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# Upstream Raise Tailings Dams from a Low Risk Appetite Perspective: A Case Study

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## Abstract

This paper explores some controversy that was experienced when dealing with an international investor with an exceptionally risk averse paradigm on the design of an upstream raise tailings storage facility to be constructed in Limpopo, South Africa. The primary points of contention will be briefly discussed, with a single point regarding liquefaction under seismic loading being explored in more detail. Some of the proposed resolutions to the liquefaction concern will be presented and critically discussed, including the option that will most likely be the final resolution.

***Keywords:*** *upstream raising; liquefaction; mine tailings*

## 1 Introduction

The design of a tailings storage facility (TSF) was undertaken for a greenfield development in the Limpopo region of South Africa. The design was conducted according to established norms and standard practice adopted in the South African mining environment, and the TSF was designed as a lined upstream raise facility, implementing a combination of established deposition methodologies consisting of daywall paddocking and spigotting.

The foreign investor with a minority stake in the project voiced some concerns they had regarding the design. The investor originated from Japan, and their primary concern was based on the fact that design and implementation of upstream raise TSFs have been banned in Japan since 1982, due to failures of upstream dams in the 1970s (usually leading to liquefaction). This argument formed the core of their resistance to the proposed design, and resulted in multiple discussions exploring possible solutions until all parties were satisfied.

The main concerns raised with the TSF design will be briefly listed and discussed, with the concern on seismic liquefaction and upstream raising being presented in more detail.

## 2 The Major Concerns

Here, the major issues of contention on the project will be presented and discussed in their own individual subsections.

### 2.1 Tailings Particle Size Distribution

There are various tailings characteristics that influence the deposition methodology implemented for each TSF. The particle size distribution (PSD) of the tailings, along with other tailings characteristics such as solids concentration, etc., will ultimately determine the practicable method of deposition for the TSF. Initial testing of tailings received from bench scale samples using laser diffraction indicated that cyclone deposition would be possible. Only later in the project when the client made the PSDs available from wet and dry sieving was a discrepancy identified, which was a large difference in the fines content of the tailings (defined as the percentage passing 75 micron). Due to the amount of wet and dry sieving results indicating a consistent outcome and the fact that sieve analysis is mostly considered to be the benchmark for determining PSDs, the TSF design was subsequently revised to include a deposition methodology consisting of a combination of daywall paddocking and spigotting.

This alteration resulted in an increase to the starter embankment height to ensure that the rate of rise could be reduced to the required level in order to initiate the hybrid paddocking method. Revising the deposition also affected the tailings delivery pipeline layout, as discharge points within the paddock as well as basin would need to be allowed for, whereas previously only a single offtake line would have been sufficient to feed the cyclones.

### 2.2 Barrier Considerations and Related Intricacies

In order to limit the exposure of the contaminated material to the receiving environment, The TSF was designed as a lined facility, in accordance with the National Environmental Waste Act (Act 59 of 2008) (NEMWA, 2008), with specific reference to the National Norms and Standards for Waste Disposal (Regulation R635 and R36). The tailings waste stream was classified as a Type 3 waste which requires a Class C landfill barrier system, or a system of equivalent performance.

The prescribed practice is environmentally responsible and in accordance with leading practice where environmentally sensitive areas prevail. It also has some benefits such as a higher return to the plant due to reduction in seepage losses. The barrier, however, poses additional stability considerations which will need to be addressed. The additional considerations for interface stability under saturated conditions plays a significant role in the overall stability, especially considering translational failures. Further to the stability and durability of the barrier system, cognisance should be taken of creep.

Another critical disadvantage due to the barrier system is its effect on the consolidation of the tailings mass, which will have an effect on the in-situ density and associated void ratio. The effects of desiccation, however, do assist in the densification of the tailings in the outer desaturated shell.

### 2.3 Seismicity and Upstream Raising

The primary issue that will be discussed is the issue of liquefaction of the tailings under seismic loading. This could result in a flow that could wreak havoc downstream of the TSF. This issue was founded in that fact that during dynamic loading of the embankment material, and under saturated conditions, the embankment may fail if the in-situ state of the material is contractive. This may even occur without dynamic loading should a triggering mechanism be present.

As this was the main area of concern during the project, the subsections to follow will focus on this subject in more detail.

## 3 Contributors to the Issue of Seismicity

Several factors contributed to a conservative approach to the assessment of seismicity such as conservative project-related reports on the projection of extreme seismic events, the current international climate due to recent failures, and the client's risk appetite. These issues are explored separately in individual subsections.

### 3.1 Japanese Design Code

A primary reason for opposing the upstream raise design was due to the fact that upstream raise TSFs were banned from practice in Japan due to poor performance during seismic loading in previously existing facilities. Aside from this, however, there were some additional grievances with the pseudo-static analysis performed.

The pseudo-static analysis that was conducted was done so in accordance with guidelines as proposed by Hynes-Griffin & Franklin (1984), and a primary assumption in the referenced text is that a factor of safety of 1.0 or higher is satisfactory (which further assumes maximum deformations of 1 m, and is related to the reduction factor stipulated for accelerations). This conflicts with the Japanese design code, which requires a factor of safety of 1.2 or higher in a pseudo-static analysis. Naturally this fact was contested, as the resulting factor of safety was exactly 1.0 (after re-analysis with a 10 000 year return period peak ground acceleration (PGA)). This will be dealt with in more detail in Section 3.3.

Another issue was the fact that a 475 year return period PGA was used for the initial analysis, whereas the Japanese design codes required a return period of 10 000 years. Most international design codes require a 10 000 year return period for seismic analysis for high hazard facilities (those that can do significant damage), and as such our methodology was altered to adopt this practice. The PGA for the maximum credible earthquake (MCE) that was projected was 0.42 *g*, which is discussed in further detail in Section 3.3.

### 3.2 Recent Major Catastrophes

Preceding the period during which the TSF was designed, a few widely-covered international catastrophes occurred, which set many investors on high-alert with regards to safe design. The three most influential events were:

- Dahegou Village, a red mud TSF that engulfed a village in 2016;
- Bento Rodrigues (Samarco), an iron ore TSF that caused 17 deaths; and
- Mount Polley, a copper and gold TSF, which polluted Polley Lake.

These unfortunate incidents all had significant social and environmental impacts. No company would wish to deal with an occurrence like this, especially considering the downstream environment of the proposed TSF (discussed in Section 3.4).

As professionals, and fellow humans, we should endeavor to ensure that heed is taken from these tragic and costly lessons.

### 3.3 Seismic Hazard Assessment Report

Another problem encountered was a seismic hazard assessment (SHA) report that had been compiled for the TSF site. The report cited a PGA of 0.16 $g$ for a 475 year return period. Interpreting the data for a 10 000 year return period PGA produced a value of 0.42 $g$. This is an extremely high value, and indeed, even the Japanese design code cites a PGA of 0.15 $g$ for a 'strong earthquake area in Japan.' The mean uniform hazard spectrum as determined for a 475 year return period is reproduced in Figure .

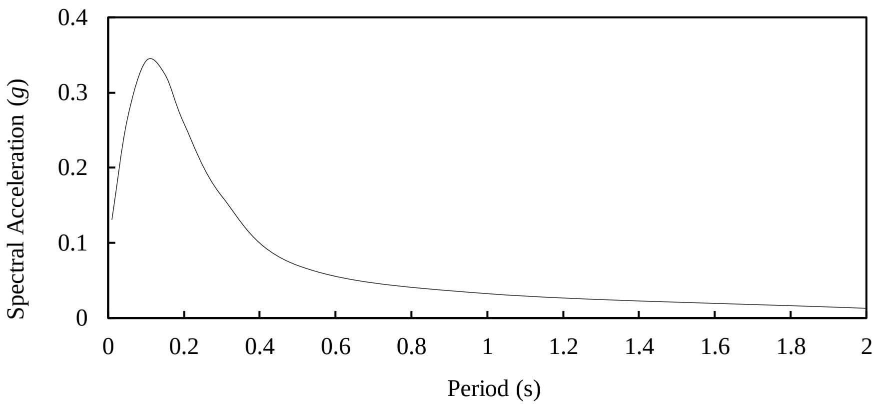


Figure 1. Mean Uniform Hazard Spectrum for a 475 year return period

Internal and external review of SHA report bore similar outcomes: the PGA values as presented in the SHA report reflected activities that were too high for the area in question, and consequently the PGA values provided were conservatively high, and not representative of reality.

### 3.4 Exposed Population

A major concern, should the TSF ever fail, is the affected downstream area. The TSF is located adjacent to a high density informal settlement, which is directly downstream of the impoundment embankment. Over 250 dwellings exist downstream of the TSF, and should the TSF fail without warning in the night, the resulting event could be truly tragic and catastrophic.

With reference to the topography and habitat, environmental damage would be limited, as there are no streams and no significant and sensitive environmental receptors.

### 3.5 Client's Investor's Risk Appetite

The client's investor's risk appetite was limited, due to previously being exposed to a TSF which failed partially due to seismic loading. The investor's tailings management background is also influenced by the conditions experienced in Japan, such as net rainfall conditions (annual rainfall exceeding annual evaporation), snow fall, and, of course, high seismic activity. Bearing this in mind, the investor was against the construction of an upstream raise TSF, due to the fact that an upstream raise TSF under such climatic and seismic conditions as they are accustomed to is most likely a high risk undertaking.

## 4 Solutions Explored

Several resolutions to mitigate the occurrence of potential liquefaction were assessed, ranging from the mundane to the outright experimental. The most widely-discussed resolutions are explored in subsequent individual subsections, including the solution that was ultimately agreed upon by all parties.

### 4.1 Downstream Raising

One of the initial, and probably the most predictable, solutions was simply to construct the TSF as a downstream raise as opposed to an upstream raise using rock fill. The logic behind this is that there would be a greater coarse barrier surrounding the fine tailings.

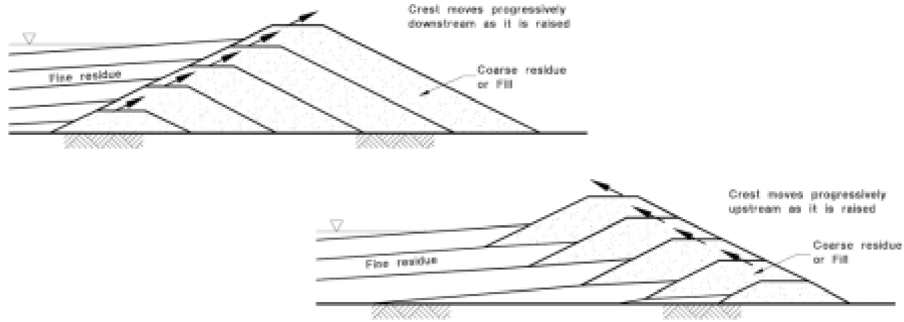


Figure 2. Depiction of an upstream (top) and downstream (bottom) raise.
(After Chamber of Mines South Africa, 1986)

The problem with this option was that, although the coarser material fraction did indeed possess more desirable material properties (such as a higher friction angle for drained conditions), the solution would not alter the susceptibility to liquefaction of the tailings. The zone of influence that would have been derived from a dam break analysis would have been the same or similar to that that would be derived for an upstream raised facility

Aside from this, two other glaring, limiting factors of this approach were the volumes of coarse material required for such a design, as well as the high cost associated with downstream raising. The material requirement would have increased sixtyfold from 249 000 $m^3$ to 15 000 000 $m^3$. The cost of the downstream raise TSF tripled the expected price to R 1 500 000 000, which would be economically unviable.

## 4.2 Dry Stacking

Another more traditional option explored was to dewater the tailings and place it at a lower water content that would facilitate compaction. This would eliminate the potential for liquefaction as the material would have very little to no water in it. This practice is not commonly used, with around 30 filtered tailings facilities existing around the world in 2010 (Davies et al. 2010).

The dewatered tailings are referred to as ‘cake’ and placing them requires alternate methods to pumping. Usually conveyors are used to transport the material before being spread and compacted as a tailing deposit. There are some advantages to this method, such as being suitable for areas of high seismic activity, and saving water.

The major problem with this method was again the costs involved, and the estimated capital requirement was R 2 010 000 000.

## 4.3 Concrete Reinforcement

A rather interesting, yet untested in practice, solution was suggested by the investors. This involved construction of concrete or soil cement columns in the outer perimeter over 25 % of the embankment footprint, which would supposedly increase the apparent cohesion of the embankments tailings to 2 000 kPa. To the best of the knowledge of the authors, this method has never been attempted in tailings designs before.

Needless to say, this suggestion was not taken to detailed design. A preliminary cost calculation from our side indicated that this solution would be economically unfeasible.

## 4.4 Cement Stabilisation

The introduction of cement into the tailings was yet another intriguing solution suggested by the investors. Theoretically, this would cement the outer layer of the TSF and would prevent liquefaction from occurring. There were several issues that were raised regarding this suggestion, with some of them being:

- Physically mixing the cement into the tailings and the associated cost of processes and equipment required to achieve this; and
- Interaction behavior of a stiff paddock over compressible contained tailings.

This potential solution was also rejected, due to the above-mentioned reasons, as well as the cost implications thereof.

## 4.5 Downstream Containment Barrier and Operational Changes

This solution involved the construction of a secondary embankment downstream of the TSF that would act as a barrier to arrest the flow in the unlikely event of a containment breach. It would need to be large enough to contain any outflow from a failure incident, which would also include the ‘wave’ that would form from the tailings flow.

In conjunction to the downstream containment barrier, some operational and design changes were suggested by the investors. These changes included the following:

- Widening of the exterior coarse layer wall to a minimum thickness of 50 m, from an original design thickness of 40 m;
- Shifting the operational pool of the TSF further inward to a minimum distance of 300 m away from the crest; and
- Shifting of the underdrainage system to draw down the phreatic surface before reaching the coarse outer shell.

This option would require an amount of 680 000 $m^3$ of additional material, and was estimated at an additional R 50 000 000 to construct.

The downstream containment barrier was accepted as the final solution.

# 5 Prominent International Regulations

During the course of the design, reference was made to international guidelines on design and analysis of storage facilities. Most prominently among these were the guidelines as stipulated by the Australian National Committee on Large Dams (ANCOLD) and the Canadian Dam Association (CDA). Some comparisons between the local regulations and the two mentioned international guidelines are provided in Table 1.

Table 1. Comparison of local and international guidelines

| **Aspect** | **SANS 10286** | **ANCOLD** | **CDA** |
|---|---|---|---|
| Zone of influence | Area dependent on the height of the TSF | Dam break analysis | Dam break analysis based on probabilities |
| Stability analysis | Overall instability<br>Local instability<br>Surface erosion<br>Deformation** | Steady state seepage<br>Rapid draw down<br>Earthquake*<br>Construction conditions | Slope stability based on probabilities of occurrence |

*Pseudo-static a screening tool only. Post-liquefaction strength.
**High hazard residue deposits only

From the preceding table, it can be seen that dam safety requirements vary quite significantly, depending on the referenced publication.

## 6 Discussion and Conclusion

Based on some of the problems experienced within this project, there are several aspects that could have been navigated more readily if there was some framework or previous work in place to ease the process. Some of the points that we feel should be looked at, in order to give comfort to international investors on large tailings or dam projects within South Africa, include:

- An improved framework for establishing the zone of influence of impoundment facilities; and
- A nation-wide, detailed probabilistic seismic hazard assessment to define the appropriate seismic loading for various recurrence intervals.

An improved or updated understanding of the national seismic environment in the country would prove invaluable for initial valuations of potential sites for TSFs. Further to this, it would prove extremely useful if a database of the characteristics of existing TSFs in terms of piezocone testing with liquefaction assessment in mind would be compiled through field investigation. This would allow designers of TSFs to have a reference of standard values of materials for analysis purposes, specifically for addressing the assessment of liquefaction.

It should also be noted that liquefaction analysis is not necessarily a requirement for every facility, but that the probability of liquefaction occurring should be assessed, which should be coupled to the probability of a liquefaction flow failure manifesting. Impoundment facilities should be designed such that the potential for liquefaction to occur is reduced, such as by introducing an ‘unsaturated prism’ around the perimeter of the facility in order to contain any tailings that may liquefy in the interior.

Liquefaction has become an increasingly discussed issue in the international environment, and South Africa should endeavor to keep abreast with international best practice guidelines.

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# Remediation of Shallow Undermined Ground beneath a Pipeline

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## Abstract

A pipeline was constructed over a historically undermined section of ground. Subsequent to the installation of the pipeline, sinkholes and subsidence became noticeable in close proximity to the pipeline with some sinkholes developing directly below the pipeline. Jones & Wagener were appointed to undertake an investigation to determine the properties of the mined-out cavity and the condition of the overburden to evaluate the risk posed to the pipeline. Based on the investigation findings, a design for backfilling the mining cavity was developed and implemented. During the implementation phase, the properties of the mining cavity were different in comparison to what was found during the investigation. The aim of this paper is to present the investigation, design and implementation phases carried out to remediate a section of shallow undermined ground traversed by the pipeline as a case study. This paper documents the process followed to determine the different conditions and measures suggested to successfully backfill the mining cavity to decrease the risk of damage to the pipeline, due to subsidence or sinkhole formation, to an acceptable level.

***Keywords:*** *Undermining, remediation, sinkhole, subsidence, risk, pipeline*

*At the request of the Client, all references to the details of the pipeline and parties involved throughout the project are omitted.*

## 1 Background

The pipeline was constructed in the early 2000s in the Mpumalanga province of South Africa. For a distance of approximately 270m, the pipeline crosses a shallow undermined area where coal was mined using the bord and pillar method, presumably between 1930 and 1972. The shallow undermining has led to the development of various sinkholes on surface in the vicinity of the pipeline. Prior to construction of the pipeline, a ground stability evaluation of the undermined area was conducted by another geotechnical consultant which found that, based on

reports by other consultants, the footwall of the mine was situated approximately 18m below natural ground level.
To protect the pipeline should a sinkhole develop below the pipe, the Pipeline Contractor elected to install precast concrete piles below the pipe centerline with square pile caps on which the pipe rests but is not fixed to. Piles were installed to a depth of approximately 18m and at a spacing of approximately 17m. Due to a lack of construction records and piling details, some doubts existed about the founding depth of the piles and whether the piles were installed through the mining cavity into the competent rock below.

In 2007, heavy rains led to a new sinkhole developing underneath the pipeline within the undermined area and further deepening of other sinkholes around the pipeline. Following this, the Main Client (owner of the pipeline) requested a risk assessment be conducted by others. The risk assessment highlighted the same concerns regarding the founding depth and condition of the piles. In light of the above, the Main Client appointed a specialist pipeline consultant (Jones & Wagener's Client) to investigate the situation and to engineer a permanent solution to ensure that the pipeline is not affected should further sinkholes form.

Jones & Wagener (J&W) were appointed by the Client to assist in determining the extent of the undermining below the pipeline, design the remedial measures and to provide specialist geotechnical assistance during the implementation phase.

## 2 Investigation

The investigation phase entailed carrying out a preliminary desk study and probe-drilling investigation to determine the extent of the undermining. The investigation was undertaken during 2013.

### 2.1 Preliminary Desk Study

The underground mining was carried out using the bord and pillar mining method. The most likely position for a sinkhole to develop, that would be large enough to propagate to surface, was where the hanging wall (mine roof) was the weakest, i.e. in the centre of where two bords cross. This is called intersection failure. Consequently, determining the bord and pillar spacing would assist greatly in determining likely positions for sinkholes to develop for backfilling purposes. (Due to the shallow mining depth (<40m), pillar failure resulting in subsidence or sinkholes was excluded.)

An undermining plan was made available to J&W, however the undermining plan was old and of poor quality thus the bord and pillar spacing, alignment and mine boundaries could not be determined based purely on the undermining plan. By overlaying the undermining plan on a Google Earth image of the site, the positions of the existing sinkholes on surface were compared to the undermining plan. By extrapolating the sinkholes centre points, the typical bord and pillar spacing and alignment of the underground workings was estimated. An alignment of 35° east of north and a bord and pillar spacing of 13m (approximately 40 feet) was estimated.

Figure 1 presents the mining plan overlain on a Google Earth image. The dark grey zones depict the undermining plan for the mine.

The natural topography of the site slopes in a westerly direction towards a stream. Based on the location of the mine adits located on either side of the stream, it was believed that the coal seam is probably fairly horizontal. Thus the thinnest overburden would be situated along the western edge of the undermined area. Most of the sinkholes visible on surface formed near the western edge of the undermined area.

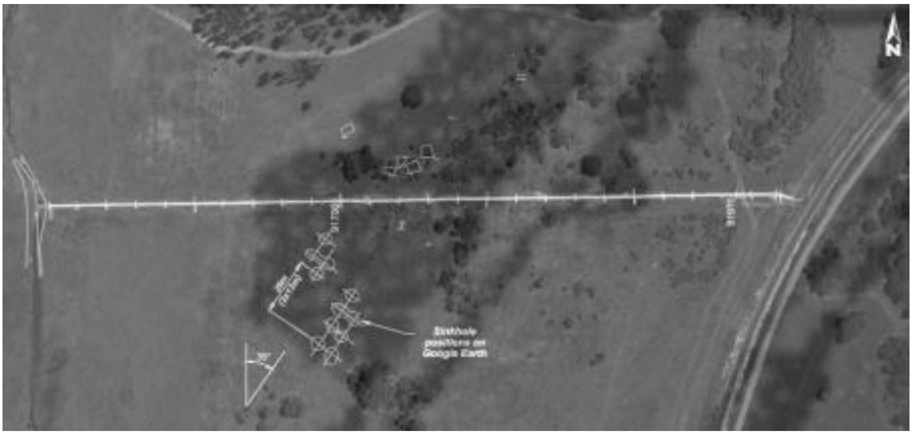


Figure 1. Undermining plan overlain on Google Earth image.

Due to it not being possible to access the underground workings and the lack of information with respect to the mining depth, mine layout and extent, the construction records of the pile installation and uncertainty of the pile founding conditions, the probe-drilling investigation was critical to the success of the investigation.

## 2.2 Probe-drilling Investigation

The probe-drilling investigation was carried out in 2013 and entailed drilling a total of 37 boreholes using a percussion rig with a down the hole (DTH) hammer. The purpose of drilling was to establish the physical dimensions of the mining cavity, especially the depth to the hanging wall, the height of mining cavity as well as the bord and pillar spacing. The boreholes were drilled until a cavity or mined out void was found. Once the hanging wall of the mine was breached, the rods were lowered to determine the depth to the footwall (mine floor). In cases where no cavity or void was found, the boreholes were drilled to a depth that exceeded the footwall depth in nearby boreholes. During the drilling, the penetration rate, air loss and hammer tempo was logged with samples taken at 1m intervals.

Based on the penetration rates recorded and samples retrieved the following typical soil profile was identified: hillwash or residual sandstone for the upper 3m underlain by medium hard rock sandstone becoming slightly carbonaceous with depth. The sandstone and carbonaceous sandstone was underlain by coal.

The depth to the hanging wall varied between 4m in the west and 26m in the east. Overall, the height of the mined-out cavity was less than 2m, except for one borehole where a 4m cavity was found. The larger cavity encountered at this borehole may have been caused by a change in mining method, i.e. the addition of bottom coaling or top coaling.

Although not noted during the probe drilling, it is possible that collapse of the hanging wall occurred in areas where the depth of the hanging wall exceeded 6m to 8m without the manifestation of sinkholes or subsidence on surface. This process is called goafing. Goafing occurs when the overburden material bulks during the collapse of the hanging wall, thus the “heap” of collapsed material catches up with the raveling face resulting in no further raveling/development of the sinkhole. Should the goafed material be removed by water, the upwards raveling may continue to ultimately result in sinkholes appearing on surface.

## 3 Remedial Works

Based on the findings of the investigation phase, a number of remedial options were tabled for consideration. The adoption of a final solution was a function of the risk and cost that the Main Client was prepared to accept. The following options were presented:
1. The do nothing option;
2. Deviate the pipeline around the undermined area;
3. Support the pipeline on a continuous reinforced concrete beam;
4. Expose the underground workings, backfill the excavation and re-route the pipeline; and
5. Stabilize the mining cavity beneath the servitude.

Taking into account the fact that Options 2, 3 and 4 required either widening the servitude or registering a new servitude altogether, and deviating the pipeline which will require shutting down the pipeline for a period of time, the Main Client ultimately selected Option 5. For the purposes of this publication, it was decided to only discuss the option that was selected.

### 3.1 Design for Backfilling Stabilization

Initially, the selected remedial option was to be applied to the entire 270m length of pipeline. However, with the depth to the mining cavity varying from the west to the east, the decision was made to divide the pipeline into three smaller zones. Each zone was evaluated to determine the risk of a sinkhole developing on surface with the most at risk zone being selected for remediation. At risk zones were defined as zones where the depth to the hanging wall is such that sudden loss of support will induce a sinkhole which will propagate to the underside of the pipeline. These are generally areas where the goaf material will not catch up to the raveling and the sinkhole will continue developing until it reaches the bottom of the pipe which leads to loss of support of the pipeline. Zone 2 (75m length) was selected for remediation.

The design objective of Option 5 was to stabilize the servitude and mitigate the risk of damage to the pipeline caused by a sudden, brittle loss of support due to sinkhole formation. The main purpose of the design was to fill the mining cavity in the servitude with concrete and grout. The main benefits of this option were that the inherent risk of sinkholes forming under the pipeline would be mitigated, and the pipeline need not be shut down for the implementation of this solution. The design for backfilling stabilization is described below.

In order to confine the concrete and grout pumped into the mined-out cavity to below the servitude (and not pump the entire mine full of concrete), the placement of shutters made of stone cones on either side of the servitude is required. A row of 254mm diameter percussion boreholes are drilled at 3m centres from surface into the mining cavity on each side of the servitude. Once drilled, 19mm stone is poured into the borehole with the aim of forming “stone cones”, stretching from the footwall of the mine to the hanging wall. A series of primary stone cones are constructed at 6m intervals with secondary stone cones spaced at 3m centres, installed between the primary stone cones. These boreholes are hereinafter referred to as SC boreholes with either prefix N or S indicating the northern or southern stone cone line, respectively (i.e. N-SC1 etc.).

Following the placement of the stone cones, a line of 254mm diameter boreholes is drilled at 4,5m centres within the servitude through which concrete is pumped into the mining cavity. The concrete would flow over the “valleys” between the cones to the outside of the servitude. However, the concrete would start building up to the hanging wall at the angle of repose of the concrete. These boreholes are hereinafter referred to as C (concrete) or G (grout) boreholes.
To make sure that the mining cavity is backfilled up to the hanging wall, a series of 114mm diameter boreholes are drilled between the concrete boreholes through which grout is pumped. The purpose of the grout is to fill the remaining cavity (if any) between the concrete level and the hanging wall.

Lastly, 100mm diameter steel pipes are grouted into the stone cone boreholes with the aim of creating small diameter piles (micropiles) on either side of the servitude. The micropiles will provide additional reinforcing to the soil to limit the potential of neighboring sinkholes extending into the servitude.

The borehole layout and sections showing the primary and secondary stone cones with the concrete placed between the stone cones are shown in Figure 2 and Figure 3.

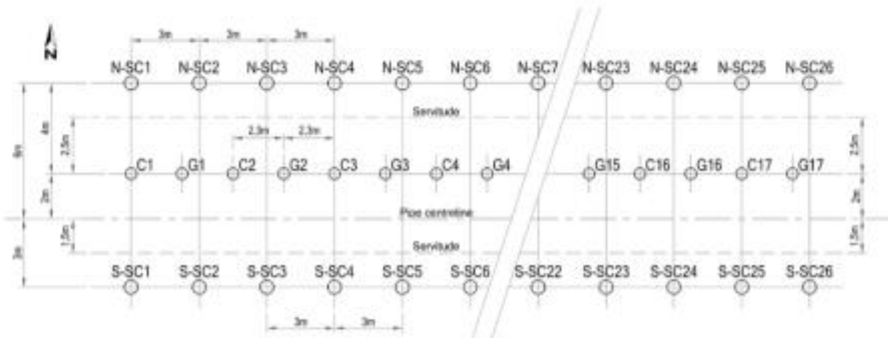


Figure 2. Typical layout for stone cone, concrete and grout boreholes.

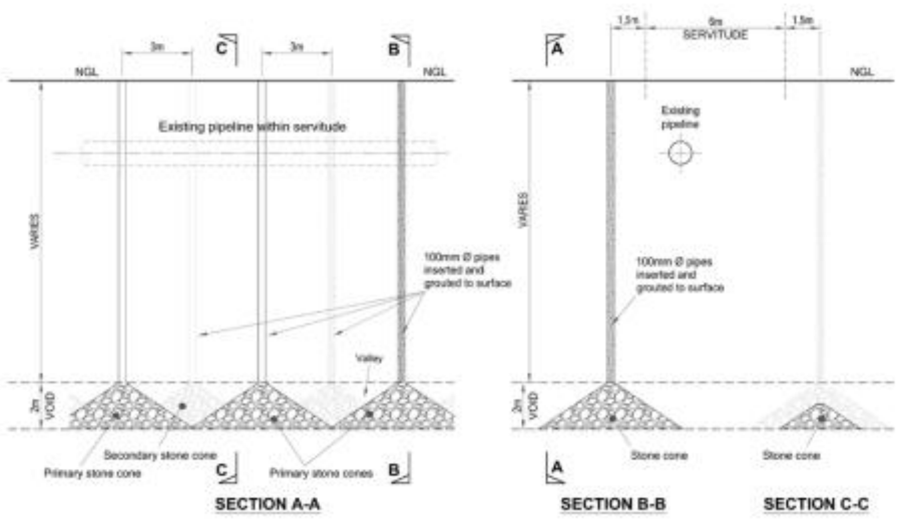


Figure 3. Section showing the concrete filling the mining cavity.

## 4 Implementation Phase

The implementation phase of this project was carried out in the second half of 2016. J&W fulfilled the role of technical consultant to the Client with the aim of providing technical input/guidance.

The design was based on the assumption that the mining cavity underwent no changes before implementation. However, since the investigation was concluded in 2013, no further investigative work was carried out to determine whether any changes had taken place within the mining cavity, i.e. collapse of the hanging wall etc. The following sections detail the construction stages and what was encountered during each.

Following the submission of the conceptual design, the Main Client decided not to carry out remedial work on the entire 270m length of undermined pipeline, but to only treat a 75m portion with the highest risk of loss of support and where the depth to the hanging wall is such that it is highly likely that a sinkhole would propagate to the bottom of the pipeline.

## 4.1 Stone Cone Borehole Drilling

The construction sequence entailed drilling all the stone cone boreholes before placing the 19mm stone. Thus the SC borehole drilling provided valuable insight into the current condition of the mining cavity, especially since they were drilled at a relatively close spacing in areas not covered during the 2013 investigation.

The SC borehole drilling commenced in the eastern section of the 75m treatment zone where the hanging wall depth below ground level was a maximum. The drilling took place from east to west.

The drill operator was instructed to record the depth at which total air loss, no hammer action/tempo and no sample return was encountered. This depth was recorded as the depth to the hanging wall. The operator then lowered the drill string to the footwall where hammer action/tempo was again encountered. This depth was measured and recorded as the depth to the footwall.

The eastern section of the treatment zone yielded no complications and the hanging wall and footwall depths were determined easily. The mining cavity was clear/open with no indication of hanging wall collapse, similar to what was encountered during the investigation.

However, the western section yielded different conditions to those encountered along the eastern section. During the drilling, it was found that the mining cavity was either completely or partially filled with broken material of unknown origin. The following aspects were noted:

- Partial air loss once the hanging wall was reached;
- Partial or no sample return below the hanging wall;
- Irregular hammer action/tempo below the hanging wall or just above the footwall; and
- Regular hammer action/tempo once the footwall was reached.

The hammer action within the cavity was not indicative of intact rock but rather of a broken rock mass. The rock fragments retrieved from the samples were angular and in no way similar to that encountered in the rock horizons situated above the mining cavity. These factors were all considered to be as a result hanging wall collapse since the 2013 investigation. In some cases, a clayey fine coal sample was recovered, underlain by intact sandstone rock which, in turn, was underlain by the broken rock mass described above. J&W suspects that two coals seams were mined in these cases and that the processed coal was used to backfill the upper seam cavity. It is believed that the mining company noted that the shallow depth of mining in the western section was unsafe and thus backfilled the upper cavity to stabilize the western section of the mine.

Due to the different conditions encountered in the western section, J&W requested that the borehole depths (which were drilled to just below the footwall) be monitored on a daily basis to determine whether any changes took place since the borehole was drilled, while the Contractor was preparing for the stone cone placement. From the monitoring data it was determined that the borehole depth decreased in some boreholes, with some borehole depths decreasing to be even shallower than the hanging wall depth. Thus it was noted that the borehole continued to collapse after the borehole was drilled. In some cases, the remainder of the partially filled cavity was filled to the hanging wall depth by the further collapsing hanging wall.

## 4.2 Stone Cone Placement

The placement of the stone cone shutters followed the drilling of the SC boreholes. Based on the hanging wall and footwall depth determined during the drilling, the height of the cavity was calculated. Based on the height of the cavity, angle of repose of the 19mm stone and the assumption that the foot wall is horizontal and free of any rubble on surface, the theoretical volume of stone required to create each stone cone was determined. Due to the uncertainty which accompanies working underground, the possibility that a stone cone is situated in close proximity to a mine pillar could not be excluded. Thus the following idealized scenarios were considered as shown in Figure 4:

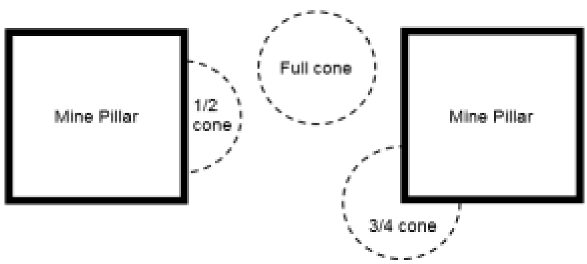


Figure 4. Stone cone scenarios.

The full cone scenario was taken as the upper bound theoretical volume with the halved cone scenario the lower bound theoretical volume. As a guideline, the volume of stone accepted by each SC borehole was compared to the upper and lower bound volumes. As mentioned earlier, many uncertainties accompany working underground thus reasonable assumptions play an important role. The volume of a stone cone is a function of the angle of repose therefore the angle of repose of the stone delivered to site was measured regularly. Taking into account the assumptions made and the fact that the angle of repose of the stone was measured regularly, the theoretical volumes were considered to be reasonable. Nevertheless, the theoretical volumes were only used as a guideline.

If a SC borehole accepted a volume of stone which fell between the upper and lower bound theoretical volume or greater than the upper bound theoretical volume and the apex of the stone cone reached the hanging wall, the stone cone was considered satisfactory. If, however, the volume of stone accepted was less than the lower bound theoretical volume, which essentially implies that the cavity was smaller than what was recorded during the drilling, concerns were raised.

Figure 5 and Figure 6 present the stone cone volume and cavity height for the northern and southern stone cone lines, respectively, with the lower and upper bound theoretical volumes shown. Note that boreholes S-SC2 and S-SC16 were drilled into mine pillars thus no cavity was encountered. Similarly, boreholes N-SC4, N-SC11, N-SC12, N-SC13, N-SC18 and N-SC19 were drilled into mine pillars. It should also be noted that the depth to the apex of each stone cone was continuously measured and compared to the depth of the hanging wall to make sure that the stone was not placed within the throat of the borehole.

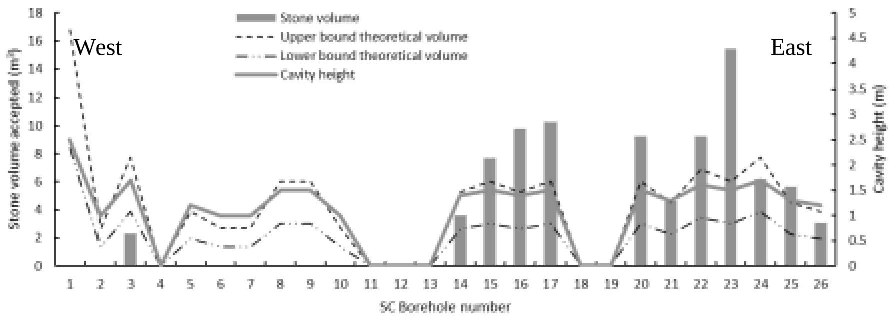


Figure 5. Northern stone cone volumes and cavity height.

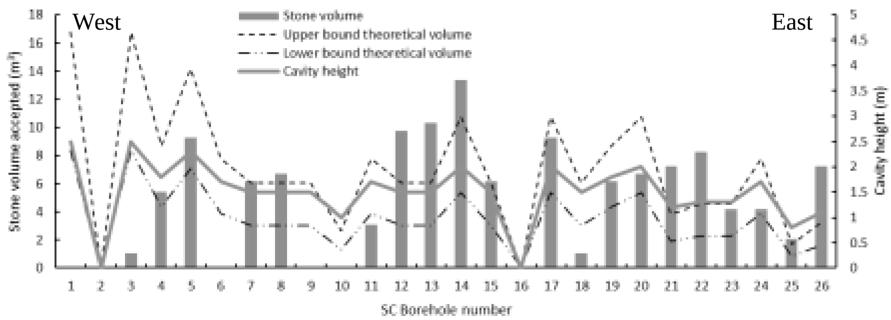


Figure 6. Southern stone cone volumes and cavity height.

As noted during the drilling of the SC borehole, a clear distinction was noted between the eastern and western portions of the treatment zone. The eastern section accepted volumes of stone that were considered satisfactory. However, the western section accepted volumes which were less than the lower bound in most cases. Figure 5 clearly shows that the volume of stone decreased for borehole SC1 to SC13 compared to boreholes SC14 to SC26. Figure 6 shows a similar phenomenon where some boreholes towards the west of the treatment zone accepted no stone compared to the eastern boreholes.

This phenomenon was ascribed to be due to deterioration and collapse of the hanging wall within the western portion of the treatment zone. Very few open cavities were found and along with the collapse of the boreholes themselves, very low volumes of stone could be placed.

### 4.3 Concrete and Grout Borehole Drilling

The drilling of the concrete and grout boreholes confirmed what was found during the SC borehole drilling and stone placement. An empty/open cavity was encountered towards the eastern end of the treatment zone compared to an either partially or completely filled cavity towards the western end.

### 4.4 Concrete and Grout Placement

The concrete and grout was pumped into the mining cavity by means of a tremie pipe placed to within 500mm of the foot wall. During pumping, the tremie pipe was gradually lifted, remaining below the concrete/grout level, until the concrete/grout level reached the hanging wall. This process was monitored by means of measuring the concrete level in the borehole into which concrete/grout was pumped and the adjacent boreholes and the concrete spread laterally due to the high slump.

A similar phenomenon was noted where a smaller volume of concrete and grout was placed in the western section than expected.

### 4.5 Suitability of Original Design to Western Section

The borehole drilling, stone cone and concrete/grout placement activities all demonstrated that the mining cavity towards the western section of the treatment zone was either partially or completely filled. The original remedial works design was based on the assumption that the cavity is clear/open thus the entire cavity could be sufficiently stabilized. By applying the initial design to a partially filled cavity, only a portion of the cavity is stabilized with concrete and grout. As a result, a risk remains as the remaining portion of the cavity was filled with material of which the geotechnical characteristics are unknown. If the collapsed material was made up of large rock boulders, the compressibility of the material would be less, thus the risk of subsidence on surface would be less. If, however, the remaining portion was filled with loose, highly compressible material, the possible deformation would be greater, resulting in subsidence on surface.

In simple terms, the structural arrangement of the broken rock mass could be described as either being like a *house of cards* with large voids and a high compressibility which may result in a brittle failure, or a *pile of bricks* with smaller voids and lower compressibility which may only result in gradual subsidence.

Due to time constraints it was not possible to further investigate the geotechnical properties of the material/s encountered in the mining cavity in the western section of the treatment zone. Also, due to contractual and programme requirements, the Main Client requested that the original design be implemented in the western portion of the site, to see if it would be successful.

## 5 Revised Remedial Strategy

Following the implementation of the original design over the entire treatment zone, a revised remedial strategy in the form of downstage grouting was recommended to treat the western zone and achieve the goal of stabilizing the undermined ground within the servitude. The downstage grouting procedure required the pumping of grout into the broken rock or coal backfill through a closed pressure system. Downstage grouting is a process whereby grout is injected under pressure in depth intervals. This involves drilling to a preselected elevation for the specific stage and then pumping grout under pressure until a pressure or volume limit is reached. Once the first stage grouting is completed and the grout has set sufficiently, the hole is redrilled to the next depth interval (deeper than the first) and the second stage grouting is commenced. This process is repeated until the required depth of treatment is reached.

