Report No. CDOT-DTD-R-2000-5

PERFORMANCE OF GEOSYNTHETIC-REINFORCED WALLS SUPPORTING THE FOUNDERS/MEADOWS BRIDGE AND APPROACHING ROADWAY STRUCTURES

REPORT 1: DESIGN, MATERIALS, CONSTRUCTION, INSTRUMENTATION, AND PRELIMINARY RESULTS

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DEPARTMENT OF TRANSPORTATION
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COLORADO DEPARTMENT OF TRANSPORTATION
RESEARCH BRANCH

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16. Abstract In July of 1999, the Colorado Department of Transportation (CDOT) successfully completed the construction of the new Founders/Meadows Bridge near Denver. In this project, both the two-span bridge and the approaching roadway structures are supported by a system of geosynthetic-reinforced segmental retaining walls. The reinforced soil system not only provides bridge support, but it was also designed to alleviate the common bridge approach bump problem. 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. 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. Three sections were instrumented to provide information on external movements, internal soil stresses, temperatures, moisture content, and geogrid strains during various construction stages and after the structure opening to traffic. This report initially presents an overview of the reinforced soil wall system design, materials, construction stages, and the instrumentation program. Results of large-size triaxial and direct shear tests on the gravelly backfill soil material and preliminary instrumentation results are presented and discussed. The instrumentation results suggest that the overall performance of this structure, before its opening to traffic, has been satisfactory. CDOT design guidelines of assuming zero cohesion for the backfill strength and testing specimens without the gravel portion of the soil lead to significant underestimation of the actual shear strength of the backfill.

Implementation The design and construction of geosynthetic-reinforced walls supporting bridge and approaching roadway structures should be considered in the future for field conditions similar to those encountered in the Founders/Meadows Structure. CDOT design procedure should be enhanced and made more flexible to employ the proper shear strength parameters of the backfill.

17. Key Words	**18. Distribution Statement**
bridge approach bumps, shear strength, backfill, shallow foundations, retaining walls	

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CONVERSION TABLE

U. S. Customary System to SI to U. S. Customary System

(multipliers are approximate)

Multiply (symbol)	by	To Get (symbol)	Multiply	by	To Get
		LENGTH			
Inches (in)	25.4	millimeters (mm)	mm	0.039	in
Feet (ft)	0.305	meters (m)	m	3.28	ft
yards (yd)	10.914	meters (m)	m	1.09	yd
miles (mi)	1.61	kilometers (km)	m	0.621	mi
		AREA			
square inches (in^2)	645.2	square millimeters (mm^2)	mm^2	0.0016	in^2
square feet (ft^2)	0.093	square meters (m^2)	m^2	10.764	ft^2
square yards (yd^2)	0.836	square meters (m^2)	m^2	1.195	yd^2
acres (ac)	0.405	hectares (ha)	ha	2.47	ac
square miles (mi^2)	2.59	square kilometers (km^2)	km^2	0.386	mi^2
		VOLUME			
fluid ounces (fl oz)	29.57	milliliters (ml)	ml	0.034	fl oz
gallons (gal)	3.785	liters (l)	l	0.264	gal
cubic feet (ft^3)	0.028	cubic meters (m^3)	m^3	35.71	ft^3
cubic yards (yd^3)	0.765	cubic meters (m^3)	m^3	1.307	yd^3
		MASS			
ounces (oz)	28.35	grams (g)	g	0.035	oz
pounds (lb)	0.454	kilograms (kg)	kg	2.202	lb
short tons (T)	0.907	megagrams (Mg)	Mg	1.103	T
		TEMPERATURE (EXACT)			
Farenheit (°F)	5(F-32)/9 (F-32)/1.8	Celcius (° C)	° C	1.8C+32	° F
		ILLUMINATION			
foot candles (fc)	10.76	lux (lx)	lx	0.0929	fc
foot-Lamberts (fl)	3.426	candela/m (cd/m)	cd/m	0.2919	fl
		FORCE AND PRESSURE OR STRESS			
poundforce (lbf)	4.45	newtons (N)	N	.225	lbf
poundforce (psi)	6.89	kilopascals (kPa)	kPa	.0145	psi

Performance of Geosynthetic-Reinforced Walls Supporting the Founders/Meadows Bridge and Approaching Roadway Structures

Report 1: Design, Materials, Construction, Instrumentation, and Preliminary Results

by

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Executive Summary

This report 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.

Implementation Statement

- CDOT design procedure should be enhanced and made more flexible to employ the proper shear strength parameters of the backfill. CDOT current design guidelines of assuming zero cohesion for the backfill strength and testing specimens without the gravel portion of the soil lead to significant underestimation of the actual shear strength of the backfill.

- The use of reinforced soil walls to support both the bridge and approaching roadway structure is recommended for field conditions similar to those encountered in the Founders/Meadows structure. The most interesting features of this application are:
    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 (i.e., noise, mobilization, and cost)
4. Allows for construction in stages and smaller work areas.

The field and design limitations of using reinforced soil walls to support bridge footing are:

1. Firm foundation is needed to minimize anticipated settlement of the reinforced soil system.
2. Bridge should have no scour concerns.
3. Bridge should require full-height abutments and wing walls.

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1.0 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 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 mechanically stabilized earth (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. In this project, both the bridge and the approaching roadway structures are supported by a system of geosynthetic-reinforced segmental retaining walls. A key element in the design was the need to support the high concentrated loads from the bridge foundation structure and the comparatively smaller loads from the approaching highway structure without inducing significant differential settlements. Figure 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 opened 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 using movement information collected at the three monitored sections.

- 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. Measured backfill material strength, structure external and internal movements, developed soil stresses, geogrid strains, and soil moisture will be compared with the values and performance expected from the design.

- Numerical analysis of the behavior of the reinforced soil system using representative stress-strain-strength parameters for the gravelly backfill material. The numerically predicted structure behaviors of the monitored sections will be compared with those assumed in the design and measured in the field.

This report initially presents an overview of the new Founders/Meadows design, materials and construction stages. 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 opened to traffic, are presented and discussed. Data collection and analysis continues at the time of preparation of this report. Subsequent publications will summarize all the findings from the instrumentation and monitoring programs, and implications of these results in the design of segmental retaining walls supporting bridge and approaching roadway structures. Appendix A presents results of external and internal stability analyses of the front MSE walls. Appendix B contains an album of pictures taken during and after the construction of the Founders/Meadows Bridge. Appendix C presents a detailed description of the instruments and techniques employed in this study and their applications.

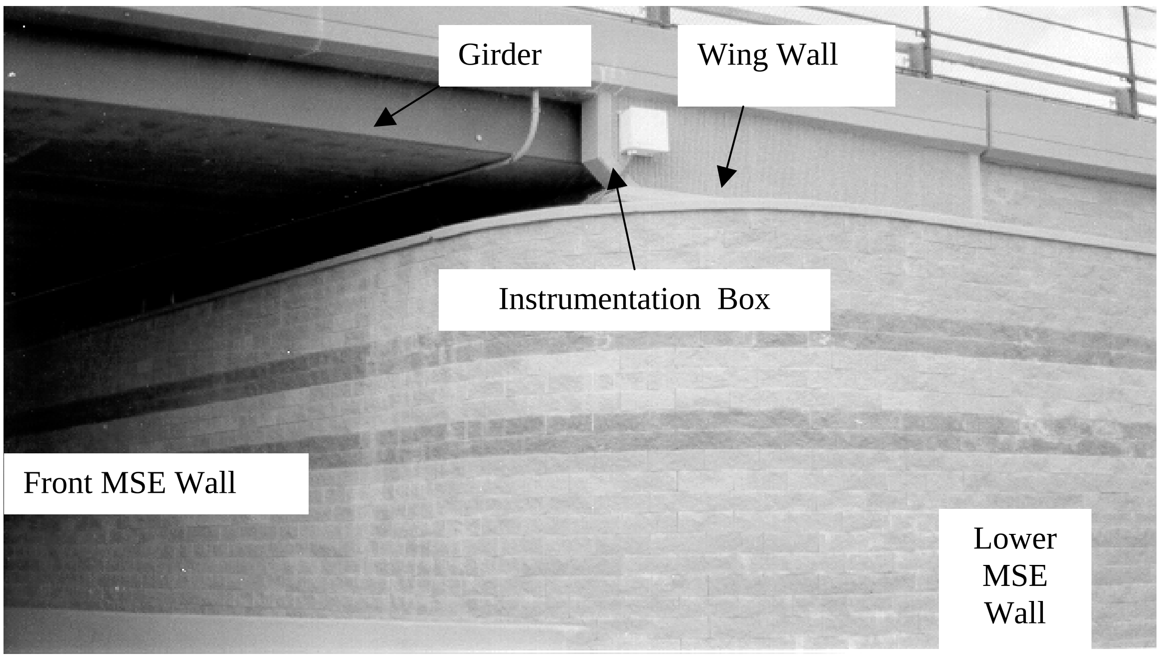

Figure 1. View of the south-east side of the completed Founders/Meadows Bridge.

2.0 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 approximately 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 (4).

Full-scale tests of geosynthetic-reinforced soil abutments with segmental block facing have been conducted by the FHWA, e.g. Adams in 1997 (1) and by CDOT, e.g. Ketchart and Wu in 1997 (6). 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 (4). In the CDOT study, the abutment structure was constructed with roadbase backfill reinforced with layers of a woven polypropylene geotextile having ultimate strength of 70 kN/m. Dry-stacked hollow-cored concrete blocks were used as facing. The abutment was rectangular in shape (4.6 m by 7.6 m) and 7.6 m tall. 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. in 1996 (5) concluded that the performance of top-loaded reinforced experimental embankments are highly promising. Won et al. in 1996 (9) 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. in 1997 (8) 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 (7,10). Three common causes for the development of bridge bumps are examined in this study. The first is 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. The second cause is expansion and contraction of the bridge decks and girders, which causes 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. As the bridge abutment wall moves due to the expansion and contraction of the bridge girders and deck, it alternately pushes into and pulls away from the backfill behind the abutment wall, leading to the development of a void near the abutment wall under the approach slab. The third cause is 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 1 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 (7). 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.

3.0 DESCRIPTION OF THE FOUNDERS/MEADOWS BRIDGE

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. Figure 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.

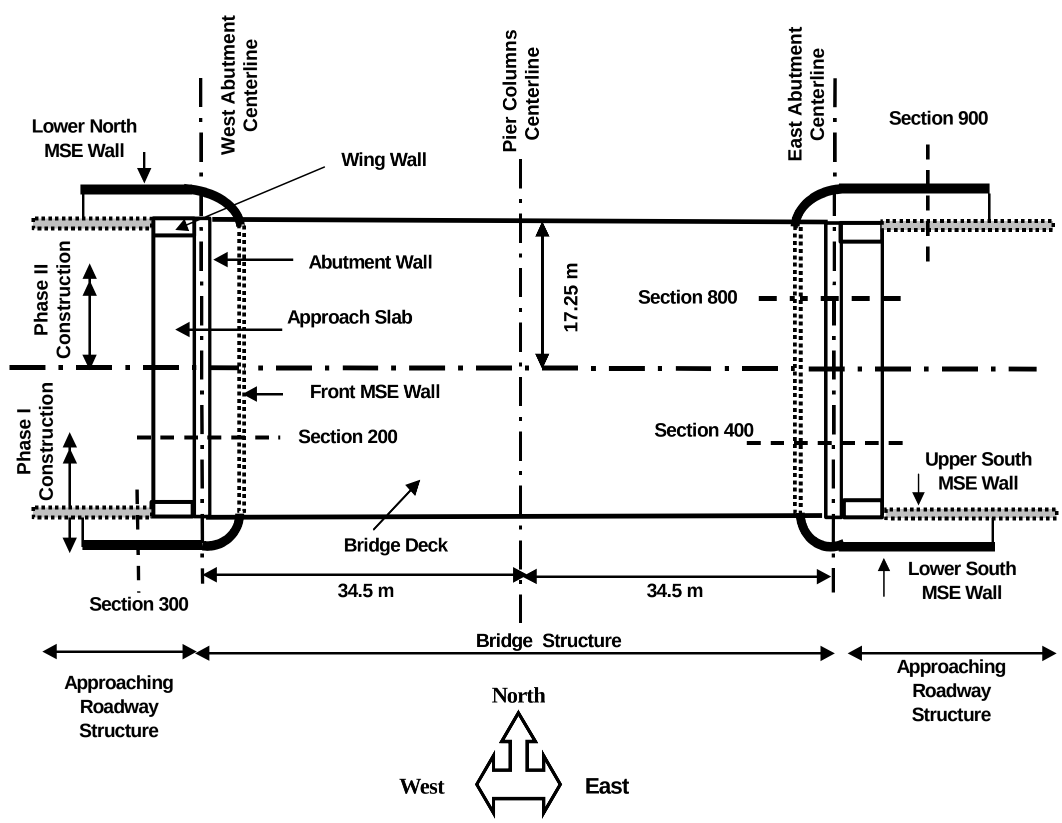

Figure 2. Top view of the Founders/Meadows Bridge.

