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Monitored Displacements of a Unique Geosynthetic-Reinforced Walls Supporting Bridge and Approaching Roadway Structures

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ABSTRACT A Geosynthetic-reinforced soil (GRS) system was used to support both the abutment shallow footings of the Founders/Meadows two-span bridge and the approaching roadway structures. This unique system has the potential to alleviate the "bump bridge" problem, and allowed for a small work area and construction in stages. This paper provides an assessment of the Founders/Meadows performance and behavior under service loads from displacement data collected through surveying, inclinometer, road profiler, and strain gages. Three sections of the GRS system were instrumented to measure movements of the front GRS wall, settlement of the bridge footing, and differential settlements between the bridge abutment and the approaching roadway. Data was collected during construction of the GRS walls, placement of the bridge superstructure and during 18 months after opening the bridge to traffic. The structure showed excellent performance: monitored overall displacements were smaller than those expected in the design and allowed by performance requirements, there were no signs for development of the bridge bump problem or of any structural damage, and postconstruction movements became negligible a 1 year after opening the bridge to traffic. The influence of compaction, construction season and construction sequence on the structure movements are discussed in this paper. Finally, design implications of the measured results and basis for estimating the movements of structures similar to the Founders/Meadows structure are presented.

INTRODUCTION

The Colorado Department of Transportation (CDOT) successfully completed in July of 1999 the construction of the new Founders/Meadows Bridge near Denver, Colorado. In this unique field application, geosynthetic-reinforced soil (GRS) walls support both the footing of the two-span bridge and the approaching roadway structure. Fig. 1 shows a picture of one of the segmental retaining walls. The "front GRS wall", which supports the bridge superstructure, extends around a 90-degree curve into a "lower GRS wall" supporting the "wing wall" and a second tier, "upper GRS wall". Fig. 2 shows a typical monitored cross-section (Sections 200, 400, and 800) along the front GRS wall and abutment GRS wall. The figure illustrates that the bridge superstructure load (e.g., from girders, bridge deck) is transmitted through abutment wall to a shallow strip footing placed directly on the top of a geogrid-reinforced segmental retaining wall. The centerline of the bridge abutment wall and front edge of the bridge footing are located 3.1 m and 1.35 m, respectively, from the back face of the front GRS wall (Fig. 2). A short reinforced concrete abutment wall and two wing walls, resting on the bridge spread footing, confine the reinforced backfill soil behind the bridge abutment and support the bridge approach slab (see Figures 1 and 2).

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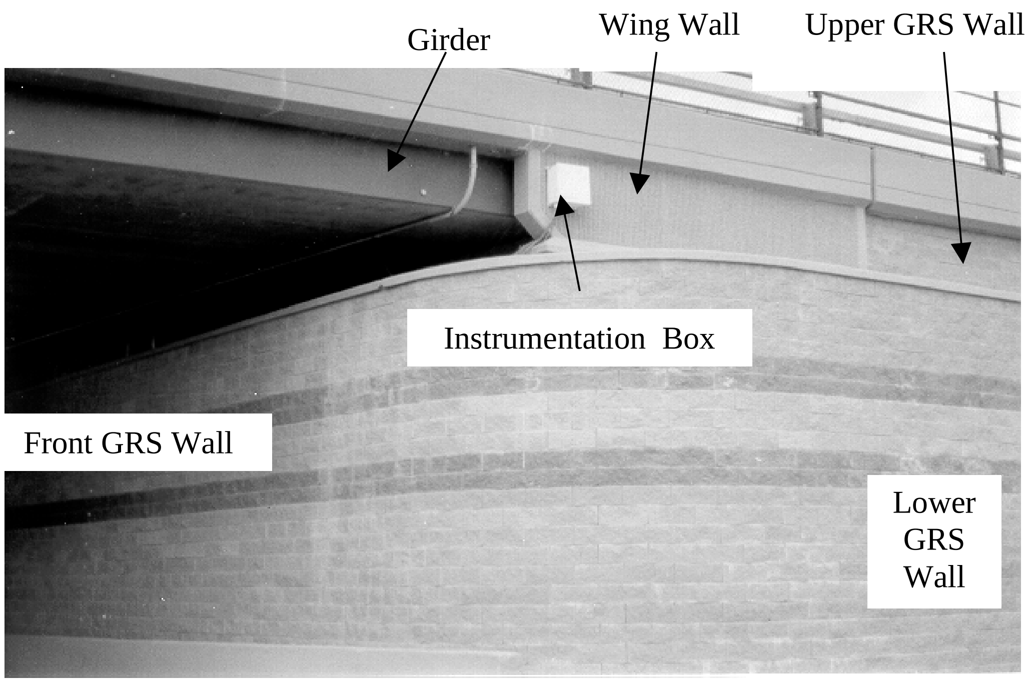


FIGURE 1 View of the South-East Side of the Completed Founders/Meadows Bridge

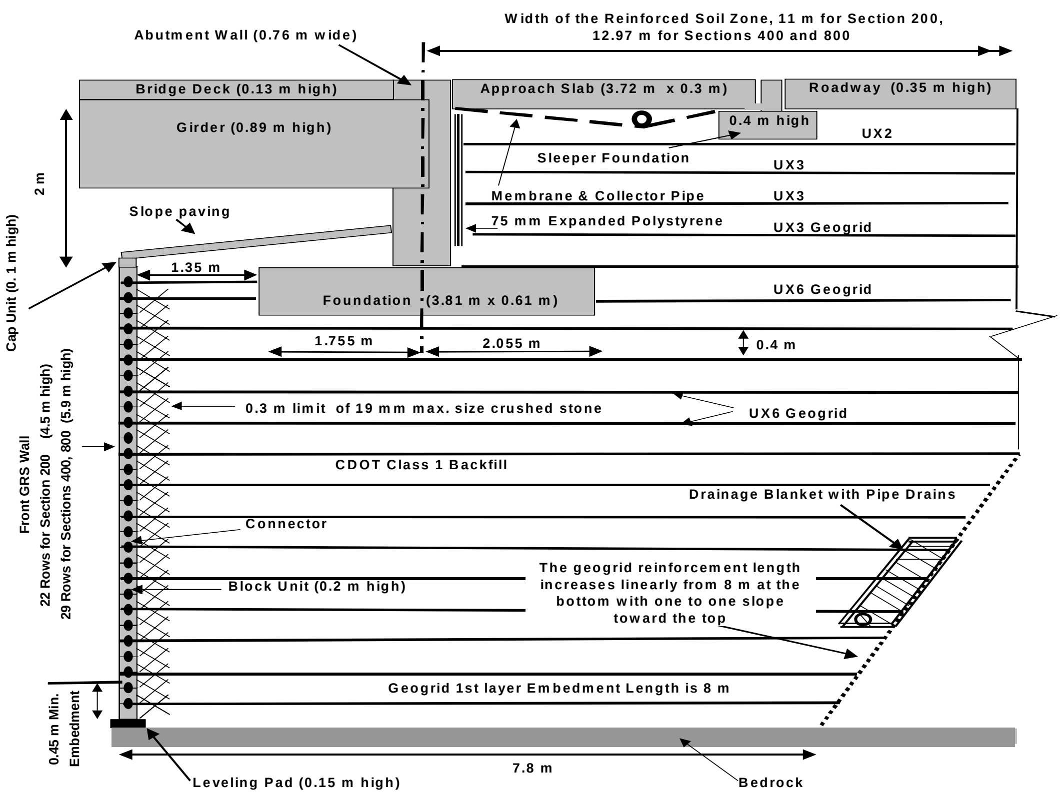


FIGURE 2. Typical Monitored Cross-Section (Sections 200, 400, and 800) through the Front and Abutment GRS walls