To ensure a closed pressure system, the revised remedial strategy entailed drilling a number of 114mm diameter percussion boreholes to 2m depth in the western section. A 2,5m stub casing is installed in the 2m borehole which is then grouted into place. Once the grout has set sufficiently, Stage 1 is commenced whereby a 75 diameter borehole is redrilled into the mining cavity through the stub casing. The Contractor then couples the high pressure grouting hose to the stub casing and pumps grout into the partially filled mining cavity under high pressure. The pressure and volume limits were set to 1 MPa or 8 pockets (960 litres) of grout. If the volume criteria are reached, Stage 1 grouting is stopped. Stage 2 then entailed repeating the process at the following depth interval once the grout has set. The process is continued until the pressure criterion of 1 MPa is met.
The main objective of the revised remedial works design was to “stitch” the broken rock mass together by filling the voids with grout. Unfortunately, due to contractual and programme requirements, it was not possible for the Contractor to implement the revised strategy.

## 6 Summary

A pipeline was constructed over historically undermined ground. Subsequent to the installation of the pipeline, sinkholes and subsidence became visible in the vicinity of the pipeline. The Main Client raised concerns regarding the risk posed to the pipeline should a sinkhole develop under the pipeline. J&W conducted an investigation to determine the extent of the undermining, the properties of the mined-out cavity and the condition of the overburden.

Based on the findings of the investigation, J&W presented several remedial works options for consideration. The selected remedial measure entailed filling the mining cavity within the servitude with concrete and grout with rows of stone shutters on either side of the servitude.

During the implementation phase, it was noted that the properties of the mining cavity in the eastern and western section varied. An open cavity, similar to what was encountered during the investigation, was found in the eastern section. In the western section, however, a partially or completely filled mining cavity was encountered. These conditions were different to those found in the 2013 investigation. In order to achieve the goal of stabilizing the undermined ground within the servitude, a revised remedial strategy in the form of downstage grouting was recommended to treat the western zone. However, due to contractual and programme requirements, it was not possible for the Contractor to implement the revised strategy.

A detailed risk analysis was conducted to evaluate the remaining risk of sinkhole and subsidence formation. The risk analysis concluded that there is a very low probability of a sudden loss of support to the pipeline in the western section whereas the risk of total loss of support in the eastern section was mitigated. However, the possibility remains that gradual subsidence may take place in the western section due to compression of the material within the cavity. The Main Client accepted the residual risk.

*The authors acknowledge with thanks the input of Mr. Gavin Wardle and Mr. Jacobus Breyl of Jones & Wagener as well as all the personnel from the Main Client and Jones & Wagener's Client who were involved throughout the project.*

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# Reinstatement of the Hazelmere Dam Grout Curtain

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## Abstract

The raising of the full supply level of the Hazelmere Dam in KwaZulu-Natal commenced in 2015. The original construction of the gravity concrete structure was completed in 1977. As part of the project scope the grout curtain was reinstated. This paper deals with the design, construction and results of the grout curtain reinstatement. The major design considerations are discussed with reference to the original grout curtain design. The reinstatement grout takes and total drilled lengths are compared with that of the original grout curtain installation.

***Keywords*** *Hazelmere Dam, Grout Curtain, Drainage Holes*

## 1 Introduction

Construction for the raising of the full supply level of the Hazelmere Dam by 7m, commenced in 2015. The project included the installation of post-tensioned anchors, replacement of the existing ogee spillway with a piano key weir and the reinstatement of the grout curtain. Ingérop South Africa was appointed by the Department of Water and Sanitation as engineers for the project. Knight Piésold was subcontracted to advise on geotechnical matters and the reinstatement of the grout curtain.

Hazelmere Dam is located in the Umdloti River approximately 5km north of Verulam and 5km west of the King Shaka International Airport in the KwaZulu-Natal Province of South Africa. The concrete gravity wall is approximately 46m high from deepest foundation level to the non-overspill crest height before raising. The original construction of the 470m long concrete gravity structure was completed in 1977 and included an extensive grouting programme.

A geotechnical investigation, conducted during the project design phase in 2012, indicated that the existing grout curtain was not functioning effectively and that most of the deeper drainage holes in the spillway section of the drainage gallery, were blocked (Knight Hall Hendry, 2012).

The reinstatement of the grout curtain commenced in October 2015 and was completed in October 2016. The dam was operational during construction even though a reduced water level was maintained. Figure 1 depicts the dam wall as seen from downstream during the raising of the dam wall. The longitudinal section through the dam wall with the length and spacing of grout curtain holes drilled from the drainage gallery is provided in Figure 2. A typical cross-section through a gravity dam wall showing the grout curtain and drainage hole location is shown in Figure 3.

Figure 1. View of the Hazelmere Dam during raising

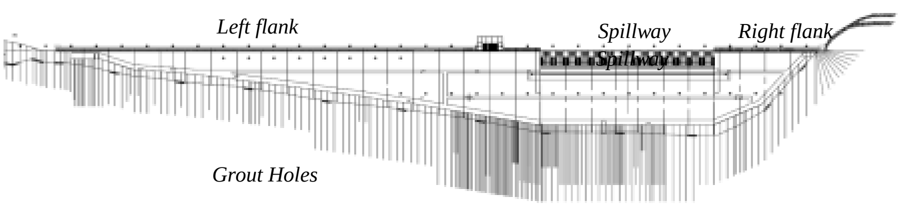


Figure 2. Longitudinal section of Hazelmere Dam wall showing depth and spacing of grout curtain holes below drainage gallery

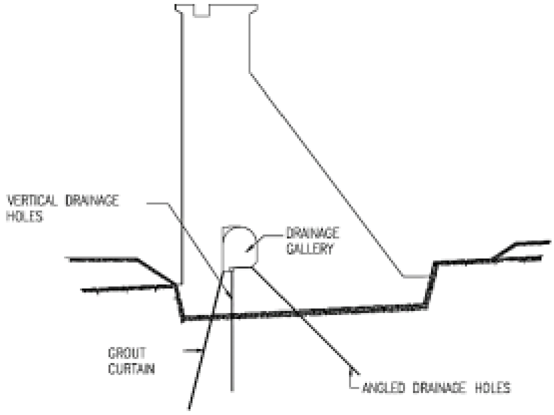


Figure 3. Typical gravity dam cross-section showing the location of the grout curtain and drainage holes.

## 2 Hazelmere Dam Foundation Conditions

### 2.1 Geology

The dam is underlain by quartzitic sandstone of variable grain sizes with interbedded, thin micaceous and ferruginous shale beds of the Natal Group. The sandstone is almost horizontally bedded, moderately to closely jointed with a rock mass permeability of between 0 and 20 Lugeon (1 Lugeon = a water take of 1ℓ/m/minute at 1MPa pressure). The sandstone bedding planes dip at approximately 7 degrees towards the right flank. Rock cliff exposures visible during low water conditions in the reservoir indicated that bedding planes are reasonably continuous over extensive distances in an upstream-downstream direction (Knight Hall Hendry, 2012).

### 2.2 Groundwater

It was reported that artesian groundwater conditions were encountered during the original pre-construction geotechnical investigation of the dam site (George, 1972). Particulars regarding the depth at which these conditions were encountered and the water pressures were however not available. Artesian conditions were also encountered during the original installation of the grout curtain and it was reported that in many instances grout was forced out of adjacent and even distant grout holes (van Schalkwyk, 1981).

Pressurised water was encountered in all the boreholes in the spillway section and in most of the boreholes in the stilling basin during the 2012 geotechnical investigation. A static pressure of up to 100kPa at collar elevation was measured in boreholes in the spillway section of the drainage gallery. The source of the water encountered was not determined, but results of borehole camera surveys indicated that a strong flow of air bubbles (assumed to be associated with water flow) emerged at depths of between 4m and 7m along prominent continuous bedding planes (Knight Hall Hendry, 2012).

Pressurised water was encountered in exploration boreholes drilled at the beginning of the grout curtain reinstatement period at depths of 15m below gallery floor level and deeper. Water samples were collected and isotope analyses were done to determine the source of the water. It was determined that water from deeper than approximately 40m below gallery floor level is not related to dam water (Knight Piésold, 2016).

## 3 Grout Curtain Background

An extensive grouting programme consisting of contact grouting and curtain grouting was performed during the original construction of the dam wall as highly pervious foundation rock conditions were encountered (van Schalkwyk, 1981).

Prior to the 2012 geotechnical investigation, it was assumed that the grout curtain was intact as no, or very little seepage was observed in the drainage holes downstream of the dam wall. However, exploration boreholes drilled from the drainage gallery and stilling basin during the 2012 investigation encountered substantial water flows which led to the conclusion that the grout curtain was damaged and that the existing drainage holes were blocked (Knight Hall Hendry, 2012).

There are reservations regarding the long-term durability of relatively lean grouts used at the time of the original grout curtain construction. There have been new developments in dam grouting technology and grouting methods and as a result there is a tendency (in the USA) to regrout existing dams as part of dam rehabilitation projects (Weaver, 2007).

## 4 Grout Curtain Design

The purpose of the grout curtain is to create a zone of low permeability below the dam wall. The grout curtain and drainage hole combination reduces water flow in the foundation, thus reducing hydrostatic uplift pressure underneath the dam wall. The risk of sliding of the dam wall is thereby reduced (Krynine and Judd, 1957).

The original single row grout curtain was constructed with an upstream inclination of 14 degrees from vertical. The depth of the grout curtain varied between 17m on the flanks and 73m in the river section. Primary grout holes, spaced at 10m intervals, secondary grout holes at 5m intervals and tertiary grout holes at a spacing of 2,5m were required along the entire length of the curtain with quaternary grout holes (1,25m spacing) and quinary grout holes ( at a spacing of 0,6m) in certain areas (van Schalkwyk, 1981).

The reinstated grout curtain layout was designed to correspond with the original grout curtain layout as far as possible. A single row grout curtain with an original maximum depth of 45m below the drainage gallery floor in the spillway section was constructed in line with the existing grout curtain. This depth was reduced to 35m as more information regarding the depth of the artesian water source became available. Grout curtain holes were inclined upstream at 14 degrees from vertical, as far as practically possible, to obtain maximum rock joint interception. Primary grout hole spacing was 8m with secondary grout holes being drilled and grouted irrespective of the water take results of primary holes (Knight Piésold, 2015).

Grouting closure was governed by residual rock mass permeability and recorded by means of Lugeon (Lu) values. A design rock mass permeability of 2Lu was adopted. Where water takes of secondary grout holes exceeded 2Lu, splitting holes were provided. Splitting holes is a term used to describe the following sequence of grout holes that split the distance between the preceeding holes. The use of rock permeability as closure criterium does not necessarily infer that a direct correlation exists between water take and grout takes. Water has zero cohesion and the water take in a continuous rock joint or fracture will be ongoing while grout, having cohesion, will reach a maximum distance from point of injection where flow will stop.

Grout pressures for each stage were selected such that maximum pressures be applied in the foundation for maximum penetration, without risking uplift or dilation. Applied pressure ranged between 500kPa and 1500kPa. When a grout take in excess of 100ℓ of grout per 1m length of stage occurred and no indication of refusal was evident, grouting was stopped for a period of four hours, thereafter the same stage was grouted again.

## 5 Methodology

The grouting methodology used at Hazelmere Dam generally followed the sequential curtain grouting methods proposed by Houlsby (Water Resource Commission, 1981).

The downstage sequential grout method with surface packers was adopted with stage lengths of no greater than 10m in rock. A test section on the upper left flank consisting of three primary grout holes and subsequent splitting holes, was used to confirm the grout mix, grout pressures and grouting procedure. The depth of the grout curtain was increased in a stepped manner per block until the maximum depth was reached in the spillway section.

The first stage of percussion drilled, primary grout holes were drilled, flushed with water, water tested and grouted with a mechanical packer at the surface of the hole. The mechanical packer was left in the hole following the completion of the stage to allow the initial set of the grout. The subsequent stage(s) of the hole was drilled between seven and twenty four hours after grouting of the former stage depending on the grout take of that stage. This process was continued until the final stage of the hole had been grouted. Secondary holes were drilled only once the second stage of adjacent primary holes had been completed. Drilling and grouting of successive splitting holes had to remain one completed stage behind previous holes.

Abbreviated water tests were conducted for every stage of a grout hole before grouting commenced (Abbreviated water tests measures water take of a test section over a 15 minute time period at a constant pressure). Water pressure tests were carried out with a mechanical packer at the surface at 100kPa surface pressure regardless of the static pressure of water coming from the hole. In certain areas, the Lu value of quaternary grout holes (1m spacing) was still greater than 2. Instead of adding a splitting hole, these holes were redrilled to the applicable depth, water tested and grouted. Only if this method proved unsuccessful were quinary holes drilled at 0,5m spacing.

The grout curtain reinstatement was mainly carried out from the confined conditions of the drainage gallery. This necessitated the use of small drill rigs with limited rod length. Drilling and grouting were done from the dam crest on the upper parts of the right flank as access for the drill rig inside the drainage gallery from the steep staircase was problematic. A drilling operation from the spillway section of the drainage gallery is illustrated in Figure 4.

Figure 4. Drilling operation inside the drainage gallery

## 6 Grout Mix

The selected grout mix was used in the test section on the upper left flank and specifically designed for injection under hydrostatic conditions caused by the water in the dam. Rapid hardening cement was used due to the smaller particle size, which allows the grouting of fissures with smaller apertures. A superplasticiser and a viscosity modifying agent were added to the grout mix to extend the time the grout could be used and to prevent the segregation of grout when coming into contact with groundwater. The resultant water:cement ratio used at Hazelmere Dam was 0,66:1 by weight. Quality tests were performed on the grout mix at random intervals during the grout curtain reinstatement.

## 7 Water and Grout Take Results

The Lugeon value for every stage of every grout hole was determined based on the water take during abbreviated water tests. The grout take was recorded in liters for every stage of every grout hole. This value was converted to a kg/m cement take. Grout takes of more than 100 liters per 1m length of stage was noted as a large grout take.

A summary of the average cement take and Lu values per block for the entire grout curtain is shown in Figure 5. Figures 6 and 7 provide a summary of the average Lu value per stage for each block and the average cement take per stage for each block respectively. The average cement take (kg) and the average Lu values follow a similar trend. Higher permeability and cement takes occurred on the upper flanks and in the spillway section along with the lower left flank. Generally, cement takes for the deeper Stages 3 and 4 were greater than those of Stages 1 and 2.

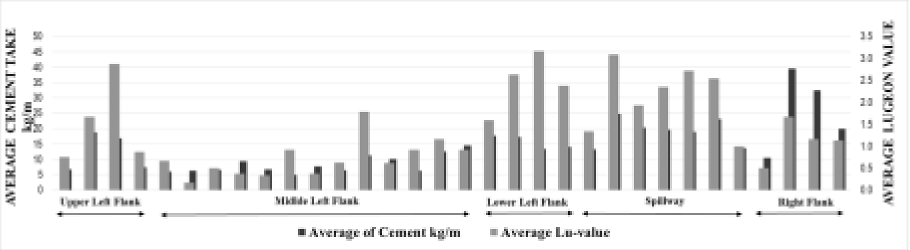


Figure 5. Average cement take and Lugeon value per Block

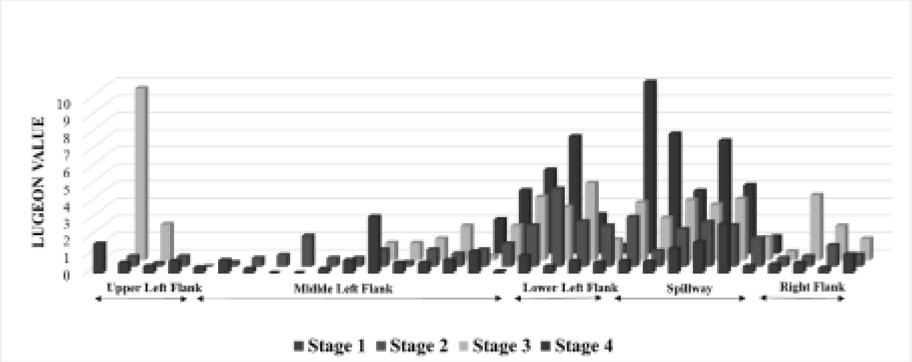


Figure 6. Average Lugeon value per Block per stage

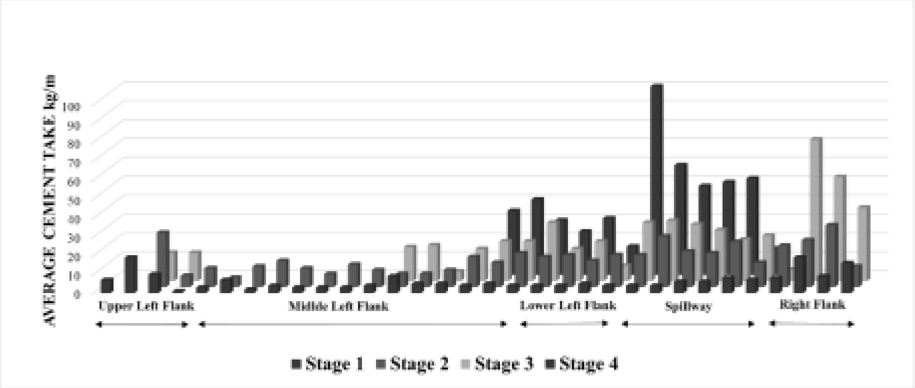


Figure 7. Average Cement Takes per Block per stage

## 8 Comparison between Results of Original and Final Grout Curtain

Little information could be found regarding the original grout curtain installation. It is clear that the process was much more extensive than first anticipated as the total consumption of grout was approximately 75% more than had been expected. The original construction of the grout curtain took 18 months instead of the expected 7 months and the total cost of grouting was 50% more than what was expected (van Schalkwyk, 1981). The grouting practice at the time of original construction was to use a grout mix with a high water cement ratio. The available data for the original grout curtain installment is compared to the final data for the reinstatement of the grout curtain in Table 1.

Table 1. Comparison between original grout curtain installation and reinstatement values

| | **Original Grout Curtain** | **Reinstated Grout Curtain** |
|---|---|---|
| Total Length Drilled (m) | 12 282 | 8 278 |
| Total Cement take (kg) | 400 000 | 132 150 |
| Average Grout take (kg/m) | 32,6 | 15 |

It follows from the above table that the average cement consumption of 15kg/m recorded for the current curtain grouting operation is significantly less than the documented 32,6kg/m for the original grouting (van Schalkwyk, 1981). This can be explained by the fact that the current grouting methodology is more efficient than what was used in the past, i.e. thicker grout mix with additives. It can also be argued that because the current grout curtain position is similar to the previous curtain, the rock mass permeability had to some extent, been reduced by the original grouting.

## 9 Verification Holes

Three inclined check holes were drilled in the plane of the grout curtain to verify that the cutoff design criterium of 2Lu had been met. The verification holes were specifically drilled in problem areas in order to verify the grouting efficiency. Verification holes were drilled on the lower left flank and in the spillway section. Abbreviated water tests were done at 5m intervals in rock for each of the verification holes. Most of the water test results indicated Lu values of less than 2, except for three section with Lu values of 3, 7 and 5. The grout take for the stages with Lu values exceeding 2 was not excessive therefore no additional grouting was done in these areas. The overall impression gained from the verification holes were that the grout curtain had been reinstated successfully, despite the odd instances where Lu values exceeded 2. The verification holes were grouted upstage at pressures corresponding to the pressure used for the grouting of the grout curtain holes.

## 10 Drainage Holes

After completion of the grout curtain, reinstatement of the drainage system followed. According to available as-built drawings, the original drainage system comprised two rows of drainage holes. The first located just behind the grout curtain and the second located at the downstream side of the drainage gallery inclined downstream. The spacings of the holes in the two rows were staggered to provide a 2,5m spacing between drainage holes.

Based on the artesian water level it was decided to limit the depth of the reinstated drainage holes to 20m below drainage gallery floor level. Existing vertical drainage holes were cleaned and the portions of holes in excess of 20m were plugged.

## 11 Conclusion

The experience gained at Hazelmere Dam raised questions about the effectiveness of grout and drainage curtains at dams, when lean, unstable cementitious grout mixes were used. Based on the experience gained at Hazelmere Dam, it is considered likely that other older dams could have similar conditions, whereby the grout curtain has lost its effectiveness due to poor durability of unstable grouts used in the past, possibly combined with blocked drainage holes. This aspect should receive more attention in the dam safety programme.

The reinstatement of the grout curtain took longer than expected due to the extensive number of splitting holes required on the lower left flank and in the spillway section. The grout curtain was reinstated along the entire length of the left flank although it was initially expected that parts of the upper left flank would not require any grouting.

For both the original grout curtain construction and the reinstatement phases, the magnitude of grouting and drilling required were underestimated. This could be attributed to the presence of the artesian water conditions, but mostly due to the non-homogeneous rock mass permeability of the foundation rock. The lesson learned is that in some rock types, it is difficult to predict rock mass permeability based on widely spaced exploration boreholes. The actual conditions can only be known once very close hole spacing (less than 8m) is applied.

## Acknowledgements
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- Ingérop South Africa
- Department of Water and Sanitation

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# Booysendal Central Development

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## Abstract

Northam Platinum Limited's new Booysendal Central Complex development, situated south of its existing Booysendal North Complex, comprises a 140m wide and 30m high box-cut that will house seven incline portals to access the upper group 2 (UG2) and Merensky platinum-group metals (PGM) ore bodies. A new 4.8 km long access road with six cuttings, each up to 10.5m high, will provide access to the development from the north. Ore from this box-cut will be transported via a rope conveyor system to Everest Mine for processing.

This paper briefly outlines the planning, investigation, design and construction of the box-cut and cuttings on the development. The brief outlines then becomes the backdrop for a case study that highlights some of the problems encountered throughout the project whilst aiming to provide possible solutions for future reference.

***Keywords:*** *Booysendal, lateral support, cutting, design adaption,*

## 1 Introduction

Northam Platinum's existing Booysendal mine is situated in the Steelpoort Valley on the border of the Mpumalanga and Limpopo Provinces. Geologically, it is situated in the Bushveld Igneous Complex which hosts the platinum rich UG2 and Merensky ore bodies.

Northam's most recent development is termed the Booysendal Central Complex and is situated to the south of its existing Booysendal North Mine. The main focus of this development is the new 140m wide and up to 30m high box-cut in which seven portals, each 10m wide, will be created to access the ore bodies. These portals will be inclined at 10° above horizontal as the upper reefs are suspended above the box-cut platform in the anorthosite geology.

The box-cut will be accessed from the north via a 4.8 km long road traversing various streams in the pristine mountainous terrain. Along this access road, six critical cuttings are encountered that are up to 10.5m high. The stability of these cuttings is essential to the construction and operation of the new box-cut.

Ore from this new box-cut mining operation will be transported via a rope conveyor system over a distance of some 4 km to the existing processing plant at the Everest Mine. This system will comprise a loading station with a 3 500 ton silo, 12 intermediate pylons or towers and a drive/off-loading station at the Everest Mine. Figure 1 below provides an indication of the locality of the existing Booysendal North Mine (BYN), Everest Mine and Booysendal Central box-cut (BYC).


Figure 1. Locality plan of the mines and new box-cut

This article highlights some of the problems encountered on the project against the backdrop of the planning, investigation, design and construction phases, whilst aiming to provide possible solutions for future reference.

This project is utilised as a case study, allowing the author to highlight the development, management and resolution of several typical geotechnical problems and challenges encountered when undertaking this form of large scale geotechnical investigation, design and construction assistance.

## 2 Investigative fieldwork

The investigation planning phase was initiated in September 2015. Since then the investigation scope has been increased from feasibility to detailed design level. Fieldwork commenced at the end of October 2015, as discussed in the following sections.

### 2.1 Box-cut

The box-cut was the main focus of the development at the onset of the project. Lateral support was always seen as the most effective means of stabilizing the slopes and thus the investigation was tailored to provide all the information that a lateral support design would require. The near-surface investigation comprised a total of 18 test pits excavated with a 20 ton excavator. The deep subsurface investigation comprised 39 rotary core boreholes amounting to a cumulative total of 988m of drilling.

Due to the natural dip of the anorthosite rock, established from outcroppings and exposed cuttings, seven of these boreholes were inclined at 20° above the vertical in a south-west direction and orientated utilising the clay-impression technique. This was aimed at perpendicularly intercepting the primary joints in the rock mass in order to determine the accompanying dip and dip-direction. Five joint sets were encountered, with the main set dipping at 29° in a direction of 134° (SE).

This information was presented on a Stereonet plot and utilised to perform a kinematic analysis with RocScience's Dips software. This kinematic analysis calculates the likely mode of failure, with the possible outcomes being planar sliding, wedge sliding, flexural toppling and direct toppling.

A deeper soil and weathered rock zone was encountered in the northern portion of the box-cut high wall position. This indicated that the rock was either dipping in this direction and/or that the adjacent natural drainage gully caused enhanced chemical weathering to the rock mass. To ascertain the extent of this comparatively weak profile, five additional boreholes, amounting to 150m of drilling, were undertaken in this area. This provided information which would establish the impact on portal stability and ultimately the feasibility of the proposal. In due course, the position of the box-cut high wall was moved some 20m to the south, away from these structures and weathering zones, where competent material was encountered at shallower depths.

## 2.2 Access Road

The localities of the critical cuttings on the 4.8 km long permanent access road were determined by design engineers responsible for the vertical and horizontal alignment of the road. The subsurface investigation involved drilling 12-15m deep rotary core boreholes at key locations, resulting in a cumulative 170m of core. Representative samples were submitted to a laboratory for strength tests.

The original construction road only partially overlapped the permanent access road, to allow quicker access to the box-cut construction whilst the permanent road was being constructed. The route alignments were later adjusted to overlap more, impacting slightly on the investigation activities.

# 3 Laboratory testing

Representative core samples from the box-cut and access road investigation were submitted to an accredited laboratory for the following tests:

- Unconfined compressive strength (UCS) – 13
- Unconfined compressive strength with modulus and Poisson's ratio (UCM) – 7
- Point load index (PLI) – 35

The tables overleaf indicate a summary of typical UCS and UCM, and PLI test results from the access road core samples.

Table 1. Summary of PLI test results for access road investigation

| BH ID | Depth (m) | | Average corrected Point Load strength IS(50) (MPa) | Diametral PLI | Axial PLI | Hardness | |
|---|---|---|---|---|---|---|---|
| | From | To | | Est. UCS (MPa) | Est. UCS (MPa) | Logged | Actual |
| BH P02a | 13.28 | 13.62 | 6.49 | | 155.8 | VHR | VHR |
| | | | 7.82 | 187.68 | | | VHR |
| BH P03a | 15.20 | 15.50 | 0.24 | | 5.76 | SR - MHR | SR |
| | | | 0.19 | 4.56 | | | SR |
| BH P11a | 14.23 | 14.38 | 0.60 | | 14.4 | MHR | MHR |
| | | | 0.58 | 13.92 | | | MHR |
| BH P12a | 3.00 | 3.17 | 9.4 | | 225.6 | MHR - HR | EHR |
| | | | 10.7 | 256.8 | | | EHR |
| BH P13a | 6.42 | 6.59 | 2.7 | | 64.8 | HR | HR |
| | | | 3.3 | 79.2 | | | VHR |

Table 2. Summary of UCS and UCM test results for access road investigation

| BH ID | Depth (m) | | E Modulus (GPa) | Poisson's ratio | UCS (MPa) | Logged hardness | Actual hardness |
|---|---|---|---|---|---|---|---|
| | From | To | | | | | |
| BH P02a | 4.49 | 4.8 | 89.5 | 0.23 | 221.1 | HR - VHR | EHR |
| BH P02a | 10.6 | 10.9 | 0.7 | 0.27 | 6.7 | MHR | SR |
| BH P11a | 13.87 | 14.12 | | | 266.6 | HR | EHR |
| BH P12a | 4.3 | 4.52 | | | 135.6 | MHR - HR | VHR |

## 4 Design parameters

Hoek-Brown rock parameters for design were determined using Bieniawski's 1989 publication on calculating a rock mass' geological strength index (GSI) and rock mass rating (RMR), in conjunction with RocScience's RocLab V1.033. Input values to the aforementioned were first based on core logs for preliminary design purposes and later adjusted according to laboratory results in the detailed design. Hardness values assigned during core logging were found to be safely conservative, albeit not overly conservative.

Table 3. Calculated GSI and RMR values for rock types used in design

| No | Rock description | UCS | | RQD | | Joint spacing | | Joint condition | | Water | | RMR |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Value (MPa) | Rating | Value (%) | Rating | Value (/m) | Rating | Value | Rating | Value | Rating | |
| 1 | VSR | 1.0 | 1.0 | 20 | 3.0 | 0.10 | 8.0 | 4 | 10 | 2 | 10 | 27 |
| 2 | SR | 6.5 | 2.0 | 40 | 8.0 | 0.25 | 10.0 | 4 | 10 | 2 | 10 | 35 |
| 3 | MHR - HR | 17.5 | 2.0 | 80 | 17.0 | 0.50 | 10.0 | 4 | 10 | 2 | 10 | 44 |

The RMR values indicated above were adjusted by a discontinuity value of -5 to obtain the GSI values which were used as inputs to RocLab to generate the design values indicated in Table 4 below.

Table 4. Hoek-Brown strength parameters and stiffness values for design

| Parameters | Rock type | | |
|---|---|---|---|
| | 1 | 2 | 3 |
| Intact compressive strength (MPa) | 1.00 | 6.50 | 17.50 |
| mb parameter | 1.542 | 0.534 | 0.876 |
| ‘s’ parameter | 2.00E-04 | 3.93E-05 | 1.00E-04 |
| ‘a’ parameter | 0.538 | 0.522 | 0.512 |
| Erm (MPa) | 20.25 | 93.50 | 385.61 |

Where Erm is the rock mass deformation modulus obtained as an output from RocLab.

## 5 Design

A three-dimensional geological model was set up using the subsurface information from the array of boreholes drilled at the box-cut high wall position. Although only basic information such as rock hardness and weathering was incorporated in the model, it proved useful as a tool to generate cross-sections for design purposes

The lateral support design was primarily undertaken using finite element methods. The 30m high box-cut comprised two 3m wide benches separating the 12m high slope faces. The design comprised a series of Threadbar 500 soil nails, varying in length, and reinforced shotcrete. Soil nails were spaced at 1.5m centres with band drains installed vertically between the grids for pore water pressure dissipation.

## 6 Challenges encountered

A summary of the main challenges encountered during the planning, investigation, design and construction stages are succinctly discussed in the ensuing subsections with probable solutions provided at the end of each main challenge.

### 6.1 Planning and investigation - Terrain

Demanding topographical conditions such as boulders, trees, streams and steep terrain caused access complications during the investigation phase, especially with drill rig setups. Ultimately, these delays and claims had a pronounced negative effect on the final cost and programme of the investigation, even though a pre-tender site visit was conducted by all the relevant parties.

These situations should be anticipated prior to any fieldwork being conducted. It is recommended that an agreement should be reached by all affected parties on possible difficult setups and all other challenging situations that might impact the investigation phase. Furthermore, the drilling contractor should be appointed through a detailed drilling contract comprising this information. This should cover all parties involved and mitigate any unforeseen delays and claims in this regard.

### 6.2 Investigation - Poor drilling quality

Core recovery rates as low as 50% were attained in some drill runs. Typical cases include the washing of soils and, more importantly, completely weathered rock at geological structures and contact zones during drilling. This is usually affected by excessive drilling pressures either due to an inexperienced drilling team, lack of supervision and/or a stringent programme leading to attempts at accelerating production, etc.

Poor information from the drilling investigation such as core loss and low recovery rates should be prevented as far as possible. It is recommended that the drilling contract clearly specifies how payment is affected by core loss and poor drilling information. A reasonable percentage of core-recovery should be agreed upon at tender stage as a criterion for payment in conjunction with linear metres. This should encourage quality drilling instead of only quantity-driven performance.

The drilling contractor should also be made aware of the requirement that records must be diligently kept of drilling processes. This should be performed on approved drilling journals as this information, in conjunction with samples or core retrieved, is important to the design engineer, e.g. penetration rates, hammer action and water loss, etc. during percussion drilling.

### 6.3 Investigation – Orientated drilling

Various orientation techniques are available to orientate the core when retrieving it from an inclined borehole. During this investigation, it was discovered that, although the "clay-impression" technique works with fair success, it is sensitive to drillers' breaks and blunt breaks/joints in the core, e.g. joints between 70° to 90° from the axis of drilling. At these joint angles, the specific drill run cannot be orientated, causing critical design information to be lost.

Alternative orientation techniques should be applied such as the Ballmark® or EzyMark™ technology. Though expensive when compared to the overall cost of such an investigation, these methods are considered worthwhile given the value they add.

### 6.4 Scope creep

This term is given to the phenomenon where a project's scope of works is increased. Large changes in scope are easily identified and variation orders put in place. Oppositely, smaller seemingly insignificant changes occurring throughout the duration of a long project add up (without being individually identifiable) and cause budget and time constraints for the

consultant. This may be due to, for example, changes in the client's outcomes and requirements after the commencement of the project. It may also be due to unforeseen complications which necessitate a design alteration such as critical geological features encountered during investigation or construction phases, etc.
Mid-way through the drilling investigation on the access road, a portion of the horizontal alignment was slightly altered. This nullified some investigations that had already been conducted but, due to severe time constraints, the investigations could not be repeated on the new layout. The existing information from the previous locations had to be extrapolated to the new layouts and confirmed during construction as the cutting excavation proceeded. This could easily have led to redesigns and unsafe working conditions had the geology not been more forgiving.

If a project's scope of works is altered in any way, its ultimate financial and contractual impact on the project needs to be determined. This should then be discussed with the client in order to agree upon a mitigating solution/strategy prior to any additional work being conducted.

## 6.5 Design programme

Designs that are rushed may be less cost-effective and ultimately result in unsafe working conditions and/or casualties in extreme cases. A delayed investigation programme will logically impede the analysis and design programme. If this cannot be reasonably managed, the client needs to be notified of delays and permission for extension of time needs to be obtained.

Although the design programme for the box-cut and other cuttings was slightly impinged upon by delays in the drilling/investigation phase, internal and/or external reviews confirmed it to be safe and suitable. Design conditions that had to be extrapolated are constantly being verified against actual site conditions as construction proceeds and the design solutions are adapted as required.

The only design adaptation to date was an increased soil nail spacing in the localised area indicated in Figure 2 as better quality material was encountered than that assumed for the original design. This was possible due to good site supervision and construction monitoring practices.

Figure 2. Very hard rock with little jointing at the box-cut where soil nail spacing was increased

## 6.6 Construction supervision

Several cuttings on the access road were fully excavated (up to 10m deep) prior to installing lateral support measures as indicated in Figure 3. This might have been done in an attempt to accelerate the programme, in which case it wrongly took precedence over the stability of the cutting.

Figure 3. Cutting on access road excavated to some 10m deep

In unstable conditions (e.g. soil and completely weathered rock), it is recommended that such cuttings and support measures should rather be advanced in 2m lifts in a top-down manner. Regulation number 3(1) of the Engineering Professions Act, 2000, on the topic of competence when work is carried out, mentions that all persons must engage in and adhere to acceptable practices. This means that the design must be adhered to unless otherwise approved by the designer.

This topic becomes critical in the event of injury or loss of life due to improper construction practices, whereby the contractor will be held liable if the design was not adhered to by him. Similarly, construction supervision should be carried out by a competent, qualified and appointed person to ensure that the design is adhered to in a safe and correct manner (Act No. 46 of 2000). It is also considered good practice for the design engineer to visit the site regularly and familiarise him/herself with the construction process in a consultative capacity.

## 7 Conclusions

The box-cut and access road cuttings at Northam Platinum Limited’s new Booysendal Central Complex development were used as a case study to highlight certain challenges that a young geotechnical engineer (YGE) may encounter in a similar project. These challenges originate and range from the planning, investigation, analysis and design, and construction phases. Possible solutions for the challenges were provided at the end of each subsection of section 6. The list of challenges highlighted in this report are non-exhaustive.

## References

Board notice 15 of 2006, Rules of conduct for registered persons: Engineering Profession Act, 2000 (Act No. 46 of 2000).

# The Viability and Feasibility of Using Recycled Concrete and Masonry Aggregates in the Pipe Laying Industry

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## Abstract

This research explores the utilization of recycled aggregates in the South African pipe laying industry; where pipeline life spans are not realized due to the excess cost of sourcing adequate pipe bedding materials as stipulated by the SABS: SANS 1200LB. The viability of using recycled aggregates in the pipe laying industry was determined, and a cost comparison was made between implementation of the SABS versus the USA standard. It was found that, with minor processing, recycled aggregates can be utilized instead of virgin/commercial sources at a fraction of the cost. The utilization of recycled aggregates over commercially supplied aggregates can save contractors between 45–76% on material alone, a further 43% saving if adopting the USA standard, and a further 25–35% saving if blending the recycled aggregate with on-site material. Blending on-site material with recycled aggregates also increases their CBR strength in turn further promoting the use of recycled aggregates on site.

***Keywords:*** *Builder’s Rubble, RCA, RMA, Pipe Bedding, SANS 1200LB.*

## 1 Introduction

The current South African standard for pipe bedding materials is the South African Bureau of Standards (SABS): Standardized Specification for Civil Engineering Construction LB: Bedding (Pipes) published in 1983 (SABS, 1983). This standard characterizes the type of bedding and fill material used during the construction of different pipe types – rigid and flexible. The focus of this research is on the materials to be utilized in the laying of flexible pipes. Three classes of material are used during the laying of flexible pipes, namely: 1) selected granular fill; 2) selected fill; and 3) main fill material. It is noted, for clarification purposes, that for flexible pipes the bedding is the selected granular fill material. The standard contains stringent specifications that each of these classes of material must follow in order to be suitable for use in the laying of flexible pipes.

In South Africa many of these pipelines are constructed in rural areas far from infrastructure and sources of construction material. As such, the classes of material for pipe laying have to be sourced locally either from pipe excavations themselves, borrow pits or quarries. The ideal situation – from a feasibility point of view – is that the material be sourced directly from pipe excavations. The next alternative would be to have conveniently located borrow pits or quarries within feasible haulage distance to the pipeline route. Sadly these ideal situations are often not the case in many parts of South Africa and given the stringent SABS specifications the only solution is to find the nearest quarry and haul the material required for the laying of the pipeline. This is not feasible and often the contractor opts to use the excavated material as selected granular and selected fill material for the pipeline. The issue with this is that the pipe does not get the support that it requires and is susceptible to deflection which causes premature cracking and ultimate failure of the section of pipe (Venter, 2008). A study conducted by Windapo *et al* (2013) into the South African construction industry revealed that one of the key challenges that contractors face is the increasing costs of building materials. This coupled with the ever growing demand for competitive tenders - due to a sluggish economy after the global economic crises of 2008 (Windapo, *et al*, 2013; PWC, 2015) – leads contractors to source the cheapest material accessible; often to the detriment of the environment and to the premature failure of infrastructure.

The aim of this research is to look into the utilization and feasibility of using recycled concrete aggregate (RCA) and recycled masonry aggregate (RMA) as a replacement to *in situ* soils or as a blended material to enhance the *in situ* material for placement as selected granular and selected fill material. Alternatively – when the *in situ* material is of such poor quality – the research aims to investigate whether the recycled aggregates can serve to replace material sourced from a commercial quarry or borrow pit and negate the need for sourcing greenfield sites for exploitation. In so doing the research also looks into the feasibility of using these recycled aggregates and how this can promote the development of recycling depots which may provide jobs whilst preserving the natural environment.

## 2 Pipe Bedding Standards

For comparative purposes both the SABS and the United States of America (USA) pipe bedding standards have been used.

### 2.1 SABS 1200 LB: Pipe Bedding

SABS (1983) highlights three classes of material utilized during pipeline construction: 1) selected granular fill; 2) selected fill; and 3) main fill material. These materials may either be sourced from pipe trench excavations or from a suitable borrow pit or commercial quarry source located along the pipeline route. For cost saving purposes these sources of materials need to be conveniently located along the pipeline route. The three classes of material are applicable to both classes of pipes (rigid and flexible). The material specifications for the laying of flexible pipes are discussed in the sections following.

#### 2.1.1 Selected Granular Fill

SABS (1983) states that selected granular fill must be of a granular, non-cohesive nature (i.e. little to no PI and < 5% fines) that is singularly graded (≥ 90% of material) between 0.6mm and 19mm sieve size (Figure 1a), is free-draining and has a compactibility factor not exceeding 0.4. This material type acts as both the bedding on which flexible pipes are laid and the material in which they are embedded (Figure 1b).

#### 2.1.2 Selected Fill

Selected fill must have a PI not exceeding 6; it must be free from vegetation and from lumps and stones with diameter greater than 30mm (SABS, 1983). This material acts as the compacted selected fill blanket that overlies the pipe bedding cradle (Figure 1b). After completion of the bedding cradle, 100mm layers of selected fill material are compacted at 90% modified AASTHO until the full height of the selected fill material is situated at least 300mm above the crown of the pipe i.e. a minimum 200mm thick layer of selected fill material above the bedding cradle (SABS, 1983).

#### 2.1.3 Main Fill Material

Main fill can be sourced directly from the excavated trench. The SABS standard does not give any strict requirements for this material, however it is generally accepted that this material should not comprise of rock or stone fragments in excess of 300mm diameter nor shall it comprise of excessive vegetation/ organic matter. This layer should be placed in 150mm layers above the selected fill blanket and compacted at 90% modified AASTHO until the entire trench has been filled.

