Settlement Mitigation by Using Cellular Concrete
at Nepewassi River Bridge, Ontario

A Case Study

Ken S.Q. Ahmad, P. Eng. Senior Foundations Engineer – MTO
Tony Sangiuliano, P. Eng. Senior Foundations Engineer – MTO
Brad Dolton, P. Eng. Geotechnical Manager, Cematrix (Canada) Inc.
Tim J. Smith, P. Eng. Pavement and Remediation Solution Manager, Lafarge

Paper prepared for presentation
at the ROADS AND EMBANKMENTS -- CASE STUDIES AND LESSONS LEARNED session

of the 2016 Conference of the
Transportation Association of Canada
Toronto, ON

Abstract:

The Ministry of Transportation of Ontario (MTO) trialled the use of cellular concrete to control the ongoing settlements of approach embankments at a bridge located in northern Ontario. The bridge carries Highway 535 over the Nepewassi River, 3.5 km south of Highway 17, in the Township of Dunnet.

Historical and recent monitoring data confirmed the approach fill embankments had been settling for over 60 years. Since 2000, the south and north approaches settled 275 and 240 mm respectively.

The site is underlain by some 15 to 20 m of soft, compressible clay deposits which overlay more competent soil comprised of compact to dense sands. It was postulated that the continuing settlement of the embankments was due to the primary consolidation of the soft clay deposits.

To remediate the grade of the embankment and mitigate the continuing settlement, the existing embankment fill was removed and replaced with a lightweight fill of cellular concrete. This was carried out with the intent of reducing the effective vertical stresses on the soft clay deposits, so that the resulting ground movement would be more tolerable.

Cellular concrete is a lightweight fill material that is listed on the MTO's Designated Sources of Material List and satisfies MTO's prequalification requirements. Environmental approvals comprised a substantial component of the acceptance of cellular concrete.

As part of this trial, extensive instrumentation and monitoring programs were incorporated to assess the geotechnical and environmental performance of the embankment.

This paper presents the results of the foundation investigation, the design methodology, the environmental monitoring and post construction settlement monitoring after unloading the approach fills by using cellular concrete.

1.0 Introduction

The Ministry of Transportation of Ontario, MTO, trialled the use of lightweight cellular concrete to control the ongoing settlements that were occurring at the approach embankment fills of the Nepewassi River Bridge in Ontario.

The Nepewassi River Bridge is located on Highway 535, 3.5 km south of Highway 17, in the Township of Dunnet, Ontario. The current bridge is a two lane, three-span steel plate girder structure with a concrete deck supported on end bearing steel H-piles that was built in 1999. This is the third structure built at this location. The approach fills of the Nepewassi River Bridge had been settling for over 60 years. In the past 12 years before the construction of the current bridge, the fills at the south and north approaches settled 275 and 240 mm respectively.

Cellular concrete was used as a lightweight embankment fill to reduce the embankment stress on the native compressible cohesive soils at the site. Cellular concrete contains a foaming agent that creates air voids within the concrete. For this project, the unit weight of the cellular concrete was 4 kN/m^3, which is significantly lighter than conventional concrete and soil or rock backfill.

2.0 Site Description

The topography at the site can be described as a low wide valley, through which the Nepewassi River meanders, flowing from the west to east. The river valley is some 300m in width, at the bridge site, with outcropping bedrock defining the south and north valley walls. The width at the bridge crossing is 50.8 m wide.

2.1 Subsurface Conditions

Earlier geotechnical investigation revealed an extensive deposit of soft clay up to 20 m in depth underlain by cohesionless silts and fine sands to sands and gravels overlying biotite gneiss bedrock, which was encountered at depths varying from 45m to 27m, (south and north abutment respectively). Groundwater, in the lower aquifer, was under an artesian condition, which indicated an elevated hydraulic gradient of 1 to 2 m above the water level in the river.

In geotechnical investigations in December 2011, four (4) sampled boreholes were taken at this site. A surficial pavement structure consisting of 200 to 275 mm of asphalt and 175 to 250 mm of crushed gravel was encountered in the boreholes. At the two borings advanced through the approach slab a short distance behind the rear of the abutments, a surficial pavement structure consisting of 240 to 200 mm asphalt and 280 to 350 mm concrete approach slab was encountered. A void of 125 to 130 mm was encountered below the approach slab.

Below the asphalt layer, the subsoil comprises of some 3.7 to 4.3m of blast furnace slag fill. This is underlain by some 15 to 20 m of soft to firm clay deposits. Beneath the clay, there exists compact to very dense silts to fine sands and gravel deposits that overlay biotite gneiss bedrock.

The underlying clay was found to have Liquid Limits that ranged from 39% to 66% and Plastic Limits that ranged from 18 to 25%; the clays were classified as medium to high plasticity (CH to CI). The natural moisture content ranged from 38 to 70%, and typically neared its liquid limit.

The results of oedometer tests carried out in 1995, and data from a 1992 preliminary alignment study, indicate that the clay stratum was slightly over consolidated in three of the tests and more normally consolidated in the lower portion of the clay deposit.

Oedometer tests were again carried out in 2011 on 4 clay samples, taken from depths of 12.5 to 18.6m. The results suggest a pre-consolidation pressure of 130 to 160kPa, with an overconsolidation ratio ranging from 0.81 to 1.23. These findings were consistent with the field observations of continuing settlement of the approach embankment.

The boreholes location plan and subsurface stratigraphy is shown on Figure 1.

2.2 Site History

<u>Pre 1999</u>

The bridge crossing the Nepewassi River was originally a timber structure supported on timber pile bents. In 1951, the original structure was replaced with a 11-span, timber trestle-type structure founded on new timber pile bents. Based on the historical information, and recent data it was evident that the approach fill embankments have been settling for a period of 60 years. It is probable that the approach embankments had been settling prior to the 1951 construction of the second trestle type bridge. The exact magnitude of settlement prior to 1999 is not known since records were not available at the time of placement of patching works to correct approach fill settlement at the bridge abutments.

<u>1999</u>

In 1999, a three span steel plate girder bridge with concrete deck supported on end bearing steel H piles was constructed. A temporary detour bridge was constructed some 15m west of the existing bridge to allow traffic flow during construction. During the construction of the embankment for the detour bridge, a slope failure occurred at the south embankment. It is estimated that the height of the embankment fill was some 5 m when the failure occurred. The slip confirmed the susceptibility of the clay deposit to short term slope instability under load increases.

Now, the 1999 three span concrete bridge required a grade raise at the approaches. Embankment stability and settlement analyses were carried to assess the implications of the grade raise on the clay stratum that is approximately 20 m thick below the south abutment fill and 13 m thick below the north abutment fill.

To prevent potential slope instability and to mitigate embankment settlements, the design for the grade-raise required a zero net load increase above the existing load. To achieve this, the 1999 design called for a partial removal and replacement of the original earth fill embankment, and building the approach embankment with predominantly lightweight blast furnace slag (LBFS). At the south abutment, the original embankment backfill was sub excavated to an elevation of approximately 204.9 m and backfilled with LBFS resulting in a finished grade of elevation 207.8 m (2.9m thick fill). At the north abutment, the original embankment backfill was sub excavated to approximate elevation 205.1 m and LBFS backfill was used to build the approach embankment up to a finished grade of 208.9 m (3.8m thick fill).

