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SYNOPSIS : A 25 m high fill embankment was constructed as part of a Trunk Road scheme in Hong Kong. In June 1985, large settlementsof the order of 250 mm were observed in the embankment. This was followed by a local slippage of surface material from the downstream slope face. To meet the schedule for opening the trunk road in late September, a number of emergency measures costing about HK$1 M (US$0.13 M) were implemented to stabilize the embankment. An investigation was subsequently carried out to assess the cause of the movement and the long term stability of the embankment.

This paper summarizes the sequence of events leading to the settlements and outlines the investigation carried out. The proposed hydrocompaction mechanism and the effects of grouting on the embankment are discussed.

INTRODUCTION

In 1982, the Government of Hong Kong started the construction of a dual three-lane highway in the north of the Territory. The road was formed by cutting into headlands, filling valleys and building bridges. One of the embankments, which was 25 m high, was completed in 1983. This embankment settled and deformed in 1985. The Geotechnical Control Office was requested to look into the causes of the deformation and to advise on the long term stability of the embankment within five months.

By analysing monitoring records and ground investigation data, the authors used the method of elimination and found that hydrocompaction might have initiated the settlements while later deformation was caused by redistribution of soil water. The hydrocompaction hypothesis was subsequently tested by a mathematical model.

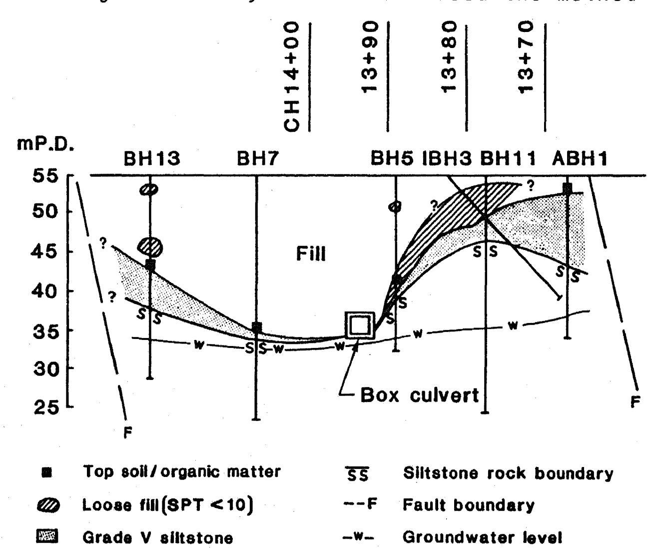


Figure 1 - Cross section through trunk road showing the geological and groundwater conditions

BACKGROUND

The Site

The embankment is located in the northern part of the Hong Kong New Territories. It has been formed by filling over a valley, with the existing stream course diverted through a 2.5 m square box culvert underneath. The maximum thickness of filling is about 25 m, and the side slopes are at 1 vertical to 1.5 horizontal.

The foundation rock is a highly fractured (RQD = 0 to 25%) sedimentary siltstone of Upper Jurassic age, the fracturing being due to faulting. The top layer of rock has been completely decomposed to a well-graded clayey sandy silt. The rock mass is fairly permeable ($k = 10^{-5}$ to 10^{-6} m/s) compared with the residual decomposed rock ($k = 10^{-7}$ m/s). The groundwater table is situated well down in rock. Though bouldery colluvium exists up-stream of the embankment, no colluvium was found under the fill. Figure 1 depicts a cross section through the trunk road showing the geological and groundwater conditions.

Construction History

Excavation for the box culvert commenced in early 1983. Because of the low groundwater table, no drainage layer was placed underneath the embankment. Filling of the embankment commenced in November 1983 after completion of the box culvert. The fill was borrowed from the adjacent slope and compacted in 300 mm layers using vibratory rollers. Filling completed at the end of 1983.

The northbound carriageway was then used for aggregate stockpiling from January to October 1984. During this period, distress was observed at the surface channels on the slope after heavy rain. The defective channels were repaired in November, and the stormwater drains and gullies adjacent to the centre-line of the road were constructed. Road surfacing was completed in May 1985.

