Stabilization of Rail Track Bed
Mile 9.0 – 9.1, CN, Dundas Subdivision

Colin Alston, P.Eng.
Alston Geotechnical Consultants Inc.
Markham, Ontario, Canada
Mario Ruel, Ing.
Geotechnical Consultant, Montreal, Quebec, Canada

ABSTRACT
A single-track rail line was constructed at the study site in a 15 m deep cutting in the 1850's, a second track was added to the north of the initial track in the 1890's. The soil profile consists of a layer of low permeability sandy silt with a trace to some clay in the base of the rail cutting which extends to a depth of about 3 m below the base of rail, where it is underlain by a layer of sand. The phreatic pressure in the sand layer was up to about 4 m above the rail track. The sandy silt layer which lies at the base of the cutting has behaved as an aquitard confining the pressure in the underlying sand layer. Breakouts of pressurized groundwater have randomly occurred in the cutting base; upward seepage through the silt has resulted in erosion of "pipes" which in turn resulted in track instability. Such instability required the imposition of severe slow orders on train movements and a 24 hour-a-day watch.

Track stabilization has been carried out in three phases over a period of 23 years. The first two phases were regarded as temporary and were intended to reduce the severity of slow orders until a permanent solution to the instability could be implemented. Both of the initial phases involved the removal of the then existing rail track and track bed, and construction of a new bed which incorporated a variety of geosynthetic reinforcing. The designs provided satisfactory support to carry a fully loaded train across a 0.5 m diameter void. The third phase of stabilization was designed to provide a long-term solution to the track instability by permanently reducing the phreatic pressure such that breakouts of groundwater would no longer occur. All of the applied corrective measures have been successful within their design expectations.

RÉSUMÉ
Une voie ferrée à voie unique a été construite sur le site à l'étude dans une tranchée de 15 m de profondeur dans les années 1850, et une deuxième voie a été ajoutée au nord de la première dans les années 1890. Le profil du sol se compose d'une couche de limon sableux à faible perméabilité avec une quantité infime à mince d'argile au fond de la tranchée qui s'étend jusqu'à une profondeur d'environ 3 m sous la base du rail, où il est recouvert d'une couche de sable. La pression phréatique dans la couche de sable atteignait environ 4 m au-dessus de la voie ferrée. La couche de limon sableux qui se trouve au fond de la tranchée s'est comportée comme un aquitard confinant la pression dans la couche de sable sous-jacente. Des ruptures d'eau souterraine sous pression se sont produites aléatoirement dans le fond de la tranchée; l'infiltration ascendante à travers le limon a entraîné l'érosion des « tubes », causant à son tour l'instabilité de la voie. Cette instabilité a nécessité l'imposition d'ordres stricts de ralentissement de la circulation des trains et d'une surveillance 24 heures par jour.

La stabilisation de la voie a été réalisée en trois phases sur une période de 23 ans. Les deux premières phases ont été considérées comme étant temporaires et visaient à réduire la sévérité des ordres de marche au ralenti jusqu'à ce qu'une solution définitive à l'instabilité puisse être mise en œuvre. Elles comprenaient l'enlèvement de la voie ferrée et de la plate-forme existantes et la construction d'une nouvelle plate-forme qui comprenait divers renforts géosynthétiques. Les conceptions ont fourni un support satisfaisant pour le transport d'un train à pleine charge dans un vide de 0,5 m de diamètre. La troisième phase de la stabilisation a été conçue pour apporter une solution à long terme à l'instabilité de la voie ferrée en réduisant de façon permanente la pression phréatique pour éviter les ruptures d'eau souterraine. Toutes les mesures correctives appliquées ont donné les résultats escomptés.

1 INTRODUCTION

The CN Dundas Subdivision extends from Hamilton Junction at its eastern limit west to London, Ontario, a distance of 78 miles (125 km). The portion of the Subdivision in which the study site lies was originally constructed by the Great Western Railway in the early 1850's as a single track. A second track was constructed on the north side of the original track in the 1890's. The study site lies in Copetown on the east side of Highway 52

The site lies at the western limit of a progressive uphill gradient which rises up Hamilton Mountain from the start of the Subdivision at Hamilton Junction (Mile 0.0). At the study site, the tracks lie in the base of a 15 m deep cutting.

