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Performance of Geosynthetic-Reinforced Walls Supporting Bridge and Approaching Roadway Structures

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ABSTRACT This paper describes a unique field application in which a geosynthetic-reinforced soil system was designed and constructed to support both the foundation of a two-span bridge and the approaching roadway structure. The reinforced soil system not only provides bridge support, but it was also designed to alleviate the common bridge bump problem. This structure was considered experimental and comprehensive material testing and instrumentation programs were conducted. These programs would allow assessment of the overall structure performance and evaluation of CDOT and AASHTO design assumptions and procedures for reinforced soil structures supporting both bridge foundations and approaching roadway structures. Large-size direct shear and triaxial tests were conducted to determine representative shear strength properties and constitutive relations of the gravelly backfill used for construction. Three sections were instrumented to provide information on external movements, internal soil stresses, geogrid strains, and moisture content during various construction stages and after the structure opening to traffic. Results from a pilot (Phase I) instrumentation program and some preliminary results from a more comprehensive (Phase II) instrumentation program are presented in the paper. The results suggest that current design procedures lead to a conservative estimation of both the backfill material strength and horizontal earth pressures, and that the overall performance of this structure, before its opening to traffic, has been satisfactory.

INTRODUCTION

The technology of mechanically stabilized reinforced soil has been used extensively in the construction of retaining walls and slopes for roadways and bridge structures to support the self-weight of the backfill soil, roadway structure, and traffic loads. The increasing use and acceptance of soil reinforcement has been

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triggered by a number of factors, including cost savings, aesthetics, simple and fast construction techniques, good seismic performance, and the ability to tolerate large total and differential settlement without structural distress. A comparatively new use of reinforced soil technology is in bridge applications, in which the reinforced soil mass would directly support both the bridge and the approaching roadway structures. Placement of shallow foundations supporting the high bridge superstructure loads on the top of MSE walls leads to reinforcement tensions and soil stresses mobilized in a different manner than in the case of MSE walls supporting small surcharge loads. However, such application has the potential of alleviating the often significant bridge "bump" problem, caused by the differential settlement between the bridge foundation and approaching roadway structures.

The Colorado Department of Transportation (CDOT) has recently completed the construction of the new Founders/Meadows Bridge near Denver, Colorado. In this project, both the bridge and the approaching roadway structures are supported by a system of geosynthetic-reinforced segmental retaining walls. Fig. 1 shows a picture of one of the segmental retaining wall systems, located at the east side of the bridge. This figure shows the "front MSE wall" supporting the bridge superstructure, which extends around a 90-degree curve into a "lower MSE wall" supporting the "wing wall" and a second tier, "upper MSE wall". This type of structure was selected mainly to alleviate the bridge bump problem, and to allow for simple construction techniques in a relatively small construction area. The performance of this unique system has not been tested under actual service conditions to merit acceptance without reservation in normal 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 were instrumented to provide information on the external movements, internal soil stresses, geogrid strains, and moisture content during construction and after the structure opening to traffic. Monitoring will continue until the long-term creep movements become negligible. The objectives of this investigation are:

- Assessment of the structure's overall performance under service loads.
- Assessment of CDOT and AASHTO design procedures and assumptions regarding the use of reinforced soil structures to support bridge foundations and to alleviate the bridge bump problem.
- Numerical analysis of the behavior of the reinforced soil system supporting the bridge and approaching roadway structure.

This paper initially presents an overview of the new Founders/Meadows design, materials and construction stages. Subsequently, the results of large-size triaxial and direct shear tests on the gravelly backfill soil material are presented and discussed. A description is then presented of the two-phase instrumentation and monitoring program. Finally, the results of the first phase of the monitoring program and some preliminary instrumentation results from the second phase, obtained before the bridge was opened to traffic, are presented and discussed. Data collection and analysis continues by the time of preparation of this paper. Subsequent publications will summarize the findings from additional monitoring results.

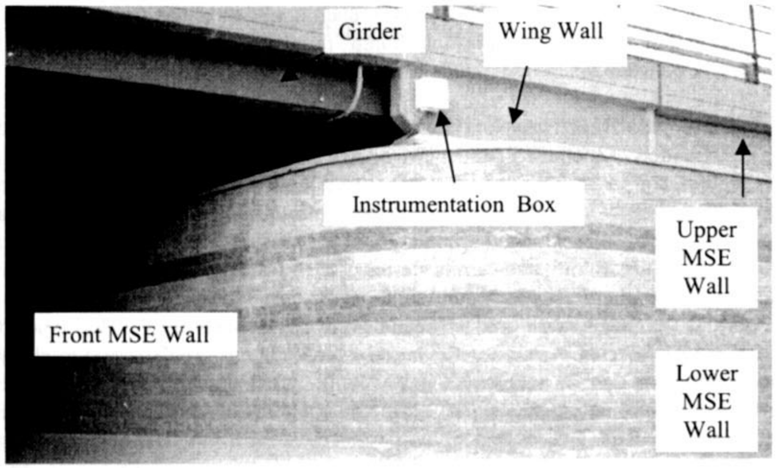

Fig. 1. View of the South-East Side of the Completed Founders/Meadows Bridge

BACKGROUND

Short bridge abutments can be either supported by deep foundation systems constructed through the reinforced soil mass, or by a shallow foundation placed directly on the top of reinforced soil mass. A recent survey sponsored by the National Cooperative Highway Research Program indicated that 86% of highway bridge foundations in the United States are supported through deep foundations. The use of deep foundations requires mobilization of large construction equipment, which requires comparatively large working areas that often induce major traffic disruptions. To the authors' knowledge, the design and construction of the Founders/Meadows geosynthetic reinforced structure, in which a short bridge abutment is supported directly by the reinforced mass, is the first of its kind in conventional highway practices. 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).

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. The load carrying capacity of the abutment was higher than the 200 kPa maximum footing pressure suggested by FHWA Demo 82 guidelines (Elias and Christopher 1997). In the CDOT study, a load corresponding to a vertical pressure of 232 kPa was applied on the top surface of the abutment structure. The measured immediate (short-term) vertical and lateral displacements were 27.1 mm and 14.3 mm, respectively. Under a sustained vertical footing pressure of 232 kPa for 70 days, the maximum vertical and lateral creep displacements were 18.3 mm and 14.3 mm, respectively. Gotteland et al. (1996) concluded that the performance of top-loaded reinforced experimental embankments are highly promising. Won et al. (1996) described the field use of sloped geogrid reinforced segmental retaining walls supporting end spans for a major bridge in Australia. The overall performance of this structure is acceptable and the obtained field results compare favorably with the calculated values. Tatsuoka et al. (1997) developed a hybrid wall system of reinforced soil backfill and a cast-in-place rigid wall facing. This system was used to support bridge girders with no problems.