The performance of bridge structures supported by GRS abutment like the Founders/Meadows structure have not been tested under actual service conditions to merit acceptance without reservation in highway construction. Consequently, the structure was considered experimental and comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations. Large-size direct shear and triaxial tests were conducted to determine the representative shear strength properties and constitutive relations of the gravelly backfill used for construction. 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 objectives of this investigation are:

- Assessment of the structure's performance and behavior under service loads from short- and long-term movement data.
- Assessment of CDOT and AASHTO design procedures and assumptions regarding the use of GRS walls to support bridge footings, and as a measure to alleviate the bridge bump problem.
- Collection of performance data for future calibration and validations of numerical models.

Previous publications (Abu-Hejleh et al., 2000) summarize the design procedure, material characterization, construction, and instrumentation program of the Founders/Meadows structure. This paper presents a summary and analysis of the collected data at different stages on the movements of the front GRS wall, of the settlements of the bridge footing, and of the differential settlements between the bridge and approaching roadway structures at different stages. This addresses the study 1st objective listed above.

BACKGROUND

Full-scale tests of geosynthetic reinforced soil abutments with segmental block facing have been conducted by the Federal Highway Administration (FHWA) (e.g., Adams 1997) and by CDOT (e.g., Ketchart and Wu 1997). These studies have demonstrated very high load carrying capacity and excellent performance of GRS bridge abutments. In CDOT demonstration project, the GRS abutment was constructed with "roadbase" backfill reinforced with layers of a woven polypropylene geotextile placed at a spacing of 0.2 m. Dry-stacked hollow-cored concrete blocks were used as facing. A load corresponding to a vertical pressure of 232 kPa was applied on the top surface of the 7.6 m high abutment structure. The measured immediate maximum vertical displacement and lateral displacement (defined as elongation of the perimeter of the abutment) were, respectively, 27.1 mm and 14.3 mm. The maximum vertical and lateral creep displacements under a sustained vertical surcharge pressure of 232 kPa for 70 days were 18.3 mm and 14.3 mm, respectively.

When compared to the typical use of deep foundations to support bridge structures, the use of reinforced soil structures to support both the bridge and the approaching roadway structures has the potential to alleviate the bridge "bump" problem (caused by differential settlements between the bridge abutment and approaching roadway). This approach also allows for construction in stages and comparatively smaller work areas. These features, other perceived advantages of GRS systems (e.g., cost-effectiveness, flexibility, etc.), and the excellent past performance of full-scale geosynthetic reinforced abutments convinced CDOT design engineers to select GRS walls to support the bridge abutment shallow footings and the approaching roadway structures of the Founders/Meadows structures. It was expected that the presence of a competent claystone bedrock formation below the base of the reinforced backfill and the use of an extended reinforced zone (see Fig. 2) would provide an adequate external stability for the structure and minimize the anticipated structure settlements. To the authors' knowledge, the design and construction of the Founders/Meadows structure is the first of its kind in conventional U.S. highway practice. CDOT designed this structure in 1996. The Federal Highway Administration (FHWA) published design details for bridge superstructures directly supported by a reinforced soil mass in 1997 (Elias and Christopher 1997).

Three of the commonly causes for development of bridge bumps are addressed in the design of the Founders/Meadows structure. First, uneven settlements between the approaching roadway structure typically constructed on compacted backfill soil and the bridge abutment typically supported on stronger soils by a deep footing. The roadway approach embankment and the bridge footing were integrated with an extended reinforced soil zone (Fig. 2) to minimize this cause of uneven settlement between the bridge abutment and approaching roadway. Second, erosion of the fill material around the abutment wall caused by surface run-off water infiltration into the fill. Several measures were implemented (e.g., placement of impervious membranes with collector pipes as shown in Fig. 2) to prevent surface run-off water and groundwater from getting into the reinforced soil mass and the bedrock at the base of the fill. Third, expansion and contraction of the bridge girders and deck strongly attached to the abutment wall (integral abutment), which cause lateral displacement of the approach backfill. A compressible 75 mm low-density expanded polystyrene sheet was placed between the reinforced backfill and the abutment walls (see Fig. 2). It was expected that this system would accommodate the thermally-induced movements of the bridge superstructure without affecting the backfill (Abu-Hejleh et al., 2000).

The backfill soil used in this project is a mixture of gravel (35%), sand (54.4%), and fine-grained soil (10.6%). The measured liquid limit and plasticity index for the backfill were 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 backfill met the construction requirements for CDOT Class 1 backfill. A friction angle of 34^{o} and zero cohesion, were assumed in the design of the GRS walls. To evaluate these parameters, conventional direct shear tests and large size direct shear and triaxial tests were conducted. In the conventional tests, the 35% gravel portion was removed from the specimens, but in the large-size triaxial and direct shear tests, the backfill soil specimens included the gravel portion. The results of conventional direct shear tests and large size direct shear and triaxial tests indicate that assuming zero cohesion in the design procedure and removing the gravel portion from the test specimens lead to significant underestimation of the actual shear strength of the backfill. The backfill unit weight measured in the field was 22.1 kN/m^3, which exceeds the design value of 19.6 kN/m^3. Hyperbolic model constitutive parameters were determined from the results of the large size triaxial tests.

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 strength. This global reduction factor accounts for reinforcement tensile strength losses over the design life period due to creep (2.7), durability(1.1), and installation damage (1.1). It also includes a factor of safety of 1.78 for uncertainty. Three types of Tensar geogrid reinforcements were used: UX 6 below the footing, and UX 3 and UX 2 behind the abutment wall. The LTDS for these reinforcements is 27 kN/m, 11 kN/m, and 6.8 kN/m, respectively.

