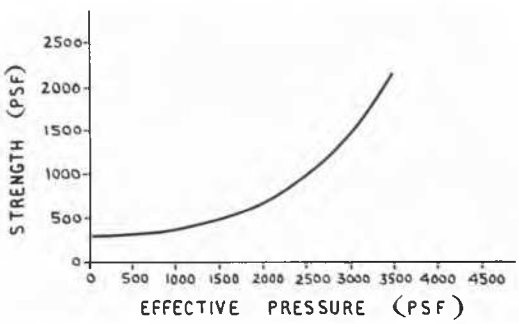
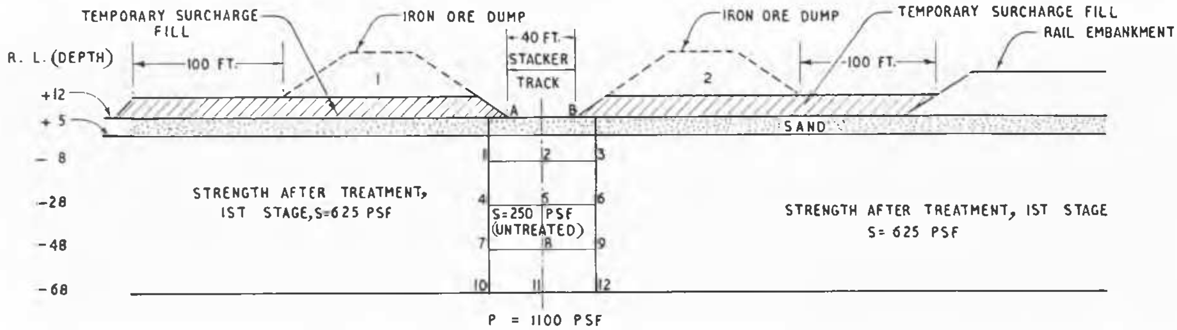


FIG. 5. Relation between strength gain due to consolidation and effective pressure.

STRENGTH GAIN *versus* CONSOLIDATION

A series of tests concerning the rate of strength gain at different percentages of consolidation for different load increments was conducted. The laboratory vane apparatus was a handy tool for conducting such tests. As a supplement, consolidated undrained triaxial tests were also done. Fig. 5 shows a typical plot of strength gain *versus* effective pressure (representing different percentage consolidation under different load increments). On the basis of such a study it would seem reasonable to estimate that the average strength of soft clay in the field would increase to $c_u = 625$ psf with $U = 90$ per cent under an overburden pressure equivalent to 1 ton/sq.ft. A 7-ft-thick, dredged sand layer recently reclaimed already exists in the area of the ore dumps. If a 10-ft high fill is built over it, it would act as a preload so that the combined weight of the 10-ft fill and the 7-ft sand layer would provide the needed pressure of 1 ton/sq.ft. for achieving the requisite degree of strength gain. In order to expedite the process of consolidation, the diameter and

POINT	CALCULATION	SHEAR STRESS DUE TO ORE DUMP 1	SHEAR STRESS DUE TO ORE DUMP 2	TOTAL SHEAR STRESS
1	0·29 P	320 PSF	0	320 PSF
2	0·22 P	232 PSF	−232 PSF	0 PSF
3	—	—	320 PSF	320 PSF



THE AREA IN WHICH STACKER TRACK IS LAID WILL NOT BE LOADED BY THE 10FT. FILL. THE STABILITY OF THE STACKER TRACK AGAINST HEAVING DUE TO THE LOADS FROM THE ADJACENT SURCHARGE FILL IS ANALYSED IN THE ABOVE TABLE. THE SHEAR STRESS IS MAXIMUM AT POINTS 1 AND 3 AND IS 320 PSF. SAND DRAINS HAVE BEEN RECOMMENDED TO BE INSTALLED AT POINTS A AND B. SHEAR STRESSES ARE ZERO AT POINTS 2,5,8,11 AS TWO EQUAL AND OPPOSITE AMOUNT OF SHEAR STRESS FROM ORE DUMPS 1 AND 2, CANCEL EACH OTHER OUT.

FIG. 6. Analysis of stability of stacker track against heave.

spacing of the sand drains were so designed as to achieve a minimum of 90 per cent consolidation within 230 days after the surcharge fill and the sand drains were in place. The important considerations in limiting the height of the preload or temporary surcharge fill were that the surcharge fill had to be discontinued over the width of the stacker track and that the stacker track should not heave due to placement of the fill. A typical set of calculations for analysing the safety of the stacker track against a possible heave is set out in Fig. 6. The stability was analysed by determining shear stresses at critical sections beneath the side slopes of the ore dumps at different depths and checking if the stresses were less than the average shearing resistance of the subsoil. Since the critical section happens to be beneath the midpoint of the side slope and in the vicinity of the toe, it was considered advisable to put in a row of sand drains 7 ft apart along the entire length and on either side of the stacker track.

