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DESIGN ASSESSMENT OF THE FOUNDERS/MEADOWS GRS ABUTMENT STRUCTURE

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ABSTRACT: Front abutment Geosynthetic-reinforced soil (GRS) walls were constructed to support the shallow footings of a two-span bridge and the embankment approach roadway structures. A key element in the design was the need to support the concentrated loads from the bridge footing and to alleviate the "bump at the bridge" problem. Past publications summarized the design, materials, construction, instrumentation, and the overall movement performance of this structure. The focus of this paper is to evaluate briefly the loading response of the front abutment GRS walls under service loads based on measured lateral earth pressures against the wall facing, vertical earth pressures, and geogrid reinforcement strains inside the wall. Data was collected during construction of the GRS wall, during five stages of bridge superstructure construction, and during 33 months after opening the bridge to traffic. This paper also presents an assessment of the design of the front abutment GRS walls. The overall loading response of the front abutment GRS wall was excellent. The measured loads, especially behind the wall facing, were below the design values, and the overall stability of the structure as measured by load eccentricity was much greater than projected in the design. The use of GRS walls to support both the bridge and approaching roadway structure has been approved by Colorado DOT (CDOT) as a viable foundation support system in future bridge abutment projects. Finally, preliminary recommendations for design and construction of future GRS abutments are provided.

1 INTRODUCTION

Geosynthetic-reinforced soil (GRS) systems have 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 differential settlements without structural distress. A comparatively new application of this technology is the use of GRS abutments in bridge applications. In this application, the reinforcement tensions and soil stresses are mobilized to different and higher levels compared to GRS walls supporting self-weight. When compared to 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 approach roadway fill.

The most prominent GRS abutment for bridge support in the U.S. is the new Founders/Meadows Parkway structure that carries Colorado State Highway 86 over U.S. Interstate 25 in Denver, Colorado (Figure 1). Figure 1 shows the segmental retaining wall system located at the southeast

side of the bridge. 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. Figure 2 illustrates how the bridge superstructure load (from girders, bridge deck) is transmitted through abutment back walls to a shallow strip footing placed directly on the top of a geogrid-reinforced modular block faced retaining wall (“front abutment GRS wall”). The front abutment GRS wall also supports the reinforced embankment of the approaching roadway structure. Figure 1 shows that the front abutment GRS wall extends around a 90-degree curve into a “lower GRS wall” that supports the “wing wall” and a second tier, “upper GRS wall”. The bridge is also supported by central pier columns, which are supported by spread footings founded on bedrock at the median of U.S. Interstate 25. Each span of the new bridge is 34.5 m long and 34.5 m wide, with 20 side-by-side prestressed box girders. The new bridge is 13 m longer and 25 m wider than the previous structure. It accommodates six traffic lanes and sidewalks on both sides of the bridge. The centerline of the bridge abutment wall and front edge of the bridge footing are located 3.1 m and 1.35 m, from the facing of the front abutment wall. Width and height of the bridge footing are 3.8 m and 0.61 m, respectively. A short reinforced concrete abutment backwall and two wing walls confine the reinforced backfill soil behind the abutment wall and support the approach slab.

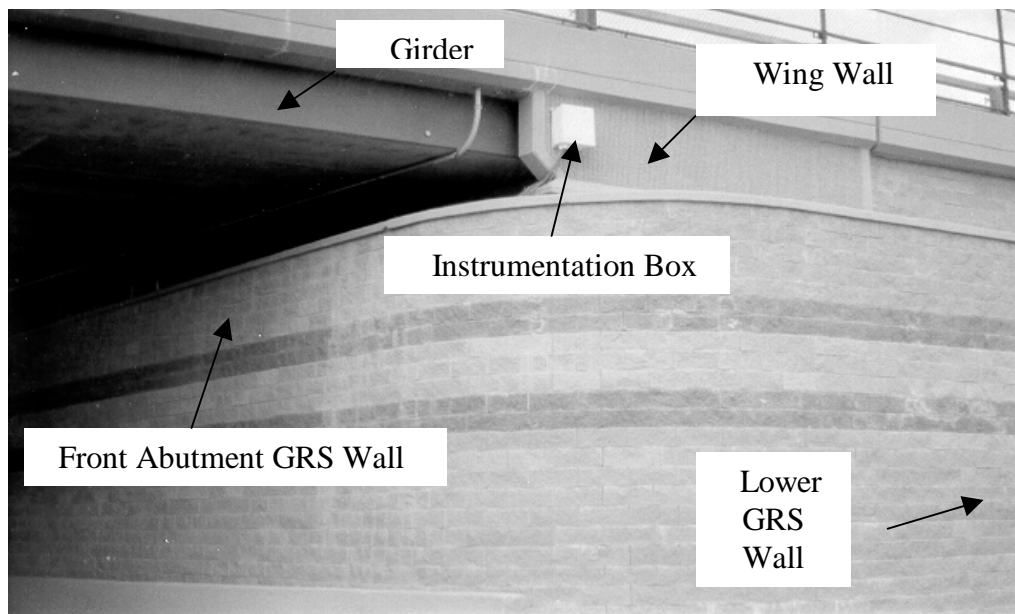


Figure 1. View of the Southeast Side of the Founders/Meadows GRS Bridge Abutment Structure