INSTRUMENTATION PROGRAM TO MEASURE STRUCTURE MOVEMENTS

The instrumentation program was conducted in two phases: Phases I and II, which 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 Phase I structure and Section 800 is located at the center of Phase II structure. The layout of the instrumented 200, 400, and 800 sections through the front and abutment GRS walls is shown in Fig. 2. Fig. 3 shows the instrumentation of Section 800. The height of the front GRS wall 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 for Section 200. Displacement data were collected during the following stages:

- Front GRS wall construction (Stage I). This stage lasted from July 16, 1998 to September 12 (warm season) for Phase I structure (Sections 200 and 400), and from January 19, 1999 to February 24 1999 (cold season) for Phase II structure (Section 800). Movements induced during this stage can be compensated during wall construction (i.e., before placement of the bridge superstructure).
- Placement of the bridge superstructure. Monitored stages include placement of the bridge footing and girders seat (Stage II), placement of girders (Stage III), placement of reinforced backfill behind the abutment wall (Stage IV), placement of the bridge deck (Stage V), and placement of minor structures (Stage VI). The total average vertical contact stress exerted directly underneath the bridge footing by the end of this stage was estimated as 115 kPa. This stage was completed on December 16, 1998 for Phase I structure, and on June 30, 1999 for Phase II Structure.
- After opening the bridge to traffic (Stage VII). The total average vertical contact stress exerted directly underneath the bridge footing during this stage was estimated as 150 kPa. Data presented in this paper were collected till the end of Stage VII, June 2000 (i.e., during approximately the first 18 months and 12 months after opening Phase I and Phase II structures, respectively, to traffic).

Continued monitoring is being performed by the time of preparation of this paper. Description of the techniques employed to obtain displacement data of the monitored sections follows.

Surveying

Survey targets made of reflectors were permanently glued to the outside face of front and abutment walls (all sections), bridge deck, and approaching slab and roadway (only Section 800, see Fig. 3). A surveying instrument was used to collect data for the northing, easting, and elevation coordinates of surveying targets at different stages. The northing-easting movements were separated into two displacement components: prependicular 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 surveying was approximately +/- 3 mm.

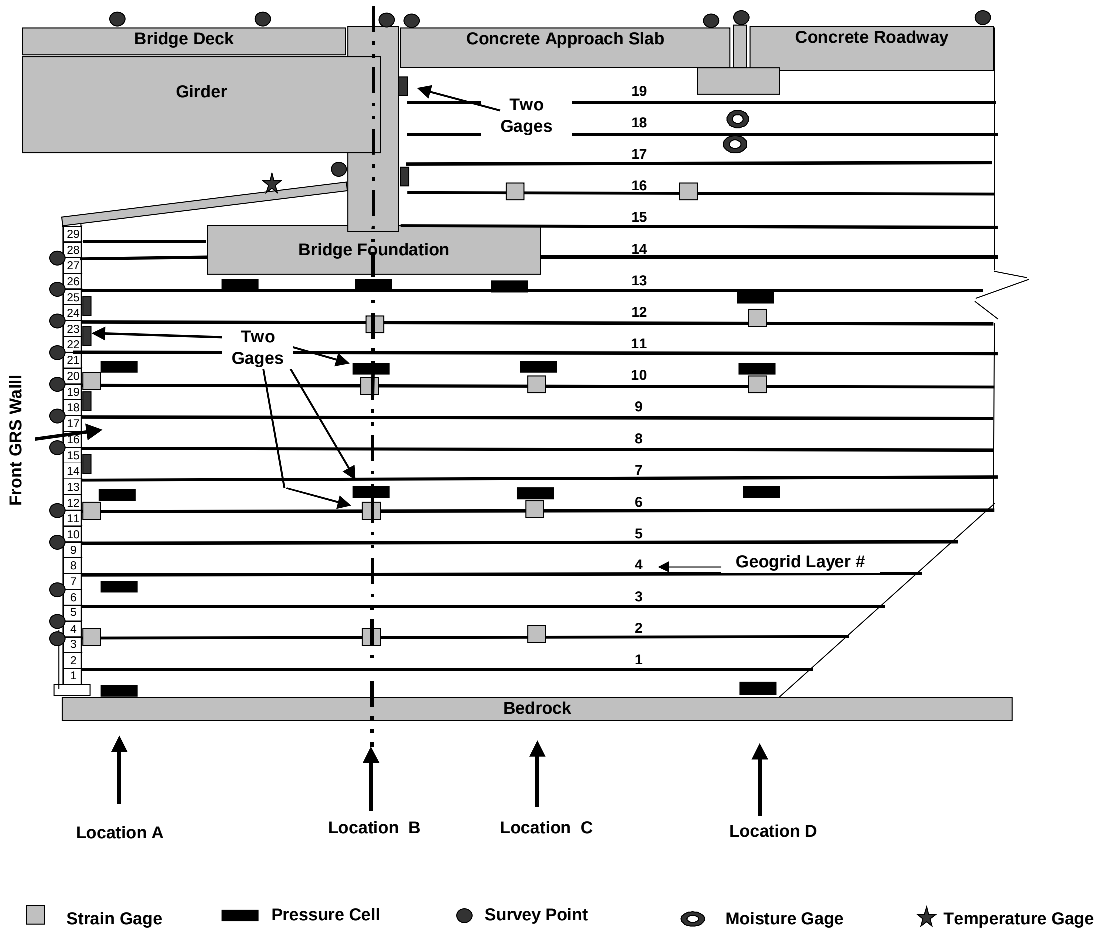


FIGURE 3 Instrumentation Plan of Section 800

Inclinometer

A vertical inclinometer tube was affixed to the back of the facing blocks of Section 400 of the Phase I structure. 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 and held in place by the fill material and the back of the blocks.

Road Profiler

This device is manufactured by Face Construction Technologies, Inc., of Norfolk Virginia. It was used to draw an elevation profile of the top surface of the transition section between bridge deck and approaching roadway structure (Figs. 2 and 3) in order to detect the potential development of the bridge bump problem.

Strain Gages

Some of the geogrid layers in Section 800 were instrumented with strain gages along four critical locations (see Fig. 3): Location A close to the wall facing, Locations B close to the centerline of the bridge abutment wall, Location C close to the back edge of the bridge footing, and Location D behind the bridge footing (approximately 7.5 meter behind the wall facing). Average geogrid strains at layers 6 and 10 along the 7.5 m wide reinforced soil mass were calculated, at different construction stages, using the geogrid strain values along Locations A, B, C, D, and wall facing. The geogrid outward displacements at the facing were obtained, at different stages, by integrating the geogrid strains along each layer, and assuming that the retained backfill did not move. These displacements assumed to represent the outward displacements of the front wall facing.

FACING MOVEMENTS OF THE FRONT WALL

Table 1 summarizes the front GRS wall movements (outward and vertical) along Sections 200, 400 and 800 measured from surveying, inclinometer, and strain gages during construction of the front GRS wall (Stage I), placement of the bridge superstructure (Stages II to VI), and after opening the structure to traffic (Stage VII). The displacements parallel to the wall measured from surveying during all stages were very small. The more sensitive inclinometer results along Section 400 indicated that the fill displacements parallel to the wall are negligible. These findings support the assumption of plain strain at the middle of Phase I and Phase II structures along the monitored sections. The vertical settlement of the wall facing at different heights was approximately uniform. 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 1st row of facing blocks. The measured settlements of the leveling pad supporting the front wall facing were approximately 8 mm during wall construction, up to 7 mm during placement of the bridge superstructure, and up to 5 mm developed while the bridge was in service for 18 months. This is shown in Table 1, which reports a total estimated settlement for the leveling pad over its entire history of 20 mm.