Bridge bumps cause uncomfortable rides, create hazardous driving conditions, and require costly, frequent repairs with unnecessary traffic delays. Numerous investigations have been undertaken during the past decades to identify the causes and minimize the differential settlements between the bridge abutments and approaching roadway structures (Yeh and Su 1995, Reid et al. 1998). Three common causes for the development of bridge bumps are examined in this study. First, uneven settlement between the approaching roadway structure, often supported by compacted backfill soil, and the bridge abutment supported on stronger soils by a deep foundation. Bridge abutments supported by shallow foundations instead of deep foundations have been observed to lead to smaller uneven settlements between the bridge and approaching roadway structures. Second, expansion and contraction of the bridge decks and girders, which cause lateral displacement of the approach backfill. This is a more critical factor with the use of integral abutment bridges, where abutment walls are strongly attached to the superstructure without joints. Third, erosion of the fill material around the abutment wall caused by surface run-off water infiltration into the fill. Conventional methods used to prevent the "bump" from developing include extension of wing walls along the roadway shoulder, and use of approach slab and granular backfill (Class I Structural Backfill) behind the abutments. Current CDOT standard practice also includes the use of a comparatively expensive flow-fill (a low strength concrete mix) behind the abutment.

A new promising method to alleviate the bridge bump problem was used by the Wyoming DOT and subsequently by the South Dakota DOT, where a very small gap (around 15 cm) is incorporated between a geosynthetic reinforced fill and the bridge abutment (Reid et al. 1998). The reinforced fill behind the abutment is used to build a vertical, self-contained wall capable of holding an approximately vertical shape and forming an air gap between the abutment and retained fill. It was hypothesized that the gap behind the abutment would allow for the thermally-induced movements of the integral abutment without affecting the backfill, thus reducing the applied passive stresses to the backfill soil to near zero. At the same time, this system would help to mobilize the shear strength of the retained approach fill and tensile resistance of the reinforcement, thus reducing the horizontal active soil pressure on the abutment wall. A "collapsible" cardboard or "compressible" expanded polystyrene (EPS) panel is often placed between the bridge abutment and the reinforced fill.

DESCRIPTION OF THE FOUNDERS/MEADOWS STRUCTURE

The Founders/Meadows bridge is located 20 miles south of Denver, Colorado, near Castle Rock. The bridge carries Colorado State Highway 86, Founders/Meadows Parkway, over US Interstate 25. This structure 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. Fig. 2 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 in order to accommodate six traffic lanes and sidewalks on both sides of the bridge.

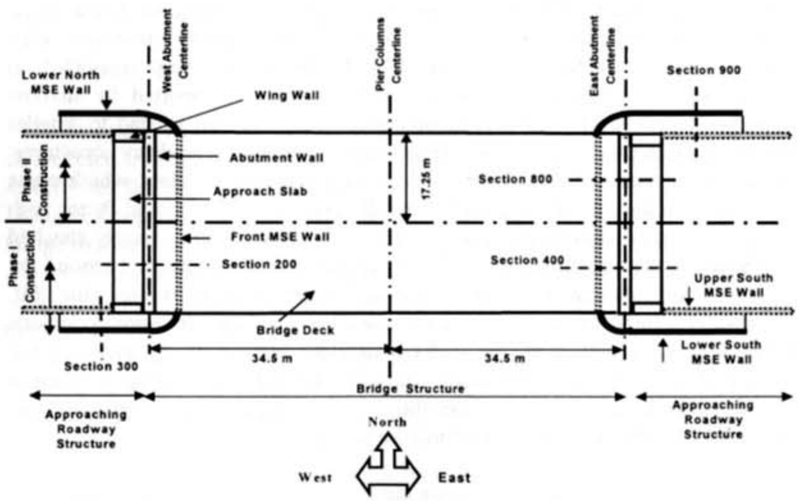

Fig. 2. Top View of the Founders/Meadows Structure

Fig. 3 shows a typical monitored cross-section through the "front MSE wall" and "abutment wall" (Sections 200, 400, and 800 in Fig. 2). The figure illustrates that the bridge superstructure load (from girders, bridge deck) is transmitted through abutment walls 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 edge of the foundation are located 3.1 m and 1.35 m from the facing of the front MSE wall. A short reinforced concrete abutment wall and two wing walls, resting on the spread foundation, confine the reinforced backfill soil behind the bridge abutment (see Figs. 1, 2, and 3) and support the bridge approach slab. Figure 4 shows a typical cross-section, along the tiered MSE walls, lower and upper MSE wall (sections 300 and 900 in Fig. 2), which support the approaching roadway structure. Sections 200, 400, and 800 are instrumented and monitored in this study. The bridge is supported by central pier columns along the middle of the structure (Fig. 2), which in turn are supported by a spread footing founded on bedrock at the median of Interstate 25.

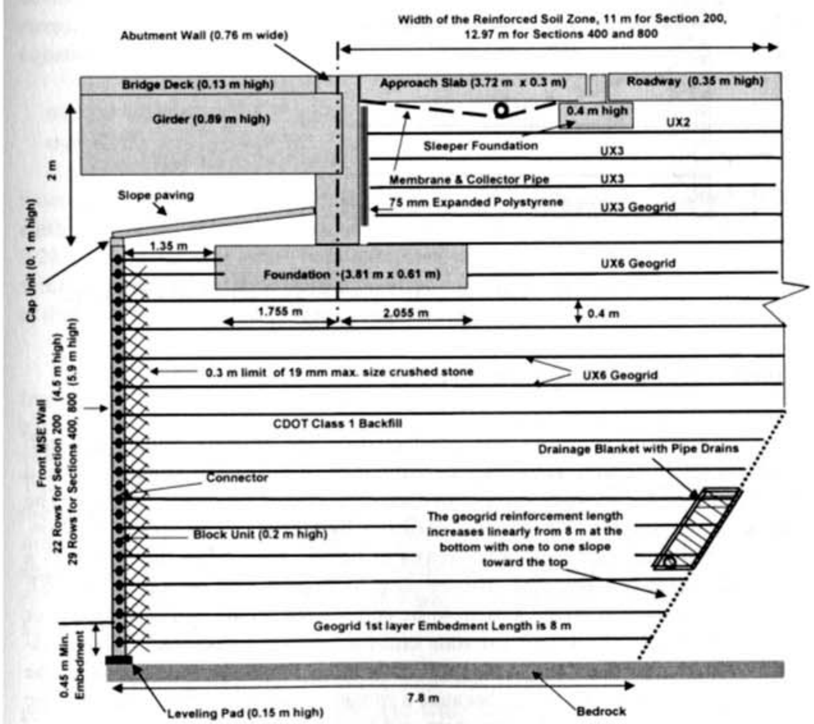

Fig. 3. Typical Monitored Cross-Section (Sections 200, 400, and 800 in Fig. 2) through the Front and Abutment MSE walls

