INNOVATIVE SOLUTIONS FOR THE CONSTRUCTION OF AN INTERCHANGE ON HIGHLY COMPRESSIBLE CLAY

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ABSTRACT: The Prospecton Interchange located near Durban, South Africa, is founded on thick, very soft, highly compressible, organic silty clays. The interchange is located in a built-up industrial area without room for fill sideslopes. The technical challenge was to accelerate the primary consolidation settlement of the clay and to construct retaining walls and bridge abutments which would be able to accommodate large settlements. This paper outlines the combined usage of mechanically stabilised embankments (MSE) and vertical band drains. Approximately 130 kilometres of drain was installed on a 2 m grid to a depth of 20 m. The predicted consolidation period of 7 years was reduced to 14 months for a consolidation of 575 mm. The MSE retaining walls and abutments settled over 500 mm during construction. Ongoing settlement was regularly measured against time and height of the fill. The MSE masses were able to accumulate these large settlements and the finished structures have claddings that are all within horizontal and vertical alignment tolerances. Once settlements were finished, the bridge deck distribution beams were cast and two simply supported decks were placed. A central pier is founded on deep piles. The interchange was constructed in 1988/89 and the solution has proved to have successfully met the technical requirements.

RÉSUMÉ: L'échangeur de Prospecton, situé près de Durban en Afrique du Sud, est fondé sur une épaisse couche d'argiles limoneuses et organiques, très molles et fortement compressibles. L'échangeur se situe dans une zone industrielle dense sans espace pour des talus de remblai. Le défi technique a été d'accélérer le tassement de consolidation primaire de l'argile et de construire des murs de soutènement et des culées de pont stables et compatibles avec de grands tassements absolus et différentiels, autant pendant la construction qu'en service. Cet article présente l'utilisation combinée de massifs en sol renforcé (MSE) et de drains verticaux en bande. Le délai de consolidation initialement estimé à 7 ans a pu être réduit à 14 mois, pour une consolidation totale de 575 mm. Les murs de soutènements et les culées, de type Terre Armée et atteignant une hauteur maximale de 8 m, ont tassé de 500 mm pendant leur construction. L'évolution du tassement par rapport au temps et à la hauteur de remblai ont été mesurés régulièrement et précisément grâce à l'emploi de cibles sur le parement des murs. Les massifs en sol renforcé ont été capables d'accumuler ces grands tassements pendant la construction, et leurs parements montrent finalement des alignements horizontaux et verticaux qui sont répondent aux tolérances. Quand les tassements se sont arrêtés, les poutres de support de tablier ont été posées sur les culées en sol renforcé et les tabliers simplement appuyés ont été construits. La pile centrale est fondée sur des pieux profonds. L'échangeur de Prospecton a été construit en 1988/89, et la solution géotechnique innovante a démontré son efficacité à répondre à l'exigeant défi technique.

GEOTECHNICAL SETTING

Introduction

Traffic congestion in the Prospecton Industrial area to the immediate south of the City of Durban, South Africa, required the construction of a new interchange across National Route 2 (N2). Its location in a built up area confined the limits of the construction, and the underlying 40m of alluvial sediments incorporated a high proportion of very soft, compressible clays.

Geology

The geology comprises alluvial sediments overlying Dwyka Tillite of the Karoo Supergroup. During the Pleistocene ice age fluctuations in sea level caused extensive erosion of the older sedimentary rocks. As a result the tillite's upper surface varies considerably. The action of streams and rivers filled in these deeply incised watercourses with unconsolidated alluvial sediments during the period of aggradation which followed melting of the ice cap. Main watercourses carried coarse sediments while backed-up streams and floodplains

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became the curators of fine muddy, often organic-rich, silt and clay sediments. The resultant estuarine deposits are a deep hotch-potch combination of coarse and fine sediments. Beneath the Prospecton Interchange are 40m of loose, fine to coarse silty sand, interspersed with firm to very soft layers of clay. The latter, up to 7m in thickness, consist of very soft, highly compressible, organic silty clay known colloquially as 'hippo mud'. It is this highly compressible material which is the chief cause of very long term differential settlements of overlying structures and hence the topic of this paper.