SETTLEMENTS

In eary June 1985, after a few days of rain, settlements were observed at the central divider of the road near Chainage 13+82. Monitoring of the settlements commenced on 6th June. Figure 2 shows the area of monitoring. The embankment was then test loaded to 20 kPa.

In early July, a defective gully close to point r (Figure 2) was found - the base of the gully had settled 20 mm relative to its side walls. The slope surface was observed to be wet. Cracks were also evident at the surface channels and catchpits on the slope. By late July, settlement at point r had reached 250 mm. The gully and a length of cracked stromwater drain were subsequently replaced. However settlement monitoring in late August indicated that the road was still settling.

In September, a series of grouting were carried out, both within the embankment body and at the interface of the fill and the insitu ground. Altogether 86 grout holes were formed and about 300 m³ of cement were used. During drilling for the grout holes on 6th September, the insitu ground/fill interface was found to be 'saturated'. On the morning of 7th September, a shallow slip, involving a 1 m deep layer, occurred at the downstream slope face (Figure 2).

Reinstatement of the road surface was carried out after grouting, for road opening on 24th September. During the works, a small crack was observed in the gully previously replaced. The 3 gullies and a manhole on the embankment were subsequently blanked off.

The settlement histories of points r and s are shown in Figure 3.

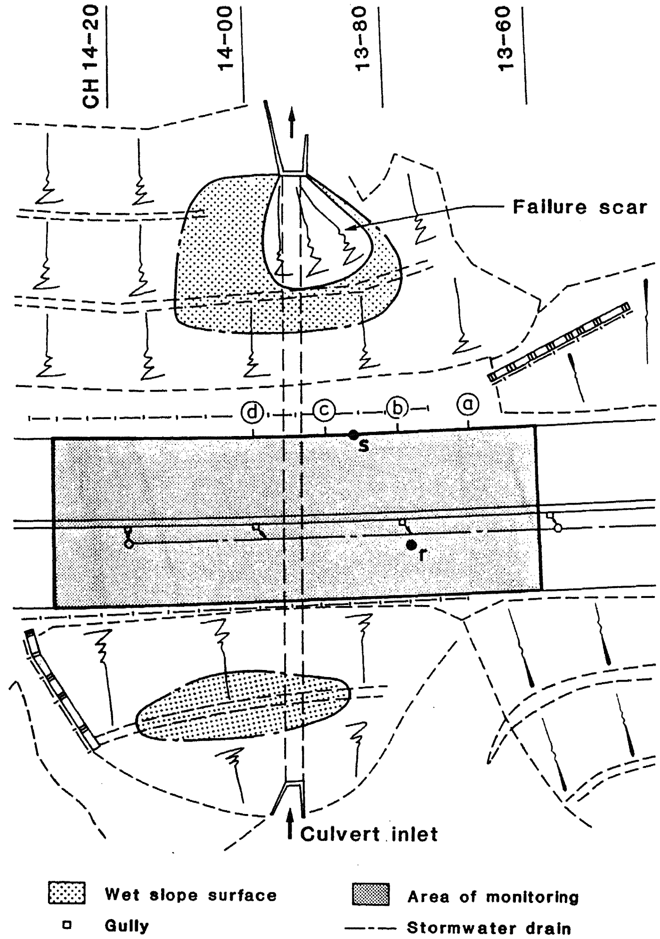


Figure 2 - Plan showing area of road surface monitored

INVESTIGATION

Site Investigation

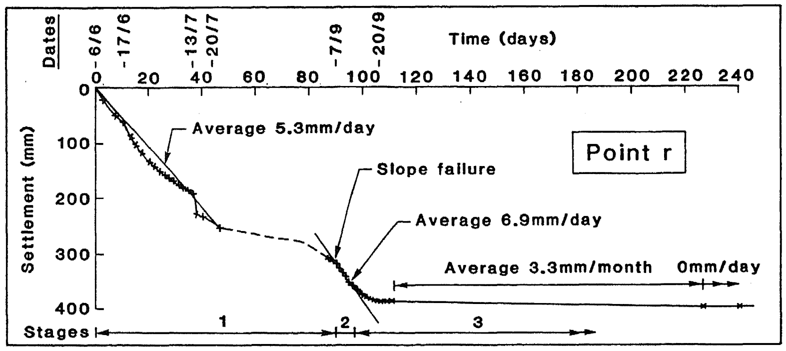
Though the emergency grouting works had stabilised the embankment, the cause of the settlements was unknown and it was felt prudent to carry out a full investigation. The investigation had to be completed within a period of about five months so that any necessary remedial works could be implemented before the rainy season arrived.

