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Selecting Backfill Materials for MSE Retaining Walls

FINAL REPORT

Prepared for
NCHRP
Transportation Research Board
of
The National Academies

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES
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W. Allen Marr, Richard P. Stulgis
GEOTESTING EXPRESS
Acton, Massachusetts
October 2013

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Author Acknowledgements

The authors gratefully acknowledge assistance and cooperation from various state DOTs, particularly with respect to their response to the survey questionnaire of current practice that comprised such an important part of this project.

The site for construction of the full-scale test walls was provided by the P.J. Keating Company, Lunenburg, MA. TenCate Geosynthetics donated the polyester geogrid reinforcement and TENAX donated the geocomposite drainage materials used in construction of the test walls. Their participation and support are acknowledged greatly appreciated.

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Abstract

This report documents and presents the results of a study to develop material selection guidelines, soil parameters, testing methods, and construction specifications that will allow the use of more fine-grained backfill materials within the reinforced zone of mechanically stabilized earth (MSE) retaining walls. A literature search and solicitation of information from state transportation agencies and private industry was conducted to determine current design and construction practices for MSE wall reinforced fill and the use of high fines and/or high plasticity. Based on that information, four full-scale test walls were constructed to rigorously evaluate issues associated with the use of "high fines" reinforced fill. The 20 ft. high by 60 ft. long wall sections were designed so that they would demonstrate acceptable performance for the normal design conditions, but show distress (deformations) when subjected to extreme conditions of high water pressures in the reinforced fill, a surcharge load placed on top of the wall, and freezing conditions in winter. The walls were fully instrumented and monitored 24/7 over a two-year period using a remote data acquisition system. This work demonstrates that the current AASHTO Limit on maximum fines content of the reinforced fill for MSE structures can be increased from 15% to 25%. Recommendations to revise current AASHTO specifications to include use of backfill materials with "higher" fines are provided.

Summary

The focus of National Cooperative Highway Research Project 24-22 (NCHRP 24-22) is to develop material selection guidelines, soil parameters, testing methods, and construction specifications that will allow the use of more fine-grained backfill materials within the reinforced zone of mechanically stabilized earth (MSE) retaining walls. The objective of the research is to provide results that allow the use of more fine-grained soils as backfills for MSE structures, thereby lowering their construction cost. The estimated potential savings from replacing AASHTO A-1-a reinforced fill materials with "higher fines" reinforced fill materials could be in the range of 20 to 30% of current MSE wall costs, especially in areas where relatively clean granular soils are not readily available.

The research consisted of three phases: synthesis of current practice; design, construction and monitoring of full-scale field test walls; and preparation of a final report for the work. GeoTesting Express, Inc. of Boxborough, MA was awarded the contract for this work, which began in August 2003. The report provides a summary of the research findings, guidelines for selecting MSE backfills, and appropriate testing methods and construction specifications for "high fines" backfill materials. Appendix A presents a synthesis of current practice of MSE retaining wall backfill design. Appendix B documents the full-scale test wall program that involved the construction and stress testing of four (4) walls, and the measured performance and our evaluation of that performance. Appendix C includes a guide technical specification for reinforced fill with "high fines" (< 25% of 0.074 mm particle size).

This work demonstrates that the current AASHTO Limit on maximum fines content of the reinforced fill for MSE structures can be increased from 15% to 25% subject to the following considerations:

- Soils with not more than 25% fines and a Plasticity Index not greater than 6% are used.
- All potential sources of water that might cause increases in pore pressure in the reinforcing zone should be addressed by the design.
- Good practices are used to measure the engineering properties of the reinforced fill for design, project specifications include requirements for reinforced fill selection and placement, and good practice construction QC/QA is employed.

The performance of MSE walls with higher fines soils is more dependent on as-placed density and moisture content than free draining materials. Using soil with higher fines requires a greater level of engineering for design and monitoring. Also, materials testing is required to determine shear strength and requirements for as-placed conditions (i.e. default values should not be used). Provisions must be included to prevent positive pore pressures from developing in the backfill. A higher level of construction monitoring and QC/QA is necessary to ensure that the wall is constructed in compliance with the plans and specifications.

This report also contains guidance on implementation of these recommendations.

CHAPTER 1

Summary of Research Findings

The work for this research was conducted in three phases, as indicated in Figure 1-1. The results of Phase 1 were utilized in developing the details of a full scale field test wall program.

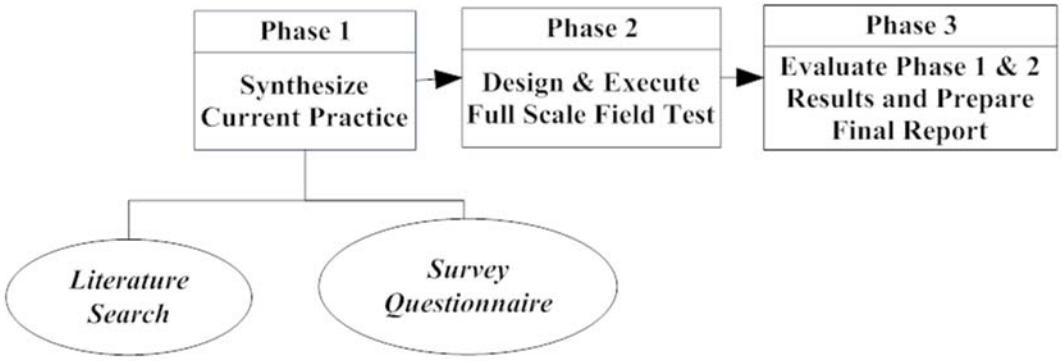


Figure 1-1. Research Plan

The synthesis of existing practice for design, specification and construction of MSE reinforced fill is included as Appendix A. It includes a literature search and solicitation of information from state transportation agencies and private industry to determine current design and construction practice for MSE wall reinforced fill and the use of reinforced fill with high fines and/or high plasticity. Cases were separated into those with geosynthetic reinforcement and those with metallic reinforcement.

Synthesis of Current Practice – Literature Search

A total of 75 case histories with geosynthetic reinforcement were located in the literature during review in 2003. Of these 75 cases, 44 are domestic and 31 are international. The majority of the walls were constructed in the period between 1980 and 2000. The earliest wall was constructed in 1974 in the U.S. The tallest wall is 35 meters high and is located in Taiwan. In these 75 cases, the performance of 23 walls was not acceptable, due either to collapse or excessive deformation of the walls. Fifteen of the failure cases occurred in the U.S. and eight in other countries. The available information on these failures indicates that water pressure in or behind the reinforced fill was the major cause for the excessive deformation or collapse of the walls. In most of the unacceptable cases, silty sand and clay reinforced fill soils contributed to the problems in the stability of the walls.

There are fewer reported case histories of the use of metallic reinforcement with “high fines” and/or “high plasticity” soils, although originally up to 25% fines were often allowed in the specifications for these systems. This may be a result of concerns about corrosion of the metallic reinforcement in these types of soils. A total of 22 case histories of walls with metallic reinforcement were identified from the literature search. Of these 22 cases, 11 were in the USA and 11 were outside the USA. Of the 22 cases, 5 were characterized as having “unacceptable” performance (excessive deformation). The domestic walls were

located in seven states, and the international walls were located in seven countries. The metallic reinforced wall cases identified in the literature review suggest the following:

- The first (1970 to early 1980s) domestic and international instrumented MSE walls generally used "low fines" reinforced soil, and wall performance was satisfactory.

- The introduction of "high fines" and/or "high plasticity" reinforced soil in full-scale production and experimental MSE walls appeared in the mid to late 1980s. The performance of these walls has varied, but serviceability problems occurred on a significant number of projects.

Subsequent to the literature search conducted for this research, Koerner (2009) further reported on 82 MSE wall failures (serviceability problems or collapse) including many of the poor performing walls from our literature review as well as a number of others. Fine grained silts and clays were used in the reinforced fill zone of 76% of the walls identified in this study and the basic failure mechanism of the failed walls was identified as external and internal water related in 68% of the cases. At the time of this report (2013), based on personal communications, Dr. Koerner has increased this data base to 156 documented failures.

The results of the literature search reported in Appendix A indicate that MSE walls on transportation projects are generally conservatively designed using granular soils, i.e. with "low fines" soil in the reinforced zone. Private MSE walls are less conservatively designed, and use a variety of reinforced soils. For example, NCMA (2009) suggests less than 35% passing 0.075mm be used, but allows >50% fines to be used if a geotechnical engineer is involved with the design to ensure that the fine grained soil does not result in unacceptable movement that may be time dependent. It is also clear from the literature that reinforced fill consisting of either "high" fines (greater than the 15% norm) or "high" plasticity (PI larger than 6%) and pore pressure resulting from lack of drainage in the reinforced zone were the principle reasons for serviceability problems (excessive deformation) or failure (collapse) of MSE walls.

Synthesis of Current Practice – Design, Specification and Construction of MSE Wall Reinforced Fill

A survey questionnaire, developed to determine current design and construction practice for MSE wall reinforced fill, was sent to each state transportation agency, District of Columbia (D.C.) and Puerto Rico (52 total) and to industry (the National Concrete Masonry Association [NCMA] and the Association for Metallically Stabilized Earth [AMSE]). Responses were received from 49 state transportation agencies and the NCMA during 2004.

The survey responses indicate that, with only a few exceptions, state transportation agencies currently conform to AASHTO requirements regarding material type and properties of reinforced fill for MSE walls. With only a few exceptions, U.S. transportation agencies specify a "low fines", non-plastic material for the reinforced fill zone of MSE walls. Most agencies specify a material that meets the requirements of AASHTO Class A-1a material, a granular soil with less than 15% fines and PI < 6%.

Private industry response was from the National Concrete Masonry Association [NCMA] and several of its individual members. NCMA guidelines allow a higher percentage of fines in the reinforced fill (suggesting 35% < 0.075 mm), but allowing reinforced fill with 50% > 0.075 mm, including soils classified as SC, ML or CL (Unified Soil Classification System, USCS). NCMA also allows soils with a Plasticity Index as great as 20%.

The following conclusions were reached from the survey responses:

- Most, if not all, U.S. transportation agencies conform to AASHTO requirements regarding material type and properties of reinforced fill for MSE walls.
- The few transportation agencies that have allowed the use of "high fines" soils for reinforced fill have had mixed results (i.e. both acceptable and unacceptable MSE wall performance).
- The level of QA/QC provided during construction of MSE walls on public transportation projects varies considerably by agency.
- It is clear that for U.S. transportation agencies to adopt the use of "higher" fines soils in reinforced fills in the future, the properties of "high fines" reinforced soils and associated design/construction controls that give acceptable performance must be demonstrated and clearly defined. Based on the survey results, we recommended a full-scale field test program to demonstrate that backfill soils with fines greater than 15% will give acceptable performance.
- The use of "high fines" and/or "high plasticity" soils for reinforced fill is much more common in the private sector, and it has been successful in many cases, especially when water did not enter the reinforced backfill.

Full-Scale Field Test

Description of the Full-Scale Test Walls

In order to establish properties for "high fines" reinforced soils and associated design controls that give acceptable performance, the NCHRP Review Panel approved the construction of a full-scale field test. The field test included provisions to demonstrate the role of pore water pressure in the reinforced fill and the importance of including a positive drainage system behind the reinforced fill zone to obtain good wall performance. Based on the survey of the literature, a few full-scale test or experimental MSE walls have been constructed (see Appendix A for the results of these studies), but none of those have rigorously evaluated the seepage issue associated with the use of "high fines" reinforced fill. Figure 1-2 illustrates the field test layout that permitted the simultaneous testing of four test sections, and Table 1-1 summarizes test section wall details. Each test section was 20 ft. high and 60 ft. long. The wall sections were designed so that they would demonstrate acceptable performance for the normal design conditions, but show distress (deformations) when subjected to extreme conditions of high water pressures in the reinforced fill and a surcharge load placed on top of the wall.

Appendix B discusses the reinforcement design of the test walls. Since the aim was to overstress the reinforcement under the extreme test conditions, a uniform and much weaker reinforcement was adopted than would be employed in usual design practice. The required geogrid strength was determined assuming lateral earth pressure equal to one-half of the Rankine active earth pressure. This was intended to result in a reinforcement that was "under strength" relative to conventional design. Polyester (PET) geogrid was selected for Walls A, B and C. A needle-punched nonwoven geotextile was used for the reinforcement of the fourth test wall (D). This was done with the idea that geotextile reinforcement could be used because it might provide horizontal drainage paths along which pore water pressure could dissipate quickly and improve the overall performance of the reinforced backfill.

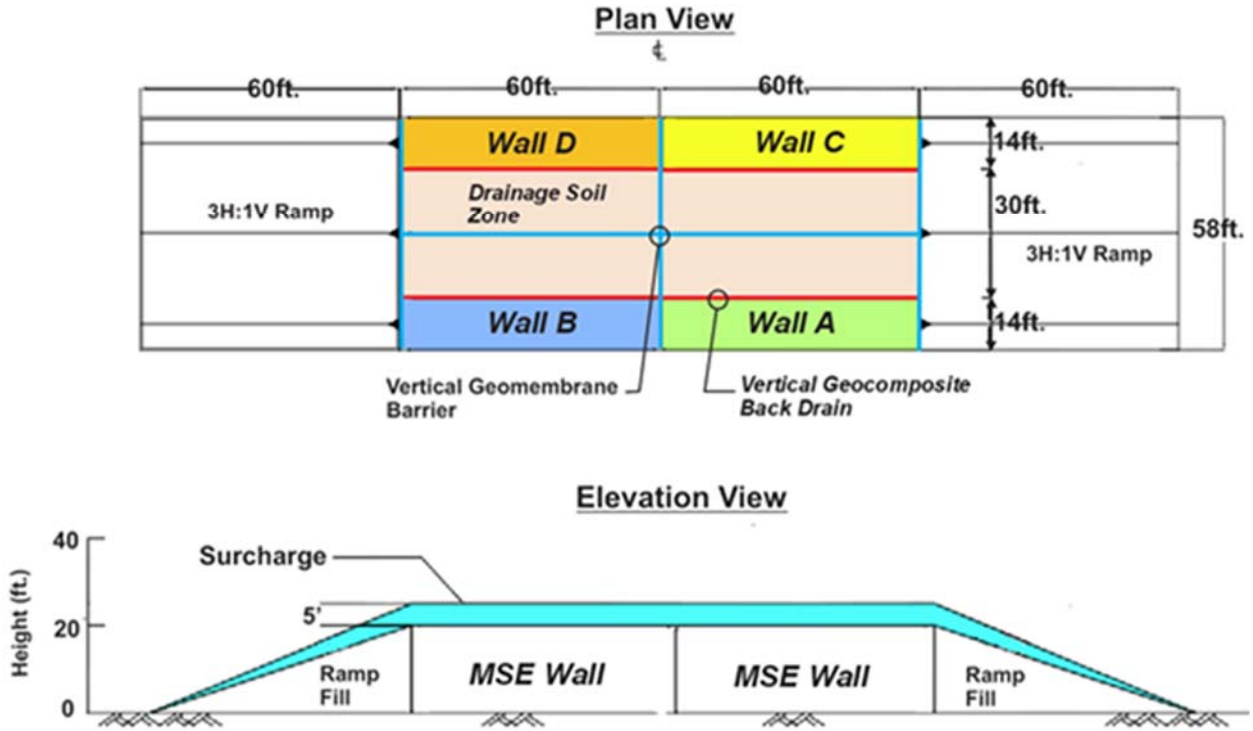


Figure 1-2. Plan/Elevation of Field Test for MSE Retaining Walls

Table 1-1: Summary of Test Wall Details

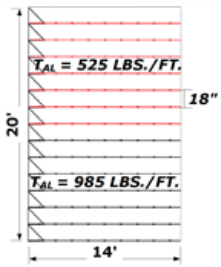
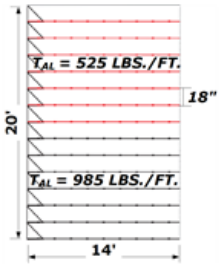
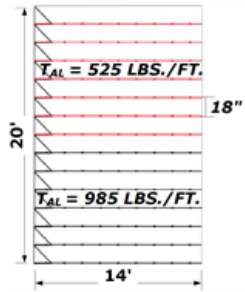
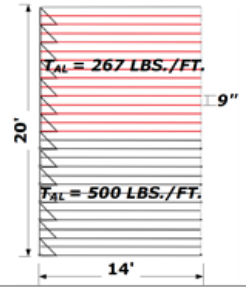
Wall	Reinforced Fill	Reinforcement	Comment
A (Control)	Target: AASHTO A-1-a (granular material with 15% or less passing the 0.075mm/#200 sieve and Plasticity Index less than or equal to 6%) Achieved: Reinforced fill with approximately 13% fines	Polyester Geogrid 	Provides a baseline of performance for current AASHTO standards
B	Target: AASHTO A-2-4 (a granular material with 35% or less passing the 0.075mm/#200 sieve and Plasticity Index less than or equal to 10%) Achieved: Reinforced fill with approximately 25% fines & PI = 0	Polyester Geogrid 	Evaluate performance of non-plastic, silty sand materials with up to 35% fines (of no plasticity)

Table 1-1 cont.: Summary of Test Wall Details			
Wall	**Reinforced Fill**	**Reinforcement**	**Comment**
C	Target: AASHTO A-4 (a silt-clay soil with more than 35% passing the 0.075mm/#200 sieve and a Plasticity Index less than or equal to 10%) Achieved: Reinforced fill with 60% fines and PI = 0 that behaves like a low plasticity silt.	Polyester Geogrid 	Evaluate high-fines silty materials
D	Target: AASHTO A-4 (a silt-clay soil with more than 35% passing the 0.075 mm sieve and a Plasticity Index less than or equal to 10%) Achieved: Reinforced fill with 60% fines and PI = 0 that behaves like a low plasticity silt.	Non-woven Geotextile 	1. Evaluate high-fines silty materials 2. Evaluate benefit of in-plane drainage of NWGT reinforcement on performance of marginal soil

The field test section walls included provisions to demonstrate the role of pore water pressure on the performance of the reinforced fill, and the importance of including a positive drainage system to obtaining good wall performance. Figure 1-3 shows how this was accomplished. A geocomposite drainage material was placed at the back of the reinforcement zone in each test section. The drainage material was wrapped around a slotted drain pipe at the bottom of the reinforced fill that facilitated the removal of water. By controlling the pressure in the water supply pipe with the slotted drain pipe open, the effect of rising groundwater level on the performance of the constructed wall can be simulated. We expected little if any

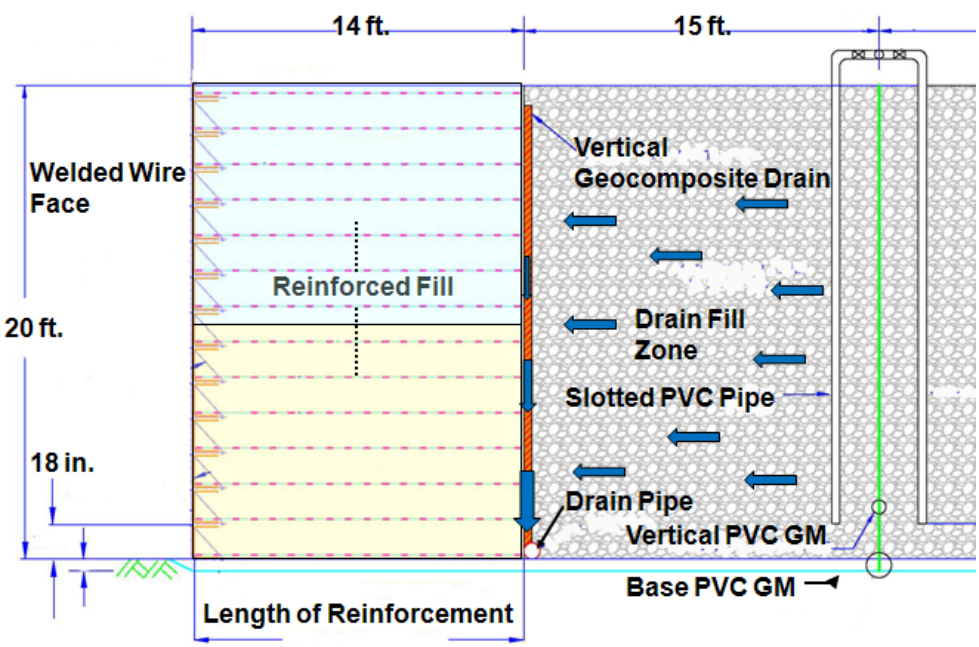


Figure 1-3. Simulation of Groundwater and Rainfall

effect on the test sections as long as the geocomposite drain functioned as designed. This phase of the test was intended to demonstrate that various reinforced fill materials will provide suitable performance, even in areas with high groundwater conditions in the natural slopes, as long as they are properly drained.