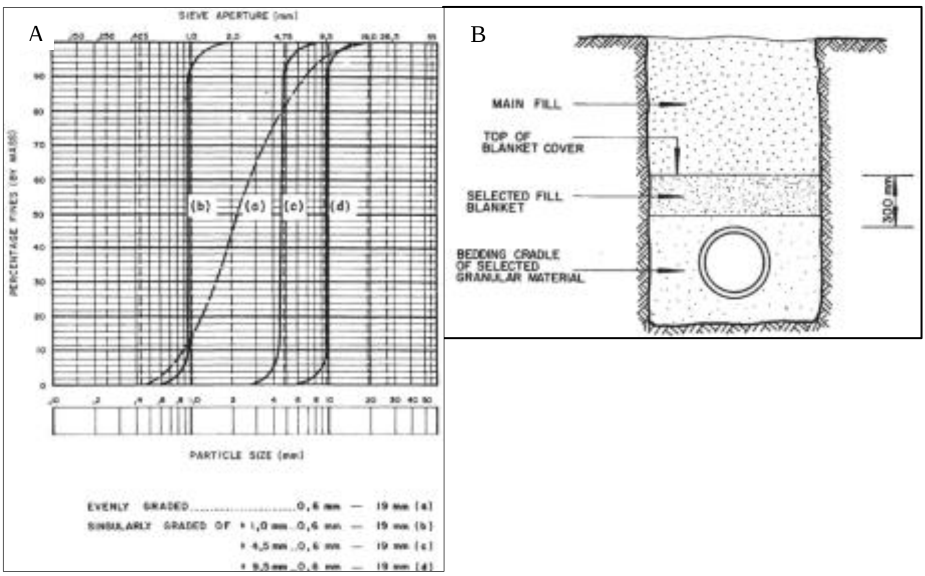


Figure 1. SANS 1200LB a) Grading curves for selected granular material; and b) Pipe bedding details – flexible pipes (SABS, 1983)

### 2.2 USA Standard: Pipe Bedding and Backfill

The most recent edition of the pipe bedding and backfill standard used by the USA was published by the US Department of the Interior Bureau of Reclamation by Amster K. Howard in 1996. The standard forms part of a geotechnical manual and is quite comprehensive as compared to SABS 1200LB. Nevertheless many of the key characteristics are similar with only minor differences that make the USA standard less stringent than the SABS.

#### 2.2.1 Differences and Similarities to SABS 1200LB

As with SABS the USA standard stipulates three types of material used in the laying of pipelines, however they are termed slightly differently: 1) bedding; 2) embedment; and 3) backfill. These terms represent the SABS materials: 1) bedding (selected granular fill for flexible pipes); 2) selected granular fill; and 3) selected fill. The USA standard’s backfill and SABS selected fill are much the same. SABS 1200LB further defines a main fill material.

#### 2.2.1.1 Bedding Material

One of the differences between the SABS and USA standards is the distinction between foundation and bedding. What the SABS standard terms bedding (with particular reference to rigid pipes) is what the USA standard covers under foundations. Bedding in the USA standard refers specifically to the material laid upon the foundation and on which the pipe is laid (Figure 2a). The bedding layer does not have specified requirements only that it must have a fine grading and be placed as a 100mm un-compacted layer in which the pipe may settle. In the SABS standard; bedding and foundation are synonymous with regards to flexible pipes and the foundation on which the pipe is constructed incorporates the bedding.

**2.2.1.2 Embedment Material**

The material used for embedment, similarly to the SABS standard, must be cohesionless; free-draining with less than 5% fines material and have a maximum size range of 19mm (Farrar, *et al*, 1998). The one difference is that the USA standard does not stipulate/confine the embedment soil to a singular grading between 19mm and 0.6mm as with the SABS standard, instead it limits the embedment soil to a maximum of 25% soil passing the 0.3mm sieve. So in essence the USA standard's specification for embedment soil is less stringent than SABS specification for selected granular fill. This means that *in situ* soils are more realistically viable for use in pipeline construction as the USA standard allows a higher percentage of finer grains in the embedment material.

In addition to the above; the USA standard requires much less embedment cover than the SABS. The USA standard requires an embedment layer with a thickness of 70% the diameter of a flexible pipe (Figure 2b) and only 37% the diameter of a rigid pipe. Whereas the SABS standard requires that the full diameter of the pipe and a minimum of 100mm above the crown of the pipe be covered in selected granular material. It is further noted that the USA standard does not specify a maximum plasticity index for this material, however it does mention that plastic clays and silts should be avoided. This statement coupled with the <5% fines is less stringent than the further specification from SABS that the <5% fines must not have a PI > 6.

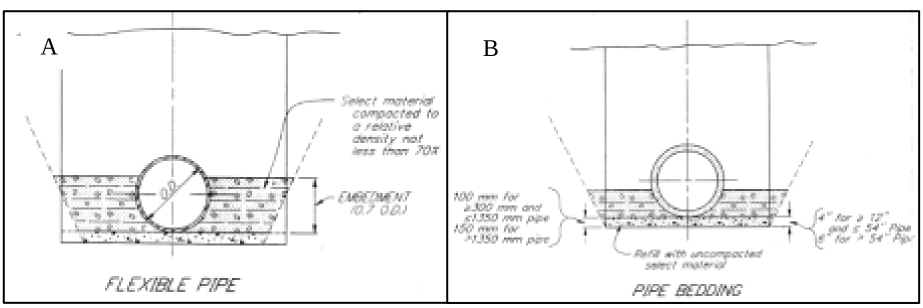


Figure 2. a) USA pipe bedding details; b) Pipe embedment details for flexible pipe (Howard 1996).

**2.2.1.3 Backfill Material**

Generally any material can be used as backfill according to the USA standard with the only limitation being size restrictions within a 300mm zone above and around the pipe (i.e. the layer immediately after placement of the embedment material). For PVC flexible pipe the size limit within this 300mm zone is a maximum size of 25mm and is equivalent to the SABS standard for selected fill material (maximum size of particles <30mm). Thereafter any material may be placed as long as individual particles are <450mm. Only where the pipeline will be beneath a road or other such crossings is an added specification included: that peat/organic material and clayey material should not be used as backfill. The SABS standard has two types of backfill: selected fill and main fill. The selected fill is much like the 300mm zone of material as per the USA standard; however with an added specification of not exceeding a PI of 6 and the maximum particle size is slightly larger at 30mm as compared to 25mm. SABS main fill material is much the same as USA backfill however the SABS particle size limit is 300mm as opposed to 450mm. Similarly no organic material or peat is allowed as backfill.

## 3 Experimental Methodology

In order to test the viability of RCA and RMA as selected granular fill or selected fill material in the pipe laying process - either as blended materials or as alternative sources to virgin materials - the following methodology was adopted:

- Obtaining a reference soil sample representative of typical conditions experienced on site and testing the soil characteristics in the laboratory with regards to the requirements as stipulated by SABS 1200LB,
- Laboratory testing to include Foundation Indicator testing (Grading; Atterberg limits and hydrometer analysis), Compactibility and Modified AASTHO and CBR testing,
- Subjecting locally sourced RCA and RMA products to the same testing parameters as with the reference soil (i.e. 100% recycled aggregate samples), and
- Blending the recycled aggregates in two ratios (50:50 and 70:30 blends) with the reference material and analyzing whether improvements were made to the reference material with regards to SABS 1200LB requirements.

Two Cape Town producers of RCA and RMA were chosen to supply the aggregates. The material supplied included the following: 1) 19mm RCA, 2) 19mm RMA, 3) 16mm RCA, 4) 13mm RMA, and 5) 6mm RMA. The laboratory testing schedule is highlighted in Table 1.

Table 1. Laboratory testing schedule.

| Sample ID | Mix Ratio | Tests Conducted |
|---|---|---|
| | **19mm RCA** | |
| A1 | 100% | FI, Compactibility |
| A2 | 50:50 | FI, MOD & CBR, Compactibility |
| A3 | 70:30 | FI, MOD & CBR, Compactibility |
| | **19mm RMA** | |
| A4 | 100% | FI, Compactibility |
| A5 | 50:50 | FI, MOD & CBR, Compactibility |
| A6 | 70:30 | FI, MOD & CBR, Compactibility |
| | **16mm RCA** | |
| A7 | 100% | FI, MOD & CBR, Compactibility |
| A8 | 50:50 | FI, MOD & CBR, Compactibility |
| A9 | 70:30 | FI, MOD & CBR, Compactibility |
| | **13mm RMA** | |
| A10 | 100% | FI, Compactibility |
| A11 | 50:50 | FI, MOD & CBR, Compactibility |
| A12 | 70:30 | FI, MOD & CBR, Compactibility |
| | **6mm RMA** | |
| A13 | 100% | FI, MOD & CBR, Compactibility |
| A14 | 70:30 | FI, MOD & CBR, Compactibility |
| | **Reference Material (Clayey Silt)** | |
| A15 | 100% | FI, Compactibility |

# 4 Results

## 4.1 Laboratory Test Results

Table 2 includes relevant laboratory test data. This table highlights in grey the failing criteria with regards to selected granular fill and selected fill material as per the SABS and in asterisk's the failing criteria for embedment material as per the USA standard. Figure 3 overleaf highlights grading curves relative to SABS and USA requirements.

Table 2. Laboratory test results.

| Sample ID | Ratio | Compactibility Index | %Particle Size (mm) | | | | PI | Classification |
|---|---|---|---|---|---|---|---|---|
| | | | 19 | 0.6 | 0.3 | 0.075 | | |
| 19mm RCA | | | | | | | | |
| A1 | 100 | 0.16 | 93 | 16 | 16 | 4 | NP | A-1-a, GW, G7 |
| A2 | 50:50 | 0.38 | 96 | 46 | 41* | 31* | 13 | A-1-b, SC, G7 |
| A3 | 70:30 | 0.43 | 90 | 38 | 33* | 25* | 12 | A-1-b, GC |
| 19mm RMA | | | | | | | | |
| A4 | 100 | 0.16 | 90 | 28 | 19 | 6* | NP | A-1-a, GW |
| A5 | 50:50 | 0.28 | 94 | 49 | 43* | 33* | 13 | A-1-b, SC, G9 |
| A6 | 70:30 | 0.41 | 92 | 39 | 34* | 24* | 10 | A-1-b, SC, G9 |
| 16mm RCA | | | | | | | | |
| A7 | 100 | 0.22 | 100 | 26 | 19 | 9* | NP | A-1-a, GM, G4 |
| A8 | 50:50 | 0.28 | 100 | 45 | 38* | 29* | 11 | A-1-b, SC, G7 |
| A9 | 70:30 | 0.32 | 100 | 36 | 28* | 17* | 6 | A-1-b, SC, G5 |
| 13mm RMA | | | | | | | | |
| A10 | 100 | 0.24 | 100 | 3 | 3 | 1 | - | A-1-a, GW |
| A11 | 50:50 | 0.32 | 100 | 41 | 35* | 26* | 12 | A-1-b, SM, G9 |
| A12 | 70:30 | 0.40 | 100 | 31 | 27* | 20* | 12 | A-1-a, GM, G7 |
| 6mm RMA | | | | | | | | |
| A13 | 100 | 0.37 | 100 | 33 | 20 | 7* | NP | A-1-b, SM, G5 |
| A14 | 70:30 | 0.36 | 100 | 36 | 25 | 11* | NP | A-1-b, SM, G4 |
| Reference Material (Clayey Silt) | | | | | | | | |
| A15 | 100 | 0.54 | 100 | 74 | 65* | 50* | 15 | A-7-5(3), CL |

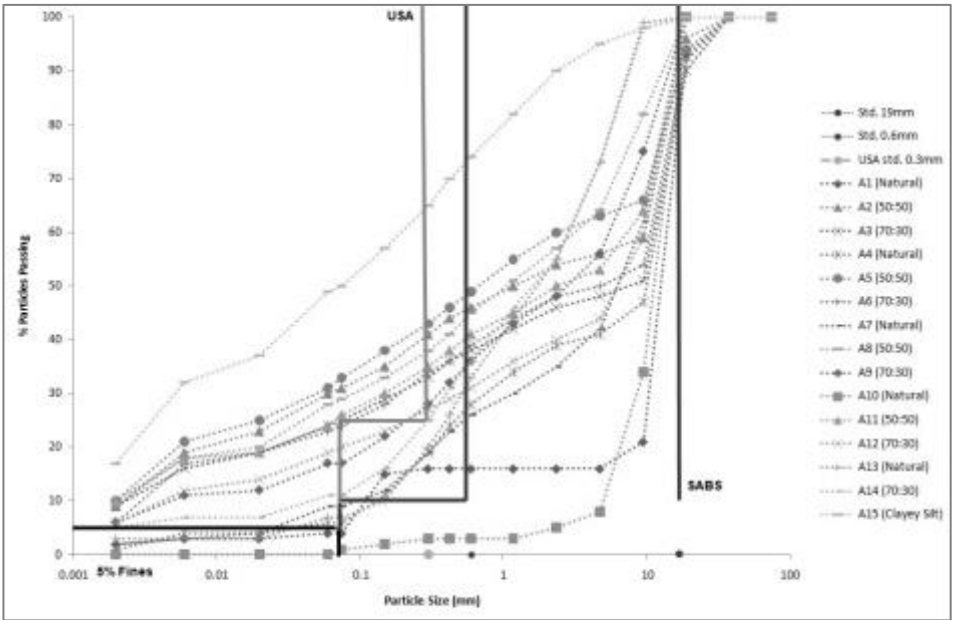


Figure 3. Grading curves indicating particle size distribution for recycled aggregates, blends and reference sample.

## 4.2 Feasibility Studies

A theoretical pipeline situation was used for feasibility studies highlighting the cost comparison between utilizing natural aggregate from a quarry or a hardware store and then with using recycled aggregates from the two local suppliers. The pipeline envisioned was a 10km long pipeline route comprising of a 300mm diameter flexible PVC pipe to be buried 1.3m below existing ground level. Table 3 indicates aggregate costs and four scenarios: 1) utilizing 100% aggregate assuming site has no suitable material for either selected granular fill or selected fill material; 2) utilizing the USA standard instead of the SABS standard; 3) utilizing the 70:30 ratio according to SABS standard; and 4) utilizing the 70:30 blend according to the USA standard.

For clarification it is calculated that the contractor would require 7400m$^3$ of aggregate material for both selected granular fill and selected fill material requirements in adopting the SABS standard, whereas for the USA standard; the contractor would only require 4240m$^3$ of embedment material. And should the contractor opt for a 70:30 blend they will require 25% less material (i.e. 5550m$^3$) if using the SABS standard and 35% less material (i.e. 2756m$^3$) if using the USA standard.

Table 3. Recycled and virgin/ commercial aggregate prices as compared in four scenarios.

| Supplier | Material | Aggregate Size | Cost (R) inc. VAT* |
|---|---|---|---|
| **Aggregate Prices** | | | |
| CDEL | RMA | 13mm | 240.00 p/m$^3$ |
| Quarry | Virgin Hornfels | 13mm | 560.00 p/m$^3$ |
| Hardware Store | Virgin Hornfels | 13mm | 935.00 p/m$^3$ |
| **Situation 1: Utilizing 100% Aggregate (SABS Standard – 7400m$^3$)** | | | |
| CDEL | RMA | 13mm | 1, 776, 000 |
| Quarry | Virgin Hornfels | 13mm | 4, 144, 000 |
| Hardware Store | Virgin Hornfels | 13mm | 6, 919, 000 |
| **Situation 2: Utilizing 100% Aggregate (USA Standard – 4240m$^3$)** | | | |
| CDEL | RMA | 13mm | 1, 017, 600 |
| Quarry | Virgin Hornfels | 13mm | 2, 374, 400 |
| Hardware Store | Virgin Hornfels | 13mm | 3, 964, 000 |
| **Situation 3: Utilizing 70:30 Blend (SABS Standard– 5550m$^3$)** | | | |
| CDEL | RMA | 13mm | 1, 332, 000 |
| **Situation 4: Utilizing 70:30 Blend (USA Standard– 2756m$^3$)** | | | |
| CDEL | RMA | 13mm | 661, 440 |

*Prices correct at time of writing. Prices exclude haulage and additional costs such as screening.

## 5 Discussion

Laboratory data has been utilized in the construction of grading curves denoted by Figure 3. Simple constructions on Figure 3 highlight the determining criteria for the SABS and USA standard for selected granular fill (SABS) and embedment material (USA). It is clear from observations that the pure recycled aggregate samples, with the exception of sample A10 (13mm RMA), do not pass the SABS criteria for selected granular fill; however these samples do pass the criteria for selected fill material. It is clear that the detrimental factor of these materials, with regards to selected granular fill, is that too much material passes the 0.6mm sieve. It is suggested however that should these recycled aggregates undergo a single stage of screening before utilization (to remove all material passing 0.6mm sieve) that they will be more than adequate for selected granular fill. With regards to the USA standard two samples (A1 and A10) pass for embedment material, the remaining materials fail by a small margin due to having >5% fines; however with washing/ screening these materials will all pass the standard for embedment material. Due to the USA standards less conservative requirements for backfill material (i.e. selected fill as per SABS) the on-site material will find use as backfill material.

As for the blended samples it is clear that blending the reference material with the recycled aggregates improves the reference material by a huge factor. However, due to the fine-grained nature and slightly high PI of the reference soil the blends fail both selected granular and selected fill material requirements as per the SABS standard as well as embedment requirements as per the USA standard. One can screen and wash the blends, however it is estimated that as much as 75% of the natural soil will be removed from this process for selected granular fill requirements. Blending may reduce the amount of aggregate required on site, however may increase overall project costs due to excessive screening and washing processes. However, due to the USA standard allowing up to 25% passing the 0.3mm sieve the blends are more likely to achieve these criteria after on-site screening at a more favorable cost.

Modified AASTHO and CBR tests provide a reference for contractors when compacting materials in pipe trenches and allow for quality control measures to take place. Test results indicate that recycled aggregates on their own are generally of G4 – G7 in quality and once blended with on-site clayey silt classified in the range of G4 to G9 in quality depending on the aggregate blended with. It is further noted that the on-site material is less than G10 in quality and by itself is considered very poor subgrade. Whilst this level of detail is not necessary for the actual pipe laying process it is pertinent to note that when on-site poor quality material is blended with recycled aggregates the level of improvement allows for greater utilization on site such as in foundation and road designs. As such the use of recycled aggregates on site has applicability on a multitude of levels.

Feasibility studies indicate that the contractor will spend between 2.3-3.9 times more on material costs if using quarry or commercial aggregates as compared to using recycled aggregates. Should the contractor opt to implement the USA standard they will make a further 43% saving on material due to the more relaxed embedment and backfill material requirements. Furthermore; should the contractor opt for the 70:30 blend they will save 1.3 times the amount on material instead of using 100% recycled aggregates with a further 50% saving if they opt for the USA standard.

## 6 Conclusion

This research promotes sustainable development for the benefit of communities, via provision of adequate infrastructure; the economy, via significant cost reduction; and the environment, via reduced waste; promotion of recycling and reducing reliance on non-renewable resources. This is in direct correlation with national policies and strategies which aim to reduce poverty and inequality, within a sustainable development framework (NDP, 2011).

The status quo in the construction industry at the moment is one of high growth, high costs and high maintenance issues. One of the largest limitations in terms of access to water supply and sanitation has been highlighted as ailing infrastructure and inadequate maintenance (Shand, 2013; Still, 2006). Challenges persist; as although communities directly benefit through the government injection of infrastructure development, this infrastructure is being rolled out at an alarming rate without much thought, or time, for innovation when challenges are met. Infrastructure is inefficient at best if it performs poorly, and communities are the first to suffer the consequences of poor planning. A sector that has seen much growth (according to the national drive for infrastructure (NDP, 2011)) is the construction industry, but a limitation to this growth at a local scale has been the economic collapse of 2008 (Windapo, *et al*, 2013; PWC, 2015). This has put significant pressure on local private contractors in terms of provision of services within tight national budgets - with local level implementers reducing pricing for projects in a bid to be competitive (CIDB, 2013). This is a recipe for disaster, in terms of provision of quality and sustainable infrastructure, as local decisions are being made to cut costs and cut corners in order to keep small companies afloat. Environmental legislation becomes an additional constraint in this regard as construction within rural areas usually triggers multiple environmental legislations from the Departments of Environmental Affairs (DEA); Water and Sanitation (DWS); and Agriculture, Forestry and Fisheries (DAFF). There are obvious challenges for sustainable development in the construction industry, but what this study provides is the necessary research and development needed to make informed decisions about alternative options on site – with emphasis on the pipe laying industry.

Sustainable development has seen an increasing support base from both the local and national level as it is seen to provide real benefits for all. In particular the National Environmental Management: Waste Amendment (NEM:WA) act of 2008 promotes waste minimization and recycling. Local municipalities are encouraged to come up with unique and innovative ways to reduce their waste production and limit the use of landfill airspace. It is estimated that, in the Western Cape alone, up to 14% of waste to landfill comprises of builder's rubble. This research outlines the availability and usefulness of recycled builder's rubble in the form of recycled concrete and recycled masonry aggregates (RCA and RMA) and their potential utilization in the pipe laying industry. In the South African pipe laying industry contractors are required to use the standards as outlined by the SABS: SANS 1200LB for pipe bedding purposes. From experience it is generally noted that this standard is so stringent that the material required for this purpose has to be sourced from a commercial quarry at huge expense. In South Africa many of these pipelines are situated in rural areas where quarries or borrow pits have to be established for this purpose, and due to huge costs this is often neglected and the contractor is forced to use on-site materials which may be totally inadequate for pipe bedding purposes. As such the full lifespan of the pipeline is not realized - at a great cost to the local municipality and all affected consumers. For comparative purposes the SABS standard has been compared to the pipe bedding standard adopted by the USA. The USA standard is more lenient and, in the author's opinion, more practical in a South African context.

If the huge cost element is removed from the equation and an alternative, cheaper material source is readily available for utilization in the pipe laying process; it is believed that local municipalities would be in a financial position to support better quality pipe installation. The conclusion for this research is thus: recycled aggregates can make for adequate pipe bedding material – in terms of both the SABS and USA standards - in the pipe laying industry at a much more affordable rate as compared to virgin/commercial sources. And by making use of RCA and RMA in the construction industry the recycling of builder's rubble is promoted not only creating jobs but also adhering to the NEM:WA act of 2008.

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# Case Study and Lessons Learnt: The Construction of Mechanically Stabilised Earth Walls at Mount Edgecombe Interchange

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## Abstract

The use of Mechanically Stabilised Earth (MSE) Walls were opted for at the Mount Edgecombe Interchange as a result of economic considerations and horizontal space constraints. Unconsolidated founding conditions as well as the mere extent of the MSE Walls called for ground improvement of the walls' foundations as well as several design reviews of both the internal and external wall stability. During construction of these walls, practical applications of the tolerances specified in SANS 207 were developed and applied as part of the quality assurance plan. The backfill of the MSEW needed to be compacted against a temporary soil nail wall, supporting 10m of cut, which resulted in potential differential settlement and drainage challenges. This paper discusses the measures that were taken to mitigate the above potential concerns and additionally addresses construction plant and material access, as well as other constructability challenges which were resolved through astute programming, surveying as well as trial and error.

***Keywords:*** *Mechanically Stabilised Earth (MSE) Walls, Geosynthetics, Reinforced Soil Walls*

## 1 Introduction

Mount Edgecombe Interchange is a four-level freeflow interchange at the juncture of the N2 and M41 in Umhlanga, Kwa-Zulu Natal. The construction of the interchange commenced in April, 2013, and forms part of the South African National Road Agency's (SANRAL) upgrade of National Route 2, Section 26, from Mount Edgecombe Interchange to Tongaat Toll Plaza.

The upgrade of Mount Edgecombe Interchange included the construction of two new bridges constructed with the incremental launching method (ILM) as well as the construction of the approach and exit ramps to the abutments of these viaducts.

The design of the exit ramp to the top level viaduct (ILM BD) proved to be a challenge due to the horizontal space constraints caused by the skew angle at which the intersection was originally built, i.e. the M41 and N2 are not perpendicular to one another. This skew angle of the interchange is shown in Figure 1. The close proximity of existing developed areas, such as Mt Edgecomebe Estate and The Crescent Shopping Centre, further restricted space adjacent to the viaducts. As a result of the horizontal space constraints, as well as the existence of a protected wetland area adjacent to ILM BD's exit, the implementation of back to back Mechanically Stabilised Earth Walls (MSEWs), as opposed to conventional sloped embankments, was selected to tie the viaduct into the existing N2 freeway.

Figure 1. Mount Edgecombe Interchange configuration

## 2 Geology

The site is characterised by alluvial and aeolian deposits of the Berea Formation, which is of Quaternary Age (10 – 30 Ma years). The Berea Red Sands are underlain by mudstone, siltstone and sandstone of the Ecca Group at a depth exceeding 50m below natural ground level.

The geology at the location of the back to back MSE Walls for ILM BD is further characterised by a stream which originates from the wetland in the south eastern corner of the site. Variable ground conditions typify the geology of this area, with alluvial deposits comprising layers of soft and stiff clays as well as intermittent boulder layers, to depth. A high water table at approximately 2m below the original surface further describes founding conditions of the back to back MSE Walls.

## 3 Design

An aerial photograph which shows an in-progress view of the back to back MSE walls is displayed in Figure 2. MSEW 6 is the wall adjacent to the wetland area and MSEW 7 is the wall adjacent to the freeway. Ramp BD constitutes both the deck of the incrementally launched bridge (ILM BD) as well as the ramp along MSE Walls 6 and 7 until it merges with South Bound Carriageway of the N2, several hundred metres south of the interchange. The length of the ramp supported by these MSE walls is 330m and the average height of MSEW 6 over this length is approximately 10m, with a maximum height of approximately 16m at the abutment to ILM BD.

### 3.1 External Stability Design

External stability design verifications include:

a) Bearing and tilt failure (ultimate bearing capacity check),
b) Sliding along base,
c) Long-term Settlement,
d) Circular/Global slip failure.

Both bearing capacity and circular slip failure checks indicated that additional ground improvement measures were required below the foundation of MSE Wall 6. An additional concern was the potential differential settlement expected at the abutment interface of MSEWs 6 and 7 and the deck of ILM BD. As a result, foundation improvement was specified under the high fills of the two MSE walls. Foundation or soil improvement consisted of Replacement Stone Columns installed with the Dynamic Compaction (DC) method in combination with a G6 raft and high strength bi-directional geotextile for areas where the wall height of MSEW 6 exceeded 7m. Where the wall height varied between 4m and 7m, only a G6 raft was specified and no foundation improvement was specified at wall heights of less than 4m to limit differential settlements between the height ranges as per SANS207:2006. The external stability checks and finite element analyses were reviewed by a peer review process.

Figure 2. Aerial photograph of MSE Walls 6 and 7

### 3.2 Internal Stability Design

According to COLTO, which was the prescribed Standard Specification for the project, the proprietary product designer is responsible for the internal stability of the MSE wall. In this case, the walls were designed according to the “Tie-back Method” as described in SANS 207:2006 with the contractor’s preferred system being a Macres T system in conjunction with polyester Paraweb grids. The two local stability checks included in this design method are the rupture check which verifies the required strength of the reinforcement in the fill layers and the adherence check which ensures that the reinforcement will not pull-out behind the wedge.
The internal stability was reviewed in accordance to SANS207:2006 by SMEC as part of their quality assurance procedures.

## 3.3 Shoring

A temporary soil nail shoring wall was constructed along the N2 southbound carriageway to allow for the construction of the stone columns, G6 raft and MSE wall. The Contractor deemed this to be the best solution to contend with the various challenges related to the construction of the stone columns and wall fill, including the close proximity of the off ramp to the existing N2 southbound carriageway, as well as the deep founding level of MSEW 6, which lies approximately 11m below the top of road level of the freeway. The soil nail wall was designed by the contractor and reviewed by SMEC, allowing for expected vibrations induced by the dynamic replacement. Additionally, the extensive height of MSEW 6 dictated reinforcement/strap lengths of up to 11m along the bottom panels of the wall which in turn created the need for space to install these reinforcement lengths sufficiently. Figure 3 details a cross section of both MSE walls, as well as the temporary soil nail wall which reaches heights exceeding 10m in certain areas. The wall was constructed at an angle of between 10 and 15° to the vertical.

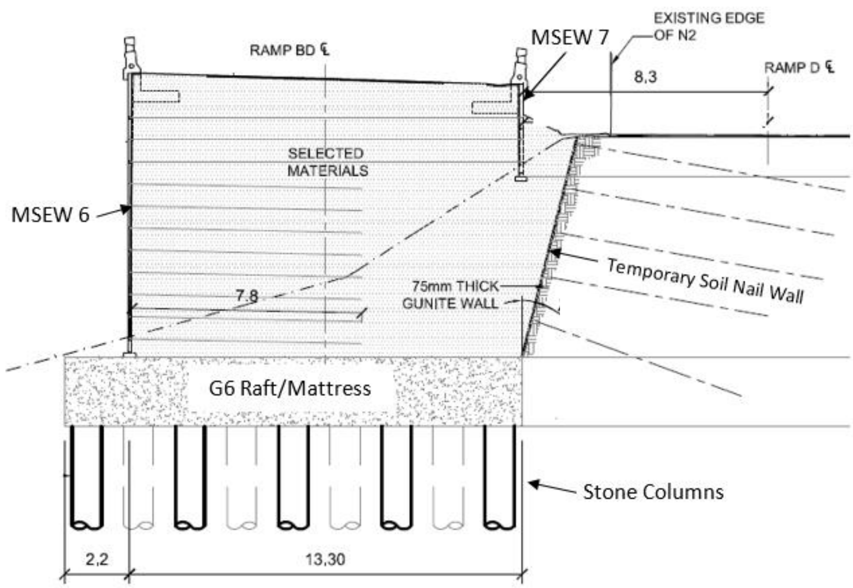


Figure 3. A typical cross section of MSE Walls 6 and 7 as well as the temporary soil nail wall

# 4 Construction and Site Related Challenges

## 4.1 Mechanically Stabilised Earth Wall Construction Tolerances

As stipulated in SANS 207, reinforced soil structures deform during construction. Given that deformation is expected during construction, primarily due to fill settlement, strain and long-term creep of reinforcement material and foundation settlement, an on-going means of monitoring was required to ensure that the MSE walls were erected according to the construction tolerances detailed in SANS 207 as shown in Table 1.

Table 1. SANS 207:2006, Construction tolerances for reinforced soil structures.

| Location of plane of structure | Tolerance ±50mm |
|---|---|
| Verticality | ± 5 mm per metre height (i.e. 40 mm per 8 m) |
| Bulging (vertical) and bowing (horizontal) | ± 20 mm in 4.5 m template |
| Steps at joints | ± 10 mm |
| Alignment along top (horizontal) | ± 15 mm from reference alignment |

Survey monitoring was undertaken at 750mm fill lifts, corresponding to the vertical position of the reinforcement material connection behind the facing panel. An XY position of the panel centre would be taken and the offset from road centreline would then be determined from this position. This offset would in turn be overlaid and compared to the design offset of panel centre to the road centreline at the top of wall level. This check was indicative of the wall's overall verticality as well as horizontal alignment and served as an early warning system for panels not conforming to the construction tolerances specified above. Bulging was monitored by comparing the periodic offset readings taken during each 750mm lift. When it was found that a certain panel column was out of specification, it was agreed that the Contractor was to correct the panel of concern in the next lift and ensure that all panels were again within the required tolerances.

In addition to the survey monitoring system, the orientation, alignment, verticality and overall surface regularity of every individual panel were checked during mandatory quality inspections, using a straightedge and spirit level. Furthermore, steps between individual panels as well as the joint spacings between panels were inspected to ensure compliance with the construction tolerances indicated above and those provided by the designer of the proprietary product used for the construction of MSE walls 6 and 7. It should further be noted that the panels were placed at an initial 1 to 2% incline towards the road centreline. This allows for slight outward movement of the panel during compaction of the layerworks. The greatest movement of panels was noted when the fill layer comprising had to be reworked as a result of the layer failing to meet the required compaction densities.

## 4.2 MSE Wall/Shoring Interface

A significant challenge faced during construction of the back to back walls was to devise a means of tying the new ramp into the existing freeway and prohibit potential differential settlements, as well as preventing the potential for preferential failure surfaces developing. The initial and instinctive solution was to bench into the newly erected temporary soil nail wall which acted as support to the N2 southbound carriageway. However, as a result of the steep 10 to 15 degree incline of this wall, benching, even at was considered the steepest functional slope of 3:1, would result in systematic demolition of the shotcrete face and severe undermining of the temporary soil nail wall, which defeats the construction of such a wall in the first place. This would in turn result in both unsafe working conditions, as well as access constraints to achieve the required degree of compaction of the layer in question. The first solution considered was to commence benching halfway up the soil nail wall and used an excavator from the top to prevent undermining of the wall below. This solution was rejected due to the close proximity of the N2 freeway to the wall and the economic feasibility of removing and reinstating the slow lane of the southbound carriageway for a length of approximately 300m.

FHWA (2006) recommends constructing the shoring interface in such a way to increase shear along the interface by one of the following methods:

- Construct the shoring wall at a batter (>4° depending on stress) or as a stepped structure,
- Extend the upper MSE reinforcements over the top of the shoring wall,
- Provide a mechanical connection between the MSE wall and shoring wall components.

Battering has the effect that some vertical stress can be transferred into the shoring system and will help mitigate differential settlement.

The above interface was considered in finite element analyses and shown to be marginally satisfactory (FOS around 1.5). However, as an additional precautionary measure, methods as described in Wu and Helwany (1990) were implemented to alleviate potential bridge approach settlements. The solution which was implemented is displayed in Figure 4. The solution agreed upon entailed placing a 2.5m width of Rockgrid, a bi-directional polyester geosynthetic, along the length of the shotcrete, with 1m placed vertically and the remaining 1.5m laid horizontally on top of the layer. The geosynthetic material was held in place by dowelling lengths of Y10 rebar through the top of the geosynthetic and into the face of the shotcrete wall. The next fill layer was then placed and worked up to the rough shotcrete wall face, thereby ensuring a degree of interlock. These “strips” of geosynthetic material were placed continuously up the wall at 2m height intervals

Figure 4. A typical cross section of MSE Walls 6 and 7 as well as the temporary soil nail wall

## 4.3 Drainage during construction

During the construction of the fill layerworks, wet spots were noted along the face of the shotcrete wall. This was attributed to the downward slope of 2-4% of the layerworks, to ensure runoff away from the MSEW panels; the wet spots were further attributed to seepage from the band drains which had been installed behind the face of the temporary soil nail wall. The solution, as photographed in Figure 5, consisted of a rudimentary subsoil drain, similar to those use behind the abutments to structures. A subsoil drain pipe, wrapped in bidim, was placed on a small concrete foundation along the length of the entire soil nail wall at a longitudinal slope of between 0.5 and 1%. This pipe then drained into a stormwater pipe inlet at the northern end of the wall which in turn drained into the attenuation dam/wetland area. Additionally, the positions of the band drains behind the shotcrete face were located, exposed and wrapped into the subsoil drain pipe to allow for sufficient drainage of hydrostatic build up behind the soil nail wall.

Figure 5. Drainage of layerworks and temporary soil nail wall band drains

### 4.4 Design changes

Given the magnitude of MSE Wall 6, in terms of both height and length, it was subdivided into several platforms along the length of the ramp. The founding level of the platforms increased in height towards the southern end of the ramp, as a result of the elevation increase in natural ground level in this direction. The Contractor's programme and progress did not allow for construction of the wall to commence at its lowest point, namely at the abutment to the ILM structure, and as a result the Contractor was forced to commence with construction on a one of the central platforms along the wall. The Contractor placed the panels at an approximate 20mm joint spacing which was within the construction tolerance of 23±5mm. However, once the construction of the wall commenced along neighbouring platforms, it was found that the length of panels stopped short of the design end point, as a result of reduced joint spacing over a significant length of the wall. This occurred in both northerly and southerly directions, towards both the bridge abutment (north) and an adjacent stormwater pipe (south).

A revised drawing was issued which incorporated the placement of "special" panels at both ends. These panels had to be manufactured with dimensions to suit the situation on site. Additionally, two construction joints in the face of the MSE wall were incorporated into the design on either side of the stormwater pipe to allow for the new panel positions as well as to allow for possible differential settlement between the fill above the concrete encased pipe and the neighbouring fill, founded on a selected G6 material. Figure 6 shows two sections of the design elevation of the wall, including the special panels required at both the abutment and stormwater pipe.

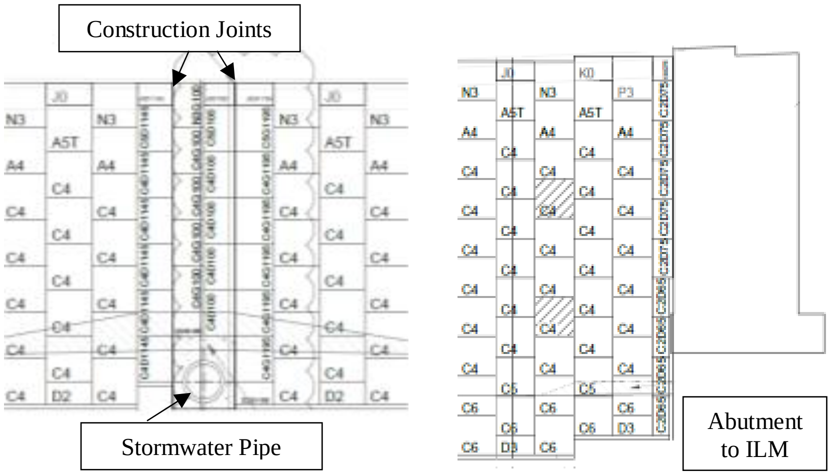


Figure 6. Extract of design elevation drawings of MSE Wall 6

### 4.5 Material and plant access

MSE Wall 6, as shown in the design elevation above, originates at the abutment to ILM BD and extends for approximately 240m until it butts up to the barrel of the Mount Edgecombe Drive Underpass. It further continues over the roof of the underpass, before tapering to its end at the junction of Ramp BD with the N2 southbound carriageway. The presence of the bridge abutment and underpass barrel at the extremities of the wall created an access challenge for the Contractor, in terms of both selected material delivery, as well as access for construction plant and machinery. Ideally, the Contractor would erect the wall in “long runs”. This entails placing panels and Paraweb grids from the lowest platform, working the wall up in long runs – from one end of the wall, to the other. The fill material would be constructed in the same manner, restricting the number of layer tie-ins. However, due to the average wall height of approximately 10m between these two landmarks, and the significant height of the neighbouring freeway, construction of the wall in the form of “long runs” from the abutment to the underpass was not feasible.

The Contractor undertook to build the wall from the abutment southwards, stopping 35m short of the underpass to allow for access via the last platform. He continued to build MSE Wall 6 up to this point, until the ramp to the top of the fill behind the wall became too steep. At this point, back to back construction of the ramp had commenced at the northern end of MSE Walls 6 and 7. Once the walls were at the required level (deemed to be approximately 1m below top of road level on the adjacent freeway), a ramp was constructed over the top of MSE Wall 7, to allow for plant and material access from the top. A photograph of the bottom access ramp is shown in Figure 7.

Figure 7. Bottom access ramp between MSE Wall 6 and Mount Edgecombe Drive Underpass

### 4.6 Mechanism to secure top MSEW panels

The cast-insitu parapet detail for the top of the MSE walls on Mount Edgecombe Interchange is detailed in Figure 8. From this detail it is noted that the height difference between the top of the MSE wall and the bottom of the parapet foundation is approximately 1.0m. Due to the nature of the wall design and parapet detail, the final row of panels, at the crest of the MSE walls, comprise of alternating fixed and unfixed panels. The term “unfixed” referring to panels which do not have reinforcement strips and are not anchored in the fill behind them. The unfixed panels all have a height of less than 750mm.

During the casting of the concrete parapet foundation elevations, it was noted that the unfixed panels were being pushed forward by hydrostatic forces exerted by the concrete during casting, regardless of the wooden clamps aligning them to the fixed panels. Although the panels would not be dislodged, as they are keyed into neighbouring panels, some panel rotation was noted. After several trial and error attempts at anchoring the unfixed panels to the wall, without fixing them to the barrier foundations, an on-site mechanism was devised to ensure that the unfixed panels did not “kick” during concrete casting. It was essential to find a solution whereby the MSE walls and parapet foundations could move and settle independently of one another.

