



# Improvements in global stability, bearing capacity, and settlement control using the Rammed Aggregate Pier® System

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## ABSTRACT:

Mechanically Stabilized Earth (MSE) retaining walls, concrete retaining walls, and earthen embankments impart high shearing stress in the underlying soil. When constructed over weak foundation soil, slope instability and bearing capacity failures can occur. Rammed Aggregate Pier® (RAP) elements were used on the One Mile House project in Saint John, New Brunswick, to increase factors of safety against global instability and bearing capacity failure and to control settlement magnitude and reduce the time required for settlement to occur.

## RESUME :

Les murs de soutènement en terre mécaniquement stabilisée (*MSE walls*), les murs de soutènement en béton et les remblais en terre peuvent induire des contraintes de cisaillement élevées dans les sols de fondation. Lorsque de tels structures sont construites sur des sols de fondation de faible résistance, ceci peut conduire à la rupture soit par instabilités de pentes ou par une capacité portante insuffisante. Les *Rammed Aggregates Pier (RAP)* ont été utilisés pour le projet One Mile House à St-John, au Nouveau Brunswick, afin d'augmenter les facteurs de sécurité contre la rupture reliée à des instabilités globales ou reliée à la capacité portante. Aussi, cette technique est a été utilisée pour contrôler la magnitude du tassement ainsi que pour réduire le temps nécessaire à ce même tassement.

## 1 INTRODUCTION

This paper describes the One Mile House Interchange Case History and discusses how Rammed Aggregate Pier® (RAP) elements were used to solve global stability, bearing capacity, and settlement issues on this project in Saint John, New Brunswick. It also details the construction and design methodology and provides field test results.

## 2 CASE HISTORY: ONE MILE HOUSE INTERCHANGE EMBANKMENT AND MSE WALL SUPPORT

The One Mile Interchange project will connect the Route 1 to Bayside Drive in Saint John, New Brunswick. The project includes the construction of an eastbound on-ramp to access Route 1 using reinforced slopes and construction of MSE walls to facilitate grade changes on the east approach to the interchange structure.

During the geotechnical exploration and design phase, problems with global stability on the Ramp 3 roadway widening portion of the project presented issues. Additionally bearing capacity and settlement control for the Main Lanes and Ramp 6 MSE walls were identified as a concern. Figure 1 illustrates the areas of importance.

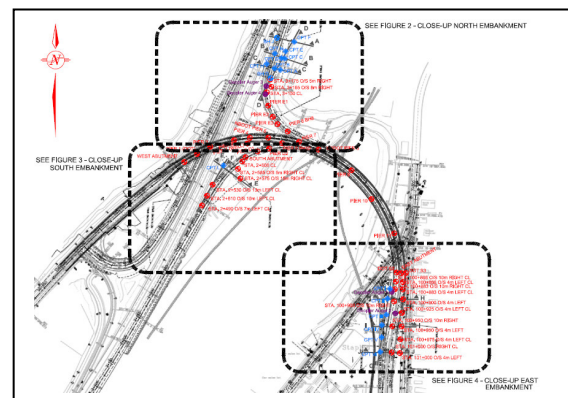


Figure 1: Areas of Concern (Courtesy of Gemtec Limited)

## 3 RAMP 3

Access to Route 1 eastbound from the One Mile House structure is provided by Ramp 3. The approach embankment is approximately 3 meters higher than the existing highway grade and approximately 9 meters higher than the existing ground on the Marsh Creek side of the ramp. Because of the close proximity of the ramp alignment to Marsh Creek, the new embankment was designed with a reinforced slope of 1:1 (H:V).

Figure 2 illustrates the embankment geometry and soil profile with CPT soundings.