Figure 1. Map of the study site

Although the original south track experienced few abnormal maintenance needs, the north rail track has experienced ongoing settlement and stability problems as a result of groundwater issues. The presence of a high water pressure in an underlying soil layer has resulted in internal erosion and the development of near vertical "pipes" daylighting in the subgrade. Soil from the underlying sand aquifer layer has transported to the ground surface with consequential surface settlement. In 1995, the severity of such unpredictable track settlement in the more recently constructed north track was affecting railway operations in two sections of track, at Mile 9.0 and Mile 9.1, this necessitated that a watchman be stationed at the site to carry out an inspection of the affected length of rail tracks prior to each train passage 24 hours-a-day, 7 days-a-week.

Figure 2. Areas of track settlement

Additionally, a 5 mph Slow Order was imposed on train movements. The operational constraints imposed by the Slow Order together with the costs of stationing a watchman on site full-time, resulted in an expenditure of several hundred thousand dollars per year. Geotechnical explorations to identify the site subsurface conditions were initiated in the 1950's, further work was carried out in the 1960's. Several analyses of those conditions have been made with a view to providing a stable track since that time.

2 SUBSURFACE CONDITIONS

The existing double rail track was constructed in a cutting which extends to a depth of about 15 m below the surrounding tableland. Borehole data show that an upper layer of clayey silt extends from the surface of the tableland to a depth of about 10 to 12 m. That layer is underlain by a layer of silt which is exposed in the cutting side slopes and extends to a depth of approximately 20 m below the tableland, which is approximately 3 to 5 m below the base of the rail cutting. In turn, the silt layer is underlain by a thick stratum consisting of layered silty fine to medium sand which includes seams of finer grained materials, the gradation of the sand becomes coarser with depth. Nearby homes and farms drew water from wells positioned in this stratum. In the base of the cutting, the rail tracks lie on rail ballast and sub-ballast granular fill materials. The fill extends to a depth of about 2 m below the base of the cutting, which illustrates the amount of track re-alignment which has been required in the past to maintain serviceability. A cross section through the cutting showing layer distribution and groundwater pressure is shown in Figure 3 below.

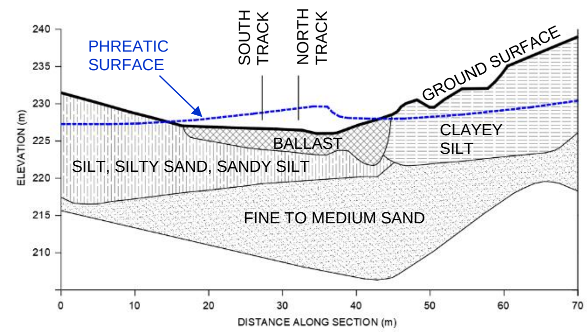

Figure 3. Soil Profile and Phreatic Surface

Water pressures within the underlying layered silty sand deposit are seasonally dependent and were up to about 4 m above the base of the cutting. The presence of the noted high water pressures resulted in breakouts of groundwater in the base of the rail cutting. The breakouts occurred through eroded "pipes" which extended from the underlying layered sand stratum to the cutting base, the breakout seepage transported sand material from the underlying sand deposit to the ground surface with consequential settlement at rail track level. In 1996, the consequences of soil erosion and transportation of soils from the sand stratum namely, large and unpredictable settlement of the north track was requiring that the north track be re-levelled and realigned every few days. Also, safe railway operation required that a watchman be on site on a round-the-clock basis.

3 DESIGN REQUIREMENTS

In the early 1990's the requirement to maintain service on the rail track was imposing severe operation challenges to rail movements and large costs. It was identified that a permanent solution to the track settlement issue would require that lowering the described high groundwater pressures within the sand soil stratum which underlies the silt layer at the base of the rail cutting be effected. However, in view of the anticipated time that design of that solution would require and with the need to cooperate with various regulatory bodies with regard to possible effects on the regional groundwater profile, a decision was made to effect a temporary solution to the track settlement issue which was to be given a design life of 5 years.