Following the construction of the three (3) span concrete bridge in 1999, embankment fill settlement at the abutments continued. A geotechnical review of the pavement repair works indicated that settlements of up to some 270 mm had occurred at the abutments since construction of the new bridge.

3.0 Embankment Performance Assessment and Design Considerations

An investigation was commissioned to assess the embankment performance for both embankment stability and settlement. Consolidation testing, compiled during a 2011 study, indicated that the effective stress on the underlying soft clay exceeded the preconsolidation (pc') values for the underlying clay deposit. As such, it was concluded that consolidation settlement would continue. To control future settlement of the approach fill, to within tolerable limits, the effective stress directly behind the abutments for a 10 m length (below approach slab) needed to be reduced to a value estimated to be less than the preconsolidation pressure of the underlying clay stratum. The relationship between the historical and existing effective overburden pressure and the estimated preconsolidation pressures, developed during the 2011 study, are shown as the Stress History plots for the south and north embankments on Figures 2 and 3.

At the south embankment, where the clay deposit is thicker, it was necessary to reduce the vertical effective stress by 50 kN/m^2 to achieve a stress level less than the preconsolidation level of the clay stratum. Likewise, at the north embankment, a reduction of 40 kN/m^2 was required.

In order to decrease the effective stress associated with the embankment fills and maintain the existing bridge elevation, reconstruction of the embankments using light weight fills was considered. A number of options were evaluated and cellular concrete proved to be a suitable candidate. On this project, cellular concrete was selected because of its competitive pricing, relatively rapid construction, whilst being structurally stable and not susceptible to common construction hazards such as fire and hydrocarbons.

3.1 Blast Furnace Slag

In 1999, the grade raise was achieved by the sub-excavation and replacement with light weight fill; 3/8” blast furnace slag (BFS) fill, with a design unit weight of 11.5 kN/m^3 was employed. The use of BFS fill had been to accommodate the new grade raises and satisfy the requirement that no additional load was imposed on the underlying clay (i.e. zero net load increase). As such, the approach embankments for the 1999 bridge were constructed by sub-excavating 2 m of the existing embankment fill and replacement with some 2.9 and 3.8 m of BFS fill, at the south and north approaches respectively. However, since the completion of the approach grade rise in 1999, ground settlements continued.

Site investigations performed in 2011 found that the BFS fill had undergone cementation and densification since it was placed. Mass/volume measurements taken on retrieved samples indicated a unit weight of approximately 15.5kN/m^3.

It is not unreasonable to conclude that the densification of the BFS fill had contributed to the overall settlement of the approach embankments.

3.2 Buoyancy and Differential Icing

The use of lightweight concrete was intended to limit the ground movements associated with primary consolidation settlement. At sites that were subject to seasonally high groundwater, due to flooding for example, consideration was also given to the effect of buoyancy on the lightweight fill. This was the case for the southern embankment.

The design of the Nepewassi bridge was based on a flood return period of 50 years (i.e. water level of 206.4m). Since the southern embankment was to be constructed to an average finished grade level of 207.8m, this meant that most of the cellular concrete would be located below the design flood level.

To cater for buoyancy forces on the cellular concrete, counterweights were used. Gabion baskets were placed at the toe of the future embankment slope, and after the initial curing, each lift of cellular concrete fill was strapped to the gabion baskets.

Differential icing of the pavement surface is also a known “side-effect” of using certain non-earth materials like cellular concrete below a pavement. This is attributed to the difference in thermal insulation/conduction properties of the non-earth materials, compared to an adjacent pavement founded on earth fill; the former pavement types tend to be more susceptible to the development of surface ice. This issue was addressed by using about half a meter of engineered fill (OPSS Granular A) to form the sub-base and subgrade.

4.0 Contract

Before this project, the use of cellular concrete on MTO contracts was rare and often only in emergency situations. To provide a contractual framework for the supply, placement and quality control of cellular concrete for use as lightweight fill, a new Non-Standard Special Provision (NSSP) was developed. The key aspects of the NSSP are given in the following paragraphs.

4.1 Prequalification of Cellular Concrete Product

Prior to the commencement of construction, a proposed lightweight concrete was to be submitted to the MTO prequalification process for Lightweight Fill and go through the approval process.

4.2 Qualifications

The suppliers and contractors were required to provide a résumé, demonstrating their experience in the production and placement of cellular concrete. The contractors had to state at least 5 past projects of comparable complexity performed in the last 3 years.

4.3 Environmental Protections

The protection of the natural environment was a priority. As such, the contract had prohibited the placement of lightweight concrete below the water table. It also ensured that the formwork was leak -proof and that the cellular concrete did not include any amount of fly-ash.

4.4 Quality Control

During construction, cellular concrete was sampled every $50m^3$ or once every 30 minutes, which ever was more frequent. The samples were cured, and tested for compressive strength and the dry unit weight. All samples met or exceeded the specified requirements.

5.0 Construction

In 2012, the construction of the cellular concrete approach embankment fill was carried out by the removal of the existing fill (combination of blast furnace slag and earth fill) and replacing it with cellular concrete.

Cellular concrete formed the main core of the new approach embankment and this approach served to reduce the effective vertical stresses on the underlying clay deposits.

The design had initially called for the approach embankment fill to be removed to elevations 204.9 m at the north approach and to elevation 202.7 m at the south approach. At the time of construction, the water level in the Nepewassi River rose and this led to the first lift of cellular concrete being placed at a grade elevation of 202.9m, and a reduction in its thickness by 200 mm. Typical cross sections of the cellular concrete embankment are shown on Figure 4.

5.1 Supply and Placement of Cellular Concrete

Cellular concrete, sometimes called foam concrete, consists of Portland cement, water, specialized pre-formed foaming agent, and air mixed together in controlled proportions to create a lightweight construction material. Fresh cellular concrete is highly flowable and can be pumped into place over large distances through flexible hoses. In the vast majority of cases, cellular concrete is cast-in-place.

Cellular concrete may be produced using either “wet” or “dry” mix processes. “Wet” mix processes use a cement and water slurry batched by a ready mix supply company. Once onsite, the temperature, density and viscosity of the slurry is measured to confirm compliance with the requirements to make cellular concrete. After quality is verified, the slurry is delivered into the cellular concrete equipment, which then injects foam into the slurry and pumps the cellular concrete into place. “Dry” mix processes are commonly used to produce cellular concrete for high volume and/or high production rate (approximately 100m^3 per hour) projects. “Dry” mix refers to the process whereby all of the constituents of the cellular concrete are blended onsite, first by mixing the cement and water into a slurry, followed by injecting the foam and pumping the cellular concrete into place. This project used a “wet” mix process.

The project documents specified cellular concrete material properties to be a wet (as-cast) density of 475 kg/m^3 (+/- 5%) with a compressive strength of 0.5 MPa at 28-days. Quality control was conducted continuously throughout the operation to measure cellular concrete properties. Cylinders were cast to verify compressive strength as per ASTM C495, Standard Test Method for Compressive Strength of Lightweight Insulating Concrete. Based on quality control data, the average cellular concrete 28-day compressive strength was 1.2 MPa. Minimum and maximum compressive strengths were 1.0 and 1.5 MPa, respectively.