Desk study and ground investigation therefore went hand in hand. Records including design and construction records, settlement monitoring data and records of emergency measures were scrutinised. Aerial photograph interpretation and field geological mapping were carried out. Trial pits, inspection trenches, GCO probings (GCO, 1987), drillholes and piezometers were ordered to assess the geological and ground-water conditions and the density of the fill. Air foam was used as the flushing medium for the drillholes that were sunk through the embankment body, to minimize the possibility of any adverse effects on the stability of the side slopes (Phillipson & Chipp, 1981; 1982). Drilling was closely supervised. Standard penetration tests carried out in the drillholes revealed the presence of a patch of loose material above the eastern valley face (Figure 1).

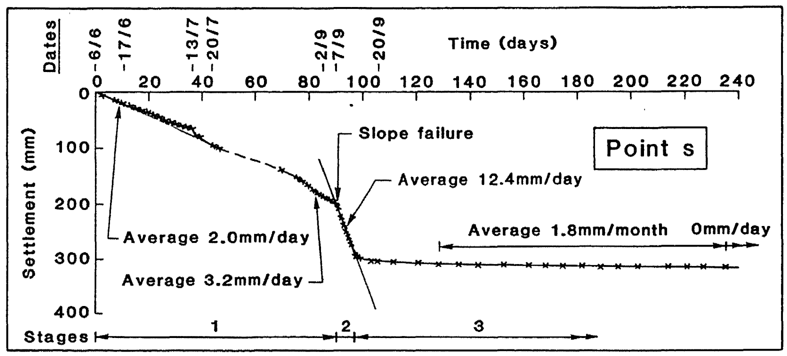
Laboratory Testing

A series of laboratory tests were carried out on the fill materials. Apart from classification and compaction tests, shear box tests (using 100 mm square direct shear box on test specimens remolded to a range of densities) and dispersion tests, including double hydrometer tests (Decker & Dunnigan, 1977), crumb tests (Standards Association of Australia, 1980), and the determination of Exchangeable Sodium Percentage (Flanagan & Holmgren, 1977; Sherard et al, 1976) were carried out. The latter tests showed that

Dates	Events
6/6/85	Monitoring started.
17/6	Test load of 20 kPa applied.
13/7	Leaking gully at Ch. 13+78 found. Testing revealed leaking stormwater drain.
20/7	Removal of test load.
Late July to Late Aug.	Replacement of defective gully and cracked stormwater drain. Road reinstatement.
6/9	Grout hole drilling commenced.
7/9	Part of downstream slope failed. Slope fully saturated.
10/9	Carriageway grouting started. Seepage from slope face during grouting.
16/9	Grouting completed.
19/9	Road surface reinstatement started. Crack observed in repaired gully. Three gullies and a manhole were blanked off.



(a) Settlement-time Relationship for Point r



(b) Settlement-time Relationship for Point s

Figure 3 - Settlement histories of points r and s with summary of events

the fill is non-dispersive.