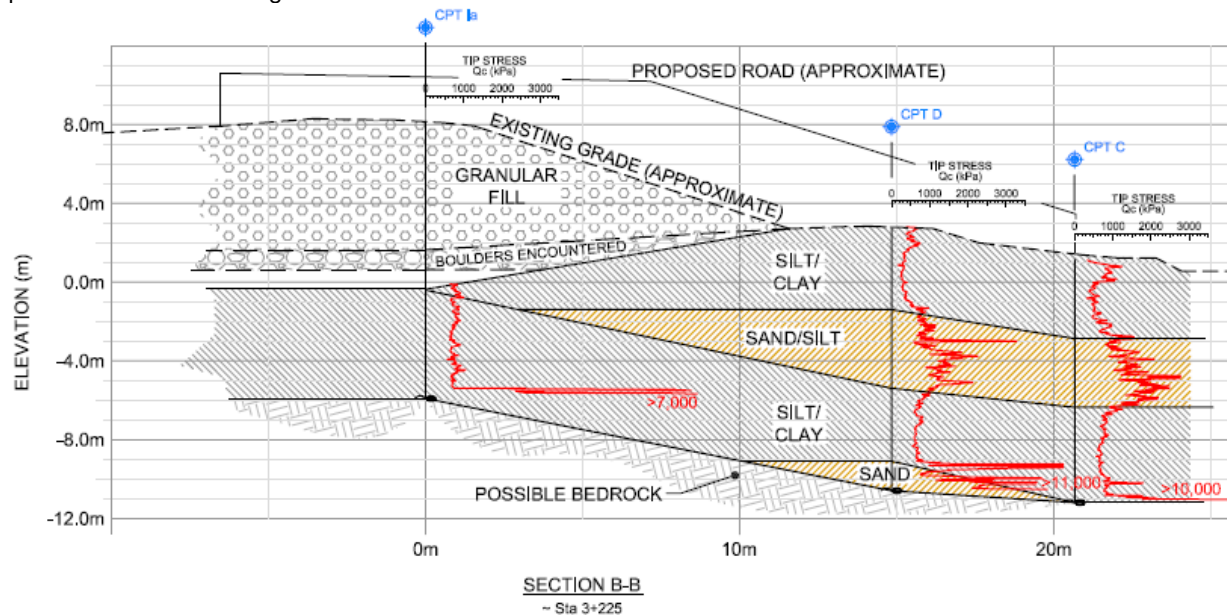


Figure 2: Ramp 3 Embankment Geometry and Soil Profile (Courtesy of Gemtec Limited)

Soil Type	Unit Weight (kN/m <sup>3</sup> )	<i>c</i> (kPa)	$\phi$ (deg)	<i>c'</i> (kPa)	$\phi'$ (deg)
Reinforced Slope	21.5	100	36	100	36
Embankment Fill	21.5	-	36	-	36
Existing Fill	21.5	-	35	-	35
Upper Silt & Clay (crust)	17	17-41*	-	-	28
Upper Silt & Clay	17	40	-	-	28
Upper Sand	19.5	-	35	-	35
Lower Silt & Clay	17	40	-	-	28
Lower Sand	19.5	-	32	-	32
Weathered Bedrock	23.5	250	45	250	45

\* 25 kPa at STA 3+200, 17 kPa at STA 3+225, 22-41 (linear increase) at STA 3+240

Table 1: Ramp 3 Soil Parameter Values

The foundation soil below this new embankment widening consisted of up to 4 meters of soft silt and clay underlain by 2 to 5 meters of sand, followed by medium stiff to stiff silt and clay that ranged from 0 to approximately 7 meters thick. The bedrock sloped to the east towards Marsh Creek.

CPT testing was performed beneath the existing embankment and within the proposed embankment footprint. Comparisons between soil that has been consolidated by the existing embankment and soil adjacent to Marsh Creek were used to select soil parameter values for design. The undrained shear strength was determined by the geotechnical engineer using correlations with CPT tests and laboratory tests, and generally ranged from less than 5 kPa to more than 50 kPa in the silt and clay layers. Friction angles in the sand layer ranged from approximately 27 degrees to over 40 degrees, but generally ranged from 32 degrees to 35 degrees. Consolidation testing indicated that the material was slightly overconsolidated near the surface but was approaching normal consolidation with increasing depth. Table 1 indicates the range of soil parameter values used for the global stability analysis.