The walls were designed so that they should experience excessive deformation and failure of the reinforcement when subjected to additional surcharge loading and high pore pressures. To evaluate the failure conditions for each test section wall, the test sections were first subjected to increased pore pressures, then drained; a surcharge was added and then subjected to high pore pressures again. It was intended that the combination of surcharge, high pore pressure and rainfall would cause Walls B, C and D to fail.

Performance of the Walls

Appendix B includes a detailed discussion of the design, instrumentation, and construction of the full-scale test walls. The performance of the walls was monitored real-time 24/7 during the period October 2005 through August 2007.

Figure 1-4 provides a typical instrumentation cross section. The wall instrumentation consisted of:

- strain gages and horizontal rod extensometers to measure strains on the geogrid/geotextile reinforcement,
- vibrating wire piezometers, to measure pore water pressure in the reinforced fill, which included integrated thermistors for recording ground temperatures,
- measurements of wall face movements using permanently mounted Robotic Total Station (AMTS) units with an array of prism targets mounted on the face of the wall,
- settlement plates to measure vertical movement of the reinforced fill, and
- vertical inclinometers to measure lateral movements of the reinforced fill near the face of the test wall.

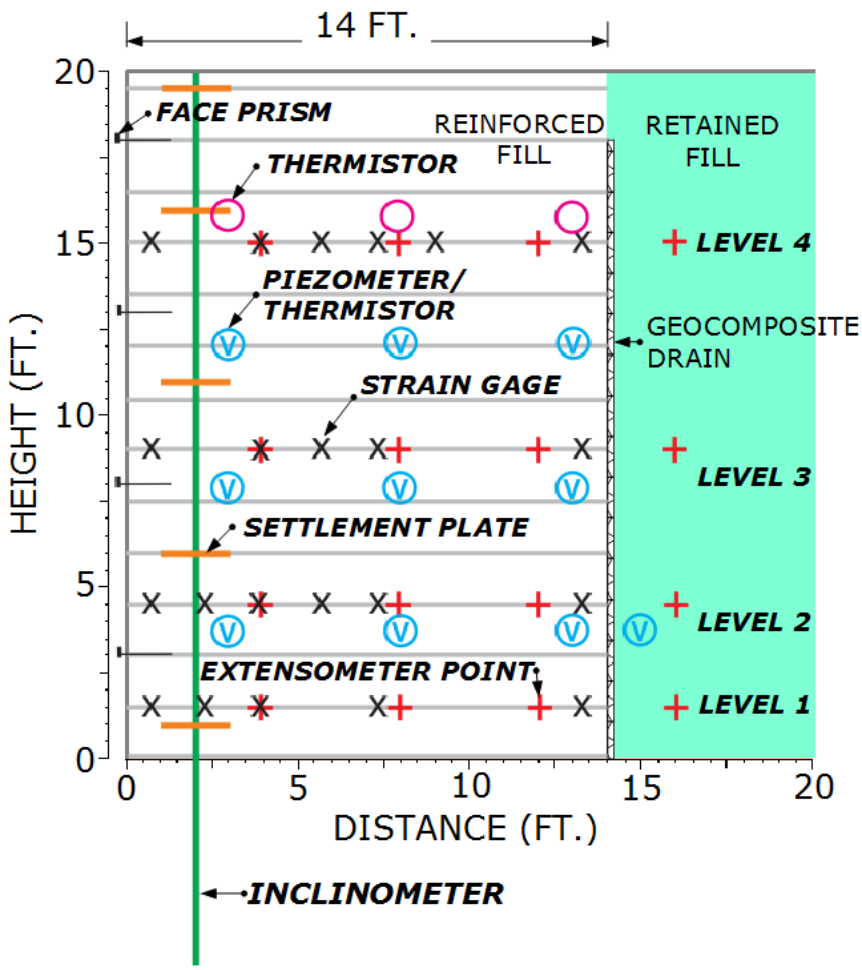


Figure 1-4. Wall Instrumentation

Figure 1-5 shows a typical instrumentation readout cluster for the test walls. Note the data logger and automated total station which provided 24/7 real-time instrumentation readings to the iSiteCentral server. Instrumentation data are summarized and presented in Appendix B.

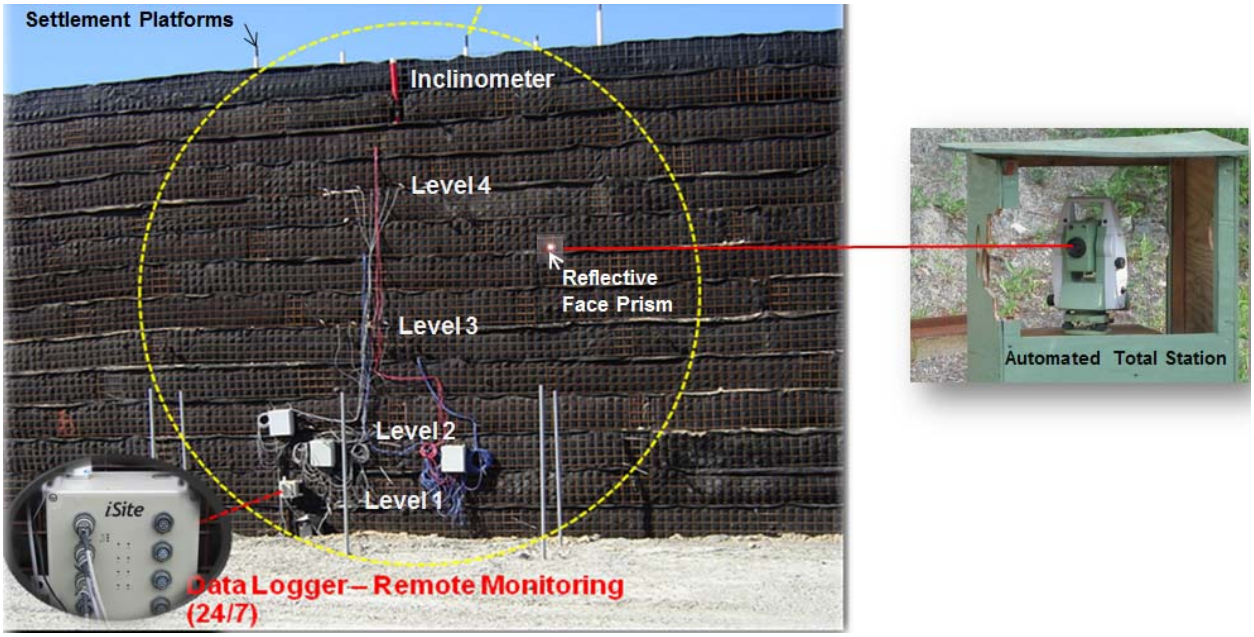


Figure 1-5. Typical Instrumentation Readout Cluster

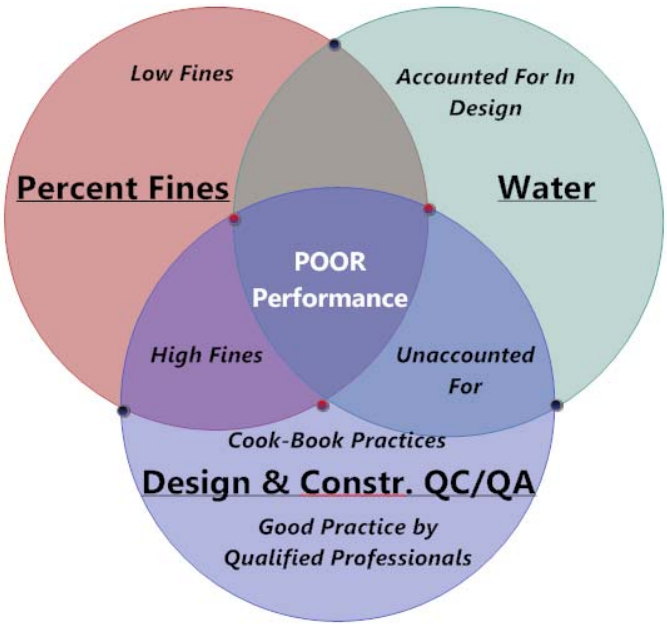
Overall the observations indicate that Walls A (13% fines) and B (23% fines) performed much better than Wall C and Wall D. Vertical and horizontal face movements were typically 1.5 to 3 times greater in Walls C (60% fines) and D (60% fines) than Walls A and B during each event and test stage (including construction and surcharge).

All four of the walls withstood extreme loading from the five foot surcharge and a buildup of pore pressure within the reinforced fill during the hydrotests without experiencing any observed failure of the reinforcing elements. This happened even though the reinforcing was purposely weakened and designed to fail during the last load event. It should be noted, however, that the hydrotests did not achieve the full pore pressure build up assumed in the reinforcement design analyses.

Wall C (with 60% fines) did experience a face/frontal zone failure at some point after completion of the field test (fall 2007) and summer 2010, most likely as a result of continued surface water infiltration. This observation is discussed further in Section 1.4.

Discussion of Results Significant to the Research

One goal of the research was to show the relationship/impact of percent fines and water in the reinforced fill on wall performance. Specifically, we expected to demonstrate that by including appropriate internal and external drainage measures, reinforced fill soils with a fines content greater than the current maximum of 15% allowed by AASHTO can be safely used.



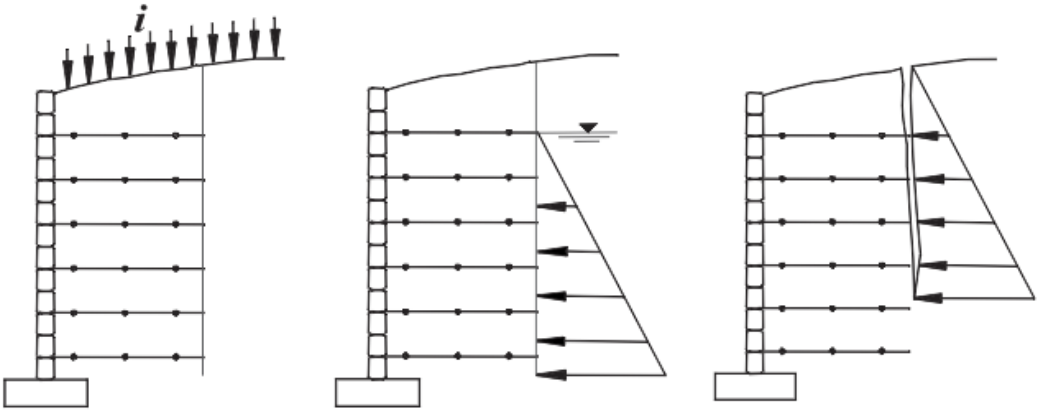
Higher Fines + Positive Pore Pressure + Bad QC/QA Practice = Poor Performance

Higher Fines + No Pore Pressure + Good QC/QA Practice = Good Performance

Figure 1-6. Relationship among Fines, Water in the Reinforced Fill Zone and Design & Construction QC/QA

As indicated in Figure 1-6, with low fines and no pore pressure (or controlled pore pressure) in the reinforced fill, we would expect acceptable wall performance. With increasing fines and no provision for pore pressure management, the likelihood of poor wall performance increases. A combination of "high" fines and no controls of pore pressure (e.g. inadequate drainage provisions) results in poor wall performance. This point is demonstrated by the number of cases with failures described in the literature and the full scale field test conducted for this research.

Figure 1-7 shows the three primary flow mechanisms that can occur in MSE structures. Conditions (A) and (B) were modeled in the full scale field test sections. Condition (C) is one that can develop at the back of the reinforced fill zone with fine grained soils, and where the tension crack can fill with water and add hydrostatic pressure.



A. Surface Water Infiltration

B. Groundwater Seepage from the retained soil zone

C. Development of Tension Crack allowing water infiltration

Figure 1-7. Water Conditions to be Evaluated in Design of MSE Walls

Surface Water Infiltration

Wetting of "high fines" soil by infiltration of groundwater or surface water causes an increase in pore pressure which reduces effective stress and results in a reduction in both the stiffness and strength of the soil. In addition, sustained rainfall increases seepage toward the wall face and, correspondingly, the increased pore water pressure effectively increases the lateral earth pressure within the wall to a greater stress than typically calculated from Rankine or Couloumb without pore pressure (Terzaghi et al, 1996 and Cedergren, 1989). Movements/deformations increase. Some densification (hydro-compaction) may also occur and contribute to increased movements. This is particularly true for soils that are placed and compacted well dry of Optimum Moisture Content.

The field test walls experienced the condition in Figure 1-7a during a record rainfall event in October 2005 shortly after construction. Fortunately the automated monitoring system captured performance during this unplanned event. Figure 1-8 presents inclinometer results for Walls A, B and C, which used the same geogrid reinforcement, after 15 in. of rainfall on the site. These results demonstrate the impact of surface water infiltration as a function of percent fines on wall performance. The buildup of pore pressure and the accompanying reduction in shear strength can be expected to increase as the percent fines increases, and, thus, increased movements/deformations would be the result.

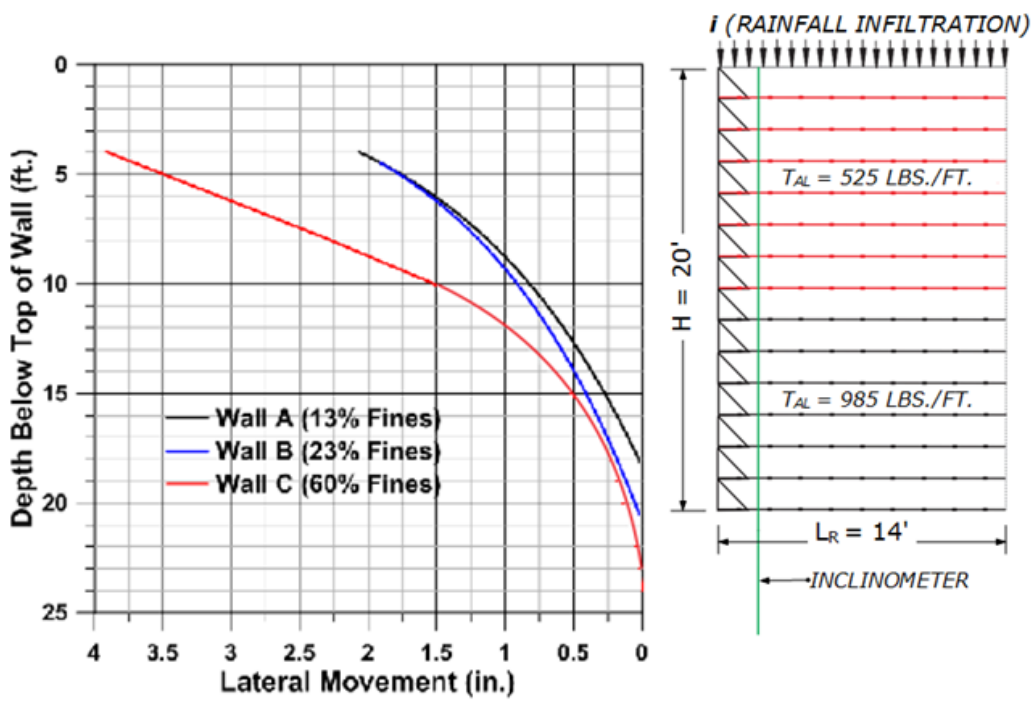


Figure 1-8. Lateral Movement of Wall Face – Oct. 2005 Record Rainfall

As indicated in Figure 1-8, the observed lateral movement in the frontal zone of Wall C (60% fines) is approximately 3 times more than that recorded at Walls A (13% fines) and B (23% fines). The increased horizontal forces from the seepage forces described by Terzaghi and Cedegren likely caused a tension crack that developed within the reinforced fill zone of Walls C and D, filled with water (Condition C (Figure 1-7)), and was largely responsible for these much larger deformations. This manifestly resulted in the introduction of hydrostatic pressure within the reinforced fill at the tension crack.

Figure 1-9 depicts the observed location of the tension crack in plan. Unfortunately most of the piezometers in the instrumentation program (as shown in Figure 1-4) were located below and behind the zone shown in Figure 1-9 and did not indicate the pore pressure increase from the water infiltration during the storm event. The one piezometer in the zone of influence also did not indicate a pore pressure increase.

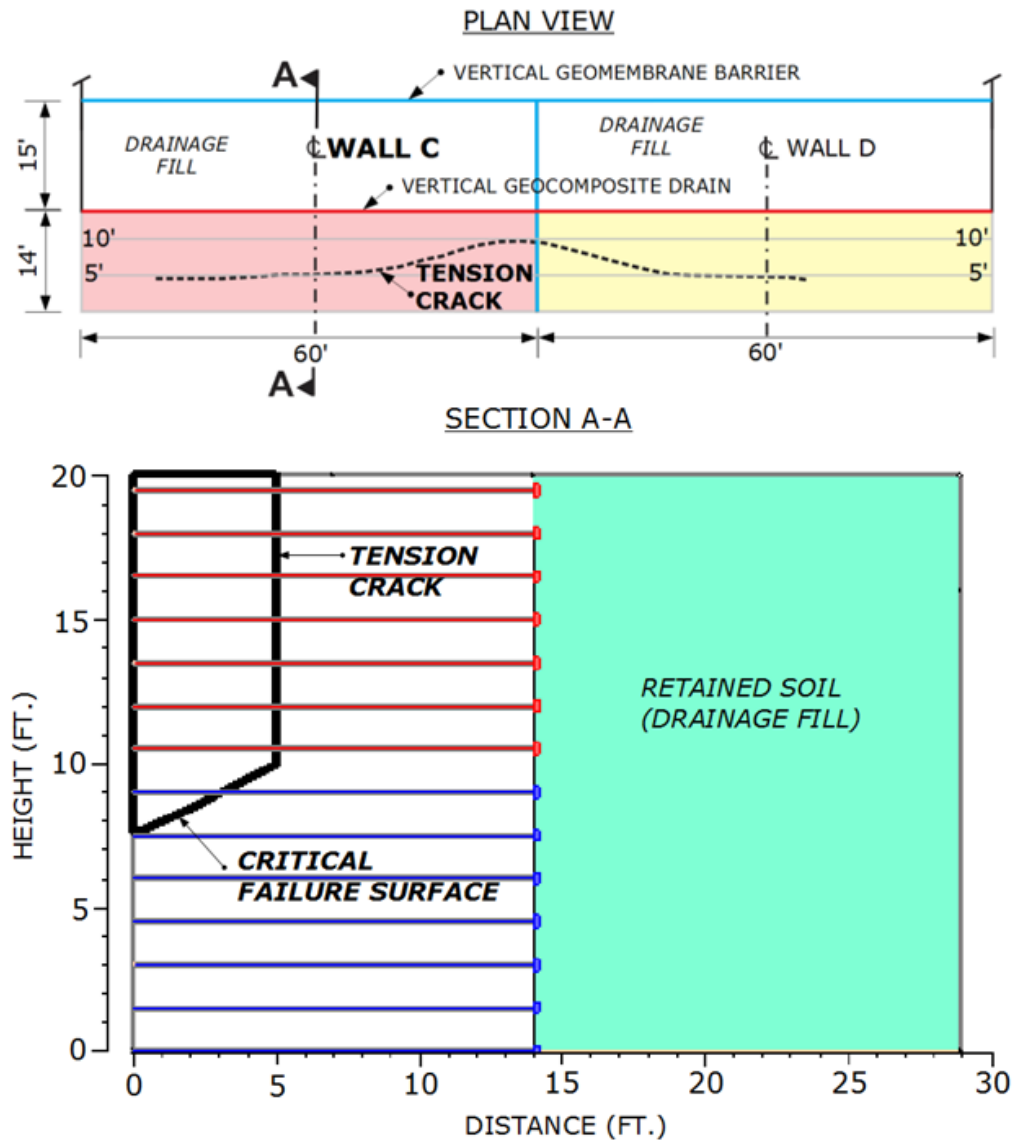


Figure 1-9. Impact of Wall C & D Tension Crack, Oct. 2005

A parametric limit equilibrium analysis was performed to evaluate the impact of the water filled tension crack on stability of the wall. The depth of the crack was varied and the critical failure surface identified is as noted in Section A-A in Figure 1-9. The calculated minimum factor of safety is 1.09. Such a reduction in the factor of safety would be expected to produce the magnitude and extent of movement recorded at Wall C. Similar large deformations were also recorded at Wall D (60% fines). As noted in Table 1-1, Wall D was constructed with a needle-punched nonwoven geotextile soil reinforcement, as opposed to PET geogrid used in the other three walls. The performance of Wall D will, therefore, be discussed separately later in this chapter.

