Technical Paper by N. Abu-Hejleh, J.G. Zornberg, T. Wang, and J. Watcharamonthein

MONITORED DISPLACEMENTS OF UNIQUE GEOSYNTHETIC-REINFORCED SOIL BRIDGE ABUTMENTS

ABSTRACT: A geosynthetic-reinforced soil (GRS) system 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, USA. This unique 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 the 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, suggesting that GRS walls are a viable alternative to support both bridge and approaching roadway structures.

KEYWORDS: Soil Reinforcement, Bridge abutment, Field monitoring, Geogrid, Instrumentation.

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1 INTRODUCTION

The technology of geosynthetic-reinforced soil (GRS) systems has been used extensively in transportation systems to support the self-weight of the backfill soil, roadway structures, and traffic loads. The increasing use and acceptance of soil reinforcement has been triggered by a number of factors, including cost savings, aesthetics, simple and fast construction techniques, good seismic performance, and the ability to tolerate large differential settlement without structural distress. A comparatively new application of this technology is the 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.

The most 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, USA (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 12 months later (June 1999). This system replaced a deteriorated two-span bridge structure in which the abutments and central pier columns were supported on steel H-piles and spread footing, respectively.

The perceived advantages of GRS abutments convinced Colorado Department of Transportation (CDOT) engineers to select GRS walls to support the bridge abutment in the Founders/Meadows structure. CDOT designed this structure in 1996. The Federal Highway Administration (FHWA) published preliminary design guidelines for

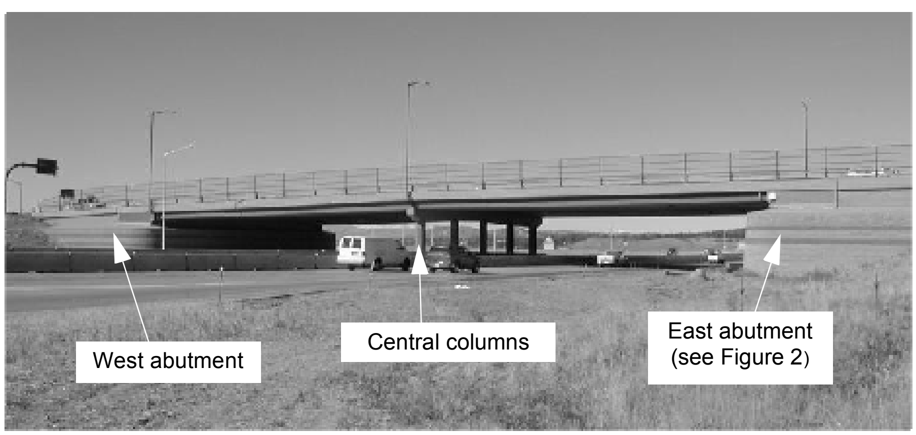

Figure 1. View of the Founders/Meadows structure near Denver, Colorado, USA, showing the east and west abutments and the central pier columns.

bridge superstructures directly supported by mechanically stabilized earth (MSE) walls with panel facings and steel reinforcements in 1997 (Elias and Christopher 1997; AASHTO 1996). Differently than in these guidelines, the Founders/Meadows structure uses segmental block facing and geosynthetic reinforcements. A recently published FHWA report (FHWA 2000) describes three studies on GRS bridge supporting structures with segmental facing: load test of the Turner-Fairbank pier (1996), load test of the Havana Yard piers and abutment in Denver, Colorado, USA (1996 to 1997), and a production bridge abutment constructed in Black Hawk, Colorado, USA (1997). These studies have demonstrated adequate performance and negligible creep deformations of structures constructed with closely spaced reinforcement elements and well-compacted granular backfill when subjected to a maximum surcharge pressure of 200 kPa. The FHWA report concludes that GRS abutments are viable and adequate alternatives to bridge abutments supported by deep foundations and to metallic reinforced soil abutments. A comprehensive literature review of studies on GRS structures supporting high surcharge loads is presented by Abu-Hejleh et al. (2000b).

The performance of bridge structures supported by GRS abutments has not been tested under actual service conditions to merit acceptance without reservation in highway construction. Full-scale instrumentation of GRS systems has provided invaluable understanding on the performance of critical structures under in-service conditions (e.g., Allen et al. (1991), Zornberg et al. (1995), and Bathurst et al. (2001)). Consequently, the Founders/Meadows structure was considered experimental, and comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations. Three sections of the GRS system were instrumented to provide information on the structure movements, soil stresses, geogrid strains, and moisture content during construction and after opening the structure to traffic. The overall objectives of this monitoring program are:

- to assess the structure's performance under service loads using short- and long-term movement data;
- to evaluate the suitability of CDOT and American Association of State Highway and Transportation Officials (AASHTO) design procedures and assumptions regarding the use of GRS walls to support bridge footings, and as a measure to alleviate the "bump at the bridge" problem; and
- to collect performance data for future calibration and validations of numerical models.

The present paper focuses on the first objective listed above by presenting an evaluation of the movements of the Founders/Meadows structure collected during various construction stages and during post-construction. This includes displacements of the front wall facing, settlement of the bridge footing, and differential settlements between the bridge and approaching roadway structures. Lessons learned from the deformation response, suitable for future GRS abutments supporting directly bridge and approaching roadway structures, are finally provided. Additional information on the design, materials, construction, instrumentation, and monitoring of the GRS walls in the Founders/Meadows structures are presented in a CDOT research report (Abu-Hejleh et al. 2000a). Results gathered during the Phase I instrumentation, including stress distributions, are reported by Abu-Hejleh et al. (2000b). In addition, a recent CDOT report (Abu-Hejleh et al. 2001) evaluates the design and performance of the front GRS walls and presents instrumentation data (displacements, stresses, and strains) collected during and after construction.

2 DESCRIPTION OF THE GRS BRIDGE ABUTMENT WALLS

The Founders/Meadows structure carries 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. Figure 2 shows that the girders from the bridge superstructure are supported by the "front GRS wall", which extends around a 90° 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. Figure 4 shows a typical monitored cross section through the "front GRS wall" and reinforced concrete abutment wall. Sections 200, 400, and 800 (Figure 3) have been instrumented and the monitoring movement results are reported in the present study. Figure 4 illustrates how the bridge superstructure load (from girders, bridge deck) is transmitted through reinforced concrete abutment walls to a shallow strip footing placed directly on the top of a geogrid-

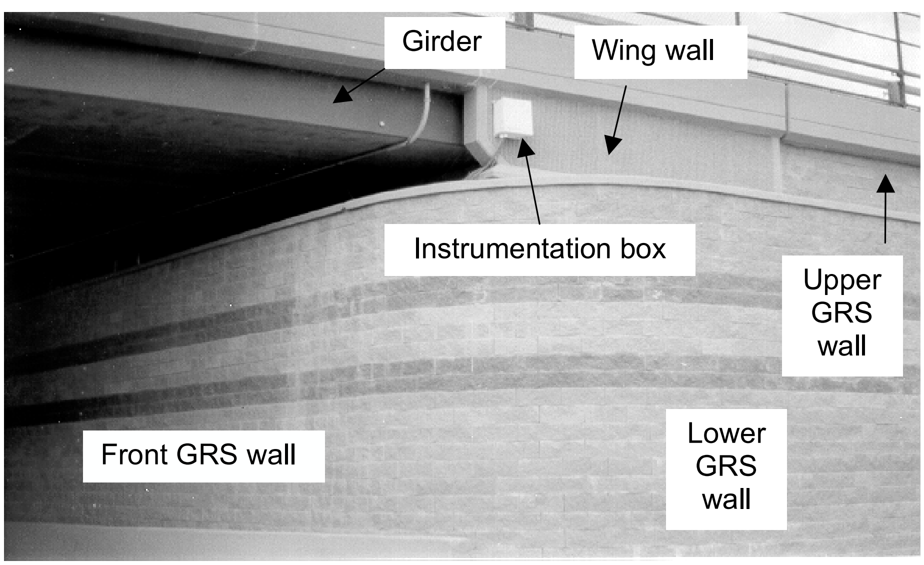

Figure 2. View of the southeast side of the Founders/Meadows bridge abutment.