### 3.1 GLOBAL STABILITY

Stability analyses were performed to evaluate the proposed embankment using the GeoStudio 2007 software package. The finite element stress-deformation program SIGMA/W was used to establish the initial stress state in the soil profile, and the SLOPE/W slope stability program was used to perform the global stability analysis. The finite element stability analysis was checked using traditional circular failure surface searches using the

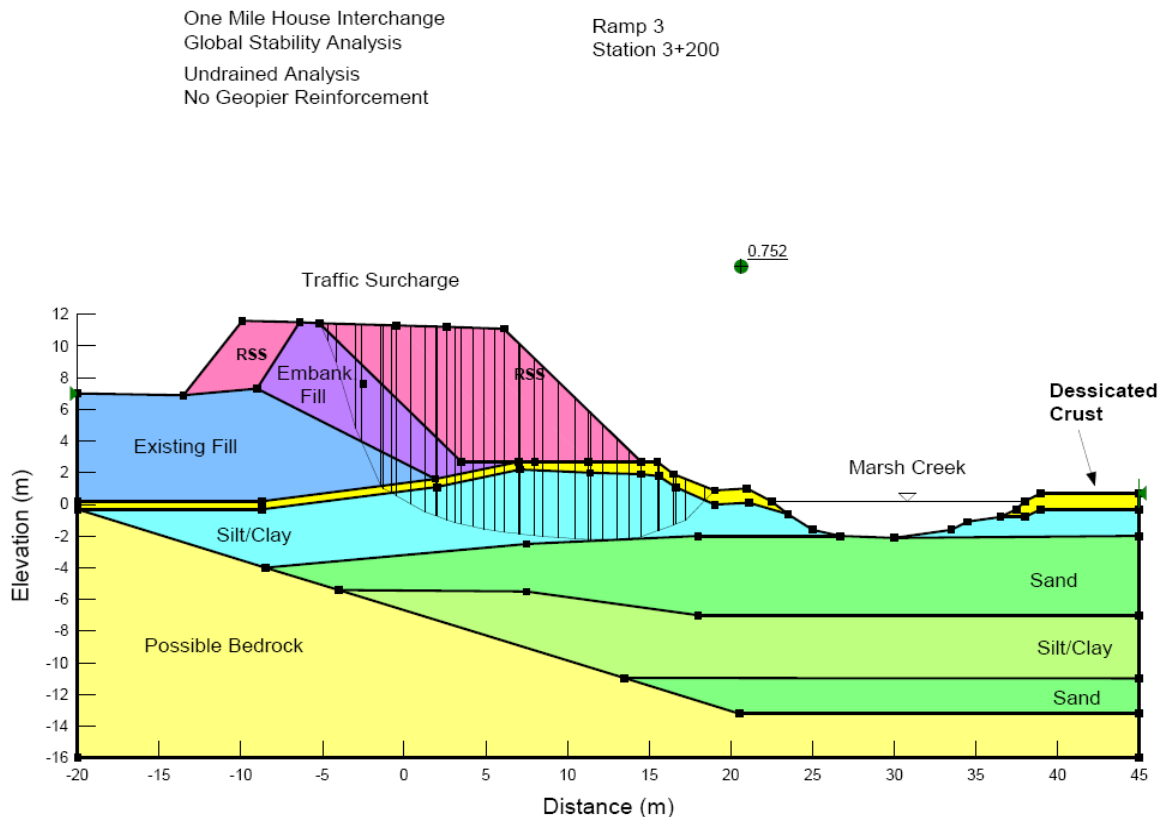


Figure 3: Unreinforced slope stability results (Courtesy of ADI Limited)

Morganstern-Price stability model in the SLOPE/W program.

The 1:1 embankment side slope requires reinforcement to create the steep slope angle. A reinforcement length of 10.7 meters (35 feet) was selected for design and was modeled as a block of soil with high shear strength.

The undrained analysis performed between station 3 + 185m and station 3 + 245m (Figure 3) indicates that the slope is unstable without reinforcement. Construction in multiple stages was considered but did not meet the schedule requirements of the Department of Transportation. Because of the close proximity to Marsh Creek and the need to keep the existing Route 1 northbound lanes serviceable, overexcavation and replacement of soft soil was not practical. The Rammed Aggregate Pier solution was selected based on the ability to work in the tight work area, the speed of construction, and cost.