One important observation is that Walls A and B performed similarly during the rainfall event, although the fines content of Wall B was 10% greater than the control section (Wall A). The following conclusions can be reached:

- This behavior can somewhat be explained by the placement water content during wall construction. As noted in Table 1-2, the reinforced fill in Wall A was placed dry of OMC by about 3%, while in Wall B the placement water content of the reinforced fill was approximately 1.5% wet of OMC. As such, the Wall A soils were placed at a lower degree of saturation, and, thus, likely had a higher apparent cohesion and was stiffer than the relatively wetter soil in Wall B. Correspondingly, the drier soil would likely support more load during construction, which would normally be transferred to the geogrid. As the dryer soils are wetted during infiltration, the apparent cohesion will reduce and the soil will soften, thus, transferring more load to the geogrid and inducing post construction strain. This is an artifact of placing soils with any appreciable fines too dry.

- It may also be that for Wall B, 23% fines do not have a significant influence on the granular behavior of the material, as the voids are not completely filled with fines. To some extent, this is apparent from the effective shear strength and unit weight of the soil in Wall B, which is not much different than the soil in Wall A, as shown in Table 1-2. Note that Wall C and D not only have a lower friction angle but also a much lower unit weight, and will thus respond more adversely to pore water pressure increases.

- More importantly regarding Wall A is the fact that, although the soil meets the AASHTO criteria for fines (0-15%), soil with 13% fines is not a "free draining" material. As the percentage of fines of the reinforced fill increases, its permeability decreases. With as little as 7% passing the No. 200 Sieve, the permeability can decrease by a factor of 100 to 400. Therefore, for reinforced fill with fines in the upper range of the current AASHTO criteria, the guidelines presented later in Chapter 2 of this report should be implemented in design of MSE walls where these materials are used.

Table 1-2: Properties of the Reinforced Fill (Index, Gradation, Plasticity, Permeability, Strength)

WALL	AASHTO Classification	% Fines	PI	C_U (D_{60}/D_{10})	Dry Unit Weight (pcf) (1)	Water Content (%) (1)	OMC (%) (2)	Permeability, k (cm/sec)	Friction Angle, φ (deg.) (3)
A (Control)	A-1a	13	0	200	131.5	4.9	8.0	6.3×10^{-5}	35
B	A-1b	23	0	83	129	7.5	6.0	9.3×10^{-5}	35
C	A-4	60	0	21	104.5	10.4	14.5	6.1×10^{-5}	33
D	A-4	60	0	21	107.5	10.5	14.5	6.1×10^{-5}	33

(1) Mean As-Compacted Value in Test Wall, (2) Optimum Moisture Content, ASTM D698, (3) ASTM D2434

Figure 1-10 presents results of a limit equilibrium analysis performed for Wall A, which models seepage conditions as a result of the October 2005 rainfall infiltration. The flow lines represent paths of fluid flow through the reinforced fill and retained soil materials. Flow occurs in the direction of decreasing total head. For a material with isotropic permeability, as assumed for the NCHRP test walls, the flow lines are perpendicular to contours of total head. Also the flow lines and corresponding seepage forces will change with permeability. This is a critical issue.

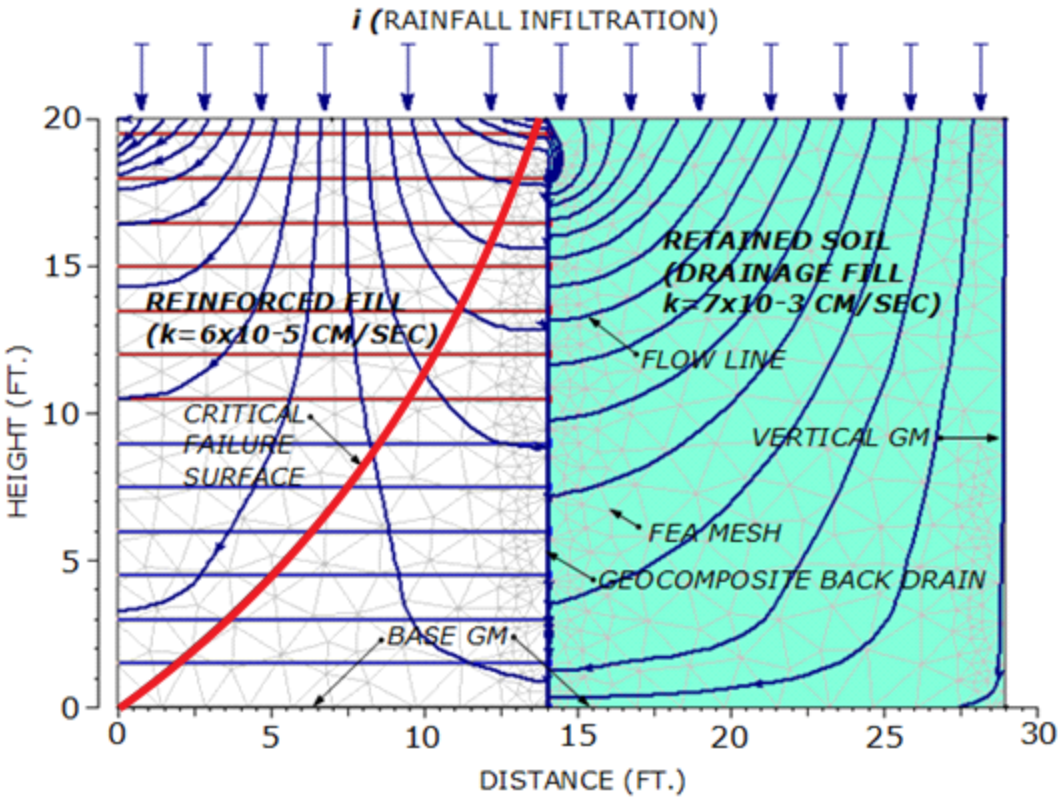


Figure 1-10. Surface Water Infiltration - Oct. '05 Record Rainfall Event, Wall A

The calculated safety factor for the critical failure surface is about 1.15 (Bishop Simplified Method), and the seepage model supports the recorded field behavior (i.e. face movements as a result of increased saturation and reduced shear strength in the frontal zone of the wall). The model also shows the destabilizing effect of preferential seepage flow toward the free draining wall face (see Terazaghi et al, 1996 and Cedergren, 1989). Since the permeability of the reinforced fill in Walls B and C is comparable to that of Wall A, similar seepage patterns would be expected in these walls.

Groundwater Seepage from the Retained Soil Zone

Two hydrotests were conducted on the full-scale test walls. During the first test, it was not possible to add enough water to increase pore pressure within the fill to more than 11 ft. It was not possible to obtain full pore pressure buildup in the reinforced fill zone in any of the walls during the short term pump test time period, mostly due to the unexpected development of substantial leaks within the backfill material where it could not be compacted to a dense condition. The inability to generate greater pressure head buildup than about 11 ft. in the first hydrotest (August 2006) resulted in a significant effort in spring/early summer 2007

to seal primary leakage zones prior to conducting the second hydrotest. Details are reported in Appendix B. Figure 1-11 indicates the phreatic surface achieved in Walls A, B and C during the second hydrotest.

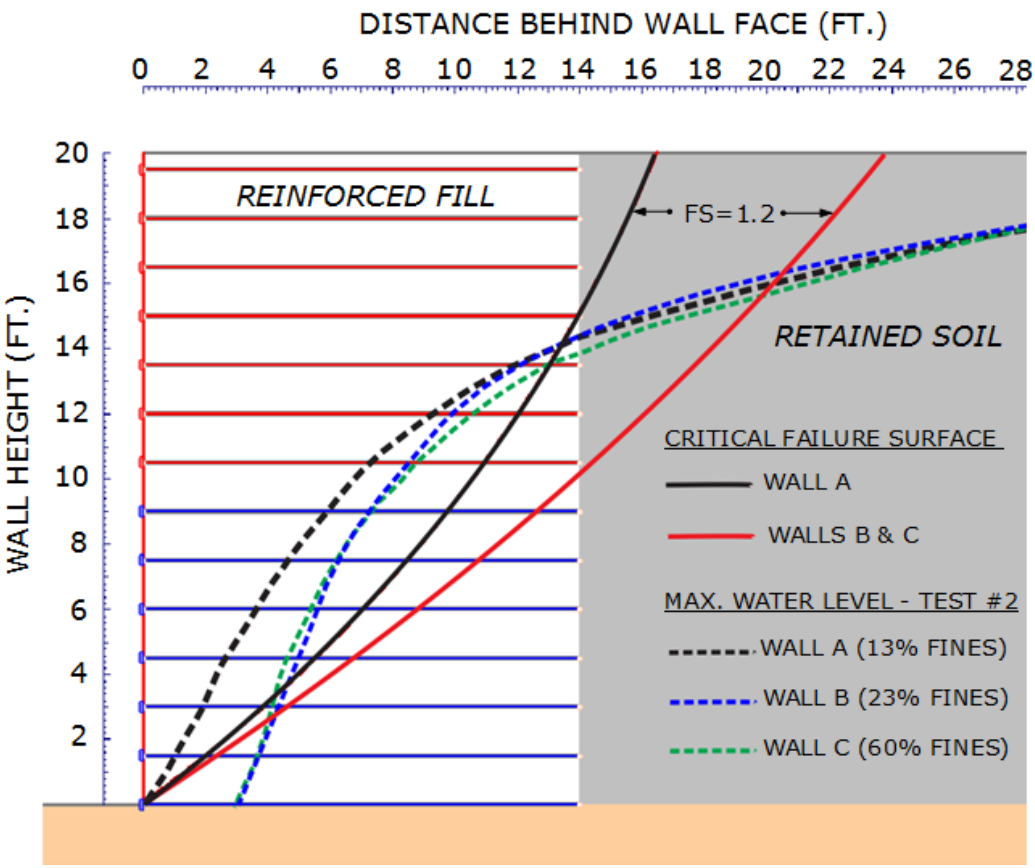


Figure 1-11. Limit Equilibrium Analysis Results – Hydrotest #2

Limit equilibrium analyses were performed to assess the impact of the induced seepage on wall stability. The results are presented in Figure 1-11. The position of the critical failure surface for each wall (compound failure) is as noted. The calculated end-of-construction minimum safety factor of 1.3 was reduced for all walls to 1.2. Relative to wall serviceability, incremental horizontal movements recorded during Hydrotest #2 were about 0.75 in. maximum for Walls A, B and C. Maximum incremental vertical movements of 0.5 in. were recorded on each wall. It would appear that the incremental performance of Walls A, B and C were comparable when subjected to the groundwater seepage conditions of Hydrotest #2. However, the phreatic surface and, thus, the head in Wall A is substantially greater than in Wall B and C (likely due to a better seal at the edges of this wall and/or more effective pumping); therefore, Wall A is effectively performing better than Wall B or C. Also, as previously indicated, a significant amount of deformation occurred in Wall C during the record rainfall event described previously. This movement likely prestressed the geogrid reinforcement and the soil, such that the horizontal movement that occurred during the hydrotest was somewhat restrained.

After completion of the first hydrotest, a 5 ft. earth fill surcharge was added and the test sequence to measure the effects of groundwater was repeated (Hydrotest #2). The walls were designed so that they should experience excessive deformation and failure of the reinforcement when subjected to additional surcharge loading and high pore pressures. Figure 1-12 presents the results of limit equilibrium analyses for the surcharge and Hydrotest #2 seepage conditions.

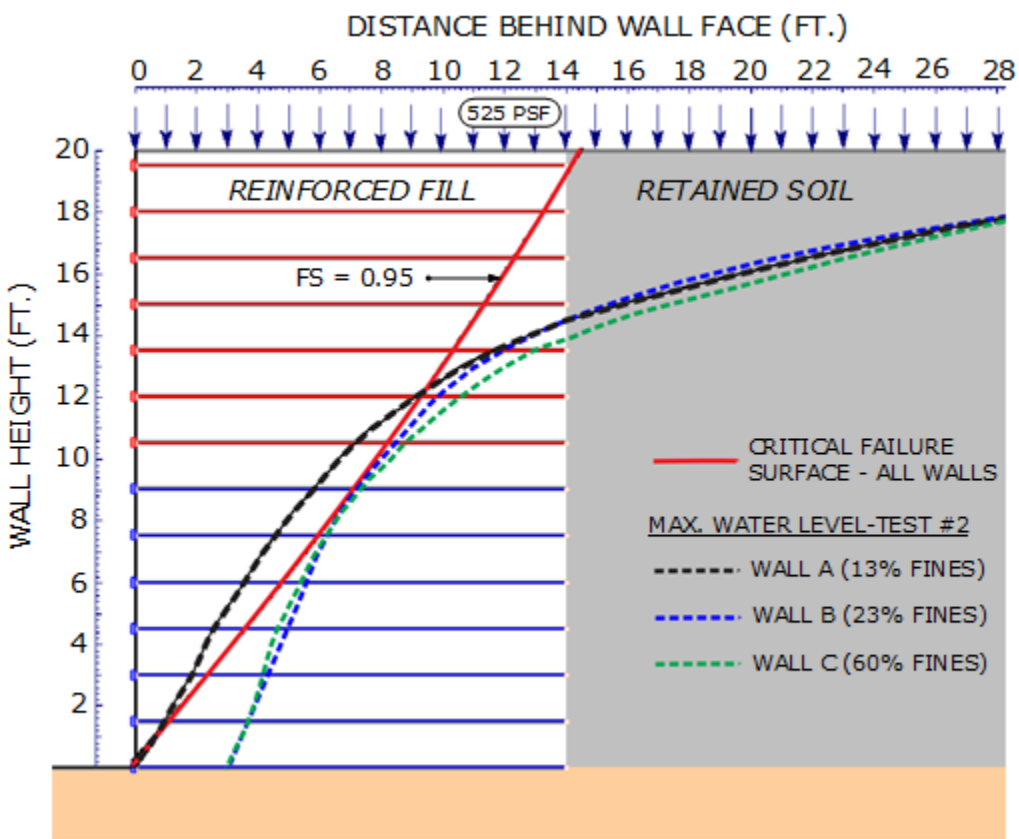


Figure 1-12. Limit Equilibrium Analysis Results – Surcharge Fill & Hydrotest #2

The calculated safety factor for Wall A, B and C is essentially at incipient failure for all three walls. However, all three of the walls withstood the five foot surcharge and a buildup of pore pressure within the reinforced fill during the hydrotests without experiencing any collapse of the wall. This outcome suggests that state of the practice methods of calculating the forces in the reinforcement for MSE walls may predict higher forces than actually develop.

Development of Tension Crack Allowing Water Infiltration

With use of high fines reinforced fill soils, tension cracks commonly occur at the reinforced fill/retained soil contact at the end of the reinforcement. This occurs due to volume decrease of the reinforced fill mass and also the outward deformation in the frontal zone of the wall. These ground surface cracks readily fill with water and exert hydrostatic pressure against the reinforced soil mass. The permeability is so low that the reinforced soil mass actually moves outward along with the facing system. With time, continuing deformations can actually produce failure of the wall face system.

This behavior was inferred at Walls C and D (both 60% fines) at the end of the test period from rod extensometer movements. It doesn't appear to have occurred at Walls A and B.

Serviceability Performance

As indicated in Table 1-3, Walls A (A-1-a soil, 13% fines) and B (A-1-b soil, 23% fines) performed similarly with respect to overall displacements. Wall C (A-4 soil, 60% fines) deformed vertically and

horizontally about 60% more than Walls A and B, based on inclinometer measurements and measurements of wall face targets with a robotic total station. This is not a surprising result because a wall constructed with a higher fines reinforced backfill is expected to deform more.

Table 1-3: Serviceability Comparison (End of 2-YR. Field Test)

WALL	Max. Horiz. Deformation at Face (in.)	Max. Vert. Deformation at Face (in.)	Max. Global Strain (%)
A (Control - 13% Fines; PET Geogrid Reinforcement)	3.5	3.25	< 2
B (23% Fines; PET Geogrid Reinforcement)	3	3.25	< 2
C (60% Fines; PET Geogrid Reinforcement)	5.5	8	< 2
D (60 % Fines; NWGT Reinforcement)	12	12	5.8

Rod extensometers were used to measure global strain occurring in the reinforced fill. The data shows that the maximum global strains in the reinforced fills at Walls A, B and C were less than 2%; generally strains were less than 1% for most calculations. For Wall D the maximum global strains were 5.8%. For all walls, the largest measured strains occurred near the wall faces.

It is interesting to note that for horizontal face movement, 60% of the movement for Walls A and B (lower "fines") occurred during the "test events" and 40% of the movement occurred as a result of "environmental factors" (rainfall, snow melt, seasonal temperature effects) during the two-year field test. Conversely, for Walls C and D (higher "fines") greater horizontal movement (70%) occurred due to "environmental factors" and 30% occurred during the "test events". As indicated earlier, the lower movement that occurred during the "test events" in Walls C and D may have been due to pre-tensioning of the reinforcement during the extreme rainfall event.

The fourth test wall (Wall D, 60% fines) has not been discussed in the earlier comparison of wall performance in this chapter, since it was constructed with a different reinforcing material (needle punched nonwoven geotextile reinforcement). Wall D experienced about one foot of vertical and horizontal deformation or roughly 5% of the wall height. Although it remained structurally intact, the face/exterior was significantly deformed and had the look of a failed wall (See Figure 1-17). Considered from the perspective of a deformation limit state, this wall, for all practical purposes, failed. It demonstrates the nature of the performance anticipated for an under-designed MSE wall system using marginal reinforced fill soil (i.e., significant lateral and vertical movements, face distortions, and tension cracks in the surface of the structure).