Investigation and results

The investigation comprised rotary cored boreholes drilled to rock; dynamic standard penetration testing (SPT) at 1,5m intervals; static cone penetration testing (Dutch Probes); undisturbed sampling and a range of laboratory testing. From an analysis of the data it was found that the Coefficient of Compressibility Mv for the very soft organic clay varied from 0,8m^2/MN to 1,6m^2/MN, with a Coefficient of Consolidation Cv of 0,4m^2/year. The geotechnical analyses indicated that 530mm of consolidation settlement could be expected beneath the interchange and that the consolidation settlement period would be 7 years.

Problem defined

The two major problems, therefore, were firstly to stop new interchange material from spilling onto adjacent properties and structures, and secondly, to accommodate severe differential movements without affecting the overall integrity of the bridge structure.

To accommodate spillage a mechanically stabilised earth (MSE) structure was adopted whereby the fill material could be confined between known limits without affecting the overall stability of the fill crossroad and ramp structures. Reinforced Earth (Pty) Ltd undertook the design for the MSE walls.

To accommodate the differential settlement required that:

- the overpass bridge would be simply supported by piers piled to rock at the bridge centre, with both ends of the two decks supported by MSE abutments.
- the MSE would accommodate differential settlements beneath the eccentrically loaded ramps and cross-roads.

A major problem still remained, however, viz. that of the time required for the clay to complete the consolidation process and this problem is discussed further in the sections following.

SOLUTION OPTIONS EVALUATION

Several options were investigated for accelerating settlement beneath the interchange and these may be summarised as follows.

Pre-loading

Pre-loading is a relatively inexpensive way of overcoming settlement problems. Sensitivity analyses showed, however, that pre-loading would take 7 years for 90% consolidation of the very soft organic clays. In calculating the consolidation period two-way drainage was expected, viz both up and down, with the shortest drainage path being 3m, assuming a clay thickness of 6m. If there were sand lenses within the clay the consolidation period could possibly be reduced to 4 years. But 4 years was still too long to have an unprotected fill open to the many coastal prevailing winds in an urban environment.

Pre-loading with surcharge

Here the method is to accelerate consolidation by building the embankment higher than the design level, and then to remove the excess material after consolidation is complete. For the technique to be effective the height of the surcharge should be at least 30% of the proposed embankment. Sensitivity analyses indicated that a surcharge of 4m to 5m (90% the embankment height) reduced the consolidation period from 7 to 4,7 years, but still not fast enough to satisfy the requirements. Also, this method would have the adverse effects of double handling of the excess fill and would reduce the global stability of the embankments.

Loading with vertical sand drains

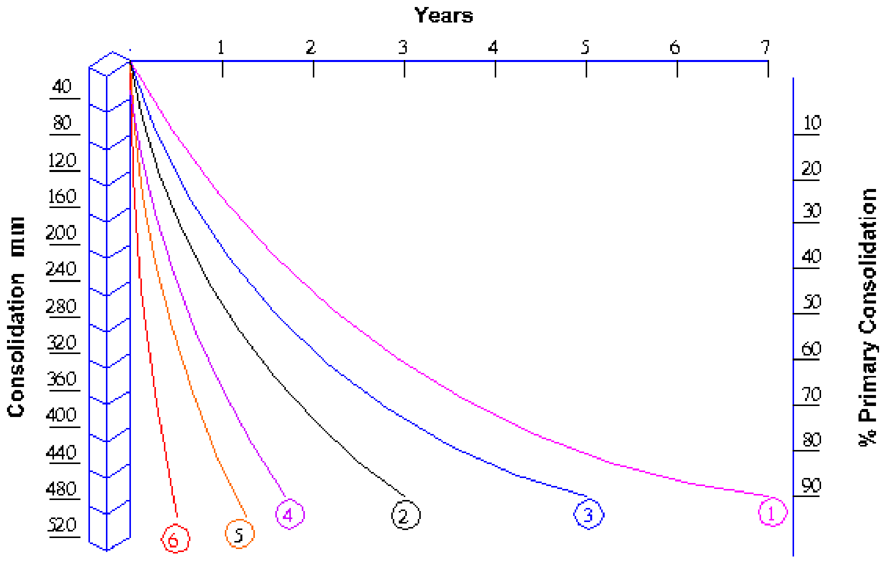
This method has proved successful in the past and if the vertical drains are spaced correctly, can have a marked reduction in the consolidation period. It can also be used in combination with pre-loading and with a surcharge. Limitations of the method, however, when compared to vertical band installation, are that the sand drains sometimes shear under load thereby breaking the required continuity; can be expensive in that infilling sand can vastly distort the cavity left when withdrawing the casing funnel and therefore large quantities of sand may be required; the construction period is time-consuming and tedious. The only perceived advantage is that the larger diameter of the sand drain allows a slight decrease in vertical drain spacing when compared to band drains.