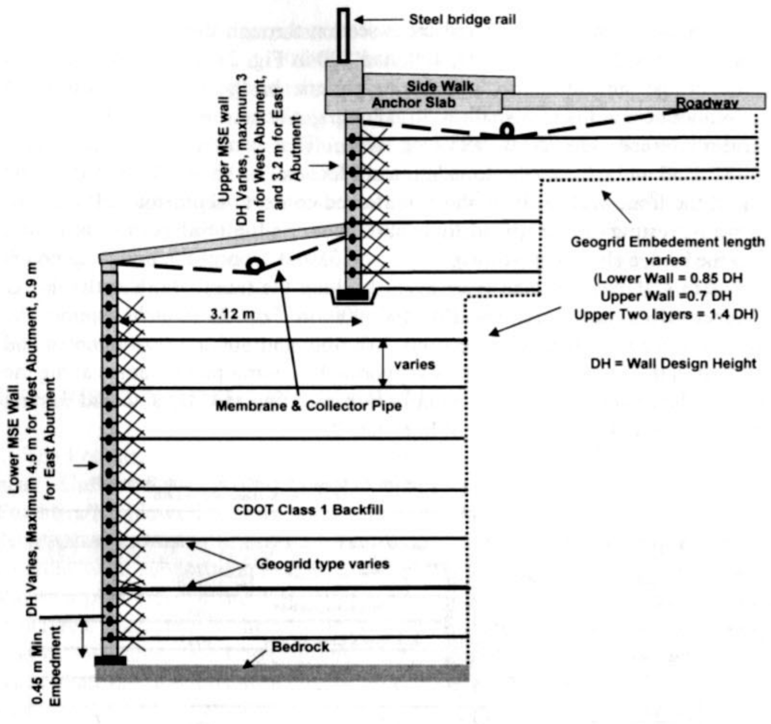

Fig. 4. Typical Cross-Section along the Upper and Lower MSE walls (e.g., Sections 300 and 900 in Fig. 2)

FOUNDATION RECOMMENDATIONS

A subsurface investigation was conducted on the existing bridge approach embankments. In general, subsurface conditions consisted of a mixture of clay and sand fill, 2.7 to 4.7 m thick, overlying sandstone and/or claystone bedrock. The Foundation Report for this project, prepared by CDOT Geotechnical Section, recommended four foundation alternatives to support the bridge abutment: (1) a fortress abutment consisting of shallow strip footings placed on top of an MSE mass with a segmental facing system to be constructed on the native claystone or sandstone bedrock; (2) drilled caissons embeded into the bedrock; (3) steel H-piles driven into the bedrock; and (4) spread footings founded on the bedrock. The first alternative was found viable because a spread foundation safely supported the pier columns of the old bridge structure and because the projected movements of the reinforced backfill and foundation bedrock were very small. Consequently, the first alternative was selected to reduce construction activities in the vicinity of the

bridge abutments, to alleviate the bridge bump problem, and because of the other perceived advantages of MSE systems (cost-effective, flexibility, etc.).

DESIGN OF THE FRONT AND ABUTMENT MSE WALLS

The design of the MSE walls along Sections 200, 400, and 800 followed AASHTO and CDOT 1996 guidelines. Fig. 3 shows the reinforcement type and layout of the structure along sections 200, 400, and 800. The wall design height (DH) is measured vertically along the blocks only.

Reinforcement and Connection Strength Requirements

The vertical stress, σ_v, within the reinforced soil mass is induced by gravity forces due to the backfill self weight, uniform surcharge load, q, and surcharge concentrated loads. AASHTO guidelines recommend the use of the 2V:1H approximation to estimate the distribution of vertical stress increment, $\Delta\sigma_v$, developed within the soil mass by concentrated surcharge loads. The soil vertical stress, σ_v, and horizontal stress, σ_h, at a depth z were estimated using conventional equations, as follows:

$$\sigma_v = \gamma z + q + \Delta\sigma_v \tag{1}$$
$$\sigma_h = K_a \sigma_v \tag{2}$$

where γ is the backfill unit weight, assumed of 19.6 kN/m^3, and K_a is the active earth pressure coefficient, calculated for an assumed friction angle of 34° to be 0.28. To determine the required long-term design strength (LTDS) of reinforcement (with 100% coverage) at any level, the estimated horizontal stress at that level is multiplied by the reinforcement spacing.

The contact pressure, induced by the bridge superstructure and full traffic load, transmitted by the bridge foundation to the top of the MSE walls, was estimated as 150 kPa. The vertical and horizontal soil stress within the reinforced soil mass at any depth z below the foundation were estimated using Equations (1) and (2). The maximum horizontal soil stresses, estimated as 55.5 kPa, occurred towards the base of the fill. The required LTDS of the reinforcement was estimated as a minimum of 22.2 kN/m assuming 100% coverage and reinforcements vertical spacing of 0.4 m. Current CDOT specifications follow those by AASHTO in recommending a connection strength equal or larger than 100% of the required LTDS at all reinforced levels.

The backfill behind the abutment wall was reinforced in both directions to reduce earth pressure on abutment and wing walls. The vertical and horizontal pressure inside the reinforced mass at depth z below the approach slab and behind the abutment wall were estimated using Equations (1) and (2) with $\Delta\sigma_v = 0$ and q = 18.84 kPa to account for the traffic and approach slab uniform surcharge load. At z = 0.4 m and z=1.9 m, the horizontal stresses were estimated as 7.6 kPa (required LTDS of 3.0 kN/m) and 15.9 kPa (required LTDS of 6.4 kN/m), respectively.

A compressible 75-mm low-density expanded polystyrene sheet was placed between the reinforced backfill and the abutment walls. It is expected that this system will eliminate the passive earth pressure induced on the fill behind the abutment from the horizontal thermal movement of the structure. In addition, CDOT engineers expect that this system will also reduce the backfill active horizontal pressure on the facing of the abutment wall to half or less of those predicted from Equation 2. This efficient and economical technique has been tried by CDOT for the first time in the Founders/Meadows structure and, if its performance is acceptable, it will replace the more expensive flow-fill technique.

Reinforcement Length Requirements

For walls exceeding 3.35 m high, CDOT requires a minimum reinforcement length of 70% of its design height. For the reinforced soil zone behind and below the bridge abutment, a trapezoid-shaped reinforced zone was adopted, in which reinforcement length increased linearly from 8.0 m at the bottom with 1H:1V (45°) slope toward the top (see Fig. 3). A comparatively long reinforced soil zone below the bridge and approaching roadway structure was considered in order to address three design issues. First, integrating the roadway approach embankment and the bridge foundation with an extended reinforced soil zone may alleviate the differential settlement problem. Second, an extended reinforced soil zone would also enhance the overall stability of the reinforced structure in terms of sliding and overturning. Third, it will provide an additional margin of safety to alleviate concerns regarding a potential shear strength loss due to soaking of the claystone bedrock.

Drainage Control

Infiltration of water into the reinforced soil mass may lead to loss of shear strength and reduced stability against sliding along the relatively impermeable claystone bedrock. Several measures were implemented to prevent surface run-off water and ground water from getting into the reinforced soil mass and the bedrock at the base of the fill. This included the placement of impervious membranes with collector pipes at the top of the reinforced soil (Figures 3 and 4) to intercept surface runoff water. Also, a drainage blanket (1 m high minimum) with pipe drains was placed directly behind the reinforced soil zone (see Fig. 3) to divert any infiltration and ground water from the reinforced soil mass.

Global Stability and Settlement Analysis

The presence of a competent claystone bedrock formation below the base of the reinforced backfill and the extended reinforced zone provided an adequate safety factor for the global stability of the structure. According to AASHTO 1996 guidelines, the two-span Founders/Meadows bridge supported at its abutments by MSE walls could safely tolerate a maximum long-term differential settlement of 50 mm without serious structural distress. Comparatively stronger and longer reinforcements were used in the MSE walls of this structure than in the CDOT demonstration abutment (Ketchart and Wu 1997) discussed earlier. Therefore, CDOT engineers expected that the movements of the front MSE wall due to the placement of the bridge superstructure will be acceptable and should not exceed 25 and 20 mm of vertical and lateral displacements, respectively.