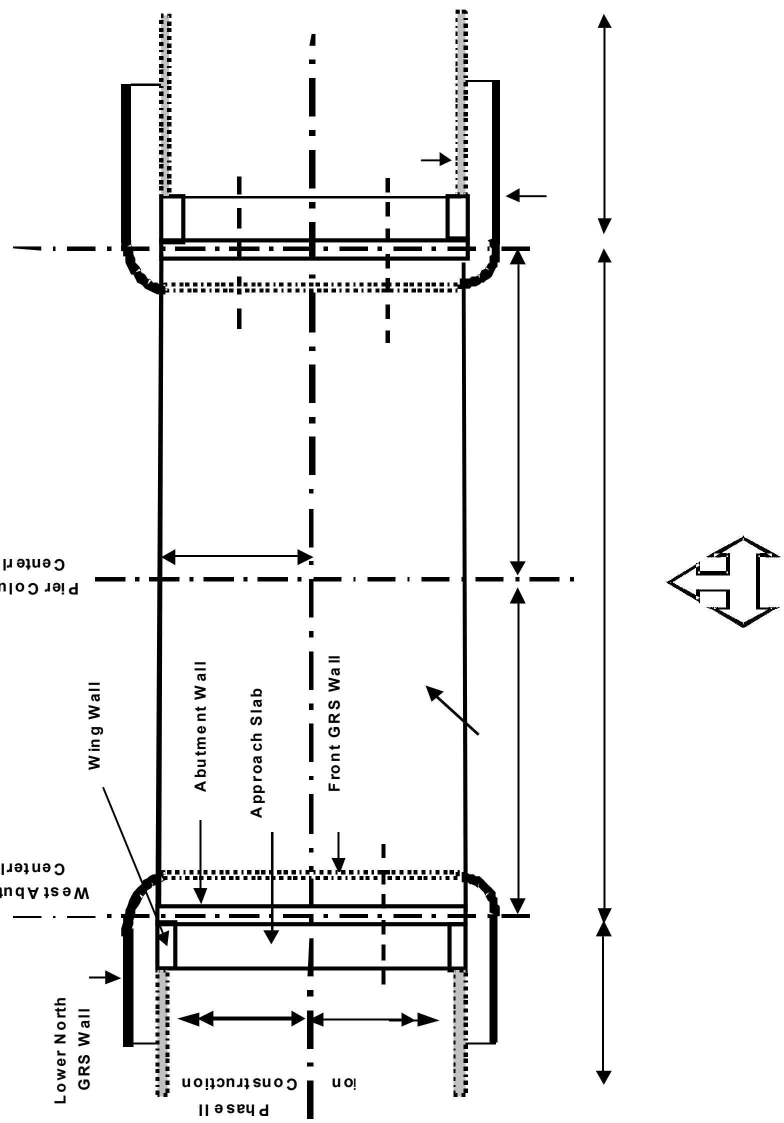

Figure 3. Plan view of the Founders/Meadows structure showing the locations of monitored sections (Sections 200, 400, and 800) and the two construction phases (Phases I and II).

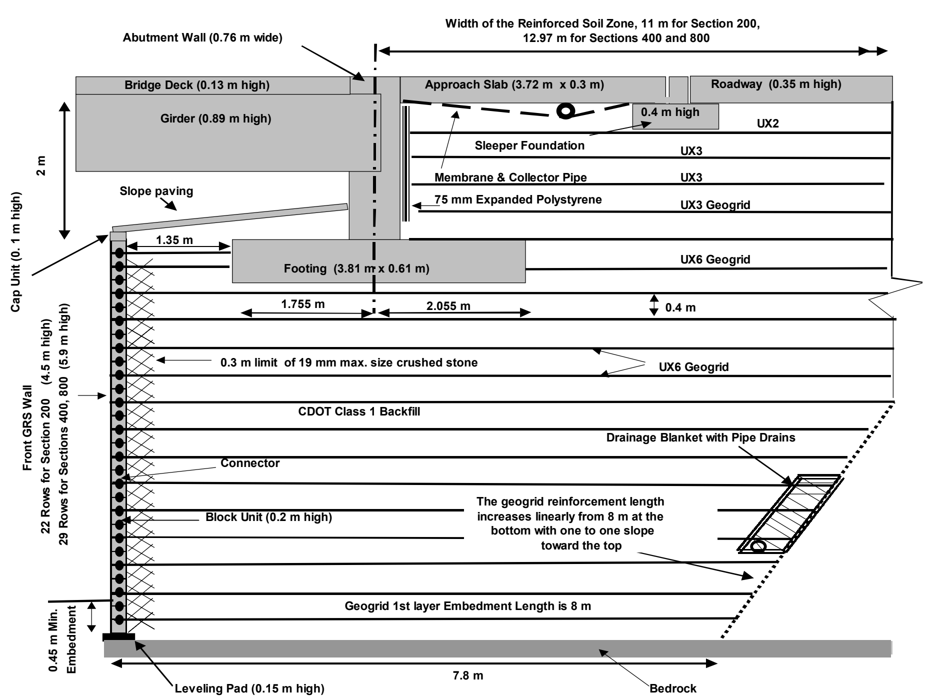

Figure 4. Typical cross section through the front and abutment GRS walls.

reinforced segmental retaining wall (front GRS wall). The centerline of the reinforced concrete abutment wall and the edge of the footing are located 3.1 and 1.35 m, respectively, from the facing of the front GRS wall. The reinforced concrete abutment wall and two reinforced concrete wing walls (Figures 2 and 3) rest on the spread footing, confine the reinforced backfill soil (upper GRS wall) behind the bridge abutment, and support the bridge approach slab. The bridge is also supported by central pier columns (Figures 1 and 3), which are supported by spread footings founded on bedrock at the median of U.S. Interstate 25. It was anticipated that the competent claystone bedrock formation below the reinforced backfill and the use of an extended reinforced zone would lead to adequate external stability and minimize differential settlements.

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. Abu-Hejleh et al. (2000b) discusses in detail several other common causes for the development of bridge bumps, which were addressed in the design of the Founders/Meadows structure. 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. Differential settlements can also be caused by erosion of fill material induced by surface water runoff. Several measures were implemented in this project to prevent surface water and groundwater from reaching the reinforced soil mass and the bedrock at the base of the fill. This included placement of impervious membranes with collector pipes as shown in Figure 4. Finally, differential settlements can also be caused by temperature changes, which may induce expansion and contraction of bridge girders attached to the abutment wall (integral abutment). A compressible 75 mm-thick, low-density expanded polystyrene sheet was placed between the reinforced backfill and the abutment walls (Figure 4) to accommodate thermally induced movements of the bridge superstructure (Abu-Hejleh et al. 2000a).