Figure 3 shows a typical monitored cross-section through the front MSE wall and abutment wall (sections 200, 400, and 800 in Figure 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 Figures 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 Figure 2). These walls 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 (Figure 2), which in turn are supported by a spread footing founded on bedrock in the median of Interstate 25.

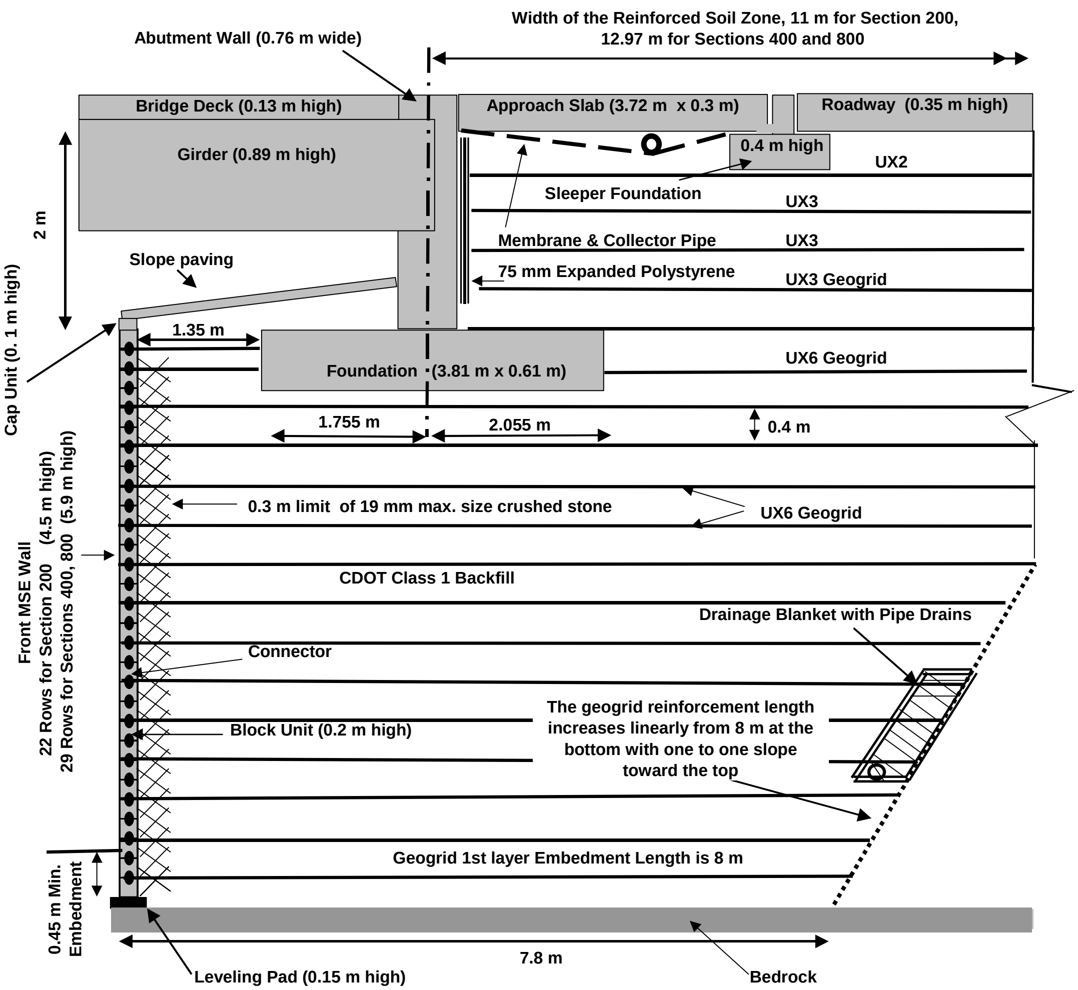

Figure 3. Typical monitored cross-section (sections 200, 400, and 800 in Figure 2) through the front and abutment MSE walls.

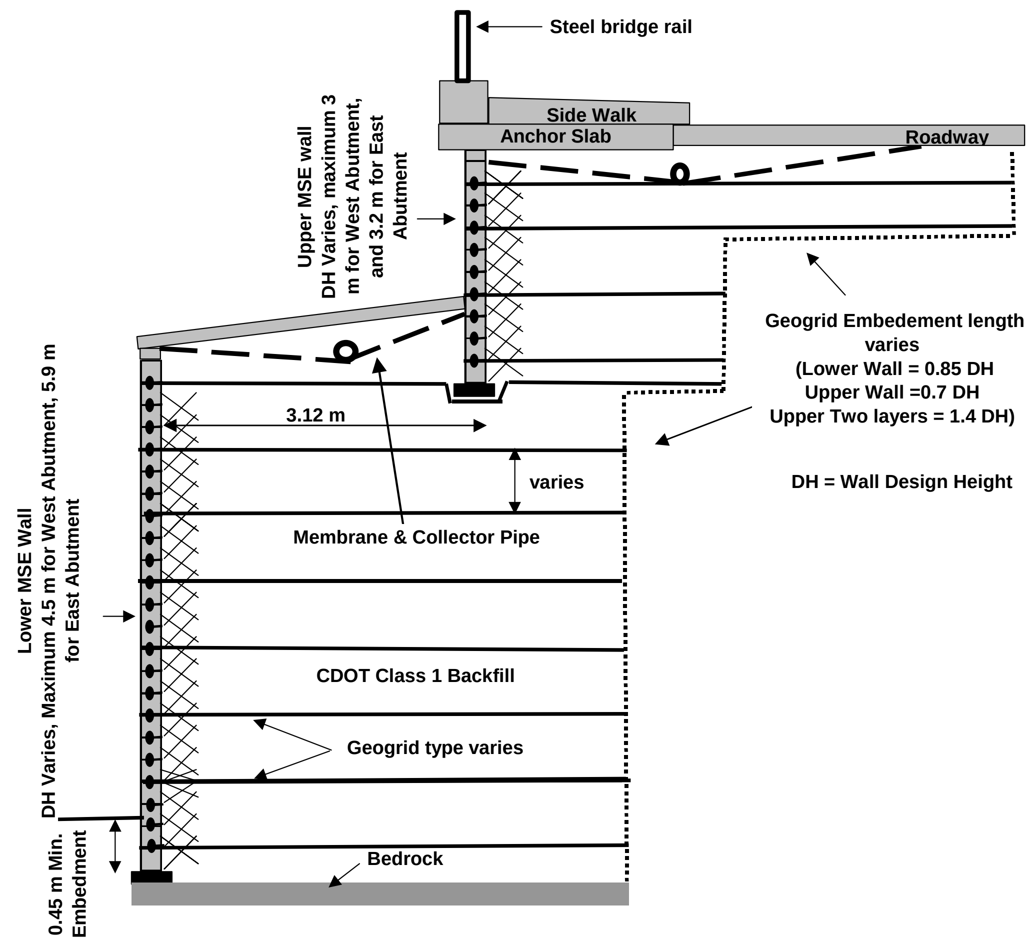

Figure 4. Typical cross-section along the upper and lower MSE walls (e.g., sections 300 and 900 in Figure 2).

4.0 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.). The geotechnical investigation report also recommended for the design of MSE walls constructed with Class 1 backfill a friction angle of 34 degrees, a unit weight of 19.6 kN/m3, and a maximum allowable bearing pressure of 240 kPa. The maximum allowable bearing pressure for the claystone bedrock was recommended as 480 kPa.

5.0 DESIGN OF THE FRONT AND ABUTMENT MSE WALLS

The design of the MSE walls along sections 200, 400, and 800 (Figure 2) followed AASHTO and CDOT 1996 guidelines. Figure 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.

5.1 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:

$$\sigma_v = \gamma z + q + \Delta\sigma_v \quad (1)$$

$$\sigma_h = K_a \sigma_v \quad (2)$$

where γ is the assumed backfill unit weight 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 stresses 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 pressures 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 (Figure 3). 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 pressures 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 Bridge and, if its performance is acceptable, it will replace the more expensive flow-fill technique.

5.2 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 Figure 3). A comparatively long reinforced soil zone below the bridge and approaching roadway structure was considered necessary 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.

5.3 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. It can also lead to migration of the fines out of the soil mass which would soften the soil mass. This will cause problems ranging from soil settlement (leading to bridge bump) to cracks at the top of MSE walls. 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 Figure 3) to divert any infiltration and ground water from the reinforced soil mass.

5.4 Stability and Settlement Analyses

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.

Analyses were performed by the manufacturer to calculate the reinforced soil structure’s internal and external stability, summarized in Appendix A. The case shown in Appendix A was analyzed using the 1996 AASHTO TF27 guideline which was reviewed and accepted by CDOT in 1996. The output of manufacturer's analyses satisfied all of CDOT requirements.

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 (6) 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.

6. MSE BACKFILL MATERIALS

6.1 Overview

The backfill material for the MSE walls was specified as CDOT Class 1 Structural Backfill. The construction requirements for CDOT Class 1structure 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).

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.

6.2 Results of Conventional Direct Shear Tests

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.

6.3 Results of Large-Size Direct Shear, and Large-Size Triaxial Tests

A large-size testing program was additionally performed using 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 from the triaxial tests for future numerical simulation of the structure and comparison between numerical and field monitoring results. The specimens were prepared at conditions that duplicates the compaction level and moisture measured in the field (see Table 2).

Compacted specimens representing the entire backfill soil were prepared at an average dry unit weight of 21 kN/m^3 and moisture content of 5.6% and tested using a large square direct shear box (300 mm wide and 200 mm high) in accordance with ASTM D3080. The tests were conducted under consolidated-drained conditions at normal stresses of 69 kPa, 138 kPa, and 207 kPa. The test results are shown in Figure 5. The change in the cross-sectional area of the specimens as shearing proceeds was considered in the calculation of the applied shear stress. 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. The specimens were prepared at an average dry unit weight of 20.6 kN/m^3 and moisture content of 5.7%, and 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 pressures of 69 kPa, 138 kPa, and 207 kPa. The test results are summarized in Figures 6 and 7, which show the deviatoric stress-axial strain and the volumetric strain versus axial strain curves, respectively. The peak deviatoric stresses at confining pressures of 69 kPa, 138 kPa, and 207 kPa were, respectively, 513 kPa (at 2.54% and +0.7 % of axial and volumetric strains), 848 kPa (at 4.68% and 0.15% of axial and volumetric strains), and 973 kPa (at 3.55% and -0.12% of volumetric and axial strains). Note that positive and negative values of the volumetric strain were referred to, respectively, dilation and contraction of the specimen. All specimens showed a symmetrical lateral bulging at failure. The Mohr-Columb shear strength envelope defined using triaxial tests 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.

Using the measured strength parameters from the large size triaxial tests, the soil active earth pressure, σ_h , should be estimated as:

$\sigma_h = K_a\sigma_v - 2C\,(K_a)^{1/2=}\ 0.22\,\sigma_v - 65.8 = 0$ for $\sigma_{v\,\leq}$ 300 kPa (3)

where K_a is calculated for a friction angle of 39.5° and C is the measured cohesion of 69.8 kPa. Equation 3 leads to much smaller horizontal earth pressure (0 if the vertical soil stress is kept below 300 kPa) than Eq. 2 used in the design. Furthermore, Equation 3 implies that the soil system itself (without geogrid reinforcement) would provide enough support up to a vertical earth pressure of 300 kPa with out failure if the soil was allowed to yield (expand or move laterally) completely. Although the vertical earth pressure encountered in the front MSE walls are much less than 300 kPa, the presence of geogrid reinforcement in the MSE walls is needed in this case to limit the lateral displacement of the wall and the backfill behind.

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 (Table 2) and the discussion above it is clear that common guidelines of 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.

The results of the three triaxial tests were used to estimate the hyperbolic model parameters (2,3), as listed in Table 3. As shown in Figure 6, the deviatoric stress-axial strain relationships of the triaxial tests and the hyperbolic model prediction are in good agreement. However, as expected, the hyperbolic model cannot represent the dilatant behavior observed in the triaxial test results because the soil is assumed to be isotropic elastic.

Table 3. Summary of the hyperbolic model parameters.