A series of oedometer tests were also performed to investigate the compressibility characteristics and collapse potential of the fill. The test specimens, which were 75 mm diameter by 19 mm thick, were prepared from samples remolded at optimum moisture content. Two types of tests were carried out :

(a) Double oedometer tests (loading sequence of 25, 50, 100, 200 and 400 kPa)

Condition of Specimen	Density of Specimen	
	80% MDD*	95% MDD
Unsoaked (air-drained)	√	√
Soaked (double-sided drainage)	√	√

* MDD = Maximum dry density

(b) Wetting tests (soaking of compressed samples)

Total stress applied (kPa)	Density of Specimen	
	80% MDD	95% MDD
25	√	√
100	√	√
400	√	√

The results of these tests are summarized in Figure 4. It can be seen that there are two components of settlement, viz compression of the unsaturated material, and collapse due to wetting.

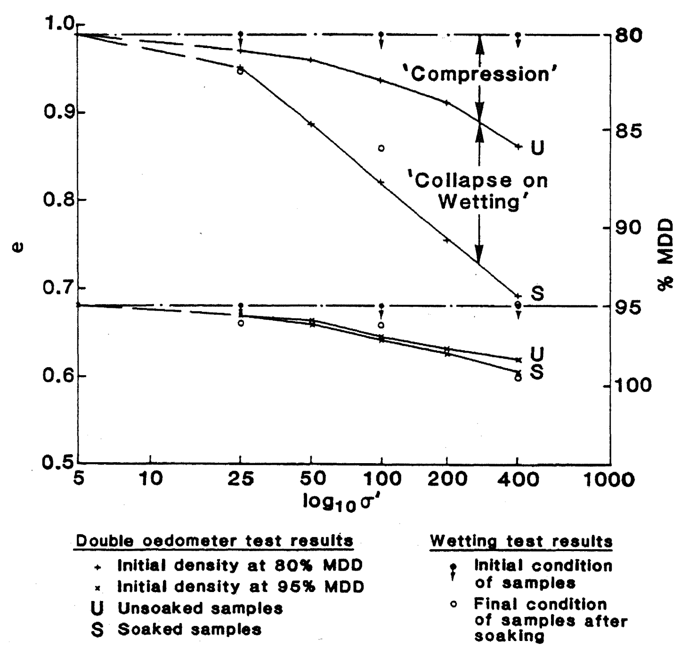


Figure 4 - Results of oedometer tests

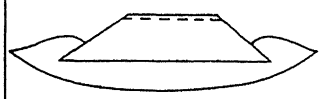
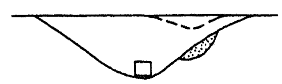
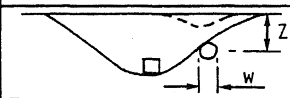
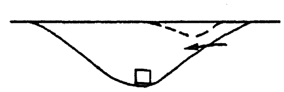
	No.	Cause	Expected Settlement Profile	Expected Settlement/Time	Supplementary Identifiers
Foundation Problems	IA	Foundation failure (in soil or in rock) due to embankment loading and surcharge		Takes relatively short time to complete and sudden.	Foundation condition. Bearing capacity of foundation.
Foundation Problems	IB	Settlement of compressible layers (e.g. colluvium and top soil) due to embankment loading and surcharge		Starts from the moment of loading. Settlement $s \propto \sqrt{t}$ (primary) or $s \propto \log t$ (secondary).	Presence of compressible layers.
Foundation Problems	IC	Collapse of cavities (e.g. solution cavities, structural cavities or voids in bouldery colluvium) due to embankment loading and surcharge	 Z W Types of cavities : (a) Sub-critical (b) Critical (Gaussian) (c) Super-critical Likely settlement profiles:  $\frac{W}{Z}<1.4$ (a) $\frac{W}{Z}=1.4$ (b) $\frac{W}{Z}>1.4$ (c)	Starts when the critical load of the cavities is reached and completes within a relatively short time. May be related to groundwater flow.	Mineralogy of rock. Groundwater chemistry. Cavities detected from drilling. Fracture state indices and nature of infill of rock joints. Effect of stress relief on joint opening. Presence of bouldery colluvium and rapid groundwater flow. Maximum settlement up to 90% of cavity size only. Volume of settlement trough less than cavity volume.
Problems with Fill Embankment Body	IIA	Collapse of deteriorating soil material (e.g. soil containing gypsum or other soluble salt) due to embankment loading and surcharge	Erratic. Depends on rain or groundwater movement. If fill is totally saturated, then settlement $\propto$ fill thickness. If local solution, then profile resembles cavity-type settlement profile.	Starts when critical void ratio is reached or shear strength reduces to limiting value. Can be gradual or stick-slip. Depends on rain or groundwater flow.	Mineralogy of parent rock. Groundwater chemistry. Sulphate and carbonate content of soil. Evidence of water flowing through fill.