3 MATERIAL CHARACTERISTICS OF THE GRS WALLS

The materials used for construction of the front GRS wall system (Figure 4) included backfill, geogrid reinforcements, concrete facing blocks, and facing connectors between the blocks and the reinforcement 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 reinforcement 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 the 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 reinforcement 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 2,000 kN/m). The connection strength for the mechanical connectors mobilized is 57.7 kN/m, measured in accordance with the National Concrete and Masonry Association (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 reinforcement (UX 3 and UX 2) were used behind the bridge abutment walls (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 (well-graded, silty sand) per ASTM 2487, and as A-1-B (0) (gravel and sand) per AASHTO (1998). The average unit weight, dry unit weight, and placement water content of the compacted backfill, as measured during construction, were 22.1, 21, 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 (1998). The backfill met the material and compaction requirements for CDOT Class 1 backfill material. A friction angle of 34^o and zero cohesion were assumed during design for the backfill material. To evaluate the suitability of the assumed shear strength parameters, conventional direct shear tests and large size direct shear and triaxial tests were conducted using the actual backfill material used in this project. A peak friction angle of 40.1° and a cohesion intercept of 17 kPa were obtained from conventional small-size direct shear tests. In the conventional direct shear tests, the gravel portion was removed from the tested specimens. However, large-size triaxial and direct shear tests were also conducted, which included the gravel portion of the backfill soil. A peak friction angle of 47.7° and a cohesion intercept of 110.5 kPa were obtained from large-size direct shear tests while a peak friction angle of 39.5° and a cohesion intercept of 69.8 kPa were obtained from large-size triaxial tests. Shear strength results obtained from both conventional and large-size direct shear and triaxial tests verified that the shear strength assumed in design was below the actual shear strength of the backfill. Also, the experimental results indicate that assuming zero cohesion and removing the gravel portion from the test specimens leads to significant underestimation of the actual backfill shear strength. Hyperbolic model constitutive parameters were also determined from the results of the large size triaxial tests (Abu-Hejleh et al. 2000a).

4 INSTRUMENTATION PROGRAM FOR MEASUREMENT OF STRUCTURE MOVEMENTS

The instrumentation program was conducted in two construction phases (Figure 3): Phase I and II that correspond, respectively, to the construction of the Phase I structure (from July to December 1998) and Phase II structure (from January to June 1999). Sections 200 and 400 are located at the center of the Phase I structure and Section 800 is located at the center of the Phase II structure (Figure 3). The layout of the instrumentation program of Section 800 is shown in Figure 5. 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 the 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 following loading sequence:

- *Construction of the front GRS wall (Stage I).* Construction took place from 16 July 1998 to 12 September 1998 for the Phase I structure (Sections 200 and 400) and from 19 January 1999 to 24 February 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). The average total vertical contact stress directly underneath the bridge footing after loading was estimated as 115 kPa. Placement of the bridge superstructure was completed on 16 December 1998 for the Phase I structure and on 30 June 1999 for the Phase II structure.
- *Post-construction performance (Stage VII).* The average total vertical contact

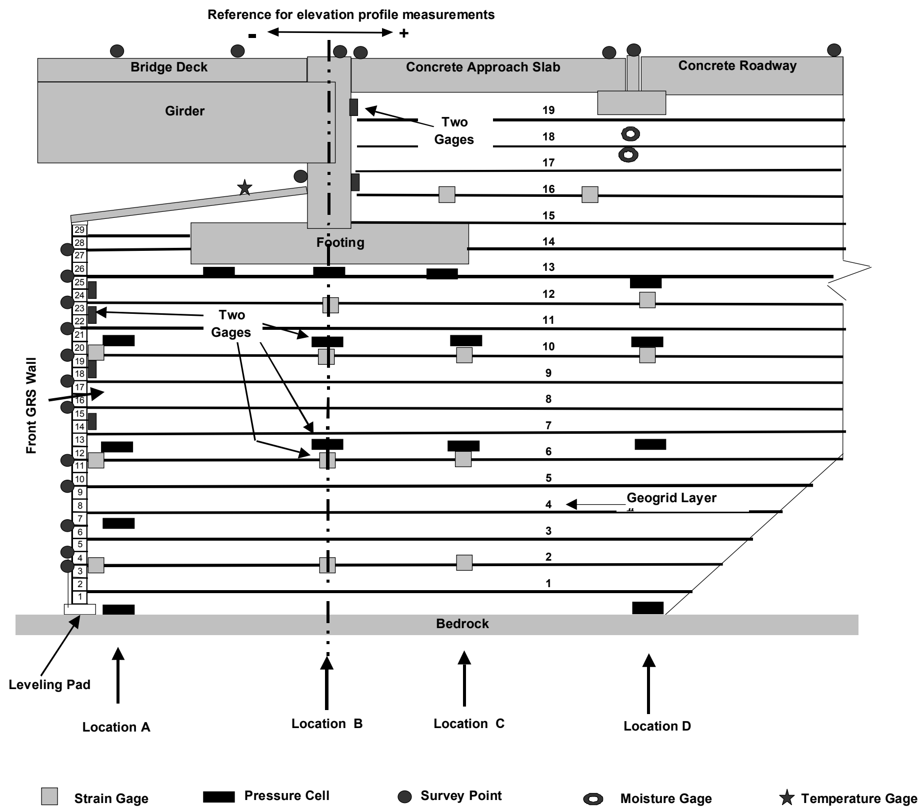

Figure 5. Instrumentation layout of Section 800 showing location and type of instrument.

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 the Phase I and Phase II structures, respectively).

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 the present paper, included survey targets, an inclinometer, strain gages, and a digital road profiler.

Survey targets used in the monitoring program involved reflectors permanently glued to the outside face of the front and abutment walls (all sections), bridge deck, approaching slab, and roadway (only Section 800). North, East, and elevation coordinates of surveying targets were collected at the different loading stages. The North-East movements were grouped into two displacement components: perpendicular to the wall (i.e., outward displacement) and parallel to the wall. The displacements collected in the vertical direction were used to estimate the structure settlements. The accuracy range of the surveying system was approximately ±3 mm.

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 5). 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. The reader is referred to Abu-Hejleh et al. (2000a) and Geokon manuals for additional information on the calibration and installation of the strain gages.

A digital road profiler was used as part of the displacement monitoring program. This device, manufactured by Face Construction Technologies, Inc. (Norfolk, Virginia, USA) was used to define elevation profiles of the road surface in the vicinity of the transition from the bridge deck to the approaching roadway in order to collect evidence of potential differential settlements between the bridge and the approach roadway structure.

5 FRONT WALL OUTWARD DISPLACEMENTS

5.1 Surveying Results for Outward Displacements Induced During Construction

Surveying data on the outward displacements induced on Sections 400 and 800 during construction of the front GRS wall is summarized in Figure 6. Displacement data for Section 400 was collected along the lower 14 facing block layers (up to 2.75 m above leveling pad) and corresponds to the construction of the front GRS wall from elevations 3.65 to 5.5 m. Displacement data for Section 800 was collected along the lower 10 facing block layers (up to 2.0 m above the leveling pad) and corresponds to construction of the front GRS wall from elevation 2.44 to 5.5 m. It should be noted that the sets of movement data shown in Figure 6 were not collected during construction of the same reinforcement lifts and, consequently, a direct comparison is not possible. Nevertheless, the outward displacement trends are consistent (note that the load applied to Section 800 is higher) and it provides an order of magnitude of the expected outward displacements. The maximum wall outward displacements measured during construction of the front GRS wall are 8.5 and 11.5 mm for Sections 400 and 800, respectively.