Stress-Strain Relationship: K, n; and $K_{ur,}$ n: are a set of parameters relating the initial tangent Young modulus to the confining stress during loading and unloading, respectively. R_f = ratio of deviatoric stress at failure to the ultimate deviatoric stress	Loading	K = 1070 n = 0.7 R_f = 0.82 Cohesion 69.8 kPa Peak friction angle= 39.5°
	Unloading	K_{ur}= 1090
Volume Change Relationship: K_b and m are parameters relating the bulk modulus to the confining stress	K_b= 700, m= 0	

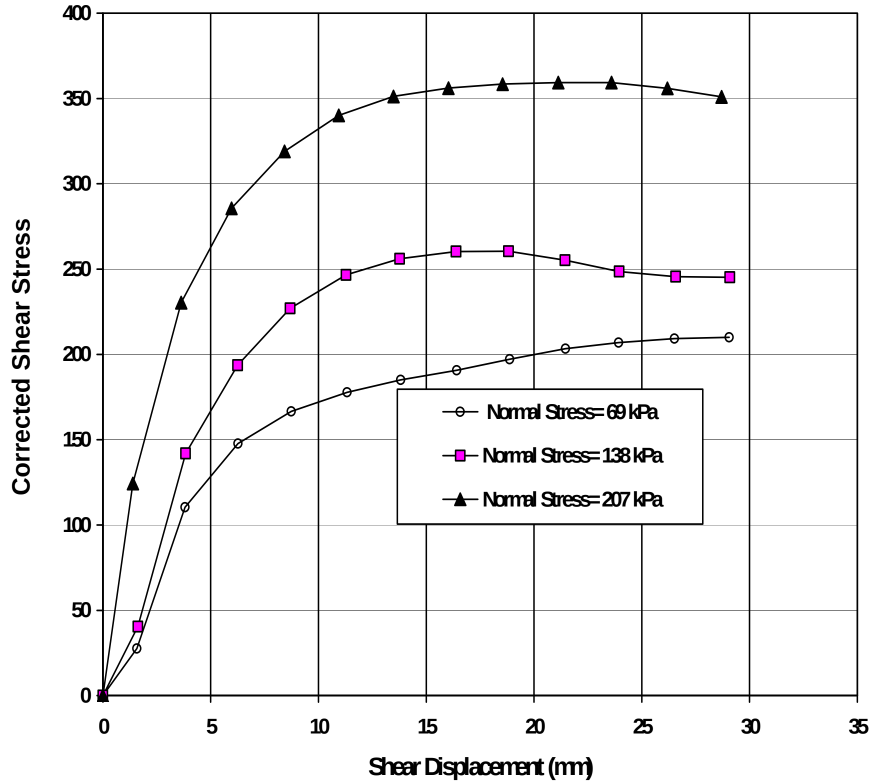

Figure 5. Results of large-size direct shear tests.

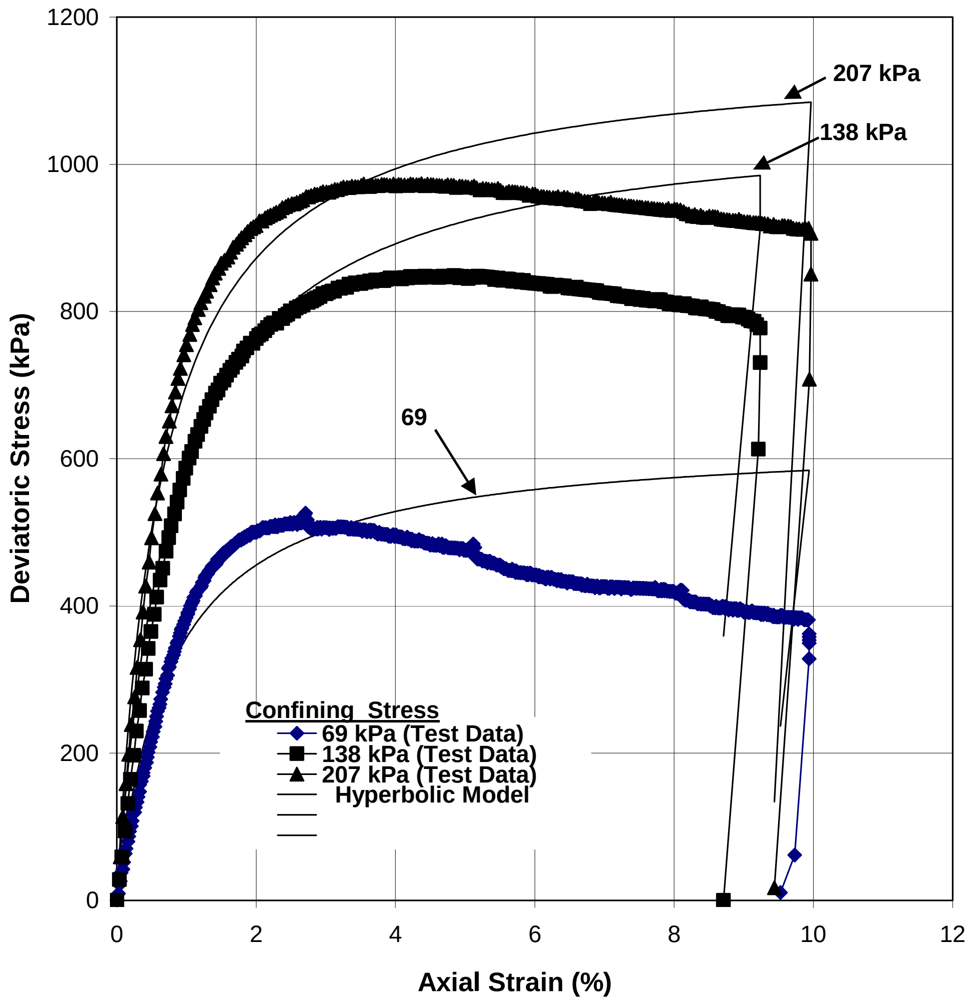

Figure 6. Deviatoric stress-axial strain test results from large-size triaxial tests and predictions from hyperbolic model.

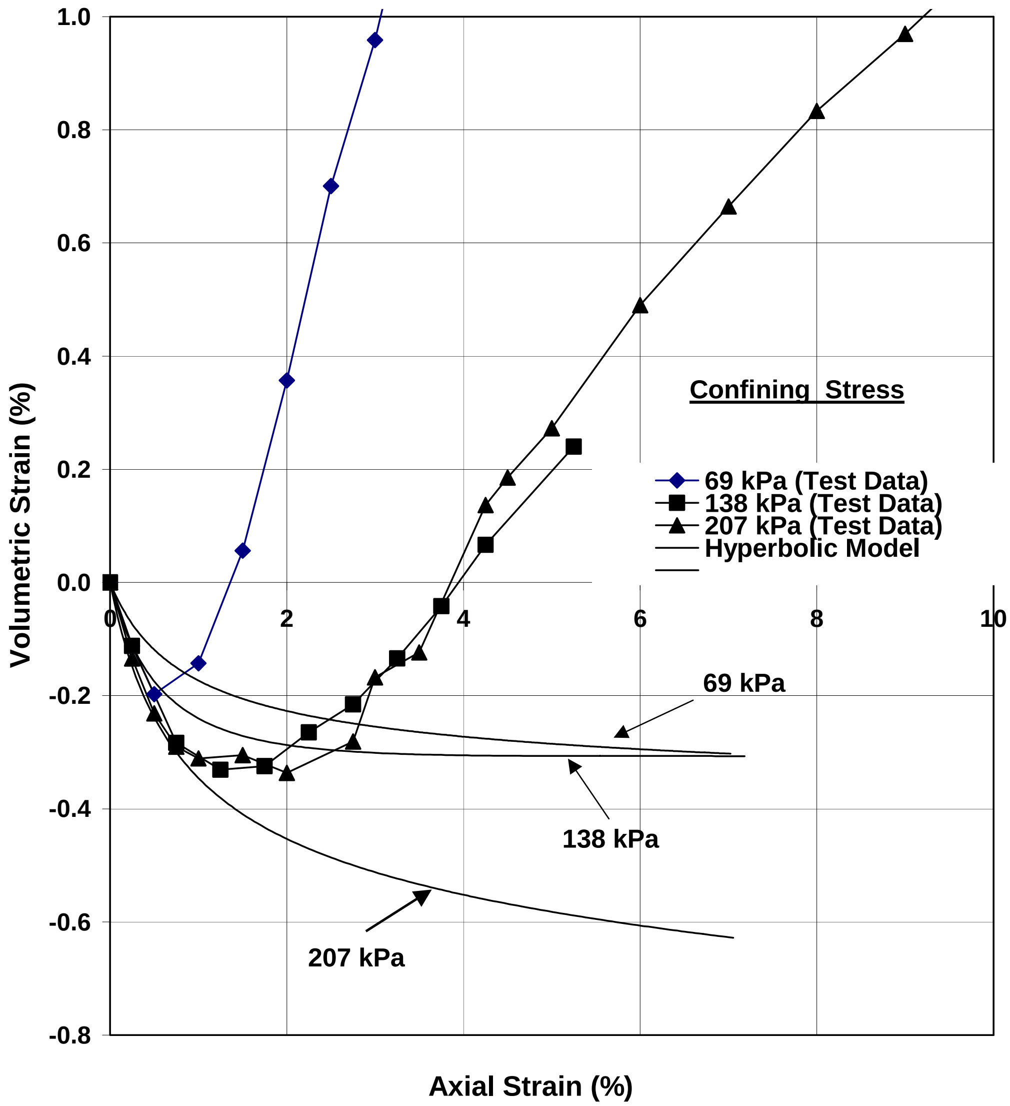

Figure 7. Volumetric strain versus axial strain test results from large-size triaxial tests and predictions from hyperbolic model.

7.0 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 Figures. 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 Figure 3, three grades of geogrid reinforcements were used: UX 6 below the foundation, and UX 3, and UX 2 behind the abutment wall. Table 4 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 Figure 3). The required and measured strength values for the connections between UX 6 geogrid and the blocks are also listed in Table 4. This table indicates that all CDOT requirements for geogrid reinforcements and connections were met. The geogrid stiffness at 0 to 2% strain range (covering the range measured in this study), measured from the wide width tensile tests (ASTM D4595) results, were approximately 2000 kN/m for UX 6 Geogrid and 1000 kN/m for UX 3 Geogrid.

Table 4. 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

8.0 MONITORED CONSTRUCTION AND LOADING STAGES

Construction of the new Founders/Meadows Bridge 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 of the 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 are shown in Figures 2, and 3, respectively. Sections 200 and 400 are located at the center of the phase I structure and section 800 is located at the center of the phase II structure. 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 Figures 3, 8, and 9) 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 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.

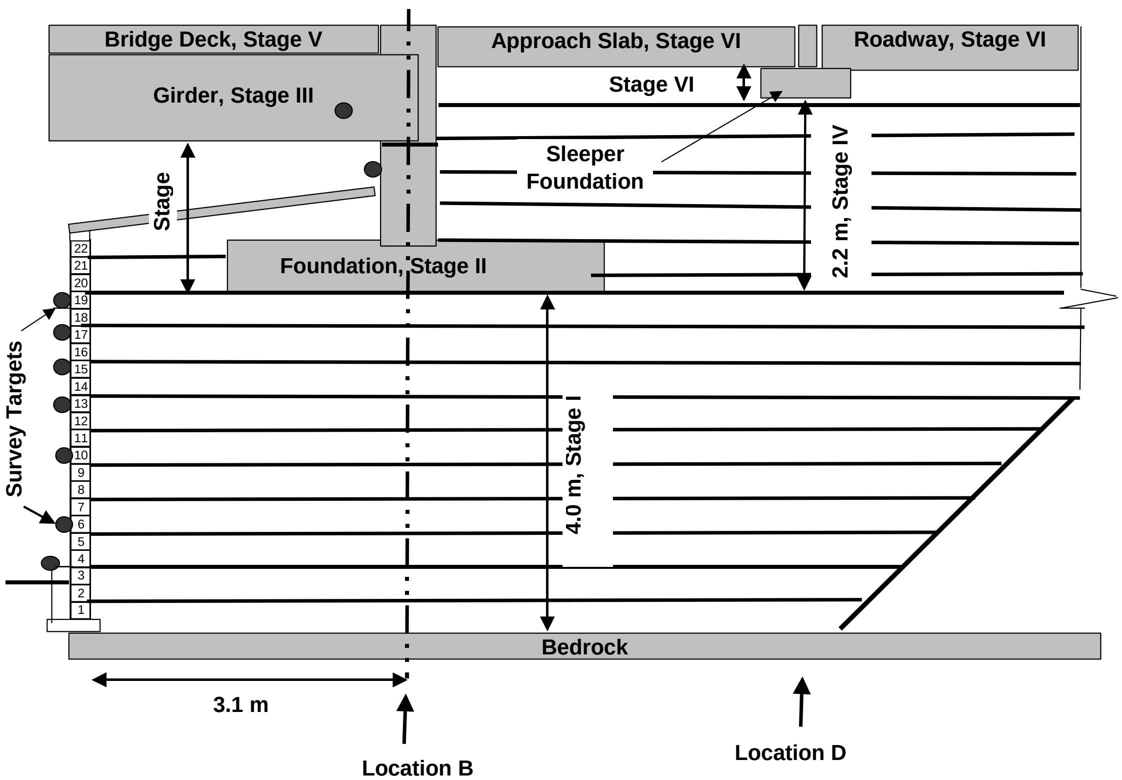

Figure 8. Instrumented section 200 indicating construction stages.