4 TRACK BED REINFORCEMENT 1996

Two sections of the north rail track were experiencing severe settlement and movement, namely at Mile 9.0 and Mile 9.1 at the Copetown site. It was decided that these two sections should be reconstructed on a reinforced track bed with a design objective that rail track settlement should be limited to not more than 25 mm in the event that a 0.5 m diameter void were to develop beneath the track bed as a result of soil erosion following breakout of the high groundwater pressure. Thus, the design should incorporate geosynthetic reinforcing sheets to carry the train loads across the design void together with a stiffening element to increase the radius of curvature at the rail track level. The design should also incorporate measures to allow upward seepage of groundwater whilst restraining the movement of soil particles.

Design guidelines for geosynthetic reinforcement sheets covering voids were available to assist in preparing the design for the reinforced track bed (Bonaparte and Berg, 1987, Giroud et al, 1990). Using these analytical methods, it was determined that the required ultimate tensile strength of reinforcing should not be less than 100 kN/m in both directions. Multistrand polyester geogrid was selected to provide a reduced creep reduction factor compared to high density polyethylene material. In order to add stiffness to the track bed, a 200 mm deep geocell component was incorporated in the track bed design to increase the equivalent thickness of track bed (Bathurst and Karpurapu 1993).

The track bed design was also required to include a system to allow for upward seepage of pressurized groundwater and for collection of that groundwater. Prior experience with construction of subdrains at the site had shown that well graded granular material which had been used as surround to those subdrains had become plugged with silt over a period of time, rendering the drains ineffective. Thus, in order to curtail soil movement and to improve the longevity of the groundwater collection system, a filtering layer of limestone screenings was laid on the subgrade at the base of the track bed (Cedergren, 1989). The selected design of reinforced track bed is shown in Figure 4 and described below:

- subgrade
- 100 mm limestone screenings
- multi strand polyester geogrid laid parallel to rail track
- 100 mm limestone screenings
- multi strand polyester geogrid laid transverse to rail track
- 100 mm sub-ballast
- double layer drainage net encased in non-woven geotextile
- 50 mm sub-ballast
- 200 mm deep geocell infilled with sub-ballast
- 50 mm thick sub-ballast
- 200 mm thick ballast (to underside of rail tie).

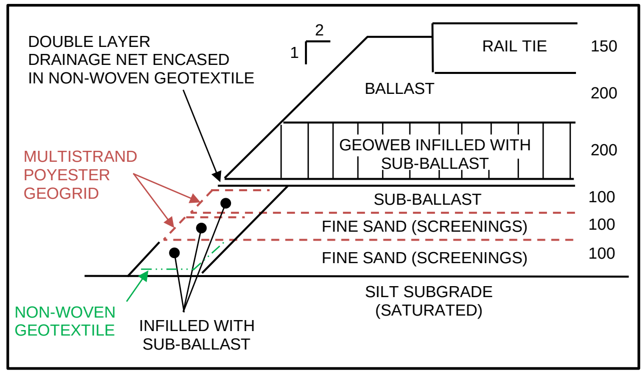

Figure 4. Track Bed Reinforcement Design 1996

In the course of reconstructing the two affected sections of track bed, instrumentation was installed to assist in monitoring performance. This consisted of installing three load pressure cells; the cells were placed immediately above the geocell unit, immediately above the upper geogrid reinforcing sheet, and immediately below the lower reinforcing sheet. The load pressure cells were complemented by placing pneumatic piezometers at the same elevation as the three load cells. Additionally, a settlement gauge was installed at the underside of the lower reinforcing sheet.

Observation of the performance of the reconstructed rail track showed that within a few days of completion, the rate of settlement of the reconstructed tracks was substantially reduced. This rate was further reduced over time such that after a period of a few months only routine track maintenance was required. The distribution of loads within the reinforced track bed was evaluated by stationing a railway engine immediately above the array of instrumentation. It was found that the upper earth pressure cell which was positioned immediately above the geocell showed an increase in pressure of approximately 10 kPa. There was no measurable response to the short-term loading recorded in either the pressure cell below the geocell or in the load cell positioned at subgrade elevation, within the limits of accuracy of the installed instrumentation. Phreatic pressures measured in the pneumatic piezometers showed groundwater heads at 2.8 m above subgrade elevation at that, 1.8 m at the elevation of the upper geogrid sheet, and 1.2 m above subgrade elevation at the underside of the drainage layer. The settlement gauge installed at subgrade elevation did not record any measurable movement in response to the stationary train load. The train was only parked above the instrumentation for the time it took to measure readings in the instruments, and then it moved away to free the track.