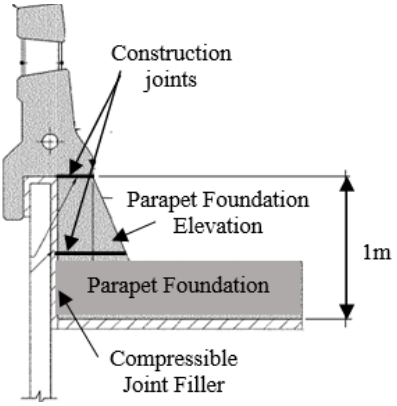


Figure 8. Parapet design detail

The mechanism, as shown in Figure 9, consists of an angle iron welded to a channel shaft. The angle slots in over the top of the MSEW panel. Two hooks were further welded onto a section of rebar which slots into one of three holes along the channel shaft. The welded hooks link onto the top of the parapet foundation elevation formwork and are tightened behind the shaft using butterfly screws. The three holes allow for varying MSEW panel sizes and ensure that no eccentricities exist when the hydrostatic force of the cast concrete is transferred to the anchorage system.

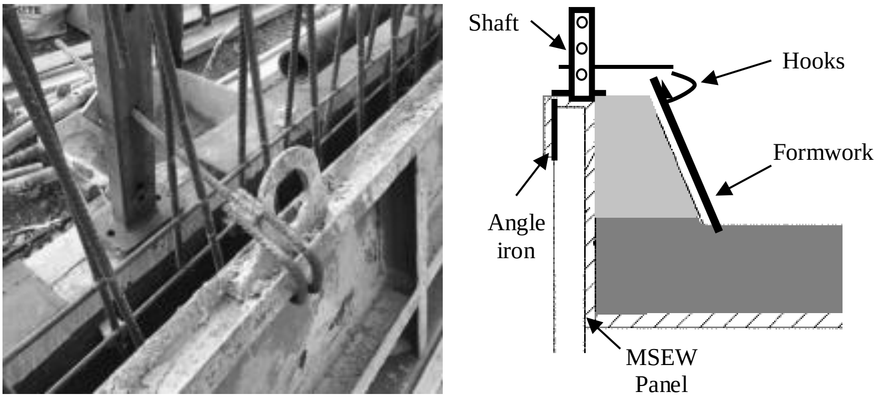


Figure 9. Mechanism used to secure MSEW panels during casting of parapet foundations

## 5 Conclusion

It is widely known that various factors may attributed to the successful construction of Mechanically Stabilised Earth Walls. This paper serves to highlight the importance of construction monitoring, good communication between all parties and innovative problem solving by site staff, the Contractor as well as the proprietary product designers to ensure the success of the end product. Quality control and quality assurance of both the layerworks construction, as well as panel and reinforcement installation, is essential to both an aesthetically pleasing and structurally sound wall.

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# Use of Crushed Concrete Aggregate Waste in Stabilization of Clayey Soils for Sub Base Pavement Construction

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## Abstract

The research aimed at stabilizing lateritic soils, using crushed concrete aggregates from demolished buildings, foundations, roads and other structures, for use as sub-base for a paved road. Lateritic soils were sampled along the Mukono-Jinja Highway from a borrow pit owned by Stirling Company LTD. Crushed concrete aggregate wastes were fairly angular and strong as they showed comparative values to the fresh aggregates as earlier researched. The lateritic soils were blended with different percentages of waste aggregates 0%, 30%, 40% and 50%, chosen basing on previous studies. The study looked at properties such as grading and flakiness of the waste aggregates, grading, atterberg limits, Optimum Moisture Content, Maximum Dry Density and 4 day soaked California Bearing Ratio for the stabilized and un-stabilized material. Mix designs with 40 % and 50 % of the waste aggregates were considered suitable for use as sub base material. They had CBR of 46 and 59, respectively, at 95 % relative compaction and PI values of 13.64 and 11.40. These met the specified standards of a CBR equal or greater than 45 and PI equal or less than 14 according to the general specifications of Ministry of Works, Housing and Communications (2004).

***Keywords:*** *Lateritic soils, Soil stabilization, Crushed concrete aggregate, Waste, Subbase*

## 1 Introduction

Use of crushed concrete waste is a process in which used-concrete is re-used for new construction. This is greatly due to the fast development of infrastructure of a country requiring huge amounts of construction material. This leads to many dangers to the environment including depletion of the natural aggregates, accidents in the quarries, noise pollution and the increasing rate of waste generated, (Abukettala, 2006). In order to reduce the usage of fresh aggregate and concrete wastes, recycled aggregates can be used as a stabilizing material in road construction. Stabilizing laterite soils, which do not meet the required engineering properties, with crushed concrete waste aggregates would increase the strength of the sub base. The major function of the sub base layer is to transfer wheel load to the subgrade, bear stresses occurring due to the wheel loads and resists wear due to abrasive action of traffic, (Tripathy & Mukherjee, 1997).

In this study, crushed concrete aggregate samples ranging from size 0.075 mm - 37.5mm mixed with lateritic soil was subjected to classification tests such as particle size distribution and plasticity, compaction and California Bearing Ratio tests to meet the required standards of the sub base layer. The results were be compared with those of the standard sub base material used in Uganda.

# 2 Materials and Methods

## 2.1 Lateritic Soil

The lateritic soil sample was obtained from a borrow pit along Mukono-Jinja road owned by Sterling Construction company. Random sampling was done at the borrow pit to obtain a representative sample. Shallow trial pits, dug by means of pick-axe and shovel, were utilized to study the soil profile in detail. Bulk samples were obtained from each pit for laboratory investigations. The sampling sites and sample specimen conditions are described below:

- Sample A: It was taken from the depth range of 0.2 – 1.0 m. It was light reddish brown, very dense sandy clayey gravel. It was underlain by yellowish brown, clayey gravel lateritic hardpan.
- Sample B: It was reddish brown clayey sandy gravel taken within a depth range of 0.3 – 0.8m. It was underlain by molted sandy gravelly clay.

According to AASHTO soil classification system, they were grouped as A-2-6(0), classified as gravels with clay good for sub base construction. A Group Index of zero (0) specifies gravel samples best for road construction. However, these gravels did not meet all the requirements for sub base construction according to the Ministry of Works, Housing and Communication (MWH&C) General Specifications for national roads of 2004, (Table 1) in reference to Maximum Dry Density (MDD), Optimum Moisture Content (OMC) and 4-days soaked California Bearing Ratio (CBR) hence the need to be stabilized

## 2.2 Crushed Concrete Aggregate Wastes

Concrete wastes were obtained from Kawempe, Kirinya and Namanve dumping centres owned by ROKO Construction Company. It involved crushing, pre sizing, sorting, screening and removal of contaminations. The obtained material was hand crushed using hammers to obtain a representative sample containing aggregate wastes of size ranging from 0.075mm - 0.375mm. The aggregates had an Aggregate Crushing Value (ACV) of 19.3%, Aggregate Impact Value (AIV) of 18.3%, Ten Percent Fine Value (TFV) of 11.2%, Flakiness Index (FI) of 16.3% and Los Angeles Abrasion value (LAAV) of 26.0%, hence highly resistant to crushing under applied loads.

## 2.3 Sample Preparation

The samples for the tests were prepared in accordance with BS 1377 Part1:1990. On account of the fact that some tropical soils are sensitive to pre-test drying methods, air-drying was undertaken. Other pre-test sample preparation methods included pulverization, sieving and subsampling (coning, quartering and riffling). After air-drying all the three bulk soil samples, index properties tests were carried out for classification.

In order to investigate the effect of crushed concrete waste aggregates on the properties of lateritic soils, specimens with specified amounts of crushed concrete waste aggregates added to the soil samples were prepared by mixing in quantities of 0%, 30%, 40% and 50% of weight. The mixing was done mechanically on a metal tray. For consistency, soil was mechanically blended before mixing with the waste aggregates. Tests of physical properties of the different soil/aggregate blends were conducted.

### 2.4 Tests

Sieve Analysis Test: The test was carried out in accordance with BS 1377:Part2:1990. In this test, representative samples of approximately 3 kg were used for the test. The sample was washed and oven dried before sieving. The sieving was carried out using an automatic shaker with a set of sieves stacked in order of decreasing sieve sizes. From the weights retained on each of the sieves, the percentage passing was obtained which was then plotted on semi log graph to give the particle distribution curve.

Atterberg Limits Test: The cone penetrometer method was used to determine the liquid limit of the gravel/aggregate mixtures. As the moisture content of the soil sample was increased by small amounts, the penetration of the cone was noted and plotted against the respective moisture content. From the same soil sample, a specimen was dried to near its plastic limit by air drying. It was then molded into a ball and rolled between the palms of the hand and glass plate to threads of nearly 3 mm in diameter. The soil was then considered to be at the plastic limit and its moisture content was determined.

California Bearing Ratio Test: The test was carried out for all design mixes in accordance with BS 1377:Part4:1990 for natural gravel and BS 1924:Part2:1990 for stabilized lateritic gravel. Fresh sets of 7000g air-dried soil are mixed with suitable amount of water to their OMC. Each layer was compacted with 65 blows using a 4.5kg hammer at a drop of 450mm. The compacted soil and mould were weighed and then soaked in water for four days. After the four days, of soaking, the samples were placed under the CBR machine following standard procedures. Load applied was recorded at varying penetrations to give a stress-strain curve from which the CBR was computed.

Proctor Compaction Test: The compaction tests were performed in accordance with BS 1377:Part4:1990 for natural gravel and BS 1924:Part2:1990 for stabilized lateritic gravel. Samples were crushed to pass through 20 mm British Standard sieve size and about 6 kg of material was used. The sample was mixed with suitable amount of water and compacted in five layers. Each layer was compacted with 4.5 kg rammer from a dropping height of 450 mm. Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) was determined from the graph of dry density against moisture content.

## 3 Results and Discussion

### 3.1 Introduction

The effect of different percentages of concrete aggregate wastes on the engineering properties of the soil are shown in Table 1. The results are further discussed graphically.

Table 1. Engineering properties of different soil blends

| **Percentages of waste aggregates** | **GM** | **LL** | **PL** | **PI** | **LS** | **MDD** | **OMC** | **CBR** | | | **CBR SWELL** |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | **93%** | **95%** | **98%** | |
| 0 % | 1.70 | 49.70 | 27.60 | 22.10 | 12.9 | 2.050 | 13.2 | 16 | 20 | 30 | 0.66 |
| 30% | 2.13 | 37.10 | 21.50 | 15.60 | 7.5 | 2.076 | 10.7 | 19 | 34 | 56 | 0.50 |
| 40% | 2.31 | 33.10 | 19.46 | 13.64 | 6.4 | 2.090 | 9.9 | 28 | 46 | 66 | 0.39 |
| 50% | 2.40 | 29.70 | 18.30 | 11.40 | 5.4 | 2.132 | 7.9 | 36 | 59 | 76 | 0.23 |

## 3.2 Particle Size Distribution

The material was not suitable for sub-base since it did not meet the gradation requirements. Particle sizes of 2.00, 0.425 and 0.075 mm were out of the grading percentage limits. However, blending it with 30% waste aggregates improved the particle size distribution within the percentage limits.

The grading modulus improved gradually as the percentages of the waste aggregates increased. Blends of 40% and 50% waste aggregates met the grading requirements for a sub base specified by the grading envelope as per the MWH& C general specification for roads and bridges (2004), Table 2.

## 3.3 Atterberg Limits

The liquid limit of the resulting blends all decreased with addition of aggregates. That is from 49.70% for neat to 37.10%, 33.10% and 29.70% for 30%, 40% and 50% waste aggregate addition respectively. The addition of aggregates which are non-cohesive reduced the binding ability of the mixture and its capacity to retain moisture. In addition, increase in aggregates decreased the samples shrinkability. All the blends had a liquid limit less than 40%, evidence of lower plasticity, within limits of the MWH& C general specification for roads and bridges (2004) in Uganda.

Table 2. Requirements for Sub-base layers of G45 materials

| Material properties | Material class G45 | |
|---|---|---|
| | **General requirements** | **Calcrete or other pedogenic materials** |
| **CBR: BS 1377: Part 4** | | |
| CBR (%) | Minimum 45 after 4 days soaking 1) | |
| CBR-swell (%) | Maximum 0.5 measured at BS-Heavy compaction | |
| **Atterberg limits: 2)** | | |
| Max Liquid limit BS 1377: Part 2 | 40 | 45 |
| Max Plasticity Index BS 1377: Part 2 | 14 | 16 |
| Max Linear Shrinkage BS 1377: Part 2 | 7 | 8 |
| **Grading: BS 1377: Part 2** | | |
| Requirements: | Grading modulus, GM shall be minimum 1.5 | |

1) CBR values shall be measured at the specified field density for the layer.
2) It is emphasized that the Atterberg limits shall be measured according to BS 1377: Part 2. Other laboratory test procedures and equipment may not give comparable results and shall not be used unless proper correlation to BS has been carried out to the satisfaction of the Engineer.

## 3.4 Compaction Characteristics

MDD of the material increased with increase in the percentages of waste aggregates, Figure 1. As seen before increment in the percentages of waste aggregates led to an improvement in the grading of the material. Grading is directly proportional to MDD of the material. The MDD was in the range of 2050-2132kg/m$^3$ of the stabilizer contents of 30% to 50%. The coarse fraction of the mixture increases, natural micro clusters breakup, grains come close together with voids filled by the fines and on compaction the particles interlock each other thereby increasing dry density.

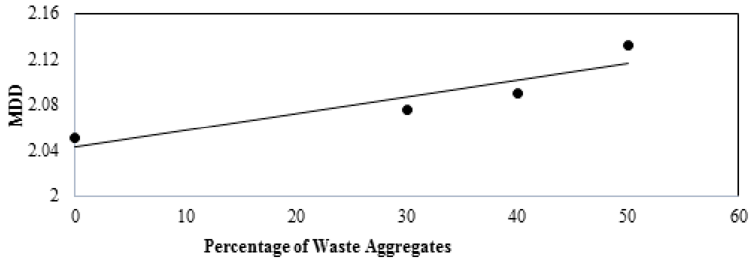


Figure 1. Relationship between MDD and % of waste aggregates

OMC is the maximum water content required to achieve maximum compaction of the material. According to the results, Figure 2, OMC reduced with increase in the waste aggregates used i.e. 13.2 for neat, 10.7, 9.9 and 7.9 for 30%, 40% and 50% of waste aggregates respectively. Addition of aggregates increases coarse fraction and reduces the proportion of fines in the mixture. As a result, less water is required to lubricate the fines thereby reducing the OMC.

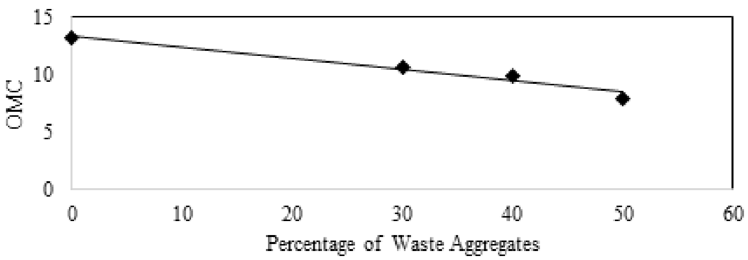


Figure 2. Relationship between OMC and % of waste aggregates

### 3.5 California Bearing Ratio

The CBR values increased from 20 for neat to 34, 46 and 59 for 30%, 40% and 50% of waste aggregates respectively considering CBR at 95% MDD. The increase in CBR was due to the increase in compaction (MDD) of the material. High compaction of the material renders it impermeable to water, expulsion of air voids and high densities which result to high bearing capacities. Only the CBR for 40% and 50% of waste aggregates at 95% MDD achieved the requirements for sub base material as per the MWH& C general specification for roads and bridges (2004) in Uganda.

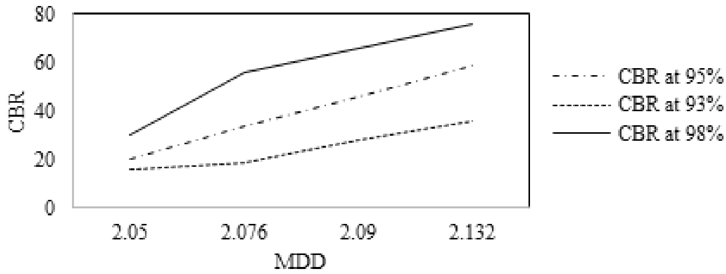


Figure 3. Relationship between CBR and MDD

## 4 Conclusion

Basing on the analysis of the results, with reference to the general specification for sub base material by Ministry of Works, Communication and Transport Uganda, mix designs of 40 % and 50 % of the waste aggregates and 60% and 50% of laterite soils respectively were considered suitable for use as sub base material as they showed CBR values of 46 and 59, respectively, at 95 % relative compaction and PI values of 13.64 and 11.40. These met the specified standards of a CBR equal or greater than 45 and PI equal or less than 14 according to the general specifications of Ministry of Works, Housing and Communications (2004).

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# Geotechnical Design Drivers for Super Tall Buildings in the Middle East

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**Abstract**

Moving into a different design realm, the design of "super-tall" buildings present new challenges for structural and geotechnical engineers. Super tall buildings present design considerations of a new magnitude compared with other buildings with issues such as large building loads, wind load effects and dynamic and cyclic response driving design. Geotechnical factors which drive the foundation design for super tall buildings in the Middle East will be explored considering geological features, modulus, bond yield strength, shaft resistance, settlement and tilt, creep, liquefaction and cyclic degradation considerations. The influence of these factors for the foundation design of the Emirates Twin Towers, Burj Khalifa and Nakheel Tall Tower will be reviewed with the aim of defining design drivers that could be considered when designing the world's next tallest building.

***Keywords:*** *Dubai, Tall Buildings, Foundation Design, Cyclic Degradation, Bond Yield Strength*

## 1 Introduction

With mankind's desire to explore new horizons, tall buildings are being built to new heights providing interestingly iconic sky lines for cities. The design of super tall buildings relates new challenges to geotechnical and structural engineers as one moves into a different design realm where traditional design procedures cannot be solely used. Super tall buildings present design considerations of a new magnitude compared with other buildings, such as large vertical loads, moments and horizontal loads along with more complex load sharing within the foundation system, earthquake effects, settlements, structural capacity of components, dynamic response of the structure to wind loads and the influence of the cyclic nature of wind and earthquakes on foundation capacity, and soil-structure interaction for all loading scenarios. A significant number of high-rise building projects are located in the Middle East.

The Burj Khalifa at a height of 828m has been the World's tallest building since 2008. The United Arab Emirates currently has approximately 26 No. super tall buildings (19% of the total number in the world) second only to China with 64 No. super tall buildings (46% of the total number in the world). Currently under construction, the Jeddah Tower in Jeddah, Saudi Arabia at an estimated completed height of 1,008m will become the World's tallest building upon completion (Arab News).

Three super tall buildings in Dubai in excess of 300m in height will be considered as part of this paper: Emirates Twin Towers, Burj Khalifa and Nakheel Tall Tower. The Emirates Twin Towers are 305m and 355m in height with the Nakheel Tall Tower planned to be in excess of 1000m in height. This paper will review some of the design challenges associated with super tall buildings in the Middle East from a geotechnical perspective. Geotechnical factors which drive design will be explored based on the influence of these factors in relation to the design of the three latter buildings. The objective thus is to define design drivers that could be considered when designing the world's next tallest building in Dubai. The latter three buildings are founded on piled rafts.

## 2 Characteristics for super tall buildings

For the foundation design of super tall buildings, Haberfield et al. (2008) define the key structural issues as substantial building weight and wind load (where designers are working in GN and MN orders of magnitude). Inclusive of the latter two issues, the key characteristics of super tall buildings that can have a pertinent influence on the design of the foundations are as follows (after Poulos, 2009, Davids et al., 2008):

1. The substantial building weight and hence the resultant vertical load imposed on the foundation will influence the settlement of the structure as well as the structural capacity of each foundation element. Moon (2008) has shown that the building weight increases non-linearly with height.
2. Wind loading imposes lateral forces on the structure and consequently high moments on the foundation, resulting in increased vertical loads on the foundation particularly on the edge piles. These increased axial loads in conjunction with the moments and lateral forces should be considered for the structural design of the piles.
3. The influence of the cyclic nature of the wind-induced loads on the foundation system should be considered in relation to cyclic loading having the potential to degrade shaft resistance of the piles resulting in increased settlements.
4. Seismic action will induce lateral motions in the ground in conjunction with additional lateral forces in the structure. The foundation system will thus be subject to additional lateral forces and moments due to two mechanisms:
   - Inertial forces and moments as a result of lateral excitation of the structure; and
   - Kinematic forces and moments in the piles due to the action of ground movements acting against the piles and the variation of these movements associated with difference in stiffness of ground strata (Nikolaou et al., 2001).
5. The potential of dynamic loading (wind and seismic) to result in resonance within the structure. The risk of dynamic resonance is dependent on factors such as natural period of the structure, the predominant period of the dynamic loading and the stiffness and damping of the foundation system.
6. Differential settlements between high and low-rise portions (typically podium structures) of the building should be controlled. In addition, the allowable angular distortion and overall allowable building tilt should be assessed from a functional and occupant experience viewpoint. Tilt limits will also be driven primarily by lift (elevator) operations.
7. The volume of ground that is potentially mobilized and the mechanism that is created in relation to local and global stability of the foundation system. Punching of the full foundation system will effect sizing of the foundation and the depth required for ground investigation data.

Tall buildings, due to their status and investment also are designed for significant design life, which challenge the way one considers ground parameters. The foundation system for super tall buildings will carry load in various ways dependent on the type of system adopted. In the past in Dubai, piled raft systems have been used whereby load is carried by the piles in shaft friction in conjunction with the raft bearing on the ground. A fairly recent change in legislation now requires pile foundation systems to be implemented whereby the load is carried solely by the piles. Thus designers require more from the ground and such designs require better ground investigation data. The depth of investigation required to supplement these designs pushes the boundaries of present ground investigation experience. In addition, the required lengths of piles approach the limits of constructed depths to date whereby issues such as verticality and machine limits drive pile length.

For super tall buildings, a significant volume of material is needed to be engaged to reach geotechnical capacity and the system actually reaches the boundaries of the structural capacity of the components before geotechnical capacity can be mobilised. Structural capacity of the foundation components becomes the design driver which is different to normal buildings. High strength concrete in the order of 80MPa is used for super tall buildings. Thus for super tall buildings, the key issues to be considered for the foundation design include: Ultimate capacity of the foundation system under all loading combinations (in particular structural capacity of the foundation components); local and global stability of the foundation system; overall and differential settlements in the short and long-term; cyclic degradation of shaft resistance and ground modulus; seismic effects considering the potential of liquefaction on the ground surrounding or supporting the foundations and the response of the structure-foundation system to seismic excitation; dynamic response of the structure-foundation system to wind loads; potential influence of externally-imposed ground movements on the foundation system arising from settlement of the ground due to fill or dewatering, heave of the ground due to basement excavation, movements arising from the installation of piles near installed piles and dynamic ground movements from seismic activity (Poulos, 2008); load-sharing between the components of the foundation system; and selection of ground parameters considering design life, cyclic degradation and induced strains for different load cases.

## 3 Super tall buildings: case studies

The Emirates Twin Towers (ETT), Burj Khalifa and Nakheel Tall Tower (NTT) are considered. The Emirates Twin Towers are located approximately 2.7km from the Burj Khalifa site and the Nakheel Tall Tower will be located about 21km from the Burj Khalifa site. All three Towers are founded on a piled raft foundation system with the Burj Khalifa and Emirates Twin Towers having circular shaped piles and the Nakheel Tall Tower having rectangular shaped piles termed barrettes. Construction of the Nakheel Tall Tower has not been completed to date. A summary of the characteristics of each Tower is included in Table 1. Note the significant pile lengths used for each Tower which range from 40 to 59m.

Table 1. Details of each case study Tower.

| | **Emirates Twin Towers (Poulos, 2009)** | **Burj Khalifa (Poulos, 2009)** | **Nakheel Tall Tower (Haberfield et al., 2008)** |
|---|---|---|---|
| Building height (m) | 305 (Hotel)<br>355 (Office) | 828 | Estimated 1200 |
| Foundation system | Piled raft | Piled raft | Piled raft |
| **Foundation elements** | | | |
| Raft thickness (m) | 1.5 | 3.7 | 4 to 8 |
| Pile Shape | Circular | Circular | Rectangular (barrettes) |
| Pile diameter (m) | 1.2 | 1.5 | 1.2 x 2.8<br>1.5 x 2.8 |
| No. of piles | 92 (Hotel)<br>102 (Office) | 1196 | 408 |
| Pile lengths (m) | 40, 45 | 50 | 42, 59 |

## 4 Geology of Dubai

The geology of the eastern Arabian Peninsula was influenced primarily by the deposition of marine sediments associated with a number of sea level changes in recent geological time (Poulos, 2009). The deposition of strata occurred in a variety of depositional environments, principally shallow marine, intertidal zones, sabkha and fluvial environments. The rocks of the Eastern Arabian Peninsula comprise a substantial thickness of carbonate, clastic and evaporate sedimentary rocks. Kent (1978) and Evans (1978) provide an overview of the geology of the Middle East.

Dubai is located on the northern edge of the Arabian Plate which is slowly colliding with the Asian Plate. This tectonic movement creates a region to the north which has significant seismic activity. Dubai is considered to be within a seismically active area and design spectra are typically related to proximity to the plate boundary. Approval authorities generally require Zone 2A to be applied for design for major building projects (Davids et al., 2008)

### 4.1 Generalised subsurface geological profile

The generalised subsurface geological profiles for the considered Towers are summarized in Table 2. Site levels are related to Dubai Municipality Datum (DMD). Typically, the ground conditions comprise a subsurface profile which is complex and highly variable in regard to strata thickness, cementation and occurrence of gypsum layers. This is due to the nature of deposition, seismic activity, river channels and the hot arid climatic conditions (Russo et al., 2013). The profile generally comprises of loose to medium dense calcareous silty sand with cemented horizons present, underlain by variably cemented materials and very weak to weak rock (sandstone, conglomerate, calcisiltite, siltstone, calcirudite and claystone and mudstone at depth). "First rock" depth ranges from -8.1m DMD to -17.3m DmD (relating to depths below ground level of 11 to 20m). Ground water is shallow typically near to surface to approximately 3m below ground level.

Table 2. Generalised subsurface geological profile.

| Subsurface strata descriptions | Average bottom of unit (m DMD) | | |
|---|---|---|---|
| | **Emirates Twin Towers (Poulos, 2009)** | **Burj Khalifa (Poulos, 2009)** | **Nakheel Tall Tower (Haberfield et al., 2008)** |
| **Level at top of strata (mDMD)** | +1 to +3 | +2.5 | +2.5 |
| Loose to medium dense, slightly gravelly to gravelly, silty sand. Cemented soil horizons | -8.1 | -13.5 | -17.3 |
| Very weak, fine to medium grained, calcareous sandstone | -26.8 | -28.5 | - |
| Calcareous silty sand, variably cemented with localized well-cemented bands | -33.1 | N/A | - |
| Very weak to weak, thickly to very thickly bedded conglomeratic calcisiltite or calcisiltite interbedded with weak, thickly to very thickly bedded conglomerate | -79* | -91 | -74 |
| Weak, thickly to very thickly bedded siltstone interbedded with very weak, conglomerate and weak, thickly to very thickly bedded calcisiltite | - | >137.79* | >217.5* |

*Note: Borehole data ends at average depth

## 5 Geotechnical Design Drivers

First and foremost, the geology, ground conditions and the variability thereof across a site is of primary interest in respect to foundation design. The geotechnical factors that drive design thereafter generally stem from the geological conditions on site and the ground's response to imposed loading conditions. The geotechnical factors that drove the foundation design of the three case study towers considered for this paper are discussed below.

### 5.1 Geological features

The following geological features are typical to ground conditions in Dubai and are significant in terms of foundation design:

- **Calcareous silty sand deposits and reclaimed land**
  Reclaimed ground is prevalent along coastal areas in the Middle East. Liquefaction of these unconsolidated reclaimed materials as well as the overlying loose to medium dense calcareous silty sand deposits is a fundamental consideration. Poulos and Bunce (2008) used the Japanese Road Association Method and the method proposed by Seed et al. (1984) to estimate the potential for liquefaction for the Burj Khalifa. Both methods indicated that the Marine Deposits and calcareous silty sand had the potential to liquefy within the top few metres below ground level. In relation to design, consideration for liquefaction effects

need to be given to buried services and shallow foundations in the upper section of the ground profile. In addition, potential downdrag forces on pile foundations constructed through liquefiable strata need to be considered.

- **Calcisiltite**
  Underlying the silty sand layer, the geological profile is typical of very weak rock with variable cementation. This is typical of the Calcisiltite unit which forms a large portion of the profile in which the piles are installed. With a relatively high carbonate content and high void ratio, the Calcisiltite unit displays a structure consistent with a formation process whereby soil particles are cemented with carbonate. This particle bonding acts to preserve the structure of the material as insitu stresses increased during deposition and burial (Haberfield et al., 2008; Poulos, 2009). If subjected to high loading conditions, the cementation bonds could be broken which would lead to densification and consolidation of the material and result in long-term settlements. Thus a fundamental design consideration is the strength of these bonds termed hereinafter the bond yield strength. This design driver is discussed in detail in Section 4.2.
- **Gypsum**
  The geological profile is also characterized by interbedded layers with variable properties (that is highly heterogeneous), typically deposits containing gypsum. Massive gypsum layers up to 3.5m in thickness have been observed (Haberfield et al, 2008). The gypsum layers are stiffer than the matrix material and highly variable. Thus for relatively small variations in pile toe level, this could result in significantly different pile performance characteristics dependent on the material at founding level (Poulos, 2009). In addition, solution of the gypsum was believed to be a risk for future degradation of ground properties (Poulos and Davids, 2008). As such for the both the Burj Khalifa and NTT, gypsum levels were considered when defining pile toe depth with pile toes generally being extended beyond gypsum levels (Poulos, 2009; Haberfield et al., 2008).
- **Chemically aggressive ground conditions**
  Due to high salt content of the insitu material and groundwater, the ground conditions of Dubai are characterized as highly corrosive. This should be considered for foundation design where such conditions could cause accelerated deterioration of foundation materials (steel and concrete) (Poulos, 2009; Haberfield et al, 2008; Davids et al, 2008).
- **Rock surface variability and conditions with depth**
  A phenomenon that appears to be prevalent within the Middle East is that the ground conditions may not necessarily improve with depth particularly within pile foundation depths. Thus it may not always be feasible to increase pile lengths to achieve design criteria (Poulos, 2009). Variation in rock surface level occurs as a result of river channels. However for tall buildings this does not have a significant effect on foundation design due to the large volume of material that is mobilized and the depth to which the mobilized material extends.

## 5.2 Bond yield strength

For all three considered Towers, the piles were socketed into weak rock strata. Due to the nature of the insitu rock (as discussed above), bond yield strength is a fundamental design limit. Bond yield strength represents the point at which bonds between particles are broken and the rock changes its compressibility and shear strength properties (essentially behaving as a different material). Currently designers are not able to predict this change in behaviour and the bond strength limit aims to keep the design within known material behaviour limits. Haberfield et al. (2009) found that there was a significant increase in creep rate when bond yield strength was exceeded.

To account for this in design of pile foundations for super tall buildings, induced ground shear stress at the pile toe as well as along the length of the pile should be checked such that the bond yield strength of the founding rock is not exceeded. The assumption is that rock withstands load through bond strength (represented either by shear strength or UCS. For the NTT, the bond strength limit was pegged at UCS/2 as shown in Equation 1. The UCS design values used for the Burj Khalifa and NTT are summarized in Table 4 and are generally low ranging from 1 to 2.6MPa on average.

$$\tau_f < UCS/2 \tag{1}$$

Where $\tau_f$ is induced ground stress and *UCS* is Unconfined Compressive Strength.

Table 3. Unconfined Compressive Strength Values.

| | **Burj Khalifa (Poulos, 2009)** | **Nakheel Tall Tower (Haberfield et al., 2008)** |
|---|---|---|
| **Layer** | **Unconfined Compressive Strength (UCS) Design Values (MPa)** | |
| Calcareous Sandstone | 1.0 | |
| Gypsiferous Sandstone | 2.0 | |
| Calcisiltite | 1.3 – 1.7 | 0.5 – 14<br>Average: 2.6 |
| Conglomerate | 2.5 | |
| Siltstone | 1.7 | 0.1 – 11<br>Average: 2.5 |

## 5.3 Young's Modulus

For the three considered Towers, Young's modulus values were derived based on a combination of data from a variety of insitu and laboratory tests including pressuremeter testing, resonant column tests, unconfined compressive strength tests, laboratory stress path tests, standard penetration tests (SPT) and p-s suspension testing. The modulus design values applied for the foundation design for the Burj Khalifa, NTT and ETT are summarized in Figure 1. For the ETT and Burj Khalifa failure of the test barrettes could not be achieved and the designers chose to cap assumptions within the known limits from the test pile data. Thus the parameters adopted are more conservative than those applied for the NTT. Poulos and Bunce (2008) reported that the movement estimates and monitored operational data for the Burj Khalifa were significantly lower than ground movements predicted during the design and were therefore quite conservative. For NTT, the designers took a different approach to parameter selection and were able to mobilise more shaft resistance in the test piles influencing their more favourable selection of design parameters. The approach adopted for the NTT design shifted the current view and pushed the boundary on the way parameters are selected.

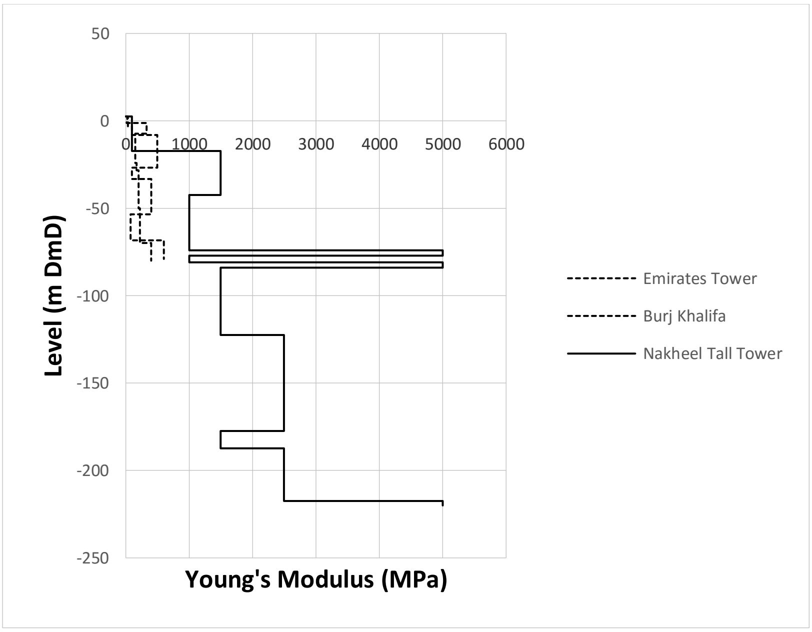


Figure 1. Design modulus with depth.

This comparison of design values shows that an approach considering design values for the strain levels expected yields more realistic design values. The applied design values should be selected in relation to small strain modulus. A reduction in small strain modulus is then applied depending on the strain levels one expects for each design case. For instance for the assessment of dynamic loading conditions such as wind and seismic loading, modulus values for smaller strain levels more representative of a dynamic response should be applied for design. Thus a number of design scenarios should be defined whereby design values are defined for each scenario. The general approach applied for the case studies was to define a characteristic best estimate for static and dynamic loading conditions and an upper and lower bound on the design value applied as additional design cases to cover variation in the parameters and ground response considered.

## 5.4 Skin Friction

The skin friction values applied for the pile foundation design for ETT, Burj Khalifa and NTT are summarized in Figure 2 and based on full-scale testing results. The skin friction design values assumed for the Burj Khalifa and ETT vary quite significantly from those applied for the NTT. For both the ETT and Burj Khalifa projects, no conclusions could be reached on end-bearing as none of the test piles appeared to have overcome frictional capacities of the piles and failure of the test piles could not be achieved. The designers chose to cap design assumptions within the known limits based on full scale testing results. The design values applied for the NTT project may be more realistic values with approximately 550kPa to 600kPa in the calcisiltite unit with increased values of 1250kPa below -80m DMD. For tension shaft capacity, the Burj Khalifa design utilised a tension shaft capacity of 50% of compression shaft capacity. This correlates well with a theoretical relationship between tensile and compressive skin friction developed by De Nicola and Randolph (1993) which indicates a factor of 0.6.

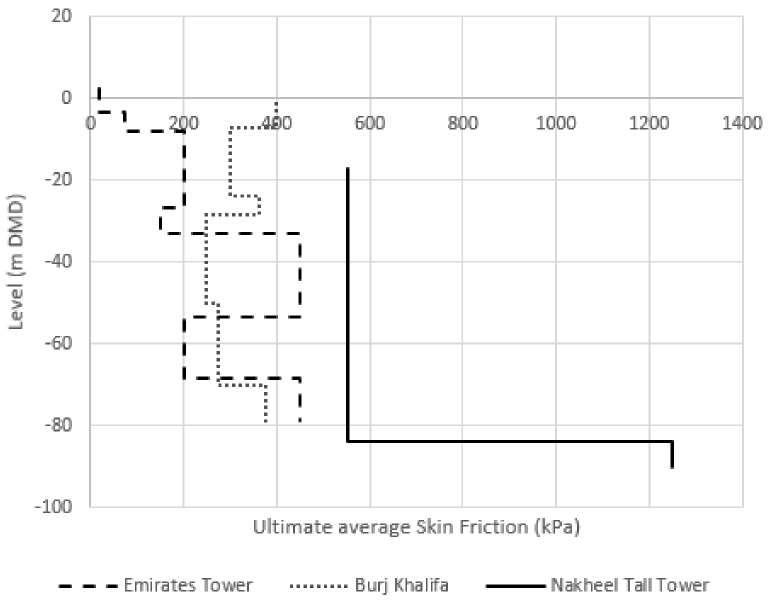


Figure 2. Skin Friction with depth.

## 5.5 Cyclic degradation

Cyclic degradation of modulus and shaft resistance due to wind and/or seismic loading is a fundamental design consideration for super tall buildings. The three case studies investigated the effects of cyclic loading on ground modulus via cyclic triaxial specialist laboratory tests and on shaft resistance via cyclic Constant Normal Stiffness (CNS) tests. The test results indicated that there was a potential for degradation of ground modulus. For design, long term behaviour and cyclic degradation shall be accounted for by means of the choice of an appropriate ground modulus. The ground modulus applied for design should be selected as a value representative of time-related cyclic degradation and based on results from cyclic full scale and/or laboratory load tests.

CNS tests were undertaken to estimate the likely shaft resistance of piles by shearing concrete-rock interfaces which represent the pile-rock interface under similar insitu conditions. The results indicated that there was limited potential for degradation of the pile-rock interface under one-way cyclic loading (Haberfield et al, 2008; Poulos and Bunce, 2009). However under two-way cyclic loading (tension loading whereby complete reversal of shear stress along the shaft occurs), a significant loss of capacity could be expected (Haberfield et al., 2008). For the Burj Khalifa it was shown that cyclic degradation could be contained if shear stresses are kept within a predetermined range. Larger strain ranges tended to cause degradation. For the Burj Khalifa (Poulos and Bunce, 2008), an analytical study indicated that if peak and trough loads are kept within a range of approximately 20%, then the effects of cyclic loading on axial capacity and settlement would be insignificant.

## 5.6 Other considerations

The case studies noted the effect drilling fluid type had on the load-bearing capacity of piles with polymer fluids appearing to give superior results to bentonite drilling fluid (Poulos, 2009). In addition, the load distribution across the piled raft was dependent on the design assumptions and method applied. For the Burj Khalifa, the pile load distribution varied depending on the type of analysis applied. This variation was dependent on raft stiffness (rigid versus flexible), pile-soil interaction effects, non-linear versus linear response and the assumed superstructure stiffening effects on the foundation response. For the three Towers in general, higher load appeared to be attracted to the edge of the foundation. In general the design process started with an evaluation of existing data from a pool of data within Dubai to peg preliminary design parameters, this is followed by a detailed ground investigation and thereafter full-scale testing to verify design assumptions and foundation design.

## 6 Conclusions

The key geotechnical factors driving foundation design of super tall buildings in Dubai were investigated based on three case studies: Emirate Twin Towers, Burj Khalifa and Nakheel Tall Tower. The geotechnical features that govern design range from a high ground water table, ground conditions that do not essentially improve with depth, a geological profile typical of very weak rock with variable cementation and interbedded layers with variable properties (particularly where interlayered with gypsum), chemically aggressive ground conditions, and the loose overlying silty sand layer being potentially liquefiable. Bond yield strength is a fundamental consideration and ground induced stresses should be kept below bond yield strength of the rock to control long term induced settlements. The designs indicate that there is potential for cyclic degradation of modulus due to wind loading and seismic loading and design values should be determined accounting for possible degradation.

The skin friction and ground modulus design values assumed for the Burj Khalifa and ETT vary quite significantly from those applied for the NTT. For ETT and Burj Khalifa, failure of the test barrettes could not be achieved and the designers chose to cap assumptions within the known limits from the test pile data. Thus the parameters adopted are more conservative than those applied for the NTT. For NTT, the designers took a different approach to parameter selection and were able to mobilise more shaft resistance in the test piles influencing their more favourable selection of design parameters. The approach adopted for the NTT design shifted the current view and pushed the boundary on the way parameters are selected.