Problems with Fill Embankment Body	IIB	(a) Consolidation and creep of fill (b) Settlement due to test loading	Settlement $\propto$ fill thickness.	Starts immediately after construction and application of test load. Consolidation $\propto \sqrt{t}$, and creep $\propto \log t$.	Estimate of rate of primary consolidation and order of consolidation and creep. Estimate of settlement due to test loading. Construction/compaction records.
Problems with Fill Embankment Body	IIC	Internal erosion (due to fast flowing water or dispersive soil) and subsequent settlement due to embankment loading and surcharge		Starts when critical void ratio is reached. Erratic and long term. Fluctuates with groundwater flow or rainfall.	Evidence of water flowing through fill. If fast flowing, water will spring out of slope face and adjacent hillside. Dispersion potential of fill. Presence of loose zones in fill and structureless materials in rock joints.
Problems with Fill Embankment Body	IID	Slope failure (or creep) due to groundwater flow, infiltration or additional loading		May be associated with rainfall. Can be rapid or slow.	Laboratory tests on shear strength of fill. Presence of perched water table. Factor of safety of side slopes.
Problems with Fill Embankment Body	IIE	Hydrocompaction (i.e. collapse of unstable soil structure, e.g. loose fill or fill compacted dry of optimum moisture content)	Depends on form of wetting. If totally saturated, then settlement $\propto$ fill thickness. If locally saturated, then profile resembles cavity-type settlement profile (see IC above).	Completes soon after first saturation but soil may take some time to be completely saturated, particularly if water is from a local source. If caused by infiltration, side slopes will gradually subside.	Estimate of collapse settlements of loose fill (oedometer wetting tests). Assessment of ground conditions for perching. Identification of source of water for hydrocompaction. Estimate of quantity of water required for saturation. Construction/compaction records. Assessment of fill density (SPT's and GCO probings).

Table 1 - Summary of the range of possible causes and their associated mechanisms

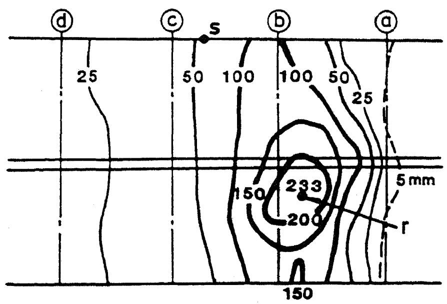
POSSIBLE CAUSES

Possible causes of the settlements can be divided into two categories, viz causes which arise from the foundation and causes which arise from the fill embankment body. Table 1 summarizes the range of possible causes and their associated mechanisms. Each of the causes was systematically evaluated in the investigation.

As the settlement monitoring records are the most valuable information available, the settlement contours and settlement-time relationship were used as primary identifiers of the causes. These were supplemented by results of the site investigation and laboratory testings.

Analysis of settlement data revealed that the settlements can be separated into three stages :

(a) Stage 1 settlements had a dish-shape and appeared to have resulted from a point disturbance around point r (Figure 5). They had a stick-slip relationship with time (Figure 3a), i.e. a 'collapse' type of settlement. Calculations indicated that the 20 kPa test load could have induced up to about 10% of the total settlements in this stage.