Surveying data for the outward wall displacements induced during placement of

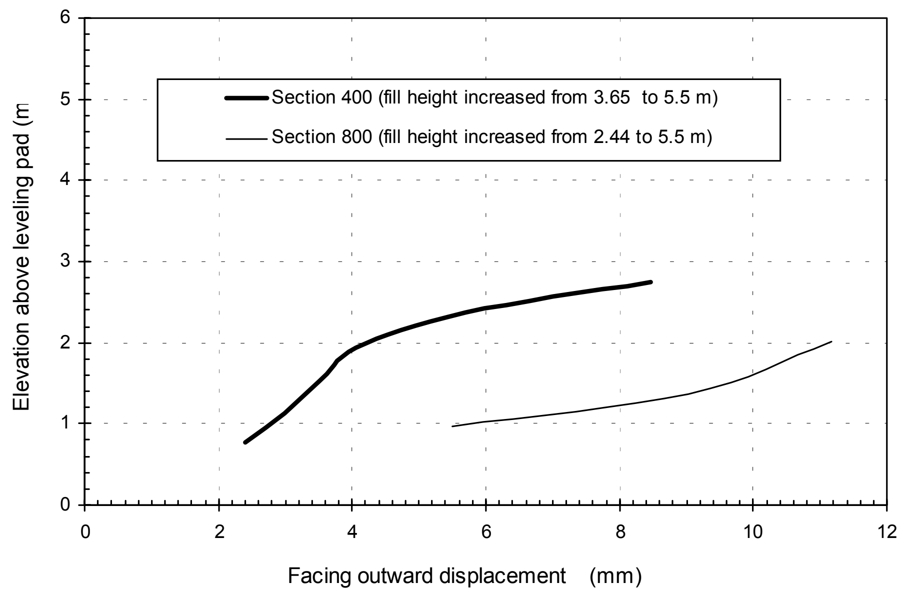

Figure 6. Outward displacements of the front wall facing induced by construction of the front GRS wall (Stage I).

Note: Data obtained by survey measurements.

the bridge superstructure is summarized in Figure 7. As observed in Figure 7, the maximum wall outward displacements experienced along Sections 200, 400, and 800 during placement of the bridge are approximately 7, 9, and 10 mm, respectively. The maximum outward displacements occur within the upper third of the wall, directly below the bridge footing. The maximum wall outward displacements experienced along Section 800 (10 mm) and 400 (9 mm) are higher that those experienced along Section 200 (7 mm) because of the different height of the structure (Section 200 is 4.5 m high while Sections 800 and 400 are 5.9 m high). Although Sections 400 and 800 are identical in configuration and applied loading, the displacements induced along the depth of Section 800 are somehow higher. Possible explanations for the difference in outward displacements between these two sections are as follows.

- *Different construction season.* Most of the Phase I structure (Section 400) was constructed during a warm season while the front GRS wall of Phase II (Section 800) was constructed during a cold season. Placement of the bridge superstructure along Section 800 occurred mostly in March and April of 1999 when thawing and wetting seasons started. This may have led to softening of the backfill and comparatively larger deformations.
- *Different construction sequence.* The backfill behind the abutment wall was placed before placement of the girders during construction of Section 400. Instead, the girders were placed before placing backfill behind the abutment wall during construction of Section 800. This induced, most probably, larger outward displace-

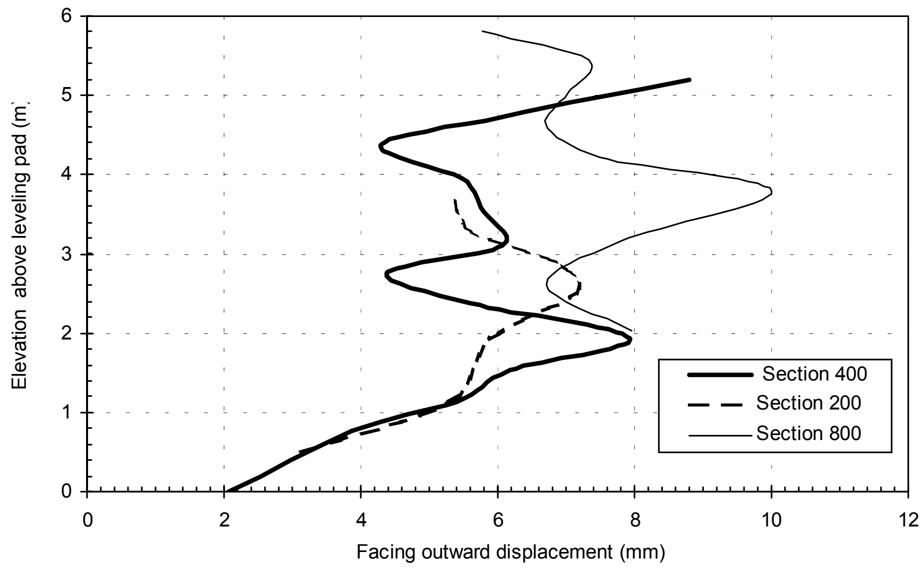

Figure 7. Outward displacements of the front wall facing induced by placement of the bridge superstructure.
Note: Data obtained by survey measurements.

ments and reinforcement strains within the GRS backfill along Section 800.

5.2 Strain Gage Results During Construction

Additional insight on the outward wall displacements can be gained from the strain gage measurements collected along geogrid layers 6 and 10 (Figure 5). 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 m behind the wall facing). Figure 8 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, Figure 5). Accordingly, the retained backfill was assumed not to move. For layer 6, the geogrid strain was taken as zero at 7.6 m from the facing, which seems reasonable as indicated by the results in Figure 8a. Figure 9 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 Figure 9 indicates the construction stage to which the data point corresponds. The average vertical soil stresses at different stages, were estimated as:

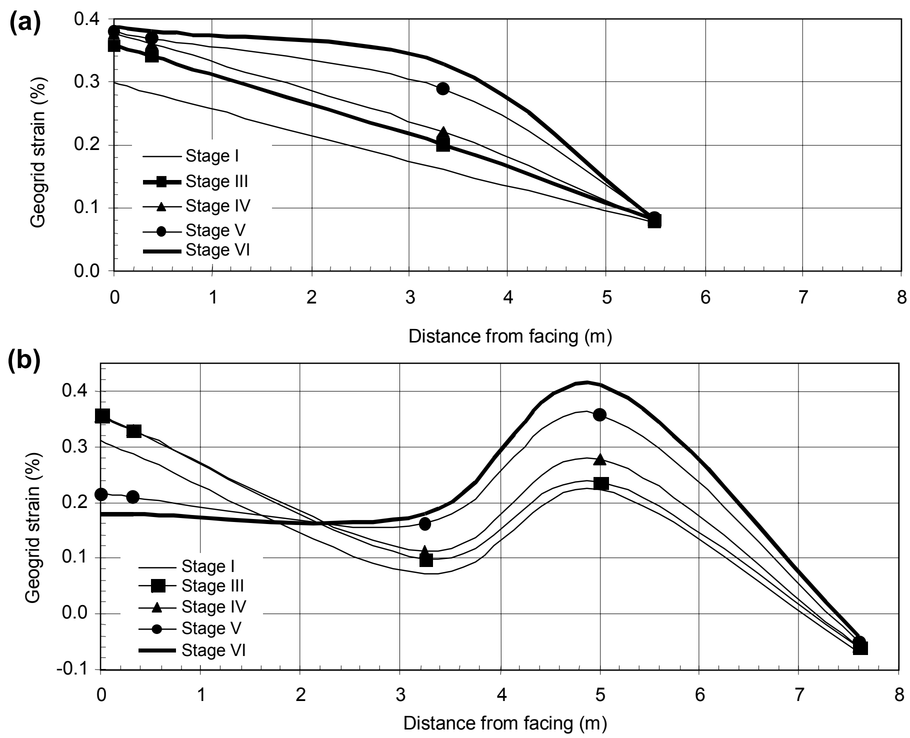

Figure 8. Geogrid strain distribution measured at the end of various construction stages along: (a) geogrid layer 6; (b) geogrid layer 10.

