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Case Study: Mount Edgecombe Interchange – investigation, design and construction of a complex geotechnical project

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ABSTRACT: This paper will discuss key observations and lessons learnt from the geotechnical investigation, design and construction for the four level Mt Edgecombe Interchange. The interchange is the largest and most complex yet to be constructed in South Africa and is situated on a weak succession of coastal sands and clays to depths of over 60 m. A significant feature of the project was the geotechnical works. In South Africa, an alarmingly large percentage of ground engineering is undertaken without adequate ground investigation data and minimal site supervision during construction. Furthermore, codes of practice are often ignored, and even non-geotechnical engineers have been known to undertake complex ground engineering projects. It is considered important to bring to the fore projects where the benefits of having adequate investigations, robust codes or client specifications, and meaningful roles for geotechnical engineers, contributed to its ultimate success.

1 INTRODUCTION

The Mt Edgecombe Interchange project comprised the improvement of an existing diamond interchange between the M41 Motorway and N2 Freeway in uMhlanga Durban, to a free flow, four level interchange. The project planning commenced in 2009 and construction was completed in late 2018 for the South African Road Agency SOC Ltd (SANRAL). The project team had to overcome numerous challenges throughout the nearly decade-long project; long since buried and forgotten were the challenging ground conditions, which resulted in significant geotechnical works for this mega interchange.

A number of deficiencies in geotechnical practice are known to exist, and for the Mount Edgecombe project an attempt was made to address some of these. This article aims to highlight these so as to improve project successes and reduce contractual claims and disputes arising from inadequate investigations and poor geotechnical design.

2 PROJECT OVERVIEW

The upgrade included six new and four upgraded bridge structures, of which the main feature was the two free flow upper level ramp viaducts of 443 and 947 metre length. The latter is the longest incrementally launched bridge in South Africa and was incrementally launched from the opposite abutment ends and designed to be stitched in the centre (Concrete Society of Southern Africa 2018).

Technically the project also boasts nine mechanically stabilised earth walls (2100 metres long, up to 16 m high), ground improvement (some 1400 dynamic replacement stone columns and 700 Dynamic Compaction positions), three soil nail retaining walls (950 metres long, up to 7 m high) and more than 11 km of piling (Van As 2018).

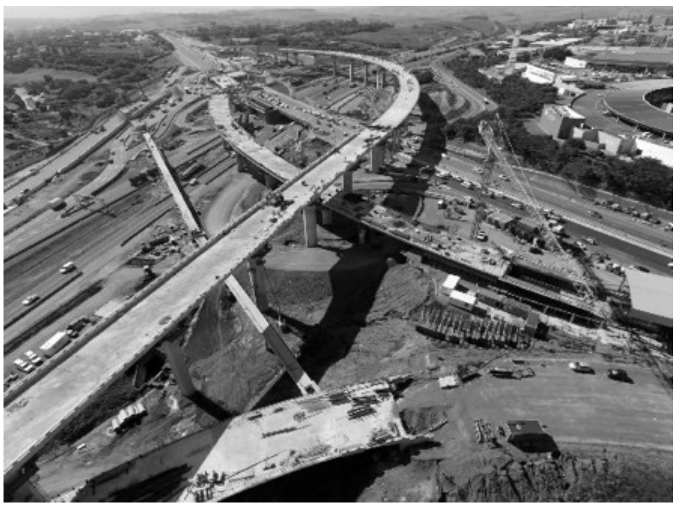

Figure 1. Aerial photo of Mt Edgecombe Interchange in March 2017 with the free flow upper level ramp in centre (SANRAL).

3 GROUND CONDITIONS

The project site is underlain by Quaternary Age (+- 30 Ma years) coastal aeolian sand deposits of the Berea Formation which gives the area its characteristic rolling topography. In geological time, several phases of uplift, erosion and deposition have created complex landforms with the Berea underlain by soft rock Sandstone, Siltstone and Shale of the Ecca Group with occasional hard rock Dolerite.

The Berea Red Sands are typically medium dense clayey fine sands with the thickness varying, according to both the surface topography and the underlying bedrock profile, both of which undulate. Consequently, this was encountered to depths of between 40 and 60m. Additionally, the properties of the Berea Formation vary both laterally and vertically.

A second feature of the local geology is the presence of a stream which flows roughly across the centre of the interchange. Ground conditions in the vicinity of the stream showed greater variability than the rest of the site with an alternating succession of alluvial deposits intercalated within the Berea formation to depth. These paleo alluvial deposits include very soft clays, stiff clays and gravel or bolder horizons. Groundwater was typically encountered at greater than 25m depth except in the vicinity of the stream where the water table was shallow (Purchase & Van der Merwe 2017).