For super tall buildings, a significant volume of material is needed to be engaged to reach geotechnical capacity and the system actually reaches the boundaries of the structural capacity of the components before geotechnical capacity can be mobilised. Structural capacity of the foundation components becomes the design driver which is different to normal buildings. In addition due to a fairly recent change in legislation, pile foundation systems are to be implemented as opposed to piled rafts. Thus designers require more from the ground and such designs require better ground investigation data. The depth of investigation required to supplement these designs pushes the boundaries of present ground investigation experience. In addition, the required lengths of piles approach the limits of constructed depths to date whereby issues such as verticality and machine limits drive pile length.

The foundation design of super tall buildings requires the use of advanced numerical and design analyses accounting for soil-structure interaction and a consideration of range of strain levels that could occur over the long-term performance of the structure. Close interaction between the structural and geotechnical engineers is fundamental as an iterative process is required for computing structural loads and foundation response and understanding the full structure-foundation-ground interaction.

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# Effectiveness of Impulse Compaction (IC): A case study of Deepwater Container Terminal extension in Gdansk (Poland)

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## Abstract

The grand opening of the 2nd terminal (T2) in Deepwater Container Terminal (DCT) Gdansk took place on 24th October 2016. The T2 project is one of the largest investments of port sector in Central and Eastern Europe. The paper describes a case study detailing the use of Impulse Compaction (IC) technology as an effective way of compacting the sand layer in the containers stacking yards of DCT. To meet the settlement requirements the soil profiles and loading conditions were analyzed in details. In the yards the aim of the soil improvement was to compact the loose fill to uniform the settlements and offer sufficient stiffness for the pavement structure. The paper presents the results of test the field trials performed to verify the efficiency of the applied grid of points. The measured results are presented and compared with the calculated and allowable values. The authors discuss the vibrations monitoring performed on site and impact of the performed works on adjacent buildings and utilities, providing the conclusions and recommendations for the successful application of Impulse Compaction.

***Keywords:*** *soil improvement, impulse compaction, rapid impact compaction, vibrations monitoring, settlement*

## 1 Introduction

The Port of Gdansk (Poland) is situated on the southern coastline of the Baltic Sea enabling connection to the high seas through the Danish Straights. The extension of Deepwater Container Terminal (DCT) concerning brand new 2nd terminal (T2) was a major step giving DCT the position of the largest container terminal in the Baltic Sea and strengthening the role of the Port of Gdansk as a key port in this part of Europe. Now it has an opportunity to meet the growing demand for the deepwater services, by the increased capacity of the containers in the port and the ability to handle the largest Ultra Large Container Vessels (ULCVs).
The industrial region of Gdansk is known for its difficult ground and water conditions, with a significant presence of marine and alluvial deposits. The geotechnical part of the design and

build process of T2 project concerned the new quay and adjacent container stacking yards – platform area.

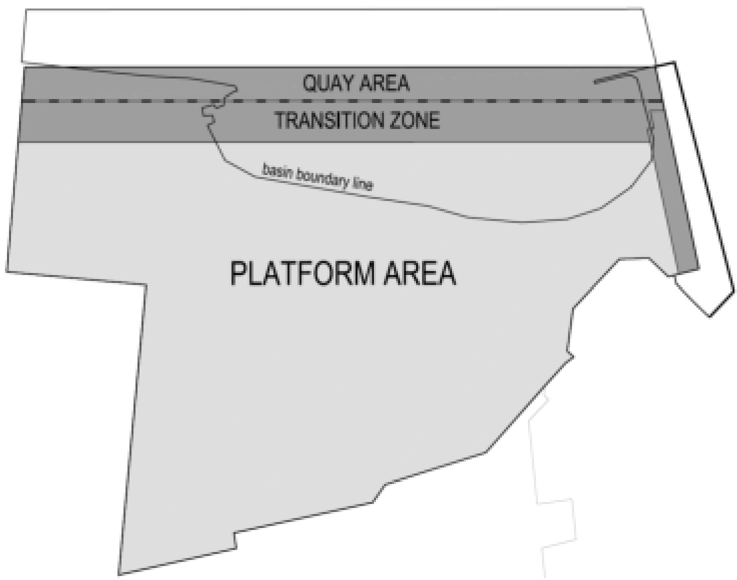


Figure 1. General layout of the 2nd terminal.

The area of 25 ha of the investment has been divided into two major parts: heavy foundation of the STS (Ship-To-Shore) gantry crane beam and deep soil improvement of the platform and quay area with connecting transition zone (Figure 1).
The soil improvement in the quay area consisted of the land part (onshore) with its origin soil layers and deep hydraulic fill of the marine part (offshore). Deep soil improvement elements, such as stone and concrete stone columns, were adopted in variable grids and lengths to ensure the allowable settlements.
In the platform area the aim of the soil improvement was to compact loose fill and upper layer of loose sands by Impulse Compaction (IC) to the reduce differential settlements and offer sufficient stiffness to the pavement structure. The improved upper layer function was to uniform the loads and distribute them to the deeper silty layer, that governed the global settlements. To meet the settlement requirements of the project the soil profiles, geological origin, loading conditions and design requirements were analyzed in details (Buca and Mitrosz, 2016).

## 2 Geological conditions

Basing on the results of the soil investigation (BAGEO soil investigation campaign, 2014), the ground-water conditions in the area were determined as varied and difficult. Soil sedimentation due to transport by the Vistula river was the main phenomenon creating the geological formations such as sand layers and soft organic silts with very low strength and deformation parameters (Table 1).

Table 1. Generalized stratigraphy, strength and deformation parameters.

| **Layer (Stratum)** | $\gamma_{sat}$ **(kN/m³)** | **ϕ (°)** | **c (kPa)** | $E_{oed}$ **(MPa)** |
|---|---|---|---|---|
| Compacted **FILL** (I) | 20 | 32,5 | - | 60 |
| Loose **SAND** with silt inclusions (II) | 19-20 | 30 | - | 42-74 |
| Medium dense to dense **SAND** (III) | 20 | 35 | - | 80-110 |
| Sandy **GRAVEL** (IV) | 21 | 41 | - | 120-190 |
| Soft **SILT** with organic content (V) | 16 | 7 | 9 | 1,4-4,0 |
| Sandy to silty **CLAY** (VI) | 20-22 | 15 | 17 | 20-29 |

The typical geological cross section (Figure 2) consists of an upper layer of fine sands (stratum II) with wide range of cone resistance, $q_c$=2÷12 MPa. The results of soil investigation showed low value of non-uniformity coefficient, what indicates the presence of single-grained soils. Samples of natural fine sand from upper layer showed $C_u$ (U) coefficient in range from 1,5 to 1,7, which unambiguously proves the mono-fraction of the tested soils. Those types of non-cohesive soils are very difficult for compaction. The layer occurs down to 3÷4 m from the existing level and is highly diversified in its parameters. Therefore, it needed some special geotechnical treatment adjusted to the designed structure.
The sensitive silty layer (stratum V) with an organic content of ca 6 m is present almost all over the site and contains inclusions of fine sand, sandy dust and peat. Identification of soils was performed on the base of PN-EN 14688-2 standard.

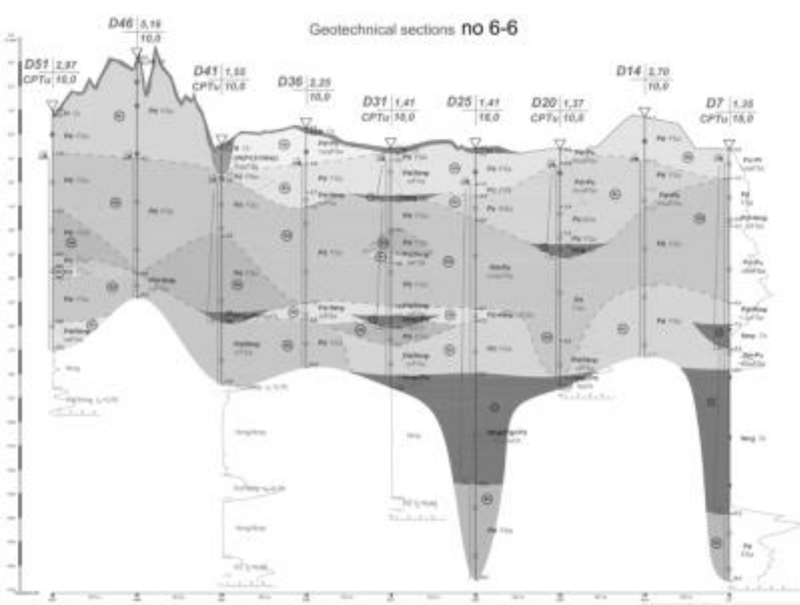


Figure 2. Typical geotechnical profile in the platform area.

## 3 Soil Improvement

In order to compact the upper loose sand layer and uniform the settlements throughout the whole platform area, IC technology was introduced.
The allowable settlements in the platform area, according to Client's requirements, were 240 mm. Due to the required settlements, the upper sand layer had to be compacted. Its purpose was to improve and uniform the parameters to avoid differential settlements, that were crucial in this part. The solution had to offer appropriate stiffness to the engineered fill and create a kind of load transfer platform to distribute the loads from the pavement down to the bottom silty layer (V). The measured cone resistance indicated that the depth of the natural ground demanding compaction is about 3 to 4 m below existing ground level. Typically the depth of treatment by Impulse Compaction is about 4 to 6 m below the working platform.

### 3.1 Implemented method

The Impulse Compaction technology is performed by means of a 9 tonne heavy hammer. The effectiveness and range of compaction is dependent on the soil type and the groundwater level relative to the working platform level. In favorable water and ground conditions the depth of compaction reaches approximately 5÷7 m. During IC a high level of soil compaction is obtained due to the reduction of porosity. In saturated soils the effectiveness of compaction is dependent on the speed of dissipation of pore water pressures, that accompanies the hammering process. The compaction works are performed with a frequency of 40÷60 blows per minute on a specially placed foot that transfers the energy deep into the ground, which results in soil improvement (Figure 3). As a result of ground compaction technological craters appear, that need to be filled with material and levelled after each pass.

Usually the most effective way of performing works can be assured by using the "Sweep & Track" method (S&T). It describes the compactor's way of changing its position by rotating around its reference axis (Figure 4). Using this method, the compactor performs a series of technological craters with a spacing specified within the testing areas. After executing the first pass, the compactor is moved to its next position and repeats the compaction cycle.

A square grid of IC points may also be used. In this method the compaction is performed in a primary grid with additional points located in between. The additional compaction is performed minimum 24 h after the primary compaction, to enable the effective dissipation of pore water pressures. This method is used mainly for heavily saturated soils with a high groundwater level. If it is necessary to obtain very high parameters of compaction or to reach higher depth it is recommended to perform a second pass, using the same grid of IC points.

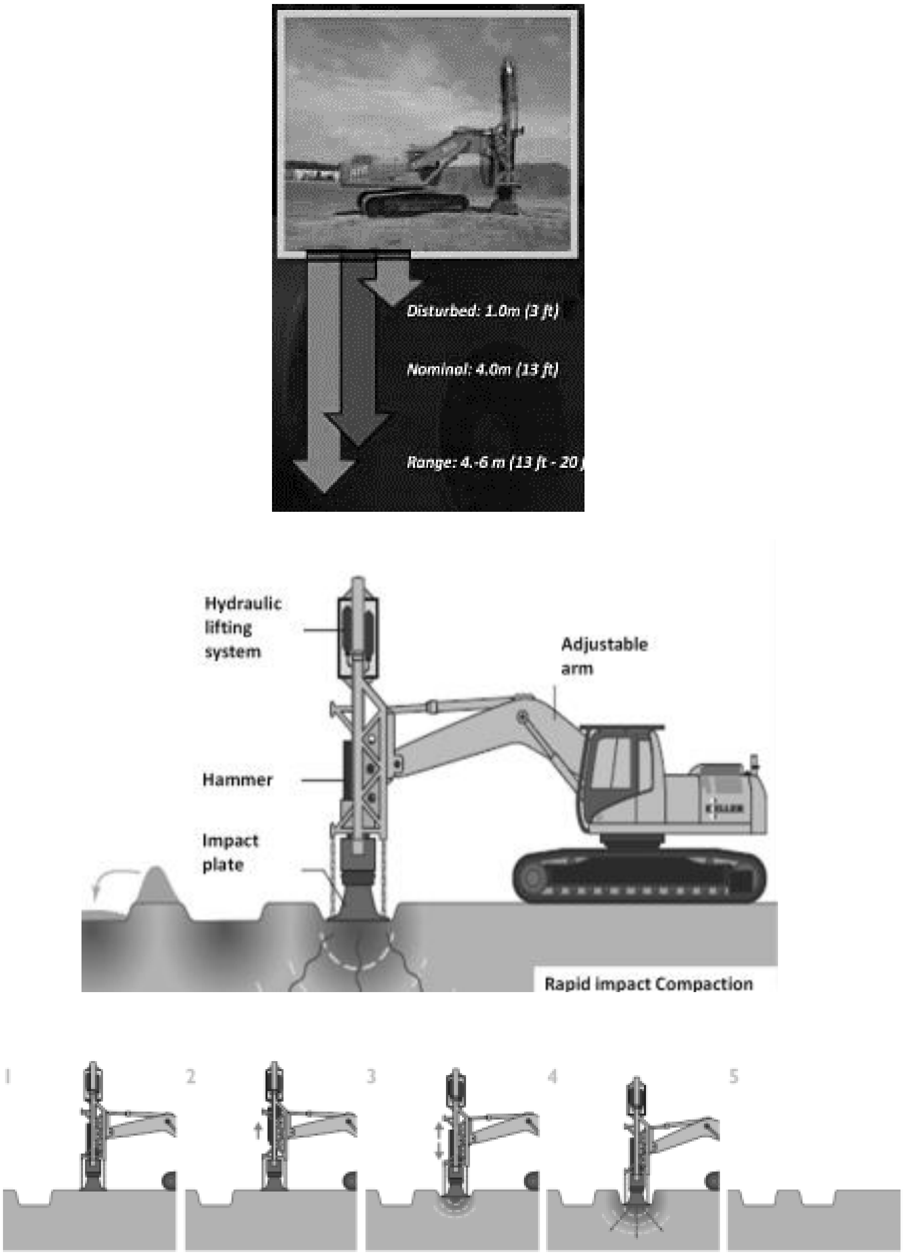


Figure 3.  Sequence of Impulse Compaction performance.

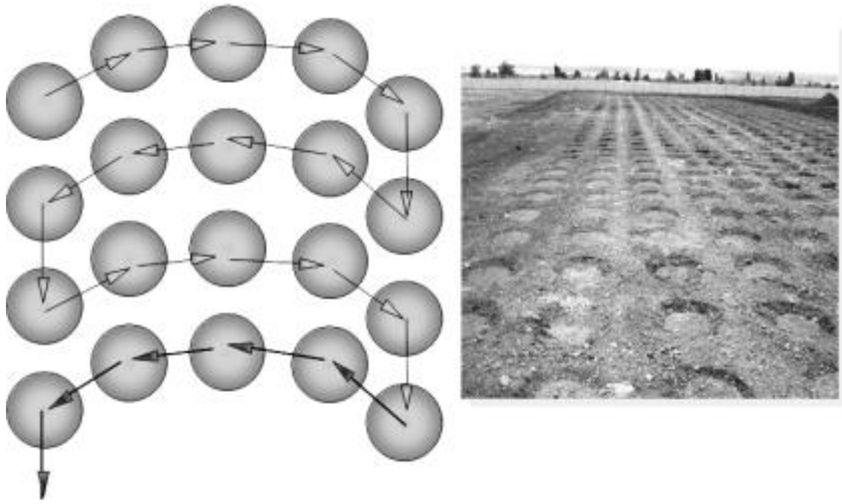

Figure 4. Sweep & Track grid diagram.

## 4 Field Trials

The Impulse Compaction trial was performed on the testing site in order to specify:

- the improvement factor after each pass of compaction (measured by CPT tests),
- the IC points spacing, grid and number of passes,
- the number of drops to obtain the total penetration of the hammer approximately 40÷50 cm.

The minimum design criteria for the ground compaction was to achieve at least 8 MPa of the cone resistance ($q_c$) over the depth of 4 m below working level (Figure 5).

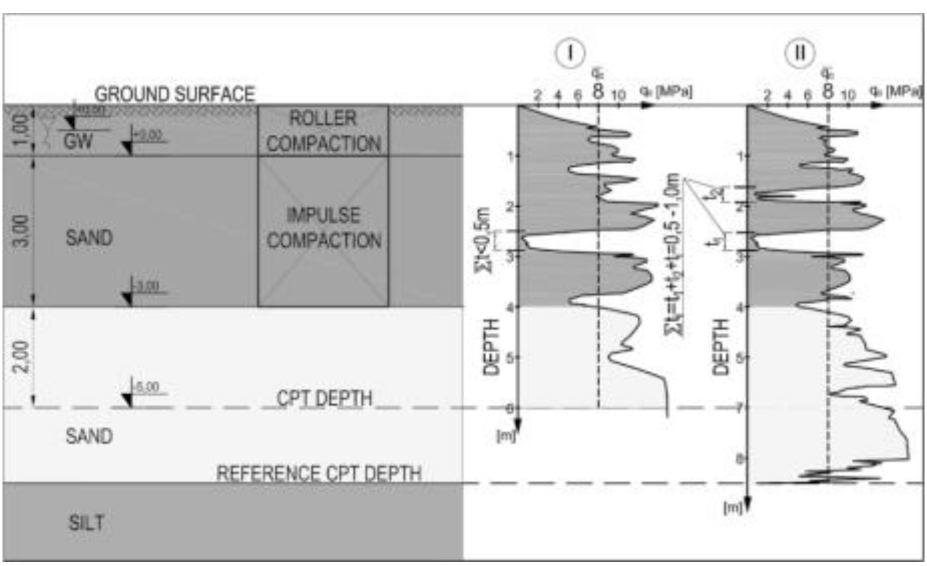


Figure 5. Impulse Compaction acceptance criteria.

During the execution of field trial, it was observed that the assumed penetration target parameters were achievable during the first compaction pass. Obtaining the high values of compaction ($q_c > 12{,}0$ MPa) to the assumed level of 4-5 m from the working level was proved by performed CPT tests (Figure 6).
In addition, all the CPT tests regardless of their position (in or between the executed IC points) showed an equal improvement factor of the treated soil, hence proving the volumetric character of the soil improvement. When comparing the performance of the various compaction methods outlined above, it has been determined that S&T provides the most effective and efficient method of compaction. Therefore it was implemented for the execution. In case high groundwater level was affecting the compaction process, square grid was used as an alternative.

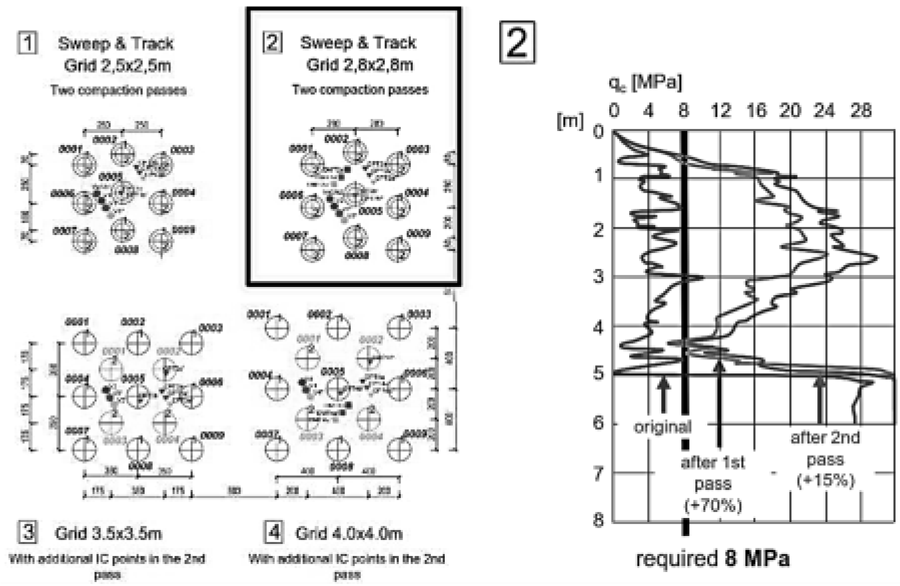


Figure 6. Results of CPT tests after S&T method.

## 5 Vibrations monitoring

A side effect of the works performed with the Impulse Compaction technology is the occurrence of inducted vibrations. Due to the fact, that the vibrations might have a negative impact on the construction of adjacent buildings, pipelines and utilities it was decided to control the impact of vibrations transferred through the ground and determine their harmfulness. The performed tests included the measurement of vertical and horizontal acceleration amplitudes of the buildings and pipelines and the determination of the frequency of inducted vibrations.
According to the Polish Standard two scales (depending on the dimensions of the building) outline five zones of the vibrations harmfulness: from I where the vibrations are imperceptible for the building to V where the vibrations cause failure of construction due to falling walls and ceilings etc.). All the vibrations of the buildings were classified to the I zone (vibrations imperceptible for the building) and II zone (vibrations perceptible for the building, but not harmful for its construction) for Impulse Compaction performed in a distance of 20÷50 m. There was no need to undertake any additional steps.
Different situation occurred, while monitoring the impact of vibrations on the pipelines. At first, vibrations were checked without any trenches limiting the range of Rayleigh waves propagation. Due to the high values of recorded accelerations and the risk of pipeline unsealing

during final works, it was decided to repeat the test after performing the 2 m wide ditch along the pipelines. Its aim was to reduce the harmful effects of vibrations on pipelines.
The effects of dynamic impact on underground installations were determined based on macro seismic scale MSK-64 (Polish Standard PN-85/B-02170). This scale distinguishes twelve vibration harmfulness zones (see Table 2).

Table 2. The relation between the level of vibration intensity and the value of accelerations in accordance to the scale MSK-64 (The European Macroseismic Scale EMS-98).

| **Intensity level** | **Description** | **Acceleration ($m/s^2$)** |
|---|---|---|
| I | Not perceptible | 0,005 – 0,012 |
| II | Hardly perceptible | 0,012 – 0,025 |
| III | Weak | 0,025 – 0,050 |
| IV | Average | 0,05 – 0,12 |
| V | Quite strong | 0,12 – 0,25 |
| VI | Strong (slight damage) | 0,25 – 0,50 |
| VII | Very strong (damage in buildings) | 0,50 – 1,00 |
| VIII | Damaging | 1,00 – 2,00 |
| IX | Destructive | 2,00 – 5,00 |
| X | Very destructive | 5,00 – 10,00 |
| XI | Catastrophic | 10,00 – 15,00 |
| XII | Exceptionally catastrophic | > 15,00 |

Due to the maximum frequency range to which the MSK-64 scale refers (0÷10 Hz) the results of vibrations at a frequency higher than 10 Hz had to be properly interpreted. Analyzing the scale of vibrations transmitted by the ground on buildings, MKS-64 scale was considered as suitable also for vibrations at a frequency higher than 10 Hz. On the base of the measured range of the vibrations frequency, the limit values of accelerations that respond to the particular zones of the harmfulness of vibrations were determined. Limit value of vibration was adopted as a = 0.5 $m/s^2$ what corresponds to zone VI in accordance to MSK-64 scale. Vibrations above this value could cause leak of the pipelines.
The results of Fast Fourier Transformation (FFT) for acceleration signals were analyzed in 1/3 octave (tierce) analysis in order to determine the extreme amplitudes of vibrations. For the particular frequency ranges (tierces) the peak values of accelerations were determined and compared with the values from MSK-64 scale. On the base of this, the harmfulness of vibrations that were transferred on the monitored pipelines during the work of the hydraulic hammer was evaluated.

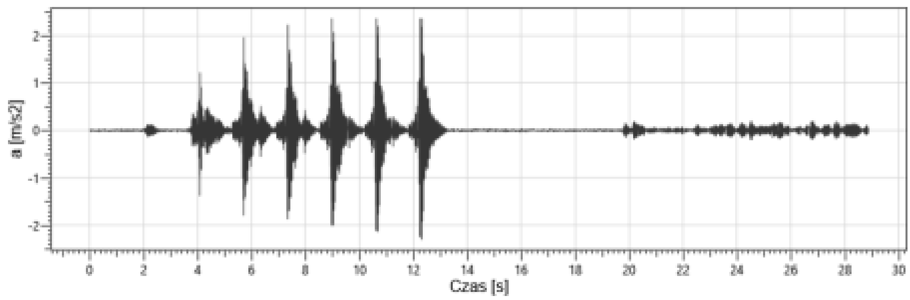


Figure 7. Example of extreme amplitudes of vibrations [$m/s^2$] before ditch performance (~15m from the pipelines).

During vibration measurements, the registered amplitudes of underground accelerations of installations (pipelines) were:

- in the VI and VII zone of intensity (water supply, sanitary and the gas pipeline in a distance from 8.5 to 28 m),
- in the V (water supply and sanitary pipeline) and in the IV zone of intensity (gas pipeline in a distance of > 60m).

Due to reduce harmful vibrations transmitted to the pipelines through the ground, the ditch between tested installations and the source of vibration was digged. Its depth was 2 m, width 2m and length 45m. After the ditch execution, vibration measurements were repeated (see Figure 8). The registered amplitudes of accelerations of underground installations were from IV to VI zone of intensity.

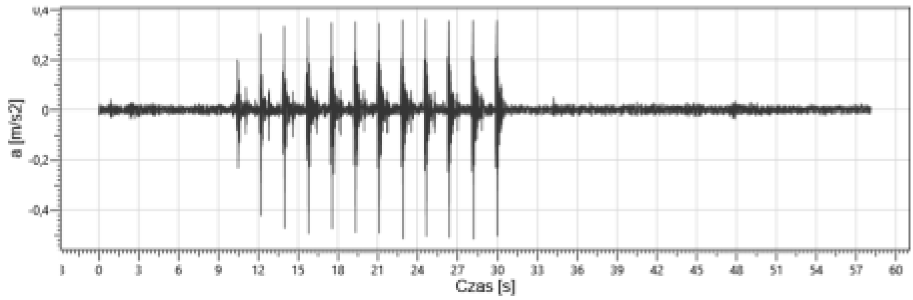


Figure 8. Example of extreme amplitudes of vibrations [m/s$^2$] after ditch execution (~15m from the pipelines).

It was decided that to perform ground compaction in IC technology safely, in a distance of more than 15m from the existing pipelines the ditch along the installations must be performed. Moreover, in order to verify the pipelines behavior during final ground compacting in IC technology, it was recommended to carry out continuous vibration monitoring of pipelines during work of hydraulic hammer, at a distance from 15 m to 50 m from the pipelines.

## 6 Embankment test

To confirm the validity of the settlement calculations, a monitored embankment test was performed. It was assumed that 3 m high overload of the embankment will be an equivalent weight that will occur during the pavement construction and as a surcharge load. The ground in the area of the embankment was previously improved with the Impulse Compaction technology.

One can observe the development of settlements on all of the settlement gauges, showing a similar type and pace of deformation. The measurements after ~1 month of monitoring showed approximately 3 cm of settlement. Above 80% of the settlement was observed in the first 2 days of the measurements. The curves showed fast stabilization of the settlements and proved that observed settlement values are in line with the calculated values (Figure 9).

| | | 1 months | 2 months | 3 months | 6 months | 12 months | 100 months |
|---|---|---|---|---|---|---|---|
| Layer | Top level (m) | Σ Δs (cm) | Σ Δs (cm) | Σ Δs (cm) | Σ Δs (cm) | Σ Δs (cm) | Σ Δs (cm) |
| SAND | 1.00 | 0.31 | 0.31 | 0.31 | 0.31 | 0.31 | 0.31 |
| SAND | -1.00 | 0.54 | 0.54 | 0.54 | 0.54 | 0.54 | 0.54 |
| SAND | -5.60 | 0.15 | 0.21 | 0.25 | 0.36 | 0.50 | 0.79 |
| SILT | -9.00 | 2.00 | 2.00 | 2.00 | 2.00 | 2.00 | 2.00 |
| SAND | -13.80 | 0.17 | 0.17 | 0.17 | 0.17 | 0.17 | 0.17 |
| | | | | | | | |
| | | | | | | | |
| Total settlement | | 3.17 | 3.23 | 3.28 | 3.38 | 3.52 | 3.81 |
| | | 83% | 85% | 86% | 89% | 92% | 100% |

3.17 cm

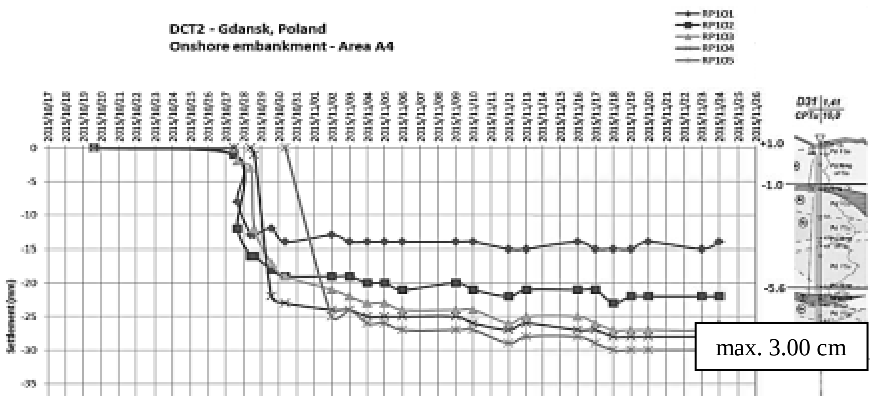


Figure 9. Correlation of calculated and measured settlements of test embankment.

## 7 Conclusions

The presented case was just a part of the complex geotechnical engineering that was implemented at the DCT site.

Before choosing any geotechnical solution, the designer has to consider a variety of components: the applicability of certain technology and its limits, type of structure, type of applied loads, structure sensitivity to settlements and type of foundation. It is also highly recommended to perform field tests prior to the commencement of works, to set appropriate QA/QC procedures and monitor the real life of the structure in order to verify the implemented solution, maintain the high quality of work and mitigate any potential risk. The applied solution also needs to fit the construction timetable and finally has to be economically viable. Thus, geotechnical engineering has to face many challenging demands.

The presented Impulse Compaction technology was very successful on DCT T2 project. All of the test results met the design assumptions. The soils were compacted quickly and economically, providing uniform parameters and required stiffness to the platform. It proved to be efficient and effective way of ground improvement, regardless difficult ground conditions with low non-uniformity coefficient values of upper sand layer and lack of compaction features. Moreover, significant improvement of parameters was observed during first compaction pass with the “Sweep & Track” method. Equal improvement factor of the treated soil was achieved, what proved the volumetric character of the soil improvement. The measured settlements were in line with the calculations as well.

However, it also has to be noted that IC technology has its limits and restrictions in terms of execution, which have to be considered. Due to significant vibrations caused by IC works, it is recommended to observe and measure the vibration accelerations of the buildings and pipelines and consider its limitation by construction of ditch near the exposed objects. On DCT site it was decided that to perform ground compaction in IC technology safely, in a distance of more

than 15 m from the existing pipelines the ditch along the installation with continuous vibration monitoring must be performed.
However, the solution should always be specified on the base of the existing ground conditions and like the other parameters, adjusted to the characteristics, requirements and constrains of a particular project.

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# Ground Stabilization Techniques in South Africa: Research and Project Study

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## Abstract

In South Africa, soft clays are most common in the coastal areas and with most localized deposits in Cape Town and around Durban, Richards bay, KwaZulu-Natal North and South Coasts. They have been characterized with low shear strength, high compressibility and severe time related settlement problems. The benefit of the inclusion of geosynthetics (geotextiles and geogrids) in a granular base, underlying a subgrade having a CBR less than 2%, showed that there was a significant improvement in bearing capacity and reduction in settlement accruing from geosynthetic inclusion as shown by the bearing capacity ratio (BCR) of 1.21, 1.29, and 1.63 for geogrid, geotextile, and geogrid-geotextile combinations, respectively. To complement the research, several case studies will be presented to highlight the broader scope of ground stabilization in pavements.

***Keywords:*** *geosynthetics, ground stabilizations, roads, cyclic loading, static loading*

## 1 Introduction

Ground stabilization is achieved by the use of geosynthetics materials to restrain the movement of soil particles through confinement, producing a soil-geosynthetic composite. The composite is able to sustain higher loads (bearing capacity), is susceptible to lower deformation in both static and dynamic case (lateral restrain). Overall, it produces a soil layer with a lower thickness or enables the use of lower quality materials incorporated with geosynthetics. This benefit is particularly useful in roads, both unpaved and paved, as well as in working platforms or storage areas. In the application of roads, the design is based on the degradation of the soil layers as a function of the traffic; the more traffic the higher will be the deformation until a limit is reached for which the layer is considered not suitable for the design, thus reaching a serviceability failure. In comparison, for working platforms or storage areas, the critical design is a very high static loading developed by low speed of the cranes, forklifts or stacked containers. The load is applied for a certain duration of time and bearing capacity takes preferences over deformations. In this case, the ultimate limit state is predominant.

The stabilization of soil with cement increases the soil's compressive strength and reduces permanent deformation under loading. The application of lime in soil reduces the potential change in volume and improves its quality. Lime reduces soil plasticity hence improving constructability and consequently pavement performance. However, cement and lime stabilization are limited to unique and very special soil types and project conditions, results in curing time and challenges to obtain the correct mix.

Geosynthetics perform two or more functions at the same time such as in the case of pavements where geosynthetics are used to stabilize selected layers, while separating from the soft in situ soil therefore a separation and stabilization function work together. It is well known and documented that the lower the mechanical property of the in situ the more geosynthetic functions are incorporated in the design (Koerner, 2005).

## 2 Research

A research conducted at the University of Cape Town (Kiptoo, 2016) was aimed at demonstrating the benefit of geosynthetics in a multi-layered system using commercial sources for both soils and geosynthetics. A steel box of dimensions 1m x 1m x 1m was constructed to model a pavement at the University of Johannesburg. The thickness of the steel box was 25 mm and it was braced on the outside. Linear variable differential transformers (LVDTs) were used to monitor lateral movements on the box. Plate load tests were conducted at a rate of 1.2 mm/min to mimic undrained conditions through a circular plate of 305 mm in a Universal Compression Machine with a capacity of 500 kN. A two-layered soil system was built with a very soft subgrade underlying a G7 material (Figure 1). The geogrid or geotextile was each placed at the interface of the base and subgrade for the initial series of tests (Figure 2). This was then followed by placement of the geotextile at the interface and geogrid within the base at a depth of 200 mm from the interface for the subsequent tests

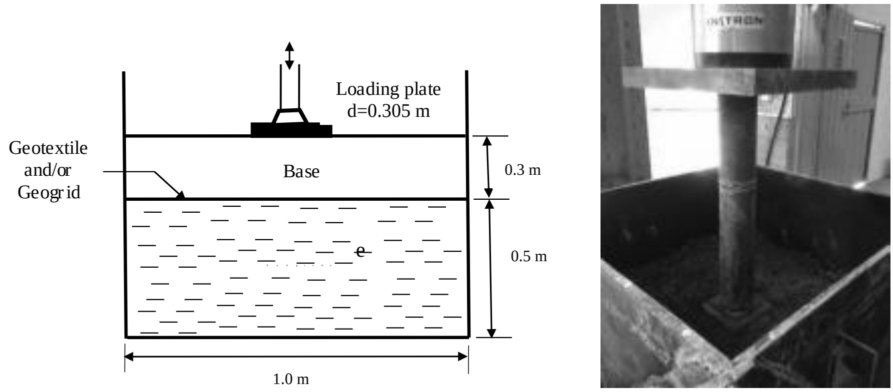


Figure 1. Schematic drawing of the test box and loading configuration

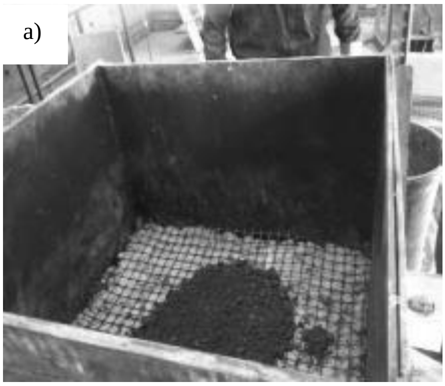

Figure 2. a) Geogrid at the interface

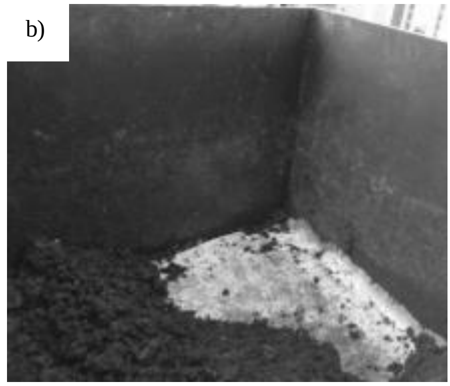

Figure 2. b) Geotextile at the interface

## 3 Materials

### 3.1 Subgrade material

A soft subgrade with an approximate CBR of 2 and an undrained shear strength of 41 kPa was modelled in the Laboratory using Kaolin (china) Clay. Standard Proctor tests were performed to obtain the compaction curve for the subgrade. The maximum dry density was found to be 1520 kg/m$^3$ and it corresponded to an optimum moisture content of 25.5%. To obtain the desired strength, the Kaolin was mixed at 31% moisture content as determined from the empirical equations of Talukdar (2014) for a CBR of 2. The mass of material required to fill the test pit for subgrade was then determined and compacted. The compaction was undertaken until the determined weight of material filled the volume for the subgrade soil at the 250 mm mark and the surface was evenly level. This process was repeated for the next layer until the entire subgrade layer was uniformly compacted and the surface level was at the 500 mm mark A torvane was used for quality control to ensure consistency in the undrained shear strength of the subgrade for all the tests.

### 3.2 Base material

G7 material, according to the South African Standards (TRH 14), was used as the base material. It was prepared by mixing a red gravelly soil with 10% Kaolin to achieve the characteristics of a G7. The base had a thickness of 300 mm and it was laid at 90% of the Maximum Dry density. The base material had the characteristics as shown in table 1.

Table 1. Base material Properties.

| Property | Soil parameter |
|---|---|
| % Passing 0.075mm | 47.4 |
| Grading Modulus | 1.95 |
| Plastic index (PI) | 13.1 |
| CBR after 4 day soak | 17 |
| CBR swell | 1.04 |
| Maximum dry density (Mod AASHTO) | 2000 Kg/m$^3$ |
| Optimum Moisture content (OMC) | 13 |

The material is classified as Clayey Sand (SC) and A-6 according to the Unified Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO) classification system respectively.

### 3.3 Geosynthetic properties

Two types of geosynthetics were used in this study: non-woven geotextile and an extruded geogrid. The index and mechanical properties are as provided by the manufacturer and are presented in Table 2. Both the geogrid and geotextile were made from polypropylene. The geogrid had a biaxial structure with a square apertures of size 38 mm by 38 mm. The geotextile had a characteristic opening size, $O_{90}$ of 300 microns. Overall, as shown in Table 2, the geotextile had a higher tensile strength than the geogrid.

Table 2. Geosynthetic Properties

| | **Woven Geotextile** *Mactex W1 5S* | **Extruded Geogrid** *Macgrid EG 20S* |
|---|---|---|
| Tensile Strength MD* kN/M | 50 | 20 |
| Tensile Strength CMD* kN/M | 50 | 20 |
| Strain at maximum strength MD* % | 20 | 13 |
| Strain CMD* % | 13 | 10 |
| Tensile strength at 2% strain Longitudinal kN/M | – | 7 |
| Tensile strength at 5% strain Longitudinal kN/M | – | 14 |
| Tensile strength at 2% strain Transverse kN/M | – | 7 |
| Tensile strength at 5% strain Transverse kN/M | – | 14 |

MD*=Machine Direction CMD*=Cross Machine Direction

## 4 Results – static testing

Failure was compared for: 1) the bearing capacity at 75 mm settlement and the bearing pressures measured to compute the bearing capacity ratio (BCR), 2) the settlement of the system for a worst case of unreinforced at 75 mm in relation to the reinforced pavement at the same pressure (Figure 3 on the following page).

The first part of the curve at small strain (0–40 mm) is not considered since compaction is taking place in the soil and the geosynthetic is straining under loading, this can be regarded as a transitory stage. From the results, the following is highlighted:

- the reinforced system outperforms the unreinforced system;
- a geogrid with a stiffness 40% lower than a geotextile achieves similar behaviour. The geogrid develops lateral restraint over and above the bearing capacity (membrane support is negligent at such deformations).
- the combination of a geotextile and a geogrid within the base generates a higher loading once the deformation has reached 60mm. The geogrid is able to spread the load through interlocking at lower deformation as it is placed closer to the loading, thus transferring a lower pressure to the layers beneath.