The Wall D design anticipated lateral transmissivity of water through the geotextile reinforcement to allow for some drainage and dissipation of pore water pressure; the intent was to improve the performance

of a marginal soil. However the piezometric measurements made during the hydrotests indicated that the pore water pressure in Wall D was approximately the same as the other walls, and the pressures did not appear to dissipate significantly faster in comparison with the other walls. The transmissivity of the selected geotextile was relatively low (an order of magnitude less than similar nonwoven, needle punched geotextiles) and the strength was 30% less than anticipated. As a result, Wall D is more an evaluation of a weaker geosynthetic, lower modulus reinforcement with high fines backfill. Although the wall did not fail, this combination resulted in unacceptable deformations for most load test conditions.

Figure 1-13 shows the wall face condition of Walls A and B in August 2006, just after the Phase 1 hydrotest and approximately eleven (11) months into the two-year field test. During wall construction, robust compaction of the reinforced fill was avoided adjacent to the vertical GM barrier out of a concern of puncturing the GM. This resulted in a less dense zone which produced the zone of vertical subsidence noted on the figure.

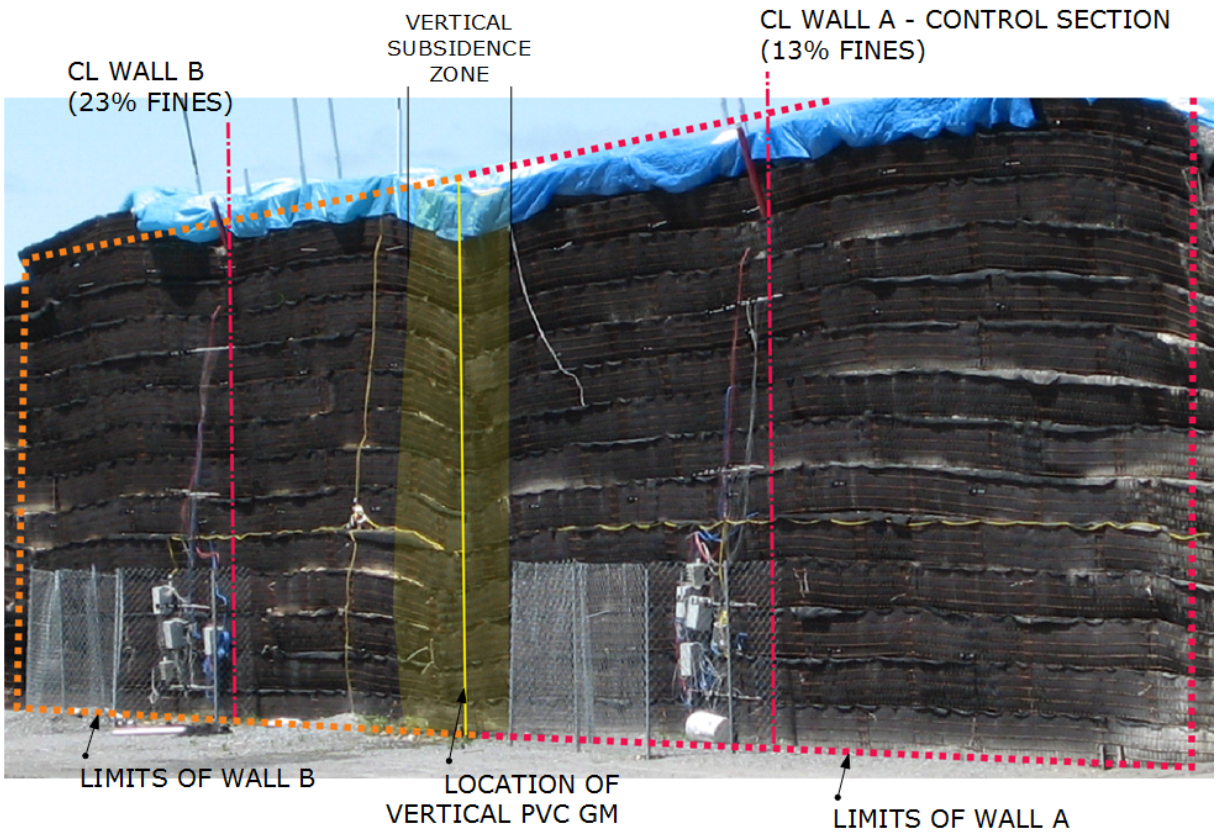


Figure 1-13. Walls A (13% Fines) and B (23% Fines), August 2006

Figure 1-14 shows the wall face condition in November 2012, five years after completion of the field test. With the exception of vegetative growth and silt staining of the face from runoff from the surcharge fill, the end-of-test conditions generally appear to have been maintained.

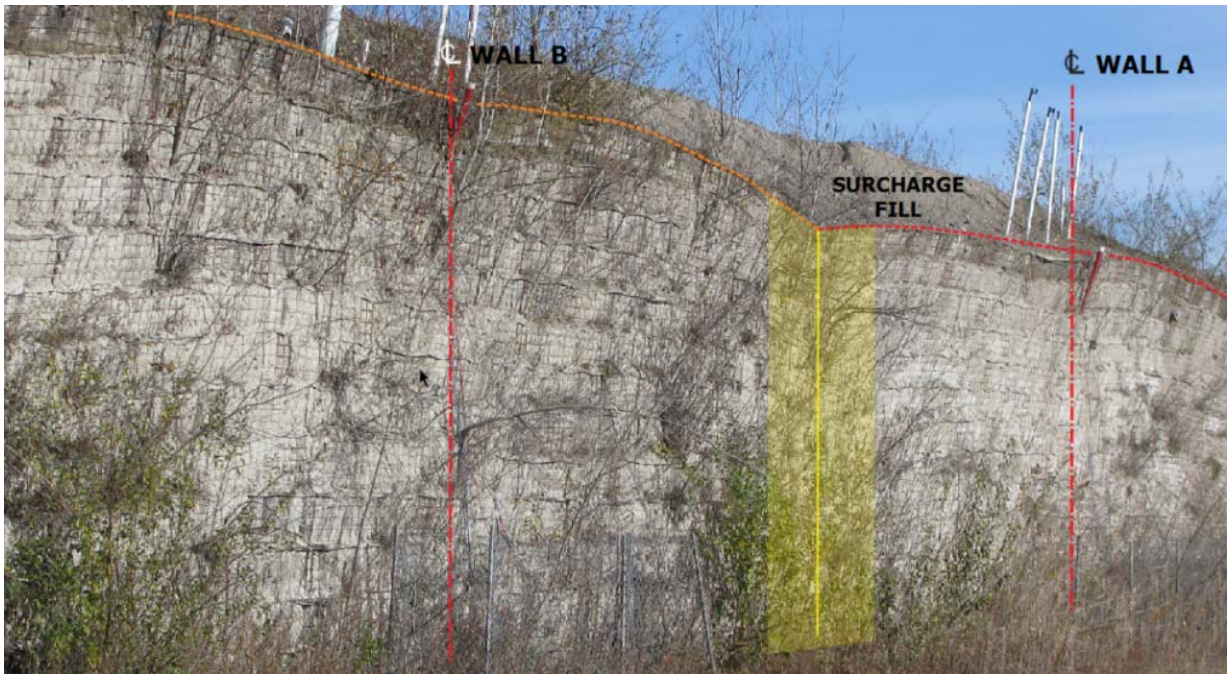


Figure 1-14. Walls A (13% Fines) and B (23% Fines), November 2012

In contrast, Figure 1-15 depicts the wall face condition of Wall C in August 2006. The larger deformations that have occurred (primarily as a result of the October 2005 rainfall events), as opposed to Walls A and B, are evident.

Figure 1-16 shows the condition of Wall C in July 2010 (approximately 3 years after the completion of the field test). The wall failed sometime between the end of the field test and summer 2010. The frontal zone of the wall basically "unzipped". This occurred as a result of continued infiltration of surface water over time. This reduced the shear strength of the reinforced fill and produced a flow slide that stripped off the geotextile face wrap and welded wire face forms in the upper half of the wall. Of note is the fact that the geogrid reinforcing

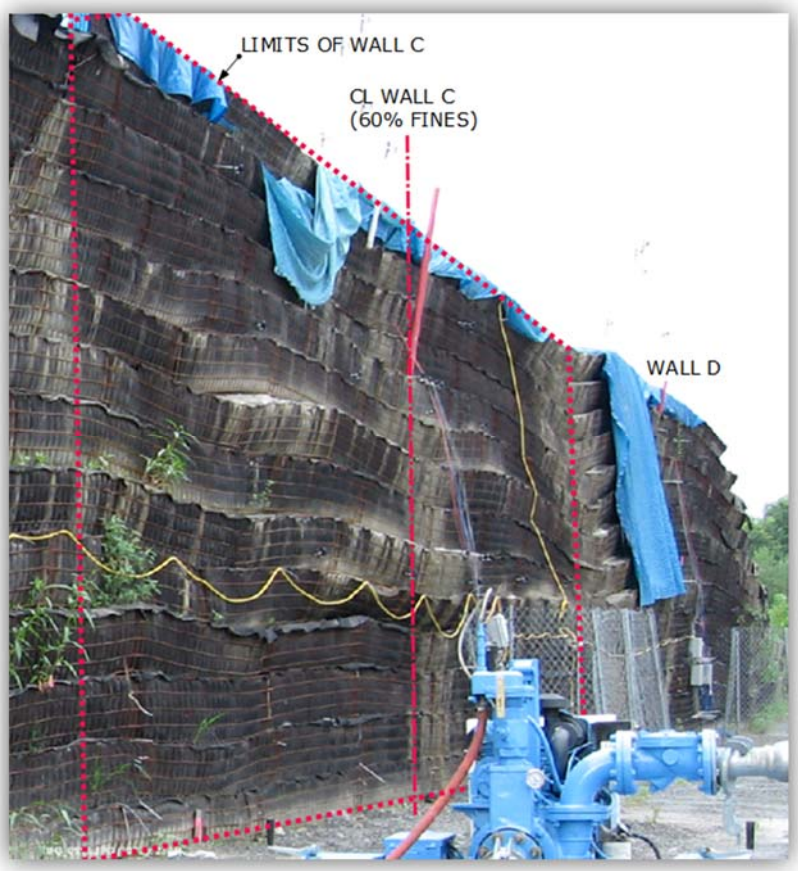


Figure 1-15. Wall C (60% Fines), Aug. 2006)

elements did not fail in tension. The geogrid layers are intact and draped down the face of the failure scarp.

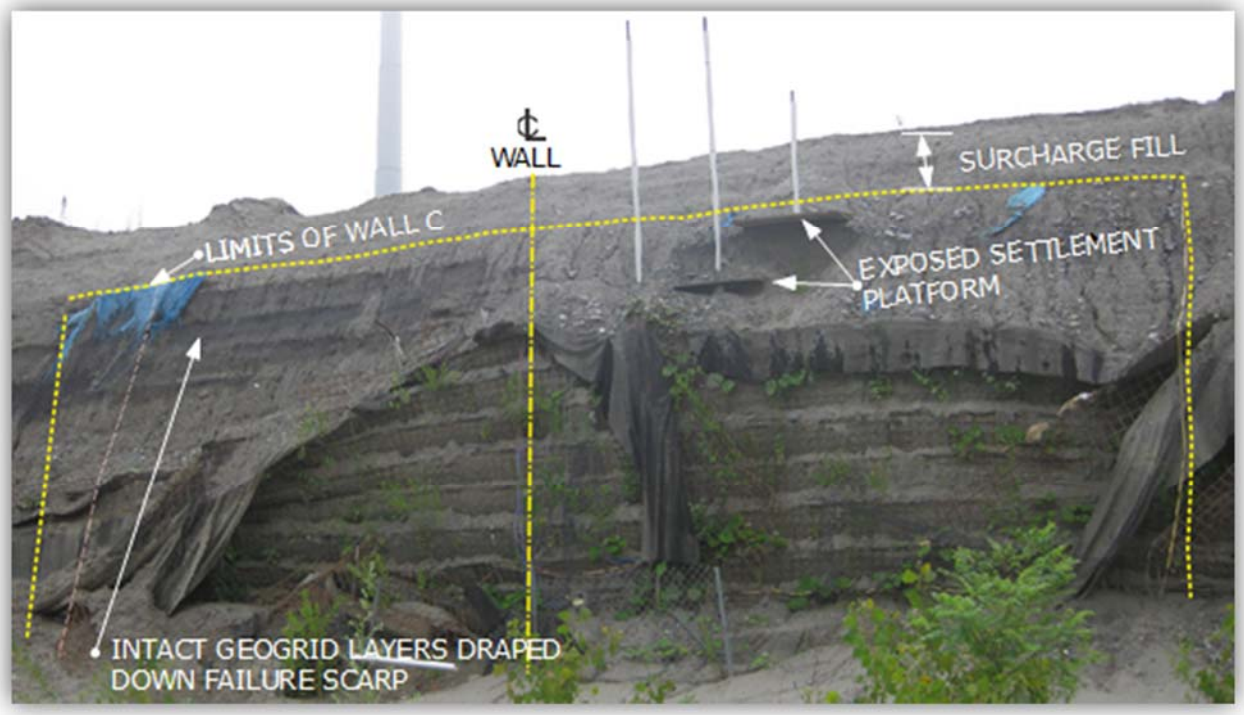


Figure 1-16. Wall C (60% Fines), July 2010

Figure 1-17 shows the wall face condition of Wall D in August 2006, just after the Phase 1 hydrotest and approximately eleven (11) months into the two-year field test. This wall, with 60% fines reinforced fill and NWGT reinforcement, experienced the largest lateral and vertical face deformations of all the walls.

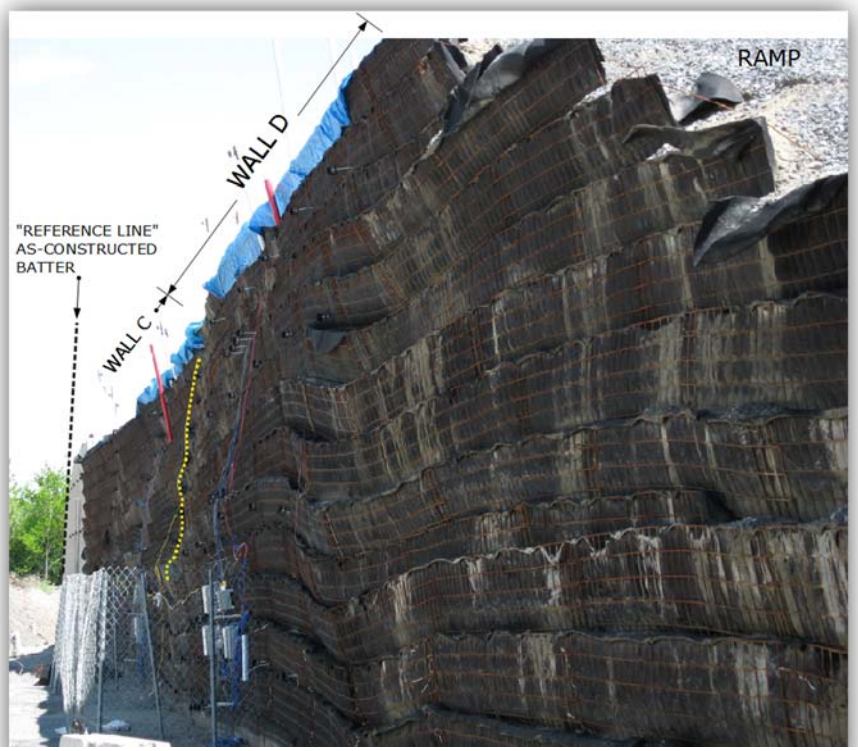


Figure 1-17. Wall D (60% Fines, NWGT Reinforcement), Aug. 2006

However, as noted in Figure 1-18, the wall unit, as opposed to Wall C, remains intact five years after the conclusion of the field test. One possible explanation is that the Wall D NWGT reinforcement is, in fact, providing some in-plane drainage and reducing pore pressures from long term surface water infiltration.

Figure 1-18. Wall D (60% Fines, NWGT Reinforcement), Nov. 2012

Conclusions from Test Sections

The test sections provided a critical evaluation of fine-grained backfill and confirmed issues that have led to poor performance of MSE walls that have used these materials. The principle issues in using fine-grained backfill and the magnitude of their impact on MSE wall performance that were demonstrated by this study include:

1. MSE walls with high fines backfills can perform adequately even with significant positive pore pressures in the reinforced backfill, but such pore pressures must be accounted for in the design. Since it may be difficult to predict what such positive pore pressures might be, the least risky design approach is to include provisions that prohibit the development of positive pore pressures in the reinforced backfill of a MSE wall during its service life. Such provisions are straight forward to apply and present relatively low increases in cost to construct.

2. There is a high potential for excess deformation and tension cracks to occur with uncontrolled seepage and groundwater, when using fine-grained reinforced fill.

3. Increased lateral stress (in terms of the impact on factor of safety) can occur due to seepage forces that can develop during extended water infiltration events (e.g. rainfall and snowmelt), as documented for conventional retaining walls with drainage only at the face by Karl Terzaghi in the 1940's.

4. Pore water pressure from uncontrolled drainage into MSE walls influences behavior of the reinforced fill.

5. Controlled drainage at the back of the MSE wall was confirmed to mitigate ground water issues, but surface infiltration requires additional control measures.

The instrumentation program including settlement plates, piezometers, inclinometers, rod extensometers, and strain gages pre-attached to the geogrid and geotextile reinforcement, was implemented to monitor the stability of the test walls during and after construction. The instrumentation connected to a remote data acquisition system with web based readout allowed for continuous, real time monitoring of the deformation response of the wall to key environmental and test events. Chapter 2, Appendix B presents the details of the instrumentation program.

In general, the instrumentation confirmed that the performance of the MSE walls was in agreement with the design assumptions, in terms of the magnitude of horizontal and vertical displacements. The instrumentation results were relied on heavily to provide information on the stability of the MSE walls. Access to the data via a WEB browser allowed the entire research team to view, discuss and make decisions based on real-time data. Use of the automated system allowed us to capture the effects of the unplanned extreme loading event of 15 in. of rain. Chapter 4, Appendix B presents details of the field performance of the test walls.

The inclinometers and the strain gages on the geotextiles provided crucial information (i.e., to assess performance during critical events such as the extreme rainfall event). Water pressure measurements indicated the effectiveness of the pumping scheme in producing a phreatic water surface within the zone of influence of the walls. In most locations, the measured strain in the geogrid reinforcements was less than 2%. Up to 5% strains were observed in the nonwoven geotextile of Wall D; however, the geotextile was well below the anticipated strength used in the design of this wall.

The results of the field tests demonstrate that MSE walls with up to 25% fines of low plasticity will perform well, provided measures are in place to prevent the development of positive pore pressures in the reinforced backfill. With additional field experience, it may become possible to increase this limit to a considerably higher fines content, as has been recommended and used by NCMA.

The project duration allowed for observations of the walls through two winter seasons. The results give some insight into the effects of freeze-thaw conditions on the performance of different backfills. The backfills contain more fines than usual, and may be more susceptible to damage from freezing. The penetration of cold was observed as 'deepest' in Walls A (least fines) and D (greatest fines). Of note is the limited extent of the freezing zone perpendicular to the wall face of all walls, particularly below about mid-wall height. Chapter 4, Appendix B discusses the thermal performance in detail.

CHAPTER 2

Guidelines for MSE Reinforced Fill with "High Fines"

Selection Criteria

Current AASHTO criteria for the selection of reinforced fill for MSE walls are presented in Figure 2-1.

7.3.6.3 – Mechanically Stabilized Earth Walls

Structure backfill material for mechanically stabilized earth walls shall conform to the following grading, internal friction angle and soundness requirements:

Sieve Size	*Percent Passing*
4.0 in. (100 mm)	100
No. 40 (425 µm)	0-60
No. 200 (75µm)	0-15

*Plasticity Index (PI), as determined by AASHTO T 90, shall not exceed 6%.