#### 4 RAP CONSTRUCTION

The construction of a Rammed Aggregate Pier solution using the Geopier system is well-described in the literature and shown in Figure 4 (Lawton and Fox 1994, Lawton et al. 1994). The piers are installed by drilling a 760 mm (30 inch) diameter hole to depths ranging between 2 m and 8 m (7 feet and 26 feet) below working grade elevations (Figure 4, Panel 1), placing controlled lifts of stone within the cavities, and compacting the

aggregate using a specially designed high-energy beveled impact tamper. The first lift consists of clear stone and is rammed into the soil to form a bottom bulb below the excavated shaft (Figure 4, Panel 2). The bottom bulb effectively extends the design length of the aggregate pier element by approximately one pier diameter. The piers are completed by placing consecutive 0.3 m (one-foot) thick lifts of aggregate over the bottom bulb and densifying the aggregate with the beveled tamper (Figure 4, Panel 3). During densification, the ramming forces stone downward while the beveled shape of the tamper forces stone laterally into the sidewall of the excavated cavity. This action increases the lateral stress in the matrix soil thus providing additional stiffening and increased normal stress perpendicular to the perimeter shearing surface.

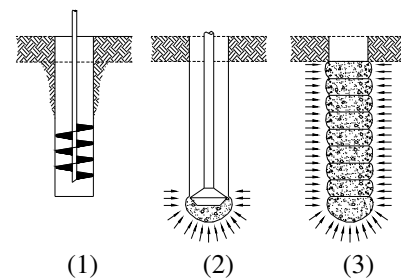


Figure 4: Rammed aggregate pier construction process

In addition to increasing the composite stiffness of the reinforced area, the densely compacted RAP elements exhibit high friction angles of approximately 50 degrees (White, et al.), thereby affording large increases in shear resistance.

The Rammed Aggregate Pier solution along Ramp 3 consisted of installing elements on a centre to centre spacing of 1.22 meters (4 feet) under most of the embankment to provide a zone of increased shear resistance beneath the proposed reinforced slope (Figure 5). Near Station 3 + 240m, the embankment height and higher in-situ undrained shear strength values afforded a centre to centre spacing of 1.83 m (6 feet). Critical failure surfaces intersect the reinforced zone and either pass through the reinforced zone, or are driven deeper below the reinforced zone through the sand layer and stronger clay layer. The result is higher factors of safety for global stability.

#### 4.1 COMPOSITE SHEAR STRENGTH PARAMETERS

The composite shear strength parameter values of RAP reinforced soils are computed using the conventional method of calculating the weighted average of the shear strength components of the aggregate piers and matrix soil materials (FHWA 1999). The composite cohesion intercept ( $c_{comp}$ ) is computed with the expression:

$$c_{comp} = c_g R_a + c_m (1 - R_a) \quad [1]$$

where  $c_g$  is the cohesion intercept of the aggregate,  $c_m$  is the cohesion intercept of the matrix soils, and  $R_a$  is the ratio of the sum of the element cross-sectional areas to the gross footprint area of the reinforced soil zone. Because the cohesion intercept of the aggregate is zero, Equation 1 reduces to:

$$c_{comp} = c_m (1 - R_a) \quad [2]$$

The composite friction angle ( $\phi_{comp}$ ) is computed with the expression:

$$\phi_{comp} = \tan^{-1} [R_a \tan \phi_g + (1 - R_a) \tan \phi_m] \quad [3]$$

where  $\phi_g$  is the friction angle of the aggregate and  $\phi_m$  is the friction angle of the matrix soils. The composite cohesion and friction angle values (Equations 2 and 3) are used to represent the composite shear strength of the soil layers reinforced by the Rammed Aggregate Pier elements.

Because the stiffness of RAP elements is 4 to 40 times greater than the matrix soil into which they are