Over the period 1996 to 2014, no more than normal maintenance was required to maintain the serviceability of the rail tracks. However, in this time period, some significant breakouts of groundwater were observed in the devil strip between the north and south rail tracks and also, on the north side of the north track. Each of the eroded “pipes” which resulted from internal erosion was quickly filled with sub-ballast material, and this action was found to maintain track serviceability.

5 TRACK BED REINFORCING 2014

As noted, performance of the reconstructed, reinforced sections of track continued to behave in a satisfactory manner. However, a significant development of internal erosion and piping occurred in 2014 in a location immediately north of the north rail track, immediately east of the westerly section of the reinforced track bed (Mile 9.1). At that time, it was decided that the affected section of track should be reconstructed utilizing geosynthetic reinforcement. To improve design, newer methods which included the probable benefit of including multiple layers of reinforcement in controlling potential settlement were available to complement prior design methods (Collin, 1997). Also available at that time was ultra-high strength woven geotextile material. Geosynthetic manufacturers were able to provide data to quantify the relationship between applied load and strain at low strain values. For the affected section of track bed, high modulus polyester woven geotextile with an ultimate tensile strength of 1000 kN/m in the main direction was selected as the reinforcement. The results of specific tensile tests carried out by the Manufacturer showed that the material would experience a strain of 1% at an applied load of 80 kN/m, which value was used for design. Collection and control of upward groundwater seepage was provided by installation of a "Drain Tube" layer beneath the lowest sheet of reinforcement; this composite comprises 2 layers of non-woven geotextile with included drain pipes set at 0.25 m horizontal spacings. The drain pipes were laid perpendicular to the track and discharged to the north side ditch. The adopted track bed design is shown in Figure 5 and is described below:

- subgrade
- 100 mm depth of 20 mm down crushed limestone
- "Drain Tube" set perpendicular to rail track
- Woven geotextile with main direction parallel to rail tracks
- 125 mm depth of 20 mm down crushed limestone
- Woven geotextile with main direction perpendicular to rail track
- 125 mm depth of 20 mm down crushed limestone
- Woven geotextile set parallel to rail tracks
- 125 mm crushed limestone
- 200 mm ballast

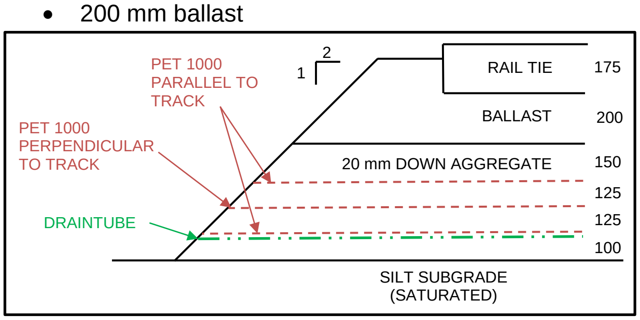

Figure 5. Track Bed Reinforcement Design 2014

Settlement of the rail tracks supported by the reconstructed track bed continued to occur at a decreasing rate for a period of one to two months as the geotextile reinforcement tensioned in response to settlement. After that time, the development of tension in the geotextile sheets resulted in track maintenance becoming a normal condition.

6 AQUIFER DEPRESSURIZATION (2016 TO 2019)

Breakouts of groundwater continued to occur outside the footprint of the reinforced track bed at the Mile 9.1 section subsequent to implementation of both the phases of reinforced track bed construction. One breakout which occurred in the devil strip was particularly severe and did not diminish with time; this was sufficient to affect serviceability of both north and south rail tracks. To mitigate this situation, the Geotechnical Team proposed that a design for depressurization of the underlying layered sand deposit which contained the local high water pressures be implemented. A conceptual design involving installation of a set of wells situated as close as possible to the north rail track was put forward. In order to advance the conceptual design to construction, the Geotechnical Team was complemented with the addition of hydrogeological design and numerical modelling expertise.

