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Failure of Sewerage Mains Constructed in Soft Estuarine Deposit

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SYNOPSIS Rising and effluent mains of 600mm and 750mm diameter were constructed at two adjacent creek crossings in New South Wales, Australia. A bund wall was constructed across each creek and the mains were laid in a trench excavated through the bund wall into the underlying soft deposit. Foundation preparation consisted of pouring rockfill into the trench and compacting with a few strikes of the excavator bucket. After construction unacceptably large joint openings of up to 120mm in width were measured. The authors were engaged to undertake a review of the geotechnical characteristics of the crossings and comment on the construction method employed by the contractor. This paper outlines the conditions encountered, discusses the analytical methods used to calculate movements of the mains and effective depth of rockfill and compares the results with the measured values. It is found that rockfilling for foundation strengthening was unsuitable for this site.

1. INTRODUCTION

Shortly after construction large movements were noted in the effluent and rising mains at Mudd Creek and Stony Creek crossings in central coastal region NSW, Australia. Also, lateral movement of the excavation support, and settlement and cracks of the adjacent road shoulder were observed. Subsequently, the authors were engaged to undertake a review of the geotechnical characteristics of the crossings and comment on the construction method employed by the contractor in the light of large movements that occurred.

Deep excavations in soft clay can cause significant settlements and lateral displacements of the surrounding ground. Adjacent structures can be damaged if the movements are not controlled by an appropriate design and construction method.

Actual behaviour of soft clay foundation is very complex. Extensive studies have been carried out on excavation and embankment in soft clay. Some of these are outlined below:

Deformation control around supported excavations is often the most important function of excavation support systems. Instructive studies of stability failures of structurally supported excavations in soft clay have been described by Bjerrum and Eide (1956) and Broms and Stille (1976), where Bjerrum examined soil heave into the bottom of the excavation and Broms reviewed deep rotational movements.

Detailed investigation of instability of a supported excavation in clay has been carried out by Clough et al (1981). Among the possible modes of instability basal heave into the excavation bottom and deep seated rotational slump-type failure have been found to be the most predominant cases. For performance of excavation the authors classified soft clays by stability number $N = \sigma_o/S_u$ in which σ_o is total overburden pressure and S_u is undrained shear strength.

Bending of pipelines in laterally deforming soil adjacent to trench excavations has been analysed by Crofts et al (1980). In the analysis the local movement of a long buried pipe was simulated by an elastic model. The elastic model consisted of an elastic beam embedded in an elastic foundation, which is locally displaced laterally.

Ladd et al (1974) found that the SHANSEP (stress history and normalised soil engineering properties) concept of normalised soil parameters which are dependant on over consolidation ratio (OCR), can be applied to determine the stability of soft clay deposits. The method is totally dependant on a good knowledge of the stress history. High quality sophisticated techniques of laboratory testing are also required.

The Finite element method (FEM) has found one of its major uses in geotechnical engineering in the analysis of embankment and excavation deformations. Mana (1977) and Wong et al (1989) have investigated the stability of braced excavations in soft clay using this method.

For stability analysis of a full scale test embankment on soft clay, Poulos et al (1990) carried out an elasto - plastic finite element analysis to consider porewater pressure effect.

The Centrifuge model has been adopted by Bassett et al (1981) and Almeida et al (1986) for analysis of an embankment on soft clay.

A finite layer elastic analysis (FLEA) method has been developed by Small et al (1984 and 1986) at the University of Sydney, Australia for settlement analysis of layered soils. The lateral displacement, the settlements and the heave of a deep excavation in soft clay can be estimated.

In this paper assessment of the longitudinal displacement of a buried pipeline adjacent to a trench excavation based on FLEA under plain strain condition will be presented. Computed soil movements have been compared with recorded data. Assessment of suitability of the employed construction method is also addressed.

2. GEOTECHNICAL PROFILE

The subsurface conditions were assessed from drill hole and static cone penetration tests (Dutch cone). The penetration tests indicated that the strata between 1m to 7.5m below the surface contains a significant proportion of fine materials consisting of silty sand and organic silty clays of estuarine origin. The estimated undrained shear strength of these materials are between 10kPa to 30kPa. The materials stay essentially as a slurry up to a depth of 4.5m becoming soft to firm between 4.5m to 7.5m below ground level.

The saturated silty sand interbeds between 1m to 7.5m depth are most likely a liquefiable material and a localised collapsing type settlement could be expected if a strong ground motion such as an earthquake happens to occur in the area in future.

A summary of insitu test results is given in Table 1.