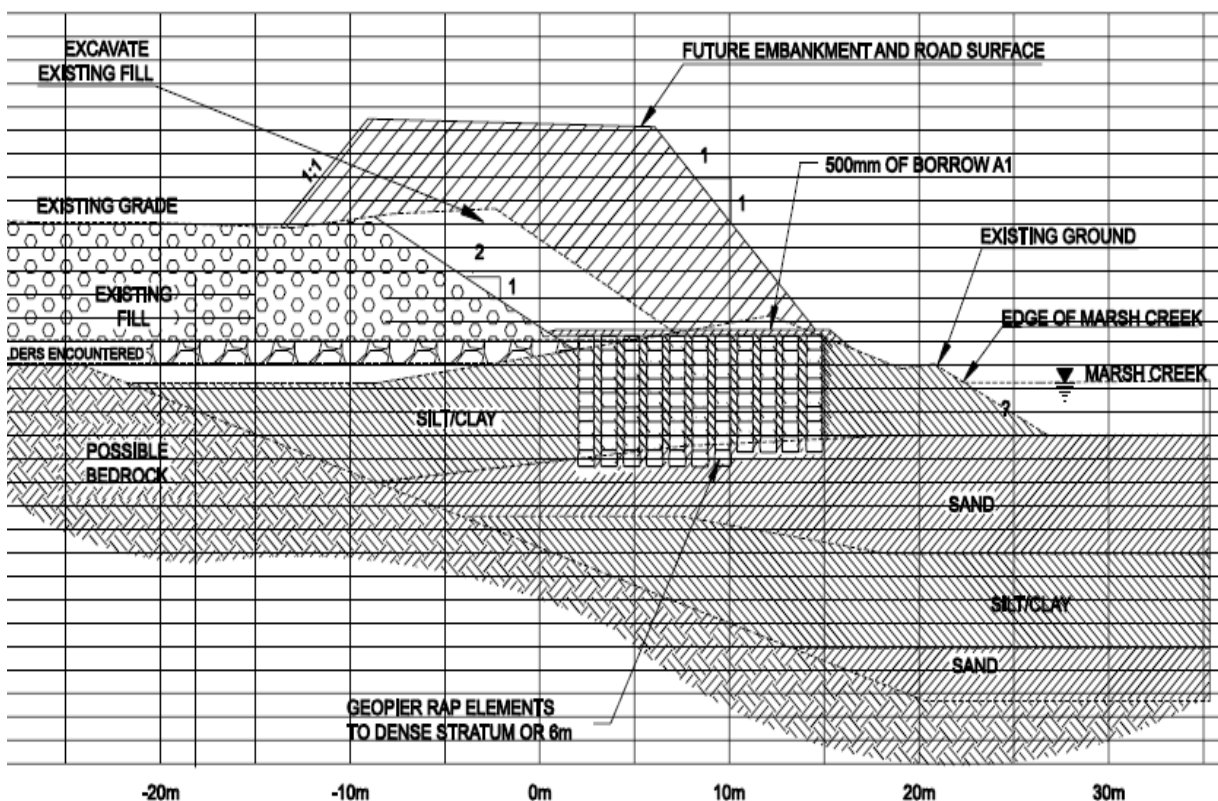


Figure 5: RAP Solution at Ramp 3 (Courtesy of Gemtec Limited)

installed, stress from the embankment is concentrated to the stiff piers. The increased stress on the piers results in an increased composite shear strength from increased pressure between particle to particle contacts within the element (Mitchell, 1981). When stress concentration occurs, the composite shear strength parameters are computed based on the same weighted average approach described above, but also incorporates factors for stress concentration (FitzPatrick and Wissmann 2002):

$$c'_{comp} = \left[ \frac{1}{R_a R_s - R_a + 1} \right] (1 - R_a) c'_m \quad [4]$$

$$\phi'_{comp} = \arctan \left[ \left( \frac{R_s}{R_a R_s - R_a + 1} \right) R_a \tan R_a \tan \phi'_g \right] + \left[ \left( \frac{1}{R_a R_s - R_a + 1} \right) (1 - R_a) \tan \phi'_m \right] \quad [5]$$

where  $R_s$  is the ratio of the RAP element stiffness to the matrix soil stiffness. Research shows that typical stiffness ratio values for reinforced embankment fill range from 5 to 9 in the upper portion of the pier (Highway Innovative Technology Evaluation Center, 2007). Because stress within the pier dissipates with depth, design values of stiffness ratio must be selected with care. For the Ramp 3 stability analysis, a stiffness ratio value of 2 was selected.

For the Ramp 3 embankment, the composite cohesion intercept varied according to the selected undrained shear strength values selected for each station. Using Equation 4, the composite cohesion intercept values along the alignment ranged from 9 kPa to 15 kPa for piers spaced at 1.22 meters (4 feet) on center, and 21 kPa for Rammed Aggregate Pier elements spaced at 1.83 (6 feet) on center. Equation 5 was used to compute the composite friction angle value to be used for analysis. For the 1.22m center to center spacing, composite friction angles of 27.5 degrees and 38 degrees were used for the undrained and drained analysis, respectively. For the 1.83m center to center spacing, composite friction angles of 15 and 32.5 degrees were used for the undrained and drained analysis.