Outward Displacements Induced during Front Wall Construction (Stage I)

Fig. 4a summarizes the outward wall displacement data measured during construction of the front GRS wall. The figure shows the outward displacements of Section 400, monitored along the bottom 18 rows (elevations 0 to 3.65 m above leveling pad), which resulted from increasing the wall height from 3.65 m to 5.5 m. The figure also shows the outward displacements of

Section 800, monitored along the bottom 12 rows (elevations 0 to 2.44 m above the leveling pad), which resulted from increasing the wall height from 2.44 m to 5.5 m. The estimated wall outward displacement along Section 800 obtained from layer 6 strain gages, which resulted from increasing the wall height from 2.23 m to 5.28 m, and from layer 10 strain gages, which resulted from increasing the wall height from 3.85 m to 5.28 m, are also shown in Fig. 4a. It is important to note that the different sets of movement data shown in Fig. 4a correspond to *differen**t*** loading conditions. The maximum wall outward displacements measured during construction of the front GRS wall of sections 400 and 800 were 9 mm, and 12 mm, respectively (Table 1).

Table 1: Summary of the Maximum Movements of the Front Wall Facing and of the Settlements of the Bridge Abutment Footing

A. Maximum Wall Outward Displacement (mm)					
	Induced ***Only*** by GRS Wall Construction (Stage I)	Induced ***Only*** by Placement of Bridge Superstructure (Stages II to VI), (115 kPa surcharge)	Induced ***Only*** While Bridge was in Service (Stage VII) (150 kPa surcharge)		
			6 months	12 months	18 months
A. Maximum Outward Displacement (mm) of the Front Wall Facing					
Section 200, Surveying		7	4		6
Section 400, Surveying	9	9	8*	12*	13*
Section 400, Inclinometer			6	11	10
Section 800, Surveying	12	10		5	
Section 800, Strain Gages	11	6	4	4	
Maximum Displacements (mm) & % of Wall Height	12 0.2 %	10 0.17 %	8	12	13 0.22 %
B. Settlement (mm) of the Leveling Pad Supporting the Front Wall Facing					
Section 200, Surveying		7	4		5
Section 400, Surveying	6	7	2	5	5
Section 800, Surveying	8	3		3	
Maximum Settlement	8	7	4	5	5
C. Bridge Abutment Footing Settlement (mm)					
Section 200, Surveying		13	7		6
Section 400, Surveying			7	11	10
Section 800, Surveying		12		10	
Maximum Settlement (mm) & % of Wall Height		13 0.29 %	7	11	10 0.17 %

*Estimated based on surveying and inclinometer data

Outward Displacements Induced by Bridge Superstructure (Stages II to VI)

Fig. 4b summarizes the outward wall displacement data of the front wall measured during placement of the bridge superstructure from surveying and strain gages. Fig. 4b and Table 1 indicate that the maximum wall outward displacements experienced along sections 200, 400, and 800 during placement of the bridge were approximately 7 mm, 9 mm, and 10 mm, respectively. Maximum displacements seem to occur within the upper third of the wall below the bridge footing. The results suggest good correlation between the wall outward displacements measured for Section 200 (height of 4.5 m) and Section 800 (height of 5.9 m). Sections 400 and

800 have an identical configuration and similar loading. However, Section 800 showed larger movements than Section 400 (Fig. 4b). This could be attributed to two factors:

- Construction Season. The front GRS walls of Phase I structure (Section 400) was constructed during a warm season but 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 the thawing and wetting seasons started. This may have led to softening of the backfill and comparatively larger deformations in Section 800.

- Construction Sequence. The backfill behind the abutment wall was placed before the girders during construction of Section 400. Instead, the girders were placed before placement of the backfill behind the abutment wall during construction of Section 800. This induced, most probably, larger reinforcement strains and lateral displacement within the GRS backfill along Section 800.

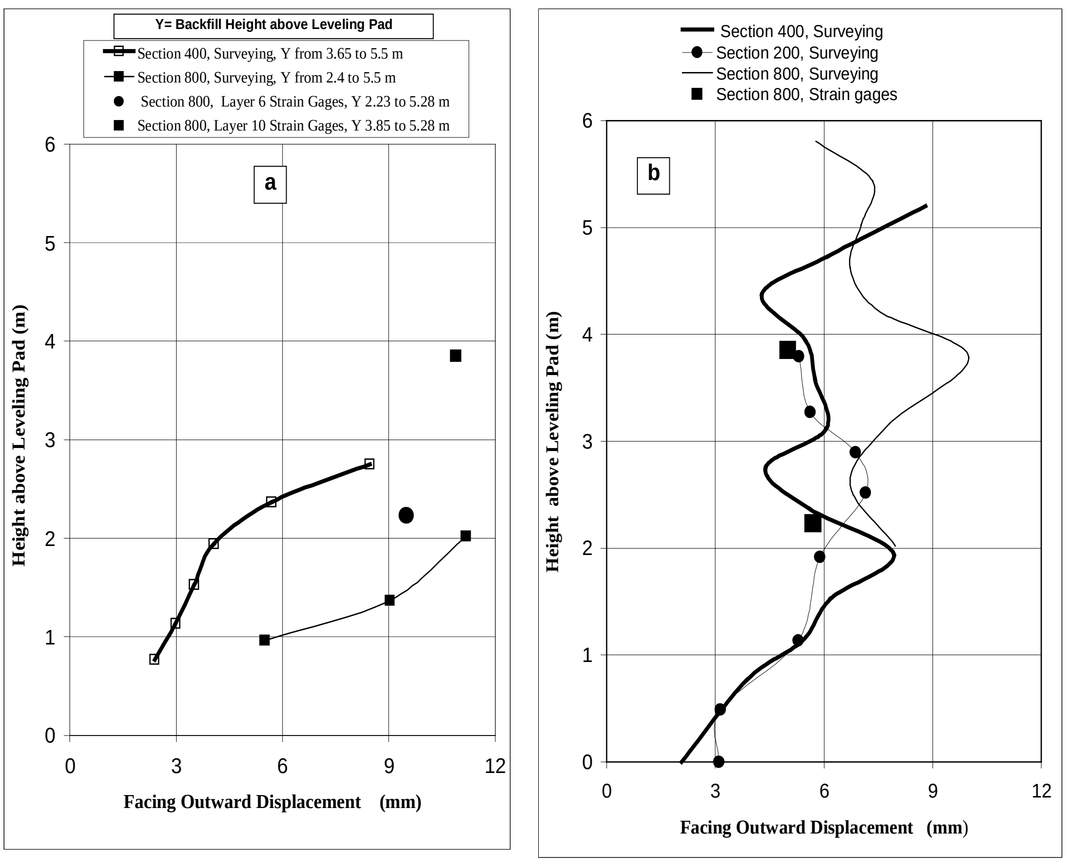


FIGURE 4 Measured Outward Displacements of the Front Wall Facing: (a) Induced during GRS Front wall Construction (sets of data correspond to different loading conditions) , and (b) Induced by Placement of the Bridge Superstructure