Table 1. Generalised insitu test results.

Depth Range (m)	Typical q_c (MPa)	Typical Friction Ratio (%)	Average SPT Value (N)
0.0-4.5	0.2-0.6	5-10	0
4.5-7.5	1.2-3.2	3-7	1
7.5-10.0	5.0-7.0	2-4	6
10.0-15.0	7.0-17.0	2-5	11

3. CONSTRUCTION METHOD

The construction method used for installation of the two mains (diameters 600mm and 750mm) in a common trench can be described briefly as follows :

* A bund wall by dumping of fill materials was constructed approximately over 2/3 of the way across each creek crossing to divert water. This was 4-5 m wide at the crest and up to 2m high.

* A trench up to 4m in depth and 2.4m in width was excavated through the bund wall using a 34 tonne excavator.

* Initially "Krings Boxes" were used as the trench support. The lower ends of these boxes were placed approximately 400mm below invert level (referred as Section 1). Following noticeable deflection of the box sides and observation of unstable condition at the bottom of the excavation, the system was abandoned in favour of trench support by steel sheet piling for subsequent lengths (referred as Section 2). The sheet piles were driven approximately 3 - 3.5m blow invert level.

* Internal dewatering by pumping out from sumps was used to control water inflow into the trench.

* Rockfill consisting of hard rock admixture was dumped into the trench and pushed down with the excavator bucket to form a relatively firm surface at invert level. Geotextile and bedding sand were then placed over the surface.

* Segments of both mains were placed over the prepared surface.

* The trench was backfilled with selective sand followed by compacted sandy clayey materials up to the ground level.

* Krings boxes/sheet piles were progressively lifted during backfilling operation.

The construction of the Section 1 for a total length of 78m and the Section 2 for a total length of 156m were completed within 24 days and 57 days respectively. Elapsed time between construction and removal of the bund wall for Mudd Creek and Stony Creek were 36 days and 155 days respectively.

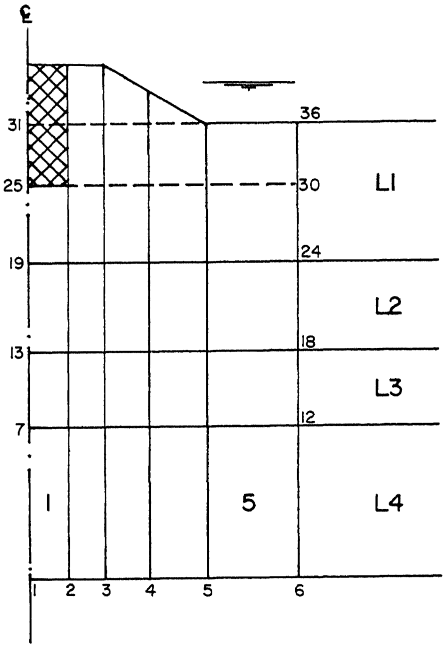
4. SETTLEMENT AND LATERAL SOIL DISPLACEMENT ANALYSES

Construction of structures such as a bund wall increases surcharge, creates downward pressure and increases porewater pressure at the substrata and induces vertical and lateral soil displacements in the soil layers. On the contrary, excavation through the bund wall releases weight, creates upward pressure at the base of excavation and reduces pore water pressure in the subsurface near the trench and induces lateral soil displacement around the trench mainly in soft layers. The soil moves towards the excavation and soft materials tend to flow into the trench from the base of the excavation.

The maximum variation in the porewater pressure values are expected to develop near the centre line of the bund wall and the trench. Evaluation of increase and decrease of porewater pressure by theoretical methods is highly complex and should ideally be made from in-situ measurements during construction. Unfortunately measurements were not available and the effect of porewater pressure had to be ignored in the analyses.

The subsoil material has been assumed to be ;aturated and incompressible. Poisson's :atios of 0.5 and 0.3 were assumed for soft :lay and sand layers respectively. The moduli)f elasticity were estimated from the static :one penetration tests after Campanella and ₹obertson (1988).

The FLEA method was used to estimate elastic novement of the subsurface due to surcharge from the bund wall and negative pressure from :he excavation. The subsoil profile was livided into 4 major layers L1 to L4 with each Layer into 3 sublayers (total 12 layers). Nodes were selected at the boundary of each Layer at the centre line of the excavation, and ıt different distances from the centre line as shown in Figure 1.



'igure 1. Finite layer and nodes used for ınalysis (sublayers are not shown).