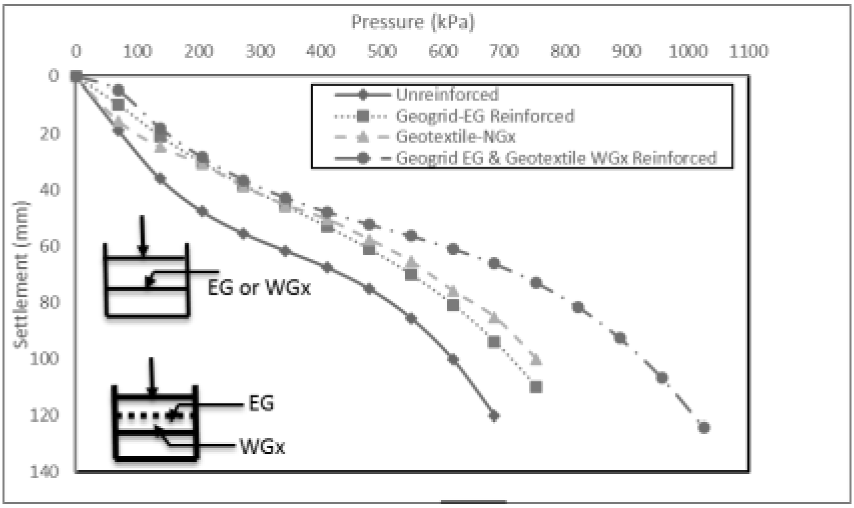


Figure 3. Pressure against settlement of the reinforced and unreinforced composite system (Kiptoo et al., 2017)

## 5 Results – dynamic testing

It is evident that permanent deformation increased with increased number of cycles for all test cases as shown in figure 4. The initial cycles (0–50) offered a high resistance to loading, this is attributed to the additional base compaction arising from the dynamic action. For every succeeding cycle, there was an accumulation of plastic non-recoverable deformations.

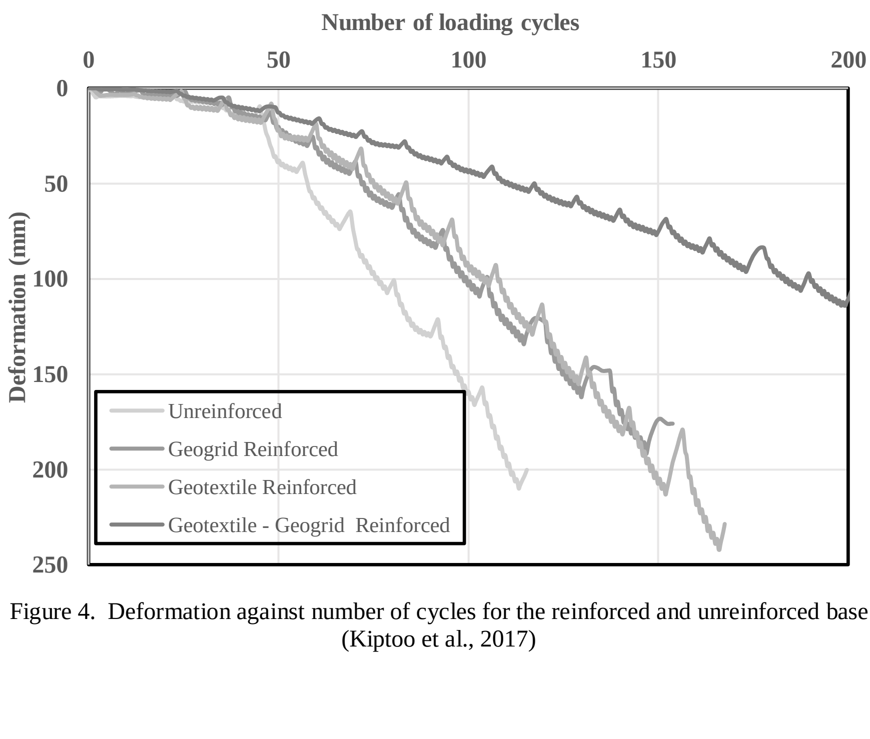


Figure 4. Deformation against number of cycles for the reinforced and unreinforced base (Kiptoo et al., 2017)

The rate of deformation was highest for the unreinforced pavement. From the results the following is highlighted:

- the reinforced system outperforms the unreinforced system
- as occurred in static test, even for dynamic test, the geogrid which has a stiffness lower than the geotextile develops similar results due to the lateral restraint;
- the geotextile at the interface and the geogrid in the layer seems to be the optimum configuration with the lowest deformation under cyclic loading.

## 6 Results – performance index

The performance in bearing capacity improvement of the reinforced pavement structure due to the provision of a geosynthetic is quantified through a non-dimensional parameter, the bearing capacity ratio. It is defined as the ratio of bearing pressure of the reinforced soil ($q_r$), at a given settlement, to the bearing capacity of the unreinforced soil ($q_u$) at the same settlement as follows:

$$BCR = \frac{q_r}{q_u} \tag{1}$$

From the plot of pressure against settlement (Figure 3), at a settlement of 75 mm, the improvement in bearing capacity as depicted by the bearing capacity ratio is 1.21 for the extruded geogrid, 1.29 for the woven geotextile and 1.63 for the combination of geotextile at the interface of base-subgrade and geogrid within the base.
Settlement reduction ratio (SRR) is defined as the percentage reduction in settlement and is expressed as follows:

$$SRR = \frac{S_o - S_r}{S_o}\, x\, 100 \tag{2}$$

Where $S_o$ is the settlement of unreinforced soil at a given footing pressure and $S_r$ the settlement of reinforced soil at the same footing pressure. From figure 3, for a settlement of the unreinforced pavement $S_o$ of 75 mm, a settlement reduction factor of 18% for the extruded geogrid, 23% for the woven geotextile and 31% for the combination of geotextile at the interface of base-subgrade and geogrid within the base was obtained.
Traffic benefit ratio (TBR) also sometimes referred to as Traffic impact factor (TIF) is defined as the number of load cycles carried by a reinforced section (Nr) at a specific rut depth divided by that of an equivalent unreinforced section (Nu).

$$TBR = \frac{Nr}{Nu} \tag{3}$$

The TBR therefore can be used to determine the number of traffic passes that a reinforced pavement can withstand compared to an unreinforced pavement for a given rutting depth (Gupta, 2010). The calculated TBR values are as shown below for a 75mm deformation.

Table 3. Calculated traffic bearing ratios

| | **Number of cycles** | **TBR** |
|---|---|---|
| Unreinforced (Nu) | 70 | 1 |
| Geotextile | 90 | 1.29 |
| Geogrid | 85 | 1.21 |
| Geotextile & Geogrid | 150 | 2.14 |

The results of the research are in line with general guideline reported by the Geosynthetic Materials Association (2000):

Table 4. Benefit of using geosynthetics in pavements

| **Benefit** | **General Anticipated Magnitude** | **Applicability** |
|---|---|---|
| Reducing undercut (i.e. the depth of excavation required for the removal of unsuitable subgrade materials) | Reduced up to 50% | CBR <3 ($M_R$< 30MPa) |
| Reducing the thickness of aggregate required to stabilize the subgrade | Reduced up to 50% | CBR <3 |
| Reducing disturbance of the subgrade during construction | Allows construction of relatively thin base (subbase) | ($M_R$< 30MPa) |
| Reinforcement of the subbase aggregate in roadway to reduce the section | Reduce up to 250mm with 75mm typical | Depends on depth of base and initial depth of base/subbase |
| Reinforcement of the base aggregate in roadway to reduce the section | Reduced up to 150mm with 75mm typical (20 to 50%) | Strong potential benefit |
| Reinforcement of the subbase aggregate in a roadway to increase its design life | TBR = 1 to 3.8 | Depends on depth of base and initial depth of base and subbase |
| Reinforcement of the subbase aggregate in a roadway to increase its design life | TBR = 1 to 10 | Strong potential benefit |
| Improved reliability | Improves performance during overload and/or seasonally weak subgrade conditions | Always a benefit |

## 7 Case Study – Static design – Cape Town Harbour

Beales et al (2017) reported on a project in 2013, where Transnet National Ports Authority (TNPA) upgraded the existing fire-fighting system of the oil tanker terminals in the Port of Cape Town. This included the construction of a new pump station, which housed booster pumps on the ground floor and a 250 kL water reservoir directly above the pump station on the first floor. The design bearing pressure exerted on the ground by this new structure was 150 kPa.
The pump station site is located inside the Port of Cape Town. The fill material in this area of the harbour consists of reclaimed soils, which were placed beginning in 1965 and comprise of variable thickness, consistency and composition. The fill material comprises hydraulically backfilled material derived from dredging activities and highly variable end tipped imported material.
The subsoil conditions are summarized as follows:

- Variable fill materials in terms of composition, low consistency and thickness creating compressible soil conditions (problem soil);
- Presence of large obstacles, such as tetrahedron dollies and very hard boulders up to 1.5m diameter as well as a rockfill layer at depth (quay construction), which could hamper piling installation;
- Soft and variable marine deposits in the order of 6.0 meters thick;

- Weathered meta-sedimentary strata associated with the Malmesbury rock with occasional discontinuities filled with sand; and
- Significant depth to competent rock

The design pressure of 150kPa and the CBR of 2% (in situ bearing capacity of 15kPa) were used for the design calculations. In unreinforced conditions, the foundation thickness required was in the order of 2m. However, with the use of geosynthetic reinforcements the thickness was reduced to 1m as shown in Figure 5. This was achieved by use of one layer of a woven polypropylene geotextile with an ultimate strength of 80kN/m in both directions that functioned as a separating layer. In addition, two layers of extruded geogrids, polypropylene bidirectional geogrids with a ultimate tensile strength of 40kN/m, were placed within a 400mm thick layer of a G5 material (TRH 14) (figure 6). The G5 material had a minimum CBR of 45 when compacted to a minimum of 95% of the modified AASHTO density.

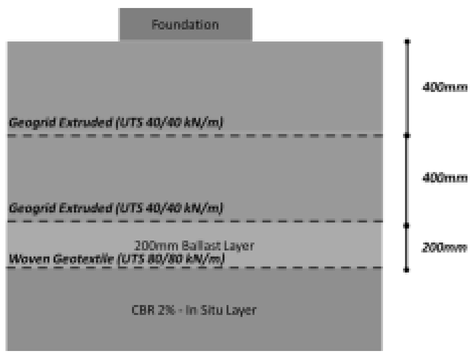


Figure 5. Foundation layout

Figure 6. Construction of the geosynthetic reinforced soil raft foundation

Plate load tests were undertaken on the in situ subgrade using a 600m plate diameter, in the pioneering layer as well as on the installed geosynthetic layers. The axial loads and the corresponding displacement were recorded at predetermined load increments and the resulting data was then used to generate the applied load vs deflection and subgrade modulus reaction curves (figure 7).

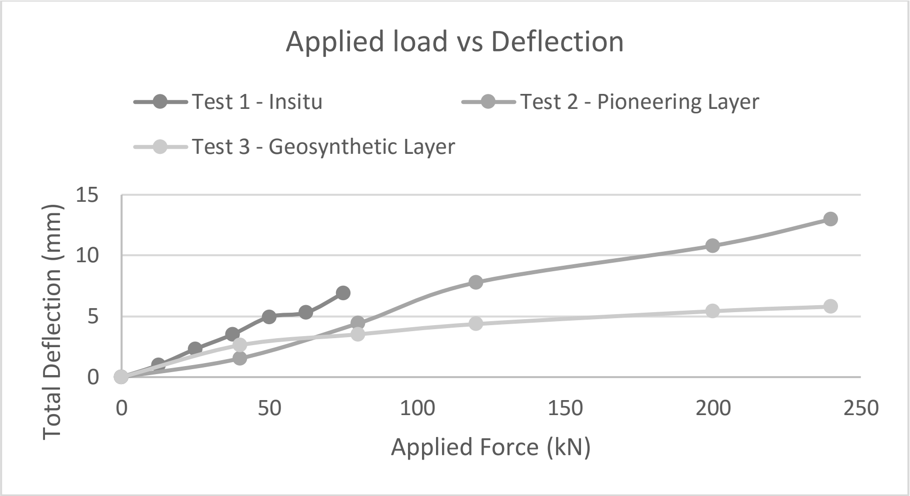


Figure 7. Total deflections vs applied force for the different layer works

The soil raft founding material (approximately 1.2m below ground level) comprised of loose, variably silty fine grained sandy material. Test 1 confirmed that the in situ material comprised generally of low strength soils (bearing capacity <40kPa), which necessitated ground improvement. During the construction phase the settlement of the Pump Station were measured on a weekly basis to record the average settlement of the structure. These readings confirmed that minimal settlement (less than 6mm) had occurred thus far (95% construction completed) and that this settlement was within the tolerance levels for the structure of less than 5mm.

## 8 Case Study – dynamic design – Morrison Road, Glentana

Zannoni and Barkhuizen (2016) reported on a project in Glentana, Western Cape – South Africa, where investigation of the road revealed that major deep-seated deformation/settlement had taken place over certain sections of the road. A geotechnical investigation using Dynamic Probe Super Heavy (DPSH) testing indicated the presence of a deep (up to 8m in certain locations), soft, low strength subgrade. SPT “N” values as low as 1 was recorded in certain locations due to penetration generally occurring under self-weight of the equipment, with no drop weight activation required, thus indicating a very poor subgrade. From the test pits, the subgrade material was classified as a sand containing organic decomposed material. One of the main design criteria was to maintain an undisturbed stress state in the soft, poor subgrade material to avoid deformation and resultant failure.
Traditional design run using the South African Mechanistic Pavement Design Method – SAMPDM considering a road Category B as per TRH 4 with an ES3 (3 million ESAL) resulted in a total pavement depth of 1.2m as shown in Figure 8a. The results from the model are shown in Figure 8b where two geogrids (30kN/m UTS) were placed, one in the G7 and one in the G4 base, thereby reducing the excavation from 1.2m to 0.7m. This resulted in a no-stress variance in the soft layer which would have failed due to the overburden pressure caused by the extra layer thickness. Furthermore, reduced excavation made it possible to maintain the he same road surface level which was paramount due to the main intersections and road annexures.

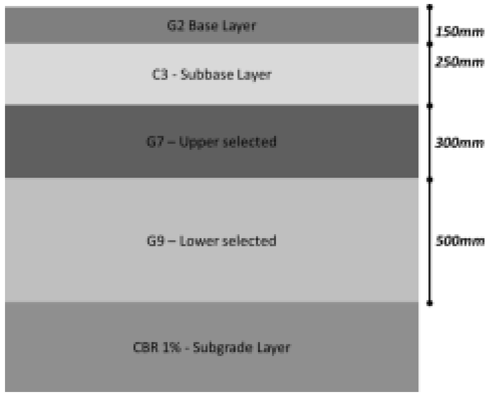


Figure 8. a) SAPDM pavement 1.2m thick

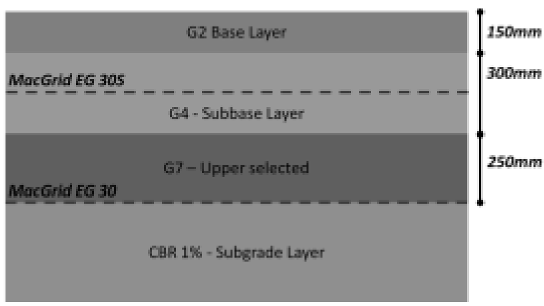


Figure 8. b) Geosynthetics design 0.7m thick


Figure 9. a) During construction

Figure 9. b) Completed project

## 9 Conclusions

Research in South Africa using local materials has proven to be valid as the results are in line with other researches undertaken around the world. The use of geosynthetics always outperform the unreinforced scenario in both performances and in total project cost between 30 and 50% for a paved road, which only considers the stabilization function discussed in the paper, excluding drainage and stress relief (asphalt reinforcement) function which can be included.

While the complexity of including a geosynthetic in the design cannot be easily assessed as it is paramount to have knowledge of the behaviour of geosynthetics, its interaction with the soil and the criteria for design. The following statements can be highlighted:

- The use of a geosynthetic in a static and dynamic loading scenario over soft soil outperform the behaviour of the system compared to an unreinforced scenario;
- A geogrid with a stiffness of 40% lower than a geotextile produces similar results due to the lateral restraint of the soil within the geogrid;
- A geogrid included in the middle of the layer together with a geosynthetic at the interface produces 30% BCR than geotextile only or geogrid at the interface;
- The SRR (Settlement Reduction Ratio) for the double geosynthetics decrease the settlement to 31% for a control settlement of 75mm.

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# True Bridge Abutment Using Geosynthetic Reinforced Soil: Principles and Applications

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## Abstract

The use of geosynthetic-reinforced soil systems for true bridge abutment solutions (where the girder rests on the soil fill) provides for a cost-effective solution compared to reinforced concrete abutments. Using a geosynthetic-reinforced soil system as a true bridge abutment may results in faster construction (approximately 15%), cost saving (approximately 25%) and higher tolerance to displacements. With geosynthetic-reinforced soil systems as true bridge abutments, geogrids and/or geotextiles are used to reinforce the soil mass in layers which, in turn, are connected to facing elements that form the outer wall and prevent erosion. Geosynthetic-reinforced soil systems have been proven to be a cost-effective solution compared to reinforced concrete or other similar solutions. This paper illustrates the major benefits of using geosynthetic-reinforced soil systems as true bridge abutments and its application on a project at Black Rock Mine near Kuruman in the Northern Cape Province, South Africa.

***Keywords:*** *Geosynthetic-reinforced, True Bridge Abutment, Cost-effective.*

## 1 Introduction

Over the past four decades, geosynthetic-reinforced soil systems have been successfully used in critical structures such as retaining walls, embankments, shallow foundations and bridge abutments as a key element to achieve the desired stability of the structure. These systems have also been receiving additional interest and attention for their use in bridge abutments.

Geosynthetic-reinforced soil systems can be adapted to a variety of site conditions, such as weak foundations or low-quality fill with presence of silt or clay. They achieve a faster construction time as there is no curing time involved (which is necessary for a concrete structure), therefore the system works immediately. Fill material must however comply with the requirements set out in BS 8006:2012. The FHWA and the Colorado DOT have conducted full-scale tests on geosynthetic-reinforced soil bridge abutments and piers with segmental modular block facings which produced high load-carrying capacity and exceptional performance (NCHRP, Report 556).

## 2 Mechanically Stabilized Earth Walls

Reinforcing a soil mass by including a material that is strong in tensile resistance is comparable to the behaviour of reinforced concrete. The mechanism of reinforcement between reinforced concrete and reinforced soil is however quite different. In reinforced soil, the soil-geosynthetic interface friction is where the bonding between soil and the reinforcement element is derived from. The reinforcement restrains lateral deformation of the soil next to the reinforcement element, through the interface friction, thereby increasing the stiffness and strength of the soil mass. The soil straining under loading will strain the geosynthetic through interface friction which in turn develops strength in the geosynthetic, thereby restraining the soil laterally.

Two Primary methods for the stabilization of soil has evolved in modern reinforced soil technology: mechanically stabilized earth (MSE) and geosynthetic reinforced soil (GRS). The predominant method for constructing reinforced soil today is MSE. One patented method was introduced in the 1960s and incorporates discrete steel strips embedded within a soil mass. Other types of reinforcement materials, classified as either extensible or inextensible, have since been developed and are used to reinforce soil.

MSE technology has however branched off into two primary paths: proprietary structures built with metallic (inextensible) reinforcements and proprietary structures built with geosynthetic (extensible) reinforcements, each having a unique combination of precast panels, connection details and reinforcements (see Figure 1).

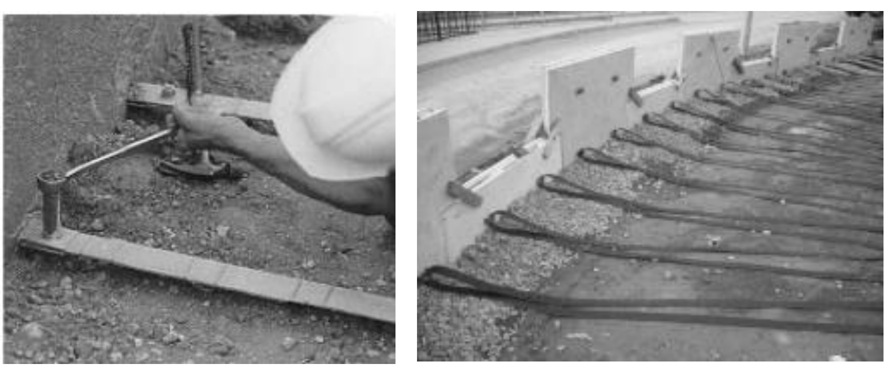

Figure 1. Different connection details and types of reinforcements used.

Extensible reinforcements were introduced in MSE structures during the 1980s, whereby geogrids were used as the main reinforcing element in the fill within a concrete modular block structure. More recently, extensible geosynthetic straps have been used to replace geogrids in structures after much research and development. Performance of these structures have been assessed through the measurements of various parameters and criterion. This concept is like that of building a MSE structure with metallic strips as was the common practice but has successfully been challenged with Geosynthetic solutions. The concept being the same and the fill specifications, method of placement and compaction, design strength of the reinforcement straps and length of embedment within the compacted fill remain the key design factors when designing these structures.

Many different reinforcement elements and facing systems have been developed over the years to suit client requirements of retaining structures that have been built with this technology. The polymeric strap, ParaWeb®, is the first polymeric material that has been successfully tried and tested as a reinforcing element in reinforced soil walls and abutments.

The motivation for the use of a geosynthetic reinforcing element in reinforced soil walls and abutments came in the 70's to satisfy the problem of corrosion encountered on the galvanized steel strips solution when exposed to highly aggressive saline conditions. Unlike other developments for this application, ParaWeb® stood the test of time and confirmed its capability to withstand such conditions. The first ParaWeb® reinforced test wall-was constructed by the Transport Research Laboratory at their Crowthorne facility in 1977. Samples are, periodically, still being exhumed for testing purposes to demonstrate the long-term performance of the material (Balderson, 2005). Tests on these samples, 28 years after the construction of the wall, show that no significant reduction in mean tensile strength have occurred as indicated by the mean stress strain relationship in Figure 2 below.

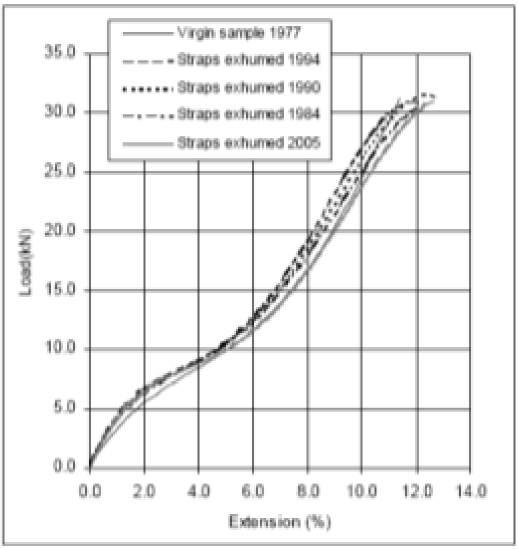


Figure 2. Stress – Strain curves of exhumed samples (Balderson, 2005).

Brady presented a detailed report in 1987 (TRRL, Research Report 111) on the observations recorded from the instrumentation of a bridge abutment reinforced with geosynthetic straps as the main element of reinforcing. The monitoring of settlement at various locations along the abutment indicated that the differential settlement between the centre and ends of the abutment was about 160mm and the maximum angular distortion was about 1/110. It is almost certain that under these conditions a conventional reinforced concrete wall would have suffered structural damage.

## 3 Design of MSE Walls and Abutments

The design of reinforced walls and abutments should be in accordance with BS 8006:2012 (code of practice for Strengthened/reinforced soils and other fills). The limit state design for reinforced soil walls and abutments should be implemented by increasing the soil weight and live loadings using appropriate partial factors, and reducing the soil properties and reinforcement strength by appropriate partial material factors. The Partial factors to be used are summarized in Table 1 on the following page.

Table 1. Partial factors to be used in the design of reinforced walls and abutments (*BS 8006-1:2012)*.

| **Partial factors** | | **Ultimate limit state** | **Serviceability limit state** |
|---|---|---|---|
| Load factors | Soil unit weight density e.g. wall fill | The appropriate value of $f_{fs}$ to be chosen according to Table 4 and Table 5 for the particular load combinations | |
| | External dead loads e.g. line or point loads | The appropriate value of $f_f$ to be chosen according to Table 4 and Table 5 for the particular load combinations | |
| | External live loads e.g. traffic loading | According to Table 4 and Table 5 for the particular load combinations | |
| Soil material factors | to be applied tan f'p | $f_{ms} = 1.0$ | $f_{ms} = 1.0$ |
| | to be applied to c' | $f_{ms} = 1.6$ | $f_{ms} = 1.0$ |
| | to be applied to cu | $f_{ms} = 1.0$ | $f_{ms} = 1.0$ |
| Reinforcement material factor | to be applied to the reinforcement base strength | The value of $f_m$ should be consistent with the type of reinforcement to be used and the design life over which the reinforcement is required | |
| Soil/reinforcement interaction factors | Sliding across surface of reinforcement | $f_s = 1.3$ | $f_s = 1.0$ |
| | Pull-out resistance of reinforcement | $f_s = 1.3$ | $f_s = 1.0$ |
| Partial factors of safety | Foundation bearing capacity: to be applied to $q_{ult}$ | $f_{ms} = 1.35$ | NA |
| | Sliding along base of structure or any horizontal surface where there is soil-to-soil contact | $f_s = 1.2$ | NA |

As shown above, the partial material factor to be applied to the design depends on the type of material used. This indicates that a variety of materials can be used if the appropriate partial factors are applied and the material meets all design specifications.

## 3.1 The Reinforcement

The design strengths of the reinforcement straps are calculated as shown below.

Ultimate Limit State (ULS): $$T_D = T_{CR}/(\mathrm{f}_n \times \mathrm{f}_m) \quad (1)$$

Serviceability Limit State (SLS): $$T_D = T_{Cs}/\mathrm{f}_m \quad (2)$$

where

$T_{CR}$ is the long-term tensile creep rupture strength of the reinforcement at the specified design life and design temperature.
$T_{CS}$ is the maximum allowable tensile load to ensure that the prescribed post-construction, limiting strain specified for the SLS is not exceeded.
$f_n$ is the partial factor for ramification of failure in accordance with BS 8006-1: 2012, Table 9.
$f_m$ is the material safety factor to allow for the strength reducing effects of installation damage, weathering (including exposure to sunlight), chemical and other environmental effects and to allow for the extrapolation of data used to establish the above reduction factors.

and

$$T_{CR} = T_{char}/\mathrm{RF}_{CR} \tag{3}$$

where

$T_{char}$ is the characteristic short-term strength supplied by manufacturer.
$RF_{CR}$ is the reduction factor for creep supplied by manufacturer.

and

$$f_m = RF_{ID} \times RF_W \times RF_{CH} \times f_S$$
(4)

where

$RF_{ID}$ is the reduction factor for installation damage supplied by manufacturer.
$RF_w$ is the reduction factor for weathering, including exposure to ultra violet light supplied by manufacturer.
$RF_{CH}$ is the reduction factor for chemical/environmental effects supplied by manufacturer.
$f_S$ is the factor of safety for the extrapolation of data supplied by manufacturer.

For serviceability limit state, the prescribed maximum allowable post-construction creep strains allowed by BS 8006-1: 2012 reinforced soil retaining walls and bridge abutments are shown in Table 2 below.

Table 2. Serviceability limits on post-construction internal strains for bridge abutments and retaining walls (*BS 8006-1:2012)*.

| **Structure** | **Strain (%)** | **Design period for the purposes of determining limiting strain** |
|---|---|---|
| Bridge abutments and retaining walls with permanent structural loading | 0.5 | 2 months – 120 years |
| Retaining walls, with no applied structural loading i.e. transient live loadings only | 1.00 | 1 month – 120 years |

Geosynthetic reinforcement straps can clearly be used as reinforcing element in bridge abutments and walls if it conforms to strain limits as set out by the BS 8006:2010. The manufacturer of the geosynthetic reinforcement element should however be able to provide certificates of the long-term creep tests being done, thereby proving the long-term creep reduction factors to be used in the design of the structure with the said element.

An example of reduction factors $RF_{CR(SLS)}$ for determining $T_{CS}$ from the characteristic short-term tensile load ($T_{char}$) for each grade of reinforcement straps are given in Table 3. The following formula is used to calculate $T_{CS}$: $T_{CS} = T_{char}/RF_{CR(SLS)}$

Table 3. Long-term creep reduction factors for serviceability limit state for a 120-year design life and design temperature of 20°C (*BBA – HAPAS Certificate 12/H191*).

| **Prescribed allowable post-construction strain (%)** | **RFCR (SLS)** |
|---|---|
| 0.50 | 2.00 |
| 1.00 | 1.54 |

It is thus shown that geosynthetic reinforcing elements do experience long term creep but it is catered for by using reduction factors to ensure conformance to the post construction strain limits for structures as set out by the BS 8006:2012.

## 3.2 Loadings

The largest loads likely to be applied to the structure should be considered in design. The partial load factors that should be applied to each component of load for different load combinations are listed in Table 4 and Table 5.

Table 4. Partial load factors for load combinations associated with walls (*BS 8006-1:2012)*.

| **Effects** | | **Combinations** | | |
|---|---|---|---|---|
| | | **A** | **B** | **C** |
| Mass of the reinforced soil body | | $f_{fs}$= 1.5 | $f_{fs}$ = 1.0 | $f_{fs}$ = 1.0 |
| Mass of the backfill on top of the reinforced soil wall | | $f_{fs}$= 1.5 | $f_{fs}$ = 1.0 | $f_{fs}$ = 1.0 |
| Earth pressure behind the structure | | $f_{fs}$= 1.5 | $f_{fs}$= 1.5 | $f_{fs}$ = 1.0 |
| Traffic load: | on reinforced soil block | $f_q$ = 1.5 | $f_q$ = 0 | $f_q$ = 0 |
| | behind reinforced soil block | $f_q$ = 1.5 | $f_q$ = 1.5 | $f_q$ = 0 |

NOTE The following descriptions of load cases identify the usual worst combination for the various criteria but are for guidance only. All load combinations should be checked for each layer of reinforcements within each structure to ensure the most critical condition has been found and considered.

Combination A: This combination considers the maximum values of all loads and therefore normally generates the maximum reinforcement tension and foundation bearing pressure. It may also determine the reinforcement requirement to satisfy pull-out resistance although pull-out resistance is usually governed by combination B.

Combination B: This combination considers the maximum overturning loads together with minimum self-mass of structure and superimposed traffic load. This combination normally dictates the reinforcement requirement for pull-out resistance and is normally the worst case for sliding along the base.

Combination C: This combination considers dead loads only without partial load factors. This combination is used to determine foundation settlements as well as generating reinforcement tensions for checking the serviceability limit state.

Table 5. Partial load factors for load combinations associated with abutments (*BS 8006-1:2012)*.

| Effects | | Combinations | | |
|---|---|---|---|---|
| | | **A** | **B** | **C** |
| Dead load of the structure | | $f_{fs}$ = 1.5 | $f_{fs}$ = 1.0 | $f_{fs}$ = 1.0 |
| Dead load of the fill on top of the structure | | $f_{fs}$ = 1.5 | $f_{fs}$ = 1.0 | $f_{fs}$ = 1.0 |
| Dead load of bridge and bank seat | | $f_f$ = 1.2 | $f_f$ = 1.0 | $f_f$ = 1.0 |
| Backfill pressure behind the bank seat | | $f_{fs}$ = 1.5 | $f_{fs}$ = 1.5 | $f_{fs}$ = 1.0 |
| Backfill pressure behind the structure | | $f_{fs}$ = 1.5 | $f_{fs}$ = 1.5 | $f_{fs}$ = 1.0 |
| Horizontal loads due to creep and shrinkage | | $f_f$ = 1.2 | $f_f$ = 1.2 | $f_f$ = 1.0 |
| Traffic loading | | Over the entire structure, $f_q$ = 1.5 | Behind the reinforced zone, $f_q$ = 1.5 | |
| Bridge vertical live load | HA | $f_q$ = 1.5 | $f_q$ = 1.5 | |
| | HA and HB | $f_q$ = 1.3 | $f_q$ = 1.3 | |
| Braking dynamic load | HA | $f_q$ = 1.25 | $f_q$ = 1.25 | |
| | HA and HB | $f_q$ = 1.1 | $f_q$ = 1.1 | |
| Temperature effects | | $f_q$ = 1.3 | $f_q$ = 1.3 | |

NOTE 1 The following descriptions of load cases identify the usual worst combination for the various criteria but are for guidance only. All load combinations should be checked for each layer of reinforcements within each structure to ensure the most critical condition has been found and considered.

NOTE 2 The designations HA and HB are currently under review by the Highways Agency.

NOTE 3 Details of the traffic loads to be used when evaluating the traffic surcharge pressures are given in NA to BS EN 1991-2.

## 4 MSEW Systems as True Bridge Abutments on Black Rock Mine, South Africa

The contract was awarded, in October 2015, to the contractor Stefanutti Stocks Road & Earthworks. Due to the time constraints Maccaferri SA (Pty) Ltd t/a Maccaferri Africa was approached to propose an alternative solution for the insitu cast cantilever steel reinforced concrete wall originally specified to serve as the bridge abutment. The solution proposed was a Mechanically Stabilised Earth Wall (MSEW) solution namely the MacRes® T system. This system generally consists of a granular structural backfilling which is reinforced with horizontal layers of high strength polymeric reinforcing strips known as ParaWeb® which produce an apparent cohesion in the direction of the reinforcement and permits the fill to function as a homogenous gravity structure. These straps are planar structures consisting of a core of high tenacity polyester yarn tendons encased in a polyethylene sheath. The purpose of the sheath is to reduce installation damage and to protect the yarn from abnormal pH levels within the soil which renders this choice of reinforcement extremely durable. Graded fill material with angular particles of approximately 35mm is recommended for use with these straps. The vertical outer face of the reinforced soil structure comprises of a concrete panel cladding which is connected to the reinforcing straps which are embedded in the structural backfill.

A total of four retaining walls, 7.5m in height, were required by Black Rock Mine to accommodate tipper trucks dumping material onto conveyor belts for further transportation. Apart from the time and cost constraints, very high loadings and weak founding material had to be overcome in the proposal. The retaining walls were designed not only to retain the fill, which is characterized by a very high density (26 kN/m$^3$), but also a more stringent requirement was the girder and subsequent bridge bearing on the walls. This generated significant loads and shear forces, thus effectively demanding performance as a true bridge abutment. Figure 3 below shows two of the four walls, just after completion, that supported the bridge deck spanning between them.

Figure 3. Two of the Four Completed Walls

The design of two sets of true bridge abutments, 7m apart and varying only in dimension, was analyzed by Maccaferri’s Technical team, using specialized in-house design software based on the BS 8006:2012 (code of practice for Strengthened/reinforced soils and other fills). The Stability analyses for all four walls were done using the Limit Equilibrium Method. Each set consisted of two walls facing each other and were submitted to the mining consultant DRA for review and confirmation. DRA in turn designed the concrete bridge slabs to span the two walls supported by the MSEW system (acting as the bridge). This was to allow trucks to drive over the center of the span and dump material through a grizzly chute onto the conveyor system below (see Figure 4).

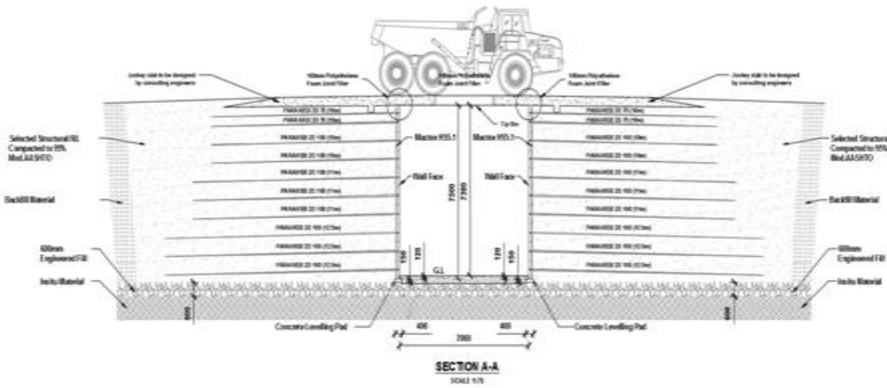

Figure 4. Typical Section of the Wall

The backfill material consisted of a blend of Manganese and sand, which was available on site as waste material of the mineral production. This is characterized by a very high unit weight of 26kN/m³ while having an internal friction angle of 34° and zero cohesion. Apart from this heavy backfill material used, 67 ton trucks were used to tip the material which generated large stresses (up to 304 kPa in the lower reinforcement levels) on the system.

After the initial set of designs were adjusted to cope with the heavy backfill material and trucks, it was found that the constraint of bearing pressure from the wall was higher than the bearing capacity of the foundation material. The foundation was accordingly improved by excavating to a depth of 600mm under the footprint of the wall and replaced with compacted engineered fill material found on site. To decrease the bearing pressure from the wall, the reinforcement lengths were increased (up to 12.5m in some sections) to allow for the load to be spread over a larger area. Different reinforcement straps with strengths of 75kN or 100kN were used at various levels in the wall depending on the requirement and which satisfied a design life of 120 years. It was decided to use galvanized steel connectors in the concrete panels to satisfy the high tensile forces that would be exerted on the structure due to heavy loadings from the backfill materials and trucks.

With the use of this rapid constructible and cost-effective solution the construction programme was accelerated and allowed for the two true bridge abutments (four walls which amounted to a face area of 850 m²) to be constructed in less than three months as required in order to satisfy the client to commission the works in December 2015 demonstrating to the speedy construction time of this solution.

## 5 Conclusions

Over the past few decades, geosynthetic-reinforced soil systems have been successfully used in critical structures such as retaining walls, embankments, shallow foundations and bridge abutments as a key element to achieve the desired stability of the structure. Numerous case histories and field observations have shown that geosynthetics can successfully be implemented in several applications, including walls and abutments, and it can also be used as the reinforcing element in true bridge abutment systems. Numerous partial factors are used to cater for events such as creep, strain, durability and installation damage of the geosynthetic reinforcement elements to meet design specifications of the structure.

The MSEW system in its totality, with geosynthetic straps being the reinforcing element, due to its inherent qualities such as simplicity, flexibility and speed of construction has been put to successful use for walls and even bridge abutments. These structures have demonstrated their ability to accommodate large differential settlements without compromising the structural integrity and functionality which confirms that their flexibility is a major advantage for these types of structures. The ability of geosynthetic reinforced structures to withstand large differential settlement (up to 300mm) can offer the client an option to reduce or eliminate elaborate ground improvements or foundation systems, that will otherwise be required for conventional structures. This can generate huge savings in total construction costs.

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# Designing a Paved Road Using Geogrids to Reduce the Thickness of the Pavement Layers

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## Abstract

Performance and durability of road pavements are significantly dependent on the strength and stability of the underlying soil layers, most especially the subgrade pavement layer. Currently, in Uganda most roads are constructed through low lying areas characterized by soft, hence weak, clay soils. The main practice, of improving the strength of such subgrade layers, has been to import stronger lateritic soils and dump them in layers over the weaker soils in thicknesses of more than 1.0 m. This is expensive, especially in terms of the haulage costs, and not environmentally friendly. Additionally, the lateritic soils are also getting depleted. Hence the need to utilize alternative means of increasing the strength of weak subgrades. This study focused on the application of Geogrids in pavement layers to reduce their overall thickness and life cycle costs of the road. A low-lying section on the Bajjo road, a bypass connecting Mukono to Seeta, was used as a case study. According to the AASHTO classification system of subgrade materials, the subgrade soils fell under the soil ranges of A-7, A-7-6, and A-6 group, therefore a poor subgrade material requiring stabilization. The average CBR was determined as 19%. The inclusion of the Geogrid reduced the overall layer works thickness by 25% and it's cost effective by 42% over the whole lifecycle of the road.

***Keywords:*** *Subgrade, Geogrids, Lateritic soils, Pavement thickness, Life cycle cost*

## 1 Introduction

Low volume roads, both paved and unpaved, usually serve as entrance or access roads to rural areas, towns and cities. They play an important role in rural economy, resource industries (forest, mining) and transportation to agricultural production areas. Constructing these roads on poor subgrade soils, usually leads to large deformations, which increase maintenance cost and interruption of traffic services. Leng (2002) states that, in general deterioration of unpaved and paved roads is faster than road replacement. Nonetheless, the increasing material, construction and maintenance costs make it important to explore alternative construction methods with longer service life, but at the same time remaining cost effective. Use of

Geosynthetics has been found to be a cost-effective alternative to improve poor sub-soils in adverse locations, especially in situations where there may be non-uniform quality or non-availability of desired soils with applications in almost all geotechnical engineering projects such as airport and highway pavements (Koerner, 2005). This study focused on utilizing biaxial Geogrids as reinforcement in pavement layer works.

## 2 Materials and Methods

### 2.1 Clay Soil

Samples of clay soil were obtained from Bajjo road, the project site, at depths between 0.5 m and 1.5 m. It was grey in color. Various laboratory classification and strength tests were carried out on clay specimens in accordance to BS1377, 1990. The results of these tests are shown in Table 1.