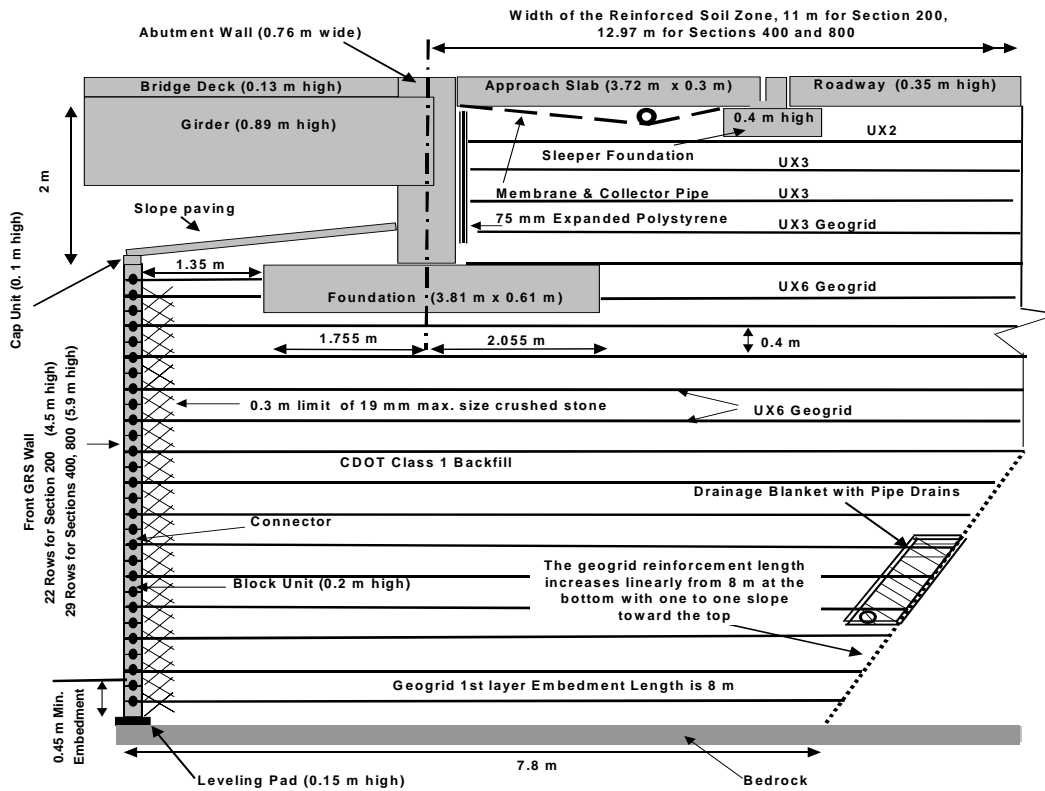


Figure 2. Typical Cross-section through the Front and Abutment GRS walls

A comprehensive literature review of studies on GRS structures supporting high surcharge loads is presented by Abu Hejleh et al. (2000b). AASHTO and the Federal Highway Administration (FHWA) had published preliminary design guidelines for bridge superstructures directly supported by MSE walls with panel facings and steel reinforcements (AASHTO 1992, Elias and Christopher 1997). The Founders/Meadows structure, which uses segmental block facing and geosynthetic reinforcements, was designed in 1996 before the release of FHWA guidelines with respect to modular block facing and connection strength criteria. A recently published FHWA research report (Wu et al, 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 (1996-1997), and a production bridge abutment constructed in Black Hawk, Colorado (1997). The report concludes that GRS abutments are viable and can be considered as alternatives to bridge abutments supported by deep foundations and to metallic reinforced soil abutments.

The design, materials, construction, and instrumentation of the GRS walls in the Founders/Meadows structures are summarized by Abu-Hejleh et al. (2000b). The front abutment GRS walls were constructed using gravelly backfill materials, Tensar UX 6 geogrid reinforcements with facing mechanical connectors, and concrete facing blocks. Reinforcement spacing is 0.4 m and block height is 0.2 m. CDOT specifications imposed a global reduction factor of 5.82 to determine the long-term design strength of the geogrid reinforcements from their ultimate tensile strength (157.3 kN/m). This global reduction factor is calculated by multiplying partial factors that account for tensile strength losses over the design life due to durability (1.1), installation damage (1.1), the maximum load to preclude a viscoelastic rupture failure over 75 years (creep reduction of 2.7), and a factor of safety to account for uncertainties (1.78).

The design of the Founders/Meadows structure met all stability and allowable stress requirements of CDOT. The presence of competent claystone bedrock formation beneath the front abutment walls ensured a small level of settlements. Several measures were considered to enhance the overall stability of the structure, support the concentrated loads from the bridge footing, and to alleviate the potential of the "bump at the bridge" problem:

- ❑ 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. The reinforcements length is often taken equal to 70% of the wall height (8 m) as measured from the leveling pad to the pavement surface (corresponds to 5.6 m in the Founders/Meadows structure). However, in the Founder/Meadows structure, a longer trapezoid-shaped reinforced zone was adopted. Specifically, reinforcement length increased linearly from 8.0 m at the base with 1H: 1V slope toward the top (Figure 2).
- ❑ Several measures were implemented to prevent that surface water and groundwater from reaching the reinforced soil mass (to prevent the soil erosion problem) and base of the fill (to prevent soaking of the claystone bedrock). This included placement of impervious membranes with collector pipes as shown in Figure 2.
- ❑ A compressible 75 mm thick low-density expanded polystyrene sheet was placed between the reinforced backfill and the abutment walls (Figure 2) to accommodate the thermally induced movements of the bridge superstructure, thus reducing the applied passive stresses to the backfill soil to near zero. CDOT engineers also expected that this system would significantly reduce the backfill active horizontal earth pressure against the abutment wall.
- ❑ The distance from bridge abutment centerline is 1.755 m to the front edge of the footing and 2.055 m to the back edge of the bridge footing (Figure 2). This and the previous measure were considered to reduce the eccentricity value of the resulting vertical load acting on the bridge footing.