MSE BACKFILL MATERIALS

Construction Requirements and Measured Values

The backfill material for the MSE walls was specified as CDOT Class 1 Structural Backfill. The construction requirements for CDOT Class 1 Structure Backfill (gradation, liquid limit, plasticity index, and compaction level) along with the measured values for the Founders/Meadows backfill are listed in Table 1. As shown in this table, the backfill soil used in this project is a mixture of gravel (35%), sand (54.4%) and fine-grained soil (10.6%). The liquid limit, plastic limit, and plasticity index for the backfill were measured in accordance with ASTM D3080. The backfill soil classifies as SW-SM per ASTM 2487, and as A-1-B (0) per AASHTO M 145. The maximum dry unit weight and optimum moisture content of the backfill, which is a function of the percentage of gravel, were 22.1 kN/m^3 and 4.2%, respectively, as measured in accordance with AASHTO T-180 Method A using 35% gravel. The average unit weight, dry unit weight, and water content of the compacted backfill soil, as measured during construction, were 22.1 kN/m^3, 21 kN/m^3 (equal to 95% of AASHTO T-180A) and 5.6%, respectively. Table 1 indicates that all CDOT construction requirements for the placed backfill were met. The measured backfill unit weight (22.1 kN/m^3) exceeds the assumed design value (19.6 kN/m^3).

Results of Conventional Direct Shear, Large Size Direct Shear, and Large Size Triaxial Tests

A friction angle of 34 degrees and zero cohesion were assumed in the design of the MSE walls. To evaluate these parameters, conventional direct shear tests and large size direct shear and triaxial tests were conducted using specimens of the backfill material used in construction. The assumed values and measured results for backfill strength parameters are summarized in Table 2.

The conventional direct shear tests were conducted in accordance with AASHTO T-236. As the conventional direct shear box is only 50 mm in diameter, the gravel portion of the backfill (35%) was removed from the specimens to be tested. The maximum dry unit weight and optimum moisture content of the backfill material without the gravel portion, were 19.9 kN/m^3 and 8.8% as measured in accordance with AASHTO T-99 Method A. The specimens were compacted to 18.9 kN/ m^3 (95% of AASHTO T-99A) and a moisture content of 9.6%. The Mohr-Columb shear strength envelope was defined, using the results from this testing program, by a 40.1° peak angle of internal friction and a 17 kPa cohesion. These measured values exceed the parameters used in the design of the reinforced soil walls.

Table 1. CDOT Construction Requirements and Measured Values for the Founders/ Meadows Class 1 Backfill

	Requirements	Measured Values
1. Gradation		
50 mm, (% Passing)	100	100
Sieve # 4 ((% Passing)	30-100	65
Sieve # 50 (% Passing)	10-60	21.1
Sieve # 200 (% Passing)	5-20	10.6
2. Liquid Limit (%)	<35	25
3. Plasticity Index (%)	<6	4.3
4. Dry Unit Weight (kN/m^3)	21 (95% of AASHTO T-180)	21

Table 2. Assumed and Measured Strength Parameters for the Founders/Meadows Class 1 Backfill

	Assumed Design Values	Measured from		
		Small Direct Shear Test	Large-Size Direct Shear Test	Large-Size Triaxial Test
Tested Material, Prepared Compacted Unit Weight (kN/m^3)*		Gravel Removed, 20.7	Entire Sample, 22.1	Entire Sample, 21.8
Peak Internal Friction Angle (Degrees)	34	40.1	47.7	39.5
Cohesion (kPa)	0	17	110.5	69.8

* Assumed backfill unit weight in the design is 19.6 kN/m^3 and measured value in the field is 22.1 kN/m^3.

A testing program was additionally performed using larger backfill soil specimens that included the gravel portion. This complementary program was

performed in order to assess the suitability of the shear strength parameters measured using the small conventional direct shear box, performed without gravel. Also, this complementary testing program would enable determination of representative constitutive parameters of the compacted backfill soil for future numerical simulation of the structure and comparison between numerical and field monitoring results. The obtained hyperbolic model constitutive parameters (Duncan and Chang 1970, and Duncan et al. 1980) will be presented and discussed in a future publication. The large size specimens were prepared at conditions that duplicate the compaction level and moisture measured in the field (see Table 2).

Compacted specimens were tested using a large square direct shear box (300 mm wide and 200 mm high) in accordance with ASTM D3080. The change in the cross-sectional area of the specimens as shearing proceeds was considered in the calculation of the applied shear stress. The tests were conducted under consolidated-drained conditions at normal stresses of 69 kPa, 138 kPa, and 207 kPa. The Mohr-Columb shear strength envelope was defined, at a shear displacement of 17 mm, by a 47.7° peak angle of internal friction and a 110.5 kPa cohesion.

In addition, triaxial tests were performed on compacted specimens representative of the entire backfill soil. Compacted specimens were tested using the large size triaxial apparatus (150 mm in diameter, and 300 mm high). The tests were conducted under consolidated-drained conditions at confining stresses of 69 kPa, 138 kPa, and 207 kPa. As shown in Figure 5, the deviatoric stress-axial strain triaxial test results and the hyperbolic model predictions show good agreement. All specimens showed dilatant behavior and a symmetrical lateral bulging at failure. The Mohr-Columb shear strength envelope is characterized by a 39.5° peak angle of internal friction and a 69.8 kPa cohesion. Possible reasons for the differences in the measured strength parameters between the large-size triaxial and direct shear test results are: different stress paths, predetermined shear plane in the direct shear test, and a small difference in the initial compaction level.

As expected, the shear strength values obtained using large-size specimens with representative gravel portion are higher than the values assumed in the design and than those obtained experimentally using specimens without gravel. From the results of the testing program, it is clear that common guidelines such as assuming zero cohesion and testing specimens without the gravel portion of the soil lead to significant underestimation of the actual shear strength of the backfill.

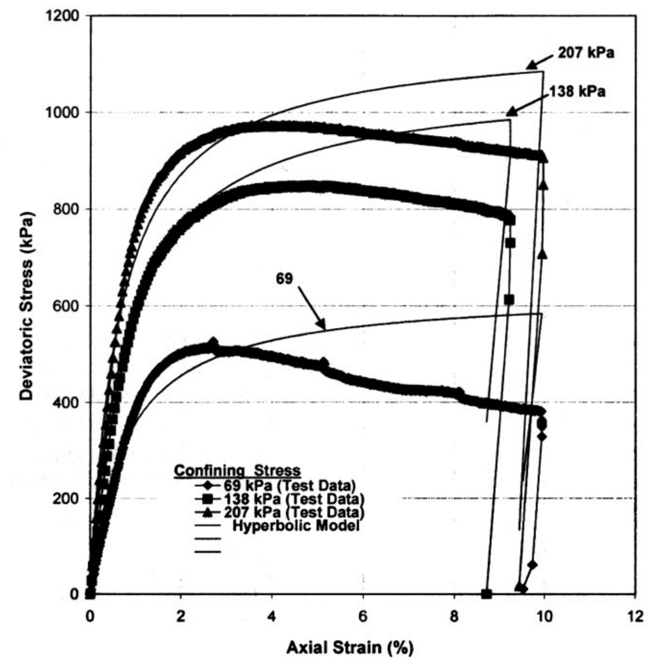

Fig. 5. Deviatoric Stress-Axial Strain Test Results from Large-Size Triaxial Tests and Predictions from Hyperbolic Model

GEOSYNTHETIC REINFORCEMENT AND FACING MATERIALS

Current CDOT specifications impose a global reduction factor to determine the long-term design strength (LTDS) of reinforcements from their ultimate strength. This global reduction factor accounts for reinforcement tensile strength losses over the design life period due to creep, durability, and installation damage of reinforcements. It also includes a factor of safety of uncertainty.