The material shall exhibit an angle of internal friction of not less than 34 degrees, as determined by the standard Direct Shear Test, AASHTO T 236 (ASTM D3080), on the portion finer than the No. 10 (2.00 mm) sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T 99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. No testing is required for backfills where 80 percent of sizes are greater than 0.75 in.

The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles.

Figure 2-1. MSEW Structure Backfill Requirements (AASHTO LRFD Bridge Construction Specifications, 3rd Edition, 2010)

In addition, "use guidelines" for MSE reinforced fill are contained in the publication entitled "Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Volumes I and II" (FHWA-NHI-10-024 and FHWA-NHI-10-025). The documents present criteria for "select granular fill material", which are the AASHTO A-1-a materials described in Figure 2-1. FHWA-NHI-10-024 also includes a discussion of the use of "marginal reinforced fill" for MSE walls (e.g. reinforced fill containing more than 15% passing a No. 200 sieve and/or a PI exceeding 6%). The report discusses a number of design considerations specific to high fines soils and provides design and construction recommendations. While no upper limit for fines is specified, this NCHRP 24-22 research study is referenced.

This research clearly demonstrates that a higher quantity of fines (at least 25%, but with PI < 6%) can be safely used in the reinforced fill, provided the properties of the materials are well defined and controls are established to address constructability issues. Accordingly, based on this research, the AASHTO gradation specification presented in Figure 2-1 can be modified to allow a maximum of 25% passing the No. 200 sieve.

Testing for High Fines Reinforced Fill Soils

Design practice for MSE structures using high fines (≤ 25%) reinforced fill soils should employ drained strength parameters for the reinforced fill, and incorporate drainage features to ensure that positive pore water pressures do not develop in the reinforced fill. Consequently parameters for strength, stiffness and permeability should be for drained conditions and stress values that cover the range of stresses within the reinforced fill.

The permeability of the reinforced fill should be at least 100 times larger than that of the retained soil, when using a single drainage pipe at the back of the lowest reinforcement layer. If the reinforced fill soil is less than 100 times the permeability of the retained soil, an aggregate or geocomposite drain should be provided at the back of all reinforcement layers. It may be necessary to measure the permeability of the high fines reinforced fill and of the most permeable portions of the retained soil, if a back drain is not used.

The following tests are recommended for high fines reinforced fill soils:

- Classification tests to determine percent fines (AASHTO T88 or T311; ASTM D422) and Plasticity Index (AASHTO T90; ASTM D 4318) should be completed on several representative samples of the potential source material prior to using it as reinforced fill. The numbers and locations of tests should be determined by someone qualified to classify soils for engineering purposes using AASHTO M145 or ASTM D2487. The design engineer should specify the number of classification tests to perform.

- Drained strength direct shear tests using AASHTO T236 (ASTM D3080). Each soil sample should be tested at three different normal stresses, approximately equal to ½, 1 and 1½ times the vertical stress at the elevation of the lowest reinforcement layer. Each material source should be tested with a minimum of one direct shear test series. The test specimens should be prepared at the highest moisture content and at the lowest dry density permitted in the field. Note that these tests are not routinely performed for MSE structures and care should be exercised in conducting and interpreting the tests. Default values should not be used unless there is sufficient supporting data on the intended backfill to establish those values.

- Laboratory permeability testing (ASTM D2434 or D5084) to determine whether the design requirement of k_{rein}/k_{ret} >100 for no back drainage is met. The test specimens should be prepared at average field moisture content and the highest dry density allowed in the field. These tests are not routinely performed for MSE structures and care should be exercised in conducting and interpreting the tests. It may be possible for an engineer to satisfactorily estimate the ratio of permeability from field classification without having to run laboratory tests, if there is a wide difference in the grain size and fines content of the reinforced fill and the retained soil and the estimated k_{rein}/k_{ret} is clearly >100.

- For steel reinforcements, electrochemical tests to establish minimum or maximum index properties, as noted in Table 2-1, to minimize corrosive potential.

Table 2-1: Recommended Tests for and Limits of Electrochemical Properties for Reinforced Fill with Steel Reinforcement		
	Criteria	**Test Method**
Resistivity	**> 3000 ohm-cm**	**AASHTO T-288**
pH	**> 5 and < 10**	**AASHTO T-289**
Chlorides	**< 100 PPM**	**ASTM D4327**
Sulfates	**< 200 PPM**	**ASTM D4327**
Organic Content	**1% max.**	**AASHTO T-267**

- For geosynthetic reinforcements, electrochemical criteria depend on the polymer (Table 2-2). The pH of the reinforced fill should be measured, if there is reason to expect low or high values.

Table 2-2: Recommended Tests for and Limits of Electrochemical Properties for Reinforced Fill with Geosynthetic Reinforcement (FHWA-NHI-09-087)			
		Criteria	**Test Method**
Polyester (PET)	**pH**	**> 3 and < 9**	**AASHTO T-289**
Polyolefin (PP & HDPE)	**pH**	**> 3**	**AASHTO T-289**

High Fines Reinforced Fill Placement and Compaction

The engineering properties of high fines reinforced fill soils also depend on the degree of compaction achieved during construction, which will be affected by the moisture content of the soil. Moisture content is highly dependent on weather conditions, nature of the borrow materials and placement procedures used by the contractor. Additional laboratory moisture-density and strength testing and field density testing will generally be required for high fines reinforced fill soils, as compared to that required for A-1-a materials.

The compaction specifications should include a specified lift thickness and allowable range of moisture content with reference to optimum. The results of the Agency survey (Appendix A) indicated a wide range of practice regarding maximum lift thickness. The majority, however, specified 8-in. maximum loose lift thicknesses, which is considered appropriate for high fines soils. Compaction moisture content limits specified by Agencies varied by geographical region. Wetting of "high fines" soil compacted dry of optimum moisture content from infiltration of groundwater or surface water can cause swelling, strength reduction, and hydro-compaction, which in turn can result in increased post construction movements/deformations. This is particularly true for soils that are placed and compacted well dry of Optimum Moisture Content. Hence, the desirability of maintaining the compacted moisture content above optimum - 2%. High fines reinforced soil compacted at moisture contents greater than optimum + 2% can be difficult to impossible to compact to the target dry density. This results in inefficiency of the compaction and lower quality in place soils. Wetter soils may also consolidate with the addition of more fill, resulting

in the potential to increase forces in the reinforcement. Therefore, compaction moisture contents for high fines reinforced fill should be between optimum moisture content minus 2% and optimum moisture content plus 2% , unless the geotechnical engineer determines that other values will work.

MSE Wall Design Considerations with High Fines Reinforced Fill Soils

When using "high fines" (low permeability) soil as reinforced fill, it is imperative that water be kept from entering the reinforced zone by collecting and discharging it away from the reinforced zone, otherwise pore pressures must be included in the analysis and design of MSE walls. Since it is difficult to predict "worst-case" pore pressures to include in a design, the recommended approach is to include elements in the design that prevent flow of water into the reinforced fill.

Figure 2-2 shows typical potential water sources. Many MSE wall designers overlook the significant flow that can occur into the reinforced fill zone through concrete joints, asphalt pavement cracks, and cracks into the pavement base materials. During the full-scale field test of this research, a record rainfall event occurred in fall 2005 that introduced significant surface water infiltration into the reinforced fill of the test walls, resulting in relatively large wall deformations. Therefore, a roughened surface geomembrane [Poly-Vinyl Chloride (PVC), High Density Polyethylene (HDPE), Flexible Polypropylene (fPP) or Linear Low Density Polyethylene (LLDPE)] with a minimum thickness of 30 mils (0.75 mm) or other equivalent water proof barrier should be provided over the top of the reinforced fill zone (Article 11.10.8 of AASHTO (2007)) of all walls constructed with high fines soils. All seams in the geomembrane should be glued or welded to prevent leakage. The geomembrane should be sloped away from the wall face. This geomembrane cover must be shaped and sloped to a pipe, swale or the back drain to collect and remove all surface water so it cannot enter the reinforced fill. Standing water on top of the wall must be prevented.

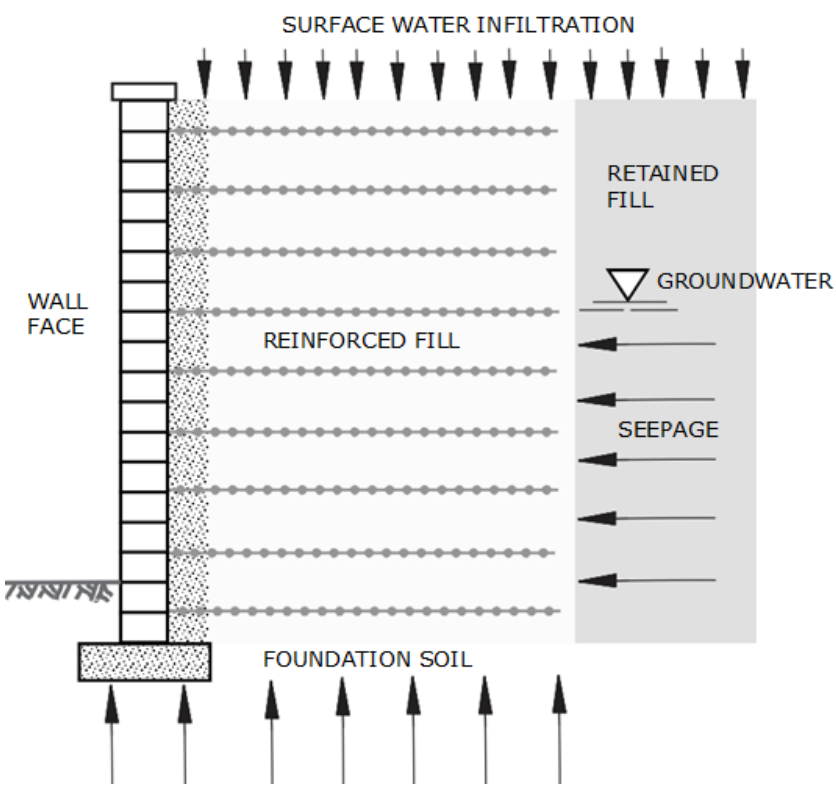


Figure 2-2. Potential Sources and Flow Paths of Water

Drainage of the retained fill zone is also of concern with respect to development of pore water pressures behind or within the reinforced fill zone. Positive pore water pressures affect the stability of a MSE wall in two important ways. Positive pore water pressures produce a horizontal seepage force on the reinforced fill that decreases stability. Positive pore water pressure also reduces the shear resistance of the reinforced fill. As demonstrated from the full-scale field test during this research (See Figure 2-3), inclusion of a vertical geocomposite back drain was shown to prevent buildup of positive pore water pressures from seepage in the retained fill zone.

These anti-seepage protection measures should be provided in all MSE walls designed with high fines (> 7%) reinforced fill.

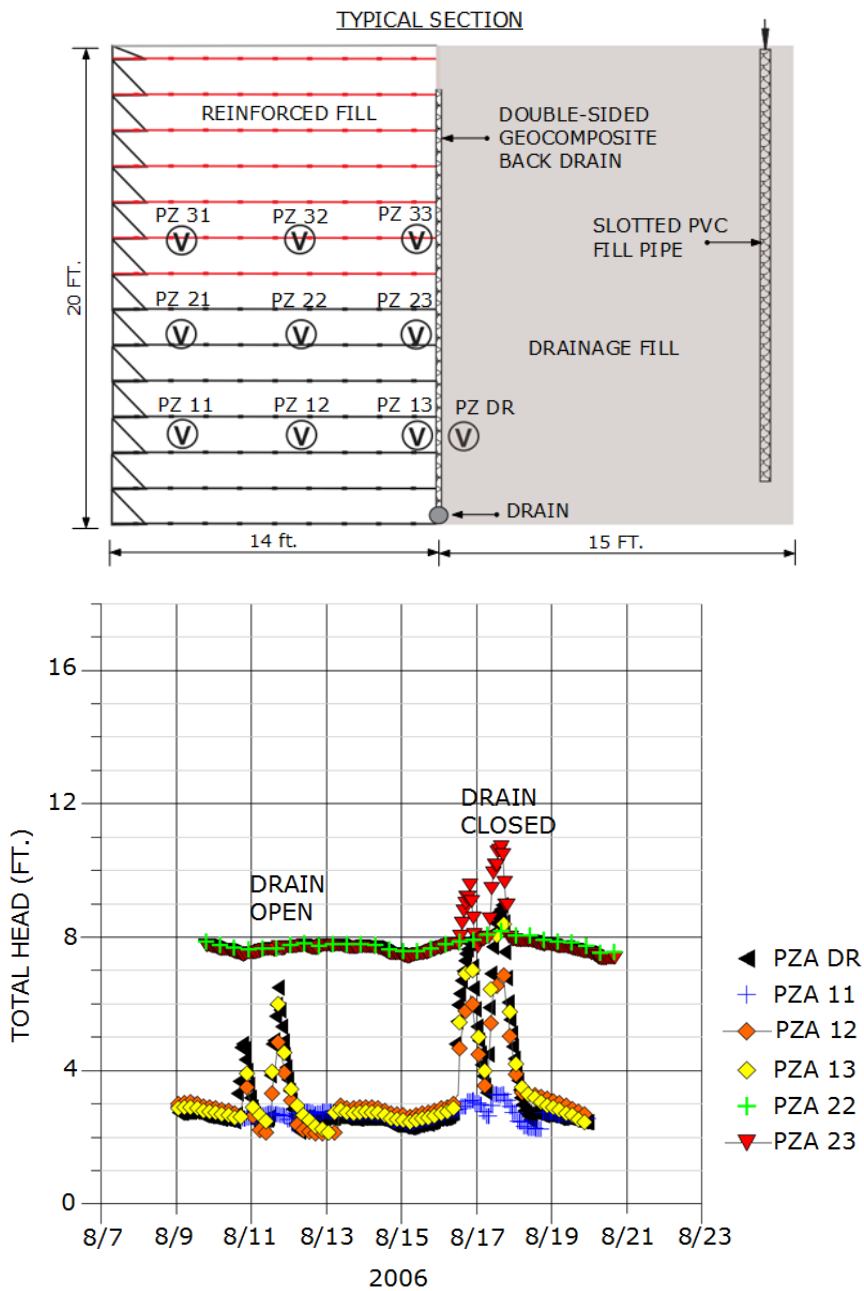


Figure 2-3. Hydrotest #1, August 2006, Wall A

The relatively cohesive nature of compacted "high fines" soil makes it prone to tension cracks that tend to form at the back of the reinforcement zone. The corresponding low permeability of high fines fill allows positive pore pressure to develop from surface water entering the crack(s). A number of wall failures have been attributed to tension cracks becoming filled with water. Upper level reinforcements should be extended to preclude the development of tension cracks. As recommended in FHWA NHI-10-024, the length of the upper 2 layers of reinforcement should be extended at least 3 to 5 ft. beyond the lower reinforcement layers to reduce the potential for tension cracks to develop directly behind the reinforced zone. If the soil reinforcement is steel, the extended layers must be contained within select granular fill (sand or gravel with < 7% fines) to avoid differential corrosion conditions.

Figure 2-4 presents conceptual details of the recommended anti-seepage measures and reinforcement design modification.

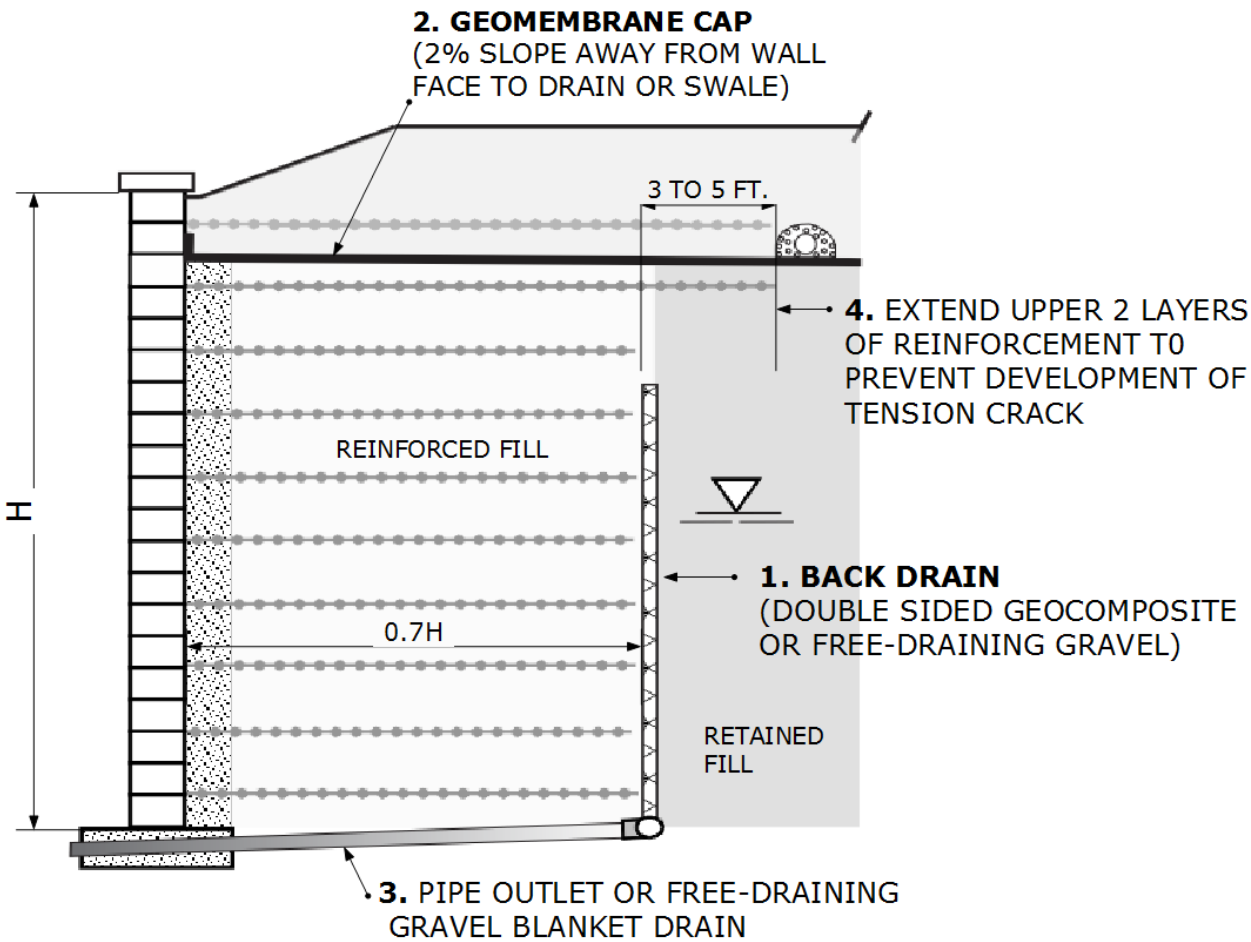


Figure 2-4. Recommended Reinforced Fill Infiltration/Seepage Protection Measures

Figure 2-5 presents an example design of a flexible face MSE roadway wall incorporating a geomembrane cap, geocomposite back drain and collection outlet.