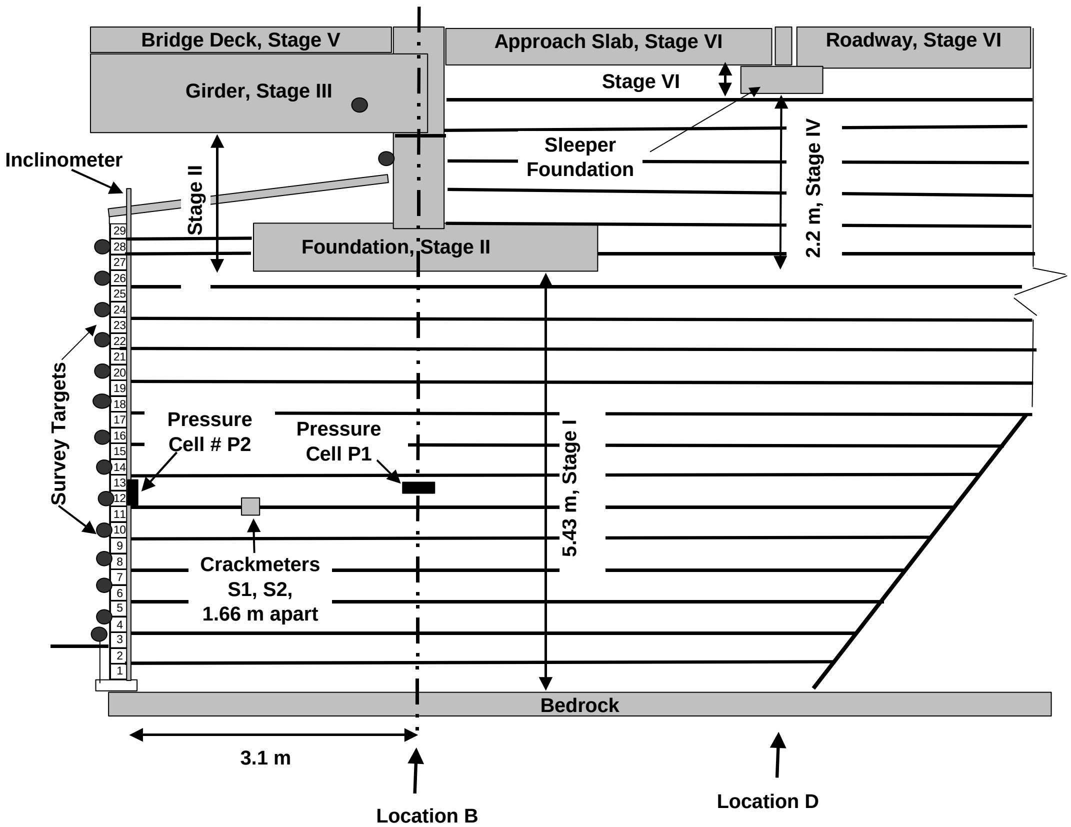

Figure 9. Instrumented section 400 indicating construction stages.

Table 5 shows the start and completion date of each stage along sections 200, 400 and 800. Note that stage IV occurred before stage III on the phase I Structure. Table 6 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 Figures 8, 9). 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) above the leveling pad during stage I or the change in vertical contact stress exerted directly underneath the bridge foundation during all subsequent stages. The vertical stresses 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 5. 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 6. Estimated changes in vertical xtresses experienced during each monitored Stage.

Stage	Vertical Stress (kPa)	
	Loc. B	Loc. D
Stage I Construction, Sections 400, 800	117	117
Stage I Construction, Section 200	86	86
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

9.0 INSTRUMENTS AND APPLICATIONS

A number of reliable instruments and techniques were employed in this study to monitor the performance of the Founders/Meadows Bridge at different stages along the monitored sections (sections 200, 400, and 800). See Appendix B for a detailed description of all the instruments and techniques employed in this study, their installation and applications. Surveying and inclinometer readings were used to measure the external movements of the structure at different stages. A number of highly reliable gages were used to measure the developed soil stresses, temperatures, and moisture level, and geogrid strains at different stages. Geokon Model 4800 and Model 4810 pressure cells were used to measure the vertical and horizontal earth pressures. Geokon Model 4420 crackmeters and Geokon Model 4050 strain gages were used to measure geogrid strains. CS615 Water Content Reflectometers manufactured by Campbell Scientific, Inc. were used to measure moisture content within the reinforced soil mass. All Geokon gages (except inclinometer) were read manually using a Geokon GK403 Readout Box during phase I, and automatically using a Campbell Scientific, Inc. CR10 data logger during phase II.

Due to construction constraints and activities beyond control, several gages and survey points were lost, survey points at the bottom of MSE walls were covered with concrete barriers before the bridge was open to traffic, control survey points were disturbed and therefore relocated, and information could not be collected at the proper time.

10.0 PHASE I INSTRUMENTATION PROGRAM

10.1 Instrumentation Plan

A pilot instrumentation plan was conducted during construction of the phase I structure in order to obtain information to 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.

The facings of sections 200 and 400, located along the phase I structure (see Figure 2), were instrumented with survey targets (Figures 8 and 9). Section 400 was additionally instrumented with two pressure cells, two crackmeters, and one inclinometer (Figure 9). Pressure cell P1, 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, used to measure horizontal earth pressures, was placed directly behind the facing 2.33 meters above the leveling pad. Two crackmeters, S1 and S2, were attached to the 6^{th} geogrid layer, 2.23 m above the leveling pad, 1.33 m back from the facing, spaced 1.7 m apart.

10.2 Pressure Cell and Crackmeters Results

Measurements from pressure cells and crackmeters (Figure 9) were collected manually during all construction stages. The obtained results are summarized in Figures 10a, 10b, and 10c. Estimated vertical and horizontal stresses at the level of pressure cells and crackmeters below the foundation were calculated using Equations (1) and (2).

Figure 10a shows a good correlation between measured and estimated vertical stress values. Figure 10b 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).

The measured geogrid strains versus the estimated soil vertical stresses at various construction stages are shown in Figure 10c. The Figure indicates that the measured geogrid strains from two crackmeters correlate well with each other. The geogrids experienced very low strains, on the order of 0.1 %. As shown in Figure 10c, 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.

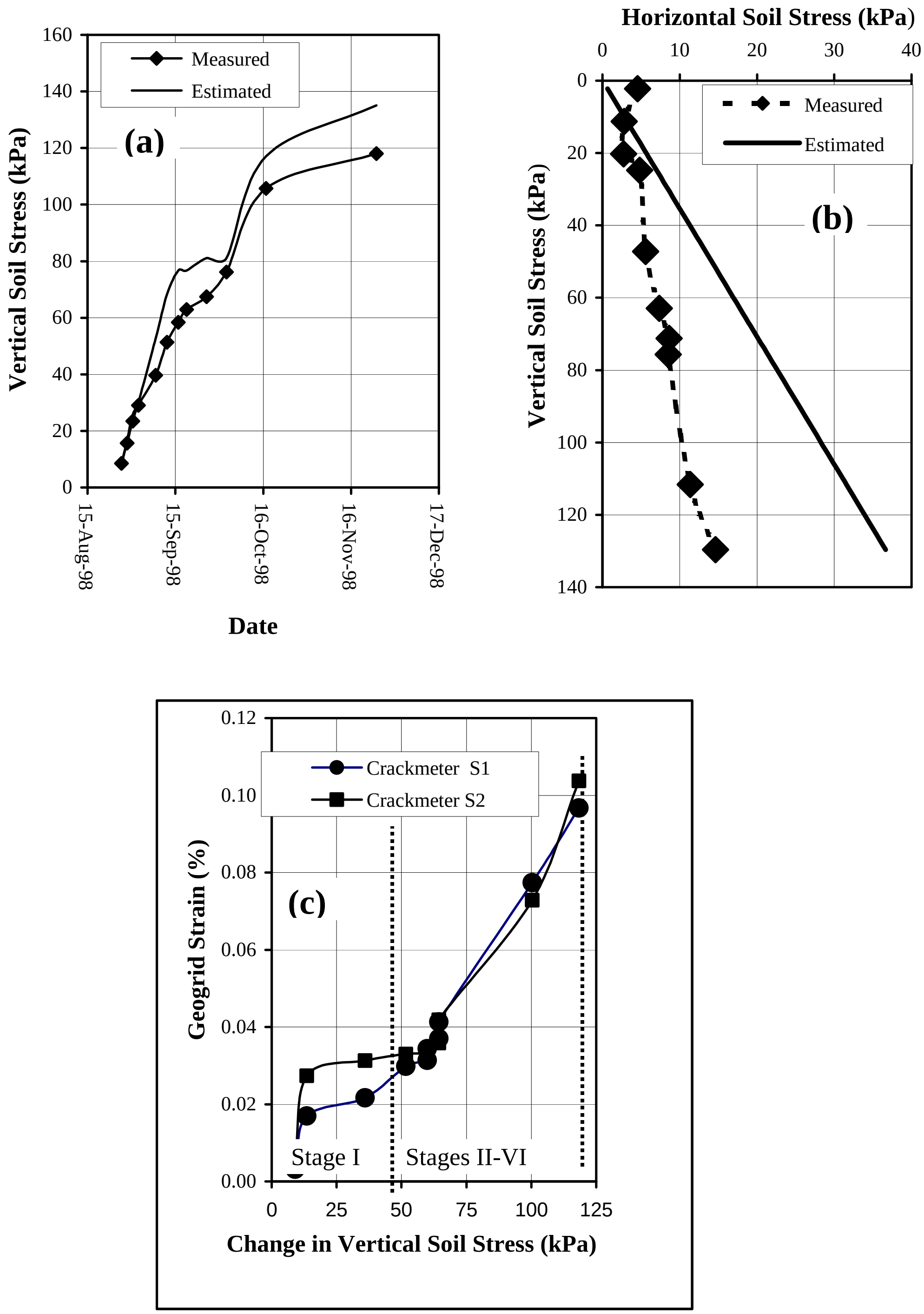

Figure 10. Measured and estimated data on section 400 of the phase I structure: (a) Vertical Stresses, (b) Horizontal Stresses, and (c) Geogrid Strains.

10.3 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.

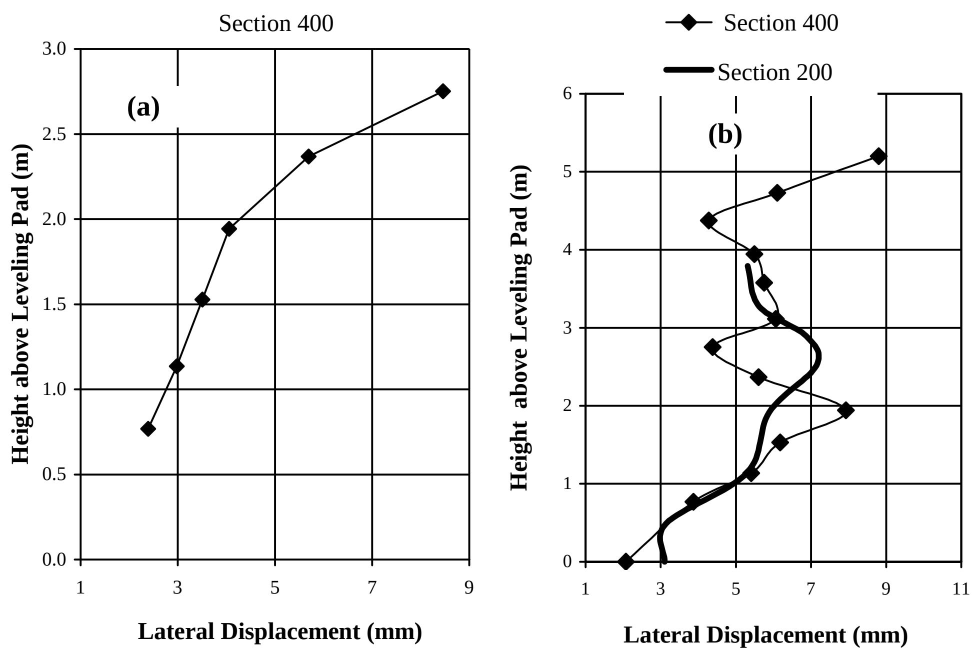

Figure 11. Lateral displacements monitored on phase I structure: (a) during stage I construction, and (b) due to placement of the bridge superstructure.