The performance of bridge GRS abutments has not been tested under actual service conditions to merit acceptance without reservation in highway construction. Full-scale instrumentation of geosynthetic-reinforced soil systems has provided invaluable understanding on the performance of critical structure under in-service conditions (e.g. Zornberg et al. 1995). Consequently, the Founders/Meadows structure was considered experimental and comprehensive material testing, instrumentation, and monitoring programs were incorporated into the construction operations. Three sections of the GRS system were instrumented to provide insight on the structure movement and loading response during six construction stages and after opening the structure to traffic. Abu-Hejleh et al. (2002) provide an evaluation of the movement response of Founders/Meadows structures based on monitored movement data of the front GRS wall, settlements of the footings supporting the bridge load, strains of the geogrid reinforcements, and differential settlement between the bridge abutment and the approaching roadway. This paper briefly summarizes the measured loading response and data analysis, provides a design assessment of the front abutment GRS walls, and offers preliminary recommendations for design and construction of future GRS abutments.

2 INSTRUMENTATION PROGRAM

The layout of the instrumentation program of Section 800 is shown in Figure 3 and list of all gages is presented in Table 1. The height of the front abutment GRS wall is 5.9 m and the bottom of the bridge footing is located 5.28 m above the leveling pad. The front abutment GRS wall along Section 800 was heavily instrumented with pressure cells and strain gages. Five pressure cells (Geokon Model 4810) were placed in the middle and upper zones of the wall to measure lateral earth pressure against the front abutment wall facing. Pressure cells (Geokon Model 4800) and strain gages (Geokon Model 4050) were used to measure, respectively, vertical earth pressures and geogrid strains beneath the bridge footing (around geogrid layers 12 and 13), the middle zone of the wall (around geogrid layers 6 and 10), and towards the base of the wall. These pressure cells and strain gages cover four vertical Location Lines: 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 7.5 m from the facing).

The gages in this study are designated by a numerical code indicating the number of the closest geogrid layer to the gage, followed by two or three letters (e.g., 10VBN, 11HN, and 6SBN). The first letter indicates the gage type: V = pressure cell to measure vertical pressure, H= pressure cell to measure lateral earth pressure, and S = strain gage. The 2nd letter indicates the closest Location Line to the gage. The third letter (optional, N or S) is used when two gages are placed at the same location, one north of the control section (N) and one south of the control section (S). Note that layer 2 gages refer to all gages placed nearby geogrid layer # 2 (Figure 3).