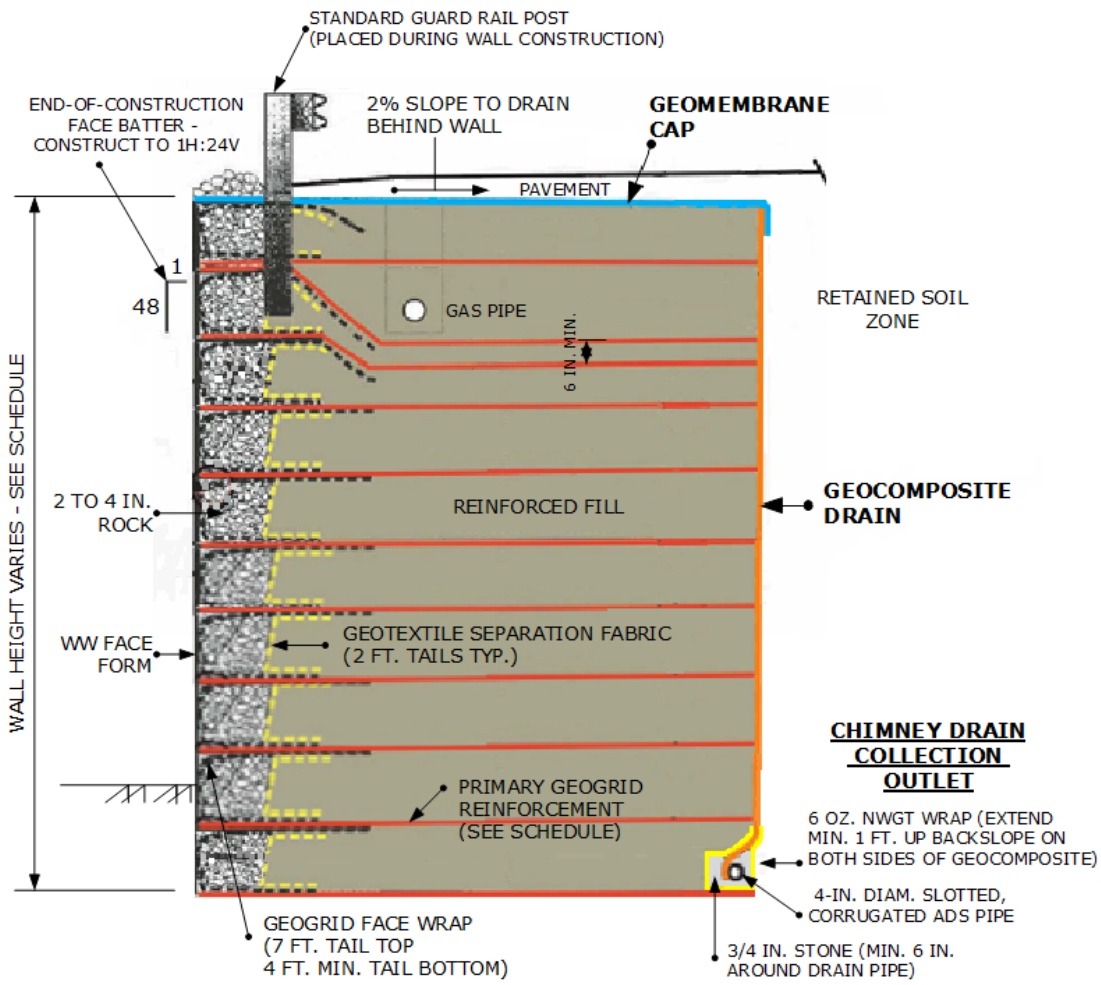


Figure 2-5. MSE Roadway Wall Incorporating Anti-Seepage Measures

One of the complications with "high fines" reinforced fill is the potential for substantial and unacceptable increases in vertical and horizontal deformation, both during and after construction. During construction, the modulus of the MSE reinforced fill is an important material property. "High fines" soils tend to deform more than clean, granular soils, and the deformation may be time dependent. Therefore, the compressibility characteristics of the backfill soil should be evaluated, depending upon the nature of the MSE structure.

Increased deformation creates several issues that must be addressed in design including:

- Maintaining wall alignment during and after construction.
- Potential deformation of supported structures and utilities.
- Downdrag on the back of facing units and facing connections.
- Increased potential for tension cracks.

With regard to wall alignment, greater care may be required during construction to meet line and grade requirements. The magnitude of post construction movement should be estimated and provided to the

designers of supported facilities. Where tolerances do not allow for post construction deformation (i.e., bridge abutments), high fines reinforced fill should not be used.

The surface of MSE structures using high fines fill should be monitored after construction to confirm that deformation has subsided before construction of supported structures. Permanent reference markers should be installed at the top and bottom of the wall at a spacing equal to the wall height, but no less than 20ft. They should be positioned so that they can be manually surveyed at any time over life of the structure.

Downdrag issues may require greater overfilling at connections, beveled and/or rounded edges on the back of modular blocks and stress relief mechanisms in rigid connections or compression pads between facing units. Alternatively, flexible facing systems should be used (e.g., geosynthetic wrapped faced walls or welded wire baskets).

Guide Technical Specification for Reinforced Fill with "High Fines"

Based on the guidelines presented in this chapter, a guide technical specification has been developed for reinforced fill with "high fines" (< 25%), and is included as Appendix C.

The guide technical specification contains proposed modifications to 7.1 - Reinforced Material (AASHTO LRFD Bridge Construction Specifications, 3rd Edition, 2010). Key elements are noted in bold italic text below.

1. Structure backfill material for mechanically stabilized earth walls shall conform to the following grading, internal friction angle and soundness requirements:

Sieve Size	Percent Passing
4.0 in. (100 mm)	100
No. 40 (425 mm)	0-60
No. 200 (75 mm)	***0-25***

2. ***The angle of internal friction of the material shall be determined by*** the standard Direct Shear Test, AASHTO T 236 (ASTM D3080), on the portion finer than the No. 10 (2.00-mm) sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T 99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. ***Each sample should be tested at three different normal stresses, approximately equal to ½, 1 and 1½ times the vertical stress at the elevation of the lowest reinforcement layer. Each source should be tested with a minimum of one direct shear test series.*** Each test in a series is to be conducted on an unsheared specimen, i.e. multi-staged shear tests are not acceptable. No testing is required for backfills where 80 percent of sizes are greater than 0.75 in. ***and fines content is less than 10%.***

3. When steel soil reinforcement is to be used:

   Organic Content less than 1% (AASHTO T-267).

4. When geosynthetic reinforcement is to be used:

   pH of 3 to 9(AASHTO T-289) – Polymer Polyester (PET)

   pH greater than 3(AASHTO T-289) – Polymer Polyolefin (PP & HDPE).

5. 7.1.1 - ***Permeability Requirements***

Laboratory permeability testing, AASHTO (ASTM D2434 or D5084) shall be performed on both the reinforced and retained fill materials, to determine whether the design requirement of krein/kret >100 for no back drainage is met. Backfill behind the limits of the reinforced fill shall be considered as retained fill for a distance equal to 50 percent of the design height of the MSE wall or as shown on the plans.

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APPENDIX A

SYNTHESIS OF CURRENT PRACTICE (LITERATURE SEARCH AND DOT QUESTIONNAIRE)

CONTENTS - APPENDIX A

FIGURES

TABLES

ATTACHMENTS

EXECUTIVE SUMMARY

Appendix A presents the findings of Phase I of NCHRP 24-22, "Selecting Reinforced Fill Materials for MSE Retaining Walls". The objective of the project is to develop selection guidelines, soil parameters, testing methods, and construction specifications that will allow the use of a wider range of reinforced fill materials within the reinforced zone of mechanically stabilized earth (MSE) retaining walls.

Phase I activities included a survey of current practice and developing parameters for lowest-quality reinforced fill soils that give acceptable performance.

SURVEY OF CURRENT PRACTICE

The survey of existing practice included a literature search and solicitation of information from state transportation agencies and private industry, to determine current design and construction practice for MSE wall reinforced fill (reinforced fill). The survey covered: a) MSE reinforced fill type and properties; b) "high fines" and/or "high plasticity" MSE reinforced fill (i.e. use of, economic implications of using, and unsatisfactory performance of walls when using); and c) enhancement of and special drainage provisions with "high fines" and/or "high plasticity" MSE reinforced fill.

Literature Search

From the published literature, 22 cases of walls with metallic reinforcement and 75 cases of walls with geosynthetic reinforcement were identified. There are very few reported case histories of the use of metallic reinforcement with "high fines" and/or "high plasticity" soils. Of the half dozen or so metallically-reinforced walls constructed with "high fines" and/or "high plasticity" soils, the performance of these walls varied; but, more often than not, serviceability problems occurred. In the 75 geosynthetic-reinforced wall cases, the performance of 23 walls were not acceptable, due either to collapse or excessive deformation of the walls. In most of the unacceptable cases, the reinforced fill consisted of silty sand and clay soils, with pore water pressure in these reinforced fill materials contributing to the excessive deformation or collapse of the walls.

Survey Questionnaire Response

A survey questionnaire, developed to determine current design and construction practice for MSE wall reinforced fill (reinforced fill), was sent to each state transportation agency, District of Columbia (D.C.) and Puerto Rico (52 total) and to industry (the National Concrete Masonry Association [NCMA]; the Association for Metallically Stabilized Earth [AMSE]). Responses were received from 49 state transportation agencies and the NCMA.

The survey responses indicate that, with only a few exceptions, state transportation agencies currently conform to AASHTO requirements regarding material type and properties of reinforced fill for MSE walls. It is clear from the responses that, for state transportation agencies to adopt the use of "higher" fines soils in reinforced fills in the future, the properties of "high fines" reinforced soils and associated design/construction controls that give acceptable performance must be demonstrated.

The use of "high fines" and/or "high plasticity" soils for reinforced fill is much more common in the private sector. However, presently, there is no rational basis for including such soils in the reinforced fill zone of MSE walls.

GUIDELINES FOR MSE REINFORCED FILL

The results of the literature search and survey indicate that MSE walls on transportation projects are generally conservatively designed, with "low fines" reinforced soils. Private MSE walls are less conservatively designed, and use a variety of reinforced soils (NCMA allows for 35% < 0.075mm). It is also clear from the literature that reinforced soil consisting of fine-grained soils (either "high" fines or "high" plasticity) and pore pressure resulting from lack of drainage in the reinforced zone were the principle reasons for serviceability problems (excessive deformation) or failure (collapse).

However, on further review, it appears that a higher quantity of fines could be safely allowed in the reinforced fill, provided the properties of the materials are well defined and controls are established to address the design issues. We have calculated that the potential savings from replacing AASHTO reinforced fill materials with marginal reinforced fill materials could be in the range of 20 to 30% of current MSE wall costs.

Experience indicates that clay soils used as reinforced fill cause too many construction difficulties, thus, offsetting any economy of their use. Therefore, "high plasticity" soils will not be evaluated for use as reinforced fill for MSE walls in Phase II of the project.

When considering the use of "high fines" reinforced fill, the following material properties have an important effect on the design and performance of MSE walls:

Hydraulic Properties

Permeability of the reinforced fill is an important operational property. As the percentage of fines of the reinforced fill increases, its permeability decreases. Wetting of "high fines" MSE reinforced fill from infiltrating groundwater, rainfall or other sources of water (e.g. snow melt, etc.) can allow pore water pressures to develop within the reinforced fill zone. Positive pore water pressures affect the stability of a MSE wall in two important ways. Positive pore water pressures produce a horizontal seepage force on the reinforced fill that decreases stability. Positive pore water pressure also reduces the shear resistance of the reinforced fill.

When using "high fines" (low permeability) soil as reinforced fill, it is imperative that either any possible water be kept out of the reinforced zone by collecting, and discharging it away from the reinforced zone, or else pore pressures must be included in the analysis and design of MSE walls.

Mechanical Properties (φ)

"High fines" soils provide less shear resistance than either "low or no" fines soils, and, thus, require stronger and longer reinforcement for MSE wall structures.

Deformation Properties

One of the more serious issues with "high fines" reinforced fill is the anticipated increase in vertical and horizontal deformation, both during and after construction. "High fines" soils tend to deform more than clean, granular soils, and the deformation may be time dependent. Therefore, the compressibility characteristics of these soils may have to be evaluated, depending upon the nature of the MSE structure.

Environmental Effects

Environmental effects become an important consideration relative to the performance of MSE walls containing "high fines" reinforced fill. These include shrink/swell potential, frost susceptibility, hydro-compaction potential, and susceptibility to surface tension cracks.

NUMERICAL ANALYSES METHODS USED FOR MSE WALL DESIGN

Chapter 3 of this report describes the range of analysis methods used since the 1970's to predict the performance of MSE walls. Limitations of the methods are discussed. An illustrative example is provided. These methods were used in Phase II of the project to design the full-scale field test.

FULL-SCALE FIELD TEST

A full-scale field test was recommended, in order to establish properties for "high fines" reinforced soils and associated design controls that give acceptable MSE wall performance. The field test included provisions to demonstrate the role of pore water pressure in the reinforced fill and the importance of including a positive drainage system to obtaining good wall performance. Based on the survey of the literature, to date, full-scale test or experimental MSE walls have not rigorously evaluated this important aspect.

Four (4) full-scale test walls (20 ft. high) were constructed in summer 2005, and were monitored through summer 2007 (two winter-spring seasons). The field test was fully instrumented, to provide answers to a wide range of technical questions. As many of the instruments as possible were electronically connected to a real-time WEB-based data acquisition system, to allow us to detect any problems at an early stage and to facilitate the dissemination of results among our team members in an efficient and effective way. During the conduct of Phase I of the project, most state transportation agencies expressed an interest in following the test results in real-time over an internet connection.

Details and results of the full-scale field test are presented in Volume III.

CHAPTER **1**

INTRODUCTION AND RESEARCH APPROACH

1.1 INTRODUCTION

AASHT0 specifications for construction of mechanically stabilized earth (MSE) retaining walls require the use of high quality, free-draining, granular reinforced fill (AASHTO Type A-1-a). High quality structural reinforced fill increases wall costs. It is anticipated that as reinforced fill supplies decrease and become too distant from MSE wall sites, costs will most likely increase. Research and current practice indicate that many soils other than those classified as AASHTO A-1-a soils have sufficient strength to be suitable reinforced fill within the reinforced zone of MSE retaining walls, but not necessarily provide adequate drainage.

An FHWA Report "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines," FHWA-SA-96-071) and a paper by Yeller (Gordon Yeller, "Experiences with Mechanically Stabilized Structures and Native Soil Backfill." Transportation Research Record No. 1474, Mechanically Stabilized Reinforced fill and Properties of Geosynthetics and Geocomposites, pp 30-38, TRB, Washington, DC, November 1995) indicate that reinforced fill materials, not meeting gradation and plasticity requirements for AASHT0 A-1-a soil, have shown satisfactory performance for reinforced soil slopes and MSE retaining walls. In addition, the National Concrete Masonry Association approves the use of reinforced fill materials other than A-1-a soil in MSE retaining walls. However, strict adherence to AASHT0 requirements for plasticity and percentage of material passing sieve #200 preclude the use of soils other than those classified as AASHT0 A-1-a as reinforced fill in the reinforced soil zone for MSE walls. Given the successful performance of reinforced slopes and retaining walls with reinforced fill other than A-1-a soils, there is a need to demonstrate that the AASHTO requirements for reinforced fill can be relaxed to permit the use of a wider range of soil types for reinforced fill within the reinforced zone of MSE retaining walls.

1.1.1 Research Objective

The objective of this research is to develop selection guidelines, soil parameters, testing methods, and construction specifications that will allow the use of a wider range of reinforced fill materials within the reinforced zone of mechanically stabilized earth (MSE) retaining walls. Success in this work will produce immediate and substantial benefits by permitting better utilization of less select and possibly on-site materials for MSE reinforced fill materials in place of A-1-a materials currently required by AASHTO specifications.

1.2 SCOPE OF STUDY

The scope of the study consists of nine work tasks conducted in three phases:

Phase I: Current Practice

Task 1: Synthesize current practice for design, specification and construction of MSE reinforced fill from literature.

Task 2: Synthesize current practice for design, specification and construction of MSE reinforced fill from survey of manufacturers and transportation agencies in US, Canada, Europe and Asia.

Task 3: Develop parameters for lowest-quality reinforced fill soils that give acceptable performance and work plan for full-scale field test.

Task 4: Provide interim report and meet with NCHRP.

PHASE II: Full-scale Field Tests

Task 5: Test reinforced fill materials and make numerical analyses for field test wall systems.

Task 6: Construct and run field tests.

Task 7: Draft guidelines and specifications for suitable reinforced fills for MSE retaining walls and meet with NCHRP.

PHASE III: New Guidelines and Specifications

Task 8: Finalize guidelines and specifications for suitable reinforced fill soils for MSE retaining walls.

Task 9: Prepare final report.

This report presents the results of Phase I.

1.3 TERMINOLOGY

For the purposes of this study, high quality, "low fines" soils are defined by AASHTO (Standard Specifications for Bridges, MSE Walls), and are essentially AASHTO A-1-a type soils. These and other terms associated with the use of these soils in MSE Walls are defined as follows:

Reinforced Fill: Soils placed and compacted in the reinforced zone of the MSE wall.

Retained Fill: Soils placed and compacted directly in back of (behind) the reinforced zone of the MSE wall.

Fines: Soils finer than a 0.075 mm (USS Number 200) sieve.

"Low fines" Soils: less than 15% passing a 0.075 mm sieve.

"Low plasticity" Soils: Soils with a plasticity index of less than 6%.

Free Draining Soils: soils with less than 3 to 5 % fines, as defined by FHWA-NHI-00-043.

CHAPTER **2**

FINDINGS FROM TASKS 1, 2, AND 3 OF THE PROJECT

2.1 LITERATURE SEARCH OF TYPE AND PERFORMANCE OF SOILS USED AS MSE WALL REINFORCED FILL

A literature search was conducted on the use and performance of different soil types for reinforced fill in MSE walls. Although the focus of the literature search was on the reinforced fill soils, it was felt beneficial to include all the available information on the wall in each case. For example, the location of the walls, wall heights, and loading conditions above the walls could serve as reference information for the design of field testing in Phase II.

2.1.1 Metallic Reinforced Walls

There are very few reported case histories of the use of metallic reinforcement with "high fines" and/or "high plasticity" soils. This may be a result of concerns about corrosion of the metallic reinforcement in these types of soils. A total of 22 case histories of walls with metallic reinforcement were identified from the literature search. Of these 22 cases, 11 were in the USA and 11 were outside the USA. Of the 22 cases, 5 were characterized as having "unacceptable" performance (excessive deformation). The domestic walls are located in seven states, and the international walls are located in seven countries.

Allen et al (2001), analyzed 16 "instrumented" case histories in a study focused on evaluating internal reinforcement stresses. The cases are of interest to this study because they include adequate information relative to the reinforced soil material properties. Figures F-1 through F-4 (See Attachment A) present information on general wall details, reinforcement information and reinforced soil information, for both the domestic and international cases reported by Allen et al.

In general, it appears that "low" fines soil was used in the reinforced soil zone of most of the walls. The walls performed satisfactorily, with no serviceability problems noted. However, two walls are of note.

However, in the 1988 Algonquin steel strip and bar mat MSE test wall (Figure F-2), the reinforced soil zone consisted of a non-plastic silt (90% < 0.075mm). An increase in lateral movement of the wall face (on the order of 50% greater than walls constructed with low fines reinforced fill) was observed during construction of the wall. A significant increase in deformation (approximately twice the initial deformation) was observed over the first winter season and was attributed to frost effects. Another important observation was the significant force measured directly beneath the facing panels, which was over five times greater than the weight of the panels themselves. This increase was attributed to down drag stresses on the back of the facing units.

In the 1988 Cloverdale, CA bar mat MSE wall, an on-site, clayey, sandy Gravel (PI < 10, 11-17% < 0.075mm) was used to construct the reinforced soil zone. There were no indications of significant wall movements. However, reinforcement loads increased with time during the first year of measurement, and was considered to be time-dependent behavior as a result of using a relatively cohesive reinforced fill.