The design study commenced with a review of groundwater pressures measured in existing functioning monitoring wells and from records of prior groundwater pressure measurements in now dysfunctional piezometers. Using those data, preliminary contours of phreatic pressure head were developed and plotted. In order to progress the design of the depressurization scheme, improved characterization of the variation of pressure in each soil unit and determination of pressure gradient was required. Thus, additional piezometers were installed in the near surface silt aquitard layer, within the upper horizons of the underlying layered sand stratum and at greater depth within that deposit. Data were also available from a deep well in the vicinity of the site which is operated by the local Regional Municipality. Drive point pneumatic piezometers were selected for installation in the base of the rail cutting in order that precautionary measures with regard to control of high above groundwater pressures in the course of installation were not required. The pneumatic piezometers were positioned on the north side of the north track, in the devil strip between the north and south tracks, and on the south side of the south track. The measurements provided useful data with regard to prevailing pressures at relevant horizons in the soil profile and the pressure gradient.

In order to determine the aquifer characteristics of the underlying layered sand stratum, a test well was constructed. To reduce the impact of complexities associated with the installation of a test well in a stratum characterized by high above ground water pressures, a 2 m high granular pad was constructed on the north side of the north track to provide an elevated platform, in the vicinity of the proposed depressurizing wells. Through the adoption of an installation system involving a succession of telescoping casing to support the sides of the well boring, each successive section of casing was installed and sealed to prevent side leakage before proceeding to the next, smaller diameter casing. Heavy-weight drilling muds were used to control the water pressures in the casing throughout the installation process. A 200 mm diameter by 4.5 m long well screen was installed below the riser in the depth range from about 6.5 to 11 m below the base of the rail cutting for the pumping test.

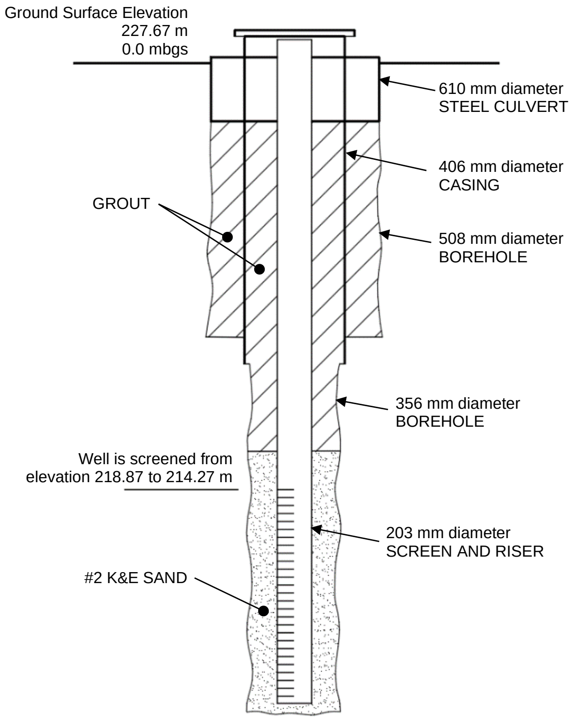

Figure 6. Construction detail of test well

Following completion of the test well, a pumping test was carried out to evaluate the hydraulic characteristics of the layered sand stratum which underlies the silt aquitard and to record the effect on the phreatic surface in each affected soil layer. The data from the well test and pressure readings taken in the array of piezometers as well as monitoring of the domestic wells situated in the vicinity of the site were input to the hydraulic model to enable design of a depressurization system to proceed. Based on the interpreted hydraulic characteristics of the layered sand deposit and the silt aquitard, it was determined that a set of five wells installed parallel to the north track installed at 5 m horizontal spacings in the affected section of track would provide adequate depressurization. Each of the wells was installed at a 6 m offset from the centreline of the north rail track. The length of well screen is 4.5 m in each well, the depth to the tip of the screen lies at a depth of approximately 11 m below the base of cutting.