Outward Displacements Induced during all Construction Stages Inferred from Strain Gages

Fig. 5 shows the estimated average geogrid lateral strains and the outward displacements of front wall facing along geogrid layers 6 and 8 of Section 800 as a function of the vertical soil stress applied on these layers during all construction stages (I to VI). The vertical soil stresses were estimated as:

$$\sigma_v = \gamma z + \Delta\sigma \quad \text{.....(1)}$$

where γ is the measured backfill unit weight (22.1 kN/m^3), z is the backfill height above the layer (below the bridge footing), and $\Delta\sigma_v$ is the vertical stress increment developed within the soil mass by concentrated surcharge loads. Fig. 5 shows a good agreement between the average geogrid strains at different depths when plotted as a function of the applied vertical soil stresses. However, for the same level of applied vertical soil stress, the wall outward displacements increase with the height above the leveling pad. This is an expected behavior because the width of the mobilized soil zone to resist lateral earth pressures increases with the height above the leveling pad. The 2nd data point in Figs. 5a and 5b was collected after compaction and placement of approximately 1 m of backfill (corresponding to approximately 20 kPa of vertical pressure) over the gages. The 3rd data point corresponds to the end of Stage I (front GRS wall) construction. The 4th data point corresponds to the end of Stage II construction and so on. The last data point refers to the end of the construction stages (Stage VI). The results of Layers 6 and 10 strain gages (see Table 1 and Fig. 5) suggest that up to 33% and 50% of the total wall displacements, induced during all construction and post-construction stages, occurred during the placement and compaction of the initial 1 m and 2 m of backfill, respectively, over the geogrid layers.

During Stages II to IV, the GRS system responded with comparatively small deformations to the increasing level of applied vertical soil stresses. Possible reasons for this behavior are the influence of compaction experienced in the previous stage (Stage I), and the occurrence of these stages (II to IV) during the winter season. Buttry et al. (1996) noticed rigid behavior for a GRS structure during the winter season. During Stages V and VI (last three data points in Fig. 5b), the GRS system responded with comparatively large displacements to the increasing level of applied vertical soil stresses. Results of pressure cells and strain gages, that will be presented in future publications, indicate that these displacements moved the structure to a more stable form and mobilized significantly the geogrid tensile resistance at locations far from the wall facing. Possible reasons for the relatively large movements are the occurrence of the thawing and wetting seasons during these stages, which may have led to the softening of the backfill, and the disappearance of the compaction influence. The observations reported by Buttry et al. (1996) for the influence of seasonal changes on the behavior of GRS structures are similar to the observations listed above.

Outward Displacements of Front Wall Facing Induced While Bridge in Service (Stage VII)

Stage VII (after opening the bridge to traffic) covers the period from December 1998 to June 2000 for Phase I structure (Sections 200, and 400) and the period from June 1999 to June 2000

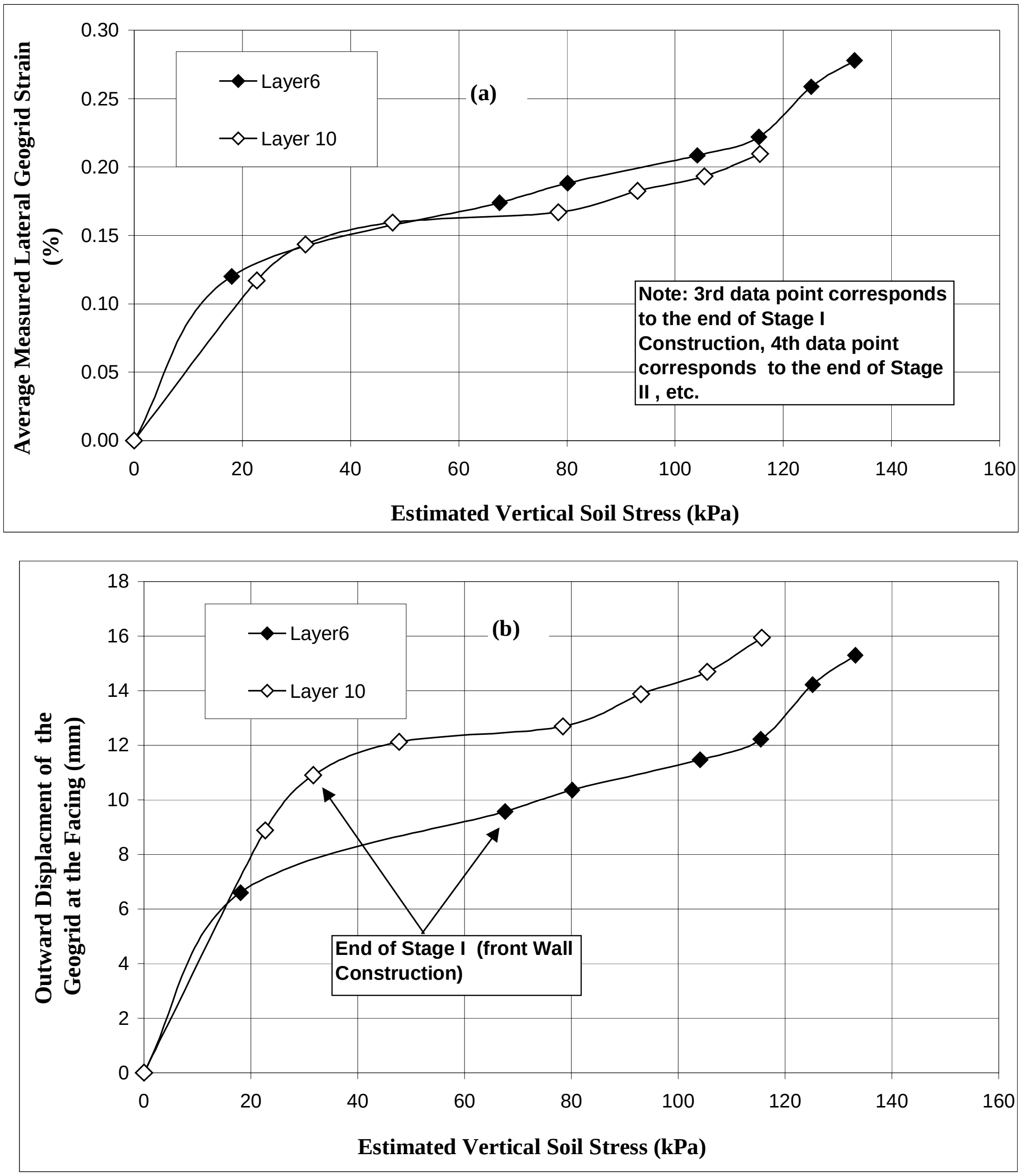


FIGURE 5 Strain Gages Results from Geogrid Layers 6 and 8 of Section 800 during all Construction Stages (Stage I to Stage VI): a) Average Lateral Geogrid Strain between the Front and Back of the GRS Wall, and b) Estimated Outward Displacements of the Geogrid Layers at the Facing