Loading with vertical band drains

The effect of band drains in compressible layers will, depending on their spacing, markedly increase the rate of consolidation. Their effect is to reduce the drainage path length within the clay, with water flowing radially inwards towards individual drains under the pressure from the new embankment. This as opposed to up and down flow without vertical drainage. Calculations indicated that 2m vertical band spacing would reduce the consolidation period from 7 years to 12 months.

Solution selection

Figure 1 is a comparison of the various options discussed, or combinations of some of these. Installation of band drains would decrease the consolidation settlement period to within the construction period; would not require any surcharge fill and would dispense of the need for long term preloading. This was consequently the design option selected.



Key

1 Proposed 7 m high fill
2 7 m fill assuming sand lenses in the clay substratum
3 Proposed 7 m fill with a 4 m to 5 m surcharge
4 Vertical Sand drains on a 3 m grid
5 Vertical Band drains on a 3 m grid + a 4- 5m surcharge
6 Vertical Band drains on a 2 m grid

Figure 1. Graph of time for consolidation versus amount of consolidation (up to 90% consolidation)

BAND DRAINS

Band drain installation

A band drain comprises an inner corrugated plastic core to allow water flow, and an outer geofabric lining to prevent clogging of the core. The band drain lengths varied between 10m and 20m depending on their total penetration of the underlying highly compressible, very soft organic clays, with invert mostly 19m below surface. The band drains were installed using two crane rigs with mandrills forced into the soft sediments using large vibrating hammers. A sacrificial shoe at the base of the mandrill with band drain inside the column, results in the drain being retained within the cavity. Individual band drains are cut off at the top; covered with a continuous geofabric, in turn overlain by 500mm of sand. The latter to ensure non-wetting – and hence stability – of the overlying fill.

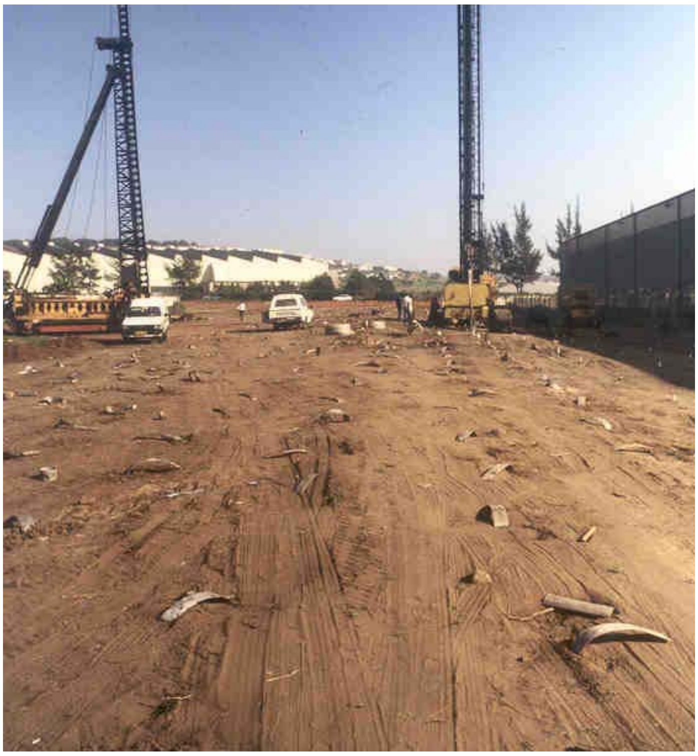


Figure 2. Installation of band drains

Construction statistics

Band drains were installed on a 2m grid pattern resulting in the installation of 8199 drains with an overall depth of 129 916m (130km), with an average length of 15,8m. Two rigs worked a 23 hour shift with a one-hour down-time for servicing. The overall installation period had an average installation time of 7 minutes per drain, although this did accelerate to 3 minutes towards the end of the installation period.