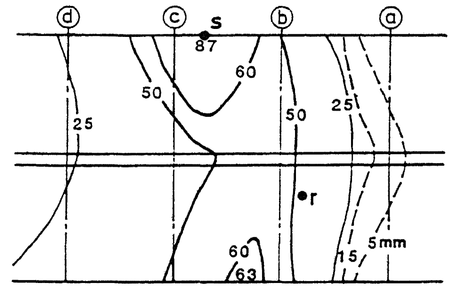
Prior to the removal of test load on 20/7

6/6 to 17/7

Figure 5 - Settlement contours for stage 1

(b) Stage 2 settlements were much more rapid (Figure 3b) and were likely to be related to slope instability (Figure 6). Stability analyses of the side slopes assuming 'dry' condition, i.e. zero pore pressures, gave factors of safety in excess of 1.2. However, measurements taken in the grout holes after drilling indicated high water levels in the holes. In one of the holes, water was found to be only 5 m below the road surface. Further stability calculations using the measured 'phreatic surface' revealed factors of safety close to unity. There was heavy rainfall between 4th and 6th September. The rain could have had some effects on the slope, but it could not be the main cause of failure as the slope had survived previously harsher rainstorms.

(c) Stage 3 settlements ranged from 1.8 to 3.3 mm per month. They could be due to effects of the previous stages or creep, which could give rise to a similar order of settlements.



Inclined holes grouting completed on 11/9

2/9 to 12/9

Figure 6 - Settlement contours for stage 2

HYDROCOMPACTION

Systematic assessment of the range of causes in Table 1 revealed that hydrocompaction is the most likely cause of Stage 1 settlements.

Hydrocompaction is defined as the collapse of an unstable soil structure upon wetting, a phenomenon which has been recognized for many years (Holtz, 1948; Hilf, 1975; Clayton & Simons, 1981). An unstable soil structure may result when the fill is loose or when compaction has been carried out at too dry a placement moisture content. The amount of collapse is dependent upon the density of the soil, the placement moisture content, the consolidation characteristics of the soil and the loading conditions.

During the ground investigation, a patch of loose fill was detected. It was not possible to identify the source of the loose fill, except that all the mechanisms for which the fill could be loosened subsequent to compaction had proved to be impossible. For hydrocompaction to have occurred, the following needs to be answered :

(a) What is the source of water entering the fill?

(b) Do the geological conditions at the site permit it to occur?

(c) Can the order of settlements be explained?

(d) Can the source of water provide sufficient quantities for hydrocompaction to occur?

Various sources of water have been examined during the investigation. The cracked gully was identified to be the main possible source. Its location is remarkably close to point r, where maximum settlement had taken place (Figure 2).

The fill was found to be resting on top of a relatively impermeable layer of decomposed siltstone, with some top soil and organic matter at the base of the fill. This fill/insitu ground interface would allow any descending water from the gully to perch on, thus permitting the formation of a wetted zone that can advance upwards to initiate hydrocompaction of the loose patch of fill (Figure 7). An idealised 1-D model was used to compute the order of settlement, using the oedometer test results. The following summarizes the analytical approach adopted.