Tensar Earth Technologies, Inc., supplied the reinforced soil system for this project. The system included polyethylene geogrids, concrete facing blocks (Mesa blocks), and positive mechanical connectors between the blocks and the reinforcements, and between blocks (standard Mesa connectors). The manufacturer provided certified test results for these materials in accordance with CDOT specifications. This system is pre-approved by CDOT with a global reduction factor of 5.82 for the geogrid reinforcements. The compressive strength of the facing blocks is 28 MPa. The length, width, and height of block are 0.457 m, 0.279 m and 0.203 m respectively. As shown in Figs. 3 and 4, a 19 mm

maximum size crushed stone was placed for a minimum distance of 0.3 m behind the facing blocks in order to facilitate fill compaction efforts behind blocks. This gravel zone also provides internal drainage system and prevents the migration of fines to the wall facing.

As shown in Fig. 3, three grades of geogrid reinforcements were used: UX 6 below the foundation, and UX 3, and UX 2 behind the abutment wall. Table 3 summarizes the ultimate strength and the LTDS for these geogrids along with the values required by design. Facing connectors were also placed below the bridge foundation (see Fig. 3). The required and measured strength values for the connections between UX 6 geogrid and the blocks are also listed in Table 3. This table indicates that all CDOT requirements for geogrid reinforcements and connections were met. Analyses were performed by the manufacturer to calculate the reinforced soil structures internal and external stability. The output of these analyses satisfied CDOT requirements.

Table 3. Required and Placed Geogrid and Connection Strength

1. Geogrid	Required LTDS (kN/m)	Placed (kN/m)	
		Ultimate Strength	LTDS
Geogrid UX 6	22.2	157.3	27
Geogrid UX 3	6.4	64.2	11
Geogrid UX 2	3	39.3	6.8
2. Connection Strength between Blocks and UX 6 Geogrid		Required (kN/m)	Placed (kN/m)
		22.2	57.7

THE MONITORED CONSTRUCTION AND LOADING STAGES

Construction of the new Founders/Meadows bridge structure was implemented in two phases (see Figure 2) to accommodate traffic needs. Phase I involved the construction of the southern half of the new bridge structure, referred to as Phase I Structure. Temporary wire mesh reinforced MSE walls were constructed to support the northern face of the Phase I structure. Construction for Phase I structure started on July 16, 1998 and was completed on December 16, 1998. Traffic was then switched from the old bridge structure to the Phase I structure. The existing bridge was then removed. During the second phase, the Phase I structure was extended to construct the northern half of the new bridge, which is referred to as the Phase II structure. Phase II construction started on January 19, 1999 and was completed on June 30, 1999.

A comprehensive instrumentation program was designed and conducted in two phases: Phases I and II, which correspond to the construction of the Phase I and Phase II structures, respectively. The location and layout of the instrumented 200, 400, and 800 sections is shown in Figs. 2, and 3, respectively. Sections 200, 400 are located at the center of Phase I structure and Section 800 is located at the center of Phase II structure. Instrumentation results were collected during eight consecutive stages, six of them during construction and two stages after the structure was opened to traffic. These stages (see Figs. 3 and 6) are:

Stage I. Construction of the front MSE wall up to the bridge foundation elevation.

Stage II. Placement of the spread footing and abutment wall where the girders will be seated, and completion of the front wall construction.

Stage III. Placement of girders.

Stage IV. Placement of the reinforced backfill behind the abutment wall from the bridge foundation elevation to the bottom of the sleeper foundation.

Stage V. Placement of the of bridge deck.

Stage VI. Placement of the approaching roadway structure (including approach slab) and other minor structures. By the end of this stage, the total vertical contact stress exerted directly underneath the bridge foundation was estimated as 115 kPa.

Stage VII. Opening of the bridge to traffic (first year of operation). During this stage, the structure will be subjected to transient live loads from passing traffic and a one complete cycle of weather conditions. By the end of this stage, the total vertical contact stress exerted directly underneath the bridge foundation was estimated to be 150 kPa.

Stage VIII. Long- term performance after one year of opening the bridge to traffic. Monitoring will continue during this stage until the long-term structure movements become negligible.

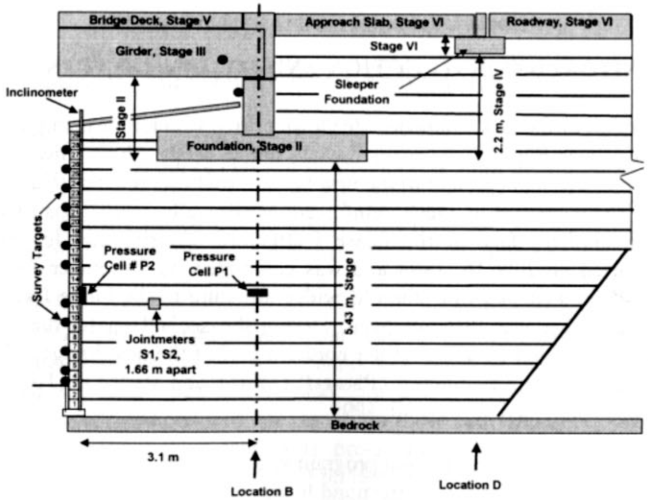

Fig. 6. Instrumented Section 400 indicating Construction Stages

Table 4 shows the start and completion date of each stage along Sections 200, 400 and 800. Note that Stage IV occurred before Stage III on Phase I Structure. Table 5 shows the estimated changes in vertical stresses developed during each stage along locations B under the center of the bridge abutment and D behind the foundation (see Fig. 6). The vertical stresses along location B are either due to the self-weight of the backfill (estimated using a unit weight of 22.1 kN/m^3) during Stage I or the change in vertical contact stress exerted directly underneath the bridge foundation during all subsequent stages. The vertical stress along location D are due to the self-weight of the backfill and approaching roadway during Stages I through VI, and due to traffic load during Stage VII.

