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Performance of a Geosynthetic-Reinforced Bridge Abutment in the US

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ABSTRACT: This paper revisits the performance of a geosynthetic-reinforced soil (GRS) system, which was constructed to support the shallow footings of a two-span bridge and the approaching roadway structures. Construction of this system, the Founders/Meadows bridge abutments, was completed in 1999 near Denver, Colorado. This system was selected with the objectives of alleviating the "bump at the bridge" problem often noticed when using traditional deep foundations, allowing for a small construction working area, and facilitating construction in stages. The primary focus of this paper is to evaluate the deformation response of this structure under service loads based on displacement data collected through surveying, inclinometer, strain gages, and digital road profiler. The overall short- and long-term performance of the Founders/Meadows structure was excellent, as evidenced from the monitored movements and loads, which were smaller than those anticipated in the design or allowed by performance requirements.

1. INTRODUCTION

A comparatively new application of geosynthetic reinforced soil technology in the US is its use of GRS abutments in bridge applications. When compared to typical systems involving the use of deep foundations to support bridge structures, the use of geosynthetic-reinforced systems has the potential of alleviating the "bump at the bridge" problem caused by differential settlements between the bridge abutment and approaching roadway. In addition, this system also allows for construction in stages and comparatively smaller construction working areas.

A prominent GRS abutment for bridge support in the U.S. is the new Founders/Meadows Parkway structure, located 20 miles south of downtown Denver, Colorado (Figure 1). This is the first major bridge in the United States built on footings supported by a geosynthetic-reinforced system, eliminating the use of traditional deep foundations (piles and caissons) altogether. Phased construction of the almost 9-m high, horseshoe-shaped abutments began July 1998 and was completed just twelve months later (June 1999). The Colorado Department of Transportation (CDOT) designed this structure in 1996. The Federal Highway Administration (FHWA) published preliminary design guidelines for bridge superstructures directly supported by MSE walls with panel facings and steel reinforcements in 1997.

Figure 1. View of the Founders/Meadows structure near Denver, Colorado.

Full-scale instrumentation of geosynthetic-reinforced soil systems has provided invaluable understanding on the performance of critical structure under in-service conditions (e.g. Allen et al. 1991, Zornberg et al. 1995). Consequently, the Founders/Meadows structure was considered experimental and comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations.

This paper focuses on the performance of the structure under service loads using short- and long-term movement data. This includes displacements of the front wall facing, settlement of the bridge footing, and differential settlements between the bridge and approaching roadway structures. Additional information on the design, materials, construction, instrumentation, and monitoring of the GRS walls in the Founders/Meadows structures have been presented by Abu-Hejleh et al. (2000a, 2000b). The information presented in this paper builds on previous presented evaluations (Abu-Hejleh et al. 2001, 2002), with emphasis on the deformation response of the structure.

2. DESCRIPTION OF THE STRUCTURE

2.1 Overall Characteristics

The structure provides an overpass to Colorado State Highway 86 over U.S. Interstate 25. Figure 2 shows the segmental retaining wall system located at the southeast side of the bridge. This figure shows that the girders from the bridge superstructure are supported by the "front GRS wall", which extends around a 90-degree curve into a "lower GRS wall". This "lower GRS wall" supports the reinforced concrete "wing wall" and a second tier, "upper GRS wall". Figure 3 shows a plan view of the completed two-span bridge and approaching roadway structures. Each span of the new bridge is 34.5 m long and 34.5 m wide, with 20 side-by-side prestressed box girders. The new bridge is 13 m longer and 25 m wider than the previous structure. It accommodates six traffic lanes and sidewalks on both sides of the bridge. The bridge is also supported by central pier columns (Figures 1 and 3), which are supported by a spread footings founded on bedrock at the median of U.S. Interstate 25.

The main cause of uneven settlements in typical bridge foundation systems is the use of different foundation types. That is, while the approaching roadway structure is typically founded on compacted backfill soil, the bridge abutment is typically founded on stronger soils by deep foundations. The approaching roadway embankment and the bridge footing were integrated at the Founders/Meadows structure with an extended reinforced soil zone in order to minimize uneven settlements between the bridge abutment and approaching roadway. A compressible 75 mm thick low-density expanded polystyrene sheet was placed between the reinforced backfill and the abutment walls to accommodate thermally induced movements of the bridge superstructure (Abu-Hejleh et al. 2000a).

