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Performance of Geogrid Reinforced Embankment Slopes Under Flooding and Scour: A case study

Ram KASTURI[a,1], Ajay SHASTRI[b] and Ram TIRUMALA[a]
[a]*Distinct Engineering Solutions*
[b]*Geosyntec Consultants*

Abstract. Design and construction of roadway embankments in scour critical and flood-prone areas poses a significant challenge. The risks and challenges are compounded further if the embankment needs to be constructed on top of highly compressible sub-surface soils. This case study describes the engineering design of one such bridge located on Rt. 209 crossing over Rondout Creek in Accord, NY. The existing bridge was only 11.3 m. long and was identified as a hydraulically deficient bridge by New York State Department of Transportation and with a scour depth of greater than 6 m. A 122 m long bridge with a 4.6 m high MSE (Mechanically Stabilized Earth) embankment was proposed with sufficient hydraulic opening as a replacement. To provide a stable foundation for the proposed high embankment, a ground improvement design was developed using geotextile in addition to protection of embankment side slopes with controlled rip-rap placement. The proposed ground stabilization for the embankment was analyzed for slope stability under various flooding conditions on both sides of the bridge and incorporated construction staging requirements.

Keywords. Slope stability analysis, geogrid reinforcement, scour.

1. Introduction

New York State is home to more than 17,000 Bridges with more than 50% controlled by the state transportation body the New York State Department of Transportation (NYSDOT) [1]. The NYSDOT is responsible for inspecting as part of their bridge inspection program where all elements of bridge from the bridge deck, supports to foundation elements are checked. Hurricane Sandy also termed Superstorm Sandy brought forth the deficiencies of several key infrastructure located in the region. Identified as part the bridge inspection program was the Critical Bridges over Water (CBOW) deemed to be hydraulically deficient structures. These bridges were prone to flooding and were effects of scour [2]. The Federal Emergency Management Agency (FEMA) developed the Hazard Mitigation Grant Program to rectify and enhance the resiliency of the bridges that are deemed scour critical/flood prone.

The bridge over Route 209 Bridge located in Ulster County, New York was identified as one such hydraulically deficient structure. Figure 1a) shows the location

[1] Ram Kastruri, P.E., Vice President, Distinct Engineering Solutions, 25 Old Georges Rd, North Brunswick Township, NJ 08902, rkasturi@distinctengineering.com

map of the bridge. The bridge was a single span 11.3 m structure constructed in 1927 [3]. The Roundout Creek runs parallel to the length of the roadway, however at the location of the bridge a section of the creek branches off and runs perpendicular to the road necessitating the bridge. Due to the hydraulic limitations, the bridge and the adjacent roadway is subjected to significant flooding during rainy seasons. In addition, the roadway embankment along the Rondout Creek is subjected to erosion specifically on the south side of the roadway. Figure 1b) shows the upstream side of bridge prior to replacement. The state awarded the replacement of the bridge as a design-build project to the team of Distinct Engineering, KC Engineering and ECCO III where the awardees would both design and construct the bridge to satisfaction of FEMA and the NYSDOT. The new bridge would have to span 122 m in length, with an increase in elevation approximately 4.6 m from existing grade. The bridge would also have an expanded roadway increasing the number of lanes from two to four.

This paper describes the slope stability analysis in light of challenges brought forth due to scour, flooding, increase in elevation and expansion of the roadway. Sections 2 describes the findings of the geotechnical borings and the scour analysis. Section 3 covers the design of the bridge addressing slope stability while addressing concerns of both flooding and scour.

a) b)

Figure 1. a) Location Map of Bridge b) Upstream side of bridge prior to construction.

The bridge approach roadways were to be reconstructed to minimize flooding with roadway grade was raised by approximately 4.6 m to counteract the 100-year flood.

2. Geotechnical investigation

2.1. Site investigations

The field investigation program consisted of drilling 16 borings. The depth of bridge borings varied from 15 m to 25 m. Rock was encountered in all 6 bridge borings. Rock coring was obtained from 4 of these borings, one at each structure. A minimum of two (2) borings adjacent to each of the two (2) piers and two (2) abutments.