Sheet piling was installed down the centreline at the approaches in order to maintain traffic across the bridge (see Photo 2).

Cellular concrete pump equipment was setup at the south end of the bridge with a flexible hose extending across the closed portion of the bridge to the north side. This allowed for placement at both ends from one setup location, which minimized disruptions to the cellular concrete production.

Cellular concrete was placed in lifts ranging in thickness from 500 to 650mm. Plywood formwork lined with polyethylene sheets was used to contain the cellular concrete where required. The edge of each lift was typically inset from the previous one. A total of 2,100 m^3 of cellular concrete was placed between August 14 and 31, 2012.

6.0 Post Construction Performance of Embankment

An extensive geotechnical and environmental instrumentation and monitoring program was incorporated into this project to assess the performance of cellular concrete in settlement mitigation and to evaluate its effect on groundwater and surrounding areas.

6.1 Settlement Monitoring

Post construction settlement monitoring comprised of settlement rods (SR) and settlement monitoring points (SP). Four settlement points and two settlement rods were installed in the new cellular concrete embankment, at both the north and south approaches. In addition, two settlement rods (SR No.1 and SR No.2) were installed within the embankment that was not “treated” with cellular concrete; this stretch of embankment was used as a control embankment. The Settlement Rods (SR) were designed to record vertical movement at the base on the cellular concrete whereas the Settlement Points (SP) were founded in the upper final lift of the cellular concrete and were intended to measure vertical movement of the surface materials. The vertical movement of the Settlement Rods (SR) and Settlement Points (SP) are presented graphically on the attached settlement data plots, Figures 5 and 6

The vertical movement of the south and north embankments are shown on Figures 5 and 6 respectively. As can be seen from the settlement data plots of both the SR’s and SP’s there is a distinct difference in the slope(s) of the settlement plots for the winter period (November to May) versus the summer periods (May to November). During the winter period of November 2013 to May 2014, the SR readings indicate 0 to +1 mm of settlement, in the area of the cellular concrete fill. This is considered to indicate essentially zero movement in the area of the cellular concrete. The SP’s also returned very low movement values varying from +2 to -1 mm over the winter months. However, over the summer periods, May to November 2013 and May to November 2014, the data indicates downward settlement varying initially from 8 to 10 mm (summer 2013) decreasing to 3 to 4 mm (summer 2014) at the SR’s located in the area of the cellular concrete. The data for the settlement points (SP) also indicates that the settlement recorded during the summer of 2014 was at a decreased rate relative to the settlement recorded during the previous summer of 2013, similar to the data at the settlement rods, Figure No. 6.

In the areas beyond the lightweight (cellular concrete) fill treatment, represented by SR 1 and SR 2, settlement has occurred during both the summer and winter periods however at a distinctly lower rate during the winter as indicated by the settlement plots SR 1 and SR 2. The relationship between the movement of the deep seated SR’s and the adjacent shallow seated SP’s has been plotted on Figure 5 and 6. This data indicates that at all four locations where SR’s were located adjacent to SP’s the movement of the rods was equal to or only slightly exceeded the downward movement of the points. This difference generally varied between 0 to 2 mm except at the location of SR5 where the difference was a maximum of 3.5 mm on November 2013 reading.

The settlement data plots, taken over a twenty-four (24) month monitoring period, indicate that the magnitude of settlement of the cellular concrete approach slab fills is essentially zero during the winter months. However, settlement still occurs during the summer months, albeit at a decreasing rate. The data also indicates that the deep seated SR's are settling slightly more than the shallow seated SP's.

The reason for the continuing, although reduced and annually decreasing, settlement of the approach embankment fill is due to the effective stress on the underlying clays exceeding the preconsolidation pressure of the deposit. The Stress History plots, evaluated during the 2011 Foundation Investigation indicate that at the south abutment the planned effective vertical stress reduction of 50 kPa was only slightly lower at some 4 kPa than the estimated preconsolidation pressure evaluated for the sample from elevation 190.5 m. The river level limited the depth of excavation and therefore restricted achieving a greater reduction in effective vertical stress at the south abutment. At the north abutment the planned stress reduction of 40 kPa was some 10 kPa lower than the preconsolidation pressure evaluated for the sample from elevation 190.5 m. All other preconsolidation values, estimated during the 2011 Foundation Investigation, exceeded the stress reduction value(s) by an amount greater than those referenced above for the sample at elevation 190.5 m.

One would initially consider fluctuations in the groundwater table as possibly being the cause of the seasonal variation in the effective vertical stress on the underlying clay, and subsequent settlement. However, the periods during which settlement occurred (summer period) appears to be out of sync with the period when a low groundwater table (generally winter period) would result in an increase in effective stress. The existing surface aquifer will be recharged during the spring freshet which, presumably, would result in an increase in the groundwater table elevation. This would subsequently result in a decrease in effective stress imposed on the underlying clay, due to buoyancy. This reduction in effective stresses would result in a decrease in primary consolidation and associated settlement. However, the current data indicates that, settlement occurs predominantly during the summer period, not the winter period.

Conversely, one would anticipate that the snow cover would limit the recharge of the groundwater table due to precipitation and result in a lower groundwater table, which would produce a higher effective stress value, resulting in consolidation and subsequent settlement. However, essentially zero settlement is developing in the winter months.

Frost penetration also occurs seasonally and the design frost penetration depth for this area is 2.0 m. The reduction in vertical stress on the underlying clay deposit may be due to freezing of the upper portion of the highway embankment. As the upper part of the embankment fills freeze, the moisture in the granular/fill material freeze, expands and exerts a compressive force on the adjacent soil particles and this results in a frozen band under compression. The compressive force may be sufficient to result in a bridging effect which would reduce the vertical effect stress by an amount roughly equal to the thickness of the frozen layer.

The cause of this seasonal variation in the settlement of the cellular concrete approach fill is unknown at this time. Current settlement data indicates that essentially zero settlement is occurring during the winter months and the settlement that has developed during the summer months has decreased from 8 to 10 mm (summer 2013) to 3 to 4 mm (summer 2014) at the SR’s located in the area of the cellular concrete. Extrapolating the summer settlement data indicates that it is likely that the settlement of the SR’s may reduce to 0 to 3 mm during the summer period for 2015, as the clay structure accommodates the load.

6.2 Groundwater Monitoring

Due to the proximity of the cellular concrete placement to the river, measures were taken to mitigate the risk of possible contamination to the river water. These measures included using liquid-tight formwork for the concreting activity, the construction of secondary spill containment around formwork and the monitoring of groundwater quality, with periodic sampling and testing.

The monitoring comprised of installation of two standpipes upstream and two standpipes downstream of the construction works. Standpipes were labelled GP 1, GP 2, GP 3 and GP 4.