4 PUBLIC VS. PRIVATE SECTOR PROJECTS; THE APPLICATION OF CODES

Experience tells us that the largest source of claims and disputes in construction are attributed to the ground works. It is thus surprising that inadequate investigations and designs are so prevalent in the industry particularly on private sector projects. The contractor is given significant advantage in such instances; he has the obvious advantage of hindsight and is effectively given an open chequebook to claim as a result of variable or unknown ground conditions even when these are actually good. Notwithstanding, (even in this scenario) unknown ground conditions disadvantage the project and all project participants including contractor.

In South Africa, codes and standards are not mandatory unless referenced in legislation or required by contract. However public infrastructure projects, especially large and complex ones, are underpinned in public law and subject to intense public scrutiny. Furthermore clients, as is the case with SANRAL, can be more technically astute and not only mandate the use of codes but also have their own specifications and guidelines which may even be more onerous.

Public sector clients may also have a more holistic view on projects, in terms of maintenance and safety risks, due to their role in the ongoing management of the infrastructure. In this case their specifications and/or the codes become prescriptive minimum standards. Being prescriptive, they are more likely to be used by the client in the assessment of professional liability when contractual disputes arise or when failures or poor performance occurs.

5 INVESTIGATION CODES

An obvious problem with large structures is that uniqueness and size make it impossible to dictate quantitively the scope of investigations, designs or desired outcomes in the same way as say a housing development. The design approach is therefore “engineer-dependent” and it has become evident that there is a varying perception between engineers of what is acceptable. It is therefore clear that minimum requirements for investigation, design and analysis are required.

The SAICE Code of Practice for Site Investigations (SAICE 2010) sets out to provide broad guidelines for investigations. This code is however, neither prescriptive nor compulsory with the result being that it is for the most part not followed especially for private sector projects.

Contrary to this, the project requirements were dictated by SANRAL’s Pavement Engineering Manual and design guide (SANRAL 2013, updated from 2004) and its tender requirements which are more prescriptive. This is particularly with regards to the quantum of investigation (it for example specifies the number of boreholes per bridge pier and per kilometre of retaining wall), what the outcome of the investigations must be, and whom may oversee the investigations and design (a Professionally Registered Geotechnical Engineer who forms part of the client’s key design personnel). This consequently results in more extensive investigations, with the involvement of the Geotechnical Engineer also ensuring that geotechnical solutions are not overlooked in lieu of more costly structural solutions.

6 INVESTIGATION STAGING

It is generally accepted that investigations should be undertaken in stages so as to inform, at an appropriate level of detail, the design as it progresses through its design phases. Attempting to undertake the investigation of a project in a single phase, can result in a number of problems, including using the wrong investigative techniques if ground conditions are found to be different, which could in turn delay the project. In another example, a common characteristic of private sector projects is that data from investigations which are cursory in nature, possibly with a different development in mind, often becomes the only data on which designs are based.

What is appropriate at any particular project stage would depend on the complexity of the project and geotechnical conditions. Investigations at Mt Edgecombe were undertaken in two phases. The initial investigation was limited to just three boreholes and some CPT-u probing and was undertaken during the preliminary design stage. These showed the thickness of the Berea to be locally substantially deeper than the expected thickness of less than 30m. Information obtained during this early stage, drove preliminary design configurations such as length of viaducts and bridges, use of larger span configurations, and likely foundation type. Crucially, these investigations aided in the planning of subsequent detailed investigations and the specific design constraints to be addressed.

During the detailed design stage more extensive subsurface investigations took place on site, with at least one borehole undertaken at each pier position (over 100 boreholes). Field testing comprised DPSHs, SPTs, and Pressuremeter tests while laboratory tests included Oedometers, Triaxials, UCS, PLI, full grading analyses, indicator and CBR testing. Geo-physical testing was limited to CSW testing which was restricted to selected positions which were not affected by various services, traffic and "noise".

7 VALIDATION OF INVESTIGATION DATA

There are various approaches to conducting a site investigation, depending on, amongst other, the purpose and extent of the investigation. However, for an investigation to be successful, it is important that the correct methods are used and that results are interpreted correctly. The methods appropriate to a particular ground profile cannot necessarily be guaranteed upfront and it is important to have methods of validating the investigation data during the investigation and design stage; as it may be too late to do so during the construction stage.