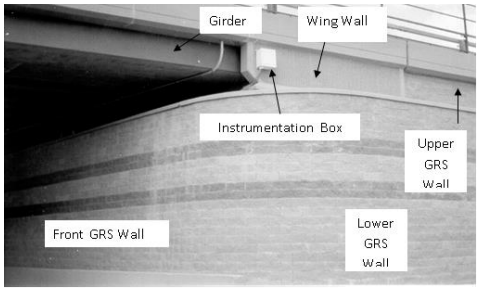

Figure 2. View of the southeast side of the Founders/Meadows bridge abutment.

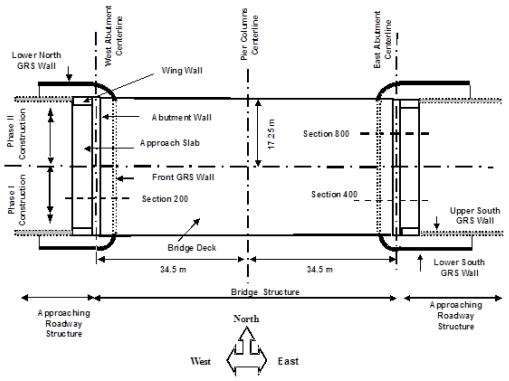

Figure 3. Plan view of the Founders/Meadows structure showing the locations of monitored sections (sections 200, 400, and 800).

2.2 Material Characteristics

The materials used for construction of the front GRS wall system included backfill, geogrid reinforcements, concrete facing blocks, and facing connectors between the blocks and the reinforcements and between blocks of the wall. The facing blocks were part of the Mesa system (Tensar Corporation), and have a compressive strength of 28 MPa. The geogrid reinforcements employed beneath the bridge footing were UX 6 geogrids, also provided by the Tensar Corporation. The ultimate strength of the UX 6 geogrid is 157.3 kN/m, measured in accordance with ASTM D 4595 test method. CDOT specifications imposed a global reduction factor of 5.82 to determine the long-term design strength (LTDS) of the geogrid reinforcements from their ultimate tensile strength. This global reduction factor includes partial factors to account for tensile strength losses over the design life due to creep (2.7), durability (1.1), installation damage (1.1), and it also includes a factor of safety to account for uncertainties (1.78). The LTDS of the UX 6 geogrid is 27 kN/m. The load-strain curve for the UX 6 geogrid is approximately linear for a range of tensile strains from 0 to 1% (the tensile load at 1% strain is approximately 2000 kN/m). The connection strength for the mechanical connectors mobilized is 57.7 kN/m, measured in accordance with NCMA Test Method SRWU-1 at a horizontal movement of 19 mm (service state). This value is above the LTDS of UX6 geogrids. Other geogrid reinforcements (UX 3 and UX 2) were used behind the bridge abutment walls, as shown in Figure 4. The LTDS of these reinforcements was 11 kN/m and 6.8 kN/m, respectively.

The backfill soil used in this structure includes fractions of gravel (35%), sand (54.4%), and fine-grained soil (10.6%). The liquid limit and plasticity index of the fine fraction are 25% and 4 %, respectively. The backfill soil classifies as SW-SM per ASTM 2487, and as A-1-B (0) per AASHTO M 145. The average unit weight, dry unit weight, and placement water content of the compacted backfill, as measured during construction, were 22.1 kN/m^3, 21 kN/m^3 and 5.6%, respectively. The placed dry unit weight (21 kN/m^3) corresponds to 95% of the maximum dry unit weight measured in accordance with AASHTO T-180A.

3. INSTRUMENTATION PROGRAM

The layout of the instrumentation program of Section 800 is shown in Figure 4. The height of the front GRS wall (i.e. elevation above leveling pad) is 5.9 m for Sections 400 and 800, and 4.5 m for Section 200. The bridge footing is located 5.28 m above leveling pad for Sections 400 and 800 and 3.86 m above the leveling pad for Section 200. The collected displacement data is organized according to the loading sequence, as follows:

- Construction of the front GRS wall (Stage I). Construction took place from July 16, 1998 to September 12, 1998 for the Phase I structure (Sections 200 and 400), and from January 19, 1999 to February 24, 1999 for the Phase II structure (Section 800). Movements induced during this stage (i.e. before placement of the bridge superstructure) are compensated during wall construction.
- Placement of the bridge superstructure (Stages II to VI). Monitoring stages include placement of the bridge footing and girders seat (Stage II), placement of girders (Stage III), placement of reinforced backfill behind the concrete abutment wall (Stage IV), placement of the bridge deck (Stage V), and placement of additional structures (Stage VI). Placement of the bridge superstructure was completed on December 16, 1998 for the Phase I structure, and on June 30, 1999 for the Phase II structure.
- Post-construction performance (Stage VII). The average total vertical contact stress directly underneath the bridge footing during this stage was estimated as 150 kPa. Post-construction data presented in this paper was collected until November 2001 (i.e. during 35 months and 29 months after the opening to traffic of Phase I and Phase II structures, respectively).

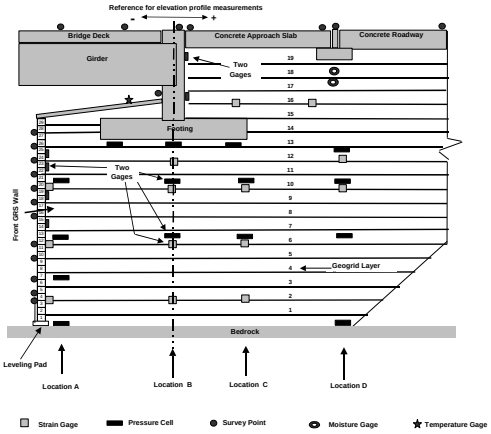

Figure 4. Instrumentation layout of section 800, showing location and type of instruments.

The monitoring program included components aimed at evaluating the deformation response and the stress distribution within the reinforced soil walls. The instrumentation used to evaluate the deformation response of the system, which is the focus of this paper, included survey targets, inclinometer, strain gages, and digital road profiler. Survey targets used in the monitoring program involved reflectors permanently glued to the outside face of front and abutment walls (all sections), bridge deck, approaching slab, and roadway (only Section 800). A vertical inclinometer tube was affixed to the back of the facing blocks of the Phase I structure Section 400. The tube was placed in segments during the construction of the front GRS wall. A Geokon Model 6000 inclinometer probe was used in conjunction with the inclinometer tube to measure lateral movement of the fill material, both parallel and perpendicular to the wall. The bottom end of the inclinometer tube was set on top of the leveling pad and held in place by the fill material and the back of the blocks.

Geokon Model 4050 strain gages with a gage length of 150 mm and range of 0.7% were installed along Section 800 (Figure 4). The strain gages were mounted using two brackets that clamp to the geogrid. The brackets were mounted to the geogrid before placement of soil, which was then placed and compacted over the clamps. After compaction, fill material was excavated at the instrumentation location, the gages were installed and soil was manually compacted at the instrument location. Geokon provided calibration and installation information for the strain gages.

3. RESULTS FROM MONITORING PROGRAM

Insight on the outward wall displacements can be gained from the strain gauge measurements collected along geogrid layers 6 and 10 (see Figure 4). These strain gages were placed along four critical locations: Location line A close to the wall facing, Location line B close to the centerline of the bridge abutment wall, Location line C close to the back edge of the bridge footing, and Location line D

behind the bridge footing (approximately 7.6 meter behind the wall facing). Figure 5 shows the geogrid strain distributions measured along layers 6 and 10 at the end of the front GRS wall construction (Stage I) and during placement of the bridge superstructure (Stages III to VI). The outward displacements of the front GRS wall facing at the elevations of layers 6 and 10 were obtained, at different stages, by integrating the geogrid strains from the facing until location line D (7.6 m from the facing). Accordingly, the retained backfill was assumed not to move. For layer 6, the geogrid strain was taken zero at 7.6 m from the facing, which seems reasonable as indicated by the results in Figure 5a. Figure 6 presents the outward displacements at the facing as a function of the estimated average vertical soil stress applied on geogrid layers 6 and 10 during all construction stages. The label shown next to each data point in the figure indicates the construction stage to which the data point corresponds.