The results of the undrained stability analysis (Figure 6) indicated that the Rammed Aggregate Pier design

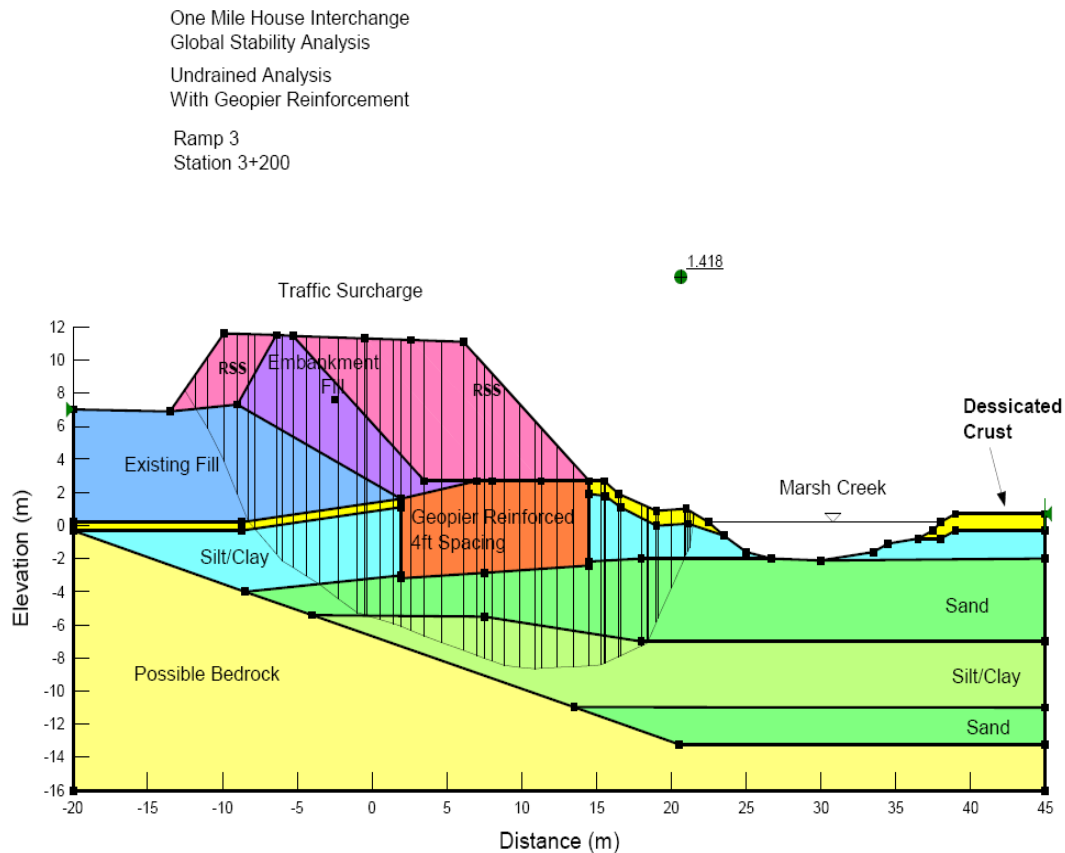


Figure 6: RAP Reinforced Stability Analysis for Ramp 3 (Courtesy of ADI Limited)



increased the safety factor to values greater than the design criterion of 1.3 at Station 3 + 200m.

## 5 EAST APPROACH AND RAMP 6

The east approach to the One Mile House Interchange structure consists of two opposing mechanically stabilized earth (MSE) walls. The MSE wall heights increased from approximately 4 meters to 8.5 meters between station 100 + 848m and station 100 + 970 m along the Main Lanes. The reinforcement length extending behind the MSE wall panels was equal to the wall height.

Ramp 6 provides access from Rothesay Avenue to Bayside Drive. The MSE wall heights range from 2.5 to 3 meters along the Ramp 6 alignment.