There are as an example, many known problems with SPT testing during drilling particularly in sand profiles below the water table. Additionally, average energy ratios at which Standard Penetration Test N-values are measured are also not entirely known. Contrary to concrete where properties are controlled in a laboratory scenario, these penetrometer results vary significantly with increasing depth, resulting in a large scatter with concomitant large co-efficients of variation. This would ultimately lead to erroneous estimates of parameters in design (Van der Merwe 2015).

Laboratory testing would be one of the obvious ways of validating in-situ testing data. However, in South Africa, variation in test results and quality are problematic (Jacobsz & Day 2008). Thus, there is an additional requirement to validate at multiple stages during the investigation, data being gathered from the drilling and testing.

So as a supplementary means, to validate the SPT testing, Pressuremeter testing was undertaken at selected boreholes. In South Africa these tests are expensive as few contractors have the equipment and the testing is not well understood by local geotechnical engineers. Fewer than 30 tests were undertaken but they were used together with laboratory testing to validate the SPT testing. SPT tests (some 1000 of them) were thus extensively used as the design basis investigation data once validated.

8 PILE DESIGN – SETTLEMENTS GOVERN

The exceptionally high vertical and horizontal loads and moments resulting from launching loads, bridge piers up to 26 m in height and individual span lengths of up to 65 m, combined with strict settlement criteria, necessitated careful consideration of the foundation design and geotechnical risks.

The ultimate load capable of being carried by a piled foundation represents the limit state for load carrying capacity. However, capacity at this ultimate state is of little consequence if the integrity of the structure founded on the piles is compromised due to excessive settlements. Thus load-settlement characteristics of the pile prior to the ultimate limit state being reached were more important. This so-called serviceability state usually represents working loads imposed on the pile caps.

The bridge engineer normally sets design criterion in terms of maximum total and differential settlements. Although on a project such as Mount Edgecombe this is more collaborative as it is also driven by the cost effectiveness of the foundation system to achieve such an outcome.

The majority of the structures were founded using Screwed-in-Casing Augered Piles (SICAP) with a diameter of 900 mm. These piles were installed to depths ranging from 15 to 40 metres and were designed as full friction piles with limited end-bearing capacity. SICAP piles were chosen for their versatility with regards to depth range and soil conditions which ultimately provided for the control of differential settlement between piers and ground variability.

The stringent settlement criteria were in the order of a total or pile cap settlement of 20mm and differential settlements between piers of 10mm. Pile cap deflections were predicted with methods described by Poulos & Davis (1980).

That the two decks would meet up within dimension tolerances required intricate design and attention to detail with close collaboration between the geotechnical and bridge engineers. It is key to appreciate that not only did the ground profile vary between piers, but significant variance occurred as a result of varying working loads. Thus, each pile group for each pier needed to be designed considering not only the ground profile but also the load and, significant variation frequently occurred between adjacent piers as a result of the variation of both. Control of differential settlement between piers, was thus an iterative process of adjusting pile designs of individual piers until the differentials converged within tolerance.

9 DESIGN VALIDATION – SCALE TESTING

In South Africa pile load testing is undertaken far less frequently than required under codes or compared to international practice. In order to validate the design three Osterberg Loadtests were undertaken. The O-cell testing was undertaken at the beginning of the project and it was a condition of the contract that the tests be conducted as a means of validating the design before the contractor was allowed to proceed with the project piles.

The Osterberg Loadtest was developed by Jorj Osterberg in the late 1980's and consists of a specially-designed negligible internal friction hydraulic jack device capable of exerting large loads at high internal pressures. The jack is placed at a specific depth in the pile where it is considered that upward capacity equates to downward capacity to eliminate the need for kentledge as per a normal top-down static pile load test. Telltales are used to establish movement at the jack and toe and strain gauges are placed along the shaft to establish development of side shear capacity. The O-cell testing proved valuable validation of the design assumptions; the results showed that the capacity derived from end-bearing was generally half the predicted value (Van der Merwe & Pequenino 2014) and the results further confirmed conditions in the stream to be substantially poorer. The reduction of end-bearing was a result of the pile installation methodology (discussed below). The halving of the end capacity was also verified by SPT tests which were undertaken through the Cross Hole Sonic Logging tubes on some of the piles on site, with SPT-N values up to half of that recorded during the investigations after pile installation.

10 PILE INSTALLATION CHALLENGES

The SICAP piles are installed, making use of a temporary steel casing being driven into the ground by means of a hydraulic oscillator, to the required piling depth. As the casing is driven down, the material inside the casing is removed using an auger drill. Once the design depth is reached, the reinforcing cage is lowered into the temporary casing. Concrete is tremied from the bottom up and the temporary casing is gradually removed as the pile is filled with concrete (Van der Merwe 2015).