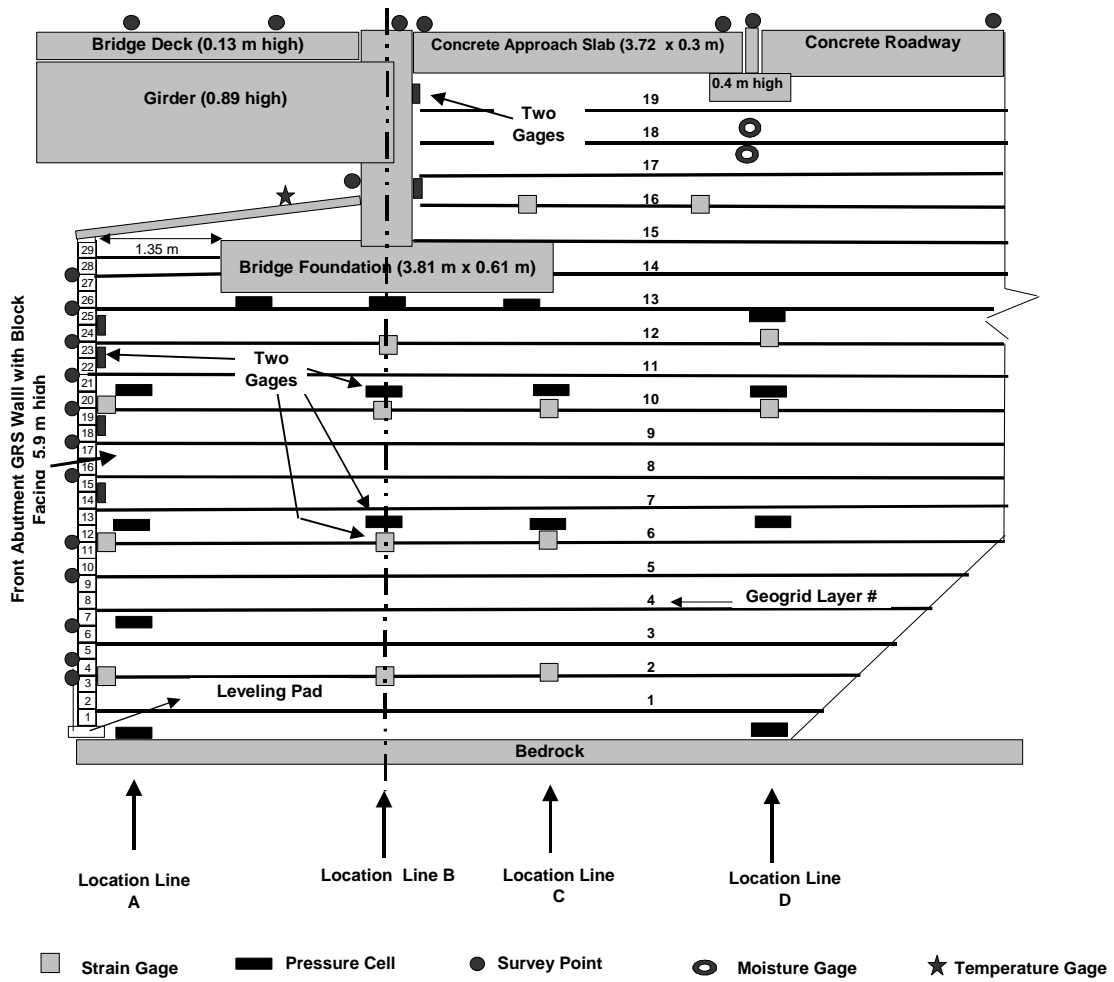


Figure 3. Layout of Instrumented Section 800

Table 1. Measured Data inside the Front GRS Wall at End of all Monitored Stages

Units: kPa for gages measuring vertical earth pressures (having letter V) and horizontal earth pressure (H), and in % for gages measuring geogrid tensile strains (S)							
Stage	I	II	III	IV	V	VI	VII
Gage #	Gages placed at or near the Base of the Reinforced Fill						
0VA	136	149	161	172	133	138	152
0VD	75	81	87	112	125	138	134
3VA*	61	68	73	74	51	50	55
2SA	(0.24)						
2SB	0.06	0.06	0.06	0.06	0.08	0.08	
2SC	0.01	0.01	0.01	0.01	0.01	0.01	
	Gages at or near geogrid layer #6						
6VA	66	79	90	92	81	75	78
6VBN	49	70	103	126	171	184	183
6VBS	36	56	83	99	130	135	124
6VC	29	35	47	68	87	100	94
6VD	47	50	54	88	98	113	105
7H*	0.5	0.5	1.0	2.0	2.0	4.4	5.9
6SA	0.28	0.3	0.34	(0.36)	0.37	0.38	(0.4)
6SBN	0.16	0.19	0.20	0.22	0.29	0.33	(0.37)
6SBS	0.05	0.06	0.07	0.08	0.15	0.18	0.22
6SC	0.08	0.08	0.08	0.08	0.08	0.08	0.11
	Gages Placed at or near geogrid layer #10						
10VA	31	39	45	45	31	30	32
10VBN	18	46	77	88	116	110	117
10VBS	29	67	114	129	171	168	183
10VC	33	43	63	95	119	139	128
10VD	30	29	28	66	69	84	81
9H	1	1.5	4.0	6.0	0.5	2.4	3.1
11HN	6.0	6.5	6.7	7.0	6.0	8.5	9.3
11HS	5.0	6.5	8.0	9.0	6.0	7.8	8.4
10SA	0.29	0.33	0.33	0.33	0.21	0.18	0.17
10SB	0.07	0.08	0.10	0.11	0.16	0.18	0.23
10SC	0.22	0.23	0.24	0.28	0.36	0.41	(0.5)
10SD	-0.07	-0.07	-0.06	-0.06	-0.05	-0.04	-0.03
	Gages Placed Beneath the Bridge Footing						
12VD	0	1	0	18	19	27	26
13VA	0	16	53	55	38	34	42
13VB	0	13	29	32	39	28	28
13VC	0	8	33	41	30	39	37
12H	0	0.5	0.5	0.5	0.5	1.5	0.6
12SB	0.45	0.45	0.48	0.52	0.08	0.01	0.05
12SD	0.18	0.21	0.24	0.30	-0.10	-0.07	-0.05

The collected data were organized according to the loading sequence of the following construction stages:

- Stage I.** Construction of most of the front abutment GRS wall up to the bridge footing elevation (height of 5.28 m). This stage started Jan. 19, 1999 and was completed on February 24, 1999 (55 days from Jan. 1, 1999).
- Stage II.** Placement of the bridge footing and girders seat and completing the construction of the front abutment GRS wall.
- Stage III.** Placement of girders (69 and 70 days from Jan. 1, 1999).
- Stage IV.** Placement of the reinforced backfill behind the abutment backwall from the bridge footing elevation to the bottom of the approach slab footing. Completed on March 26, 1999 (85 days from Jan. 1, 1999).
- Stage V.** Placement of the of bridge deck, completed on May 25, 1999.
- Stage VI.** Placement of the approaching roadway structure (including approach slab) and other minor structures. Completed on June 29, 1999. The design bridge footing pressure by the end of this stage (end of construction) was calculated as 115 kPa.
- Stage VII.** Post-Construction Stage while structure was opened to traffic and started June 30, 1999 (180 days from Jan. 1, 1999). The design bridge footing pressure while the structure was in service was calculated as 150 kPa.

3 INSTRUMENTATION RESULTS DURING THE CONSTRUCTION STAGES

Table 1 summarizes the vertical earth pressures, lateral earth against wall facing, and geogrid tensile strains measured during all construction stages. Because of the small strain anticipated, the strain gages used in this study are of high sensitivity. Accordingly, few gages maxed out at various stages. The missing strain data was estimated based on extrapolation of data from neighboring gages and measured data before the strain gage maxed out. Strain data obtained by extrapolation is shown in Table 1 between parentheses. The following discussion is based on the measured data listed in Table 1.

Significant incremental changes in measured geogrid strain, especially close to facing, and facing lateral earth pressure occurred during placement and compaction of the initial 1 m of backfill over the gages. It appears that some or all of the compaction-induced lateral loads in the soil and reinforcements remained locked-in after removal of the compaction vertical loads. At geogrid layer 12 (beneath the bridge footing) questionably large incremental increases in strains were measured (recorded by end of Stage I in Table 1). These large strains are questionable and could be attributed, in addition to compaction, to slack and bending in the geogrid, and erroneous reference reading.

As expected, sharp increases in the measured vertical and lateral earth pressures and geogrid tensile strains were measured around days 69 and 70 from Jan. 1, 1999 when girders were placed (Stage III). The highest increases in vertical earth pressures in this stage were along Location Line B, followed by C and A, and the smallest along Location Line D. Placement of backfill and surcharge loads behind the bridge abutment during Stage IV and VI increased the loads in the interior side of the reinforced soil zone, primarily along Location Lines D and C, as expected. During stages II to IV, the front abutment GRS wall system along geogrid layers 6, 10, and 12 experienced comparatively small strains. Beneath the bridge footing (from layer 12 gages), changes in measured geogrid tensile strains occurred during stages II to IV were larger than those at geogrid layers 6 and 10, as expected. The vertical earth pressures behind the wall facing (Location Line A) at all levels increased to their maximum values by the end of Stage IV (higher than at end of construction: Stage VI). At many other levels, not all, measured geogrid strain data and lateral earth pressures behind wall facing were highest at the end of Stage IV construction. Possible reasons for the wall stiffer response during stages II to IV is the influence of compaction experienced in the previous stage (Stage I), and construction took place during the winter season.