The strength gain is related to the degree of consolidation. It is significant that the value of $c_u = 625$ psf, desired after the first stage of preloading, is the average value for the treated layer. It is difficult to compute the per cent consolidation at a specified depth in a clay layer for the three-dimensional case where sand drains are used and it was found expedient (as suggested by Lobdell, 1959) to assume a value between the per cent consolidation at the given depth due to one-dimensional consolidation and the average per cent consolidation for the layer due to three-dimensional consolidation with sand drains. In this project the building of the ore dump to a partial height of 22 ft itself constitutes the second stage of preloading. This would be undertaken only when field tests confirmed that the specified average value of shearing resistance of the clay layer, viz. 625 lb/sq.ft., had been attained.

The sequence of operations involved is explained in the following set of recommendations.

1. Install sand drains at 8-ft centres in a triangular pattern over the entire area. The sand drains will penetrate through the dredged sand layer and also through the entire depth of the soft clay (65 ft) layer so as to reach firm bottom.

2. Place a surcharge fill 10 ft high (weight = 120 lb/cu.ft.) or equivalent to the height of any other fill material, covering the entire area. The fill material will be discontinued over the stacker tracks. The side slopes of the temporary fill on either side of the stacker track will be 2:1.

3. Allow a time lapse of 230 days subsequent to the installation of sand drains and the placing of the surcharge fill.

4. During the earlier part of the specified waiting period, check, by means of settlement platforms and piezometers, that the rate of consolidation and strength gain is in accord with the computed rate of consolidation and strength gain. If the rate happens to be slower, increase the height of surcharge fill sufficiently to reach the target value of average shearing resistance of subsoil, viz. 625 psf.

5. The anticipated subsidence of the ground which is of the order of 5 ft should have occurred at this stage. Whatever the actual amount of total subsidence, now remove the surcharge fill for a width equal to the base width of the proposed ore dumps.

6. Place the iron-ore dumps (weight = 160 lb/cu.ft.) up to a height of 22 ft. At this stage, the factor of safety with respect to stability will be 1.3.

7. At the same time, raise the height of surcharge fill

remaining on either side (for the 100-ft width) by another 4 ft of iron ore (weight = 160 lb/cu.ft.) or an equivalent height of any other fill material.

8. When both the increased height of surcharge fill and the weight of iron-ore dumps have been in place for a period of 230 "effective days," it is expected that the average shearing resistance of the subsoil would increase to 950 psf. When this criterion is fulfilled, as testified by test results obtained at that stage of the project, the ore dumps can be taken to a height of 30 ft.

9. The surcharge fill can finally be removed, after ensuring a minimum factor of safety of 1.3 with respect to stability of the ore dumps.

DESIGN OF SAND DRAINS

The spacing of sand drains was 8 ft on a triangular pattern and the diameter of the sand drain was fixed as 15 inches from considerations of (*a*) peripheral smear, (*b*) well resistance, (*c*) achieving $U = 90$ per cent in $t = 230$ days, (*d*) difficulties of arching of sand inside the mandrel during installation. The design was based on the theory suggested by Barron (1948).

The ratio, C_r/C_v was determined to be 2.0 and C_v was equal to 0.05 sq.ft/day. The permeability ratio was determined in the laboratory by conducting pairs of consolidation tests, one on a sample placed in the conventional manner in the consolidometer and the other on a sample placed after orienting through 90°. It is also proposed to conduct field permeability tests making use of the Casagrande open-type piezometer to check the permeability ratio.

In some stretches, partially penetrating vertical sand drains will be adopted, leaving the bottom 20 ft of the 70-ft-deep deposit untreated. Stability computations for the ore dumps indicated a factor of safety of 1.2 provided the surcharge fill is not removed but is left in place. The stability analysis was performed by (1) the slip-circle method, assuming circular surface of sliding, (2) the method of sliding block analysis assuming composite surface of sliding such that part of the failure surface is horizontal and lies in the soft untreated clay layer. The thickness of deposit which is left untreated is based on results of stability analysis and percentage consolidation achieved in the treated thickness in a given time interval. It was assumed conservatively, for purposes of estimating the strength gain in the layer treated with partially penetrating sand drains and for purposes of choosing the diameter and spacing of the drains, that the clay below the depth of penetration remains in its original state of strength. The time rate of consolidation in the treated layer was computed according to the numerical procedure of analysis for partially penetrating sand drains developed by Hart, Kondner, and Boyer (1958) and the strength gain estimated on the basis of correlation between strength and effective pressure established by a series of tests on the type of clay treated. The use of partially penetrating drains necessarily means that a higher average value of shearing resistance must be reached than with the use of fully penetrating drains in order to secure the same factor of safety with respect to stability. Thus, with partial drains, the intensity of preloading would have to be greater and be done in a greater number of stages.

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