Table 4. Time Progress of the Monitored Construction and Post-Construction Stages

Construction Post-Construction Stages	Phase I Structure		Phase II Structure, Section 800	
	Section 200 Date	Section 400 Date	Date	# Days from Jan. 1, '99
Leveling Pad	7/16/98	8/15/98	1/19/99	19
Stage I Construction	8/ 15/98	9/12/98	2/ 24/99	55
Stage II Construction	9/12/98	9/26/98	3/8/99	67
Stage III Construction	10/6/98	10/12/98	3/10/99	69
Stage IV Construction	9/19/98	10/3/98	3/26/99	85
Stage V Construction	11/25/98		5/25/99	125
Stage VI Construction	12/15/98		6/29/99	179
Stage VII, Traffic	12/16/98		6/30/99	180
Stage VIII, long-term	January 2000-		July 2000-	545-

Table 5. Estimated Changes in Vertical Stresses Experienced during Each Monitored Stage

Stage	Vertical Stress (kPa)	
	Loc. B	Loc. D
Stage I Construction, Sections 400, 800	120	120
Stage I Construction, Section 200	89	89
Stage II Construction	22	0
Stage III Construction	42	0
Stage IV Construction	20	49
Stage V Construction	17	0
Stage VI Construction	14	16
Stages VII and VIII (Bridge in service)	35	12

PHASE I INSTRUMENTATION PROGRAM

Instrumentation Plan

A pilot instrumentation plan was conducted during construction of the Phase I structure in order to obtain information that will tailor the design of a more comprehensive monitoring program to be implemented during construction of the Phase II structure. Specific information to be obtained during this phase included the ranges of structure movements, internal soil strain, and stresses that the structure components would experience under service loads.

Sections 200 and 400, located along Phase I structure (see Fig. 2), were instrumented with survey targets, and Section 400 was additionally instrumented with two pressure cells, two jointmeters, and one inclinometer, as shown in Fig. 6. A surveying instrument was used to collect data to measure the structure movements during different stages. Pressure cell P1 (Geokon Model 4800), used to measure vertical earth pressures, was embedded 2.53 m above the leveling pad, 3.1 m back from the face. Pressure cell P2 (Geokon Model 4810), used to measure horizontal earth pressures, was placed directly behind the facing 2.33 meters above the leveling pad. Two jointmeters (Geokon Model 4420), S1 and S2, located 1.7 m apart were attached to the 6th geogrid layer, 2.23 m above the leveling pad, 1.33 m back from the facing. A vertical inclinometer tube was installed behind the block facing to measure lateral movement of the fill material, either parallel to or perpendicular to the wall.

Pressure Cell and Jointmeter Results

Measurements from pressure cells and jointmeters (Fig. 6) were collected manually using the portable Geokon reader box during all construction stages. The obtained results are summarized in Figures 7a, 7b, and 7c. Estimated vertical and horizontal stresses at the level of pressure cells and jointmeters below the foundation were calculated using Equations (1) and (2).

Fig. 7a shows a good correlation between measured and estimated vertical stress values. Fig. 7b indicates that soil horizontal stresses measured on the facing are less than those estimated using Equations (1) and (2). This implies that current design procedures overestimate the loads carried by the facing. Due to compaction, horizontal soil stresses are locked-in within the soil mass. This induces comparatively high overconsolidation ratios within the reinforced soil mass. Ratios between measured horizontal and estimated vertical stresses were estimated. This ratio was 2.01 for the first measurement, when just 0.1 m of backfill had been placed and compacted on top of the P2 gauge. However, the measured horizontal stresses decreased in subsequent measurements. The decrease in horizontal stresses can be attributed to a small lateral outward movement of the

facing. As increasing vertical loads are applied, the initially overconsolidated fill gradually reaches a normally consolidated state. A ratio of approximately 0.11 was achieved towards the end of construction (Stage VI).

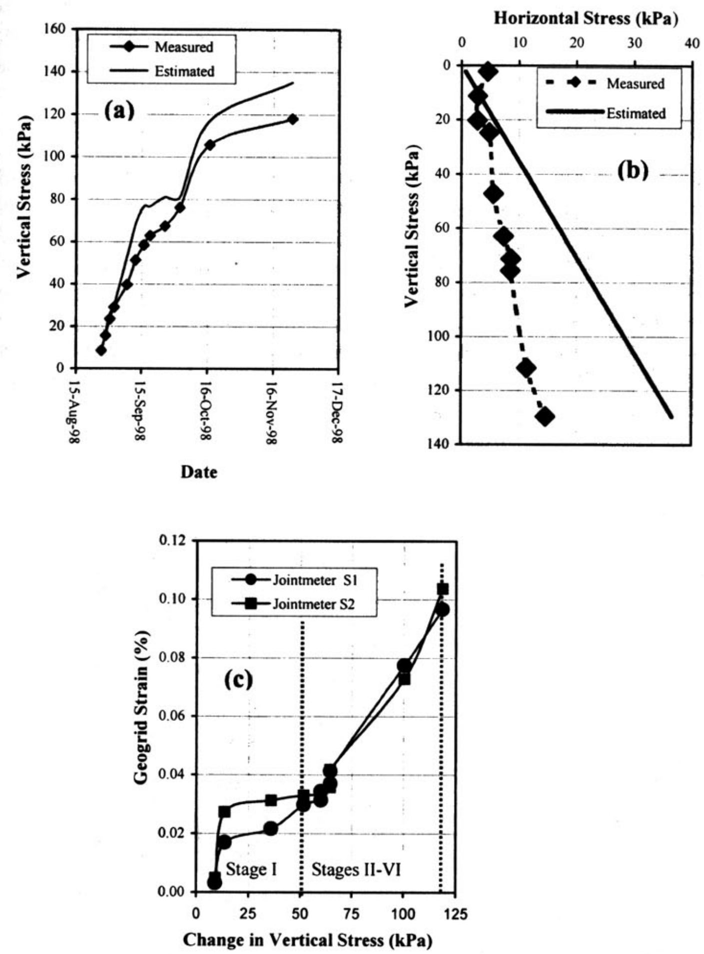

Fig. 7. Measured and Estimated Data on Section 400 of Phase I Structure: (a) Vertical Stresses, (b) Horizontal Stresses, and (c) Geogrid Strains.

The measured geogrid strains versus the estimated soil vertical stresses at various construction stages are shown in Fig. 7c. The figure indicates that the measured geogrid strains from two jointmeters correlate well with each other. The geogrids experienced very low strains, on the order of 0.1 %. As shown in Fig. 7c, sharp increases in geogrid strain developed immediately after the backfill was placed on top of the gauges, possibly influenced by compaction. The rate of strain decreases significantly during the rest of Stage I construction and during Stage II Construction. Finally, the rate of geogrid straining increased steadily during construction Stages III through VI.