The soil profile in the Main Lanes and Ramp 6 area generally consisted of a thin layer of surficial granular fill underlain by 4 to 8 meters of soft silt and clay, underlain by a thin veneer of glacial till underlain by bedrock. The depth to rock generally increased as the wall height increased along the alignment.

Global stability analysis was performed for the Main Lanes and Ramp 6 MSE walls using the same techniques described for the Ramp 3 alignment. The results indicated factors of safety less than 1.0 for the Main Lanes for unreinforced conditions, suggesting global stability failure if the MSE was constructed without ground improvement. The results for RAP reinforced foundation soil below the MSE walls indicated factors of safety greater than the project requirements.

The tall walls at Main Lanes and Ramp 6 impart substantial bearing pressures on the foundation soils. Lateral earth pressure and overturning forces cause the bearing pressure near the face of the MSE walls to exceed the allowable bearing capacity of the soft silt and clay. Ground improvement was required in order to support the proposed wall heights. RAP elements were selected to increase the allowable bearing pressure below the MSE walls based on construction schedule and cost.

Traditional bearing capacity approaches are used to evaluate the improvement in the allowable bearing pressures. However, this approach considers the influence of the composite zone as described in Barksdale and Bachus (1983), with a slight modification suggested by Hall et al. (2002). The installation of the RAP elements typically improves the allowable bearing pressure by a factor of 2 to 3 compared to the unreinforced case.

The design solution to satisfy both global stability and bearing capacity requirements consisted of RAP elements at a centre to centre spacing of 1.22m (4 feet) beneath the MSE wall panels and reinforcement, and a center to center spacing of 1.83 m (6 feet) beneath the center of the Main Lanes grade separation between the opposing reinforced zones. An additional row of piers was included outside the face of the MSE wall, as well. The results of the bearing capacity analysis indicated factors of safety equal to or greater than 2.0 for the Rammed Aggregate Pier reinforced soil.

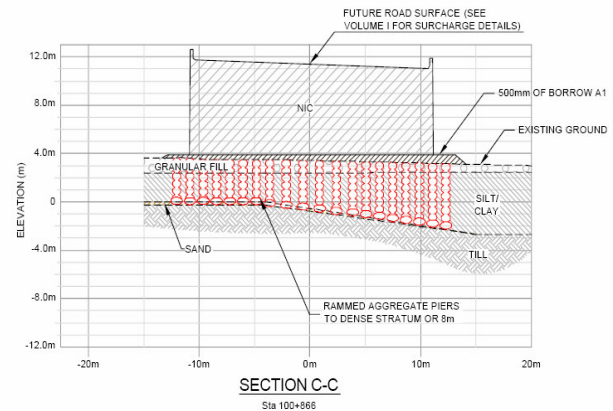


Figure 7: Main Lanes RAP Design Solution (Courtesy of Gemtec Limited)

### 5.1 SETTLEMENT CONTROL

Unreinforced settlement estimates for the Main Lanes were on the order of 1 to 1.2 meters. Differential settlement estimates along the alignment were more than half that amount because the thickness of the compressible layer increased as the embankment height increased. The project design team was faced not only with large total and differential settlements, but also with slow consolidation rates. Estimates to reach 90% consolidation were nearly 1.5 years. Rammed aggregate pier elements were used to bring total settlements and time for consolidation well within the project limits.

The RAP elements control settlements by improving the composite stiffness characteristics of the reinforced zone and by acting as drainage elements to shorten radial drainage pathways (Han and Ye, 2001). The settlement control design methodology is based on a two-layer settlement approach is described by Lawton and Fox (1994). The installation process creates a stiffened reinforced zone (called the upper zone) with reduced compressibility that controls settlement of embankments and MSE walls. The settlement below the aggregate pier-reinforced zone, referred to as the lower-zone, is evaluated using conventional geotechnical analysis approaches. The total settlement of the transportation structures is evaluated as the sum of the upper zone settlement and the lower zone settlement. In the case of the Main Lanes and Ramp 6 MSE walls, because the piers extend down to the very stiff glacial till layer which is underlain by bedrock, the lower zone settlement is negligible.