During Stage V, the measured vertical, lateral earth pressures, and geogrid strains behind the wall facing, and vertical earth pressures and geogrid tensile strains beneath the bridge footing reduced significantly (Table 1). While these data were dropping, vertical earth pressures and geogrid tensile strains were increasing at comparatively large rates along Location Lines B and C of geogrid layers 6 and 10. This redistribution of stresses significantly enhanced the overturning and facing stability of the structure as will presented next. This response could be attributed to the relatively large movement experienced during Stage V that mobilized the tensile resistance in the geogrid reinforcements at locations far from the facing and reduced the loads behind and against the wall facing. Thawing and wetting of the backfill during the spring of 1999 may have led to softening of the backfill during this stage. Also, it is possible that the load addition in this stage increased the lateral earth loads to a level that exceeded the compaction locked-in lateral loads (i.e., load response entered the normally-consolidated range).

By the end of all construction stages (Stage VI), the measured vertical earth pressures and geogrid tensile strains behind the wall facing and the facing lateral earth pressures (except from Gage 7H) were reduced to almost the values measured before placement of the bridge structure (Stage I, see Table 1).

4 INSTRUMENTATION RESULTS WHILE THE STRUCTURE IN SERVICE

Table 1 summarizes the measured data while the structure was in service for one year. Figure 4 shows the time history of geogrid strains collected from several strain gages during approximately 33 months after opening the bridge to traffic (Stage VII). The small increases in

measured values from some pressure cells and strain gages occurred immediately after opening the bridge to traffic, possibly due to placement of minor structures on the bridge structure (even after opening the bridge to traffic, see Figure 4). After that, all pressure cells showed negligible changes in the measured vertical and lateral earth pressure. All strain gages registered small increases of tensile strains during the 1st year of service (maximum of 0.09%), except Gage 10SA which registered a small decrease in the geogrid tensile strains. The geogrid strain measurements during the first year of service compare very favorably with the movements of the facing column of the front abutment GRS wall obtained through surveying and inclinometer readings (Abu-Hejleh et al., 2002). Additional geogrid tensile strains developed after one year in service were small and showed a clearly decreasing rate with time. These strains seem to increase during the fall seasons of years 2000 (maximum of 0.04 %) and 2001 (max. of 0.02%) and to level out during other times.

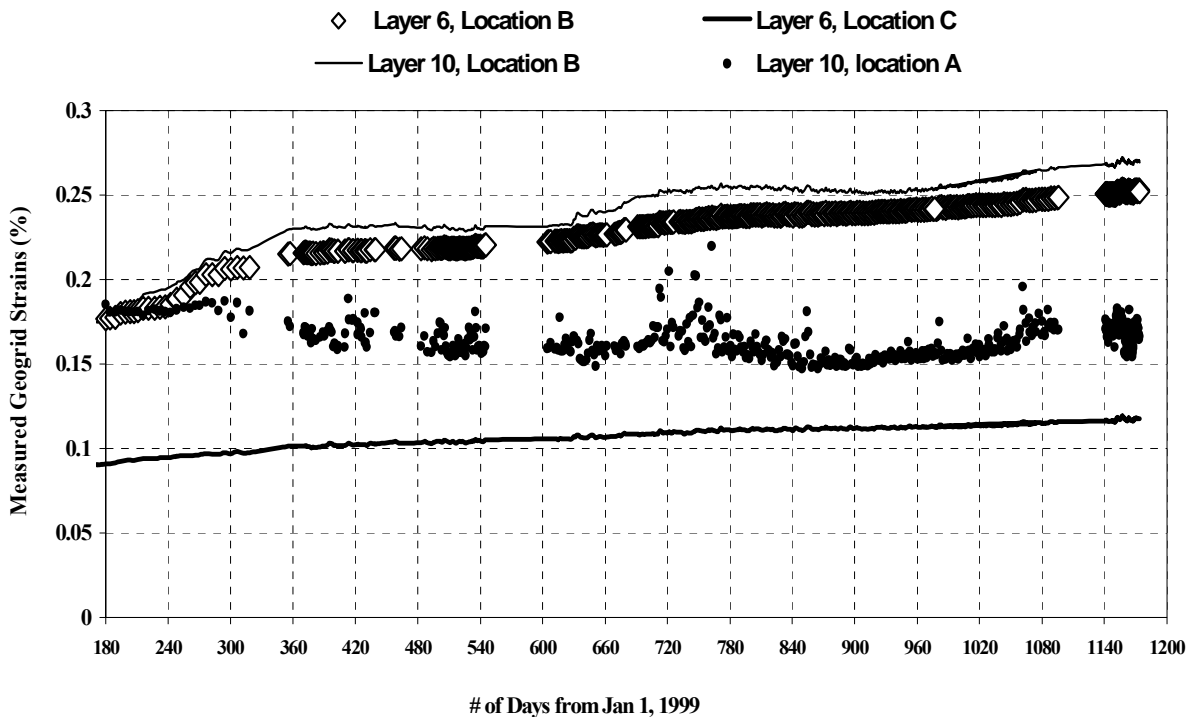


Figure 4. Typical Time History Records of Geogrid Strain Data Measured while the Structure is in Service (Stage VII).

Note: The period shown in the horizontal axis ranges from June 1999 (180 days from Jan 1 1999) to March 2002 (1185 days from January 1, 1999).