Results

The total consolidation settlement predicted amounted to 530mm with a consolidation period of 12 months. Monitoring indicates that the final settlement amounts to 575mm with a consolidation period of 14 months. A very slight increase from that predicted. Figure 3 is a log-time graphical representation of the primary consolidation settlement and secondary compression as measured after 12 years. The final total settlement is measured as 615mm of which 40mm is purported to be secondary long-term compression.

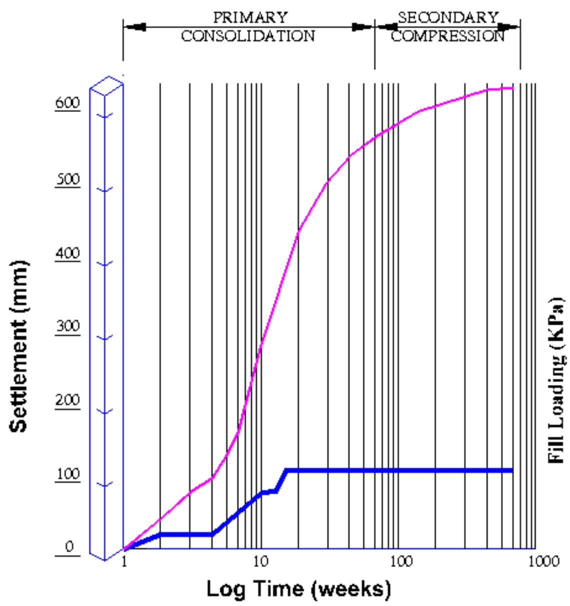


Figure 3. Maximum measured settlement, Joyner Road - Cross Road

MSE/REINFORCED EARTH®

Description

The approach embankments and access ramps rise some 8 metres above natural ground level and incorporate 4 MSE structures.

- The SE and NW vertical retaining walls prevent the N2 on and off ramps and Joyner Road from spilling into neighbouring industrial sites. The SE wall is 270 m long and rises to a total height of 8.3 metres, while the NW wall is 81 metres long, has a maximum height of 2,5 metres where it is surcharged with a positive slope, 5m high.
- The SE and NE Reinforced Earth abutments and retaining walls carry the bridge decks and prevent the fill from spilling onto the N2 freeway. They are respectively 243 m and 84 m long and have a total height of about 8,5 metres at the bridge abutment.

The entire complex of approach ramps and abutments incorporating these Reinforced Earth structures was expected to settle up to 500 mm during the construction period.

The Reinforced Earth construction at Prospecton Interchange has two functions viz:

- to contain the access embankments in order to prevent them spilling into neighbouring industrial sites and the N2 freeway
- to support distribution beams which in turn support the bridge deck

Reinforced Earth construction enables the simultaneous construction of approach embankments, retaining walls and abutments all under the same earthworks operation. A continuity between access embankments, retaining walls and abutments is realised with the same nature and magnitude of settlements. This is particularly useful in the case of poor foundations, as was the case for the Prospecton Interchange, since a common foundation improvement technique could be used.

Settlements of the Reinforced Earth, both during and after construction, occur together with the approach embankments and need not be detrimental to the works. Settlements which take place after placement of the distribution beams must, however, be taken into account. These settlements are of little consequence for simply supported bridge decks provided longitudinal profile and clearance are considered. Continuous decks must however be designed to accommodate post construction distribution beam settlements.

Constituent materials used for Reinforced Earth at Prospecton

Reinforced Earth is a composite material consisting of a frictional backfill reinforced in horizontal layers with reinforcing strips. Cladding elements are connected to the reinforcing strips. At Prospecton Interchange precast concrete cladding elements were used. These elements are designed to accommodate settlements of the backfill and also a differential settlement of up to 1½%. The panels are 140 mm thick and have sizes ranging from 0,8 m^2 to 2,8 m^2. Tie strips are cast into the panels in order to connect them by way of a single bolt in double shear to the reinforcing strips. Figure 4.