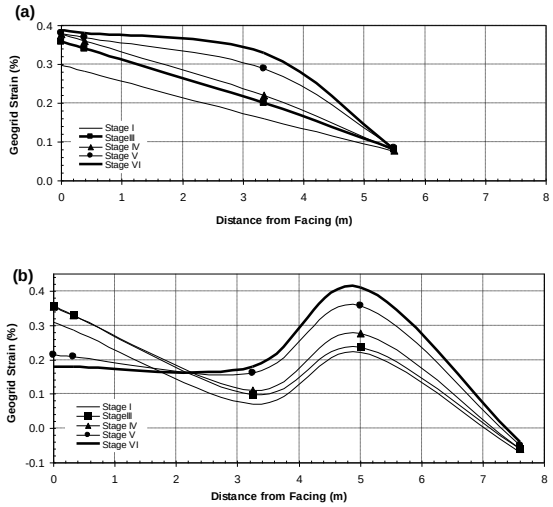

Figure 5. Geogrid strain distribution measured after construction stages along: a) geogrid layer 6, and b) geogrid layer 10.

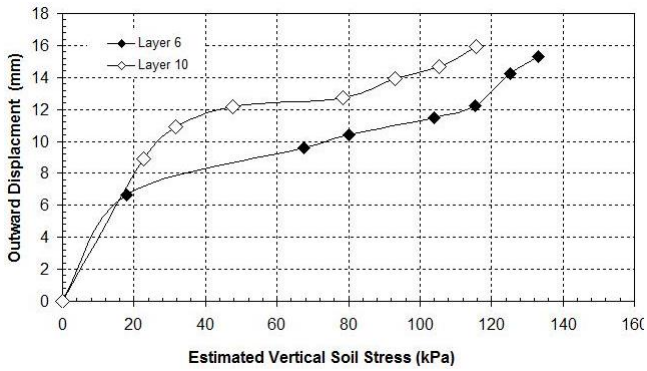

Figure 6. Outward displacements of the front GRS wall facing (section 800) at the elevation of geogrid layers 6 and 10, obtained from the strain gages results.

The results presented in Figure 6 indicate that, for the same level of applied vertical stress, the wall outward displacements along geogrid layer 10 are higher than those obtained along geogrid layer 6. This is an expected behavior because the width of the active zone (defined by the locus of maximum tension line) increases with the elevation above the leveling pad.

Construction of the front GRS wall (Stage I, before placement of the bridge structure) corresponds to the first three data points shown in Figure 6. The second data point in Figure 6 was collected after compaction and placement of approximately 1 m of backfill (corresponding to approximately 20 kPa of vertical soil stresses) over the gages. These results indicate that a significant portion of the wall displacements occur during the initial stages of backfill placement and compaction. As indicated in the figure, the maximum wall outward displacement at the elevation of geogrid layer 10 induced by wall construction was 11 mm. The maximum wall outward displacement due to placement of the bridge superstructure (Stages II to VI) was approximately 6 mm (at the elevation of geogrid layer 6). This indicates that the structure responded with comparatively small deformation to the increased vertical soil stresses induced by bridge loads. A possible reasons for the stiffer response is the influence of compaction experienced in the previous stage (Stage I). An additional justification is the fact that Construction Stages II to IV took place during the winter season. Buttry et al. (1996) reported a comparatively more rigid behavior during the winter season for a GRS structure. During Stages V and VI, the GRS system response shows comparatively larger displacements to the increasing vertical soil stresses. Thawing and wetting of the backfill, as well as smaller influence of the compaction effect, may have led to softening of the backfill during these stages. Overall, strain gage results shown in Figure 6 indicate that, in spite of the large surcharge loads due to bridge superstructure, the largest component of wall displacements occurred during compaction of the backfill.

3. CONCLUSION

Some aspects of the deformation response of a geosynthetic-reinforced soil abutments system, the Founders/Meadows bridge abutments, are documented in this paper. The following conclusions can be drawn from this evaluation:

- The monitored movements were significantly smaller than those expected in design or allowed by performance requirements.
- The use of a GRS bridge abutment was successful in preventing development of the "bump at the bridge" problem, as no signs of differential settlements have been observed after more than two years following opening of the structure to traffic.
- The use of redundant instrumentation was useful to provide confidence on the monitoring results. In particular, outward displacements obtained from surveying, inclinometers, and inferred from strain gage measurements showed good agreement.
- Most of the outward displacements at the wall facing occurred during the initial stages of backfill placement and compaction. Strain gage results indicate that approximately 50% of the total outward displacements of front GRS wall facing occurred during placement and compaction of approximately 1 m of soil over the geogrid layers (approximately 20 kPa vertical soil stress).

4. REFERENCES

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