Figure 11a 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). Figure 11b 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:

- The maximum outward movement experienced along section 400 during stage I construction was approximately 9 mm (Figure 11a).
- As shown in Figure 11b, 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).

11.0 PHASE II INSTRUMENTATION PROGRAM

11.1 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 crackmeters 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.

Along section 800 (see Figure 22), survey targets were placed on the facing blocks, abutment walls, bridge deck, 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 (1st objective of the study). Four critical locations along section 800 below the foundation were heavily instrumented with pressure cells, strain gages, and moisture gages:

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.

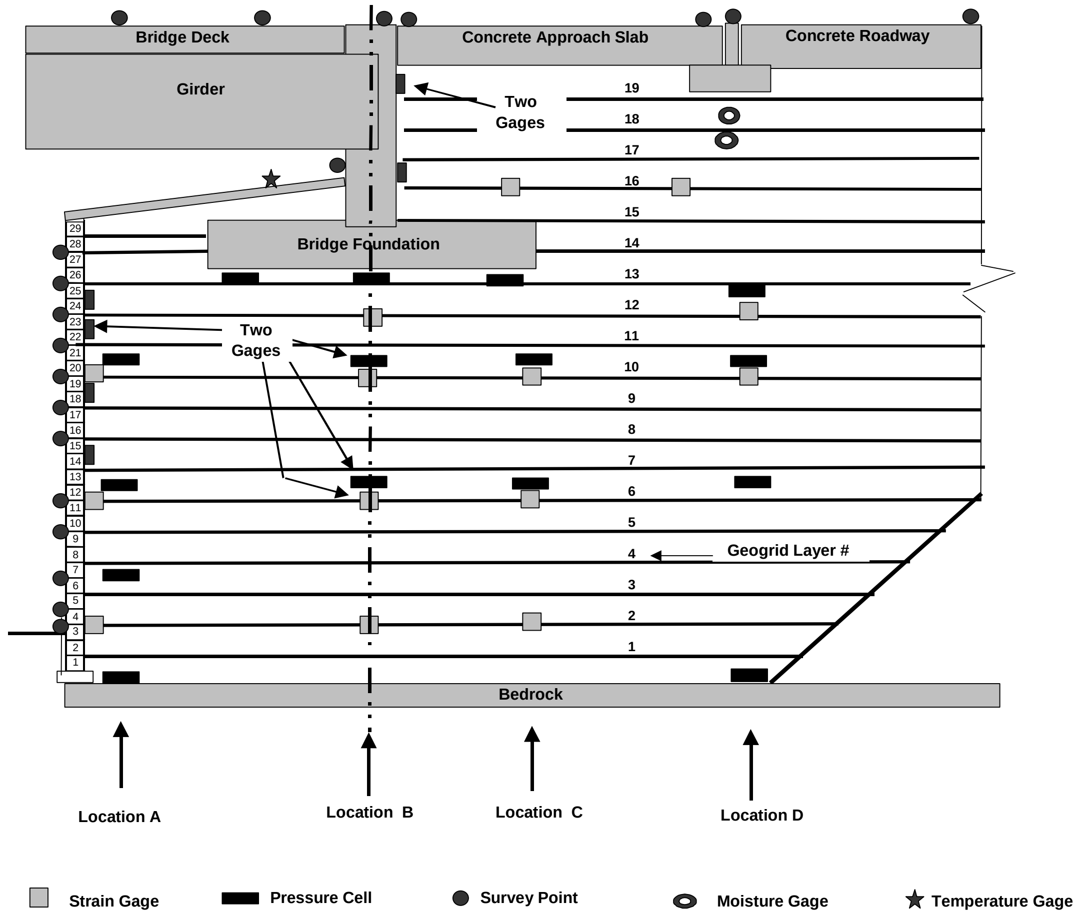

Figure 12. Instrumentation layout at section 800 (phase II structure).

As also shown in Figure 12, the reinforced backfill behind the abutment of section 800 was also instrumented with strain gauges, pressure cells and moisture gauges. Two moisture gages were employed to measure changes in soil moisture below the approach slab. Two pressure cells were installed against the back of the abutment wall and one was installed against the Styrofoam to measure developed earth pressure on the abutment wall at different stages and times. Instrumentation results to be collected at the different locations within the reinforced soil mass (Figure 12) 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. This will allow assessment of AASHTO and CDOT design procedures regarding the use of reinforced soil structures to support bridge abutments and alleviate the bridge bump problem (2nd objective of the study).

The reliability of the measured data and the suitability of assuming plain strain conditions were 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. Figure 12 shows locations where two gages were placed at the same elevation and distance from the facing, both north and south of the control section.

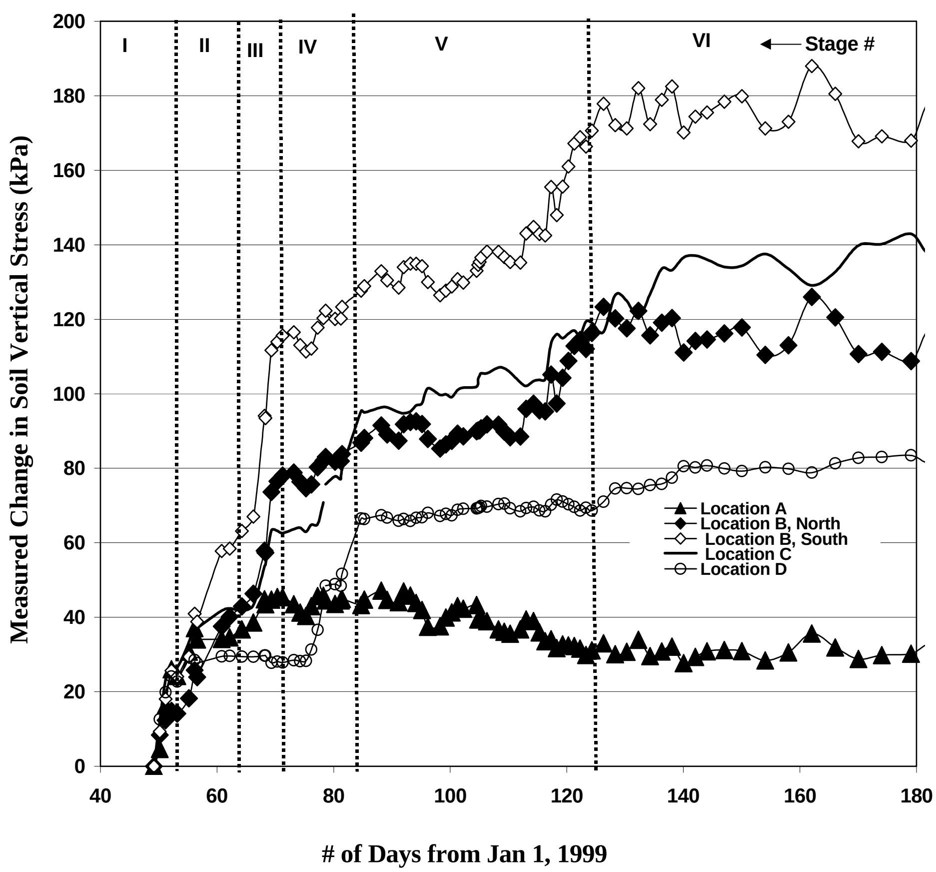

Figure 13. Measured vertical soil stresses during all construction stages (measured between geogrid layers 10 and 11 along section 800).

11.2 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 report. Examples of the monitoring records collected during the construction stages (until June 30, 1999, or 180 days from January 1, 1999) are presented. Figure 13 shows the vertical stresses measured by the pressure cells placed between the 10^{th} and 11^{th} geogrid layers (see Figure 12). Figure 14 shows the geogrid strains measured by the strain gages placed along the 6^{th} geogrid layer (see Figure 12). Evaluation of this preliminary information provides significant insight regarding the trends in the behavior of the structure. The following observations can be made:

- 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 13 and 14 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 Figure 13) 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.
- Figure 13 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 Figure 14, 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 Figure 14, 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 %.

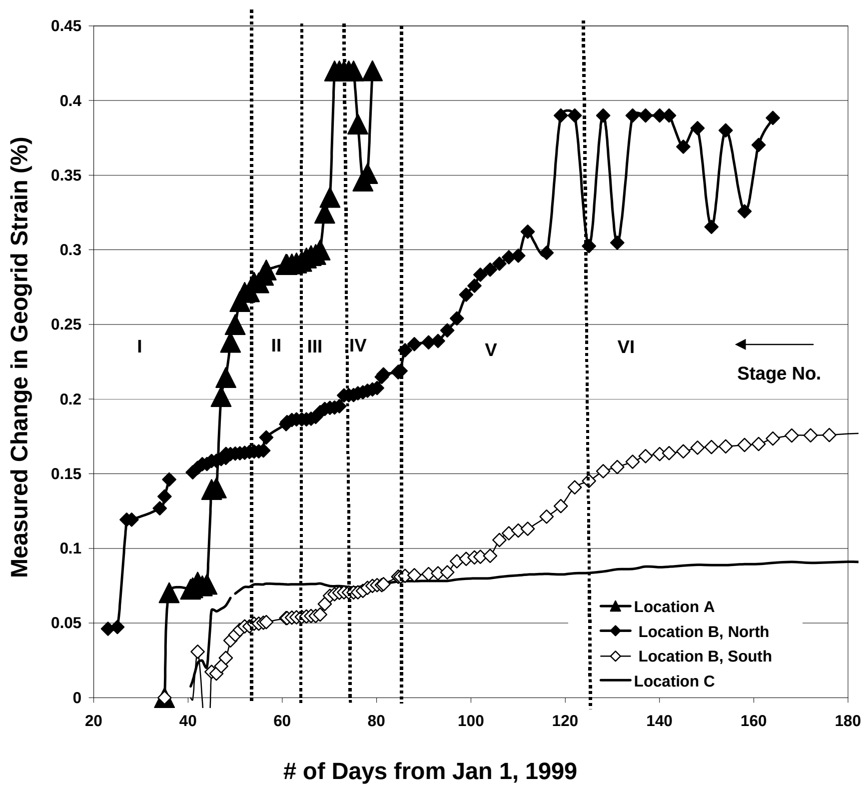

Figure 14. Measured geogrid strains during all construction stages (measured on the 6th geogrid layer of section 800).

- The maximum geogrid strain occurred close to the facing (location A) along the 6th geogrid layer (see Figure 14), 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.

12.0 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 a 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 provided comfortable rides with no signs of the development of the common bridge bump problem.

The Founders/Meadows Bridge was considered experimental, and 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 report. 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 bridge opening to traffic has been satisfactory showing very small movements.

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 redistributes 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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APPENDIX A

Tensar Earth Technologies, Inc. TENSWAL

5775-B Glenridge Drive TENSAR GEOGRID REINFORCED
Lakeside Center, Suite 450 RETAINING WALL ANALYSIS
Atlanta, Georgia 30328

Version 3.4
DESIGNER: JSB Revision Date 9/12/96

Project: SH 25 Project #: D98201 Date: 04/15/98
Case: LOWER N&S & FRONT - ABUT. #3

** Program Analysis for TENSAR Geogrid Reinforcement Only

INPUT

REINFORCED WALL FILL DATA

Height (m)	=	5.99
Angle of Face (deg, max = 15)	=	0
Density of fill (kN/m3)	=	19.6
Phi in degrees	=	34
* Cohesion c (kN/m2)	=	0

RETAINED BACKFILL DATA

Density of fill (kN/m3)	=	19.6
Phi in degrees	=	27
* Cohesion c (kN/m2)	=	0

FOUNDATION SOIL DATA

Phi in degrees	=	27
Cohesion c (kN/m2)	=	0
Allow. bearing press. (kN/m2)	=	300

LOADING DATA

Height of backfill slope (m)	=	0.47
Slope angle in degrees	=	9.4
Surcharge on top of slope (kN/m2)	=	153.2

TENSAR GEOGRID DATA

Geogrid designation	=	UX6
'-. Coverage of TENSAR geogrids	=	100
Minimum Geogrid length (m)	=	4.2
Wall soil interaction coeff.	=	0.8
F.S. for geogrid pullout	=	1.5
Fdn. soil interaction coeff.	=	0.7

MISCELLANEOUS DATA

F.S. for sliding	=	1.5
F.S. for overturning	=	2
Design Methodolgy	=	AASHTO TF27 Guidelines
Construction Damage based on	=	Sand, Silt, or Clay

Tensar Earth Technologies, Inc.
5775-B Glenridge Drive
Lakeside Center, Suite 450
Atlanta, Georgia 30328

TENSWAL
TENSAR GEOGRID REINFORCED
RETAINING WALL ANALYSIS

Version 3.4
Revision Date 9/12/96

DESIGNER: JSB

Project: SH 25 Project #: D98201 Date: 04/15/98
Case: LOWER N&S & FRONT - ABUT. #3
** Program Analysis for TENSAR Geogrid Reinforcement Only
OUTPUT