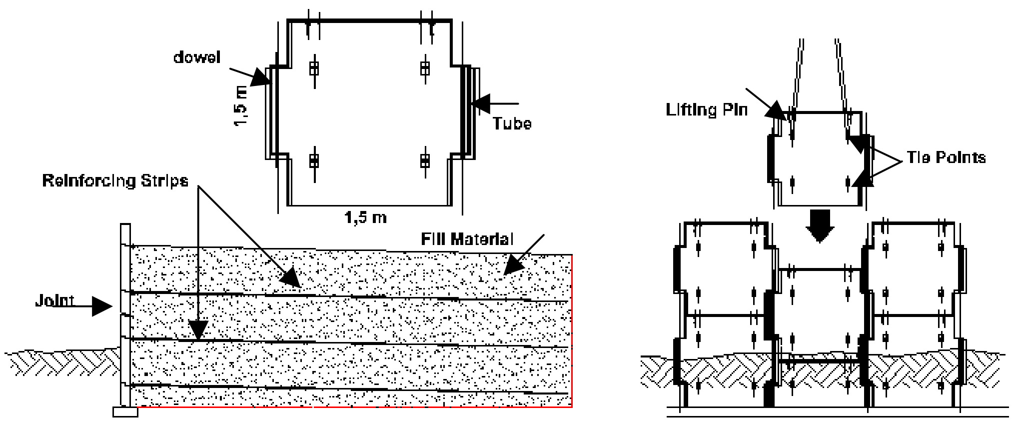


Figure 4. Cruciform panels

The reinforcing strips are made of medium tensile steel with 40 x 4 mm and 50 x 5 mm cross-sectional area. The length of the reinforcing strips varies from 6 m to 10 m. The 50 x 5 mm strips are padded at regular intervals. A hole punched through the thickened pad does not result in any nett cross-sectional loss in area. The strips are hot dip galvanised and are ribbed to improve frictional properties with the backfill.

The Reinforced Earth backfill is defined as the volume contained by the face area of the strips and the length of the reinforcing strips. A Berea Red Sand was used at Prospecton both for the Reinforced Earth backfill and the common backfill. The mechanical properties of this backfill are compared with the standard Reinforced Earth specifications in Figure 5.