All the borings indicated surficial soil layers consisting of loose to medium dense sand mixed with silt and gravel to a depth of 5.5 m. Below the surficial soil layers, a predominantly silty soil layer was observed. The thickness of this silty soil layer varied from approximately 3 m. Silty clay layer was observed below the silt layer. The thickness of this layer varied from 2.3 m to 6z m. Some of the borings indicated presence of approximately 1.5 m thick sandy gravel soil layer below the silty soil layer. Bedrock was observed in all the bridge borings. The depth to bedrock was at approximately between 15 m to 22 m.

2.2. Scour and flooding analysis

The design required critical cases of flooding and scour to be considered. Based on the design requirements set by the state, the bridges were designed for a 100-year storm event. The predicted rise in the water level from a low water level (NAVD-83 elevation of 217.5) to Flood water elevation (NAVD-83 elevation of 237.75) was 6.2 m. The high-water elevation was designated as 227.5 (increase by 3 m. from the low water) was used as the case for calculating the scour depths.

Contraction scour, Pier scour and Abutment scour were calculated based on the recommendation of the FHWA-HEC 23 which provides guidelines and governing equations for computing scour for highway bridges [4]. The velocity of water in the middle of the channel under the governing case was expected to 0.6 m/s under the high-water case. The analysis utilized a D50 value of 2.42 for the existing soil to compute the shear bed stress and the scour depth.

The contraction scour in the middle of the channel was calculated to be around 1.4 m, while that near the bridge piers was 2.2 m. The abutment scour was maximum at 6.4 m at the left bank. The huge amount of scour around the bridge abutments was a cause of concern as this could undermine slope stability of the abutments as well as endanger the foundation of the bridge both of which would lead to catastrophic loss of both life and property.

The new design had to therefore, consider the abutment slope to be engineered in such a way so as to counteract the effects the flood, as well withstand the scour without compromising either the roadway or the bridge.

3. Engineering slope stability analysis

3.1. Engineering challenges

The geotechnical engineering involvement in the design centered around two activities namely the design of the foundation and the slope stability of the abutments. The design of the roadway would be performed in conformance with the NYSDOT geotechnical design manual [3]. The existing roadway was to be elevated to a maximum of 4.6 m from existing grade. This would be achieved by gradually increasing the elevation from grade approximately 244 m away from the center of the bridge in either direction. The design was not allowed to impinge on any existing stream channels and the easement for construction activities were extremely restricted. As one leg of the Roundout stream travers parallel to length of the road (on the right), this implied that the elevation of the right abutment would produce a very steep slope. The proposed elevation changes on the existing roadway would be achieved by constructing an MSE wall on the right abutment of the roadway. On the left abutment the increased elevation required a smaller MSE wall 122 m on either side of the bridge. The remaining sections of the left abutment was allowed with a much gradual slope of 2H:1V in most sections as there were bodies of water nearby. Figure 2 shows the initial section close to the bridge for accommodating the increase in the elevation.

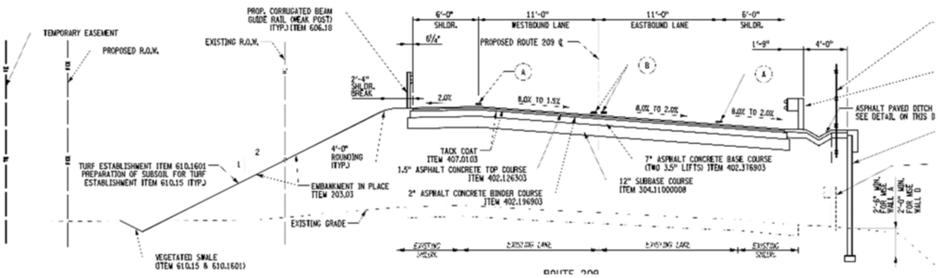

Figure 2. Initial section close to the bridge.

The design also had to contend with an increased highway live load on the bridge because of the change in the highway codes from when it was constructed. The asymmetrical slopes produced along the length of the bridge produced an increased loading on the right embankment. It was determined that the additional design loads due to the increased elevation, increased live load and the actions of the MSE wall compounded with scour effects would destabilize the existing slope. Additionally, due to environmental concerns the use of grout/concrete was not allowed on any material for the protection of the slope. A decision was made at this juncture to reengineer the right abutment slope and strength it against potential slope failures and the scour using locally sourced material which would be internally stabilized.