SICAP piles are more commonly associated with end-bearing piles which made its application on this project challenging (Van der Merwe & Pequenino 2014). As the many of the piles were installed below the water table, there was a constant difference in head between the in situ material outside the temporary casing and the void being augered within the casing. The in situ material, being of poor quality and saturated, meant that this "fluid" material would boil into the piling casing not only activating the in situ material around the piles but also rendering it impossible to keep the pile clean and excavated. The influence of "boiling" becomes more problematic with in- creasing pile length and with a larger head difference between the water level on the inside of the casing and that outside the casing (see Fig. 2).

Although designed as friction piles, this loss of the in situ state and integrity of the material surrounding the pile toe could prove to be detrimental to side shear and the quality of the piles. These problems were mitigated through the further downgrading of the end-bearing capacity whilst the contractor modified and controlled the soil "plug" left inside the casing after evaluating actual conditions at each pile position.

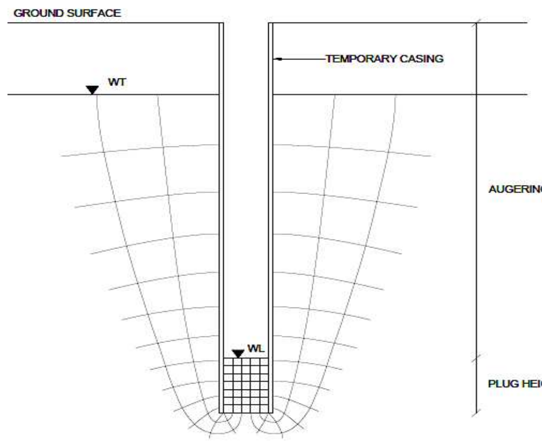

Figure 2. Illustration of soil plug to prevent boiling into pile

11 PILE END-BEARING REDUCTION

Fairly large safety factors (FoS) normally apply to calculated ultimate end-bearing capacity, predominantely due to the large load-deflection required in end bearing to derive full capacity. An FoS of 1.5 is normally applied to the shaft friction of an auger pile founded in cohesionless material to limit settlements. To correspond to an FoS = 1.5 on side shear at similar settlements an FoS on end-bearing would normally have to be in the order of 3 to 4.

However, this is dependent on various factors and Van der Merwe (2015), stated that one should rather review load-settlement curves derived and base limiting loads on a specific corresponding settlement value as the serviceability condition is normally the governing factor on settlement-sensitive structures.

Given the above, the reduction (halving) of end-bearing capacity actually had limited result on the project; it amounted to less than 5 % increase in pile length quantities, as the piles had originally been designed with limited end-bearing in mind.

12 PILE INTEGRITY

This test is based on the principle of density and the way different densities conduct waves. By interpolating the travel times of the sonic pulse one can very easily determine whether the concrete in the pile is of sound quality or not (Van der Merwe 2015).

13 MSEW

Mechanically Stabilised Earth Walls (MSEWs) were utilised for a number of the approach ramps as a result of economic considerations and space constraints. There were nine MSEWs varying from 3 to 16 m high, with four of the largest walls constructed as back to back. At 16m high, these are some of the highest walls in South Africa and presented the designers with some unique challenges (Purchase & Van der Merwe 2017). The first being geotechnical and the second contractual.

As a result of the poor geotechnical conditions, high fills like these on small footprints are at high risk of excessive settlement. The main concern with the settlement is the fact these fills are the approaches to the two highest viaducts and differential settlement between the structure and the approach could result in transfer of loads to the bridge which would have catastrophic consequences. To solve this problem the in situ founding material would have to be significantly improved prior to constructing the high fills and bridge abutments. The improvement was done by means of Dynamic Compaction (DC) and the installation of Dynamic Replacement Stone Columns (SC). This process involves a 13-ton weight being dropped from 19 metres high. Where the weight is dropped a crater is formed in the in situ material. This crater is then filled with a suitable dump rock and the process is repeated until refusal is reached. After the stone column has reached refusal, selected plate load tests were carried out to confirm that the required stiffness and bearing capacity has been reached (Van der Merwe & Purchase 2017a).

Contractually a known disadvantage of the MSEW is the shared design responsibility between the design consultant, supplier and contractor (Van der Merwe et al. 2017b). This results from the design aspect being dealt with somewhat ambiguously in the client's Standard Specification (COLTO) and most believing that these walls are specified as "design-and-build", when actually the licence holder has very limited responsibility and none of the contractual mechanisms enabling a true design-and-build approach effectively being in place.