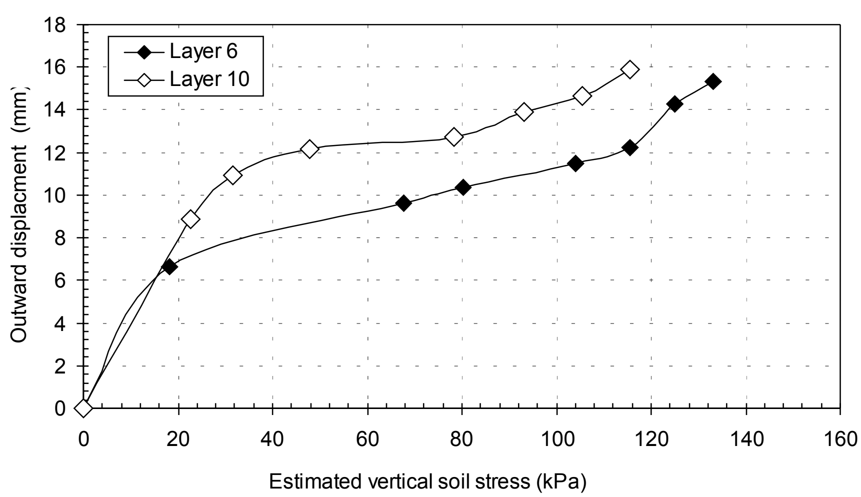

Figure 9. Outward displacements of the front GRS wall facing (Section 800) at the elevation of geogrid layers 6 and 10 obtained from the strain gages results.

$$\sigma_v = \gamma z + \Delta\sigma_v \tag{1}$$

where: γ = measured backfill unit weight (22.1 kN/m^3); z = backfill height above the layer of interest; and $\Delta\sigma_v$ = vertical stress induced within the soil mass by concentrated surcharge loads. AASHTO (1996) recommends the use of the 2V:1H approximation to estimate the distribution of vertical stresses induced within the soil mass by concentrated surcharge loads. This approximation was adopted to estimate $\Delta\sigma_v$ at different construction stages. The results presented in Figure 9 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 9. The second data point in Figure 9 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 Figure 9, the maximum wall outward displacement at the elevation of geogrid layer 10 induced by wall construction is 11 mm. The maximum wall outward displacement due to placement of the bridge superstructure (Stages II to VI) is 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 reason 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 the smaller influence of the compaction effect, may have led to softening of the backfill during these stages. Overall, the strain gage results shown in Figure 9 indicate that, in spite of the large surcharge loads due to the bridge superstructure, the largest component of wall displacements occurred during compaction of the backfill.

5.3 Outward Displacements During in Service Conditions (Stage VII)

The Phase I structure (where Sections 200 and 400 are located) was opened to traffic in December 1998, and the Phase II structure (where Section 800 is located) was opened to traffic in June 1999. Post-construction movements in all monitored sections have been collected during this period. Surveying results collected until June 2000, inclinometer results collected until August 2001, and strain gage information collected until October 2001 are reported herein. The outward displacements of the front wall facing, induced while the structure was in service, are presented in Figure 10. Outward displacements of the front GRS wall collected along Sections 200 and 800 are shown

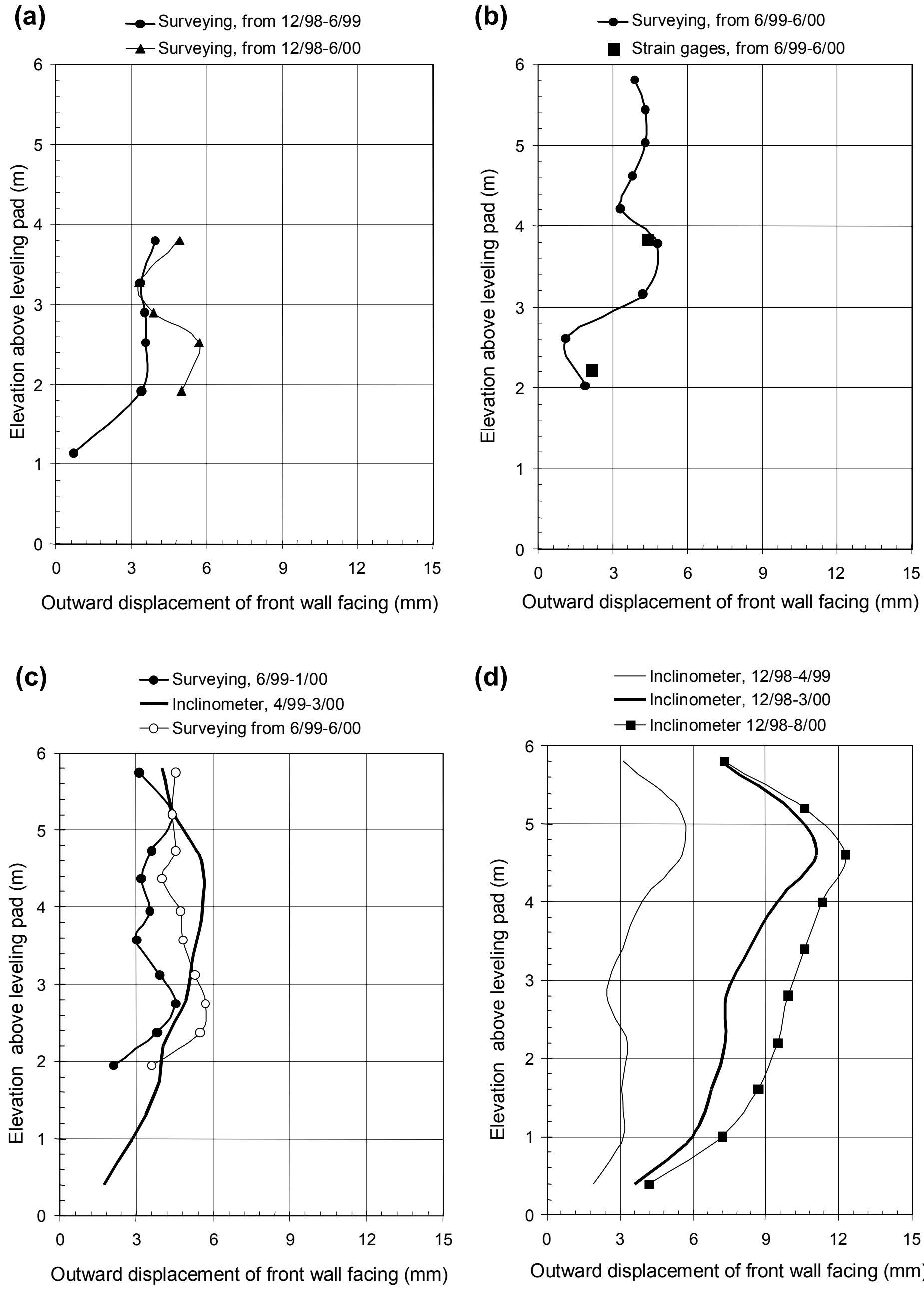

Figure 10. Outward displacements of the front wall facing induced while the structure was in service: (a) Section 200 (from surveying); (b) Section 800 (from surveying and strain gages); (c) Section 400 (from inclinometer and surveying); (d) Section 400 (from inclinometer).

in Figures 10a and 10b, respectively. Outward displacements of the front GRS wall collected along Section 400 from surveying and inclinometer measurements are shown in Figures 10c and 10d. Possible causes for post-construction movements are traffic load, deformation under sustained load (creep), and seasonal changes.

Post-construction outward displacements obtained in Section 200 (Figure 10a) are comparatively small, with a maximum outward displacement of approximately 6 mm obtained 18 months after opening the structure to traffic. As is also the case in the other monitored sections, the outward displacements in Section 200 decrease toward the leveling pad.

Post-construction outward displacements obtained in Section 800 (Figure 10b) are also comparatively small, with a maximum displacement of approximately 5 mm during 12 months. The surveying data reported in Figure 10 was selected for a period of time that allows comparison of surveying with strain gage results (June 1999 to June 2000). Very good agreement can be observed between displacements collected by surveying and displacements inferred from strain gages. The front wall displacements inferred from strain gage measurements for Section 800 were obtained at the elevation of reinforcement layers 6 and 10.