One-dimensional loading of 40kPa and -80kPa lue to bund wall and excavation respectively rere considered in these analyses. The :esulting vertical and lateral displacements of :he soil at different distances from centre .ine are plotted in Figures 2 and 3.

ı summary of the analyses are given below:

ı Effect of Bund Wall Construction

· Downward vertical movement (settlement) of up to 186mm could be expected at the centre line under the bund wall.

· Upward vertical movement (heave) in the range of 30-40mm could be expected at a distance of 5m from the centre line.

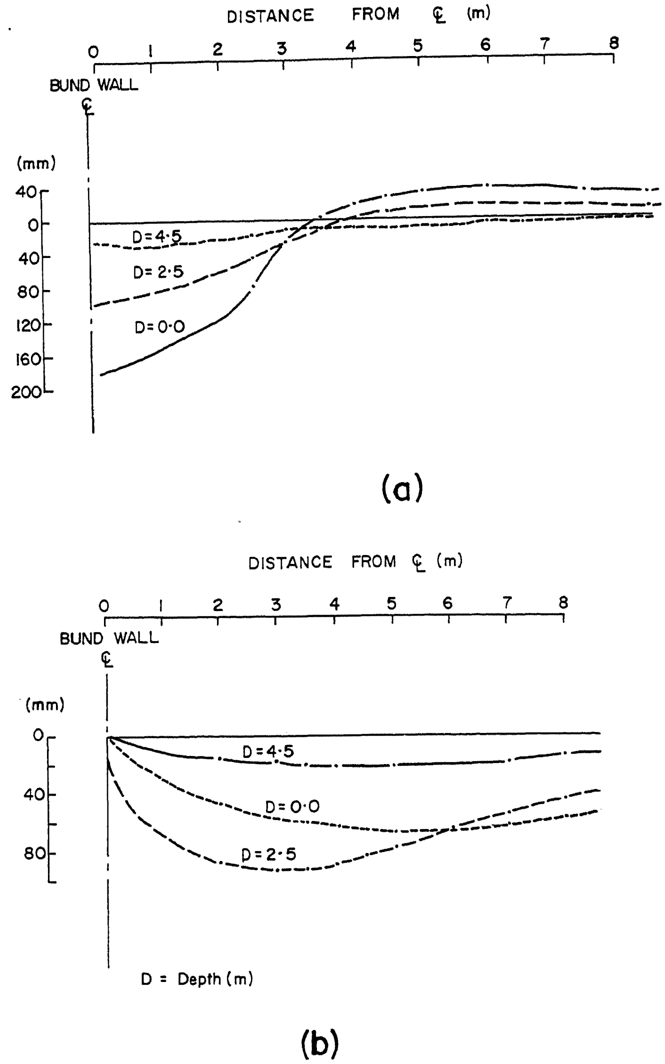


Figure 2. Soil displacements due to effect of bund wall at different depths and distances from centre line. (a) vertical movements. (b) lateral movements.

- Lateral movement (outward from bund wall) in the range of 90-100mm could be expected at a distance of 2-4m from the centre line.

B Effect of Trench Excavation

- Upward vertical movement (heave) of up to 320mm could be expected at the centre line.

- Downward vertical movement (settlement) in the range of 20-30mm could be expected at a distance of 4-6m from the centre line.

- Lateral movement (towards excavation) in the range of 90-100mm would be expected at a distance of 2-4m from the centre line.

Lateral movement in the direction of the mains axis would cause opening of joints in longitudal direction.