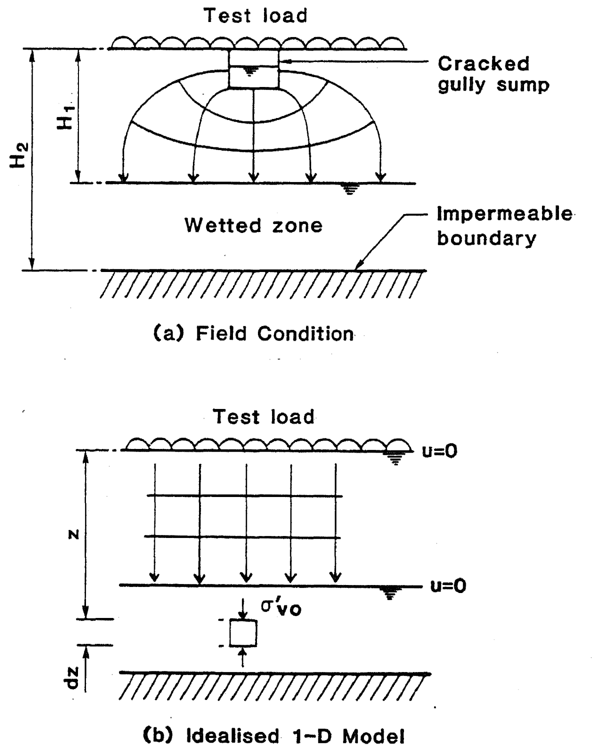


Figure 7 - Proposed hydrocompaction mechanism for collapse of loose fill

Figure 8a shows a typical set of wetting test results. A soil element denoted by point 1, upon wetting, will collapse to a denser state denoted by point 2. In the field, this will occur when the wetting front has advanced beyond the soil element at depth z (Figure 7b). The compression, ds, of the element is given by :

$$ds = \varepsilon dz \qquad (1)$$

where $\varepsilon = (e_o - e_f)/(1 + e_o)$ is the strain due to collapse, e_o and e_f are the initial and final void ratio of the soil element respectively. From the results of wetting tests, a relationship between strain ε and vertical effective stress σ'_v can be derived (Figure 8b).

For 1-D condition, the total settlement at ground surface, s, is given by :

$$s = \int_{H_1}^{H_2} \varepsilon dz \qquad (2)$$

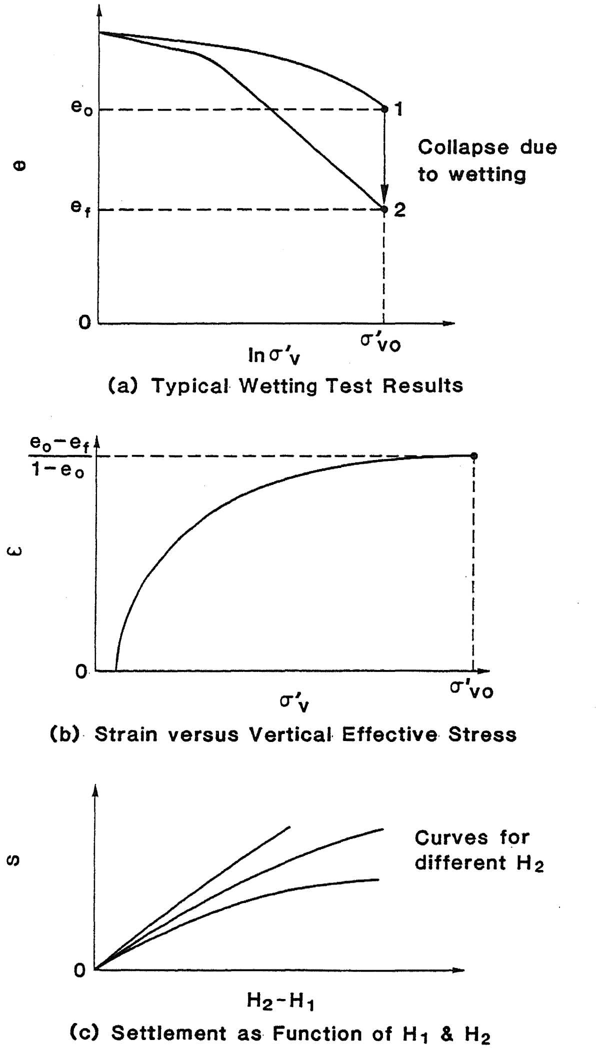


Figure 8 - Interpretation of oedometer test results

Using the above equations, a series of base curves were produced (Figure 8c) and these curves were used to evaluate the wetted depth H_1 at each settlement monitoring point.

The quantity of water, Q, required for hydro-compaction can be shown to be given by :

$$Q = \int_{vol.} (m_f \rho_{df} - m_o \rho_{do})\, dv \qquad (3)$$

where m_o, m_f are the initial and final moisture contents, ρ_{do}, ρ_{df} are the initial and final bulk densities, and dv is the unit volume of the soil element.