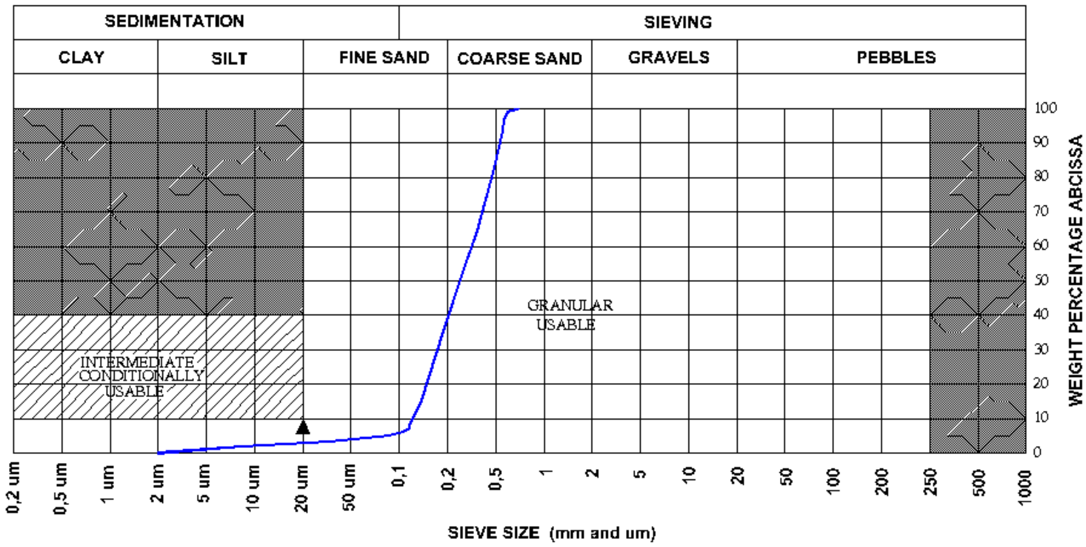


Figure 5. Backfill specification graph for Reinforced Earth®

The design of the Reinforced Earth

The Reinforced Earth structures behave as coherent gravity masses and must be designed for both external and internal stability. The effect of earth pressure from the retained fill must be added to the superimposed loads and the self weight of the Reinforced Earth mass. As a general rule the slope of its resultant becomes steeper as the Reinforced Earth becomes narrower.

The width of the structure, equal to the length of the reinforcing strips is dependent on overall stability and economical considerations since the shortest strips which satisfy sliding requirements of the structure are not always the most economical strip arrangement.

Embedment

A minimum embedment depth is required to prevent the pressure beneath the Reinforced Earth mass in the vicinity of the facing exceeding the allowable bearing capacity of the foundation soil. This embedment also guards against undermining due to future excavations which might take place in front of the mass.

Reinforced Earth abutments - the distribution beam

The conception of the beam is that of a ground slab, but in the case of use for a Reinforced Earth abutment must satisfy certain criteria. The width of the beam must be such that the pressure transferred to the Reinforced Earth mass under the action of permanent loads is less than 150 KPa.

EXTERNAL DESIGN OF THE REINFORCED EARTH STRUCTURES

Sliding on the base of the structure

The pressure exerted by the random backfill on the back of the structure tends to make it slide on the base. In order to verify that the safety against a failure by such sliding is large enough, the mobilizable friction at the base of the Reinforced Earth mass must be larger than the horizontal resultant force from the distribution beam (in the case of the abutments) and the backfill pressure.

Bearing pressure

The bearing pressure at the base of the MSE mass should be less than the allowable bearing capacity.

INTERNAL DESIGN OF THE REINFORCED EARTH STRUCTURES

The vertical stress at particular levels in the Reinforced Earth mass is a function of the external and superimposed loads and the self weight of the fill. This stress is higher closer to the facing and use of the Meyerhoff formula provides a good estimate of the maximum stress at each level in the structure.

In the case of Reinforced Earth bridge abutments the Principle of Superimposition must be applied to combine the stresses derived from the retaining wall conditions with the stresses caused by the bridge loading.

STRESSES FROM BRIDGE LOADING

Distribution

A Boussinesq distribution of stress adequately defines the distribution of stress beneath the bridge distribution beam. This applies to whether considering stress distribution to the rear of the beam or laterally.

As vertical stresses from the various surcharge loadings diffuse with depth, the centre of gravity of the Boussinesq distribution moves away from the wall facing. This movement creates an overturning moment that increases with depth which must be considered in the overall stability of the structure.

Settlement design

The first criterion then was to provide Reinforced Earth retaining walls and bridge abutments with the ability to absorb a total settlement of about 500 mm and a significant differential settlement since the settlements would be approximately proportional to the height of the embankment.

Provision was conservatively made to cater for the possibility of no settlement at all and also for the outside possibility of total settlement up to 1000 mm.

The final elevation of the top of the retaining walls and wing walls was not critical since they were all surcharged with fill spilling from the shoulder breakpoint of the roads. If the structures did not settle then the slope from shoulder breakpoint to the top of the wall would be flatter than if full design settlement occurred, in which case the slope would be 2 horizontal to 1 vertical. Refer to Figure 6.