Surveying and inclinometer measurements collected in Section 400 are shown in Figure 10c. Even though the Phase I structure was opened to traffic in December 1998, reliable surveying for this section was collected only from June 1999. The inclinometer was placed behind the facing of the front GRS wall of Section 400 (Figure 5). Data obtained from the inclinometer along Section 400 represents the movements of the wall relative to the leveling pad (i.e., not the total movements of the wall). However, neglecting leveling pad displacements was considered adequate since small outward displacements were obtained from surveying of post-construction movements in all monitored sections. As shown in Figure 10c, the post-construction outward displacements obtained from inclinometer measurements during a period of approximately one year are in good agreement with outward displacements obtained from surveying measurements also during a period of approximately one year. This agreement provides confidence on the inclinometer results.

Although Sections 400 and 800 are identical in configuration and applied surcharge loading, post-construction movements observed along Section 400 (Figure 10c) are larger than those observed along Section 800 (Figure 10b). This is consistent with the observations made regarding the displacements induced by placement of the bridge superstructure (Figure 7) and may also be attributed to the construction sequence and season. From January to June 1999, the Phase I structure was in service while the Phase II structure was under construction. Consequently, the Phase I structure (Section 400) may have been subjected to additional loads during this period due to construction of the Phase II structure (Section 800). Thawing and wetting may have also contributed to softening of the backfill in Section 400 while the Phase I structure was in service.

Figure 10d shows the outward displacement profiles obtained by inclinometer measurements along Section 400 at different periods of time. The last displacement profile shown in Figure 10d was collected in August of 2001 (32 months after the Phase I structure was opened to traffic). Data shown in Figure 10d confirm that post-construction displacements of the front of the GRS wall tend to decrease toward the leveling pad. The maximum wall outward displacements seem to occur directly below the bridge footing. The data presented in Figure 10d also shows that the rate of post-construction displacements decreases with time. Outward displacements collected along Section 400 are particularly small after approximately 12 months of service. The average outward displacement along Section 400 from March 2000 to August 2001 is approximately 2 mm.

Figure 11 shows the time history of geogrid strains collected from several strain gages. The data shown in Figure 11 corresponds to strain gages placed along Section 800 and was collected during approximately 28 months after opening the structure to traffic. Geogrid strain results shown in Figure 11 support observations made regarding outward displacements (Figure 10), which show relatively small (though not negligible) post-construction movements. Geogrid strains at location C (toward the back of the bridge footing, Figure 5) are particularly small throughout the entire monitoring period, as shown in the data presented for geogrid layer 6. Geogrid strains on the order of 0.05% accumulate at location B (center of the bridge footing, Figure 5) in geogrid layers 6 and 10 during the six months following opening of the structure to traffic (June to December 1999). Geogrid strains at location B level out after this period and remain approximately constant until September 2000 (approximately 630 days from 1 January 1999), when geogrid strains on the order of 0.02% accumulate during approx-

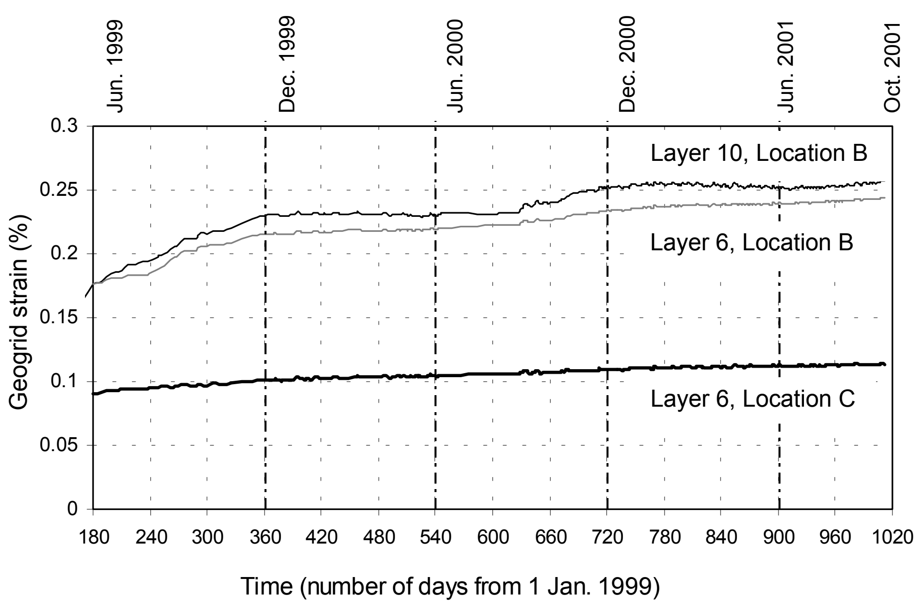

Figure 11. Geogrid strain results obtained along Section 800 below the bridge footing while bridge was in service (Stage VII).

Note: The period shown in the horizontal axis ranges from June 1999 (180 days from 1 January 1999) to October 2001 (1020 days from 1 January 1999).

imately three months (September to December 2000). Geogrid strains at location B level out after December 2000 and have remained approximately constant until the time of preparation of the present paper (October 2001). Although seasonal post-construction straining has been observed, which appears to coincide with the fall season, the magnitude of post-construction straining is small and shows a clear decreasing rate with time.

6 SETTLEMENT OF THE BRIDGE AND APPROACHING ROADWAY STRUCTURES

6.1 Settlement of Bridge Abutment Footing

Survey targets were placed at the bridge abutment walls (Figure 5) to measure settlement of the bridge abutment footing. The maximum surveyed vertical displacements of the bridge footing due to the placement of the bridge superstructure (Stages II to VI) was 13 mm (13 mm on Section 200 and 12 mm on Sections 800). The maximum surveyed vertical displacements of the bridge footing induced while the bridge was in service (until June 2000) were 7, 11, and 10 mm on Sections 200, 400, and 800, respectively. The most conservative estimate of the bridge footing (23 mm) is roughly one-third the tolerable differential settlement considered in design (70 mm). According to the information available thus far, the post-construction settlement of the bridge footing took place during the first year in service, but become negligible after this period.

6.2 Differential Settlement Between Bridge Abutment and Approaching Roadway

The development of differential settlements across the transition section from bridge deck to approaching roadway was monitored using a digital road profiler and surveying (see survey targets on this transition section in Figure 5). Digital profiling was conducted along four lines covering the eastbound and westbound traffic lanes of the east and west abutment walls (i.e., four edges of the bridge superstructure) in February 2000 and November 2001. In addition, surveying data was collected along Section 800 (east abutment, westbound lane) in June 1999 and June 2000. Figure 12 shows relative elevation data collected along the four profiling lines. The elevation data is obtained in relation to the concrete abutment wall, where the relative elevation is zero. Distances from the concrete abutment wall to the concrete approach slab are taken as positive values, while distances to the bridge deck are taken as negative values (Figure 5). Note that the bridge deck is lower than the concrete approach slab across the east abutment and higher than the concrete approach slab across the west abutment.