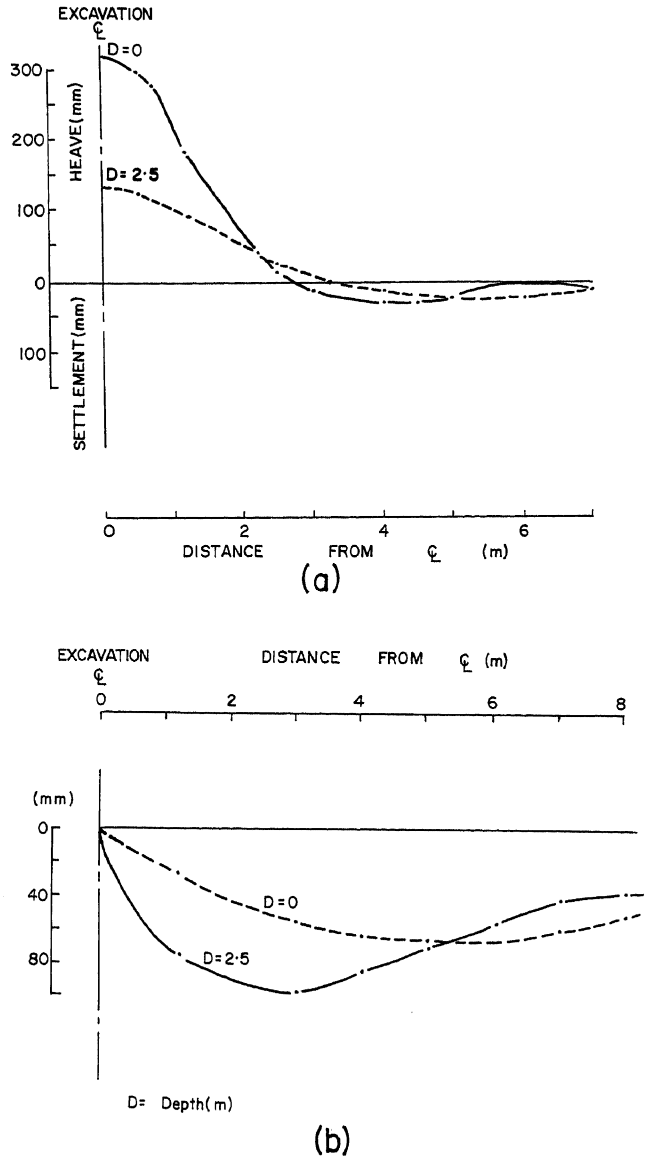


Figure 3. Soil displacements due to effect of excavation at different depths and distances from centre line. (a) vertical movements. (b) lateral movements.

5. SHEET PILING TRENCH SUPPORT SYSTEM

Stability analysis has been carried out to find minimum embedment depth required for the sheet piles to resist overturning. It is found that for a safety factor (S.F) of 1, minimum embedment required is at least 4.3m. Further limit analysis after L'Herminier (1967) has been carried out to find minimum embedment depth required to achieve a stable condition of the soil at the base of the excavation. It is found that for a S.F. = 1, minimum embedment required is at least 4.6m. The two approaches show good agreement as to the required embedment depth for the sheet piles to be just stable.

As stated before, information on porewater pressure, resulting from construction of bund wall was not available and its effect was discarded in the analysis. Consideration of the pore water pressure should show an increase in the required depth of embedment.

Therefore, for an embedment depth of less than 4.6m, base heave, soft soil flow into the trench, subsidence of excavation side and formation of tension cracks at the ground surface as evidenced on site could be expected.

During construction the possible lateral movements of soft soils sideways into the excavation was prevented or greatly restrained by the sheet piling. However, no such restraint existed longitudinally along the pipeline and the lateral loads on the previously backfilled pipes would cause movement of these pipes resulting in opening of the pipe joints.

6. ROCKFILL DUMPING

Rockfill consisting of hard rock admixture of sizes varying from 100mm to about 300mm was dumped into the trench and pushed down by repeated strikes with an excavator bucket. The objective was to apply dynamic compaction on the rock dumped into the trench and prepare a suitable foundation for laying of the mains.

Dynamic compaction, as generally applied, consists of dropping a large weight from a considerable height. Dynamic compaction may be used on most types of soil, but is reputed to give best results on medium grained, relatively free-draining soils. The depth of compaction achieved depends on the weight of the falling compactor, the height of fall, the number of drops at each location, the diameter of the falling weight and the strength of the soil (Rahimi et al 1987 and Charles et al 1981). The compacted depth has been analysed by method suggested by Rahimi et al (1987). For the purpose of the analysis it is assumed that the applied dynamic compactive effort with the excavator bucket is equivalent to a dynamic effect of 1 tonne weight dropping from 1m height with 6 drops at each location (this is most probably over estimated).

The analysis has indicated that the effective depth of compaction was unlikely to have exceeded 540 mm. This is confirmed by drilling through the rockfill encountered between 4.9m to 5.3m depths. Therefore it seems that with this process a thin layer (blanket) of compacted rockfill confined between the sheet piles virtually floating over soft soil was constructed. Although a firm trench bottom was achieved, the energy applied by the excavator bucket was not sufficient to push back soft soil and ineffective in creating a foundation of high bearing capacity at the base of the excavation.