Furthermore, MSE structures result in highly complex interactions between internal and external design factors, and between the soils and structural members making up the MSE system. These are thus complex geotechnical engineering structures that must be designed by geotechnical engineers as part of his design or project responsibilities (Van der Merwe et al. 2017b).

Figure 3. Back-to-back MSEW at one of the Viaduct Abutments

For Mount Edgecombe an initial detailed design was undertaken using typical and varying parameters for the MSE structure to account for different systems which could potentially be used. This required the selection of a number of performance criteria against which various MSE technologies could be evaluated, and a cost benefit and optimisation exercise was undertaken. This optimisation duly considered various influencing elements - such as ground improvement on overall stability and settlement, and the availability and selection of fill material on internal stability. Although performance criteria were set at tender stage, these governed the range of several design parameters important to the design.

The finalisation of the design could only occur during construction, as this was dependent on having discrete design values, which were only known once the final MSE system was selected (i.e. successful tender was known). This stage also included peer review and collaboration on the design with the supplier on their internal stability designs and compliance with the specification. Similarly, the supplier was able to review the geotechnical designer's external stability designs. This ensured that design assumptions and interfaces were understood.

The geotechnical design engineer thus took ultimate responsibility for MSEW design; this included ground improvement works, overall stability, specification of materials used as backfill behind walls and checking of internal stability.

14 PEER REVIEW

Given the complexity and significance of the project the client appointed two independent consulting firms to conduct peer reviews of the (structural) design of each of the respective ramp viaducts. The peer review provides the client with some assurance that designs are consistent with best practice and that technical innovation is considered together with any risk.

However, the peer review process brought to the fore one of the major challenges in current geotechnical engineering practice; that is, the lack of and inconsistent use of geotechnical design codes or design methodologies in South Africa.

In comparison to the bridge engineers who, even for a unique project such as Mount Edgecombe, are bound to the application of just a handful of design codes and methodologies. Geotechnical engineers use a broad range of methodologies, varying factors of safety at various stages of the design and analysis, and different interpretations of the same investigation data. This can make the peer review process contentious, as the point of departure can be very divergent.

15 THE TENDER STAGE

One of the contradictions in civil engineering occurs at tender stage. On the one hand the designer has developed a design which is sufficiently robust and cautious so as to overcome the risks he has evaluated. On the other hand, the contractor makes his own assessment of these risks and normally assumes a less cautionary approach during the short tender period, to increase his chances of being awarded the project.

As with the geotechnical peer review process, evaluating alternative tenders, can become challenging due the different results gained from different theories and assumptions.

16 CONCLUSION

A significant feature of the Mt Edgecombe Interchange project was the geotechnical works undertaken. A number of lessons were learnt over the decade long project; the authors regard the most significant being, the need for more comprehensive and prescriptive South African geotechnical design and investigation code(s), and greater attention paid to geotechnical aspects at the design stage.

The use of these codes must be mandated by industry or enforced by clients. Investigation and testing requirements must be defined by the geotechnical design engineer and peer review should be a requirement for major structures to assure the safety of the public and economically optimised/viable design.

Robust planning/design and the holistic consideration of geotechnical risks in the early design has significant benefits to the client. This improves project success and reduces contractual claims. Though it adds a small amount of time and cost to the early portion of a project, these costs are minor compared to the alternative of the costs and effort required to make changes at a later stage in the project which typifies private sector projects in South Africa.

By extension, this includes making the contractor responsible for geotechnical design at the later construction stage. Contractors are not in a position to adapt the project constraints once these have been fixed and the project is due for construction.

SANRAL's specification ("code") for minimum investigation and design requirements, including the design by consultant, need for peer review and specification of key personnel as part of the design ultimately led to the success of the project.

17 PROJECT PARTICIPANTS

The Mt Edgecombe Interchange was funded by the South African National Roads Agency SOC Limited, in partnership with KwaZulu-Natal Department of Transport and Ethekwini Municipality.

SMEC South Africa (Pty) Ltd were the design engineers. The contractor, CMC Di Ravenna South Africa was awarded the contract in February 2013 for an award value of approximately R715 million (excluding VAT and CPA) (or US$75million, 2013) (Van As 2018). KellerFranki undertook the geotechnical works including piling, ground improvement and lateral support works, MSEW panels and grids (internal elements) were supplied by Maccaferri.

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