Settlement in the upper zone is related to the average pressure applied by the MSE wall, the cross-sectional area replaced by RAP elements, the stress concentration ratio between the RAP elements and the matrix soil, and the stiffness of the Rammed Aggregate Pier (FitzPatrick and Wissmann 2003). The stress applied to the top of the pier is computed by:

$$q_g = q \left( \frac{R_s}{R_s R_a - R_a + 1} \right) \quad [6]$$

where  $q$  is the average applied bearing pressure,  $R_s$  is the stress concentration ratio,  $R_a$  is the area replacement ratio.

The bearing pressure is computed through normal MSE design approaches while the area ratio is determined by spacing of piers and geometry. The stress ratio between the Rammed Aggregate Pier element and the matrix soil is selected based on experience, but generally varies between 5 and 9 for reinforced soil.

Settlement of the reinforced upper zone is related to the top of pier stress and pier stiffness by:

$$S_{uz} = \frac{q_g}{k_g} \quad [7]$$

where  $k_g$  is the RAP stiffness modulus. Settlement control is achieved by limiting the top of pier stress, often by adding piers or using tighter pier spacing.

A conservative estimate on the anticipated RAP stiffness modulus value is selected based on similar results from piers installed in similar soils. A database of over 2000 modulus tests is used to select an appropriate pier stiffness value. A value of 25 MPa/m was selected for design. Based on these upper zone parameter values and incorporating the negligible lower zone settlements, total settlement estimates for the MSE walls supported on Rammed Aggregate Pier reinforced soil were 1 to 2 cm.

To verify the selected pier stiffness, a full scale stiffness test (called a modulus tests) is performed on Rammed Aggregate Piers at the site. The modulus test evaluates the stress-deflection behavior. The setup is similar to a pile load test, however, the goal is to measure the stiffness of the element and evaluate its deflection characteristics rather than test an ultimate capacity. On the One Mile House project, one test was performed at the Ramp 3 site and one was performed at the Main Lanes site. The results for both sites were nearly identical, and indicated a RAP element stiffness value of 45.9 MPa/m (169 pci) at the top-of-pier design stress of approximately 355 kPa (7.4 ksf). This value was nearly twice the stiffness selected for design.

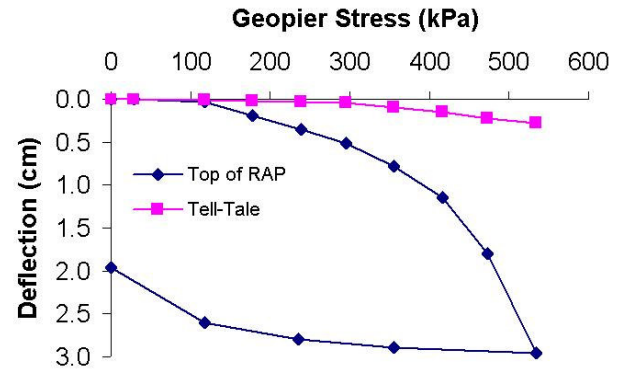


Figure 8: Rammed Aggregate Pier Modulus test results

## 5.2 Settlement Rate

The long delay period for unreinforced soil did not meet the project schedule. The radial drainage afforded by using clear stone to build the RAP elements substantially reduces the time for consolidation. The stiff elements also attract stress, which reduces the stress applied to the matrix soil and therefore reduces the total amount of consolidation that needs to occur. The design approach uses horizontal consolidation rates, horizontal drainage path lengths from pier spacings, and the stress concentration ratio to estimate the rate of consolidation based on a modified time factor for radial flow (Han and Ye 2001).

At the close spacings used for bearing capacity and stability control, and using this approach, nearly 90 percent of the consolidation settlement within the reinforced zone was estimated to occur within a one to two week time period. When compared with the one to one and a half year consolidation rate estimated for the unreinforced case, the RAP elements provided a substantial time savings on the project.

## 6 CONCLUSIONS

The project team was faced with very weak soil, aggressive construction schedules, steep slopes and tall MSE walls. Traditional geotechnical options such as overexcavation or staged construction could not provide practical solutions that were cost effective or time sensitive to the project. By using a Rammed Aggregate Pier® solution incorporating the Geopier® system, the geotechnical engineer provided an economical solution that improved global stability, improved bearing capacity, controlled total and differential settlement, and increased the rate of consolidation to meet the project schedule. Site specific modulus tests confirmed the design and indicated an engineering response nearly twice the design value.

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