5 DATA ANALYSIS OF THE MEASURED DATA

The geogrid tensile loads were obtained directly from the measured geogrid strain and the load-strain curve for UX 6 Tensar geogrid. The measured geogrid strain-load response for UX 6 geogrid, developed using ASTM D4595 test method, was almost linear from 0 to 2% strain range with a load stiffness of 2000 kN/m. The use of an unconfined modulus obtained from in-air testing can be justified on the basis that modulus obtained in a confined mode for uniaxial HDPE geogrids varies only in the range of 5 to 10 percent (Elias et al., 1998). The increase in post construction geogrid strain observed in this study could be attributed to traffic load, seasonal changes, and the time dependent response of the geogrid (i.e., creep). Since the post construction measured vertical earth pressures were constant after 1 year in service, it can be assumed that post construction strains developed during the first year of service are due to traffic load and seasonal changes, and those developed after that are due to the time dependent response of the geogrid (creep). Assuming a log-log relation between the creep strains and time, it is roughly estimated that the developed maximum time dependent strains along geogrid layer 10 will be 0.5% over the structure service period of 75 years.

Design and measured load data along Section 800 at the end of construction have been compared and used to assess the design of the front abutment GRS wall. It was shown previously that there is a strong correlation among the measured vertical earth pressures, geogrid tensile strains, and the applied load during construction stages. Measured performance data from different types of gages showed similar trend of response.

5.1 Analysis for the Internal Stability of the Front abutment GRS Wall

To estimate the geogrid pullout resistance requires knowledge of the location of the maximum tensile line. AASHTO 1992 recommends that the shape of the maximum tensile force line for MSE walls supporting high surcharge, using extensible or inextensible reinforcements, be modified to extend to the back edge of the footing. The location of the maximum geogrid strains across geogrid layers 2, 6, and 10 were employed to determine the location of the maximum tension line. The results, as summarized in Figure 5a, suggest that the maximum tension line was indeed shifted to the backside of the bridge footing. This is in support of AASHTO and FHWA recommendations and of CDOT design procedure to extend the length of reinforcements.

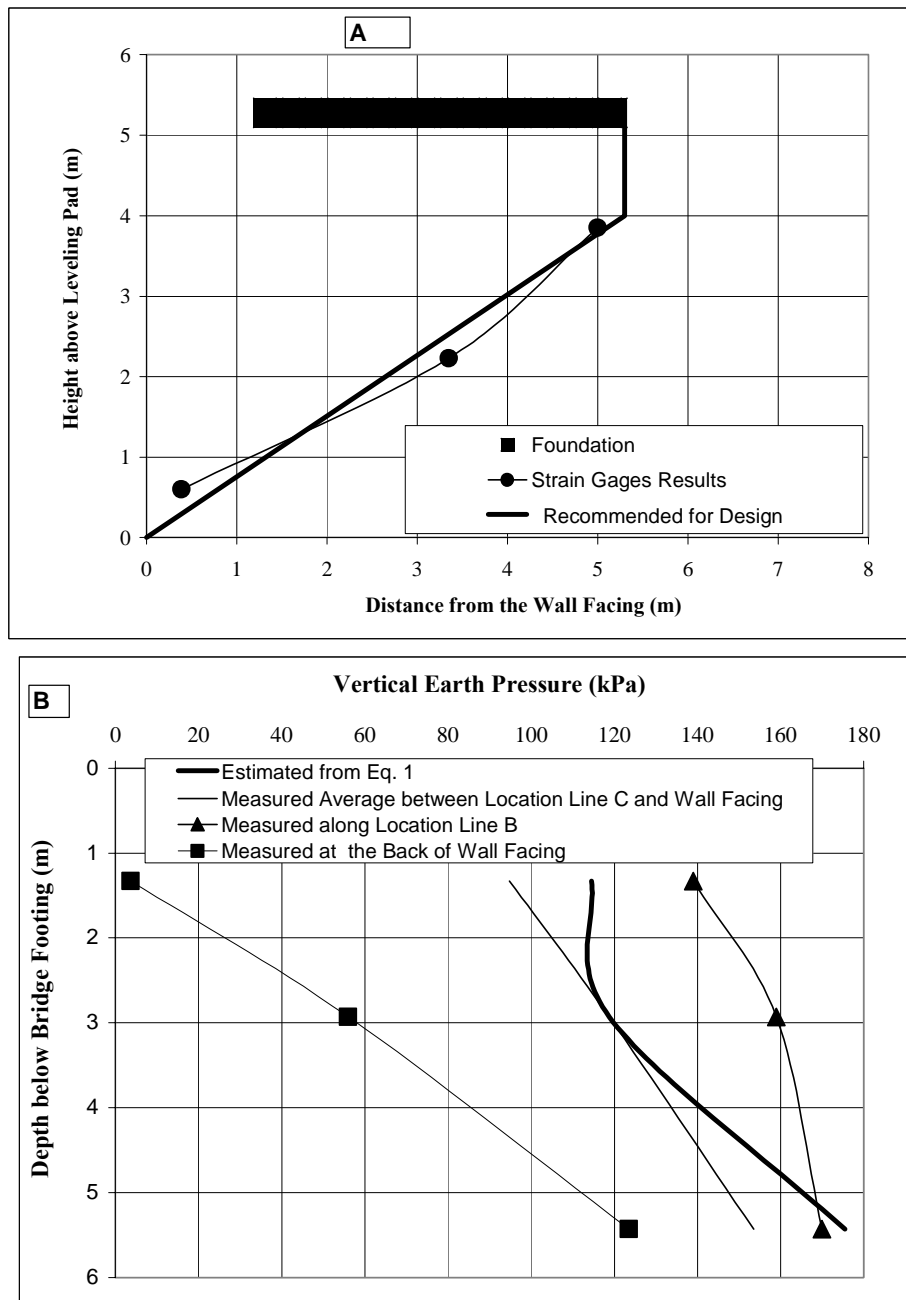


Figure 5. Measured and Estimated Results in the Front Abutment GRS Wall for: a) Maximum Tensile Force Line, and b) Vertical Earth Pressure Profiles