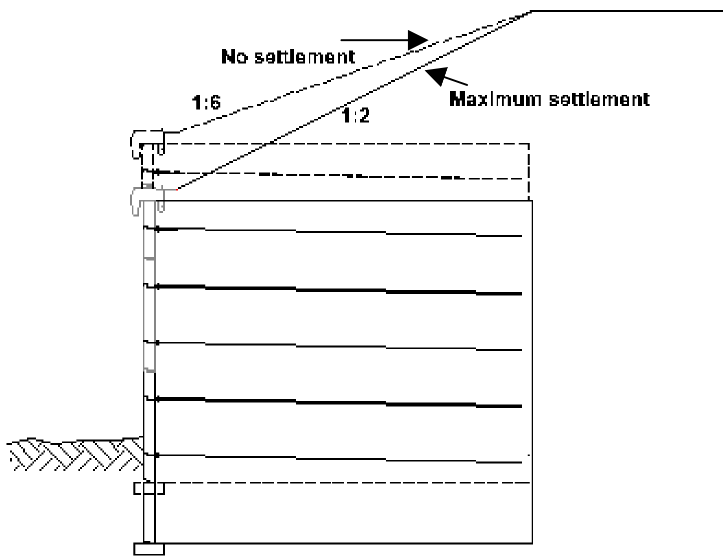


Figure 6.

On the other hand the level of the top of the panels of the abutments was important. The protrusion of the panel in front of the distribution beam and the level of the first reinforcing strip beneath the distribution beam enabled a settlement of between 0 and 200 mm to be accommodated before placement of the distribution beam. These panels were increased in height during the construction process to accommodate a settlement of between 380 mm and 580 mm. Larger settlement could be taken up in the thickness of the distribution beam. Figures 7 and 8 illustrate this provision. Figure 9 shows one of the abutments prior to construction of the distribution beams.

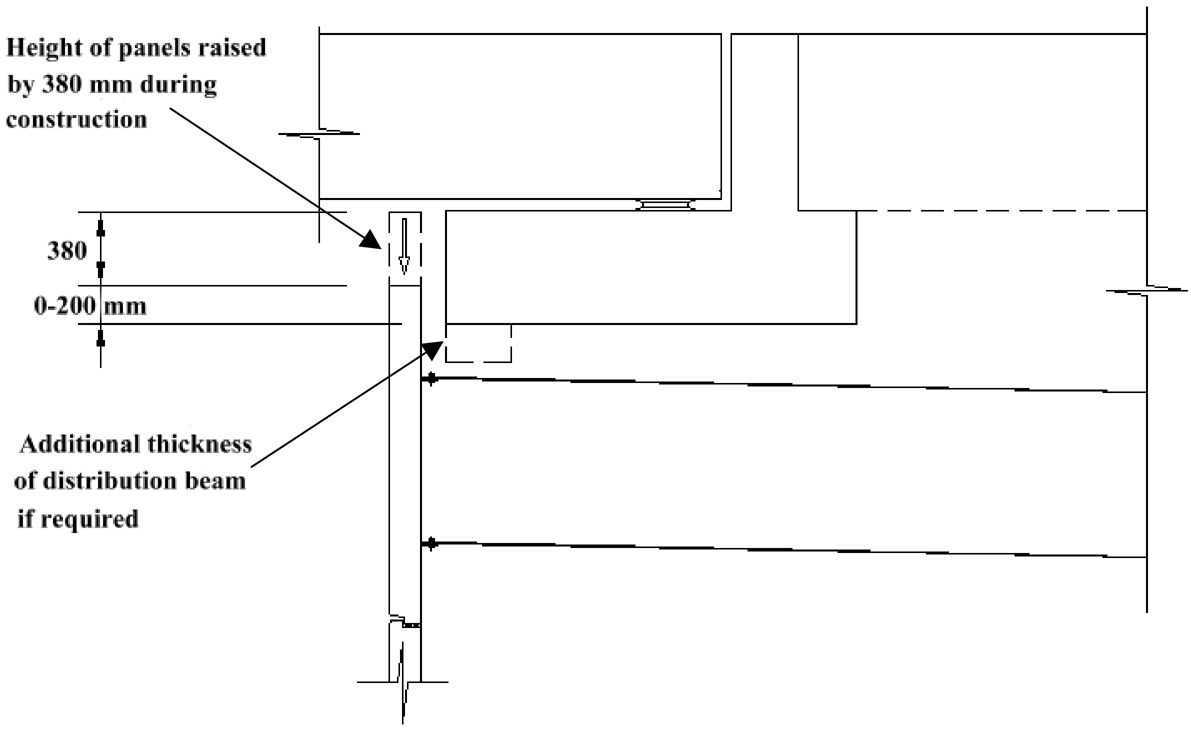


Figure 7.