Following placing the relief wells in service, it was found that the wells could operate passively to lower the phreatic surface to below the base of the cutting.

Maintaining a phreatic surface at that level eliminates the propensity for internal erosion and piping to develop in the soil profile beneath the rail track bed.

The depressurization system was put into service in the Fall of 2018. It has operated satisfactorily since that time with no subsequent breakouts of groundwater nor any requirement for rail track maintenance beyond normal conditions.

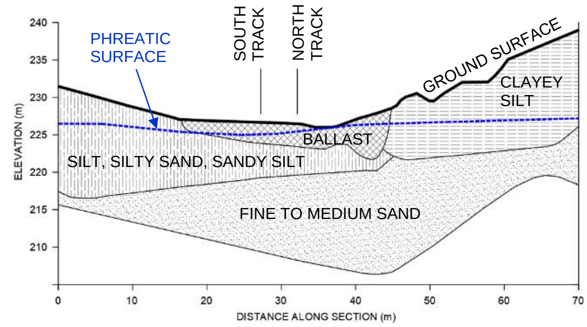

Figure 7. Phreatic Surface Following Depressurization

7 TRACK PERFORMANCE

The initial design premise to provide a temporary solution to very severe settlement impacts on train movements at the two original problem locations was effected quickly and at low cost (less than $200,000, including contractor, construction materials, and engineering design and field inspection costs). The anticipated design life of that system was set at five years. Implementation of that design enabled the zone speed for train movements to be increased from 5 mph to 30 mph within a period of a few weeks. The full-time watchman was removed, detailed track inspections were reduced initially to a single daily inspection and then further reduced.

Prior to construction of the 1996 track reinforcement sections, the annual costs associated with a full-time watchman, together with fuel costs resulting from slowing down and speeding up trains amounted to more than $500,000 per year, excluding the operational costs of slower rail service. The track reinforcement at Mile 9.1 remained effective for more than 23 years prior to the permanent solution involving depressurization being implemented. The track reinforcement at Mile 9.0 has remained effective and has not been supplemented; there have been no breakouts of groundwater adjacent to that section of rail track. At Mile 9.0 a track monitoring and warning system was attached to the rail ties which can detect any movement significant to rail movement and which is connected to an alarm in the Railway Dispatch Office as a precautionary measure.

The track bed reinforcement system which was constructed as an easterly extension to the reinforced section at Mile 9.1 in 2013 resulted in satisfactory control of track movements.

Reconstruction of rail track beds must be carried out within a “Block” that is of a few to several hours’ duration. The actions that must be completed within that short time frame are:

- Cut and remove affected section of track
- remove existing rail track bed
- profile subgrade
- reconstruct sub-ballast layer, including all reinforcement layers
- lay ballast layer
- relay and level track.

Time pressures preclude desirable construction practices such as dense compaction of each lift of granular materials in the sub-ballast layer and tensioning of the reinforcement. Dense compaction of the newly laid sub-ballast layer occurs as a result of loading by train movements, tensioning of reinforcement occurs as a result of settlement of the reconstructed track bed. Hence, settlement of the rail track is to be anticipated in the period following construction. In the situations of both the 1996 and the 2014 track bed reinforcement, much of the track bed settlement was complete within a few weeks of reconstruction. Track maintenance requirements became normal within a few months after reconstruction as tensioning of the geosynthetic reinforcement occurred.

The results of monitoring instrumentation installed in the track bed using a load cell show that inclusion of a geocell component in the track bed design reduces loads on the cell commensurate with an increase in equivalent thickness of the overlying granular materials, as anticipated from prior laboratory studies. Measurements taken in load cells positioned lower in the reinforced track bed show that short term load application consisting of a stationary work train parked above the load cells did not have a measurable increase in applied load at those positions within the reinforcement.

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The load cell and settlement monitoring devices installed beneath the first phase of track reconstruction were provided by the geosynthetic manufacturer, Tencate Mirafi.

Hydrogeological analysis of the site situation and numerical modelling of the effects of passive depressurization and the effects of pumping on groundwater pressures were carried out by Gunther Funk, P.Geo. and Chris Neville, P.Eng., respectively.

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