7. TRENCH BACKFILLING AND LIFTING TRENCH SUPPORT

After placing the mains the excavated trench was backfilled and Krings boxes/sheet piles were lifted. After lifting of the trench

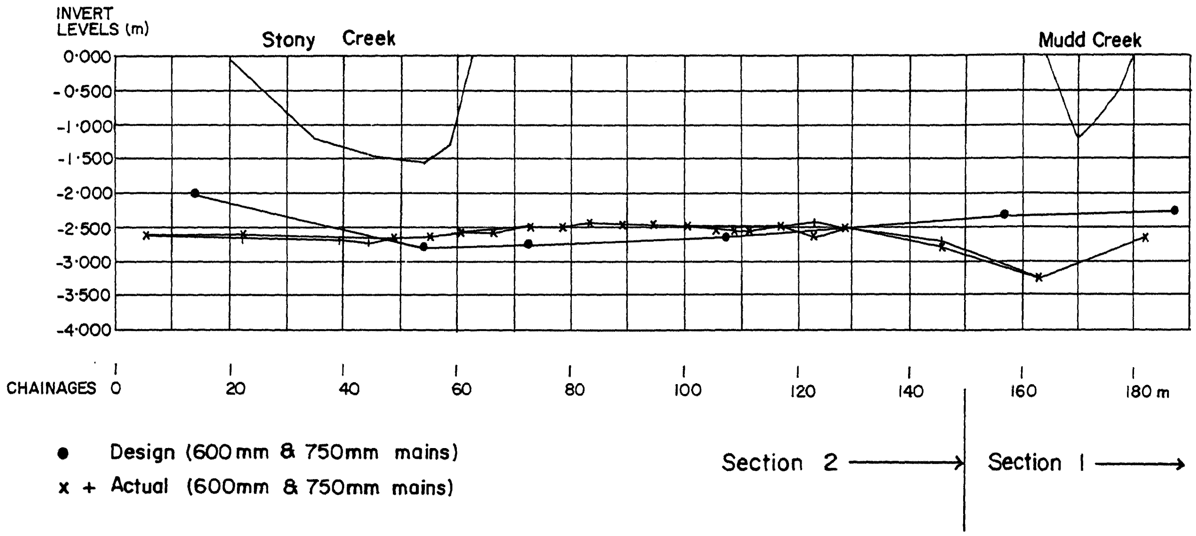


Figure 4. Design and actual position of mains.

support, the blanket of compacted rockfill would be in a condition of little lateral confinement and susceptible to lateral displacement. This lateral displacement would be non uniform due to non uniformly compacted rockfill.

Non uniform settlement also occurs from backfill loads and non uniform compaction of the backfill materials. In addition, non uniform deformation would be expected from long term stabilisation of localised internal stresses along the pipeline. It therefore follows that the pipelines would experience non uniform movements in both lateral and vertical directions.

Thus, accurate evaluation of the mains displacement using a theoretical model without factual data from the real situation would be very difficult.

8. ACTUAL MOVEMENTS

Vertical movements of the mains were monitored during construction by levelling and are presented in Figure 4. In this figure the new position of the mains after displacement in vertical direction is shown. The heave between the two creeks is due to the bund wall effect as predicted. The figure shows considerable relative displacements in vertical direction from joint to joint.

The 90-100mm movement calculated by FLEA in the direction of mains axis shows good agreement with the measured 120mm longitudal movement.

9. CONCLUSIONS

- Unconsolidated estuarine deposits encountered at a depth of 1m to 4.5m comprised admixture of sand, silt, clay and organic materials. The materials are very soft, saturated and of low bearing strength. Improvement of these materials to achieve a strong foundation by rockfilling with low energy compaction as discussed above is not possible.

- Due to the subsurface conditions, the trench excavation for construction of the pipelines would induce large soil flow towards the excavation and lateral soil movements of up to 100mm could be encountered.

- The rockfill used provided a floating foundation and due to its non-uniformity, and the variance of backfill compaction, the pipelines would undergo non-uniform movements in both the lateral and vertical directions.

- Compaction of a deep soft clayey estuarine deposit, using a excavator bucket is not effective to provide a structural foundation. Analysis indicated that the effective depth of such compaction is of the order of only 500mm. This is in good agreement with field data of 400mm.

- The technique suggested by Rahimi et al (1987) is suitable for modelling the behaviour of soft deposit treated with rockfill.

- Finite layer method (Small et al, 1984 and 1986) would appear to be capable of modelling soil movement in soft clay. The results of settlements, heave and lateral soil movements calculated from the elastic solution, finite layer method is in good agreement with the actual measured values.

- Effective stress analysis taking account of pore-water pressure should be more accurate than the procedure described in this paper which is based on total stress analysis. Additional field data are, however, necessary and given normal constraints of time and construction expediency such accurate analysis may not be possible. For practical purposes, total stress analysis should provide acceptable results.

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