Integration was carried out over a volume of the ground above which settlements had occurred. Because the initial moisture content of the loose material is unknown, a range of likely values (m = 20% to 26%) were used to derive a corresponding range of Q values.

The next step in the analysis involved calculation of the quantities of leakage water from the cracked gully. This water came from the rain that fell onto the road carriageway during the period of settlements. The permeability of the soil is unknown but considered to be of the order of 10^{-4} to 10^{-5} m/s, based on the nature of the fill material. The total amount of leakage was estimated for this range of permeabilities, and the results were found to agree with the range of Q values calculated previously.

GROUTING

Drilling and grouting in September 1985 had some effect on the slope but it eventually stabilized the embankment.

Because of the low permeability of the fill, the cement grout could not penetrate the fill mass. The grouting pressures imposed horizontal stresses which fractured the fill and allowed stiff columns and sheets of grout to be formed within the embankment body. These pressures momentarily increased the pore pressures, forcing water within the embankment towards the slope surface. Some of the free water in the grout had also leached into the fill mass before it had time to hydrate with the cement. The drainage of this water together with the water driven off by hydrocompaction of the fill are considered to have caused the rapid movements of the slope shoulders.

The water from the grout gradually spreaded to other parts of the embankment body, causing collapse of any loose pockets it encountered. This redistribution and downward migration of water had caused the post-grouting movements, which became stabilized after the grout had hardened.

CONCLUSIONS

A rigorous investigation was carried out for a fill embankment which settled and showed signs of distress. A range of causes was considered, and, by a process of elimination, it was found that hydrocompaction, i.e. collapse of an unstable soil structure upon wetting, could have caused the early stage of settlements. A model was proposed for hydrocompaction of the loose fill. This model, together with the results of site investigation, lends support to the suggested cause.

Drilling and grouting was considered to have contributed to the slope movements. As the grout hardened, stability of the side slopes was restored.

As a result of the investigation, no major remedial works were considered necessary. Some minor works were recommended to deal with the loose fill and improve the stability of the downstream slope surface.

ACKNOWLEDGEMENTS

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REFERENCES

Clayton, C.R.I. & Simons, N.E. (1981). "The Collapse of Slabs on a Compacted Hoggin Fill" Ground Movements and Structures, Pentech Press, London, pp 655-670.

Decker, R.S. & Dunnigan, L.P. (1977), "Development and use of the Soil Conservation Service dispersion test", Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects. American Society for Testing and Materials, Special Technical Publication no. 623, pp 94-109.

Flanagan, C.P. & Holmgren, G.G.S. (1977), "Field methods for determination of soluble salts and percent sodium from extract for identifying dispersive clay soils", Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects. American Society for Testing and Materials, Special Technical Publication no. 623, pp 121-134.

GCO (1987), Guide to Site Investigation (Geoguide 2), Geotechnical Control Office, Hong Kong, 362 p.

Hilf, J.W. (1975). "Compacted Fill", Foundation Engineering Handbook, Van Nostrand Reinhold Company, New York, pp 244-311.

Holtz, W.G. (1948), "The Determination of Limits for the Control of Placement Moisture in High Rolled Earth Dams", Proceedings of the American Society for the Testing of Materials, Philadelphia, pp 1240-1248.

Phillipson, H.B. & Chipp, P.N. (1981), "High Quality Core Sampling - Recent Developments in Hong Kong", Hong Kong Engineer, vol. 9, no. 4, pp 9-15.

Phillipson, H.B. & Chipp, P.N. (1982), "Air Foam Sampling of Residual Soils in Hong Kong", Proceedings of the Specialty Conference on Engineering and Construction in Tropical and Residual Soils, Honolulu, pp 339-356.

Sherard, J.L., Dunningan, L.P. and Decker, R.S. (1976), "Identification and Nature of Dispersive Soils", Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 102, pp 287-301.

Standards Association of Australia (1980), "Determination of the Emerson Class Number of a Soil", Australian Standard Methods of Testing Soils for Engineering Purposes, no. AS 1289, C8.1-1980. Standards Association of Australia, Sydney, 3 p.