for Phase II (Section 800) structure. The front wall outward displacements measured along Sections 200 and 800 during Stage VII are shown in Figs. 6a and 6b, respectively. The data obtained from the inclinometer along Section 400 represent the movements of the wall relative to the leveling pad, not the total "absolute" movements of the wall. Since the outward displacements of the leveling pad during placement of the bridge superstructure were small (Fig. 4b), the leveling pad displacements were neglected in the analysis of the inclinometer results. Fig. 6c shows the front wall outward displacement profile along Section 400 measured from the inclinometer at different times during Stage VII. Surveying results for Section 400 from December 1998 to June 1999 are not presented because the surveying control point that monitored Section 400 was relocated during this period. Surveying results for Section 400 displacements from June 1999 to January 2000 and from June 1999 to June of 2000 are shown in Fig. 6d. The surveying results show good correlation with the inclinometer results (Fig. 6d).

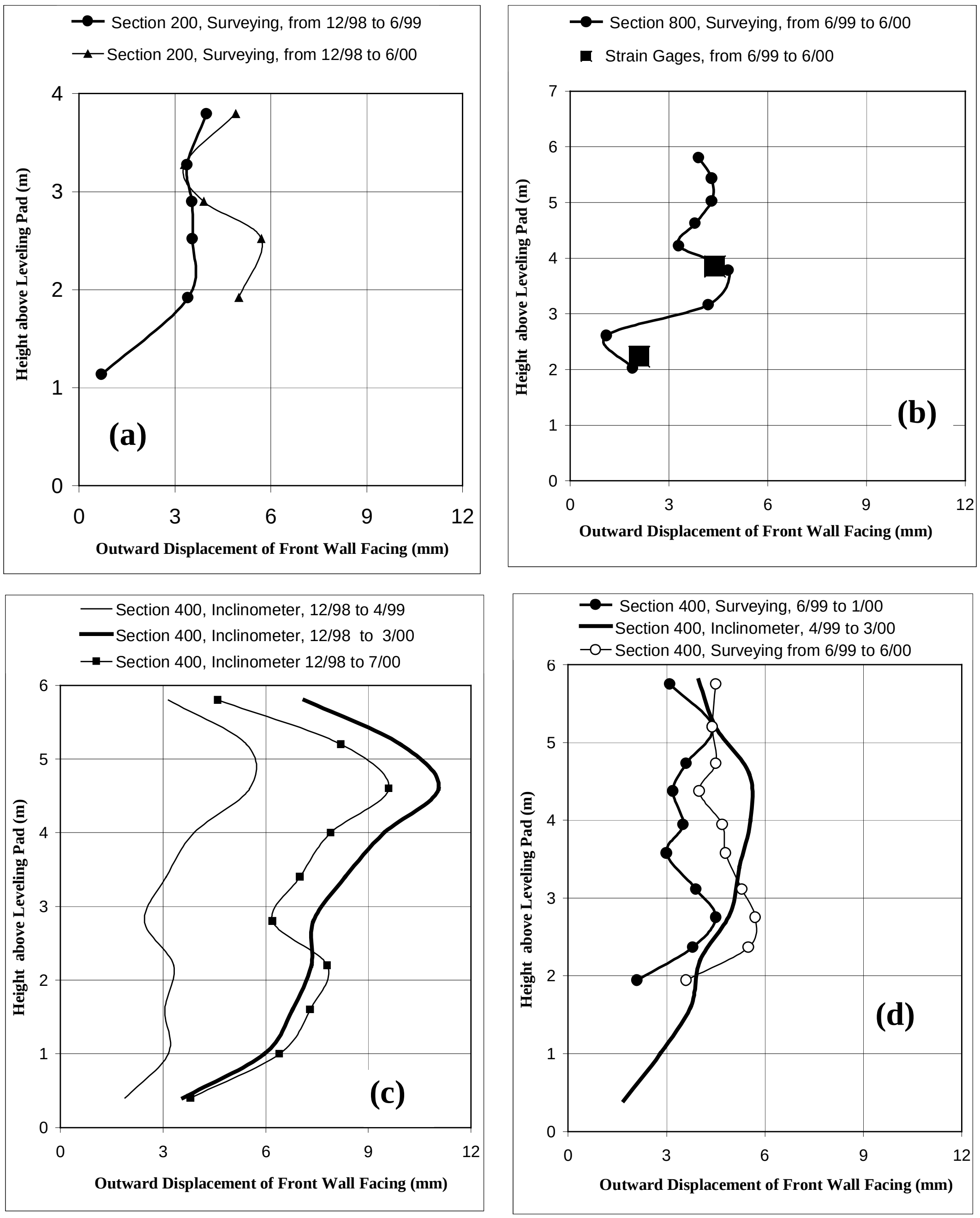


FIGURE 6 Measured Outward Displacements of Front Wall Facing Induced while the Structure in Service: (a) Section 200 from Surveying (b)Section 800 from Surveying and Strain Gages, (c) Section 400 from Inclinometer, and (d) Section 400 from Inclinometer and Surveying

Excellent agreement existed between the front wall displacements along Section 800 inferred from layers 6 and 10 strain gages and from surveying (see Fig. 6b). Fig. 7 shows some results of the geogrid strain measurements with time. The results were obtained from strain gages placed along geogrid layers 6 and 10 (referenced with the first digit in Fig. 7) along different locations (referenced with last letter in Fig. 7) of Section 800. The strain data was collected within one year after opening the structure to traffic (Stage VII).

The results in Fig. 6 suggest that the wall displacements decreased toward the leveling pad. The maximum wall outward displacement seems to occur directly below the bridge footing. The results in Figs. 6 and 7 and Table 1 indicate that Phase I structure (Sections 200 and 400) continued to move during the first 12 month of service and that Phase II structure (Section 800) continued to move during the first 6 months of service. These displacements were relatively small for Sections 200 and 800 (less than 6 mm) and large for Section 400 (12 mm). Possible causes for these movements are traffic load, creep, and seasonal and temperature changes. From January to June 2000, the results along all monitored sections from surveying and inclinometer suggest that the front wall experienced negligible movements (Table 1 and Fig. 6). This is also supported by geogrid strain readings in Fig. 7, which indicate that the strains measured from January 1999 (approximately 365 days from Jan. 1, 1999 in the figure) to June 2000 leveled out and remained approximately constant.