The results shown in Figure 12 indicate that the transition between the bridge and approaching roadway after almost three years in service is smooth and shows no signs of developing differential settlements between the bridge abutment and the approaching roadway (i.e., “bump at the bridge” problem). The elevation profiles collected at different times essentially match each other, suggesting that settlements have been uni-

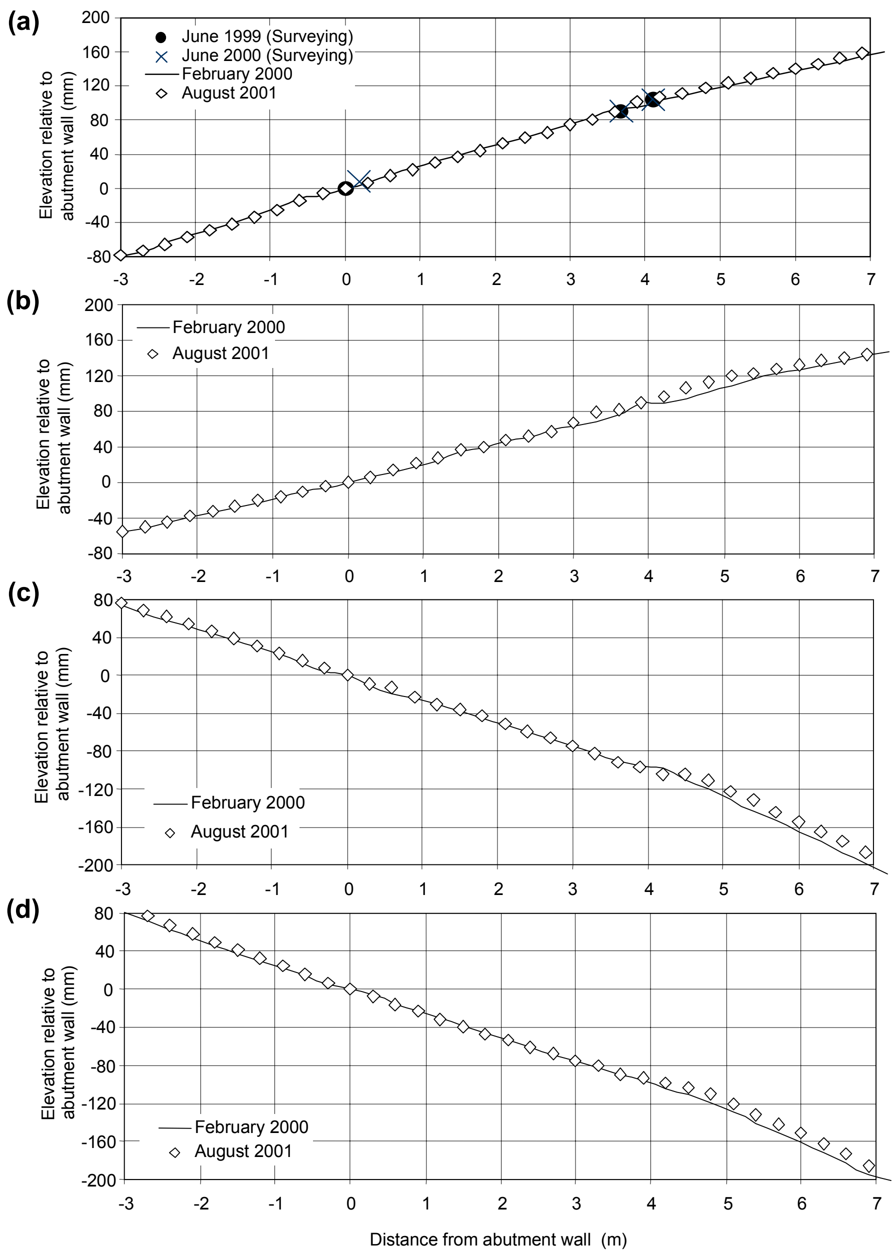

Figure 12. Elevation profiles along the transition from bridge deck to approaching roadway: (a) east bridge abutment, west-bound Lane; (b) east bridge abutment, east-bound lane; (c) west bridge abutment, west-bound lane; (d) west bridge abutment, east-bound lane.

Notes: Distance from bridge abutment is (+) toward the concrete approach slab and (-) toward the bridge deck (Figure 5). All data obtained using digital road profiler, except the June 1999 and June 2000 data, which is survey data.

form. That is, no evidence of differential settlement has been observed between the bridge superstructure and approaching roadway.

7 SUMMARY AND DISCUSSION

Table 1 summarizes the displacement results collected from surveying, an inclinometer, and strain gages for the Founders/Meadows structure along Sections 200, 400, and 800. The summarized displacement information includes: (i) the maximum outward displacements of the front GRS wall facing; (ii) vertical settlement of the front GRS wall facing; and (iii) settlements of the bridge abutment footing. The movements shown in the table were induced during construction of the front GRS wall (Stage I), during placement of the bridge superstructure (Stages II to VI), and during different periods after opening the structure to traffic. The vertical settlement of the wall facing was approximately the same at different elevations. This indicates that most of the wall vertical settlements are due to the settlement of the leveling pad and compression of the joint materials located between the leveling pad and first row of facing blocks. Although not shown in Table 1, the components of the facing displacements parallel to the wall, measured from surveying during all stages, were essentially negligible. The more sensitive inclinometer results along Section 400 also indicated negligible displacements parallel to the wall. These results support the assumption of plain strain conditions at the middle of the Phase I and Phase II structures.

The maximum displacement values obtained from all monitoring techniques and along all monitored sections during different stages, are also shown in Table 1. These movements are normalized with respect to the wall height. Evaluation of the information summarized in Table 1 leads to the following observations regarding the overall deformation response of the GRS system.

- *Movements induced during construction of the front GRS wall.* The wall experienced comparatively large movements during this stage. The relatively large movements can be attributed to the effect of compaction operations, low soil confinement of surficial soils, and the presence of slacks in the geogrid reinforcements. Strain gage results suggest that approximately 50% of the total outward facing displacements induced during all monitored stages (Stages I to VII), occurred during placement and compaction of approximately 1 m of soil over the geogrid layers (approximately 20 kPa vertical soil stress). The maximum front wall outward displacement induced by wall construction was 12 mm, which corresponds to 0.2% of the front wall height. The measured settlement of the leveling pad supporting the front wall facing was approximately 8 mm.
- *Movements induced during placement of the bridge superstructure.* The maximum front wall outward displacement and bridge footing settlement induced by placement of the bridge superstructure were 10 and 13 mm, respectively. These movements correspond, respectively, to 0.17 and 0.29% of the height of the front GRS wall (0.29% is normalized in relation to a wall height of 4.5 m). The measured settlement of the leveling pad supporting the front wall facing was approximately 7 mm.

Table 1. Summary of the maximum movements of the front wall facing and settlements of the bridge abutment footing.

	Induced only by GRS wall construction (Stage I)	Induced only by placement of bridge superstructure [1] (Stages II to VI)	Post-construction movements (in service) [2]		
			6 months	12 months	18 months
Maximum outward displacement of the front wall facing (mm)					
Section 200, survey	---	7	4	---	6
Section 400, survey	9	9	8 [3]	12 [3]	13 [3]
Section 400, inclinometer	---	---	6	11	11 [4]
Section 800, survey	12	10	---	5	---
Section 800, strain gages	11	6	4	4	---
Maximum displacement (% of wall height)	12 (0.2%)	10 (0.17%)	8	12	13 (0.22%)
Settlement of the leveling pad supporting the front wall facing (mm)					
Section 200, survey	---	7	4	---	5
Section 400, survey	6	---	2	5	5
Section 800, survey	8	3	---	3	---
Maximum settlement	8	7	4	5	5
Settlement of the bridge abutment footing (mm)					
Section 200, survey	---	13	7	---	7 [5]
Section 400, survey	---	---	7	11	11 [5]
Section 800, survey	---	12	---	10	---
Maximum settlement (% of wall height)	---	13 (0.29%)	7	11	11 (0.18%)

Notes: [1] Estimated surcharge is 115 kPa. [2] Months in service are counted from December 1998 for Sections 200 and 400, and from June 1999 for Section 800 (estimated surcharge is 150 kPa). [3] Displacements are estimated based on surveying and inclinometer data. [4] Displacement after 32 months was 12 mm. [5] Actual measured settlement was 1 mm less than the value reported in the table.