A set of water samples was collected from the four standpipes (monitoring wells) and two surface water locations for the six month post-construction phase of the water quality monitoring. Surface water locations, upstream and downstream of the north bridge approach were named as SW 1 and SW 2. The odour and color of the water at each groundwater monitoring well was observed and recorded. Sampling and testing of groundwater was undertaken throughout the construction and the results of some key parameters are shown on Figures 7 to 10.

It is interesting to note that the concentration of chloride and total dissolved solids, and the alkalinity of the water increased during the construction period, and eventually returned to pre-construction levels. This is characteristic of all measured chemical content of the river water.

It is reasonable to assume that the water samples upstream of the bridge were not affected by the construction activity. The chemical contents of the upstream water samples were consistently lower than the downstream water samples, even before construction.

7.0 Conclusions

The application of lightweight cellular concrete at the Nepewassi River Bridge was successful. The effective stresses on the bridge approaches were reduced by the use of Cellular Concrete resulting in reduced settlement. In addition, there was no adverse environmental effect from the use of cellular concrete. Based on the performance of cellular concrete on this job the ministry will use cellular concrete in future projects to mitigate any stability and settlement concerns.

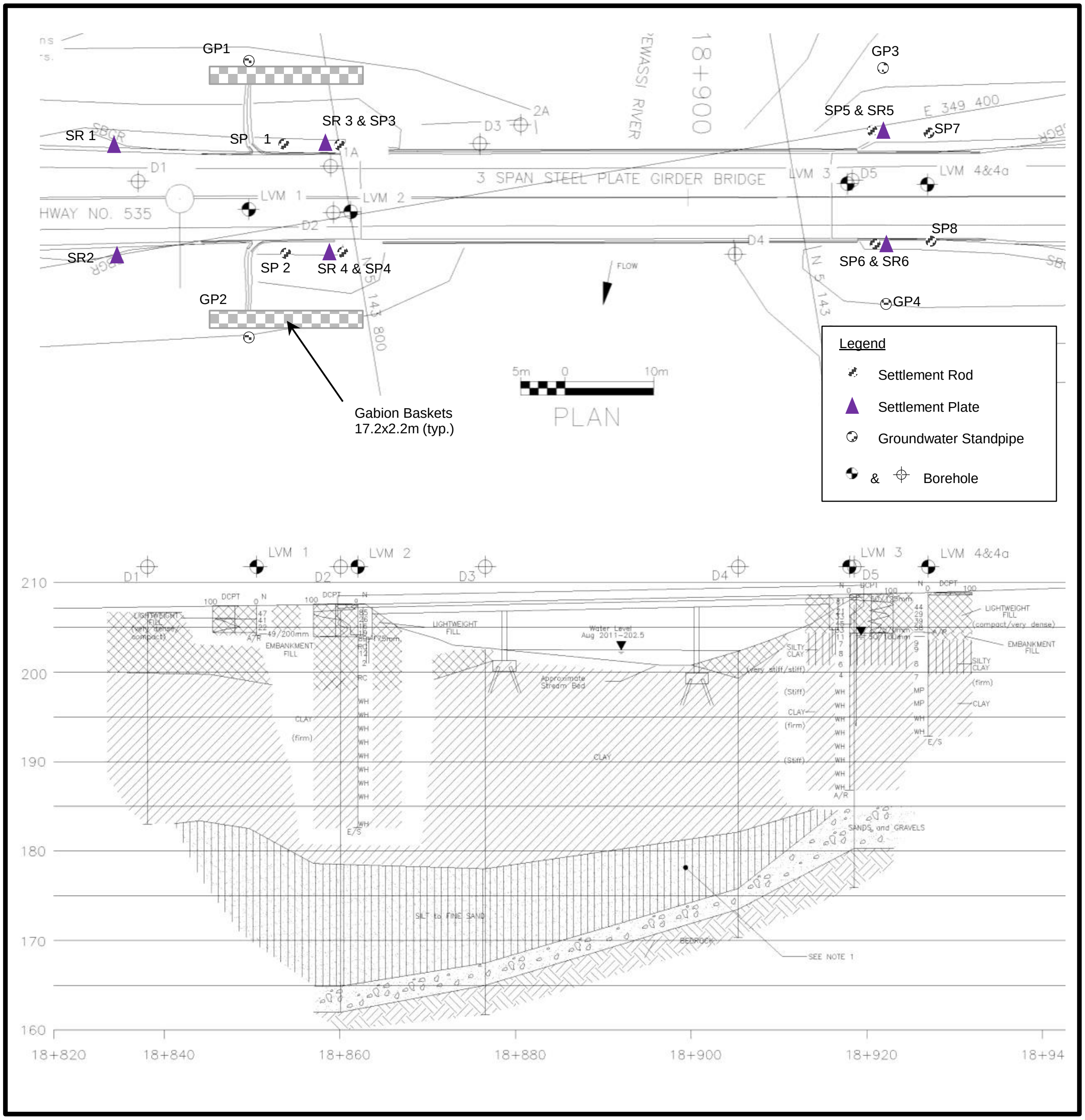

Figure 1 – Borehole & Instrumentation Layout Plan and Soil Stratigraphy

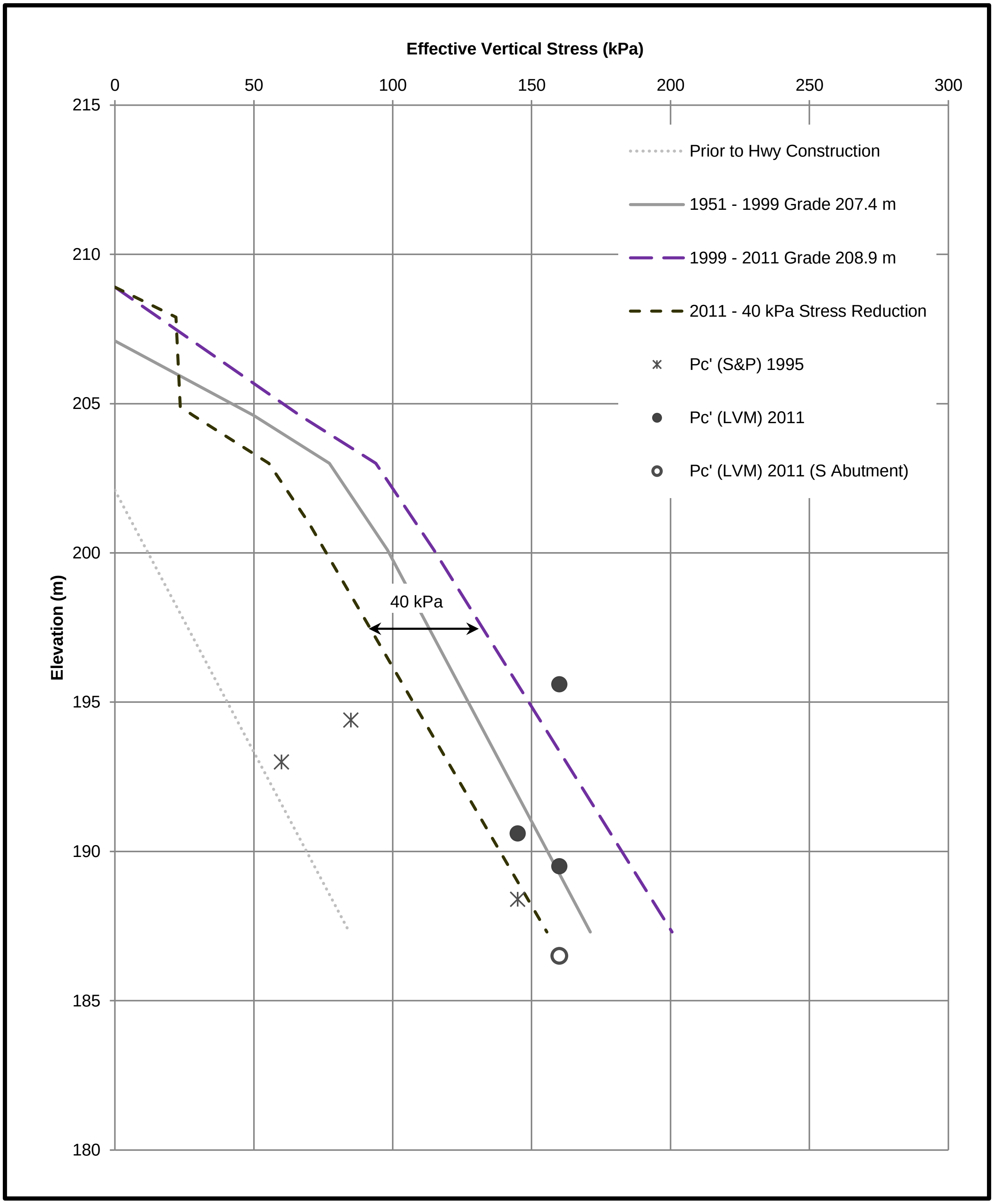

Figure 2 – Stress History of Northern Embankment