Mitchell et al (1995) reported six case histories in which "poorly draining" soils were used to construct the reinforced soil zone of the MSE walls. We interpret "poorly draining" to

mean "high fines" and/or "high plasticity" soils. Table T-1 (See Attachment B) presents information on general wall details, reinforcement information, reinforced soil information, and wall performance, for both the domestic and international cases reported by Mitchell et al.

The performance of the MSE walls reported by Mitchell et al generally indicates serviceability problems (excessive movement and cracking) were encountered with the use of "high fines" and/or "high plasticity" reinforced soils. Of the three domestic MSE walls listed in Table T-1, two of the walls experienced significant serviceability problems. The international cases suggest similar serviceability issues. Two of these walls were experimental.

Summary

The metallic reinforced wall cases identified in the literature review suggest the following:

- The first (1970 to early 1980s) domestic and international instrumented MSE walls generally employed "low fines" reinforced soil, and wall performance was satisfactory.
- The introduction of "high fines" and/or "high plasticity" reinforced soil in full-scale production and experimental MSE walls appeared in the mid to late 1980s. The performance of these walls varied, but serviceability problems occurred on a number of projects.

2.1.2 Geosynthetic Reinforced Walls

A total of 75 case histories with geosynthetic reinforcement were in the literature. Of these 75 cases, 44 are domestic and 31 are international. The information on each case has been organized into four categories: general information of the walls, reinforcement information, reinforced fill soil information and the full citation of the referenced literature.

Furthermore, the cases in which the walls exhibited either excessive deformation or collapse are identified and summarized in a separate table (Table T-4). Of the 75 cases identified, 23 are categorized as having "unacceptable" performance.

Domestic Cases

Tables T-2 (a) to (e) [See Attachment B] present the information on the 44 domestic reinforced wall cases. The walls are located in 18 states.

General Information. General information on the walls is summarized in Table T-2 (a). Figure 2-1 presents a bar chart of the date of wall construction in 5-year increments (Two of the walls did not report the construction date.). The earliest wall was constructed in 1974 and was re-evaluated in 1999, 25 years after construction. Thirty-seven of the 44 walls were constructed during the 15-year period from 1985 to 1999.

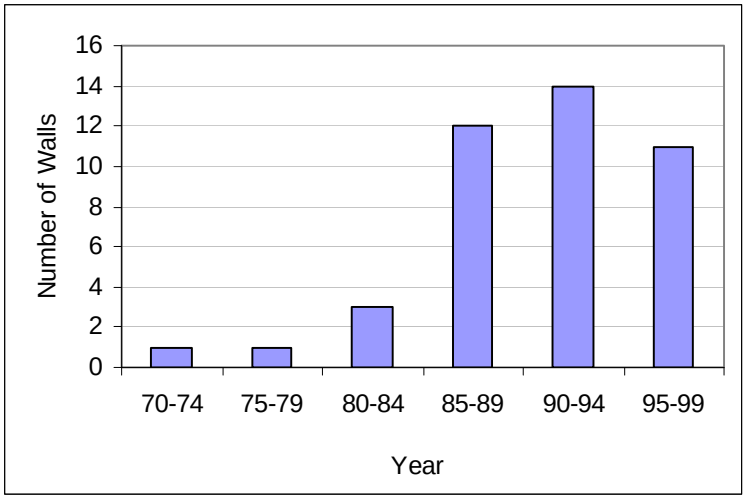

Figure 2-1 Number of Domestic Walls constructed in five-year increment periods

The height of the walls ranged from 2 to 18 meters. The number of walls, in 1.0 m wall height increments, is shown in Figure 2-2 (Note that the last column covers wall heights from 10 to 20 m.). The majority of the walls have a maximum height less than 10 meters.

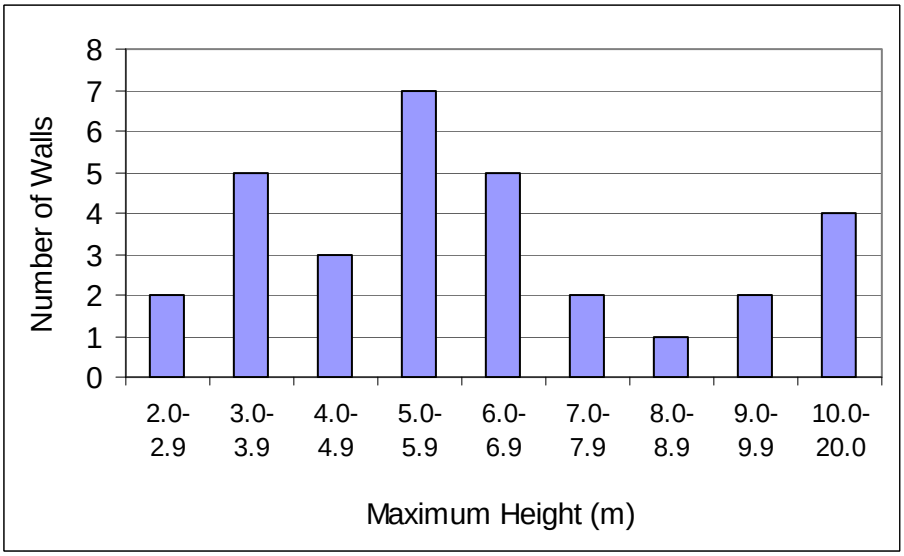

Figure 2-2 Number of Domestic Walls, in 1.0 meter wall heights (except last column)

The types of wall facing included modular concrete blocks, full height pre-cast panels, gabion baskets, shotcrete, wrap-around geotextiles and steel mesh. The majority of the walls were built in cut areas. Loading conditions above the walls included roadways, bridges, parking lots and soil surcharges.

Reinforcement Information. Information on the geosynthetic reinforcement is summarized in Table T-2 (b). Thirty-seven of the 44 walls were reinforced with geogrids; the other seven walls were reinforced with geotextiles. The vertical spacing and length of the reinforcement layers are included in the table.

Soil Information. The reinforced fill soil information is shown in Table T-2(c). The table includes soil type in the reinforced zone, the retained soil, and the fill and cut zones, as illustrated in Figure 2-3.

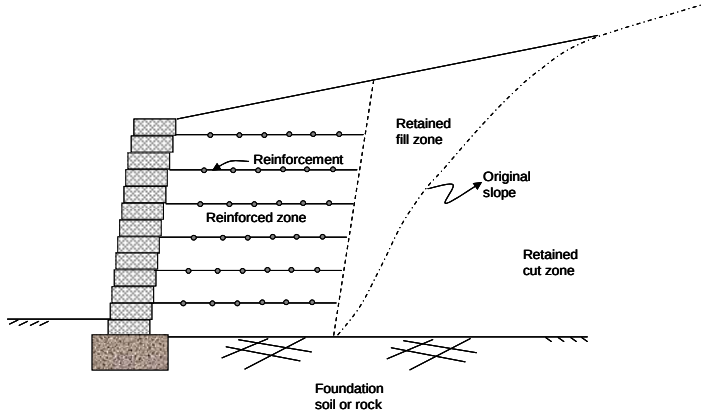

Figure 2-3 Cross-section of a MSE wall illustrating the different soil zones

Of the 44 domestic cases, three did not provide the description of the soil type in the reinforced zone; however the friction angle of the soil was included. The range of friction angles of the soils in the reinforced zone is between 32° and 40°, except one case indicated soil with a friction angle of 22°. In 14 cases, the soil in the reinforced zone consists of silty sand, with small amount of clay. The soil types in the reinforced zone have been categorized into three groups, in which the friction angle range and "fines" content are identified, as shown in Table 2-1. The densities of the soils range from 18 to 22 kN/m^3. Regarding placement and compaction procedures of the soil in the reinforced zone, 12 cases provided a detailed description.

The soil information in the retained fill and retained cut zones is very limited. For Cases, #5 and #18, soil with friction angles lower than that in the reinforced zone was used in the retained filled zone. In Case #22, the soil in the reinforced zone was the same as the in-situ soil.

Table 2-1 – Characteristics of the soil types in reinforced zone

Soil in Reinforced Zone	Friction angle range	Fine soil (< 0.075 mm)
Silty sand with clay	30° to 34°	Yes; undefined amount
Sand	32° to 34°	Yes; undefined amount
Gravelly sand and gravel with some amount of fines	34° to 40°	Yes, 3 to 15% in some cases

Wall Performance. The performance of the walls is described in Table T-2(d). "Acceptability" is judged based on the performance of the wall after construction. Fifteen out of the 44 cases (Cases #3, #10, #29 and #33 to #44) are categorized as unacceptable. The failure of the wall in Case #3 was caused by improper surface grading that led to infiltration of surface water behind the wall. For Cases #10, #30 and #41 to #44, failures were caused by improper design that led to wall collapse. Case #29 was an experimental wall that was intentionally under-designed to achieve internal stability failure. In Cases #34 to #40, the walls collapsed due to water forces that were not considered in the design.

The design method for some of the cases is also included in the table. Table 2-2 indicates the design method and the number of the walls constructed based on that design.

Table 2-2 – Design methods used in the domestic cases

Design Method	Number of cases
AASHTO, FHWA and DOT	6
NCMA	2
"MSWALL" computer program	4
Manufacturer's design program	4
Rankine method (Bell, Koerner)	4

References. Table T-2(e) lists the full citation of the domestic cases that were identified from the literature search.

International Cases

Tables T-3 (a) to (e) present the information on 31 international reinforced wall cases. The walls are located in 9 different countries: Canada, France, Germany, Italy, Norway, South Africa, Taiwan, United Kingdom and Panama. Fourteen of the wall cases were located in France.

General Information. General information on the walls is summarized in Table T-3 (a). Figure 2-4 presents a bar chart of the date of wall construction in 5-year increments (Three of the walls did not report the construction date.). The earliest wall was constructed in 1982. It was re-evaluated in 1992, 10 years after construction. The majority of the walls were constructed in the period of 1982 to 1998.

The height of the walls ranged from 1.4 to 35 meters. The number of walls, in 1.0 m wall height increments, is shown in Figure 2-5 (Note that the last two columns have different maximum wall height ranges.). The majority of the walls have a maximum height less than 10 meters. The two tallest walls have a maximum height of 27.5 and 35 meters, and they are located in Italy and Taiwan, respectively.

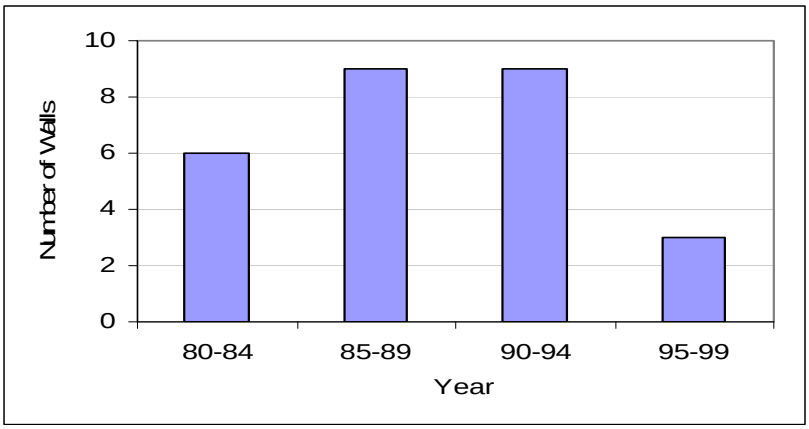

Figure 2-4 Number of International Walls constructed in 5-year increment periods

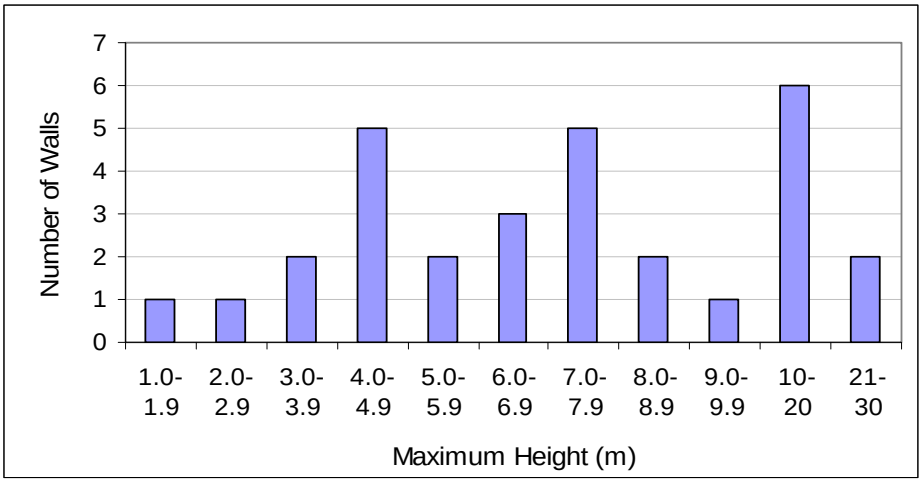

Figure 2-5 Number of walls, in 1.0 meter wall heights (except the last two columns)

There are variety of wall facing types, including modular concrete blocks, full height pre-cast panels, vegetated, wrap-around geotextiles, steel mesh, bituminous, timber lagging and used tires. The loading conditions above some of the walls include roadway, parking lots and embankments.

Reinforcement Information. Information on the geosynthetic reinforcement is summarized in Table 2-5(b). Sixteen of the 31 walls were reinforced with polyester (PET) woven geotextiles; twelve walls were reinforced with geogrids. The vertical spacing and length of the reinforcement layers are included in the table.

Soil Information. The reinforced fill information is shown in Table T-3(c). The table includes soil type in the reinforced zone, the retained soil, and the fill and cut zones, as illustrated in Figure 2-3. Of the 31 cases, five cases did not provide the description of the soil type in the reinforced zone; although one of the cases did include the friction angle of the soil. The range of friction angles of the soils in the reinforced zone is between 30° and 41°, except one case is 26°. In twelve cases, the soil in the reinforced zone consists of silty sand, with a small amount of clay. In Cases #12 and #19, the soil in the reinforced zone exhibited cohesive properties, indicating the presence of clay. In Cases #24 and 31, the reinforced fill soil consisted of 25% "fines" (less than 0.075 mm; #200 Sieve).

The soil types in the reinforced zone have been categorized into three groups, in which the friction angle range and fines content are identified, as shown in Table 2-3. Only five cases provided the soil density values and they ranged from 17 to $21 kN/m^3$. Regarding placement and compaction procedures of the soil in the reinforced zone, only four cases provided a detailed description. Soil information in the retained fill and retained cut zones was not provided.

Table 2-3 – Characteristics of the soil types in reinforced zone

Soil Type in the Reinforced Zone	Friction angle range	Fine soil (< 0.075 mm)
Silty sand with clay	26° to 31°	Yes; undefined amount
Sand	30° to 35°	Yes; undefined amount
Sandy gravel and gravel with small amount of fines	35° to 41°	Yes, up to 12% in some cases

Wall Performance. The performance of the walls is described in Table T-3(d). "Acceptability" is judged based on the performance of the wall after construction. Eight out of the 31 cases (Cases #1, #19, #20, #24, and #28 to #31) are categorized as unacceptable. In Cases #1, #19 and #20, the walls collapsed within 3 years after construction. The failure of the walls in Case #1 and #19 were caused by water in the reinforced fill that induced unexpected pore pressure. In addition, some of the reinforced geotextile layers in Case #19 were omitted during construction. For Case #20, the failure was caused by an incorrect spacing between the reinforcing layers. In Case #24, a significant amount of deformation was observed after three years of construction. The soil in the reinforced zone of this wall consisted of a high percentage of fines, leading to saturation and loss of reinforced fill soil. The walls in Cases #28 to #31 collapsed due to high pore pressure within the reinforced fill.

The design methods are also included in the table; however, only three cases included such information. Two cases were designed based on the standards in the corresponding countries, and one case was designed using the FHWA method. All three walls had acceptable performance.

References. Table T-3(e) lists the full citation of the international cases that were identified from the literature search.

Summary

From the published literature, 75 reinforced walls with geosynthetic reinforcement have been identified and cataloged. Forty- four of the cases are located in the United States (U.S.) and 31 are located in nine other countries. The majority of the walls were constructed in the period between 1980 and 2000. The earliest wall was constructed in 1974 in the U.S. The tallest wall is 35 meters high and is located in Taiwan.

The types of reinforcement used in these 75 walls included both geotextiles and geogrids. There were a variety of wall facing types, including modular concrete blocks, full height pre-cast panels, vegetated, metal mesh, used tires and timber lagging, etc.

The soil types in the reinforced zone of the walls were not well documented, particularly for the international cases. In general, the soil types can be categorized into three groups: silty sand with clay, sand, and gravel. Furthermore, the percentage of soil that is finer than 0.075 mm (# 200 Sieve) is also noted. The highest "fines" content were greater than 50% in domestic cases 34 through 44.

In these 75 cases, the performance of 23 walls were not acceptable, due either to collapse or excessive deformation of the walls. Fifteen of the failure cases occurred in the U.S. and eight in other countries. Table T-4 lists the unacceptable wall performance cases, and includes a description of the reinforced fill soil, description of the failure and the possible cause of the failure. Table T-5 summarizes the reinforced fill soil properties of the 23 poor-performing cases.

The information in Table 2-7 indicates that water pressure in or behind the reinforced fill was the major cause for the excessive deformation or collapse of the walls. In most of the unacceptable cases, silty sand and clay soils have contributed to the problems in the stability of the walls.

The characterization of the soil properties was limited to the friction angle of the soil and density in some cases. Regarding the field test procedures for the reinforced fill soil, no information was found. Only a few cases provided the compaction method used during the placement of the reinforced fill soil.

2.1.3 Well Instrumented MSE Walls With Marginal Reinforced Fill

Two well instrumented test walls constructed with marginal reinforced fill were identified from the Task 1 literature search. The first was the Algonquin test wall constructed in 1988, and described in Chapter 2.1.1 and 2.1.2. Non-plastic silt (90%<0.075mm) was used to construct the reinforced fill zone. The second was the Louisiana Transportation Research Center (LTRC) test wall constructed in 1998, using silty clay soils as the reinforced fill.

The LTRC test wall was constructed at the LTRC Pavement Research Facility. It consisted of a 20 ft. high vertical wall with modular block facing. It was constructed using medium plastic (PI = 15) silty clay soils for the reinforced fill. The wall was reinforced with various types of geogrids. Details of the test wall are presented in Figure F-5 (See Figure Tab). The test wall was constructed to evaluate the behavior of MSE walls constructed with silty clay soils, through comparison of predicted performance and field measurements. The results were published by Farrag et al, 2004.

The instrumentation results from these projects will be compared to the results of the full-scale test walls constructed for this study, as discussed in Chapter 4.

2.2 SURVEY OF CURRENT PRACTICE FOR DESIGN, SPECIFICATION AND CONSTRUCTION OF MSE WALL REINFORCED FILL

A survey questionnaire (See Attachment C) was developed to determine current design and construction practice for MSE wall reinforced fill (reinforced fill). The respondents were informed that the survey was for permanent MSE walls for typical highway applications (maximum wall height of about 7m (23 ft.)). The survey was conducted via email, with the survey questionnaire attached in MS-Excel format.

Respondents were asked to provide answers to approximately 30 questions grouped under the following general categories:

1. MSE reinforced fill type.
2. MSE reinforced fill properties.
3. Use of “high fines” and/or “high plasticity” MSE reinforced fill, as opposed to “standard” reinforced fill.
4. Economic Implications of using “high fines” and/or “high plasticity” reinforced fill.
5. Unsatisfactory performance using “high fines” and or ”high plasticity” reinforced fill.
6. Enhancement of “high fines” and or “high plasticity” reinforced fill.
7. Special drainage provisions with “high fines” and or “high plasticity” reinforced fill.
8. Additional comments.