- *Movements induced after opening the structure to traffic.* The maximum front wall outward displacement and bridge footing settlement induced while the bridge was in service (Stage VII) were 13 and 11 mm, respectively. These movements correspond, respectively, to 0.22 and 0.18% of the front wall height. The measured settlement of the leveling pad supporting the front wall facing was approximately 5 mm. From the time of opening the bridge to traffic (December 1998 for Phase I structure and June 1999 for Phase II structure) until approximately January 2000, the structure experienced post-construction movements. However, additional post-construction movements experienced since January 2000 were negligible. In addition, the front GRS wall abutment has achieved the important design objective of minimizing differential settlements between the bridge abutment and approaching roadway.

CDOT engineers had anticipated that the maximum outward displacement of the front GRS wall and settlement of the bridge footing due to placement of the bridge superstructure would not exceed 20 and 25 mm, respectively. The measured values are well below these anticipated displacements (10 mm of maximum outward displacement and 13 mm of maximum settlement were induced by bridge superstructure placement). According to guidelines from the AASHTO (AASHTO 1996, in the 1997 interim specifications), the two-span Founders/Meadows bridge supported at its abutments by GRS walls could safely tolerate a maximum differential settlement (due to placement of the bridge superstructure and after opening the bridge to traffic) of 70 mm without structural distress. An additional consideration was the need to maintain a 4.95 m minimum clearance between the I-25 Highway and the bottom of the bridge superstructure, which implies that the settlement of the bridge footing should not exceed 100 mm. These maximum settlement criteria are clearly satisfied, as the maximum settlement recorded for the bridge footing is 23 mm (13 mm induced by placement of bridge superstructure plus 10 mm induced after 18 months of service). This suggests that less conservative and more cost-effective design alternatives involving smaller size bridge spread footings and placement of the footing closer to the wall front face may prove feasible. Future research should quantify appropriate bearing capacity for GRS abutments under different loading and field conditions. Additional information regarding the monitoring results of the front GRS wall is provided in the paper by Abu-Hejleh et al. (2001).

8 CONCLUSIONS AND FINAL REMARKS

The deformation response of a geosynthetic-reinforced soil abutment system, the Founders/Meadows bridge abutments, is documented in the present paper. 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 altogether. A comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations. The present paper evaluates the deformation response of this critical structure. The following conclusions can be drawn from this evaluation:

- Based on the overall deformation response of the structure thus far, the structure has shown excellent short- and long-term performance. Specifically, 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 the 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).

- The order of magnitude of displacements reported for this project provides a reference for future projects constructed under similar conditions. In particular, placement of the bridge superstructure induced a maximum outward displacement of the front GRS wall equal to 0.17% of the wall height and a settlement of the bridge footing equal to 0.29% of the wall height.
- Post-construction outward displacements of the structure facing were small and showed a clearly decreasing rate with time. Also, post-construction geogrid strains were small and showed a clearly decreasing rate with time. The post-construction maximum outward displacement of the front GRS wall (after 18 months in service) was 0.22% of the wall height and the post-construction settlement of the bridge footing (after 18 months in service) was 0.18% of the wall height. Post-construction movements became negligible approximately one year after opening of the structure to traffic.

The good performance of the Founders/Meadows system and other GRS abutment structures reported in the literature (FHWA 2000) suggests that the use of reinforced soil walls to support both the bridge and approaching roadway structure is an adequate alternative for bridge abutment projects. In particular, the experience reported in the present paper shows that geosynthetic-reinforced soil bridge abutments work well for multiple span bridges, have the potential for eliminating the "bump at the bridge" problem, avoid disadvantages associated with the use of deep foundations, and allow for construction in stages and within a smaller working area. The use of this system should be limited to projects founded on firm soils and with no scour potential until further research is conducted.

To achieve satisfactory performance of future GRS abutments, CDOT will incorporate the following considerations in future GRS bridge abutments: (i) use a reinforcement layout with a smaller vertical spacing than that used in the Founders/ Meadows Structure; (ii) use well-compacted granular backfill meeting the requirements for CDOT Class 1 backfill; (iii) use reinforcement with stiffnesses consistent with that employed in the Founders/Meadows structure; and (iv) place the backfill behind the abutment wall before the girders. Current criteria for bearing capacity and creep reduction factors for geogrid reinforcement should be evaluated in future studies. Since the maximum settlement of the bridge footing is well below the tolerable differential settlement, a less conservative spread footing design may prove feasible.

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REFERENCES

AASHTO, 1996, "*Standard Specifications for Highway Bridges*", 16th Edition, as amended by 1997 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, DC, USA, 628 p.

AASHTO, 1998, "*Standard Specifications for Transportation Materials and Methods of Sampling and Testing*", 19th Edition, American Association of State Highway and Transportation Officials, Washington, DC, USA, 1124 p.

Abu-Hejleh, N., Outcalt, S., Wang, T., and Zornberg, J.G., 2000a, "*Performance of Geosynthetic-Reinforced Walls Supporting the Founders/Meadows Bridge and Approaching Roadway Structures, Report 1: Design, Materials, Construction, Instrumentation, and Preliminary Results*", Report No. CDOT-DTD-R-2000-5, Colorado Department of Transportation, Colorado, USA, 58 p.

Abu-Hejleh, N., Wang, T., and Zornberg, J.G., 2000b, "Performance of Geosynthetic-Reinforced Walls Supporting Bridge and Approaching Roadway Structures", *Advances in Transportation and Geoenvironmental Systems Using Geosynthetics*, Zornberg, J.G. and Christopher, B.R., Editors, ASCE Geotechnical Special Publication No. 103, proceedings of sessions from GeoDenver 2000, pp. 218-243.

Abu-Hejleh, N., Zornberg, J., Wang, T., McMullen, M., and Outcalt, W., 2001, "*Performance of Geosynthetic-Reinforced Walls Supporting the Founders/Meadows Bridge and Approaching Roadway Structures, Report 2: Assessment of the Performance and Design of the Front GRS Walls and Recommendations for Future GRS Abutments*", Report No. CDOT-DTD-R-2001-12, Colorado Department of Transportation, Colorado, USA, 143 p.

Allen, T.M., Christopher, B.R., and Holtz, R.D., 1991, "Performance of a 12.6 m High Geotextile Wall in Seattle, Washington", *Geosynthetic-Reinforced Soil Retaining Walls*, Wu, J.T.H., Editor, Balkema, 1992, Proceedings of the International Symposium on Geosynthetic-Reinforced Soil Retaining Walls, Denver, Colorado, USA, August 1991, pp. 81-100.

ASTM D 2487, "*Standard Test Method for Classification of Soils for Engineering Purposes*", American Society for Testing and Materials, West Conshohocken, Pennsylvania, USA.

ASTM D 4595, "*Standard Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method*", American Society for Testing and Materials, West Conshohocken, Pennsylvania, USA.

Bathurst, R.J., Walters, D.L, Hatami, K., and Allen, T.M., 2001, "Full-scale performance testing and numerical modelling of reinforced soil retaining walls", *Landmarks in Earth Reinforcement*, Ochiai, H. et al., Editors, Balkema, Vol. 2, Proceedings of the Fourth International Symposium on Earth Reinforcement, IS Kyushu 2001, Fukuoka, Japan, pp. 3-30.

Buttry, K., McCullough, E., and Wetzel, R., 1996, "Temperature and Related Behavior in Segmental Retaining Wall System", *Transportation Research Record 1524*, pp.

19-23.

Elias, V. and Christopher, B.R., 1997, "*Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*", FHWA Demo Project 82-1, Washington, DC, USA, 367 p.

FHWA, 2000, "*GRS Bridge Piers and Abutments*", FHWA-RD-00-038, Sponsored by Turner-Fairbanks Highway Research Center, 146 p.

Zornberg, J.G., Barrows, R.J., Christopher, B.R., and Wayne, M.H., 1995, "Construction and Instrumentation of a Highway Slope Reinforced with High Strength Geotextiles", *Proceedings of Geosynthetics '95*, IFAI, Vol. 1, Nashville, Tennessee, USA, February, 1995, pp. 13-27.