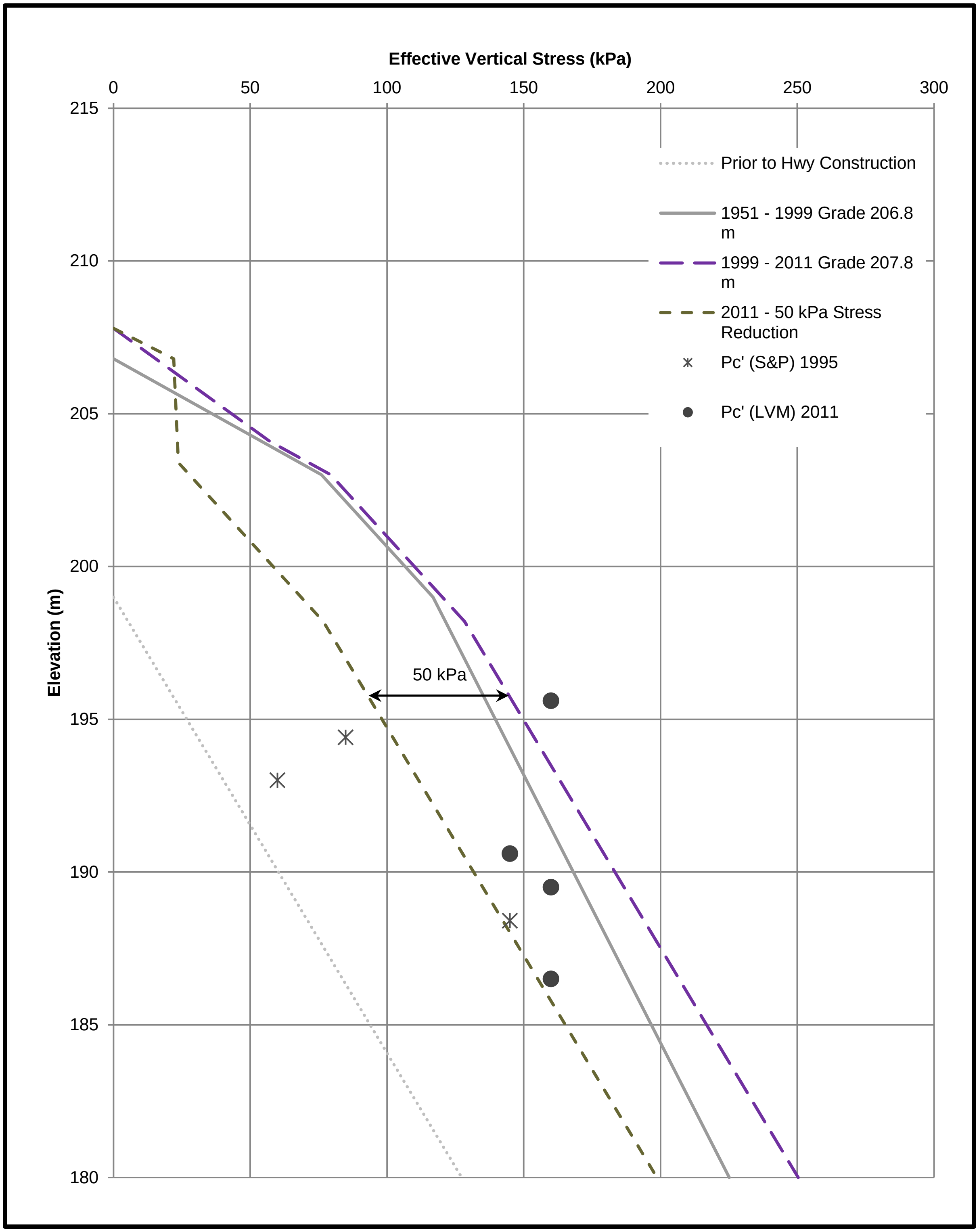

Figure 3 – Stress History of Southern Embankment

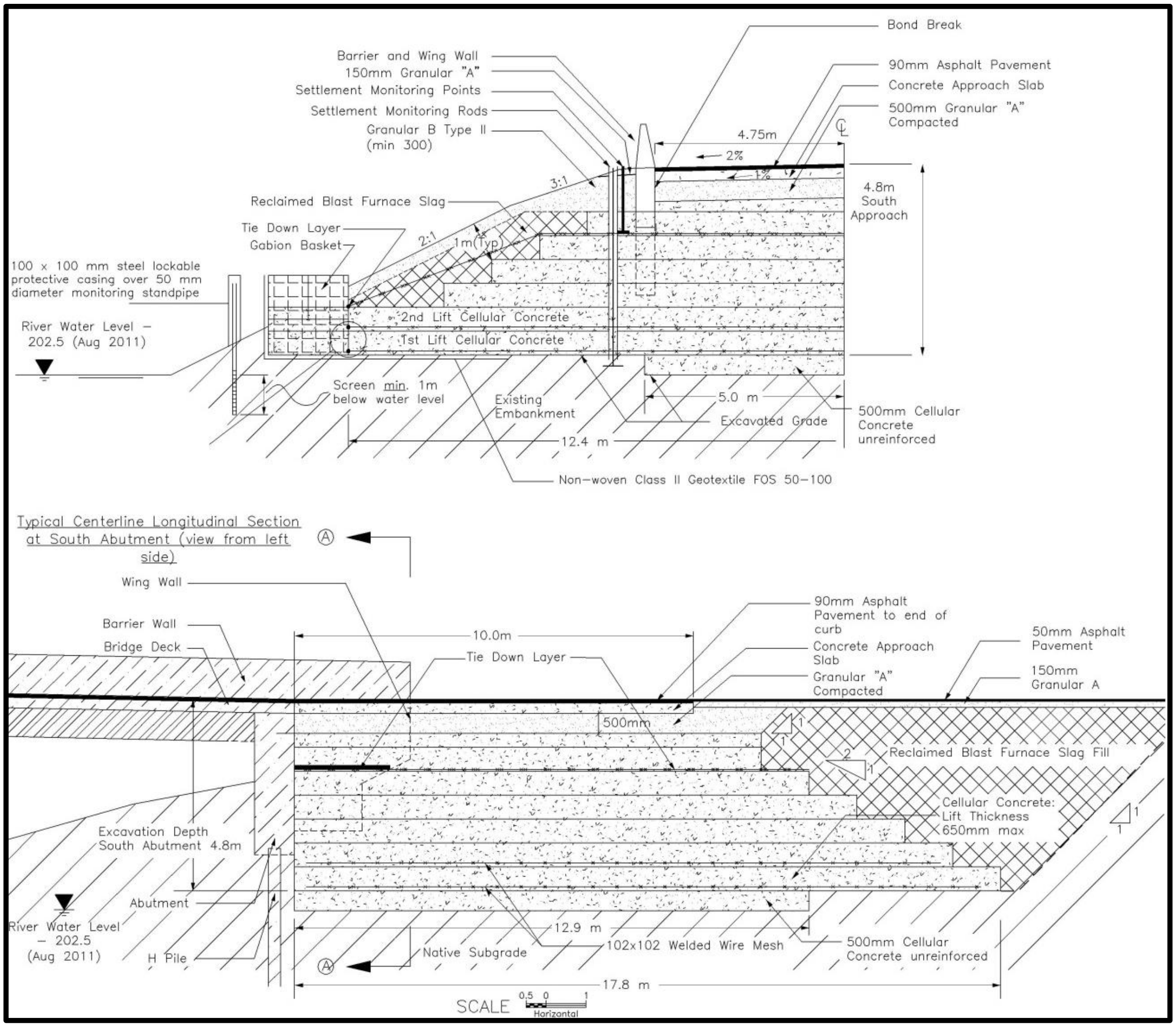

Figure 4 – Typical Section of South Approach Embankment

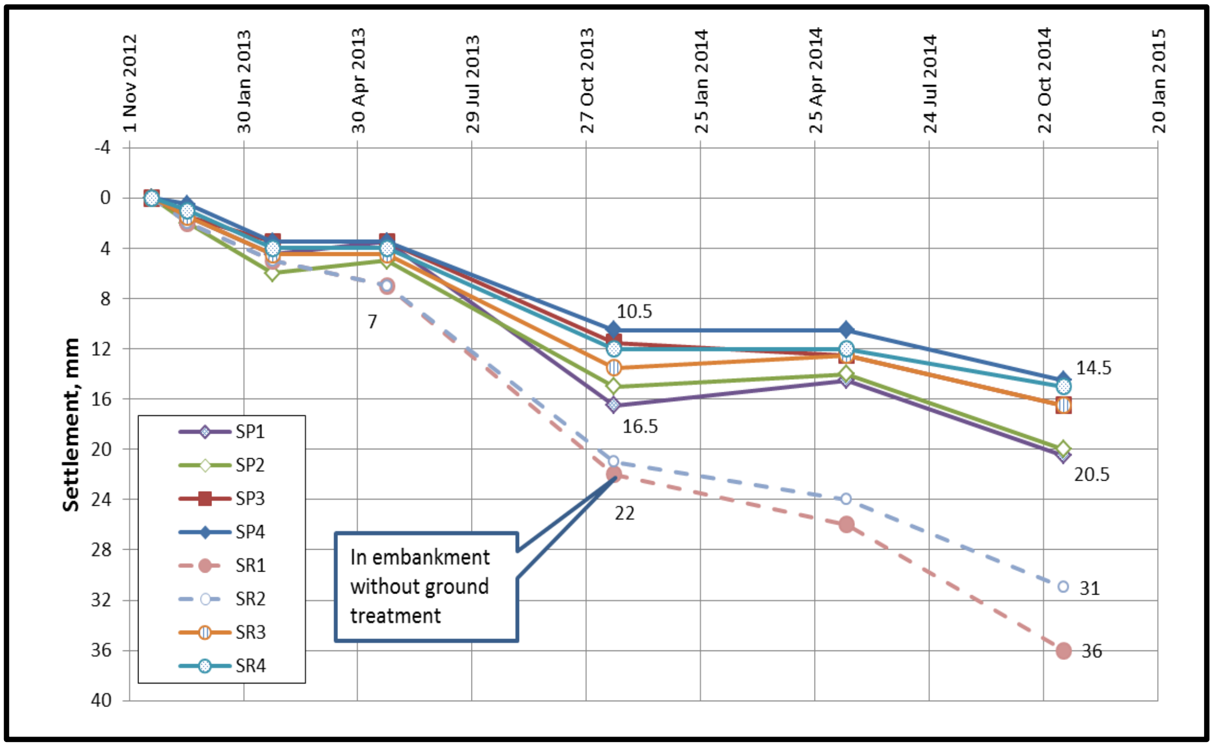

Figure 5 – Settlement of South Embankment

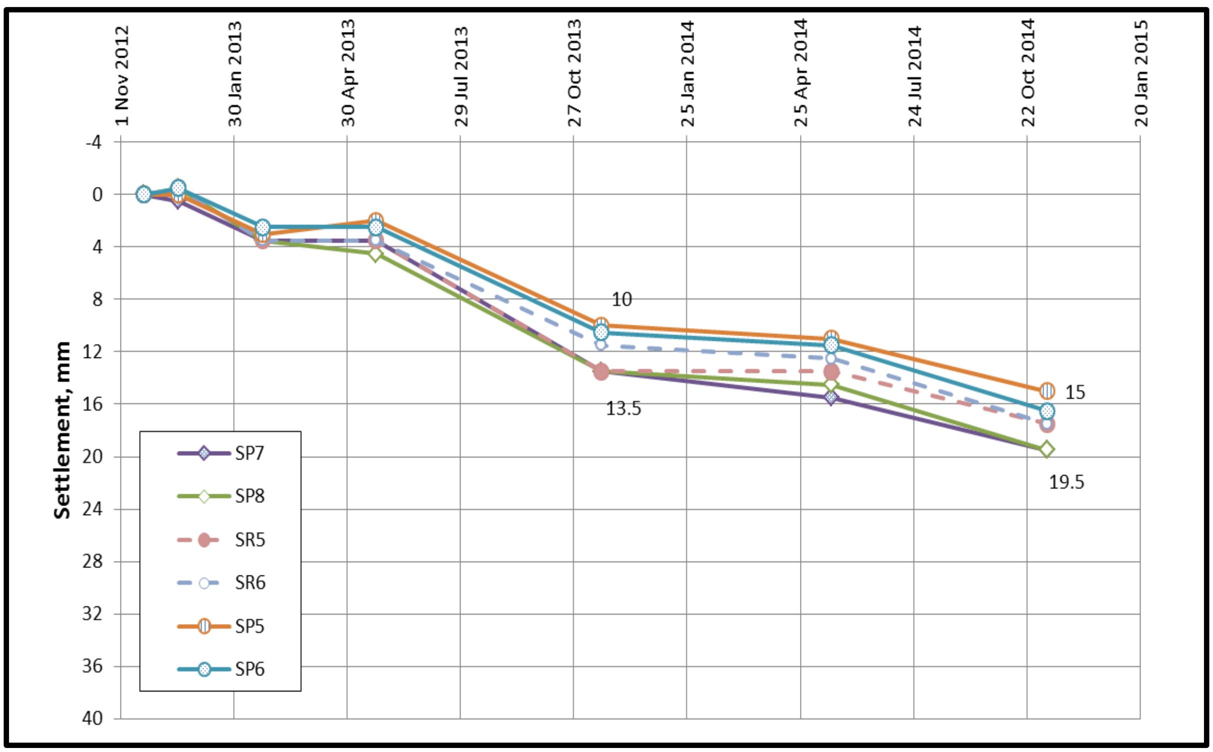

Figure 6 – Settlement of North Embankment

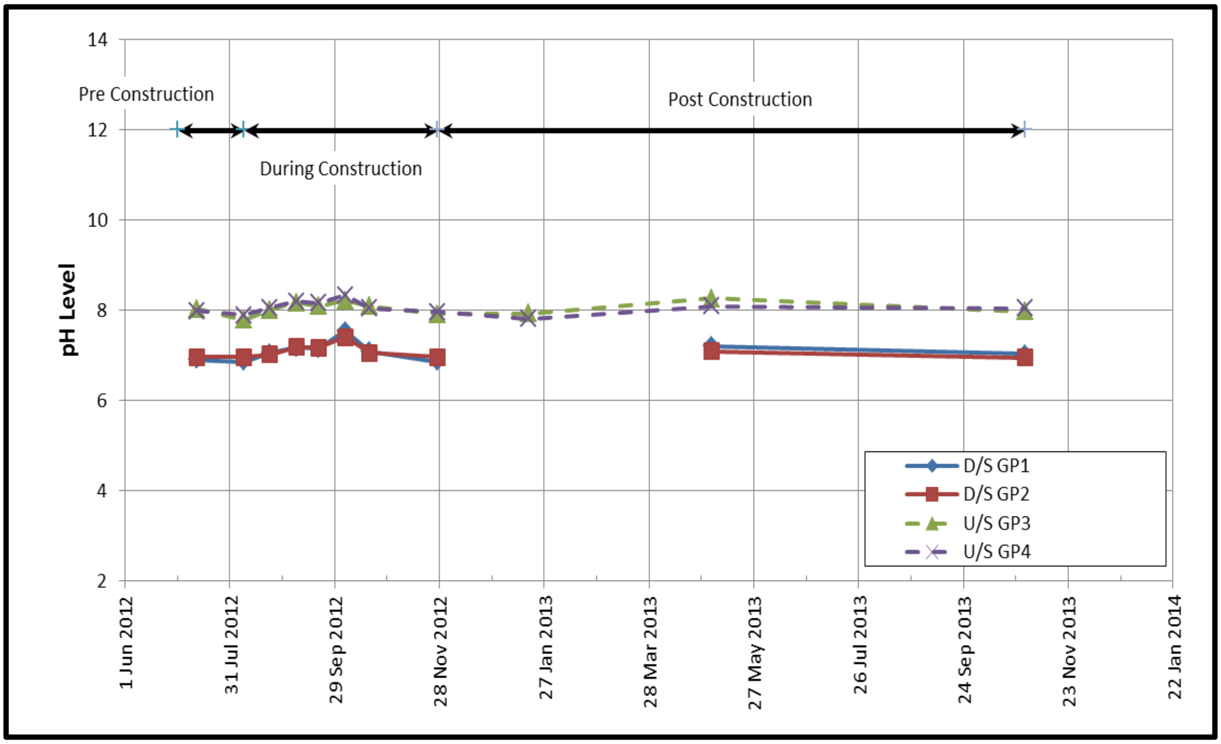

Figure 7 – pH of Groundwater
Upstream (U/S) & Downstream (D/S) of Embankment

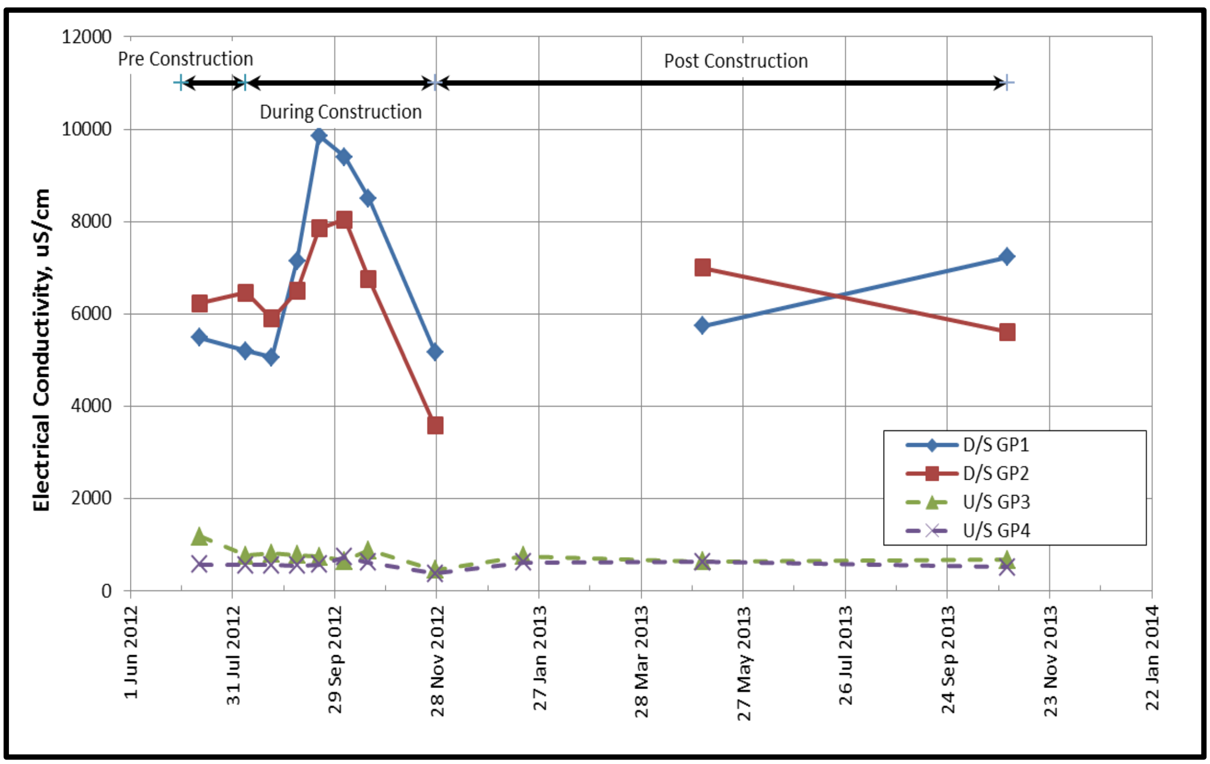

Figure 8 – Electrical Conductivity of Groundwater
Upstream (U/S) & Downstream (D/S) of Embankment