Displacement Monitoring Results

Movements were monitored for Sections 200 and 400. The measured movements of the structure were very small and relatively close to the accuracy range (+/- 3mm) of the surveying results. Fig. 8a summarizes the total outward movement of the front MSE wall (Section 400) induced during Stage 1 Construction as the wall height increased from 18 rows (elevation 3.65 m above leveling pad) to 27 rows (elevation 5.48 m). Fig. 8b summarizes the total outward movements of the front MSE walls (Sections 200 and 400) induced during Construction Stages II to VI. The following observations can be inferred from the monitored displacement data:

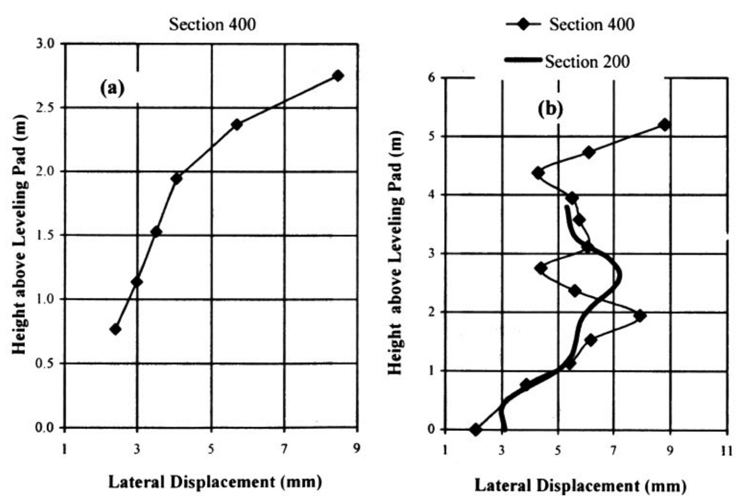

Fig. 8. Lateral Displacements Monitored on Phase I Structure: (a) during Stage I Construction, and (b) due to Placement of the Bridge Superstructure.

- The maximum outward movement experienced along section 400 during Stage I construction was approximately 9 mm (Fig. 8a).

- As shown in Fig. 8b, the maximum outward movements experienced during placement of the bridge superstructure (Construction Stages II to VI) along sections 200 and 400 were approximately 7 mm and 9 mm, respectively. Note that Section 400 is 1.4 m higher than Section 200.
- The bridge spread foundation settled almost 14 mm due to the load of the bridge and approaching roadway structures (Construction Stages II to VI).
- Along section 400, the leveling pad settled vertically almost 5 mm during Stage I Construction. It additionally settled vertically almost 6 mm when the bridge and approaching roadway structures were placed (Construction Stages II to VI).

PHASE II INSTRUMENTATION PROGRAM

Instrumentation Plan

The lessons learned from the pilot Phase I instrumentation were evaluated and considered in the design of the more comprehensive Phase II instrumentation program. These findings included:

- The movements observed in the structure and the strains developed in the geogrids were comparatively small. Consequently, more accurate and sensitive strain gauges (Geokon Model 4050) having a smaller (0 to 0.7%) measurement range than the jointmeters used in Phase I were selected for the Phase II instrumentation plan.
- More accurate pressure cells, having a smaller (0-345 kPa) measurement range than those used in Phase I, were selected for the Phase II instrumentation plan.

Section 800, located along the front MSE wall of the East Abutment (see Fig. 2), was heavily instrumented with survey targets, pressure cells, strain gauges, moisture gauges, and temperature gauges as shown in Fig. 9. Survey targets were placed on the facing blocks, girders, abutment walls, bridge deck, and approach slab and approaching roadway. Displacement monitoring data along the three monitored sections should provide a picture of the overall movements of the structure, including information on the differential settlement between the bridge and approaching roadway structures.

Four critical locations along section 800 below the foundation were instrumented, as follows:

1. Location A, close to the facing. Data to be measured here would be particularly useful for guiding the structural design of the facing and of the connection between facing and reinforcements.
2. Locations B and C along the center and interior edge of the abutment foundation. Information collected at these locations would be particularly relevant for the design of the reinforcement elements.

3. Location D, behind the bridge foundation, and horizontal plane at the base of the fill. Data measured at these locations would be useful to estimate the external forces acting behind and below the reinforced soil mass.

Instrumentation results to be collected at the different locations within the reinforced soil mass (see Fig. 9) will provide a complete picture of the distribution of internal soil stresses, geogrid strains, and soil temperatures below the bridge foundation and behind the abutment wall, as well as soil moisture information below the approach slab. The measured data during various stages will be used to evaluate design procedures and assumptions of the front and abutment MSE walls.

The reliability of the measured data and the suitability of assuming plain strain conditions was investigated by comparing the results of gages placed at the same elevation, same distance from the facing, but at different locations in the longitudinal direction along the wall. Fig. 9 shows locations where two gages were placed at the same elevation and distance from the facing, either North or South of the control section.

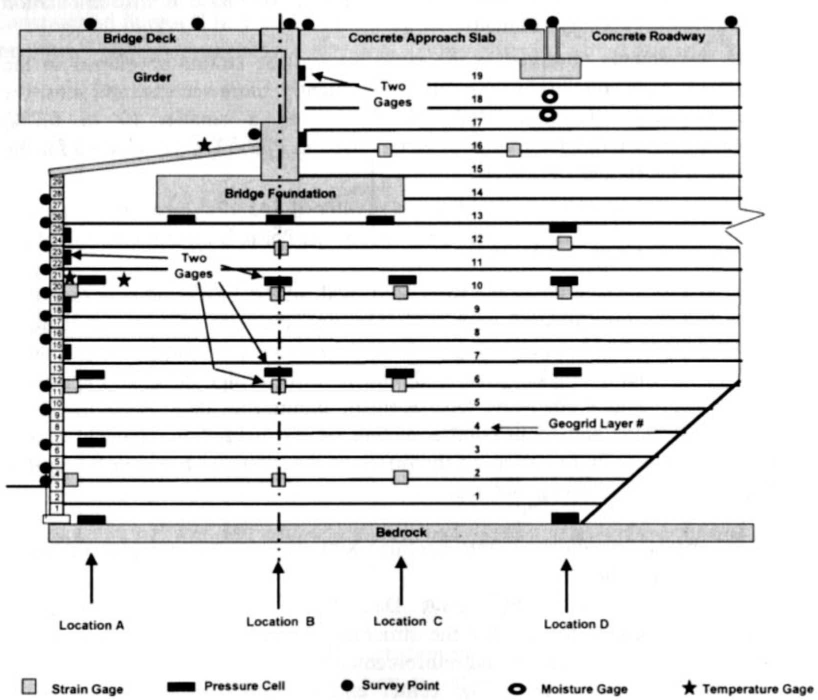

Fig. 9. Instrumentation Layout at Section 800 (Phase II structure)

Preliminary Phase II Instrumentation Results

Soon after each gage was placed during Phase II construction, data was then collected and recorded continuously during all stages using a data logger. A significant amount of data was collected during the construction stages. Analysis of existing raw data and collection of additional monitoring data continues by the time of preparation of this paper. Examples of the monitoring records collected during the construction stages (until June 30, 1999, or 180 days from January 1, 1999) are presented. Fig. 10 shows the vertical stresses measured by the pressure cells placed between the 10th and 11th geogrid layers (see Fig. 9). Fig. 11 shows the geogrid strains measured by the strain gages placed along the 6th geogrid layer (see Fig. 9). Evaluation of this preliminary information provides significant insight regarding the trends in the behavior of the structure. The following observations can be made:

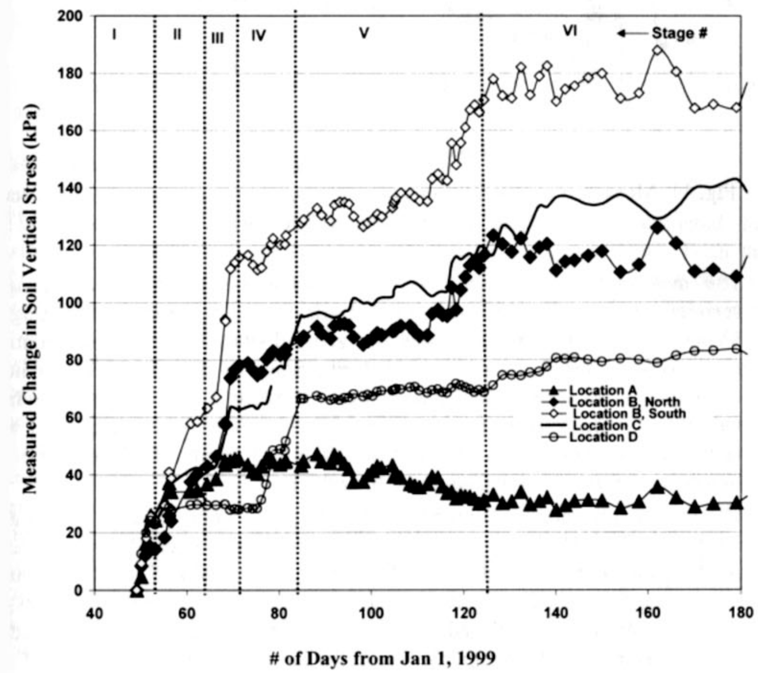

Fig. 10. Measured Vertical Soil Stresses during all Construction Stages (Measured between Geogrid Layers 10 and 11 along Section 800)

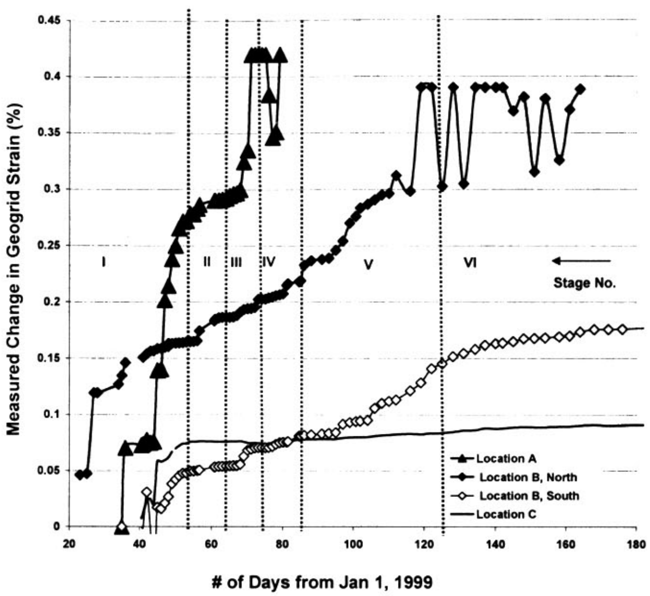

Fig. 11. Measured Geogrid Strains during all Construction Stages (Measured on the 6th geogrid layer of Section 800)

- The measured response from both the pressure cells and strain gauges correlates very well with the applied loads during the construction stages. Sharp increases in vertical stresses and geogrid strains below the foundation can be observed in Figures 10 and 11 during Stage III construction (placement of girders). The pressure cell placed along Location D responded to loading only during Construction Stages IV and VI, when the overlying backfill and approach slab were placed, but did not show any response when the bridge superstructure was placed (Stages II, III and V).
- The results from pressure cell gauges placed at the same location (North and South in Fig. 10) run almost identical to each other during Stage I construction, and parallel to each other during Construction Stages IV through VI. During Stage III Construction (Placement of girders), the vertical stresses measured by the gauge placed on the south side of the control line exceeded those measured by the gauge placed on the north side. The placement of the girders in two days starting from the south side might have caused this difference.
- Fig. 10 indicates that vertical stresses differ significantly from location to location (same elevation), and that they are not uniform as often assumed in the design procedure. The lowest vertical stresses occurred close to the facing and the highest vertical stresses occurred along Location B, the centerline of the bridge abutment. This distribution of vertical stresses suggests no potential for overturning the structure.

- As shown in Fig. 11, sharp and different increases in geogrid strain developed immediately after backfill was placed on the top of the gauges (during Stage I construction), possibly under the influence of compaction. Geogrid strain information collected by strain gauges placed at the same location (see Fig. 9, North and South gages) exhibit almost parallel response during the subsequent construction stages. Consequently, the differences in initial geogrid strains can also be attributed to the effect of compaction during placement of the backfill and also to differences in the initial tensioning of the geogrid reinforcements.
- The maximum geogrid strains experienced during construction of Phase II structure is almost 0.45 %.
- The maximum geogrid strain occurred close to the facing (location A) along the 6th geogrid layer (see Fig. 11), and along Location C for the 10th geogrid layer (not shown here).
- The preliminary monitoring results of horizontal earth pressures at the facing and of the reinforcement maximum tensile strains are well below design values.

SUMMARY AND CONCLUSIONS

The Colorado Department of Transportation successfully completed the construction of the new Founders/Meadows Bridge in July of 1999. Both the bridge and the approaching roadway structures are supported by a system of geosynthetic-reinforced segmental retaining walls. The design and construction of this structure was possible because the predicted settlements of the reinforced soil structure and bridge foundation soil were small. This is a unique structure in terms of design and construction because it provided bridge support, has the potential to alleviate the bridge bump problem, and allowed for a relatively small construction area when compared to the use of deep foundation. The reinforced soil mass was extended beneath the bridge foundation and the approaching roadway structure in order to minimize differential settlements between the bridge and the approaching roadway structures. A thin compressible material was incorporated between the reinforced backfill and the integral bridge abutment wall to allow for the thermally-induced movements of the bridge superstructure without affecting the backfill. By January, 2000, this structure provides comfortable rides with no signs for the development of the common bridge bump problem.

The Founders/Meadows structure was considered experimental and a comprehensive material testing, instrumentation and monitoring programs were incorporated into the construction operations. These programs would allow assessment of the overall structure performance and evaluation of CDOT and AASHTO design assumptions and procedures regarding the use of reinforced soil structures to support bridge and approaching roadway structures. The results of conventional 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. Results from a pilot (Phase I) instrumentation program and some preliminary results from a more comprehensive (Phase II) instrumentation program are also presented and discussed in the paper. Three sections were instrumented to provide information on the external movements of this structure, developed internal soil stresses, geogrid strains, and moisture content during various construction stages and after the structure opening to traffic. The overall performance of this structure under service load before the bridge was opened to traffic has been satisfactory because of the comparatively small movements monitored so far.

The measured vertical stresses change significantly across different locations (same elevation), and they are not uniform as often assumed in the design. The lowest vertical stresses occurred close to the facing and the highest vertical stresses occurred along the centerline of the bridge abutment. This distribution of vertical stresses suggests no potential for overturning the structure. This can be explained by the flexibility of the reinforced soil structure which redistribute any overturning stresses. Geogrid strains developed during initial stages of filling and compaction were typically larger than those developed during subsequent construction stages. Differences in initial tensioning in the geogrid reinforcements during placement and in the effect of compaction were observed to have significant effect on the developed geogrid strains. The preliminary results of horizontal pressures at the facing and of the reinforcement maximum tensile strains are well below the design values. This suggests that current CDOT and AASHTO design procedures are conservative.

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