3.2. Design methodology

In order to meet the challenges of the project the design team considered several options in order to stabilize the right abutment.

To protect the existing soil from scour the bottom of the slope would be covered by a heavy stone fill. The heavy stone fill designated as Item spec 620.05 [5]) in the NYSDOT, is used specifically as rip rap. It was determined to withstand effects of shear stress produced under extreme loading and would be acquired from a local quarry. The stone fill would be built to have a shear key at the bottom approximately 1.2 m from existing grade. The shear key was added to provide additional stability to entire slope. The heavy stone fill was to be laid in layers in a 1:1 slope. The existing grade was excavated between 1.8 m to 2.7 m to accommodate the stone fill. Due to size and angularity of the stones, the slope here was determined to be safe from any local slope instabilities. In order to avoid loss of material below the fill the heavy stone fill was wrapped in a geotextile bedding. The upper part of the slope of the right abutment was covered with dry rip rap aggregate (Item 620.3-04 [5]) fill approximately 0.2 m thick to ensure slope stability during flood conditions. The rip-rap material was underlain by coarse aggregate and a geotextile bedding. For aesthetic purposes a 0.2 m thick topsoil with vegetation was added on top of the dry rip rap. In order to prevent an of the construction material from falling into the creek a turbidity curtain was installed along the length of the roadway.

During the course of the analysis it was determined that the existing material behind the stone fill and the rip on the slope had insufficient strength. This deficiency would undermine and destabilize the slope during flooding conditions. It was decided to incorporate geogrid reinforcement extending up to a maximum 12.2 m from the face of the rock of the stone fill. The extent of the geogrid reinforcement was shortened in the upper parts of the slope and to form a vertical line on the far side of the slope. The lengths and number of the geogrid reinforcements were appropriately reduced at sections away from the bridge where the increase in elevation and thereby the loads were not as severe. Two section of the roadway close to bridge are shown in Figures 3 and 4 to illustrate the final design implemented. It can be noted here that while the length of the geogrid reinforcement remains at 12.2 m the number of geogrid reinforcements reduce from 4 to 3 within the slope from Station 208 to Station 207.

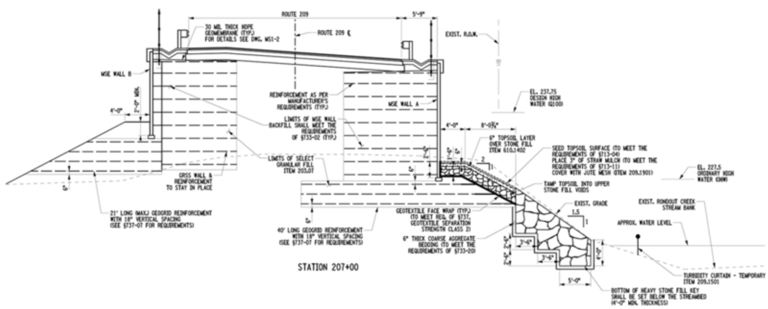

Figure 3. Roadway section at Station 207 (approximately 46 m. from center of ridge).

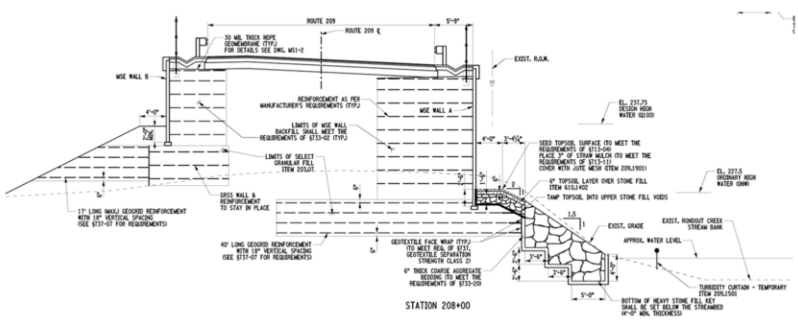

Figure 4. Roadway section at Station 208 (approximately 15.2 m from center of ridge).