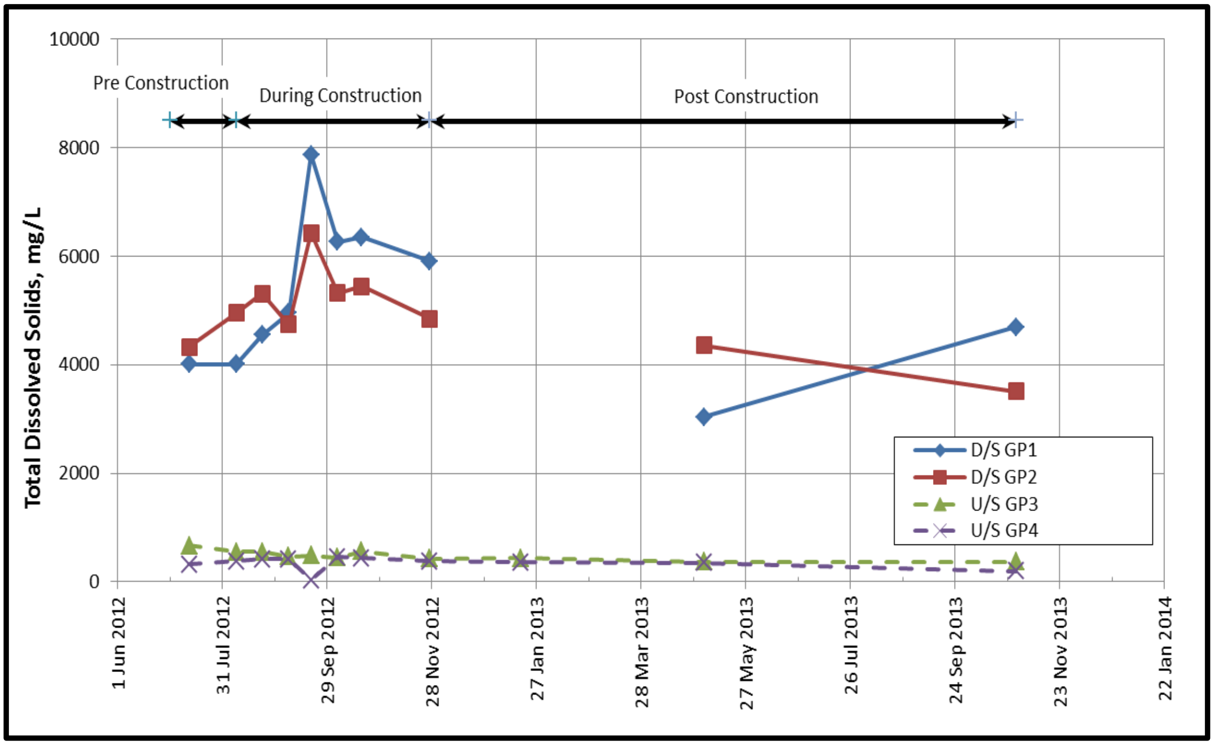

Figure 9 – Electrical Conductivity of Groundwater Upstream (U/S) & Downstream (D/S) of Embankment

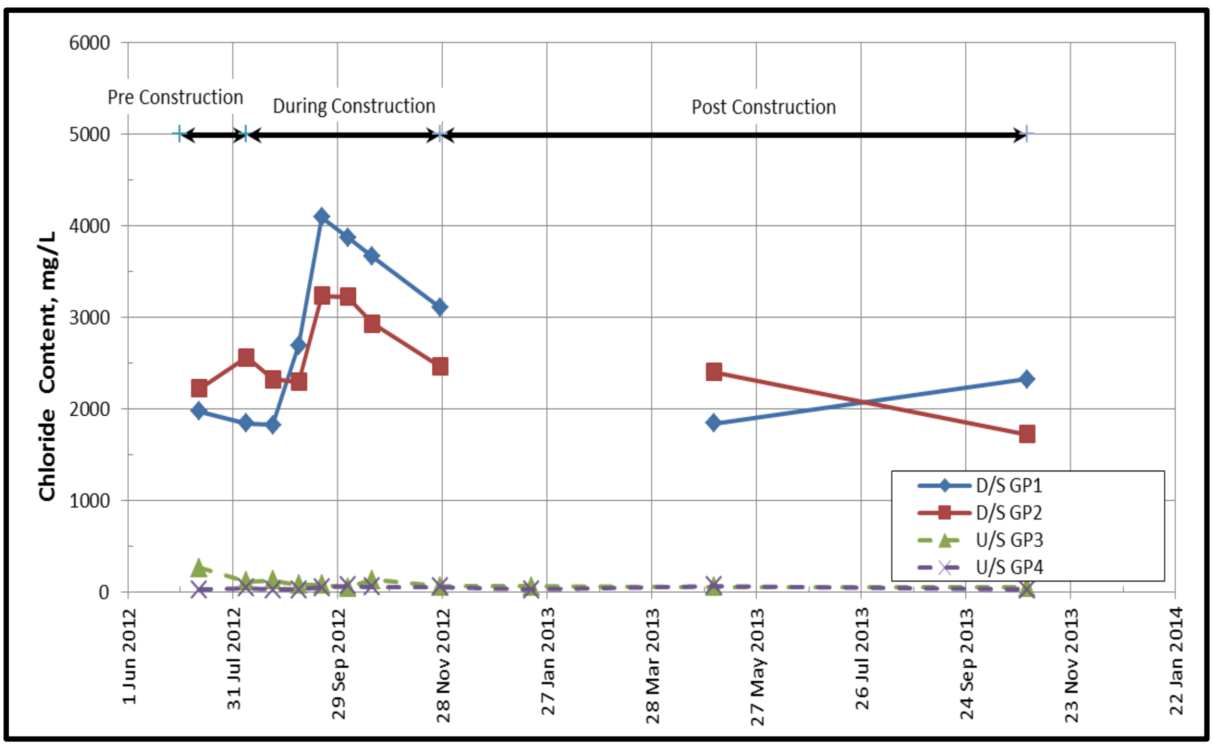

Figure 10 – Electrical Conductivity of Groundwater Upstream (U/S) & Downstream (D/S) of Embankment

Photo 1 – Construction of southern embankment. Lighter formwork was used due to the lower unit weight of cellular concrete. Wire mesh was used as part of tie-down system using gabion blocks.

Photo 2 – Placement of cellular concrete. Plastic sheets were used to make the forms impermeable and secondary containment was installed to guard against unplanned concrete discharge or spills

Photo 3 – Completed Northern Embankment

Photo 4 – Staged Construction of Embankment. Exposed subsoil of slag fill seen on the background