Table 1. Clay soil properties.

| **Soil property** | **Result** |
|---|---|
| Color | Dark greyish |
| Liquid Limit (%) | 33.3 |
| Plastic Limit (%) | 19.2 |
| Plastic index (%) | 14.1 |
| Optimum Moisture Content (%) | 14.2 |
| Linear shrinkage (%) | 7.1 |
| Maximum Dry Density (g/cm$^2$) | 1.699 |
| California Bearing Ratio at 95% MDD (%) | 19 |
| Gravels (%) | 0.5 |
| Sand (%) | 6 |
| Fines (%) | 93.5 |

The clay soil was classified as a fine grained lean clay of medium plasticity (CL) based on the Unified Soil Classification System. According to AASHTO classification system of subgrade materials (ASTM D 3282, 2004), the soil fell under the soil range A-7, A-7-6 and A-6 group hence a poor subgrade material that required stabilization. This was because the liquid limit and plastic limit exceeded the minimum values of 41% and 11% respectively. According to ETL1110-1-189 (2002, 2003), Geogrids can be applied to soils with CBR of 19% for reinforcing the base.

### 2.2 Lateritic Soil

Samples of lateritic soils were obtained from the Uganda Christian University borrow pit. It was reddish brown in color. Various laboratory classification and strength tests were carried out on lateritic soil specimens in accordance to BS1377, 1990. The results of these tests are shown in Table 2.

Table 2. Lateritic soil properties.

| Soil property | Result |
|---|---|
| Color | Reddish brown |
| Liquid Limit (%) | 37.5 |
| Plastic Limit (%) | 24.3 |
| Plastic index (%) | 13.2 |
| Optimum Moisture Content (%) | 14.3 |
| Linear shrinkage (%) | 6.4 |
| Maximum Dry Density (g/cm$^2$) | 1.994 |
| California Bearing Ratio at 98% MDD (%) | 61 |

The CBR value was found to be 61% satisfying the minimum base course CBR requirements according to TM 5-822-5 of 50%.

## 2.3 Geogrids

The specifications for the Geogrids were obtained from ETL 1110-1-189 (2003). It outlines the common engineering Geogrid properties with lower limits below which the Geogrids should not be used.

## 2.4 Traffic counts

Traffic counts, along Bajjo road, were carried out for seven consecutive days starting at 7:00 am to 10:00 pm at an interval of 15 minutes. The traffic was assembled into groups 1, 2, 3 from the lightest to the heaviest respectively according to TM 5-822-5. The traffic was projected for 25 years at a growth rate of 7% in accordance with UNRA (2008) and the design hourly volume (DHV) estimated, according to TM 5-822-2. Table 3 shows the obtained Average Daily Traffic (ADT).

Table 3. Average daily traffic.

| Traffic Category | ADT |
|---|---|
| Boda | 1087 |
| Passenger cars | 761 |
| Mini buses | 115 |
| Small trucks | 107 |
| Medium buses | 53 |
| Coasters | 24 |
| Larger buses | 10 |
| Heavy trucks (2 axles) | 16 |
| Heavy trucks (3 axles) | 2 |
| Total | 2175 |

The traffic was of category II and group 1 with 14.31% two-axle trucks in accordance to TM-5-822-5. An expression for projecting the traffic was obtained as shown in Equation 1.

$$DT = T * \frac{(1+0.07)^{25}-1}{0.07} = 63.249T \tag{1}$$

Table 4 indicates the projected traffic. The percentage of the heavy traffic was acquired from the projected values so that the design hourly volume could be obtained. Bajjo road is located in a flat terrain and open area. Total percentage of heavy traffic was 7.33%, with a total daily heavy traffic count of 5059. The design hourly volume (DHV) for roads can be estimated to be 15% of vehicles from ADT, (TM 5-822-2).

$$DHV = \frac{15*5059}{100*24} = 32 \quad (2)$$

Hence a road class of E as obtained from Table 5. In accordance with TM 5-822-2, for traffic category II and road class E the design index was 2, determined from Table 6.

Table 4. Traffic projections

| **Type of traffic** | **Current traffic** | **Unidirectional traffic (T)** | **DT = 63.249T** | **Group according to TM** | **% of total DT** |
|---|---|---|---|---|---|
| Boda | 1087 | 544 | 34408 | 1 | 49.91 |
| Passenger cars | 761 | 381 | 24098 | 1 | 34.95 |
| Mini buses | 115 | 58 | 3668 | 1 | 5.32 |
| Small trucks | 107 | 54 | 3415 | 2 | 4.95 |
| Medium buses | 53 | 27 | 1708 | 1 | 2.48 |
| Coasters | 24 | 12 | 759 | 2 | 1.10 |
| Larger buses | 10 | 5 | 316 | 2 | 0.46 |
| Heavy trucks (2 axles) | 16 | 8 | 506 | 3 | 0.73 |
| Heavy trucks (3 axles) | 2 | 1 | 63 | 3 | 0.09 |
| Total | 2175 | 1090 | 68941 | | 100 |

Table 5. DHVs for different road classes (TM 5-822-2)

| **Class** | **Road** | **Street** |
|---|---|---|
| A | >=900 | >=1200 |
| B | 720-899 | 1000-1199 |
| C | 450-719 | 750-999 |
| D | 150-449 | 250-749 |
| E | 10-149 | 25-249 |
| F | <10 | <25 |

Table 6. Pavement Design Index (ETL 1110-1-189, 2003)

| **Traffic Category** | **Pavement Design Index by Road/Street Class** | | | | | |
|---|---|---|---|---|---|---|
| | **A** | **B** | **C** | **D** | **E** | **F** |
| I | 2 | 2 | 2 | 1 | 1 | 1 |
| II | 3 | 2 | 2 | 2 | 2 | 1 |
| III | 4 | 4 | 4 | 3 | 3 | 2 |
| IV | 5 | 5 | 5 | 4 | 4 | 3 |
| IVA | 6 | 6 | 6 | 5 | 5 | 4 |
| V(60-kip tracked vehicles or 15-kip forklifts) | 7 | 7 | 7 | 7 | 7 | - |
| 500/day | 6 | 6 | 6 | 6 | 6 | - |
| 200/day | 6 | 6 | 6 | 6 | 6 | - |
| 100/day | 6 | 6 | 6 | 6 | 6 | 6 |
| 40/day | 6 | 6 | 6 | 5 | 5 | 5 |
| 10/day | 5 | 5 | 5 | 5 | 5 | 5 |
| 4/day | 5 | 5 | 5 | 5 | 4 | 4 |
| 1/day | 5 | 5 | 5 | 4 | 4 | 4 |
| VI (90-kip tracked vehicles or 25-kip forklifts) | | | | | | |
| 200/day | 9 | 9 | 9 | 9 | 9 | - |
| 100/day | 8 | 8 | 8 | 8 | 8 | 8 |
| 40/day | 7 | 7 | 7 | 7 | 7 | 7 |
| 10/day | 6 | 6 | 6 | 6 | 6 | 6 |
| 4/day | 6 | 6 | 6 | 6 | 6 | 6 |
| 1/day | 5 | 5 | 5 | 5 | 5 | 5 |
| 1/week | 5 | 5 | 5 | 4 | 4 | 4 |
| VII (120-kip tracked vehicles) | | | | | | |
| 100/day | 10 | 10 | 10 | 10 | 10 | 10 |
| 40/day | 9 | 9 | 9 | 9 | 9 | 9 |
| 10/day | 8 | 8 | 8 | 8 | 8 | 8 |
| 44/day | 7 | 7 | 7 | 7 | 7 | 7 |
| 1/day | 6 | 6 | 6 | 6 | 6 | 6 |
| 1/week | 5 | 5 | 5 | 5 | 5 | 5 |

## 2.5 Design procedure

The following procedure was utilised in the design of the pavement layers:

- The California Bearing Ratio (CBR) value for the subgrade was determined and noted down. This was used to obtain the subgrade class.
- From the traffic counts Average Daily Traffic and Daily Hourly Vehicle values were calculated to get the traffic class.
- Figure 1 was used to get the thickness of the pavement.
- Table 7 was utilised to determine the minimum asphalt concrete (AC) thickness values for the surface. Final pavement structure is dependent on the minimum AC values.
- The geogrid-reinforced aggregate thickness (base) was taken from the equivalency chart, Figure 2. The reinforced aggregate thickness was determined by subtracting the minimum AC thickness from the equivalent reinforced aggregate thickness in the pavement design. Hence forming the reduced thickness and reinforced pavement layers.

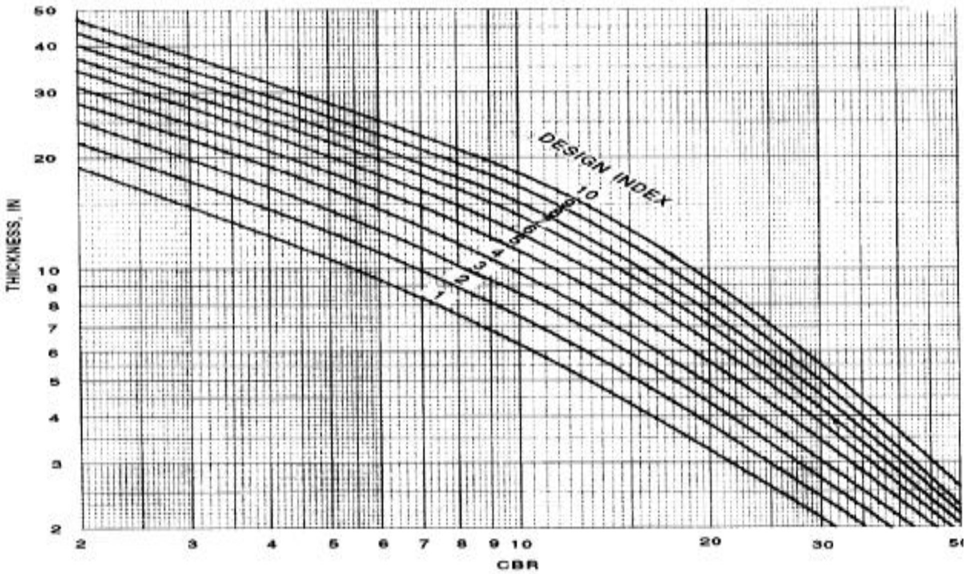


Figure 1. Flexible pavement design curves for roads and streets (TM 5-822-5).

## 2.6 Cost comparison

This was done by first assuming unreinforced pavement road and comparing it to the reinforced road with the Geogrid designed using the same materials covering same dimensions of both roads. Then the total materials cost used for the two roads were compared by way of estimating the volumes for the material used and the area of Geogrid used to cover the materials that fit in the dimensions and multiplying with unit costs of materials, to ascertain the cost effectiveness of the design. The service life was projected to 25 years. Maintenance cost was taken as a percentage of the initial cost.

Table 7. Minimum pavement layer thickness (TM 5-822-5/AFM 88-7)

**Minimum Thickness of Pavement Layers (extracted from Table 6-1 of TM 5-822-5)[1]**

| | Minimum Base Course CBR | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 100 | | | 80 | | | 50[2] | | |
| Design Index | Pavement (in.) | Base (in.) | Total (in.) | Pavement (in.) | Base (in.) | Total (in.) | Pavement (in.) | Base (in.) | Total (in.) |
| 1 | ST[3] | 4 | 4.5[5] | MST[4] | 4 | 4.5[5] | 2 | 4 | 6 |
| 2 | MST[4] | 4 | 5[5] | 1.5 | 4 | 5.5[5] | 2.5 | 4 | 6.5 |
| 3 | 1.5 | 4 | 5.5[5] | 1.5 | 4 | 5.5[5] | 2.5 | 4 | 6.5 |
| 4 | 1.5 | 4 | 5.5[5] | 2 | 4 | 6 | 3 | 4 | 7 |
| 5 | 2 | 4 | 6 | 2.5 | 4 | 6.5 | 3.5 | 4 | 7.5 |
| 6 | 2.5 | 4 | 6.5 | 3 | 4 | 7 | 4 | 4 | 8 |
| 7 | 2.5 | 4 | 6.5 | 3 | 4 | 7 | 4 | 4 | 8 |
| 8 | 3 | 4 | 7 | 3.5 | 4 | 7.5 | 4.5 | 4 | 8.5 |
| 9 | 3 | 4 | 7 | 3.5 | 4 | 7.5 | 4.5 | 4 | 8.5 |
| 10 | 3.5 | 4 | 7.5 | 4 | 4 | 8 | 5 | 4 | 9 |

[1]Table 6-1 extracted from TM 5-822-5, Chapter 6.
[2]In general, 50 CBR Base Courses are only used for road classes E and F.
[3]Bituminous surface treatment (spray application).
[4]Multiple bituminous surface treatments.
[5]Minimum total pavement thickness for road classes A through D is 6 inches.

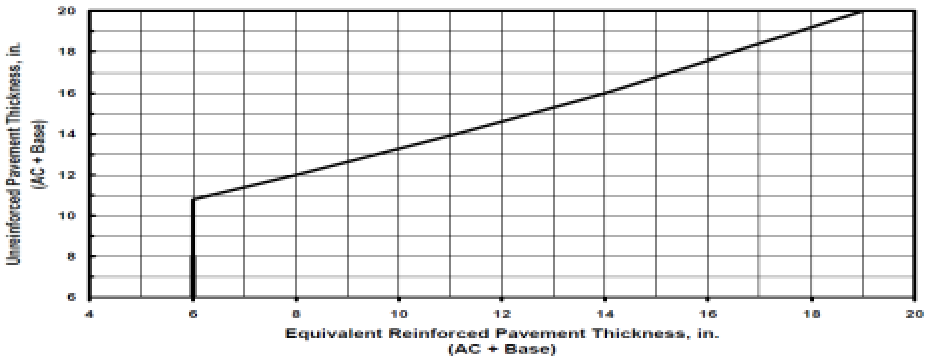


Figure 2. Webster's reinforced pavement thickness equivalency chart.
(ETL 1110-1-189, 2003)

## 3 Design of Pavement Layers

For the design subgrade CBR of 19 and the Design Index (DI) of 2, the total required pavement thickness was determined as 4in (101.6mm), Figure 1.

According to Table 7, for DI 2 and a road class E the minimum pavement thickness is 2.5in (63.5mm) and a base thickness of 4in (101.6mm). The minimum total thickness was taken to be 6.5in.

Additionally, the design index of 2 requires that a minimum thickness be 8in (203.2mm) at the CBR value of 95% MDD as indicated in Table 8. Consequently, the minimum total thickness was taken to be 8in.

Table 8. Depth of compaction for selected materials and subgrade.

| Design index | Depth of compaction* for percent compaction shown, in. | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Cohesive soils PI > 5; LL > 25 | | | | | Cohesionless soils PI ≤ 5; LL ≤ 25 | | | | |
| | 100 | 95 | 90 | 85 | 80 | 100 | 95 | 90 | 85 | 80 |
| 1 | 3 | 7 | 10 | 14 | 17 | 7 | 13 | 19 | 25 | 33 |
| 2 | 4 | 8 | 12 | 16 | 20 | 8 | 15 | 22 | 29 | 38 |
| 3 | 4 | 9 | 14 | 18 | 23 | 9 | 17 | 25 | 33 | 43 |
| 4 | 5 | 11 | 16 | 21 | 26 | 11 | 20 | 28 | 37 | 48 |
| 5 | 6 | 12 | 18 | 23 | 28 | 12 | 22 | 31 | 40 | 50 |
| 6 | 7 | 14 | 19 | 25 | 31 | 14 | 24 | 35 | 44 | 58 |
| 7 | 7 | 15 | 21 | 28 | 34 | 15 | 26 | 38 | 48 | 63 |
| 8 | 8 | 16 | 23 | 30 | 37 | 16 | 29 | 41 | 52 | 68 |
| 9 | 9 | 18 | 25 | 32 | 40 | 18 | 31 | 44 | 56 | 74 |
| 10 | 10 | 20 | 28 | 35 | 43 | 20 | 34 | 47 | 59 | 77 |

* Depth of compaction is measured from pavement surface.

From the Webster's design chart Figure 2, the minimum unreinforced thickness of 8in. gives an equivalent reinforced thickness of 6in (base plus AC) therefore the minimum AC thickness should be 4in. (101.6mm) and the aggregate thickness should be 2in. (50.8mm) Minimum, Figure 3.

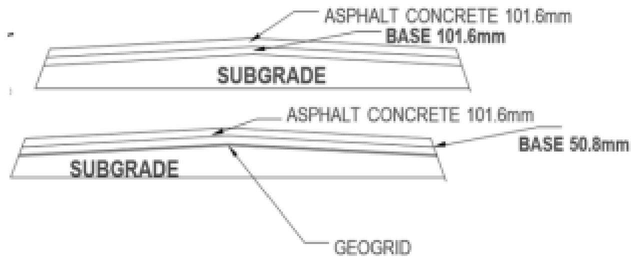


Figure 3. The geometric design of the pavement layers for both reinforced and unreinforced.

The camber used for the cross falls was 2.5%. This helps in attaining good drainage on the road surface. All other thicknesses remained the same apart from the reinforced base layer. The Geogrid is placed between the subgrade and the base interface for base thicknesses less than 14 inches and in the middle of the base layer for aggregate thicknesses greater than 14 inches.

## 4 Cost Comparison

The addition of Geogrids reduced the thickness of pavement by 2in, which is from 4in to 2in of the base material. This comparison was based on the difference in cost of the 2in. aggregate layer and the Geogrids. Both the reinforced and unreinforced pavements had 4in thickness of asphalt concrete layer. The comparison was done based on a 1km long, 6m wide and 2in (50.8mm) thick road.

The cost of lateritic soil per truck was determined at 40,000 Ugandan shillings (/=) of Tipping truck of 3.5m length x 2.0m wide x 0.5m deep, and cost of Geogrid as US \$0.50/$m^2$

### 4.1 Unreinforced road

Take a 1km section of a 6m wide road, 4in thick section without Geogrids.

$$V = L * W * H$$
$$V_{un} = 1000x6x4x2.54x10^{-2} = 609.6m^3 \quad (3)$$

Where: $V_{un}$ is the volume of the unreinforced pavement considering the base only

Volume of 1 truck is equal to

$$V_t = 3.5x2x0.5 = 3.5m^3 \quad (4)$$

Where: $V_t$ is the volume of a truck

So cost of $1m^3$ is equal to:

$$\text{Cost} = 40000/3.5 = 1429/= \quad (5)$$

Therefore: 609.6m3 cost (initial construction cost)

$$= 609.6x11429 = 6,967,118/= \quad (6)$$

## 4.2 Reinforced road

Take a 1km section of a 6m wide road, 2in thick section with Geogrids between the base and subgrade.

$V_r = 1000x6x2x2.54x10^{-2} = 304.8m^3$ (7)

Where $V_r$ is the volume of the reinforced road.

So using $V_t = 10.404m^3$ and cost of a truck 40,000/=
Cost of $1m^3$ = 11429/=
Therefore initial cost of $304.8m^3$ will be
= 304.8x11429 = 3,483,559/= (8)

Then add the cost of the Geogrids
$1m^2$ = \$0.5 (https://www.alibaba.com)
1\$ = 3,500/= from (http://www.xe.com)
So $1m^2$ = 0.5x3500 = 1750/=
Required Geogrid area
= 1000x6 = $6000m^2$
Considering an overlap of 1ft = 0.3048m for CBR >4% (ETL 1110-1-189 2003)
= $0.3048x1000m^2 = 304.8m^2$
Giving a total of $6304.8m^2$
Total cost of required Geogrid is
= 6,304.8x1750 =11,033,400/=
The estimated (shipping + transportation cost) = 5,000,000/= (9)

Therefore total initial construction cost
= 5000000+11,033,400+3483559 =19,516,959/= (10)

This shows clearly that initial construction cost using the Geogrid layer was higher than constructing without, after deducting the 2in thickness of the aggregate layer. Initial percentage cost increase due to reinforcement

$$= \frac{19{,}516{,}959 - 6967118}{6967118} * 100$$
$$\underline{\mathbf{= 180\%}} \quad (11)$$

## 4.3 Maintenance comparison

Geogrids are said to outlast the life of the road, (Meyer & Elias, 1999), implying that the Geogrids, if not intentionally damaged will always perform their functions for as long as the road exists. This reduces the need for frequent maintenance and rehabilitation activities. On the other hand, maintenance operations for the unreinforced road must be more frequent to avoid quick degeneration of the road. Therefore refilling and resurfacing will cost more on the unreinforced road.

Unreinforced road
Total initial cost was 6,967,118/=
Maintenance cost was said to be 50% of the initial cost after every three year. (UNRA, 2008)

$$= \frac{50}{100} * 6{,}967{,}118 = 3{,}483{,}559/= \quad (12)$$

The life cycle of the road was projected to 25years minus the year of commissioning or opening the road. So total maintenance cost will be:

$$= 3483559x\ (25/3) = 29{,}029{,}658/= \quad (13)$$

Total cost after 25 years

$$= 29{,}029{,}658+6{,}967{,}118 =35{,}996{,}776/= \quad (14)$$

Reinforced road
Total initial cost was equal to 1,171,956/=)
Maintenance cost was said to be 5% of the initial cost after every three years.

$$= 5/100x3483559 = 174178/= \quad (15)$$

The life cycle of the road was projected to 25years minus the year of commissioning or opening the road. So total maintenance cost will be:

$$= 174178x25/3 = 1{,}451{,}483/= \quad (16)$$

Total cost after the 25years for the reinforced pavement was

$$= 1{,}451{,}483+19{,}516{,}959 = 20{,}968{,}442/= \quad (17)$$

The total percentage reduction in cost of reinforced over unreinforced after the 25years.

$$= \frac{35{,}996{,}776-20{,}968{,}442}{35{,}996{,}776} * 100$$
$$\underline{= \mathbf{42\%}} \quad (18)$$

Therefore, the initial cost of constructing with Geogrids for reinforcement was greater by 180% and the reinforced design is more cost effective over the unreinforced by 42% over the 25years.

## 5 Conclusion

From the cost analysis comparisons, it was found out that reinforcing the road reduces the total cost over the life of the road. Reinforcing using Geogrid was cost effective by 42% over the unreinforced pavement. However, the initial cost of reinforcing using Geogrids was greater by 180%.

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# A Review of Sonic Echo Pile Integrity Testing on Cast In-Situ Piles Founded on Soil or Rock

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## Abstract

Cast in-situ piles are extensively used as deep foundation solutions for medium to highly loaded structures ranging from bridges to multi-storey buildings, hence their structural integrity is of great importance. Cast in-situ piles are constructed by pouring concrete into a preformed bore in the ground. As such, the condition of the concrete cannot be visually inspected and its quality needs to be verified through other means.

Sonic Echo Pile Integrity Testing, also known as Pulse-Echo or Low Strain Pile Integrity Testing offers such a solution. This technique is also highly economical, efficient and user-friendly. Pile integrity testing can be administered on various pile types of varying sizes founded on soil or rock. This paper will introduce the Pile Integrity Testing technique. Furthermore the benefits, limitations and review of case studies covering several pile integrity scenarios will be discussed.

***Keywords:*** *Sonic Echo- and Low strain integrity testing.*

## 1 Introduction

### 1.1 Background

Over the past millenniums structures have evolved from light structures such as dwellings and wooden bridges, to medium to highly loaded structures such as multi-storey buildings and complex structures. Cast in-situ piles have been used since the late 19th to early 20th century and are used extensively throughout the world to carry higher and more complex loads thus making their structural integrity of great importance.

Cast in-situ piles are constructed by pouring concrete into a preformed bore in the ground. The condition of the concrete cannot be visually inspected and needs to be verified through other means. Sonic Echo Pile Integrity Testing, also known as Impulse-Echo or Low Strain Impact Pile Integrity Testing offers such a solution.

Sonic Echo Pile Integrity Testing is a procedure where the integrity of a singular vertical or inclined pile is assessed by measuring and analysing the velocity response of the pile to an external impulsive force. The exerted force is low enough so that the pile-soil system behaves in a linear-elastic manner and the pile head returns to its original position. The results obtained from this test are used to establish the uniformity of the pile material, the physical pile dimensions (length and relative cross-section throughout the pile) and continuity of the pile.

According to van Koten and Middlendorp (1980) this echo-type integrity testing method was developed in Holland by the Dutch research organisation TNO Institute for building Materials and Structures (TNO-IBBC) in 1968 and early 1970s (CIRIA 144, 1997). Sonic Echo Integrity testing literature, however, first surfaced in 1968 with Jean Paquet's paper on non-destructive testing of piles in the publication French National Building and Engineering (Hertlein and Davis, 2006). A divergence between Paquet's frequency-domain method and Holland's time-domain appeared and Paquet concluded that the frequency domain method was best.

Although this test method is relatively new and still holds room for extensive research, it has fast shown its significance and subsequent benefit to the construction industry. It is highly economical, user-friendly and efficient. It also gives useful information when used by a person with good knowledge of the testing method, results and good engineering judgement. Today, Sonic Echo Pile Integrity is considered to be the one of the most used integrity testing techniques for quality assurance (ASTM, 2016).

This paper presents the analysis of 127 tested piles from 10 sites with 66 piles from 5 sites founded in soil. There are 41 piles in 4 sites with significant top portion in soil and the bottom end socketed into rock. There are also 20 piles in 1 site socketed into rock. It is to be noted that for the purpose of this paper, piles with a small portion (+/- 500mm) of their length going through overburden soil before entering into rock are considered to be completely socketed into rock as the influence of the portion through the soil is 'negligible'.

## 2 Method of Testing

Sonic Echo Pile Integrity Testing is carried out using the Pulse Echo Method (PEM) where an impact device strikes the pile head surface and generates a downward traveling stress-wave/ echo which propagates through the pile and partially or totally reflects where the pile and/ or surrounding material properties change significantly. These changes can be interpreted as either the pile toe or changes along the cross section and are denoted in terms of impedance (Z) change:

$$Z = \rho . c . A \tag{1}$$

Where:
$\rho$ = density of pile material
$c$ = velocity of stress-wave
$A$ = cross-sectional pile area

A sensor attached to the pile head surface receives these reflected stress-waves and measures their return time (NBS, 1984). The time domain record is depicted as a signal response curve (reflectogram), and is then evaluated to determine the pile's integrity. If the wave velocity in the pile is known, the round-trip travel time of each stress-wave can be used to determine the probable pile defect location or the pile-soil or pile-rock interface at the base of the pile thus indicating the pile length.

## 3 Required pre-test information

Before an integrity test is conducted on a pile the cast date, physical pile dimensions, inclination, pile type, surrounding material (i.e.: soil and-/or rock) and watertable level are required. This information is obtained from the pile layout plan, daily pile reports and the geotechnical investigation report.

## 4 Testing and Preparation Apparatus

The following pieces of equipment are required to conduct the field tests.

1. Chisel
2. Brush
3. Hand-held hammer
4. Sensor
5. Putty
6. Computerized hand-held device (i.e.: tablet)

## 5 Pre-Test Preparation

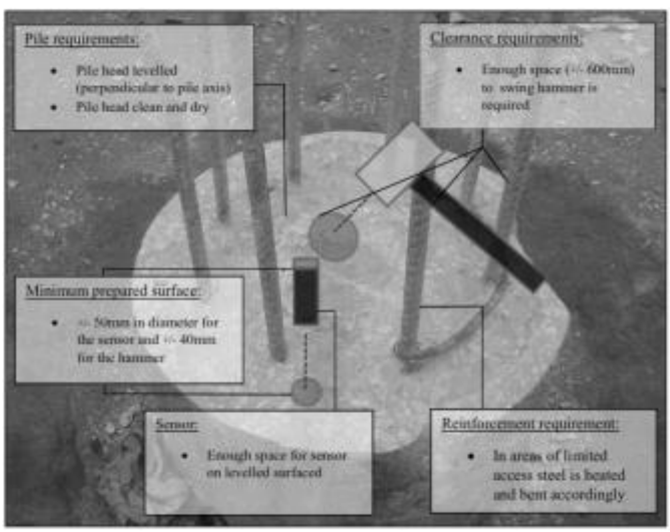



Figure 1. Well prepared pile head testing surface

Once the pile has reached 7 days after casting or 75% of its design strength, it is considered ready for testing. The pile should be trimmed down to the specified depth and the pile head adequately prepared by smoothing the entire surface or at least, the location(s) where the hammer will be striking and the sensor placed (Figure 1). The pile head test surface(s) should be accessible, and loose concrete, dust particles and standing water removed. If the pile is contaminated the pile should be trimmed down until sound concrete is reached.

## 6 Testing Procedure

### 6.1 Fieldwork

The schematic flow of work from the fieldwork to the final report production is indicated in Figure 2. Once the pile head surface is prepared (Figure 1) a thin layer of putty (or similar bonding material) is placed at the base of a sensor. The sensor which is used to measure the axial pile head motion is placed firmly and upright at the center of the pile head surface. Three locations should be considered for piles with diameters of more than 500mm.

A hand-held hammer (Figure 3) with a hard plastic tip is used to repeatedly strike the pile head perpendicularly at a distance no more than 300mm from the sensor. This impact force generates a stress-waves that propagates through the pile with the shaft acting as a travel guide. As the waves continue to travel they will be totally or partially reflected by impedances from material defects or interfaces between phases of different densities or elastic moduli. These reflected waves will be received by the sensor and the accelerometer in it will integrate them and where necessary, reduce the information to get velocity data.

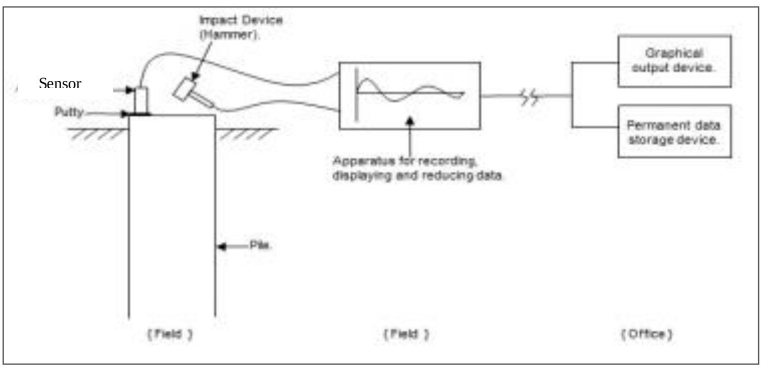



Figure 2. Schematic diagram of the flow of work from the beginning of the pile integrity test to the end stage where the data is analysed in the office (ASTM D5882, 2007).

Figure 3. Fieldwork component (Computerized hand-held device, sensor and hammer)

According to the ASTM D5882-07 the accelerometer used must be calibrated linearly to at least 50g and the device be either A/C or D/C. A D/C device should have a frequency response of up to 5000Hz with less than -3 dB reduction of content whereas a A/C device should have a time constant of 0.5s and a resonant frequency of at least 30 000 Hz. Alternatively, velocity or displacement transducers of equivalent performance may be used.

The signals from the sensor are then transferred to a handheld computerized device using Bluetooth technology, where they will be recorded, reduced and presented as a function of time. This data is viewed as a reflectogram and this is where the operator reviews it for the first time. This amplifies and filters the graph as per need to ensure the quality of the results. A minimum of three good results are required but it is advised to take considerably more readings to average out the noises and have enough data to make an informed decision.

If the results reflect potential problems (i.e.: decrease in shaft diameter, discontinuities, etc.) or inconclusive (uncharacteristic wave profile), the operator should ensure that the pile head surface is in its specified condition. If not request that the specifications be meet before continuing the test. However, if the specifications have been met the pile should be retest until the noises are removed. If potentially problematic or inconclusive results persist, a person with extensive experience and necessary engineering judgment should look at the result and instruct what immediate action should to be taken. Trimming down the pile to sound concrete might be necessary. Alterations made, troubles encountered and additional relevant observations made before or while testing, should be mentioned in the comments section of the software of the handheld computer device. This computerized device should have a storage facility so that the test data can be retrieved and reviewed at a later stage.

### 6.2 Data Analysis

The Bluetooth Pile Integrity Tester (Figure 3) is calibrated during the manufacturing process in accordance with ASTM D5882 and generally does not require recalibration during its lifespan after which it is to be replaced. The stored data on the device is then transferred to a computer where the data is further reviewed and analyses by an individual with significant experience (i.e.: suggested to be at least one year) with this testing method and good engineering judgement. The competence of the tester and more so, the analyser is important to the success of this testing method.

The velocity of the sonic wave through a concrete pile is considered to be 4000m/s, based on the assumption that the stress-wave velocity through concrete ranges from 3500m/sec. to 4500m/sec (average 4000m/sec.). However, it is possible for the stress-wave velocity to be greater than 4500m/sec or smaller than 3500m/sec.; depending on the concrete quality and Young’s Modulus. In addition, the filter and amplification was adjusted accordingly to suit the stress-wave propagation characteristics, the effect of the surrounding material and the change in stress-wave properties as it moves through the pile.

## 7 Types of Responses and Typical Results

There are generally three types of responses obtained from the Sonic Echo integrity time-based test, namely the free end situation response (Figure 4 and 5), the fixed-end situation response (Figure 6 and 7), and the combined end situation response (Figure 8 and 9).

In a free-end pile situation no force can be transmitted across the boundary, particle velocity at a free end is twice that of the initial stress-wave while the resultant force is zero, whereas at a fixed-end the resultant force is twice that of initial force and the resultant velocity is zero (CIRIA144, 1997).

The interface between the pile toe and the underlying soil usually entails reduction in impedance as the pile is usually stiffer than the soil. Hence, the toe reflection is a free-end response, as is a reduction in pile diameter and a pile discontinuity (i.e.: crack) (Figure 4 and 5). However, a relative increase in pile impedance results from increased pile cross section, material properties or both; would result in a fixed-end result (Figure 6 and 7). A reflection of a free-end would appear on the same side of the reflectogram’s horizontal (time) axis as the initial hammer impulse whereas the fixed-end reflection would appear on the opposite side (CIRIA 144,1997).

In some instances, multiple partial reflections due to a change in more than one factor (i.e.: change in soil layer, change in pile cross sectional area, etc.) affecting the impedance occur within a pile resulting in combined responses as indicated in Figure 8 and 9.These responses are generally more complex with the pile toe being mostly unclear due to the increased degree of partial reflections occurring along the pile shaft.

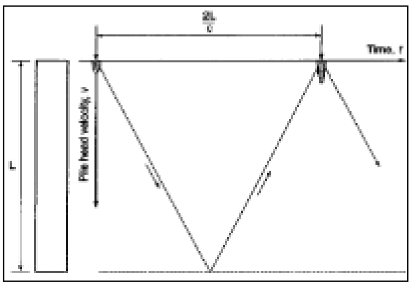



Figure 4. Free-end response at pile toe (CIRIA 144,1997)

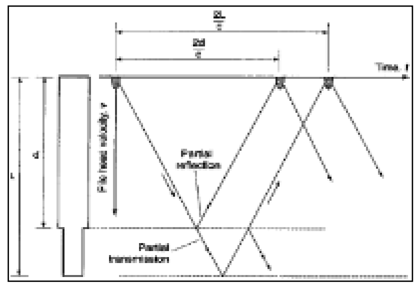



Figure 5. Free-end response intermediate decrease in cross section (CIRIA 144,1997)

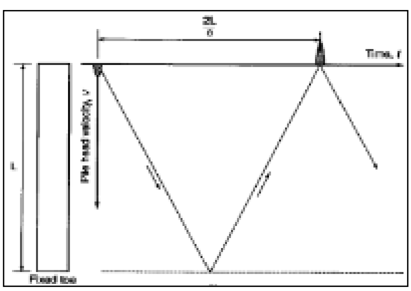



Figure 6. Fix-end response at pile toe (CIRIA 144,1997)

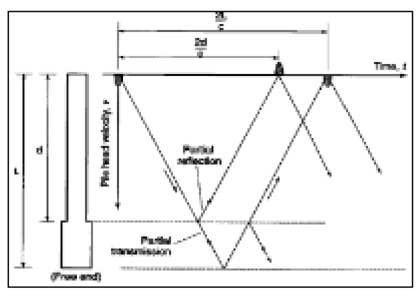



Figure 7. Simplified fix-end response at intermediate increase in cross section (CIRIA 144,1997)

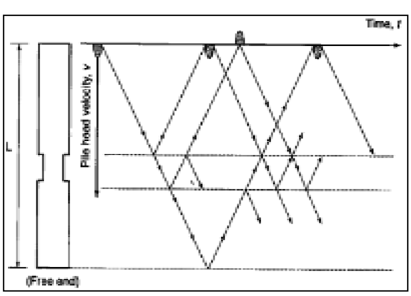



Figure 8. Simplified combined response with free-end and a neck. Partial reflection at both changes with a reduced toe reflection (CIRIA 144, 1997)

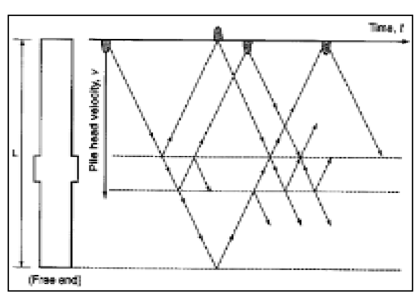
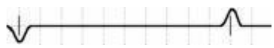
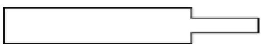
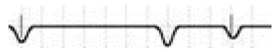



Figure 9. Simplified free end with bulb. Partial reflection at both changes with a at reduced toe reflection (CIRIA 144, 1997)

The table below shows the correlation of the different types of reflectograms, resulting from the responses mentioned above, to the different pile geometries and founding material.

Table 1. Typical reflectograms and their meanings (PileTest; Unknown)

| **Pile Profile** | **Description** | **Reflectogram** |
|---|---|---|
|  | Straight continuous pile with no noticeable defects. Free-end condition |  |
|  | Straight continuous pile with a fixed end and no noticeable defects. Fixed end condition |  |
|  | Increased impedance. Fixed end condition. |  |
|  | Decreased impedance. Free end condition. |  |
|  | Locally increased impedance. Fixed end then free end condition. |  |
|  | Locally decreased impedance. Free end then fixed end condition. |  |
|  | Irregular shaft profile or inconclusive result |  |

## 8 Results and Analysis

The speed of stress-wave propagation in an elastic solid is related to the modulus of elasticity, Poisson's ratio, density and geometry of the solid. The relationship between the propagation of the stress-wave and pile properties allows inferences about the pile characteristics (i.e.: relative dimensions, etc.) to be made (NBS, 1984). As stress-waves travel through the pile, damping effects occur as signals are reduced by the pile- and surrounding material (soils and-/ or rock). However, the effects the surrounding material has on the pile integrity test results has not been precisely quantified due to several reasons, including the difficultly incurred at times when distinguishing between soil and rock.

Nonetheless, a few general trends and conclusions could be made from the different piles in the different materials. Piles socketed and founded in rock were adequately cleaned whenever practically feasible and where water would be an issue the piles were cased to minimise probable contamination. However, the influence of cross sectional variances along the pile shaft and ingress water from the watertable were taken into account where applicable during the analysis of the test results.

## 8.1 Piles founded in soil

66 piles founded in soil were tested. 45% of these piles were CFAs with a diameter of approximately 650mm, lengths ranging from 16 to 24m and L/D ratios of between 25 and 37, whilst 55% were DCIS piles with diameters ranging from 355- 610mm, lengths ranging from 5 to 11m and L/D ratios of between 14 and 18.

All the tested piles yielded conclusive results with only a few CFA piles from the L/D ratio between 25 and 37 range requiring retesting before a conclusive result was obtained. The reflectograms below, with the x-axis representing the depth/length of the pile and y-axis the amplitude of the graph, show the general display of the stress-waves propagated through the shorter DCIS piles (Figure 10) and longer CFA piles (Figure 11).

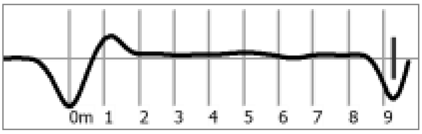



Figure 10. Typical Reflectogram from the tested shorter DCIS piles

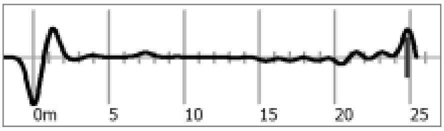



Figure 11. Typical Reflectogram from the longer CFA piles

Piles founded in soil are generally easier to analyse because the propagated wave only interacts with one surrounding material (i.e.: soil) of a significantly different velocity property to the pile (Figure 10). The longer the pile shaft, the larger the signal damping effects across the pile shaft hence the less pronounced the pile toe (Figure 11).

## 8.2 Piles embedded in soil with a socket in rock

41 piles with a significant top portion embedded in soil and the bottom end socketed into rock of variable consistency were tested. 67% were Auger piles with diameters of approximately 600mm to 1080mm and lengths ranging from 7 to 18m; and L/D ratios ranging between 6 and 17. 33% of these piles were CFA piles with a diameter of approximately 450mm and lengths ranging from 7 to 9m and L/D ratios ranging from 16 – 18. For CFA piles heavy machinery was used to get the required pile length in zones of harder rock.