The raising of the grade has an impact on left abutment also. Here the slope above the natural soil is filled with locally available material up to Elevation 237.75 (high water level) sloping back to the natural ground at 2H:1V. Depending the profile of the existing ground on the left the length of the reinforcement is extended to appropriate lengths to ensure slope stability. Here it can be seen that in Station 208 the geogrid reinforcement is extended 5.2 m from the face of the slope while in Station 207 it is extended 6.4 m from the face of the slope.

The MSE walls were designed separately by the company reinforced earth. Because the design is propriety in nature, the local slope stability of the MSE wall could not be conducted in the initial stage of the operation. Before the production of the construction drawings the analysis was conducted assuming that MSE wall supporting the embankment would be internally stable. A revision in the form of a compound analysis was undertaken after the detail design of the MSE wall. The compound analysis included the all aspects of the support structure from the strength of the MSE wall reinforcements to the properties of the geogrid reinforcement in checking for slope stability under various cases of flooding.

NYSDOT required the road to be operational during the demolition and construction of the new bridge. This always demanded that a minimum of one lane of the roadway to be open which was achieved by stage construction of the roadway. The right lane of the roadway was shutdown shifting all traffic on the left lane in engineer the right abutment and erect the MSE wall. After the construction of the right side, the left half was closed, and traffic was shifted. A Geosynthetic Reinforced Earth Systems (GRESS) wall was constructed and the slope was filled in front of it, in layers placing the geogrid reinforcements where required. After the construction of the slope the MSE wall was placed on top of the GRES wall. The addition of the GRES wall ensured that soil was reinforced continuously throughout the height of the slope improving the factor of safety of the slope. The GRES wall was eliminated in sections of the bridge which did not warrant an MSE wall on the left slope based on the elevation of the natural ground. Typically, these sections occurred at either end of the bridges.

3.3. Slope Stability analysis

The analysis of the slope stability was carried out in the software package SLIDE by Rockscience [6]. Slide is a 2D limit equilibrium slope stability program for determining safety factors or probability of failure, of circular or non-circular failure surfaces in soil or rock slopes [6]. Slide analyzes the stability of slip surfaces using vertical slice or non-vertical slice limit equilibrium methods. Slide also includes groundwater analysis modules to undertake steady state and transient seepage analysis. The program as inbuilt library of various methodologies for undertake slope stability analysis. For the current project Bishop's simplified and the Spencer method both of which are vertical slice methods were adopted. The Bishop's method is a force-based method wherein the Factor of safety is computed as a ratio of the sum of mobilizing forces to sum of the resisting forces. This methodology neglect interslice shear forces and satisfies only moment equilibrium. NYSDOT recommends the use of a minimum methodology to verify slope stability. However, the design team felt that in order to maintain the integrity of the design, it needed a 2nd method of analysis. The Janbu method was initially was considered for this purpose, however due to inherent conservative nature of the method precluded it use in favor of a more robust method like the Spencer method. The spencer method includes both normal and shear interslice forces and considers moment equilibrium. It is generally considered to provide a more accurate value of the Factor of Safety compared to other methods based on vertical slices.

Slide uses a grid pattern in searching for the most critical slip surface. A large search area was designated in order to obtain the most critical slip surface. The slope stability of the left and right embankments was analyzed separately with each having its own search areas. The minimum factor of safety was designated to be achieved was set to 1.3 for all loading conditions.

The geometry of each section of the roadway was analyzed. The discretization of the analysis was spread over station width of 30.5 m. The critical sections for slope stability formed around the bride where the increase in elevation of the roadway was the most. The list of soil and material properties used in the analysis are provided in Table 1. The properties of the reinforcing materials were obtained from the spec sheets [5]. Due to the stage construction of the project the Soft-Clay later would undergo consolidation during construction. The shear strength was computed to increase due to consolidation using the SHANSEP [7] methodology. The approximate increase in vertical effective stress over the clay layer is approximately 10%. The new shear strength $s_{u,new}$ is calculated as:

$$s_{u,new} = s_{u,original} * \left(\sigma'_{v,new}/\sigma'_{v,orignial}\right) = s_{u,original} * 1.10 \quad (1)$$

where, $s_{u,original}$ is the original shear strength, $\sigma'_{v,new}$ is the vertical effective stress after construction of the embankment and $\sigma'_{v,orignial}$ is the original vertical effective stress.