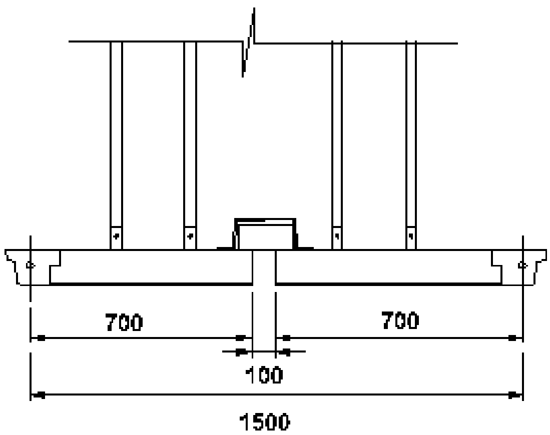


Figure 8. Plan view on cladding at settlement joint

Design for differential settlement

The cladding elements are designed to accommodate settlements of up to 1½%. As a precaution against larger differential settlements, settlement joints were introduced at regular intervals effectively separating one section of wall cladding from another.

Figure 9. An abutment prior to construction of distribution beams

MONITORING

Monitoring requirements

Slope stability analyses indicated safe embankment structures with Factors of Safety in excess of 2 but only once all excess pore water pressures beneath the embankment had dissipated. Slow construction was therefore a very necessary requirement, with measurement of pore water pressures to ensure stable conditions. Monitoring would also be required to ensure no excess lateral movement of the substratum beneath the fills – especially because of the effect on any vertical and raked piles beneath adjacent buildings – and for this vertical inclinometers would be required. Finally, monitoring of the long-term settlement would be required both for technical monitoring and also for contractual arrangements. Last-mentioned achieved by repeatedly adding galvanised rods onto one another with start-off from a round metal base plate. Rods are added as the fill rises during construction. All measuring stations required protection, but even so, three of the thirteen stations were damaged beyond repair during construction.

Monitoring results

Settlement measurements, as measured using survey from stable benchmarks, proved successful in measuring actual settlements as previously alluded to.

Excess pore water pressures, as measured using Casagrande standpipe piezometers, indicated an initial surge in piezometric level at the start of construction. This levelled and stayed static for most of the construction period before diminishing to pre-construction levels toward the end of the nearly 3-year construction period. Not once was there any indication that pore water pressures would rise to a height which would have an affect on the stability of the fill. This can be attributed to the close spacing of the band drains, but more importantly, the slow construction rate in having to construct an MSE wall as opposed to ordinary earthworks.

Inclinometers were installed alongside a building with raked piles on the NE corner of the interchange. A gantry crane with low tolerance for settlement made it imperative that foundations would not be disturbed by the new construction. The maximum movement recorded was 145mm, which, although in excess of the predicted 100mm or less, was not considered excessive. There was no effect on the piled portions of the building although a free-standing store butting on to the main building was so badly damaged it had to be demolished. This had been predicted prior to construction. An interesting element of this aspect of the monitoring was that the maximum horizontal settlement zone adjacent to the highest MSE wall amounted to 7m, viz. practically the same height as the wall.

CONCLUSION

To the authors' knowledge this is the first time MSE, in conjunction with vertical band drains, has been used in South Africa, and shows what can be achieved using innovative, geotechnical engineering via a combination of solutions rather than reliance on one. The 8% difference in predicted and actual settlement is considered acceptable considering the variable nature and unknowns within the substratum beneath the interchange.

The construction team comprised:

Client : South African National Roads Agency (Pty) Ltd
Consultant: Jeffares & Green (Pty) Ltd
Main Contractor: Murray & Roberts Civils (Pty) Ltd
MSE Sub-consultant/Contractor: Reinforced Earth (Pty) Ltd.

All those associated with the project are thanked for their invaluable contributions which saw a project completed on time and within budget. Figure 10 is a view of the completed bridge structure

Figure 10. Completed bridge structure

REFERENCES

Price, GV, NPA Roads Department (1985), Prospecton Interchange Geotechnical Report
Jones, GA, Davies, P (1985), Soft Clays: Problem soils in South Africa. The Civil Engineer