A significant amount of piles yielded conclusive results with some requiring retesting before they could be deemed conclusive. There were signal losses due to the damping effects of the surrounding soil and rock material, and the concrete material along the pile shaft. The reflectogram and the Fast Fourier Transform (FFT) curve were interchangeably used to evaluate the integrity of the pile. The reflectogram in these piles was generally used more than FFT curve. A FFT algorithm determines the Discrete Fourier transform (DFT) of a sequence, or the converse thereof. This impact response method transfers the measured time history data into the frequency domain and peaks on the FFT curve displays significant changes and features in the pile and surrounding material occur (Figure 13).

The 4 reflectograms below show the general display of the waves propagated through the all the piles in the different sites.

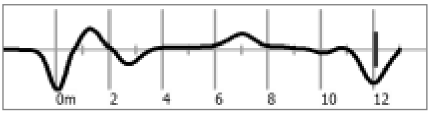



Figure 12. Typical Reflectogram of piles socketed through rock ( 7m for this pile) with the pile toe clearly shown

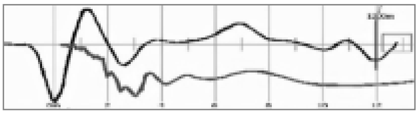


Figure 13. The reflectogram as shown in Figure 12 with a lightly shaded FFT curve underneath

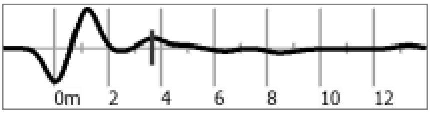



Figure 14: Typical Reflectogram of piles socketed through rock (at approximately 3.5m for this pile) with the pile toe not clearly shown

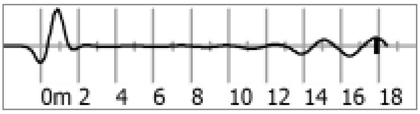



Figure 15: Typical Reflectogram of piles socketed through rock (at approximately 14m for this pile) with the pile toe not clearly shown. The shaft condition between the socket and presumed pile toe unclear.

Piles embedded mostly in soil and socketed into rock generally have one of three behaviours. The soil-rock interface is either shown as an anomaly and the toe of the pile reflected clearly on the reflectogram (Figure 12). The other scenarios would be that the interface of the soil and rock is either reflected as a toe on the reflectogram and the actual toe either shown as a secondary or insignificant reflection (Figure 14), or seem completely unclear (Figure 15). It is however important to note that in instances where the consistency of the soil and soft rock are closely related, it might not possible to distinguish that interface from the reflectogram.

### 8.3 Piles socketed in rock.

20 piles socketed into rock were tested. These piles were 600mm Augers with lengths ranging from 3 to 5m, and length to diameter (L/D) ratio of these piles is approximately 5 and 8 respectively.

All the tested piles yielded conclusive results with no retesting required. There were signal losses due to the damping effects of the concrete material and surrounding rock along the pile shaft. In this instance the reflectogram and the Fast Fourier Transform curve were both used to evaluate the integrity of the pile.

The reflectograms below shows the general display of the waves propagated through the all the piles in rock.

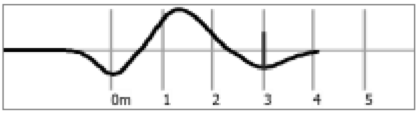



Figure 16. Typical Reflectogram of piles socketed into rock with the pile toe shown

The piles socketed into rock are significantly harder to analyse. They require an equal amount of use of the reflectogram and FFT curve. This method has only been carried out on short piles and it is unknown exactly where the limit of usage will occur for longer piles in rock.

The reflectorgram will not look identical to the longer piles founded in the soil but with the right level of engineering judgement the relevant data can be derived from the interpretation of the projected reflectogram and the crest and troughs of the FFT curve .

## 9 Benefits and Limitations of Integrity Testing

This testing method is not only user-friend and efficient; it is also cost effective and gives immediate results, without altering the structural integrity of the pile. In addition, the following can be detected by this testing method (PileTest, unknown):

- Pile length,
- Inconsistencies in the concrete material,
- Horizontal cracking and planned jointing,
- Abrupt changes in cross sectional area,
- Distinct changes in surrounding material layers (i.e.: soil and rock)

However, Sonic Echo Integrity Testing like most physical tests, have their operational limitations, at which point they yield results that are inconclusive (not meaningful). This test will generally not detect the following (PileTest, unknown):

- Toe reflection when the L/d ratio roughly exceeds 20 (in hard soils) to 60 (in very soft soils),
- Gradual changes in cross sectional area,
- Small changes (approximately 25%) in shaft diameter,
- Length variations of less than approximately 10%,
- Features below a significant discontinuity or shaft diameter change,
- Residue at the toe,
- Deviations from vertical alignment,
- Bearing capacity

## 10 Conclusion and Recommendation

Sonic Echo Integrity testing is used to determine the uniformity of the pile material, the physical dimension (relative cross-section throughout the pile and pile length) and continuity of the pile shaft. It has shown itself to be efficient, user-friendly, fast and inexpensive, and is in turn probably one of the most used methods of integrity testing today. The results of the test are however highly dependent on the pile preparation, the manner in which the test was carried out and the engineering judgement of person(s) conducting and analysing the test.

Analysis of piles in soil is generally relatively easy but the dynamics can be altered if significant ingress of water and sidewall collapse is encountered. The analysis of piles embedded in soil and socketed in rock is slightly more difficult to interpret due to the coupled effects of the pile-soil, soil- rock and pile-rock interaction; and in some instances the FFT curve is relied upon to give greater insight to the pile integrity. The analysis of piles socketed into rock is considerably more difficult, requiring relatively equal degrees of the evaluation of the FFT curve and the reflectogram. In all instances, testing of the pile integrity becomes more difficult when significant over-break and ingress of water leading to localized pile contamination is encountered. However, with the input of a competent person useful data can be derived from these tests and the correct remedial process undertaken.

In addition to detecting pile geometry and pile material inconsistencies, this testing method can identify horizontal cracking, abrupt changes in cross sectional area and distinct changes of surrounding material layers. Conversely, it cannot detect toe reflection when the L/d ratio roughly exceeds 20 (in hard soils) to 60 (in very soft soils), small pile diameter changes, gradual changes in cross sectional area, small length variations, features below a significant discontinuity and bearing capacity

More research and insight into the Sonic Echo Integrity Testing technique would result in more benefits being realised and allow for further, allowing for the advancement of the testing method.

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# Temporary Structures: Sheet Pile Applications

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## Abstract

The use of sheet piles as temporary structures is not only common for offshore construction projects but also used more and more for onshore projects to act as a temporary retaining wall against ground and water to enable the construction of an underground structure. Sheet piles are relative easy to install and to remove and can be used again in different projects. Common applications for such structures: Underground Car Parks (Basement) Construction, Building Pits (Trench excavations & Tunnels), Cofferdams (offshore construction) and Deep Foundations.

Firstly, the temporary function of these structures is to retain water and soil while providing a dry working site. Secondly, a permanent function as engineers incorporate the sheet pile design onto the structural design. If the total costs of the projects are considered using sheet piles permanently, instead of removing and replacing them with new building material, this can be a more cost effective solution.

This paper reviews basic design and construction considerations for the use of sheet piles as temporary support structures for basement construction (underground car parking) and other temporary applications.

***Keywords:*** *Temporary structures, onshore / offshore, cost effective, sheetpiles, temporary & permanent function, Basement Parking / Underground Car Park and retaining wall.*

## 1 Introduction

Use of sheet piles for underground car parks or basement parking for high rise buildings are a solution to limited space in highly urbanised areas, same can be said about open trench excavations and underground tunnels.

However, underground construction has numerous design and construction issues that normal structures above ground level do not have. That is why building underground is so costly (with conventional methods) and only considered when all other options have been exhausted.

The innovative use of sheetpiles for such construction projects eliminates the need to construct a permanent wall within a shored excavation, thereby greatly reducing the construction schedule.

The reduced schedule equates to significant project savings.

*Case Study: New Amsterdam Court House NACH*: Construction of a basement parking garage and multifunctional space under new Courthouse.

- 2 sub surfaces levels
- Demolition of the old basement (for extension of the new be easement).

Sheet Pile Construction Pits as temporary works (later become permanent basement walls). During design phase assessment, two alternative solutions were discussed:

- Construction with one row of struts and submerged concrete.
- Construction with two rows of struts and drainage.

In general the Netherlands has a very high water table and very soft ground which makes it a perfect place for installing and removing of sheet piles, this also makes it a difficult place for underground construction with conventional methods, hence the use of sheet piles (which are water proof) and a submerged concrete slab at the bottom level of the basement parking to ensure a dry working area.

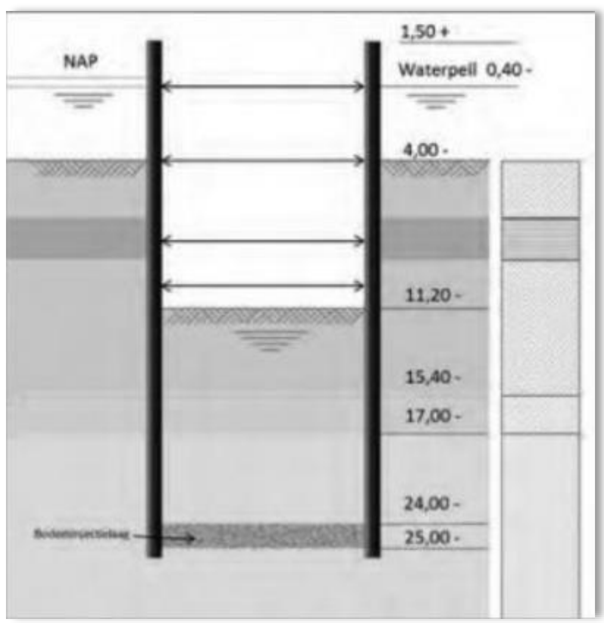


Figure 1. Cross-Section of a concept design model – 4 rows of struts and submerged concrete slab

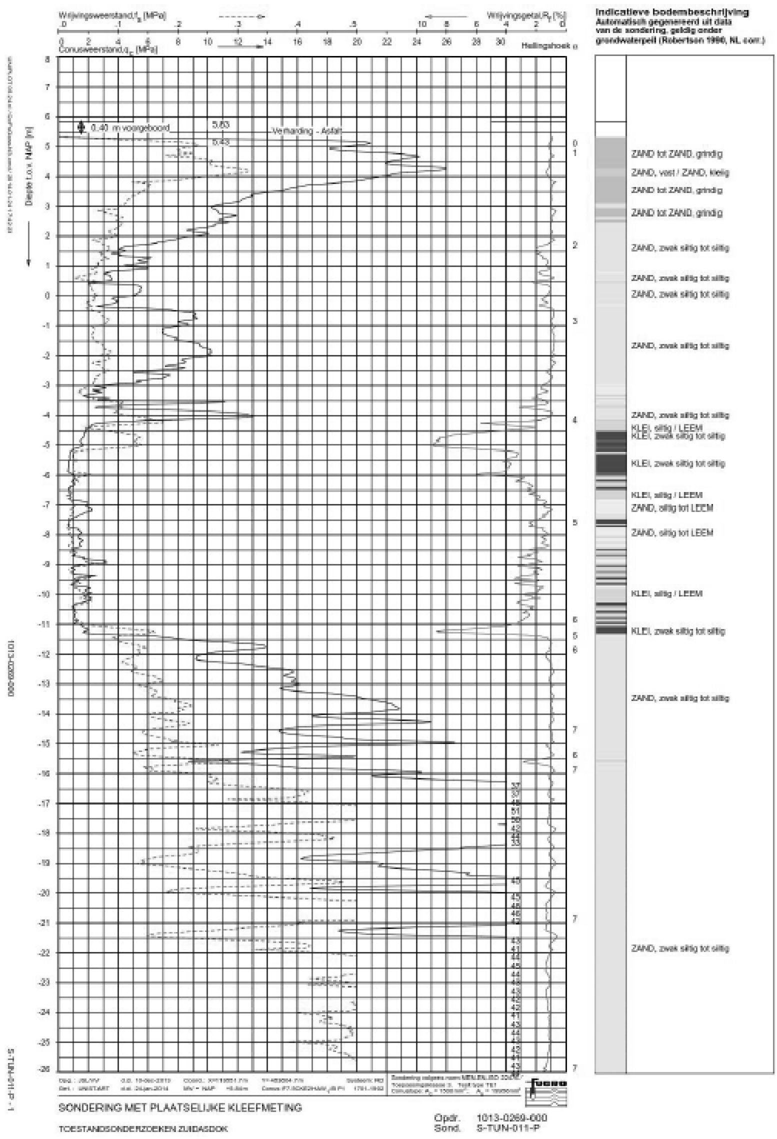


Figure 2. Sample of Geotechnical Report

## 2 New Amsterdam Court House, Nach – Basement Parking construction

### 2.1 Design considerations – Basement Parking

Amsterdam is a highly urbanised area with limited construction space, with existing buildings and roads situated along the edges of the construction site. The foundations of the adjacent buildings and roads were to be protected from collapsing due to excavation works and at the same time providing a dry working site. Since this was an inner city project, noise levels during construction had to be controlled.

Figure 3. Plan View – Construction Site close proximity to neighbouring buildings

Important design factors to consider for construction:
- Strength of the sheet pile (ULS)
- Stability of the sheet pile (ULS)
- Deflection of the sheet pile (SLS).
- Concrete & sheet pile interface (inside wall of the basement)

Dependent on design philosophy and environment (site geometry and constrains, geotechnical parameters, working space requirements) we can either use ground anchors or struts (see Figure 5) to reduce bending moments, control deflection and insuring stability of the basement walls.

While both are good options, below are few factors to be considered:

- *Ground anchors* – give you freedom of movement on the construction site but space might be limited and building permits needed for adjacent buildings.
- *Strutting or Struts* – Strutting is possible within your own site boundaries, no building permits required and these struts can be temporary and replaced after casting basement floors (these basement floors then take over function of permanent struts).

Strutting system is more common option for basement construction projects and it was used on this project. Adjacent buildings meant limited space for ground anchors.

The use of submerged concrete slab has a dual purpose: (see Figure 1.)

- Help in keeping the working site dry.
- Reducing the lengths of sheet pile which equates to project savings.

They are advantages and disadvantages of submerged concrete solution:

Advantages:
- Submerged concrete reduces sheet pile length.
- One layer of strut instead of 2 or more and no drainage required.

Disadvantages:
- Dredging takes longer than dry excavation.
- Pouring submerged concrete and the logistics that come with it.

Table 1. Proposed Sheet Pile Lengths – With application of submerged concrete, sheet pile lengths were reduced to 16m and 18m, around deep excavation.

| | | **Damwand** | | | | |
|---|---|---|---|---|---|---|
| **Positie** | **Omschrijving** | **Staalkwaliteit** | **Lengte** | **Aantal** | **Punt** | **Top** |
| N1 | damwand AZ36-700N | S355 | 14500 | 60 | -13500 | +1000 |
| N2 | damwand AZ48-700 | S355 | 19000 | 42 | -18000 | +1000 |
| N4 | damwand AZ42-700N | S355 | 20000 | 26 | -19000 | +1000 |
| N4 | damwand AZ48-700 | S355 | 20000 | 90 | -19000 | +1000 |
| O1 | damwand AZ48-700 | S355 | 21500 | 131 | -20500 | +1000 |
| W1 | damwand AZ48-700 | S355 | 19000 | 130 | -18000 | +1000 |
| Z1 | damwand AZ36-700N | S355 | 15500 | 168 | -13500 | +2000 |
| Z2 | damwand AZ48-700 | S355 | 20000 | 103 | -19000 | +1000 |
| Z3 | damwand AZ48-700 | S355 | 20000 | 55 | -19000 | +1000 |

*Strutting system definition* – Steel or concrete profiles that can absorb axial compression forces so that the main structure is supported in one or more directions and these members are connecting to waling beams (Steel or concrete profiles which distribute the active soil pressures from the main wall to the occurring anchors and/or struts) which are connected to the main wall.

Figure 4. Plan View – Strutting system – During construction

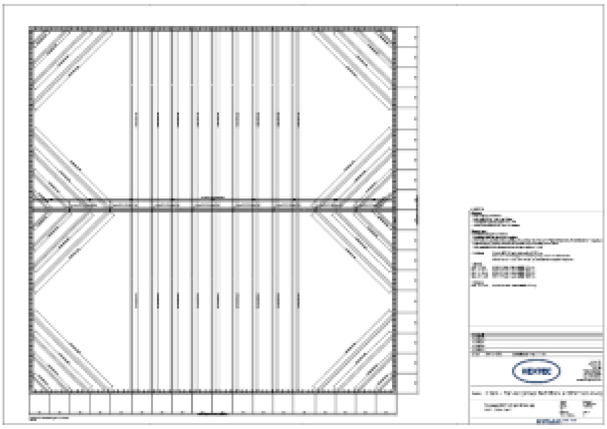

Figure 5. Plan View – Strutting System – Concept design

## 2.2 Construction Phases – Basement Parking

Construction method /sequence used for this project is what is commonly referred to as "Top-Down" construction method. Excavation was done in stages from the top-down and not the entire depth (dredging level) excavated in one process.

*Phase 1: Installation of sheet piles*

Figure 6. Phase 1 – Installation of Sheet Piles

Figure 7.  Phase 1 – Installation of Sheet Piles

Before any excavation works within the site boundaries can begin, the first phase was to install the sheet piles to the required depths all around the site boundary (Figure 7 & 8).

Due to strict restrictions on inner city construction projects – noise levels and vibrations had to be controlled. Vibrations could destabilised the neighbouring buildings’ foundation and the noise from the installation of sheet piles (Vibratory hammer & pressing machine) had to be kept to a minimum.

So, selection of the equipment used was important to stay within regulations. The vibratory hammer was used for the first few meters to get the sheet piles into position and from then on a silent piler was used for the rest of the required depths.

With the silent piler, there are no ground vibrations, this process might be slower than the vibratory hammer but with prior knowledge of the ground (see Figure 2) proper planning was in place to account for the time needed for the silent piler to get the sheet piles into position.

*Phase 2: First row of strutting system installed*

Figure 8.  First row of Struts – Installation

To avoid over-stressing the sheet pile walls (Basement walls), excavation was done in stages. Only excavate a few meters below ground at a time to provide enough working space for the installation of the waler beams and struts – first level (see Figure 7).

When installation of the first row of struts row was complete, second stage of excavation then followed.

This process was repeated until basement floor level was reached and the submerged concrete slab constructed. When all the struts levels were constructed including the submerged concrete slab. The construction site was then dried-off and ready for the casting of basement floors.

The strutting levels were all temporary structures, once the basement floors have been cast, these struts are then removed and used on another construction site.

The sheetpiles walls (Basement walls) which began as temporary structures are now permanent basement walls, and basement floors (concrete slabs) are now permanent struts (see below Figure 8)

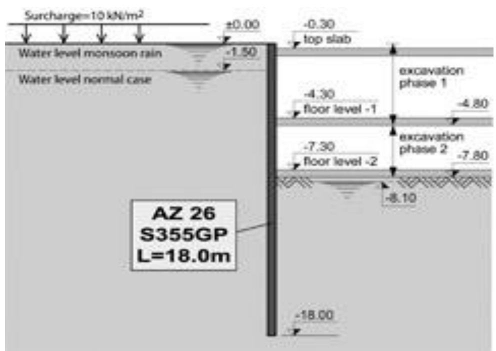


Figure 9. Permanent sheet pile wall and concrete floors as permanent struts

Figure 10. Basement Parking - Sheetpiles

## 2.3 Other design considerations – Basement Parking

*Corrosion:*

Corrosion in non-marine structures is often not a problem but this is dependent on the surrounding environment. Corrosion for permanent basement walls (Sheet pile walls) can be prevented or treated:

- Coating (inside basement)
- Concrete façade (inside basement)
- Sacrificial thickness (outside)

Table 2. Corrosion rates used in the design based on British standards (BS 8002, 1994)
(Note: corrosion rates for peat, contaminated or disturbed soils can be much higher)

| Environment | Corrosion rate mm/side per year |
|---|---|
| Embedded in undisturbed soil | 0.015 (maximum) |
| Exposed to atmosphere | 0.035 (average) |
| Immersed in fresh water | See note 1 |
| Exposed to marine environment | |
| - below bed level | 0.015 (maximum) |
| - seawater immersion zone | 0.035 (average) |
| - tidal zone | 0.035 (average) |
| - low water zone | 0.075 (average) |
| - splash zone | 0.075 (average) |

*Note: 1. Fresh waters are variable. Corrosion losses in fresh water immersion zone are generally lower than for seawater.*

*Water seepage, interlock sealant or welding of interlocks:*
Basement parking's are required to be water tight, during construction and service life. The sheet pile interlocks could either be welded or interlock sealant used.

Figure 11. Welded interlocks inside basement to ensure water tightness.

Table 3. Available Interlock sealant – for water tightness

| Sealing system / method | ρ {10 $^{-10}$ m/s} 100kPa | 200kPa | 300kPa | Application of the system |
|---|---|---|---|---|
| No sealant | > 1000 | - | - | - |
| Beltan Plus | < 600 | * | - | easy |
| Arcoseal | < 600 | * | - | easy |
| Roxan Plus | 0.5 | 0.5 | - | with care |
| AKILA | 0.3 | 0.3 | 0.5 | with care |
| Welded Interlocks | 0.3 | 0 | 0 | |

## 3 Other Temporary Applications – Sheet piles

### 3.1 Building Pits

Building pits are temporary waterproof constructions in which an excavation is executed for the realization of an underground building structure. When enough space is available a pit can be dug under natural slope when enough space is available to do so.
However, usually this extra space is not available as roads, railways, or other buildings will be situated along the edges of the excavation. In that case sheet piles can be applied as an earth-retaining structure. The function of this construction is not only to retain ground, but also has a waterproof function.

Figure 12. Tunnel construction in Neherkade, Den Haag, Netherlands

Sheetpiles walls were a temporary support structure, which were later removed after construction was completed.

### 3.2 Cofferdams

Cofferdams are temporary enclosure structures built within, or in pairs across, a body of water and soil. By pumping the water out the enclosed area a dry work environment is created so that the actual building activities can proceed. Enclosed coffers are commonly used for construction and repair of oil platforms, bridge piers and other support structures built within or over water. These cofferdams are usually welded steel structures, with components consisting of sheet piles, walings, and cross braces. Such structures are typically dismantled after the work is completed.

Figure 13. Cellular Cofferdam Breakwater project, Tuban. Indonesia (2005 – 2006)

### 3.3 Trenches

Trenches are excavations in the ground that are generally deeper than its width. In the civil engineering field of construction or maintenance, trenches are created to install or search for underground infrastructure or utilities (such as gas and water pipe lines or telephone lines, etc). The construction of a trench is usually done in an environment with limited space and sheet piles are an ideal solution to dig a trench under a steep slope with limited occurring displacement in the environment while providing a dry construction space at the bottom of the trench.

Figure 14. Trench protection: With heavy construction plants operating next to the trench, the excavation had to be protected from collapse. Once the underground pipe was laid the sheet piles are then removed and used on another construction site.

## 4 Conclusion

Temporary structures are used to provide trench / slope protection in soft or unstable soils with high water table and provide a dry working area. Sheetpiles sections available for such projects vary in size, shape (geometry) and in section modulus (stiffness). All temporary sheet pile structures can be re-used in other projects if still in good working conditions (number of times the sheet pile can be re-used is limited).

Installation is fairly a straight forward process, with pre-installation techniques available for hard driving conditions.

Factors to consider when selecting a sheet pile profile:

- Ground Profile & water table (Geotechnical Report).
- Retained Height (Influence on your deflection, bending moments & required embedded depth for stability).
- Construction Ability of the contractor.
- Site constraints.
- Forces to resists.

Big advantage of sheet pile temporary structures – fairly easy to install & remove, less man-power/labour force required. Once no longer needed on site they can be removed and re-used again on different projects.

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# The Geotechnical Design Verification Process Followed for the Qatar Redline South Underground Metro Rail Project

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## Abstract

This paper outlines the design verification process as for the Doha Metro Red Line Project. Aurecon is currently appointed for the design verification of the Doha Redline project in Doha, Qatar. Aurecons' role was to certify design compliance in terms of safety, functionality and employers requirements. The report documents verified for the project included design basis, factual, groundwater ingress, temporary stability, geophysical, interpretative geotechnical, instrumentation and monitoring, pre-construction condition survey, construction effects on existing structures and general engineering notes. All documents were managed using a web based document control system utilised by the lead contractor, designers and design verification engineers. A design verification record (DVR) was used for each document wherein the verifiers produced their comments and designers responded. Once the DVR was closed out with all comments addressed the document would be processed for external review. The aim of DD1 stage was to achieve design freeze of the rail and basic layouts of the tunnels, stations, switchboxes and shafts. Following closure of the DVR and an external review, the DD2 stage commenced. For the geotechnical design, the change from DD1 to DD2 was seamless and document workflow did not always follow the expected technical trail of deliverables. This posed numerous challenges and a prolonged verification process.

***Keywords****: Doha, Verification, Tunnel*

## 1 Introduction

Aurecon is currently appointed as Design Verification Engineer for the Doha Redline project which involves the design and construction of 15 km of twin bored tunnel, five underground cut and cover stations, four switchboxes, three emergency escape shafts and numerous cross passages between the tunnels (Figure 1). The Doha Metro Redline South (RLS) Underground Rail is expected to be a dual-tube underground line between Doha and Msheireb in Qatar. The geology of the region is underlain by near surface Aeolian sands and silts (Holocene and Pleistocene) which is underlain mainly by uniform horizontal to sub-horizontal dipping limestone beds. The area comprises upper Dammam which is the Simisima Limestone

Formation. This is underlain by the Lower Dammam comprising Dukhan Formation (large foraminifera bearing limestone) and Midra Shale Formation (Gypsiferous shale). These lithology's are underlain by the Rus Formation which are the oldest exposed rocks of the Lower Eocene and consist mainly of dolomite and limestone. Karstic features are common in the strata of Dammam and Rus Formation (Cavelier, 1970). The Qatar Railways Company engaged a design and build contractor, RLS-JV to undertake the Red Line South Underground section. The role of Aurecon for this project was to verify RLS-JV designs, independently. Although various disciplines within Aurecon were involved in the project, this paper discusses the verification process for the geotechnical works.

Figure 1. Schematic of the Redline Tunnel South route

## 2 Verification Scope

The design verification services was essentially defined by various volumes of the employer's requirements as well as Qatar Rails system assurance plan. The role of the design verification engineer (DVE, herein referred to as the "DVE") was to verify the design documents for work under the contract and to ensure that it met the employers requirements for safety, functionality and RAM (reliability, availability, maintainability), as well as Qatari regulations (including Qatar National Construction Standards, QCS 2010), applicable local and international codes and standards (including British and Eurocode), system assurance documents and other applicable codes and standards. This included comprehensive independent analytical work and calculations. There are three design stages, ie. Design Basis, Detailed Design Stage 1 (DD1) and Detailed Design Stage 2 (DD2), all of which needed to be certified by the DVE prior to submission to Qatar Rail for acceptance. The general objectives of checking at each design stage was to evaluate the design documents in order to determine that they comply with the outputs of the previous stage. The DVE was also required to identify issues with the output and suggest necessary actions.

## 2.1 Design Stage 0

Design Stage 0 comprised the design basis which included:

a) Accounting for all applicable Employers Requirements and other requirements including, but not limited to, selection of appropriate design codes and standards;
b) Interpretation of each of these requirements and application to the design;
c) Identification of the agreed acceptance criteria;
d) Checking the suitability of design and standards and or codes of practice adopted in the preparation of the geotechnical design (Design Basis Report); and
e) Verifying the adequacy of the proposed site investigations, namely, type, extent (quantity, layout and depth) and laboratory test results relating to the design of the works (Geotechnical Appraisal Report)

This was a fundamental process as it allowed alignment of the verification. This included viewing and interpretation of the ground profile, assessing and deriving the characteristic input parameters.

## 2.2 Detailed Design Stage 1

Detailed Design Stage 1 was conducted to allow "design freeze" in which the basic layouts were defined. The main aim within this stage was to verify the input parameters to ensure that the design complied with the outputs of the previous stage (Stage 0) as well as verifying if the designers recommended geotechnical design parameters were suitable and sufficient for the anticipated ground conditions and were consistent with local and international best practice. Various design software was also used including Plaxis (2d and 3d), GeoStudio package (Slope /w and Seep/w) and Wallap. The documents that were verified at this stage was the:

a) Geotechnical Interpretative Report (GIR) – indicating geotechnical findings, interpretation (summarized ground profile and characteristic parameters) and considerations. The verification included checking all assumptions of the design parameters as well as the investigation results and the geotechnical parameters for the design of the works including consideration of onerous water conditions, seepage pressures and surcharge, earth, construction and accidental loadings;
b) Geotechnical Factual Report (GFR) *(*only against the employer's requirements*).*
c) Geophysical Reports *(*only against employers requirements*);*
d) Construction Effects Report – Assessment of ground movement due to the construction activities (including ground loss due to tunneling, ground loss due to groundwater drawdown and vibrations induced by construction and impacts) and leading into the instrumentation and monitoring requirements. The verification included the method or model adopted for the analysis and design including the consideration of drained, undrained and consolidation analyses, and other appropriate drainage conditions and evaluating the risk within the zone of influence (including measures to limit ground movements). The suitability of the structure types and schemes, and the method and sequence of construction to be applied was also reviewed;
e) Groundwater Ingress Reports – Assessment of the groundwater inflow into the proposed temporary excavations in order to determine the dewatering system design. The DVE review included allowable limits of ground deformation and changes in groundwater and peizometric levels, and measures to control groundwater where required;
f) Temporary Stability – Stability assessment of the temporary open excavations. This included checking the stability of the excavation works, taking into consideration groundwater, drainage, and seepage conditions, basal heave, toe stability, hydraulic uplift and piping, assessing drawdown and any ground stabilization or improvement works as appropriate (This included evaluating the need to flatten the slope or install lateral support - retaining walls or anchors); and

g) Instrumentation and Monitoring – Proposed instrumentation plan including the monitoring programme. The review comprised the instrumentation and monitoring plan including the consideration of location, type and number of instruments, and frequency of monitoring and reporting.

Following the verification of the above, the documents were then certified by the DVE.

## 2.3 Detailed design Stage 2

The Detailed Design Stage 2 allowed further development of the design for construction. The main aim of this stage was to incorporate all the DVE's comments so as to close out comments, again check that the input parameters complied with the outputs, verifying that the design was fully coordinated at a detailed design level between design disciplines, interfacing contracts and stakeholders.

The documents verified included all reports from Detailed Design Stage 1 as well as the:

a) Pre-Construction Condition Survey Report– A summary of the survey and risk assessment of the existing structures within the influence zone prior to construction*;*
b) Post Construction Condition Survey Report - A summary of the survey and risk assessment of the existing structures within the influence zone after construction and a comparison to the pre-construction condition survey*;* and
c) General Engineering Notes

Following the verification of the above, the documents were then certified by the DVE.

# 3 Verification Process

## 3.1 Document Management

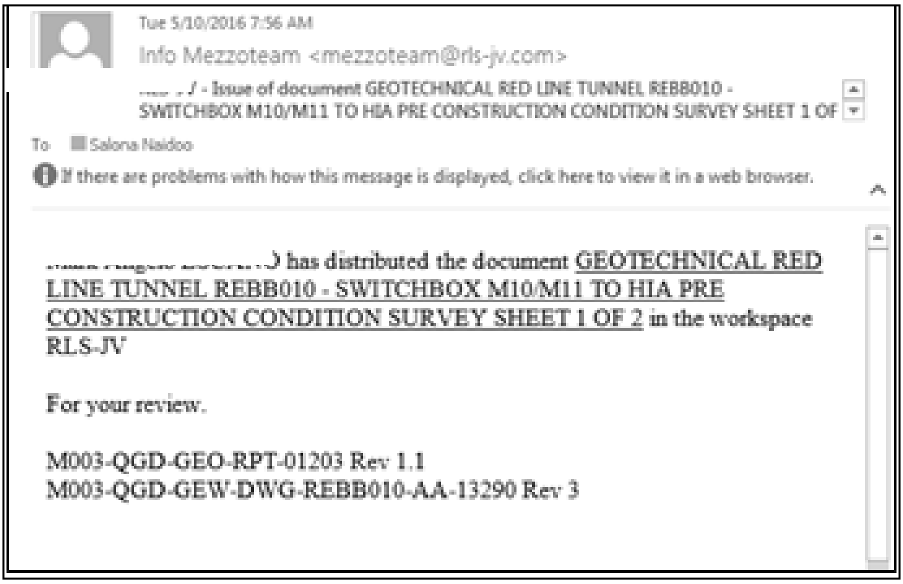
The design reports were segmented into various rail sections along the route as well as for each station, switchbox, and emergency exit shafts. This meant that for each of these sections or structures, there were typically between seven to ten reports with three to five revisions on average. In order to track and manage the flow of documents, the project as a whole (designers, contractors, Employer, and DVE) utilized Mezzoteam. This is a document management tool that allowed the authorized user to create, save, validate, share and update documents in a secured collaborative environment. Authorized users connected to the software via the internet through a login and password. All documents were loaded and downloaded from this web based document management system. A notification was generally received via email informing the user that certain documents have been uploaded in Mezzoteam for review with an associated link (Figure 2). A duplicate folder system with all the necessary native files and software models were also saved on the Aurecon server. An Aurecon tracking register was also set up for the project to keep record of the verification of the reports.

## 3.2 Process Followed

The following steps were conducted as part of the verification process:

**Step 1**

An email notification was received via Mezzoteam indicating that a geotechnical report had been uploaded for the DVE review. (A one to two week comment or response cycle was assumed for each of the design packages. Typically a response target of seven days was expected following the date of receipt of the design.)



Tue 5/10/2016 7:56 AM
Info Mezzoteam <mezzoteam@rls-jv.com>
- Issue of document GEOTECHNICAL RED LINE TUNNEL REBB010 - SWITCHBOX M10/M11 TO HIA PRE CONSTRUCTION CONDITION SURVEY SHEET 1 OF
To Salona Naidoo
If there are problems with how this message is displayed, click here to view it in a web browser.

has distributed the document GEOTECHNICAL RED LINE TUNNEL REBB010 - SWITCHBOX M10/M11 TO HIA PRE CONSTRUCTION CONDITION SURVEY SHEET 1 OF 2 in the workspace RLS-JV

For your review.

M003-QGD-GEO-RPT-01203 Rev 1.1
M003-QGD-GEW-DWG-REBB010-AA-13290 Rev 3

Figure 2. An example of the Mezzoteam notification

**Step 2**

The report was downloaded and a blank Design Verification Record (Herein after referred to as DVR) was set up with the first revision number. The DVR comprised a table (Figure 3) which indicated:

- The report name, number and revision that should be reviewed;
- The unique DVR and revision number – Aurecon project specific;
- The date indicating when the report and comment response was received;
- The discipline, namely, Geotechnical;
- The summary of findings (to overall identify if the design complied, required further development to comply or did not comply);
- The DVE comments as well as the designer's response; and
- The date that the response had been closed out.

**Step 3**

The report was reviewed independently by the DVE geotechnical team. The report was reviewed against four items, i.e (1) Employer's requirements, (2) system assurance, (3) technical and (4) general. Each comment compiled by the DVE was allocated a category i.e (1) suggestion, (2) discussion item, (3) important issue and (4) critical issue. Major issues (3 and 4) generally needed to be resolved before proceeding to the next design stage. Minor issues or comments were likely to be carried forward and addressed during the next design stage.

DESIGN VERIFICATION RECORD – COMMENT AND RESPONSE FORM

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Project Name: | Doha Metro Red Line South Underground | | | Aurecon Project No: | | | | |
| Design Package: | | | | Checking Leader: | | Discipline | Geotech | |
| Design Codification: | | | | DVR Codification: | | | | Rev |
| Design Stage | | | | | | | | |
| Date Received: | Click here to enter a date. | Design Revision: | | Checked By: | | Date: | Click here to enter a date. | 1 |
| Date Received: | Click here to enter a date. | Design Revision: | | Checked By: | | Date: | Click here to enter a date. | |
| Date Received: | Click here to enter a date. | Design Revision: | | Checked By: | | Date: | Click here to enter a date. | |
| DVE Summary Finding: | Design Document Complies | | | | | | | |

DS2 Phase Comments

Comment Category: 4 – Critical issue 3 – Important issue 2 – Discussion item 1 – Suggestion

Items to be reviewed in Rev 1 of DVR:

| DOCUMENT CODIFICATION | REV. | TITLE |
|---|---|---|
| DRAWINGS | | |
| M003-QGD-GEO-ENN-00D48 | 0.1 | ENGINEERING NOTE – TEMPORARY STABILITY UNDER MOBILE CRANE (<100T) LOADS UMM GHUWAILINA (RSST020) SUBWAY ENTRANCES 1, 2 AND 3 |

Comments:

| Item No. | Drawing No. / Document Reference | DVE's Comments | Cat. | Contractor / Designer's Response | Open / Closed (Date) |
|---|---|---|---|---|---|
| Compliance Against Employers Requirements | | | | | |
| 1. | | | | | |
| | | DVE response (Reviewer initials – Date) | | Contractor / designer's response (if required) (Date) | |

DVR Codification: M003-AUR-ENG-DVR-0708-1 Date:04/01/2017 Page 1 of 2

Figure 3. An example of the DVR

**Step 4**
As part of the Aurecon quality control process, the DVE geotechnical checking leader then checked the review with all supporting documents i.e calculation sheets, software models, input parameters, ground profile assumed by the DVE as well the internal interdisciplinary interfaces etc.

**Step 5**
The DVR was then submitted to the DVE coordinator (based in Doha) who then uploaded the DVR on Mezzoteam. All other supporting documentation and revisions were also saved on the Aurecon server as well.

**Step 6**
The contractor or designer was then notified of the comments.

**Step 7**
The contractor or designer then responded to comments with the aim to clarify the assumptions, edit the report and close out the comments.

**Step 8**
The process from step one is then repeated until all the comments are closed out. Where there has been numerous responses and the designers were at loggerheads, a formal discussion was then carried out to try and close out the comment.

**Step 9**
Once the comments were closed out, the DVE certified the report and it was then submitted to Qatar Management Consultant for comment. If there were comments from Qatar Management Consultant (QMC) that required the designer to edit the reports or design, the DVE was then required to review and recertify the document once the comments were closed out.

## 4 Process and Project Challenges

Challenges regarding the verification process and the overall project included the following:

1) Working remotely from site (the geotechnical verification occurred in the Tshwane Aurecon office) and the verification leader only occasionally visited the site;
2) Aligning the independent analysis with regards to the design parameters and ground models against the designers analysis;
3) Reiterative discussions on design alignment – technical compliance;
4) Difficulty in planning resources, since the submission schedule provided by the designer and contractors was continuously changing;
5) Ever changing design verification priorities;
6) Various revisions of the design resulting in various iterations of the design reports;
7) Verifications requests that did not entirely follow the design sequence such as verifying the temporary stability report prior to the GIR or verifying the GIR prior to the factual report;
8) Changes in the reports and associated references to all other reports which needed to be updated and resulted in further reports requiring verification and certification;
9) Initially the designer did not indicate the changes from the previous report which meant that the next review was not focused on the changes and this resulted in longer verification periods;
10) Submission of the revised design report to the DVE but excluding the accompanying DVR; and
11) The above then resulted in numerous financial variations and cost implications as the number of reports and revisions had considerably increased which also increased the review period

## 5 Process and Project Successes

Successes regarding the verification process and the overall project included the following:

1) The geotechnical team developed various internal models and spreadsheets;
2) Exposure and interactions of Aurecon to other geotechnical designers in the industry;
3) Robustness of the geotechnical team to handle changing priorities;
4) Developing design verification procedures or method statements for the works which could be implemented on future similar projects;
5) Improved knowledge on Doha geology and ground conditions; and
6) Developing project costs for similar types of works since the process allowed tracking of all design and verification reports.

## 6 Proposed improvements to the Process

The following improvements are proposed:

1) Automating the Design Verification Report unique Number and revision as well as the tracking register;
2) Creating a restriction where the updated report loaded on Mezzoteam should indicate a comment as to the reason for the update;
3) Creating a restriction where the updated report loaded on Mezzoteam must accompany the updated DVR as well;
4) Assessing and fixing the important design parameters upfront; allowing a gate review for parameters which would limit the “cart before the horse” scenario and therefore reducing the number of report revisions;

5) Introducing a "live" verification schedule on Moezzoteam and tracking status of the reports and drawings. Once a document has been uploaded for review, the "live" tracking register immediately updates to compare if the schedule is on track and automatically calculates the due date based on the agreed contractual turnaround time;
6) Detailed workshops in country should be carried out continuously to allow alignment between the verification engineers (DVE) and the designer or contractor; and
7) Creating ranges of acceptance within the verifications assessment or sensitivity, so if there is a slight change in the input parameter, the approval process is just a formality and the range of acceptance is known.

## References

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