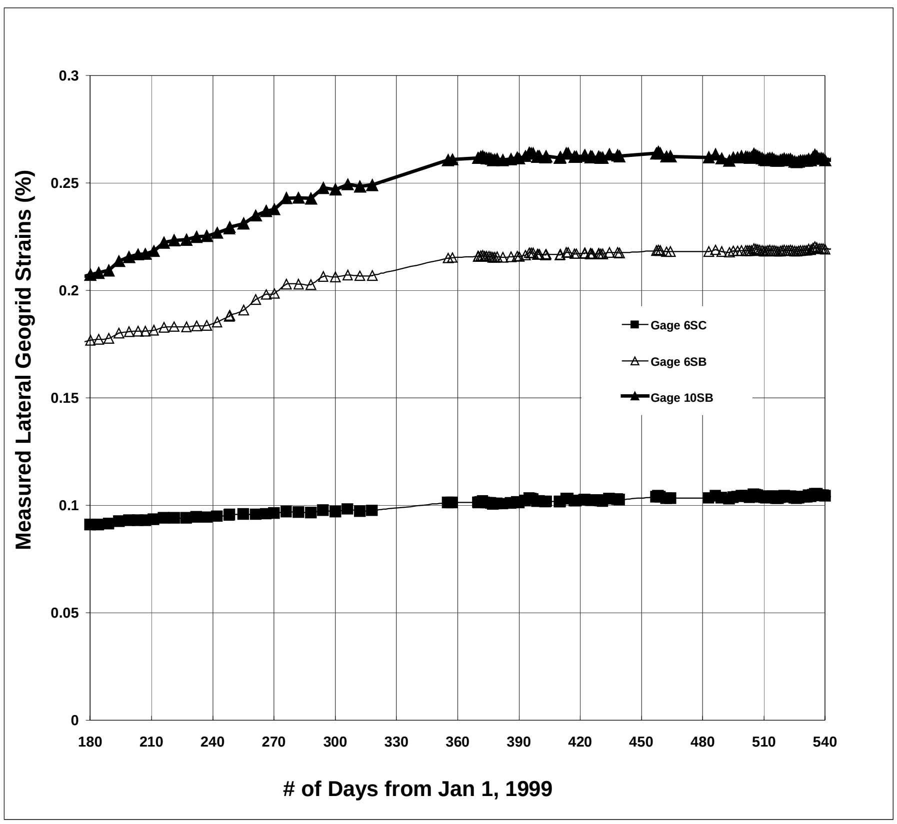


FIGURE 7 Geogrid Strain Gage Results obtained along Section 800 below the Bridge Footing While Bridge in Service (Stage VII)

SETTLEMENT RESULTS FOR THE BRIDGE AND APPROACHING ROADWAY STRUCTURES

Bridge Abutment Footing

Table 1 summarizes the measured settlement results of the bridge abutment footing along the monitored sections. The maximum measured vertical settlements of the bridge footing due to the placement of the bridge superstructure were 13 and 12 mm along Sections 200 and 800, respectively. The maximum measured vertical settlements of the bridge footing induced while the bridge was in service (Stage VII) along Sections 200, 400, and 800 were 7, 11, and 10 mm respectively. Table 1 indicates that the bridge footings continued to settle over the first 12 months of service. The bridge footing of Phase I structure (Sections 200 and 400) experienced negligible settlements from January to June 2000.

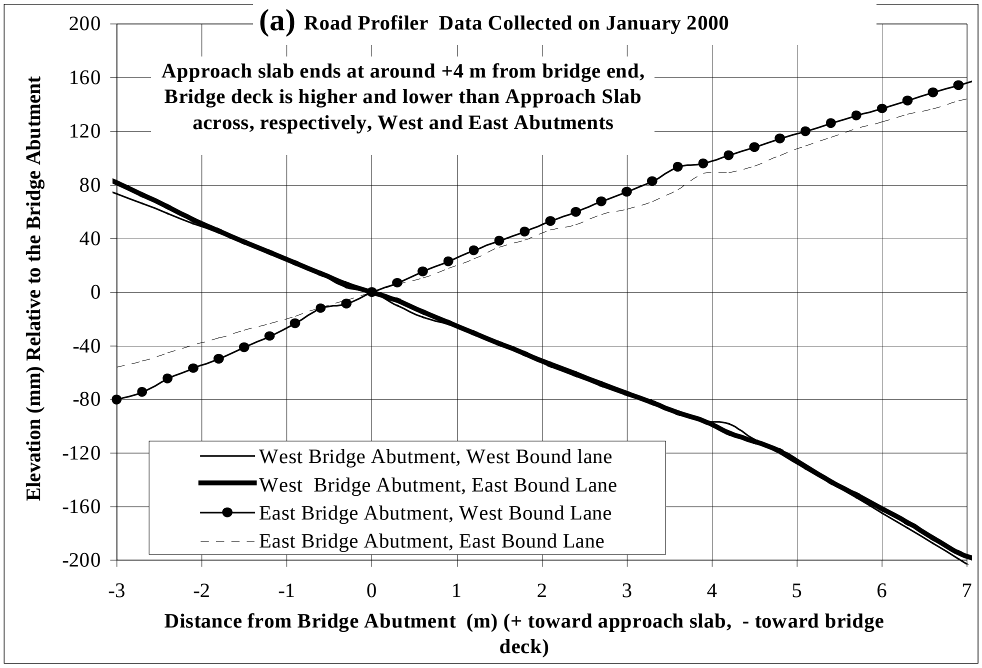
Differential Settlement between Bridge Abutment and Approaching Roadway

The road profiler technique and surveying were used to obtain an elevation profile of the top surface of the transition section between bridge deck and approaching roadway through the approach slab (see Figs. 2 and 3). Profiling was conducted along the east and west bound traffic lanes of the structure, across the east and west abutment walls. Surveying was conducted along Section 800 (Fig. 3) located across the east abutment wall, west bound traffic lane. The elevation data of the transition sections relative to the current bridge abutment top elevation (zero for the abutment wall), obtained at various times during Stage VII, are shown in Fig. 8. Note that the bridge deck is higher than the approach slab for the sections across the west abutment and lower than the approach slab for the sections across the east abutment (see Fig. 8). Distances from the end of the bridge abutment to the roadway are taken positive and to the bridge deck direction are taken negative (see Fig. 8).

The graphical results clearly indicate that the transition between the bridge and approaching roadway is smooth and shows no signs of developing a bridge bump. Slight changes in the elevation grade occurred between the approaching roadway and approach slab of the east abutment, west bound lane (Fig. 8). Results of Fig. 8b suggest also almost even settlements between the bridge abutment wall and the approaching roadway.