F of S against sliding = 1.51

F of S against overturning = 4 . 4 7

Maximum Bearing Pressure (kN/m2) 240

Angle of Inclination, Delta (deg) = 0.0

Eccentricity of Pres. Resultant (m) = 0.33

Total Number of TENSAR Geogrid Layers 14

Elevation of TENSAR Geogrid (m)	Allow. vi (m)	Length of TENSAR Geogrid (m)	Working Strength (kN/m)	Percent Cov	Max Force of TENSAR Geogrid (Kn/m)	F.S.	Control Mechanism
5.60	0.87	7.8+5.6	UX6: 47.6	100	18	2.62	Tension
5.20	0.81	7.8+5.2	UX6: 47.6	100	13	3.61	Tension
4.80	0.76	7.8+4.8	UX6: 47.6	100	18	2.71	Tension
4.20	0.69	7.8+4.2	UX6: 47.6	100	19	2.47	Tension
3.80	0.66	7.8+3.8	UX6: 47.6	100	16	2.92	Tension
3.40	0.62	7.8+3.4	UX6: 47.6	100	17	2.77	Tension
3.00	0.59	7.8+3	UX6: 47.6	100	18	2.64	Tension
2.60	0.56	7.8+2.6	UX6: 47.6	100	19	2.51	Tension
2.20	0.54	7.8+2.2	UX6: 47.6	100	20	2.40	Tension
1.80	0.52	7.8+1.8	UX6: 47.6	100	21	2.30	Tension
1.40	0.20	7.8+1.4	UX6: 47.6	100	22	2.20	Tension
1.00	0.47	7.8+1	UX6: 47.6	100	23	2.10	Tension
0.60	0.44	7.8+0.6	UX6: 47.6	100	24	1.95	Tension
0.20	0.41	7.8+0.2	UX6: 47.6	100	26	1.82	Tension

Efficiency of Strength Used vs. Available = 73.21 %
Area of geogrids required (m2/m) = 102.31

In accordance with the "Tenswal" licensing Agreement,
the designer must determine the suitability of program results.

APPENDIX B

Figure B-1. A view of the northwest side of the completed Founders/Meadows bridge-approaching roadway structure showing lower and upper MSE walls, wing walls, and girders.

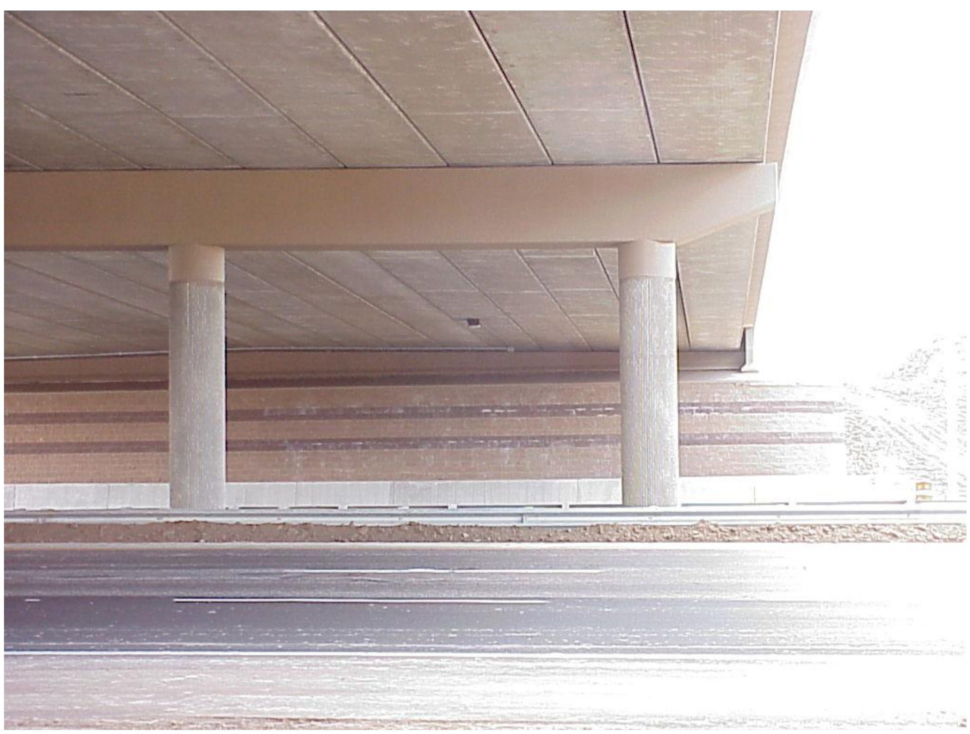

Figure B-2. A view of the east side of the Founders/Meadows Bridge showing front MSE wall, girders and pier columns.

Figure B-3. Southwest view of the completed Founders/Meadows bridge structure.

Figure B-4. Picture showing facing blocks, crushed stone used to fill in and 0.3 m behind the blocks, and the compacted CDOT Class 1 backfill. The yellow inclinometer tube was used to measure lateral movement in the structure along Section 400.

Figure B-5. Backfill material meeting requirements for CDOT Class 1 backfill.

Figure B-6. The 19 mm maximum size crushed stone aggregate used to fill in and for .3 m behind the blocks to facilitate compaction and drainage behind the blocks, prevent washout of the fines in the fill material, and facilitate compaction behind the facing.

Figure B-7. Rolls of Tensar UX 6 geogrid placed below the bridge foundation.

Figure B-8. Top view of the Mesa Block - the outside face of the block is toward the top of the picture.

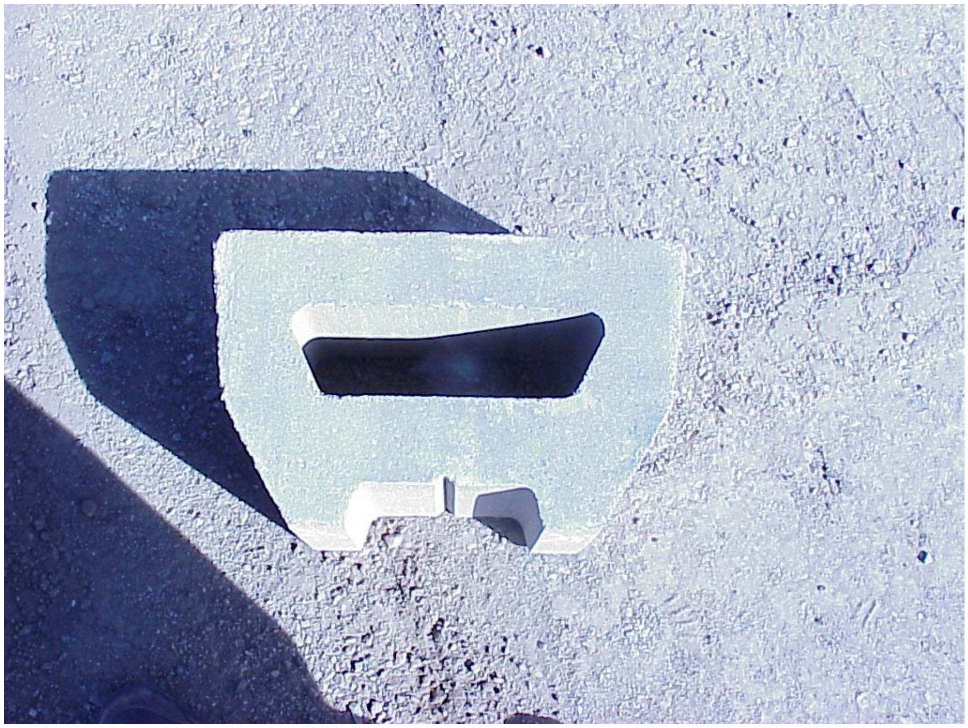

Figure B-9. Bottom view of Mesa block showing smooth flat surface with no grooves or offsets.

Figure B-10. The Standard Mesa Connectors which provide the positive mechanical connection strength between blocks and between blocks and geogrid. This connector has four "teeth" that fit down into the groove of the block (Fig 8) and two "flags"where the geogrid is hooked. The rear of the cavity of the next row of blocks rests against the two large flags. This provides connection strength between blocks.

Figure B-11. The facing blocks are fastened to the geogrid by the use of Mesa Standard Connectors which fit into the grooves of the blocks.

Figure B-12. The rear of the block (to the right of the picture under the geogrid) is lower than than the front to allow for the thickness of the grid.

Figure B-13. Expansion joint material was placed between the leveling bed and the first row of blocks.

Figure B-14. Water was added before the fill was compacted.

Figure B-15. Backfill levels of compaction were checked with a nuclear gage.

Figure B-16. Around the corners of the lower MSE walls, where geogrid from the front and side of the structure overlapped, a thin layer of fill material was placed between these geogrid layers to provide friction between them.

Figure B-17. The first layer of fill material at the east side of the Phase I Structure has been placed. In the background, H-beams and shoring support the approach to the old bridge during the construction of the Phase I Structure.

Figure B-18. The northern side of Phase I structure was internally stabilized with steel wire mesh. The steel extended past the edge of the wall and was tied to the geogrid placed to stabilize the front MSE wall.

Figure B-19. Temporary wire mesh facing was used to support the northern face of the Phase I structure until the Phase II structure was built and tied to the Phase I structure. The backfill in between the wire mesh reinforcement was contained using wrapped fabric that also facilitated backfill drainage.To the right is a closer look at the shoring supporting the old bridge structure.

Figure B-20. The geogrid was connected at the wall end, rolled out, and then stretched by hand. The back end was staked to prevent geogrid movement while the fill was placed using a small loader. In the background is the steel reinforcement used to temporarily support the northern face of the Phase I structure.

Figure B-21. A vibratory roller was used to compact the fill material.

Figure B-22. Near the block facing, a small walk-behind compactor was used to prevent pushing the blocks out of position.

Figure B-23. A vertical drain pipe was installed inside the MSE Walls.

Figure B-24. The geogrid was cut to fit over a drain pipe that went down through the middle of the structure. The pieces of geogrid were offset from one layer to the next so the longitudinal joints did not line up vertically.

Figure B-25. The pipe used with the drainage blanket placed behind the reinforced backfill.

Figure B-26. The abutment wall footing has been poured and the forms are being set for the abutment wall.

Figure B-27. The top two rows of blocks were filled with concrete. Steel rebars inserted in the plastic concrete were later bent over to tie the concrete cap at the top of the front MSE wall.

Figure B-28. Expansion joint material was installed below the sides of the abutment wall on top of the footing. In the center zone below the abutment wall on top of the footing, galvanized sheet metal centered on a bearing pad was placed (see figure 29).

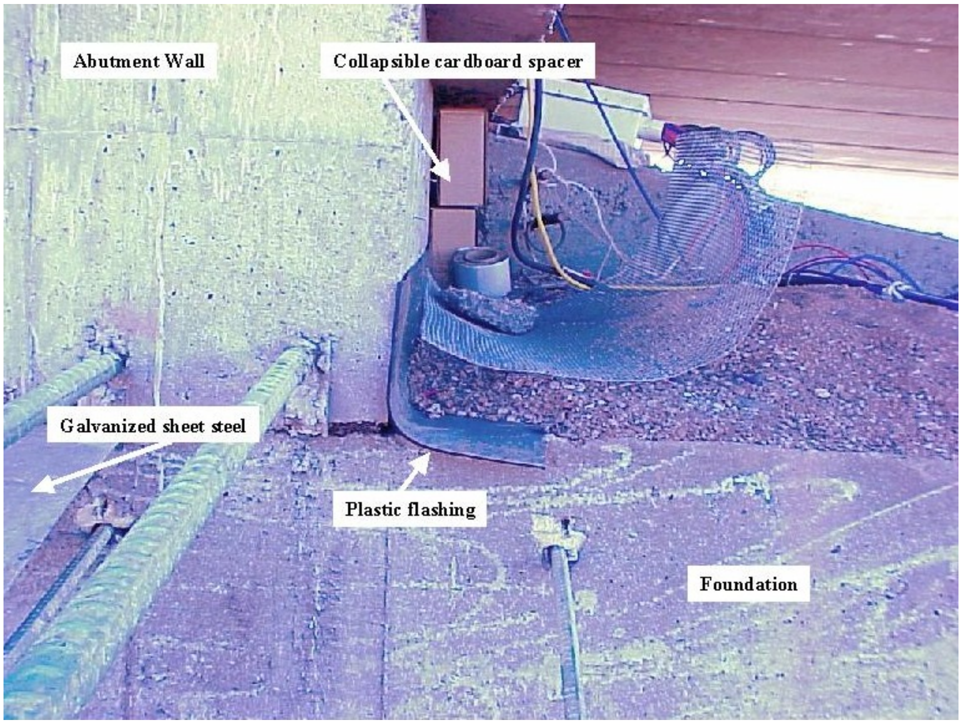

Figure B-29. This picture shows the galvanized sheet steel between the abutment wall and the foundation, and the plastic flashing and collapsible cardboard spacer in front of the bottom of the abutment wall. Following this stage is placement of the concrete slope paving cap between the top of the front MSE wall and the base of the abutment wall.