Table 1. Material and soil engineering properties.

Material Name	Unit Weight (kN/m^3)	Shear Strength (kPa)	Friction angle φ (°)
Medium Dense Sand	18.8	-	32.0
Loose Sand	18.1	-	28.0
Soft Clay	18.8	36.8	-
Medium Stiff Clay	19.6	47.8	-
Dense Sand	20.4	-	34.0
Rock	22.0	-	40.0
Vegetated Fill	17.3	4.8	-
Rip Rap	20.4	-	36.0
Heavy Stone Fill	20.4	-	-

The traffic surcharge load from the roadway was kept at a constant 9.6 kPa, based on Federal Highway Administration (FHWA) design manual [8]. The slope stability was analyzed under three varying hydraulic conditions namely low water, high water and flood conditions at elevations of 217.5, 227.5 and 237.75 respectively. Figures 5a – 5d represents the computed factors of safety at Station 208 for the left and right sections of the bridge for the Bishop's and Spencer methods under high water conditions.

3.4. Observations

The following observations were made

- Sections closer to the bridge showed more critical factors of safety compared to the farther ones.
- The addition of the geogrid membrane proved to be a decisive addition to the design in its ability to bolster the stability of both the left and right abutments.
- Critical failure surfaces passed through areas which saw in absence in the coverage of reinforcements while analyzing the right abutment. Providing longer length reinforcements ensured that the factor of safety was above the desired 1.3 by producing a larger slip circles and ensuring enough resisting forces.
- Impact of height of water in the three cases is not seen to affect the left abutment, however in the right abutment several sections see a critical slip surface forming at the low water scenario. The water at the high-water level and flood conditions is seen to provide a resisting force on the soil at the toe of the slope therefore increasing the factor of safety. The absence of this resistance force in low water level case produces a more critical case.
- A local slope failure on the left is more likely to occur. The critical slip surfaces are formed primarily below the placement of the geogrid reinforcements.

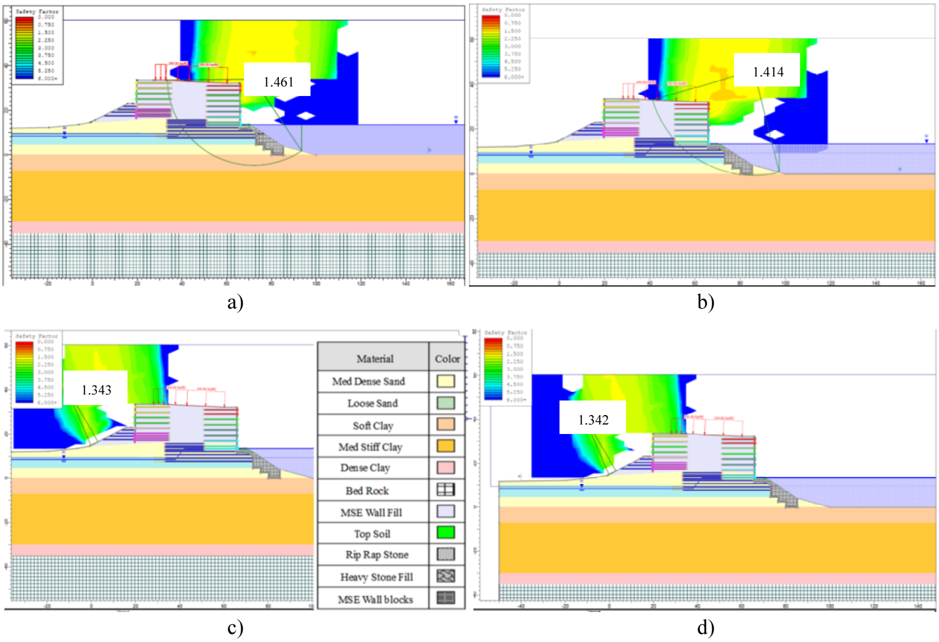

Figure 5. Results of the slope Stability at Station 208 at high water condition a) Right Abutment Bishop's Method b) Right Abutment Spencer Method c) Left Abutment Bishop's Method d) Left Abutment Spencer Method.

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