SUMMARY OF THE STRUCTURE MOVEMENTS

The maximums measured wall outward displacements and bridge footing settlements from all techniques and along all monitored sections are listed in Table 1. These movements are normalized with respect to the wall design height. Monitoring results of the structure movement (Table 1, Figs. 5b and 8) show the following overall response:



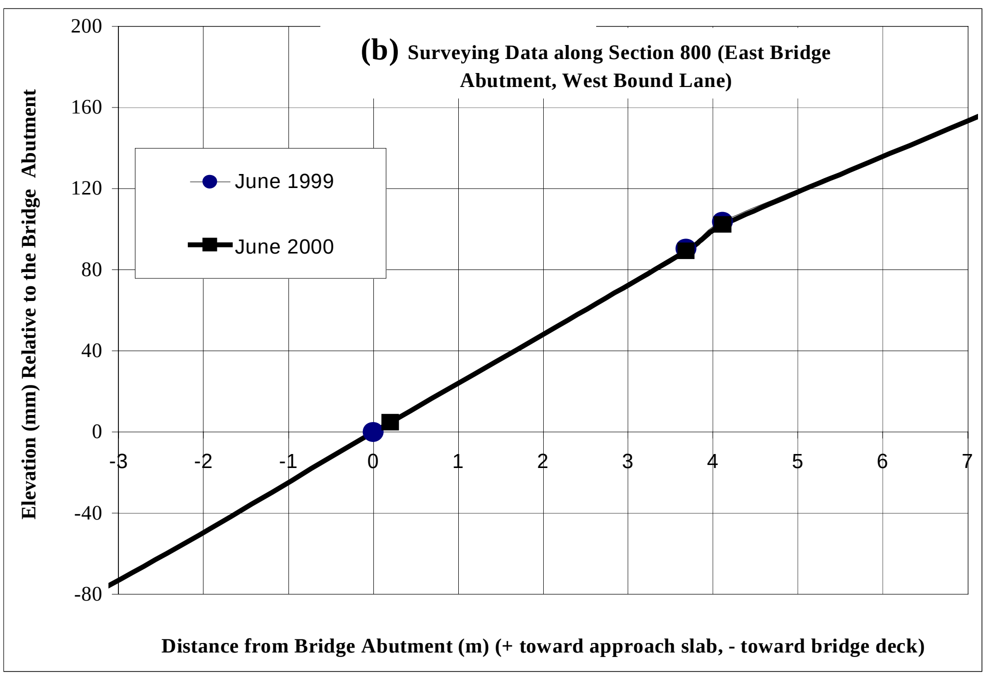


FIGURE 8 Measured Elevation Profiles Relative to the Bridge Abutment along the Top Surface of the Transition Section from Bridge Deck to Approaching Roadway: (a) Road Profiler Data, and (b) Surveying Data

- Induced by construction of the front GRS wall. The wall experienced comparatively large movements during this stage. The relatively large movements were attributed to compaction operations and presence of slacks in the geogrid. Compaction loads induced relatively large locked-in lateral strains in the backfill and geogrid.

- ♦ The results of strain gages suggest that more than 33% and 50% of the total outward displacements of front wall facing, that induced during all monitored stages (Stages 1 to VII), occurred during placement and compaction of approximately the initial 1 m (~ 20 kPa of vertical stress) and 2 m (~40 kPa) of backfill, respectively, over the geogrid layers.
- ♦ The maximum measured 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.

- Induced by placement of the bridge superstructure. The maximum measured bridge footing settlement and front wall outward displacement induced by placement of the bridge superstructure were 13 and 10 mm, respectively. These movements correspond, respectively, to 0.29 % and 0.17% of the front wall height. The measured settlement of the leveling pad supporting the front wall facing was approximately 7 mm.
- After opening the structure to traffic. The maximum measured bridge footing settlement and front wall outward displacement induced while the bridge was in service for 12 to 18 months (until June 2000) were 10 and 13 mm, respectively. These movements correspond, respectively, to 0.17 % and 0.22 % 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 January 2000, the structure showed continued movements.
- ♦ From January to June of 2000, the results along all monitored sections from all techniques suggest that the front GRS wall and bridge footing experienced negligible movements. The geogrid strain readings leveled out and remained approximately constant during this period.
- ♦ Elevation profiling results show no signs of development of the bridge bump problem and indicate even settlements between the bridge and its approaching roadways.

According to AASHTO 1996 guidelines, the two-span Founders/Meadows bridge supported at its abutments by GRS walls could safely tolerate a maximum long-term differential settlement (due to placement of the bridge superstructure and after opening the bridge to traffic) of 50 mm without serious structural distress. To maintain the 4.95 m minimum clearance between I-25 and the bottom of the bridge superstructure, the maximum tolerated settlement of the bridge footing is 100 mm. A total maximum bridge footing settlement of 23 mm was measured due to the placement of the bridge superstructure and after a service period of 18 months (Table 1). CDOT engineers expected that the maximum settlement of the bridge footing and outward displacement of the front GRS wall due to placement of the bridge superstructure would not exceed, respectively, 25 mm (measured 13 mm) and 20 mm (measured 11 mm). Therefore, it is concluded that:

- The Founders/Meadows bridge structure showed excellent short- and long-term performance: the monitored movements were significantly smaller than those expected in design or allowed by performance requirements, there were no signs for development of the bridge bump problem or any structural damage, and post-construction movements became negligible after a service period of 1 year.

DESIGN IMPLICATIONS

- The use of reinforced soil walls to support both the bridge and approaching roadway structure is an adequate alternative for field and loading conditions similar to those encountered in the Founders/Meadows structure. Current limitations of this application include the need of firm foundation for the GRS backfill and no scour potential. The most interesting features of this application include:

  1. Works well for multiple span bridge
  2. Has the potential for eliminating the bridge bump problem.
  3. Disadvantages associated with the use of deep foundations are avoided.
  4. Allows for construction in stages and a smaller work area.

- Fig. 5 and Table 1 provide insight on the range of movements that might be expected during and after construction of structures similar to the Founders/Meadows structure.
- Preliminary results of all techniques employed to measure structure movements show very low creep potential of the geogrid reinforcements. The total geogrid strains developed during all construction and post-construction stages inside the front GRS wall along Section 800 were small and remained almost constant after January 2000. Note that a creep reduction factor of 2.7 was adopted in the design to determine the long-term design strength of geogrid reinforcements from their ultimate strength.
- Compaction operations, construction season and construction sequence can influence the magnitude of movements experienced by the structure. Strain gages results suggest that more than 50% of the total front GRS wall lateral displacements occurred during the placement and compaction of approximately the initial 2 m (~40 kPa) of backfill. The backfill behind the abutment wall should be placed before the girders to minimize outward movements. Construction of the GRS backfill during the warm season is recommended.
- The maximum recorded settlement of the bridge footing (25 mm) is roughly one half the tolerable differential settlement (50 mm). Consequently, less conservative and more cost-effective design alternatives involving reduction of the bridge spread footing size 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.

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