Figure B-30. These elastomeric bearing pads were installed on the abutment wall where the bridge girders are seated.

Figure B-31. The elastomeric bearing pad and expansion joints material placed on the abutment wall where girders will be seated.

Figure B-32. A reinforced concrete "kicker" strongly attached to the bridge foundation was placed behind the bottom of the abutment wall. This kicker was placed to provide an extra bracing at the top of MSE wall for global stability.

Figure B-33. Abutment wall and some girders. The stage that follows is pouring the concrete for the bridge deck and the rest of the abutment wall and girders. The overlapping steel from girders, abutment wall, and bridge deck will strongly attach the bridge superstructure to the abutment wall without any joint (integral abutment).

Figure B-34. A plastic sheet was placed between bridge abutment and bridge foundation to facilitate water drainage.

Figure B-35. After the girders were placed, 75 mm sheets of low-density expanded polystyrene were placed against the back of the abutment wall. The area in the foreground between the lower and upper MSE walls will be covered with a membrane and paved with concrete (referred to as slope paving in Fig. 40).

Figure B-36. The wrapped geogrid behind the abutment wall.

Figure B-37. Leveling pad for the upper MSE wall.

Figure B-38. The edge of the wing wall and upper MSE wall. Expansion joint material was placed between blocks and leveling pad and between blocks and the wing walls.

Figure B-39. This panel was used to give texture to the outside of surface of the concrete wing wall.

Figure B-40. Collapsible cardboard was placed in front of the bottom portion of the abutment wall behind slope paving (see figures 29 and 35).

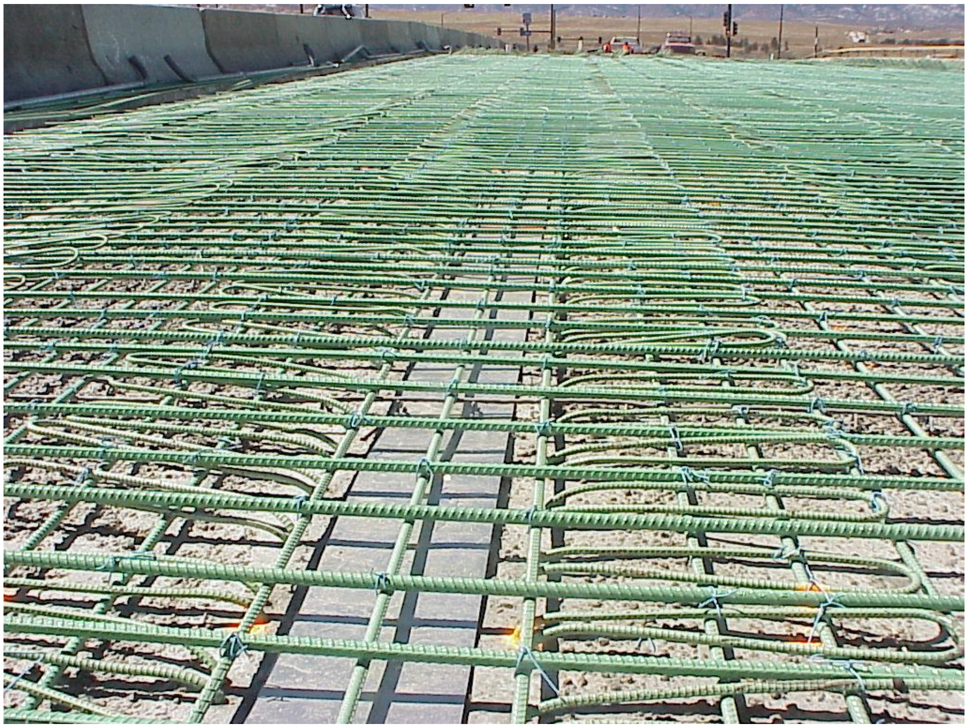

Figure B-41. Reinforcement Steel for the Bridge Deck

Figure B-42. The forms and reinforcing steel for the sleeper foundation supporting the approach slab.

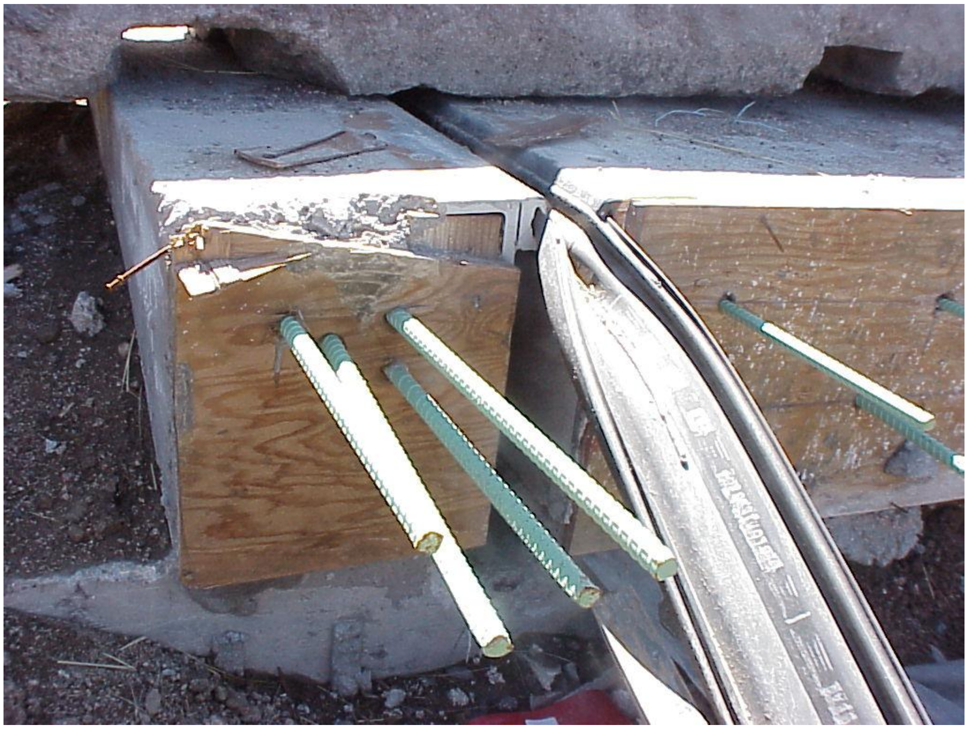

Figure B-43. The approach expansion device placed between the sleeper foundation and approach slab.

APPENDIX C

C.1 Overview

In this study a number of reliable instruments and techniques were employed to monitor the performance of the Founders/Meadows structure at different stages. Figure C-1 shows a plan view of the completed Founders/Meadows structure and the location of the monitored sections. Three Sections were instrumented, Section 200 (Figure C-2) and Section 400 (Figure C-3) along Phase I Structure, and Section 800 (Figure C-4) along Phase II structure. Figure C-5 is a photograph of the instrumented front wall of section 800.

The instruments and techniques employed to monitor the performance of the structure during and after construction are described in the following. For a comprehensive description of the operation of the gages and instruments, the reader is encouraged to refer to the manufacturer's literature.

C.2 Structure External Movements

The instruments and techniques employed to monitor the external movements of the MSE walls, and approaching bridge and roadway structure are described in the following. Due to construction constraints and activities beyond control, several survey points were lost, survey points at the bottom of MSE walls were covered with concrete barriers before the bridge opening to traffic, control survey points were disturbed and therefore relocated, an inclinometer placed along Section 800 of Phase II structure was severely distorted under compaction and therefore eliminated from this study, and some information could not be collected at the proper time.

C.2.1 Surveying

See Figures C-2, C-3, and C-4 for locations of all survey points. Survey points made of reflective sign sheeting were permanently glued to the outside face of walls, girders, and abutment walls. Since the leveling pad would be covered with compacted fill, a long steel post with a reflector was inserted into the plastic concrete of the leveling pad (Figure C-6). Survey points marked by nails were glued into recessed holes in the bridge decks, approaching slab, and approaching roadway. Several permanent control points were established around the structure. A surveying instrument (Sokkisha Set 3 SN) was used to collect data for the northing, easting, and elevation coordinates of surveying targets at different stages. These data were analyzed to determine the structure's external movements induced between different stages.

C.2.2 Dipstick Road Profiler

The Dipstick Road Profiler (Figure C-10) is a device manufactured by Face Construction Technologies, Inc., of Norfolk Virginia. It is a digital level with two pivoting feet that are 12 inches (.305 m) apart. Two digital readouts show the difference in elevation, up to one inch, between the feet in 1/1000ths of an inch (0.025 mm). This device was used to draw an accurate elevation profile of the bridge deck and approaching roadway. The road surface was profiled along a line near the outside edge of the westbound and eastbound #2 lanes as shown in Figure C-11. The profiling extended 15.5 m from the east and west ends of the bridge to cover both the roadway and bridge.

Each reading from the Dipstick gives the change in elevation over 1 foot (305 mm) of distance. The readout is signed to show whether the change is up or down hill. By adding the readings together a very accurate profile of the road surface can be graphed. Heavy texture may affect one reading, but over a long section they cancel out leaving an accurate profile.

C.2.3 Inclinometer

One vertical inclinometer tube (Figure C-7) was affixed to the back of the blocks of Section 400 (Phase I Structure). A Geokon Model 6000 inclinometer probe is used in conjunction with the inclinometer tube to measure lateral movement of the fill material, either parallel to or perpendicular to the wall. The inclinometer tube is a fiberglass tube with an inside diameter of 6.35 mm (2-1/2 in.). It has four grooves spaced at 90-degree intervals around it to guide the instrument (Figure C-8). The inclinometer (Figure C-9), which has a pair of wheels at each end to ride the two grooves on the opposing sides of the tube, is lowered into the tube until it reaches the bottom of the inclinometer. Then a reading is taken every 0.6 m (2 feet) as the inclinometer is raised in the tube. Each reading tells how far, at that particular elevation, the tube is out of plumb. The inclinometer measures its angle of tilt in the plane of the wheels and at 90-degrees to that plane so tilt of the tube in any direction is measured.

The bottom end of the first section of tube was set on top of the leveling bed at the bottom of the wall and held in place by the fill material and the back of the blocks (Figure C-7). Since the tube extended above the fill material in an area where the contractor was working, the tube was installed in short sections about 1.3 m (4 feet) long to make it as little of an obstruction as possible. Since the face of the wall was built with a negative batter and the tube was against the facing, the tube was not plumb. Figure C-8 shows the inclinometer tube splice. The inside piece is the inclinometer tube. The piece on the outside is a coupler, about 0.3 m (12 in.) long, that slides over the outside of the tube. Two pieces of tube are butt jointed and the coupler is slipped over the joint and pop-riveted to the two pieces of tube to hold them together. Even at the joints, the tube is very stiff. That stiffness prevented measurement of different amounts of movement in individual layers. Only the average overall movement of the tube could be monitored - it was impossible to tell if one layer moved 3 mm and the one above or below it did not move at all.

The inclinometer readings were recorded manually and stored using Geokon 603 Readout Box. Each time a new set of readings is taken, they are added to the file so changes in the tilt of the tube can be determined. These data were used to estimate changes in the wall outward and parallel wall movements relative to the leveling pad.

C.3 The Structure Internal Response

Highly reliable gages were employed measure the developed soil stresses and geogrid strains inside the reinforced soil mass at different stages. Geokon Vibrating Wire pressure cells and strain gauges were selected because of their rugged design, long-term stability in difficult environments, and high accuracy and resolution. These instruments are read by measuring frequency rather than resistance or voltage, therefore, gage accuracy is not influenced by variations in cable resistance caused by moisture, temperature, contact resistance or leakage to ground. A vibrating wire gage contains a spring that is attached to a diaphragm. Changes in pressure on the diaphragm increase or decrease the tension of the spring. This raises or lowers its resonant frequency similar to changing the pitch of a guitar string. (A tighter spring resonates at a higher frequency.) When a gage is read, a scanned series of frequencies is used to pluck the spring contained in the gage. Ideally, all but the resonant frequency will quickly die out and the spring will continue to vibrate at that frequency for a short period of time. By measuring the frequency of vibration of the spring, the pressure applied to the gage can be determined. A thermistor contained in each strain gage or pressure cell provides temperature information at the location of the sensor so corrections for temperature variations can be made.