For MSE walls supporting high surcharge loads, the FHWA (Elias and Christopher, 1997) recommends the following sets of equations to estimate the maximum reinforcement tensile force per unit width (100% coverage), T_{max} along the maximum tension line, connection force per unit width T_o , and lateral earth pressure against the wall facing, σ_{ho} , at depth z below the bridge footing by the end of the construction:

$$\sigma_v = \gamma z + \Delta\sigma_v \dots\dots\dots(1)$$

$$\sigma_h = \sigma_{ho} = K_a \sigma_v + \Delta\sigma_h \dots\dots\dots(2)$$

$$T_{max} = T_o = S \sigma_h \dots\dots\dots(3)$$

Where σ_v , σ_h are respectively the vertical and lateral earth pressure; γ is the backfill unit weight assumed 19.6 kN/m³ (measured in the field as 22.1 kN/m³); K_a is the active earth pressure coefficient, calculated as 0.28 for a backfill soil with friction angle of 34°, zero cohesion, and S is the reinforcement vertical spacing equal to 0.4 m (every two blocks); $\Delta\sigma_v$ is stress increment at each depth from bridge footing loads, estimated using the 2V: 1H pyramidal distribution; $\Delta\sigma_h$ is the increment of horizontal stress from the backfill behind the bridge footing. $\Delta\sigma_h$ is taken zero due to the large distance from back edge of the bridge footing to the wall facing, and placement of reinforced backfill with EPS material behind the bridge abutment back wall. The actual shear strength parameters for the Founder/Meadow's gravelly backfill materials were obtained from large size triaxial tests as 39 degrees for the peak friction angle and 69 kPa for the cohesion, larger values than assumed in the design (Abu-Hejleh et al., 2000b). By conservatively dropping the measured cohesion component of the backfill shear strength, the value for K_a for soil with a friction angle of 39 is 0.22. Data analysis was made using $K_a = 0.22$, and $\gamma = 22.1$ kN/m³.

Calculated values for the vertical pressures, σ_v , and corresponding measured values (average of data at Location Lines A, B, and C) are shown in Figure 5b. These results are in support of the AASHTO and FHWA procedure to calculate the vertical earth pressures, σ_v . Also shown in Figure 5b are the measured vertical earth pressures behind the wall facing and along Location Line B. Table 2 summarizes for geogrid layers 6 and 10 the measured maximum reinforcement tensile loads (from measured maximum geogrid strains) and connection loads (from measured geogrid strains behind wall facing), and the measured facing lateral earth pressures. Table 2 also summarizes the calculated values from Eqs. 1, 2, and 3, together with the percentages of the measured to calculated values.

Table 2. Design and Measured Load Data at End of Construction

Geogrid Layer # or Gage #	Design	Measured	Percentage Ratio of Measured to Design (%)
Maximum Geogrid Tensile Forces (kN/m)			
Layer 6	11.7	7.7	66
Layer 10	10.1	8.2	82
Connection Forces (kN/m)			
Layer 6	11.7	7.7	66
Layer 10	10.1	3.6	35
Lateral Earth Pressure against Wall Facing (kPa)			
7H	27.5	4.4	16
9H	25.5	2.4	9
11HN	24.5	8.5	35
11HS	24.5	7.8	31
12H	25	1.5	6

5.2 Analysis for the External Stability of the Front Abutment GRS Wall

The horizontal profiles of the vertical earth pressure measured in the reinforced soil mass from the wall facing to Location Line D at the base of the fill and along layers 6 and 10 gages (obtained with the vertical pressure values listed in Table 1) were utilized to determine: 1) the eccentricity value of the resulting vertical load acting on these levels during various construction stages, and 2) the average vertical earth pressure acting on the base of the fill at end of construction. Note that at the base of the reinforced fill, no pressure cells were placed at Location Lines B and C (Figure 3), and therefore the vertical earth pressures value at these locations were estimated by extrapolation. The horizontal profiles of vertical earth pressures measured just beneath the bridge footing (Table 1 at Location Lines A, B, and C) were used determine: 1) the eccentricity value of the resulting vertical load acting on the bridge footing during various construction stages, and 2) the average vertical earth pressure acting on the bridge footing by end of construction.

Positive eccentricity values mean that the location of the resulting vertical force shifted from the center location toward the wall facing. The overturning potential of the structure increases with increasing eccentricity. Maximum permissible eccentricity is 0.64 m for the bridge footing (3.8/6), 1.25 m on the reinforced soil mass (7.5/6), and 1.88 m for the rocky base of the fill (7.5/4). Stage III (placement of girders) produced the maximum eccentric loading on the reinforced soil mass. The measured eccentricity values at end of Stage III is 0.2 m beneath the

bridge footing, 0.23 m along layer 10, 0.4 m along layer 6, and 0.4 m at the base of the fill, all less than the allowable. The placement of reinforced backfill behind the abutment during Stage IV and redistribution of stresses observed during Stage V reduced significantly the measured eccentricity values. By end of construction, the measured eccentricity values at all horizontal levels were almost zero.

The measured average vertical earth pressure at the base of the reinforced fill at the end of construction (Stage VI) was 154 kPa, less than the design calculated value of 199 kPa. Beneath the bridge footing, the measured average vertical earth pressure beneath the bridge footing by the end of the construction was 34 kPa, significantly below the calculated value of 119 kPa.