C.3.1 Geokon Model 4800 Earth Pressure Cell (EPC)

These gages were used to measure the vertical pressure exerted by the fill material. Two gages (P1 & P2 in Figure C-3), with a range of 370 kPa, were used on the Phase I structure, and 18 gages with a range of 345 kPa were used on the Phase II structure (Figure C-4). Figure C-12 is a picture of a Model 4800 Earth Pressure Cell. The disk part of the gage is made up of two stainless steel plates welded together around their edges, leaving a small space between them. The space is filled with hydraulic oil that has had all of the air removed. A short tube connects the reservoir to the pressure transducer, which converts the oil pressure to an electrical signal to be transmitted through the signal cable to the readout location. This cell is designed to be placed in the soil and was positioned horizontally to measure vertical pressure.

C.3.2 Geokon Model 4810 Contact Pressure Cell (CPC)

These gages were used to measure the horizontal pressure exerted by the fill material on the facing. One gage was used on the Phase I structure (see Figure C-3), and 8 gages were used on the Phase II structure (See Figure C-4). The CPC differs from the EPC only in the fact that one of the steel plates is thicker (Figure C-13). The CPC is mounted with the thick plate against a solid surface and the thinner pressure sensitive plate toward the soil. CPC’s were placed vertically against the back of the block facing and the abutment wall to measure the horizontal pressure exerted by the soil fill.

Geokon's installation instructions called for a layer of grout between the structure and the gage to be sure there was even contact between the surface of the structure and the back of the sensor. There was some concern as to whether the temperature of the blocks would have an effect on the gage readings. For that reason, most of the gages installed on Phase II were fastened to a piece of plywood and the plywood set against the cement (Figure C-17). The plywood was used to provide some thermal insulation between the blocks and the gages. Two gages were installed between geogrid layers 11 and 12 (see Figure C-4). One of them was fastened to a piece of plywood, and the other was set directly against the grout layer.

Three pressure cells were installed on the back of the abutment wall. The bottom gage (Figures C-17 & C-18) was nailed to a piece of plywood. The plywood was then nailed to the abutment wall using a power nailer. The polystyrene foam was installed over the gage. The foam sheet was hollowed out so it would lie flat against the abutment wall over the gage.

Near the top of the wall two pressure cells were installed. One was fastened to a piece of plywood and the plywood fastened to the concrete (Figures C-19 & C-20). The foam sheet was removed in this area. The second upper cell was installed on the soil side of the polystyrene foam sheet (Figures C-21 & C-22).

C.3.3 Geokon Model 4420 Crackmeters

Two of these gages, shown in Figure C-23, were used on Section 400 (Figure C-3) in the Phase I structure to measure geogrid strains. The crackmeter gage was .305 meter (12 in.) long, and had a range of movement of 12.5 mm (1/2”). Vertical threaded posts on each end of the gage were inserted in holes in the web part of the geogrid and held with nuts. The right end of the gage in Figure C-23 is a threaded shaft that can be lengthened or shortened to preset the gage to span a specified distance. The gage operates by increasing or decreasing the stretch of an internal spring when the shaft moves in or out of the gage body. It can read either extension or compression. An initial zero reading is recorded at the time of installation. Changes in the gage output can be converted to displacement in inches by using a gage calibration factor supplied by the manufacturer. The leads from the gages run to the back of the front wall, then inside a 50 mm diameter plastic pipe up to the top of the wall where they are connected to the monitoring system.

A crackmeter , as its name implies, is designed to measure expansion or contraction of a crack in a solid surface such as concrete. In these applications, the gage is solidly mounted to a rigid surface. The geogrid was comparatively flexible and it is speculated that it could not hold the gage solidly enough. There was also some concerns for using crackmeter in Phase I because of their high strain range (4%) and comparatively smaller sensitivity. Therefore, the measured geogrid strains from these gages (presented later) were of questionable value.

C.3.4 Geokon Model 4050 Strain Gages

More accurate and sensitive strain gauges (Geokon Model 4050) than the crackmeters used on Phase I were employed in Phase II to measure geogrid strains. Total of 15 gages were installed along Section 800 (See Figure C-4 and C-24). These gages were similar to the crackmeters used in phase I except that they were mounted using brackets (Figure C-25) that clamp to the geogrid . This mounting system provided more solid and stable connection between the geogrid and the gages than the crackmeters used in Phase I. By using a 150 mm (12 inch) gage slightly modified by Geokon to mount in clamps spaced 150mm (6 in.) apart, the sensitivity of the gage was doubled while maintaining the same range of movement. The gage clamps had to be installed on the geogrid before it was stretched and covered because it was very difficult to get under the grid after the fill had been placed.

Alignment of the clamps was critical. Collars on the gages fit the mounting holes in the clamps with very little play. Any misalignment along the axis of the gage made installation very difficult. Also, spacing between the clamps needed to be 152 ± 3 mm (6” ± 1/8 in.) to allow for proper mounting and calibration of the gages. The CDOT technician who worked on the project made steel jig rods (Figure C-25) to use to position the clamps on the geogrid. Before the geogrid was installed, the rods were tightened in the holes in the clamps and the clamps fastened to the geogrid. The grid was then stretched and the fill material placed and compacted. After compaction, the fill material was dug away from around the clamps, the rods removed, and the gages installed. The rods kept the clamps in proper alignment during the placing of the geogrid and the fill material. Wood blocks were wired to the geogrid (Figure C-26) over the clamps to keep dirt out of the screws and to make it easier to dig up the clamps.

After the gages were inserted into the clamps they were connected to the GK-403 readout box and calibrated as follows: The left end of the gage shown in Figure C-24 was tightened in one clamp. Then the body of the gage was pulled to the right to stretch the sensor spring and preset the gage output to the desired range on the GK-403 readout box. The right clamp was then tightened without allowing the gage to move. The reading was checked to be sure it was still in range and recorded for use as a zero reference.

After calibration, the gages were covered to a depth of 50-75 mm (2-3 in.) with fill material that had been screened to remove the large rocks (Figure C-28) and then compacted gently by hand. The screened fill around the gage helped protect it while the next lift of fill material was being spread using bobcats and a small front loader.

C.3.5 Resistive Temperature Probes

To record air temperature, a Geokon temperature probe was placed on top of the Phase I Front MSE wall below the girders, where the sun and precipitation could not reach it. The thermistors associated with each of the vibrating wire sensors were used to measure backfill temperatures at the locations where these sensors were placed.

C.3.6 CS615 Water Content Reflectometers

These gages were used to measure moisture content changes within the reinforced soil mass. Two gages were installed below the approaching slab of the Phase II structure (see Figure C-4). There were no suitable vibrating wire moisture sensors available at the time of construction, so CS615 Water Content Reflectometers from Campbell Scientific, Inc. were used (Figure C-29). The CS615 measures the water content of porous media by monitoring the effect of changing dielectric constant on electromagnetic waves propagating along a wave-guide. The two metal legs in Figure C-29 form the wave guide. Because they does not use the scanned frequency excitation used by the vibrating wire gages, the CS615's were not connected through a multiplexor. They required direct connections to the data-logger, and a special program module in the software.

C.4 Data Collection System

The data output of Geokon gages can be read manually by a Vibrating Wire Readout Box, such as the Geokon GK403, or automatically by a data logging system like the Campbell Scientific, Inc. CR10 data logger. Both of these methods were used on this project. Because only 4 gages plus a vertical inclinometer were installed in the first phase of construction, installation of a data logger for automatic continuous monitoring was delayed until the second phase was under construction.

For phase II, continuous data collection was necessary so changes in the gage readings could be correlated to the construction operations. The data collection system consisted of a CR10 measurement and control unit (MCU), a data storage module, a vibrating wire interface, four multiplexors, a deep cycle 12-volt battery, and software to control the CR-10. The data collection equipment, except for the 12v battery, all fits in a weather proof, lockable box (Figure C-30). The box was bolted to the concrete wing wall on the south side of the phase I part of the bridge.

C.4.1 Measurement and Control Unit (MCU)

A Geokon Micro-10 data-logger built around a Campbell Scientific CR-10 MCU was used to monitor all the gages installed in phase II and to store the data. The MCU functions as a microcomputer, clock, multi-meter, calibrator, scanner, frequency counter, and controller. The CR10 is capable of reading practically any gage from any manufacturer.

The operating instructions for the MCU are set up on a laptop computer using software called MultiLogger© which operates under MS Windows. It allows the operator to build an instruction set telling the MCU how to control multiplexors, provide excitation signals to gages, read gage outputs at a set interval, and store data in memory or a separate storage module. The program can direct the MCU to store gage data raw for later conversion, or do mathematical operations on the data such as temperature corrections and/or conversions to units appropriate to the type of gage. After the instruction set has been developed on the PC it is downloaded through an SC232 interface to the MCU. MultiLogger© also allows the operator to monitor the most recent gage readings on a computer connected directly to the CR-10 system.

The system was programmed to take readings at timed intervals. During construction, the gages were scanned at 1-minute intervals; the interval was later increased. After construction was completed the scan rate was set to 6 hours and later to 12 hours.

C.4.2 Data Storage Module

A Campbell Scientific SM-716 data storage module was used with the system. The SM-716 stores data in battery backed RAM. It provides 716K bytes of storage, enough for several weeks to months of operation depending on how frequently the gages are read.

One of the advantages of using a 716 was that the module could be easily unplugged and replaced. This allowed the module to be connected to a desktop computer and downloaded in the office rather than requiring downloading at the site. At the final scan rate of once every 12 hours, the module will store several months of data, although it is collected every 2-3 weeks.

C.4.3 Vibrating Wire Interface

The vibrating wire gages were routed through a Campbell Scientific AVW4 Vibrating Wire Interface. The AVW4 provides signal conditioning that: 1. Increases the MCU swept frequency amplitude from 2.5 volts to 12 volts, 2. Completes the thermistor bridge for temperature measurement, 3. Provides transformer isolation and consequent noise reduction for the vibrating wire signal, and 4. Gives additional transient voltage (lightening strike) protection for both temperature and vibrating wire circuits.

C.4.4 Multiplexors

The number of gages used in the structure was several times the number of channels available on the AVW4 Vibrating Wire Interface. To make the system function, four Campbell Scientific AM416 multiplexors were used to connect the gages to the AVW4 and the MCU. Through the multiplexors, the MCU and AVW4 were connected in turn to each vibrating wire gage to excite and read it and its temperature probe.

Each multiplexor had sixteen 4-conductor channels. A separate gage and its thermistor were connected to each channel. The MCU provided a switched signal to turn each multiplexor on, step through its 16 channels reading the gages and thermistors, then turn it off and turn on the next multiplexor, etc.

The Campbell Scientific temperature probe was not associated with a strain of pressure gage. It was connected to the thermistor terminals for its respective channels on the multiplexor leaving the gage terminals for those channels open.

The moisture probes, which did not use vibrating wire technology, required different excitation voltages from the vibrating wire gages. They were connected directly to separate channels on the MCU and read separately through the software.

C.4.5 Battery

A 12-volt deep cycle battery was installed to power the MCU, AVW4, SM716, and AM416's. Power for the entire system is distributed through the MCU. Every time the gages were scanned the battery voltage was checked and recorded by the software. Current drain was very low: the battery provided power for the system from early March through August and still read 12.25 volts.

C.5 Instrumentation Recommendations

- Make the strain gage clamps (Phase II) with the bolts on top. Install the clamps on the grid with a spacing rod in place. Then, before removing the jig rod, loosen and re-tighten the bolts to be sure the clamps are properly aligned. With the bolts were inserted from the top of the clamp, it would not be necessary to install the clamps on the grid before placing it in the structure.

- To use the crackmeters used in Phase I on geogrid, use just the top and bottom plates of the clamps used in Phase II. Drill the top plate to fit the mounting post at the ends of the crackmeter. Position the plates to match the end posts on the gage, tighten the clamping bolts on the geogrid and then install the crackmeter.

- Plan, as closely as possible, the number of gages of each type to be used in the project. When they are connected to the multiplexors, connect all gages of one type to adjacent channels. This will arrange the data files with similar gages together and make it easier to monitor changes that occur during construction and to compare changes that occur in different gages. On the multiplexors one or two blank channels could be left between groups to allow the addition of extra gages if necessary.

- Keep the wiring neat and orderly. This is especially important in a situation where more instruments are being added over a relatively long time as they were on this project. Where the gage leads need to be spliced to extend their length, as they were on this project, use a set of terminal strips in the junction box to make the splices. This will facilitate a neat junction box and make it easy to locate possible problems in the splices.

- A few specialized tools were found to be helpful:
  - A screen and a bucket for sifting fines to use to cover the gages.
  - A dummy pressure cell plate to use to compact a level base for the cell to rest on.
  - A small “torpedo” level to facilitate leveling pressure cells.
  - A small tamper to compact the fine fill material around the gages after installation.
  - Jig rods for proper spacing of strain gage clamps.