6 PERFORMANCE AND DESIGN ASSESSMENT OF THE FRONT GRS WALL

The overall performance of the Founders/Meadows bridge structure is excellent, based on the deformation response reported by Abu-Hejleh et. al. (2002) and the loading response and design assessment of the structure summarized in this paper. Abu-Hejleh et. al. (2002) reported:

- 1) The monitored movements of the Founders/Meadows structure were smaller than those anticipated in the design or allowed by performance requirements;
- 2) Post-construction movements became negligible after an in-service period of 1 year, and
- 3) There was no evidence of the “bump at the bridge” problem after 35 months in service.

The results reported in this paper indicated that the maximum geogrid creep strain that will develop over the structure service design life of 75 years was roughly estimated as 0.5%. The measured tensile loads in the reinforcements (less than 10 kN/m, Table 2) are well below the geogrid creep rupture load of 58 kN/m. This suggests that the long-term time dependent movements of the structure will be very low and the service life of the geogrid reinforcements will be much more than 75 years.

Additionally, analysis of the measured performance loading data at end of construction (when all dead loads were placed) and comparison with the design calculated values reveal the following:

- By using assumed friction angle for the backfill of 34° instead of the actual measured value of the 39° , the reinforcements loads were overestimated in the design by almost 30%.
- Support of the AASHTO and FHWA recommendations with respect to the location of the maximum tension line (Figure 5a) and the calculation (Eqs. 1, 2, and 3) for maximum geogrid tensile loads (Figure 5b, and Table 2).
- Support of the CDOT design decisions to
 1. Provide a significant offset from the front footing face to the back of the wall facing. Very small influence from the bridge superstructure loads was measured behind and

against the wall facing by end of construction. The measured vertical earth pressures and connection loads behind the wall facing after placement of the bridge superstructure (Stage VI) were very close to the values measured before placement of the bridge structure (Stage I, see Table 1).

2. Place EPS geofoam material and reinforced backfill behind the abutment back wall, and proper position of the bridge abutment centerline with respect to the bridge footing centerline. These two measures produced zero eccentricity of the vertical load on the bridge footing.
 3. Use of longer reinforced soil zone to alleviate the bridge bump problem and enhance the overall stability of the structure.
- The measured loads behind and against the wall facing were smaller than those estimated in the design. The measured vertical earth pressure behind the wall facing ranges from 2% (at a depth of 1.33 m below the bridge footing) to 67% (at the bottom of the wall, depth of 5.43 m) of the vertical earth pressures estimated with Eq. 1 (Figure 5b). The measured connection forces in the middle zone of the wall range from 35% to 66% of the design calculated value with Eq. 3 (Table 2). The measured facing lateral earth pressures against the wall facing in the middle and upper zones of the wall range from 6% to 35% of the design calculated values with Eq. 2 (Table 2).
 - There is no potential for overturning the Founders/Meadows structure as evidence by the measured negligible eccentricity of vertical load at all horizontal levels. The measured vertical earth pressure was almost uniform beneath the bridge footing. The measured horizontal distributions of vertical earth pressures at other deeper levels in the wall were non-uniform (i.e. varies significantly from location to location) and almost symmetrical around the centerline of the reinforced soil zone. The lowest measured vertical earth pressure occurred behind the wall facing (Location Line A) and the highest value occurred along the bridge abutment centerline (Location Line B).
 - The measured bearing pressures at the base of the reinforced fill and below the bridge footing were below the estimated design values.

7 RECOMMENDATIONS

The use of GRS walls to support both the bridge and approaching roadway structure has been approved by CDOT as a viable foundation support system in future bridge abutment projects. GRS 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 general layout and design of future GRS abutments would be as in the Founders/Meadows structure (Abu-Hejleh et.

al. 2000b). The maximum tension line needed in the internal stability analysis should be assumed bilinear (Figure 5b). It starts at the toe of the wall and extends through a straight line to the back edge of the bridge footing at the mid height of the wall (wall height is 8 m from leveling pad to top pavement surface), and from there extends vertically to the back edge of the bridge footing. The use of this system should be limited to projects founded on firm soils and with no scour potential until further research is conducted. The projected foundation settlement should occur rapidly and be smaller than the value specified by the bridge design engineer. Elias and Christopher (1997) recommended that the tolerable differential settlements between abutments or between piers and abutments of MSE abutments be limited to $0.002L$ (L is the span length) for bridges with continuous spans and $0.0025L$ for bridges with simple spans. For field and loading conditions similar to those encountered in the Founders/meadows structure, the designer should plan for a bridge footing settlement of at least 25 mm. It is preferred that construction takes place during the warm and dry seasons, and that the backfill behind the abutment wall placed before the girders. The actual friction angle of the backfill should be measured and employed in the design.

It is also recommended to use a smaller vertical spacing for the reinforcement layers (i.e., every block level) and reinforcements having a stiffness level as those employed in the Founders/Meadows structure (i.e., for reinforcement spacing of 0.2 m, the reinforcement initial load stiffness modulus should be larger than 1000 kN/m). For GRS system meeting these conditions, CDOT is planning to make the following changes from those employed in the Founders/Meadows structure:

- The allowable design pressure for the bridge footing will be increased from 150 kPa to 200 kPa;
- The base length of reinforcements will be reduced from 100% to 80% of the wall height (measured from leveling pad to the top pavement surface).
- The distance between centerline of the bridge abutment wall and wall facing (rear side) will be reduced from 3.1 m to 2.7 m.

Future research should investigate the dependent (interrelated) relation between the creep reduction factor, factor of safety, and other factors utilized to determine the design strength of geosynthetic reinforcements.

Additional research is needed to explain the measured loading response of the front GRS wall of the Founders/Meadows structure and the low vertical earth pressure and geogrid strain values measured beneath the bridge footing at end of construction. It is speculated that, if the entire wall construction took place during the warm and drying seasons, the sharp reduction of the measured loads observed in this study behind the wall facing will not occur and the loads measured at end of construction will be close to those measured in this study.

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