The questionnaire was sent to each state transportation agency, District of Columbia (D.C.) and Puerto Rico (52 total) and to industry (the National Concrete Masonry Association [NCMA]; the Association for Metallically Stabilized Earth [AMSE]). Responses were received from 49 state transportation agencies and the NCMA (including individual responses from 3 NCMA member firms). The survey was not expanded to transportation agencies outside the U.S., since it is highly unlikely that international practices would influence current AASHTO requirements.

The following sections of this chapter discuss the survey responses and present our conclusions from the survey. These conclusions, taken as a whole, reflect the current state of the practice with regards to the design and construction of reinforced fill for MSE walls. Attachment D presents the responses from state transportation agencies to questions 1c. ("Are there special requirements within 3 feet of the face of the wall?"), 1e ("What level of QA/QC is required during placement of the reinforced fill?"), and 8 ("Additional comments").

2.2.1 U.S. State Transportation Agency Responses

MSE Reinforced fill Type

The first category of the questionnaire was focused on identifying the type of soils used for the reinforced fill of MSE walls, and quality assurance/quality control procedures that are used in the placement of the reinforced fill. Related questions addressed specifications and compaction requirements, and special requirements for reinforced fill within 3 feet of the face of the wall.

Question 1a - MSE Reinforced fill Specification. Thirty-two (32) of the responding states have modified the AASHTO specifications into a more specific state specification. Fifteen (15) states reference the AASHTO specifications directly.

Six states indicated that they did not allow the use of geosynthetic reinforcement, and do not have any specifications for geosynthetic reinforcement. One state (North Dakota) indicated that they did not allow metallic reinforcement, but did allow geosynthetic reinforcement. Unless noted otherwise, the following sections summarizing the survey responses refer to MSE reinforced fill with metallic reinforcement.

Question 1b - Soil Classification System Used. Thirty-seven (37) States provided responses to this question. The responses are summarized as follow:

- 12 States use their own classification system
- 14 States use the AASHTO classification system
- 10 States use the Unified Soil Classification System
- 1 State uses the Soil Conservation Services system.

Question 1c - Special Reinforced fill requirements within 3 feet of the wall. Nine states specifically indicated that special, coarse, free-draining material is required in this zone, with the width of the zone varying from 12 in. to 2 ft. Several states indicated that free-draining material was only required for MSE walls in which geosynthetic reinforcement is used. Thirty states indicated that the size of the compaction equipment that is allowed to operate within this zone is restricted to "light" or hand-guided equipment, in order to reduce the likelihood of face distortion that could occur with usual, heavy compaction equipment. Several states indicated that the lift thickness is reduced in this zone, so that the required reinforced fill density can be achieved with the lighter weight compaction equipment. One state reduces the reinforced fill density requirement in this zone.

Question 1d – Placement and Compaction Requirements for MSE Reinforced fill. Forty-four (44) States provided responses to this question.

Compaction Requirements for the Reinforced Fill Zone. Forty-four (44) States provided responses to this question. The responses are summarized in Figure 2-6.

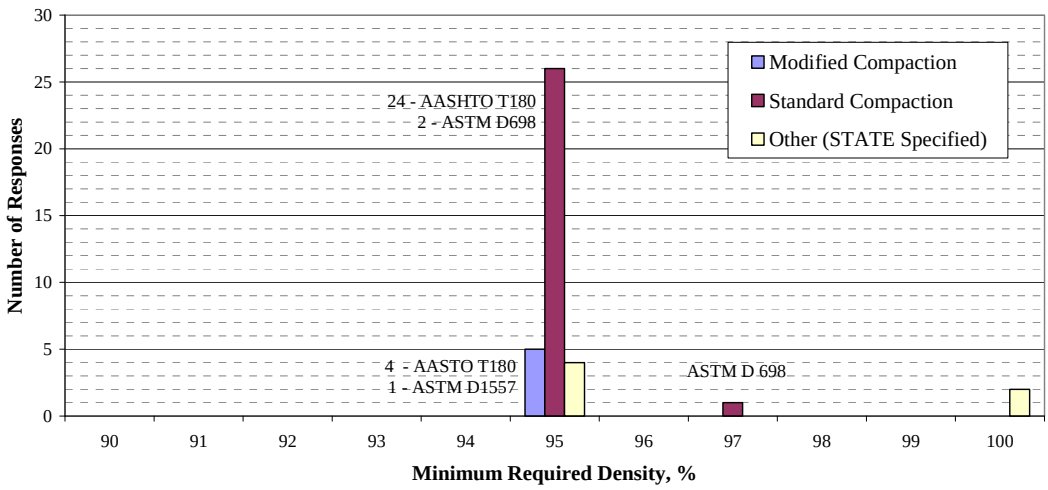

Figure 2-6 Compaction requirements in the reinforced zone from State responses to survey.

Thirty-seven (37) of the respondents require that the soil in the reinforced fill zone must be compacted to a minimum of 95% of the maximum dry density determined by AASHTO T99 or by ASTM D 698. This is commonly referred to as Standard Proctor density. Only 4 States specify a greater compactive effort, i.e. greater than 95% of Standard Proctor or specify the Modified Proctor test methods (AASHTO T180 or ASTM D1557). Six (6) of the respondents indicated that they specify compaction requirements using a method that is a State standard.

Placement Moisture Content of Reinforced Fill. Twenty-nine (29) States provided information about specifications for the placement moisture content of the soil in the reinforced fill zone. The range of moisture contents specified is show in Figure 2-7.

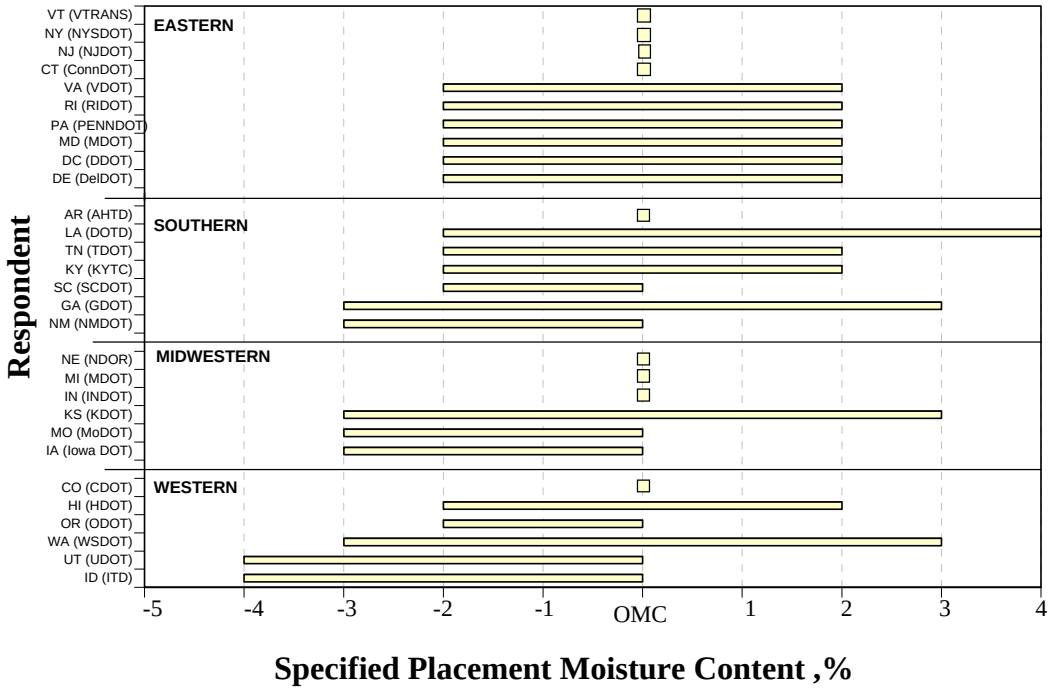

Figure 2-7 – Summary of specified placement moisture content of reinforced fill from State responses to survey

Maximum Lift Thickness of reinforced Fill. Forty-six **(**46) States provided information on the maximum lift thickness specified for the reinforced fill zone. The responses are summarized in Figure 2-8.

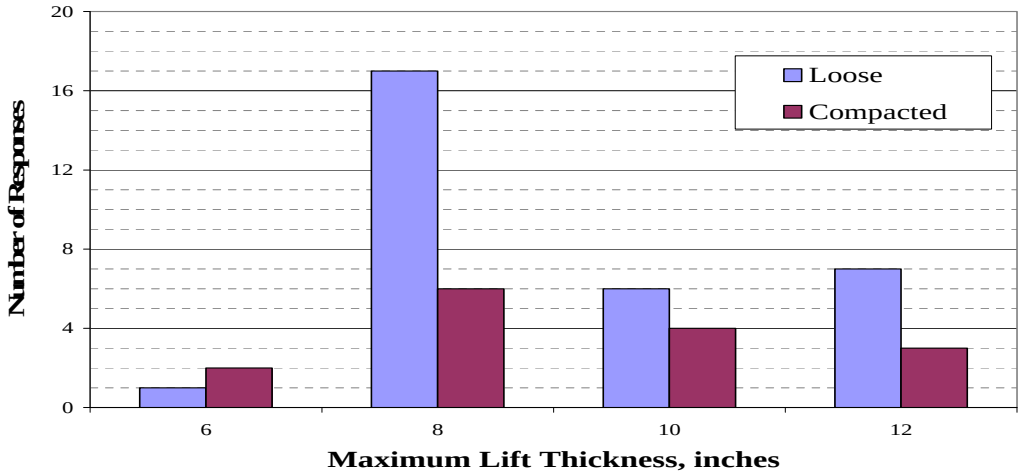

Figure 2-8 – Summary of maximum lift thickness specified in the reinforced fill zone from State responses to survey

Question 1e - QA/QC during placement of the reinforced fill. The level of QA/QC varies considerably agency to agency. A compilation of the actual responses from the state transportation agencies is contained in Attachment D. Typically the contractor performs the Quality Control, and the state or engineer performs the Quality Assurance. Generally, the material source is tested to verify that the material meets the gradation requirements, and field density tests, by nuclear density method) are performed at schedules determined by each agency.

MSE Reinforced fill Properties

This section of the questionnaire focused on required properties of the MSE reinforced fill (e.g. particle size, plasticity index, electro-chemical, and strength) for both metallic and geosynthetic reinforcement.

Question 2a - Particle Size Distribution. Figure 2-9 presents the acceptable upper limit (i.e. material cannot be finer) of the gradation of materials acceptable for use as reinforced fill by the various state transportation agencies (for metallic reinforcement). For comparison purposes, the upper limit allowed by AASHTO is depicted by the dashed bold line in the figure.

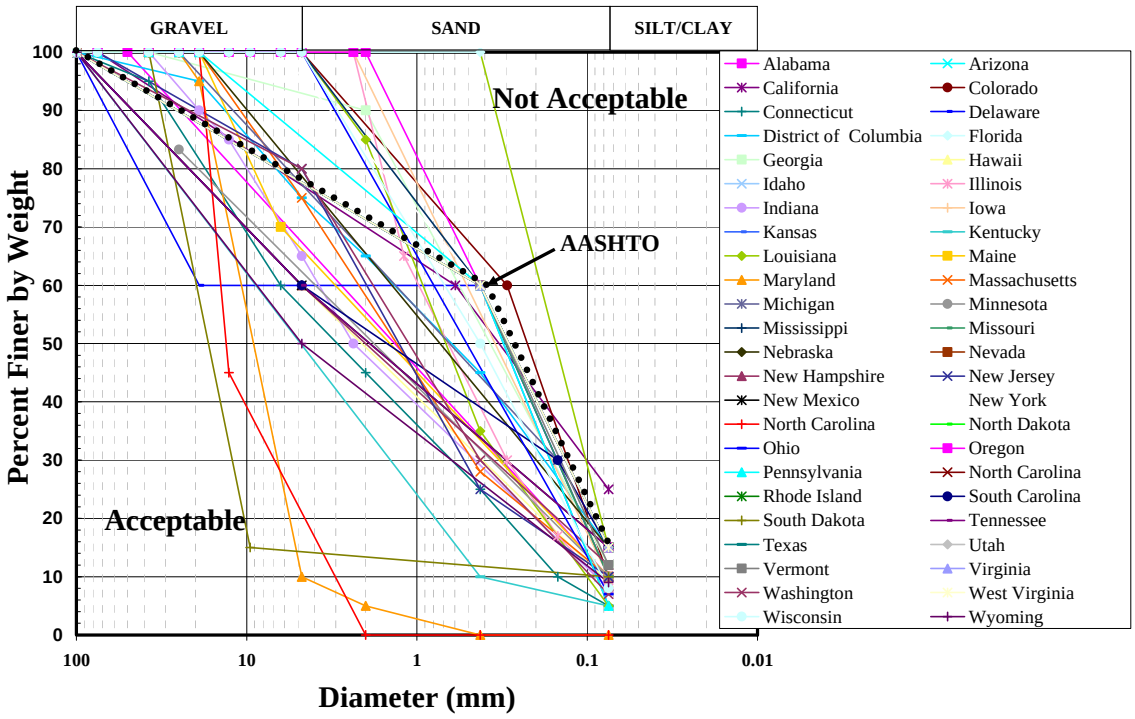

Figure 2-9 Acceptable "Upper" Gradation Limit for Reinforced Fill from State responses to the survey

With the exception of California (CALTRANS) and Arkansas (AHTD), all responding states limit the material passing the #200 sieve (< 0.075 mm) to no more than 15%, which conforms to AASHTO requirements. At present, CALTRANS allows up to 25% passing the #200 sieve. CALTRANS indicated, however, that they are experiencing an increasing number of construction related problems associated with the reinforced fill that is allowed by their specifications. They are currently working on several specification changes, including restricting the amount of soil passing the #200 sieve to no more than 15%. AHTD indicated that they have allowed the use of materials with a high fines content (i.e. greater than 25%, but generally less than 35%, passing the

#200 sieve). These soils, however, have a high internal angle of friction. They do not allow a material with high plasticity.

Table T-6 (Attachment B) summarizes the gradation limits for reinforced fill specified by each state transportation agency.

All responding states, with the exception of Maine, do not have different requirements for placing reinforced fill in below freezing conditions. For winter conditions, Maine requires the use of crushed stone, with a maximum particle size of 2.5 inches and no more than 5% less than ¾ inch, for MSE walls with metallic reinforcement.

Question 2a - Maximum Plasticity Index. Of thirty-five (35) States responding to this question, 24 indicated that they specify a maximum plasticity index (PI) of 6%. This value is in accordance with that recommended by AASHTO. Five (5) States specify a PI less than 6%, and 6 States allow a PI greater than 6% (including two States that allow a maximum PI of 20%). Table T-6 summarizes the maximum PI allowed by each state transportation agency.

Question 2b – Electro-chemical. Thirty-seven (37) states responded to this question, and the responses are summarized as follows for metallic reinforcement;

	AASHTO	
■ Resistivity	≥3000 Ω cm.	32 States
■ pH	5.0 to 10.0	29 States
■ Sulfate content	≤200 ppm.	35 States
■ Chloride content	≤100 ppm	35 States

Resistivity. Only one state specifies a resistivity lower (≤1500 Ω cm.) than AASHTO, while three states specify a value higher (≥5000Ω cm. min).

pH. There are some slight variations of pH specifications around the AASHTO minimum and maximum values. Figure 2-10 shows the variation in specified minimum and maximum pH from the responses to the questionnaire.

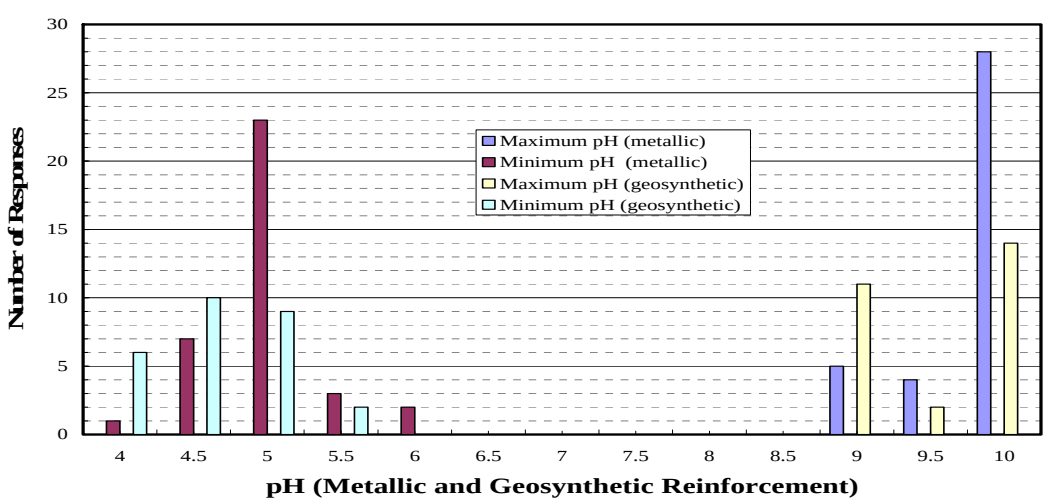

Figure 2-10 – pH requirements for reinforced fill from State responses to survey

Sulfate and Chloride Content. Only two states allow a Sulfate and Chloride content higher than those recommended by AASHTO. One State responded with a value of ≤1000 ppm for Sulfate and ≤200 for chloride, and another State responded with ≤2000 ppm for sulfate and ≤500 ppm for chloride. No states set allowable sulfate or chloride contents lower than those given by AASHTO.

Organic Content. AASHTO recommends that the reinforced fill material be free from organic matter. Thirty(30) of the states surveyed responded, and indicated a maximum limit on organic content ranging from 0 to 2%.

- 6 States indicated that they don't allow any organic matter
- 1 State allowed a maximum organic content of 0.1 %
- 1 state allowed a maximum organic content of 0.5%
- 20 States allow a maximum organic content of 1%
- 2 States allow a maximum organic content of 2%.

Table T-7(Attachment B) summarizes the individual state transportation agency responses relative to acceptable electro-chemical properties and organic content of the reinforced fill.

Question 2c - Mechanical Properties. Forty-one (41) States provided information on the mechanical properties of the reinforced fill materials used in the construction of MSE walls in their respective states. Of these, 34 adhered to the criteria recommended by AASHTO:

friction angle = 34 degrees
cohesion = 0

The other 7 state transportation agencies indicated a friction angle less than 34 degrees and a cohesion of 0. The lowest specified friction angle is 28 degrees.

No state transportation agencies identified a cohesion value. The inference is that only granular soils are to be considered and/or any cohesion of a reinforced fill is not considered in MSE wall design.

The majority of the states that specify a minimum soil strength indicate that the strength should be determined using AASHTO T-236, which is a consolidated drained direct shear test. Six states responded that they do not specify a reinforced fill strength, and use either the AASHTO default values for design or the MSE wall vendor recommendations for reinforced fill strength.

Table T-8 (Attachment B) summarizes the individual state transportation agency responses relative to reinforced fill mechanical properties assumed for design and test methods used to confirm these properties.

Use of "High Fines" and/or "High Plasticity" Reinforced fill

This purpose of this section of the questionnaire was to establish what the respondents considered to be a "standard", "high fines", and "high plasticity" reinforced fill. They were then specifically asked if "high fines" or "high plasticity" materials are allowed in the reinforced soil zone of MSE walls.

Question 3a – Definition of "Standard" Reinforced Fill. All respondents considered a "standard" reinforced fill as that required by their specifications (Refer to Figure 2-9 and Table T-6).

Question 3b – Definition of "High fines" Reinforced Fill. Thirty-eight **(**38) state transportation agencies provided responses indicating their definition of a "high fines" reinforced fill. The responses are summarized in Figure 2-11.

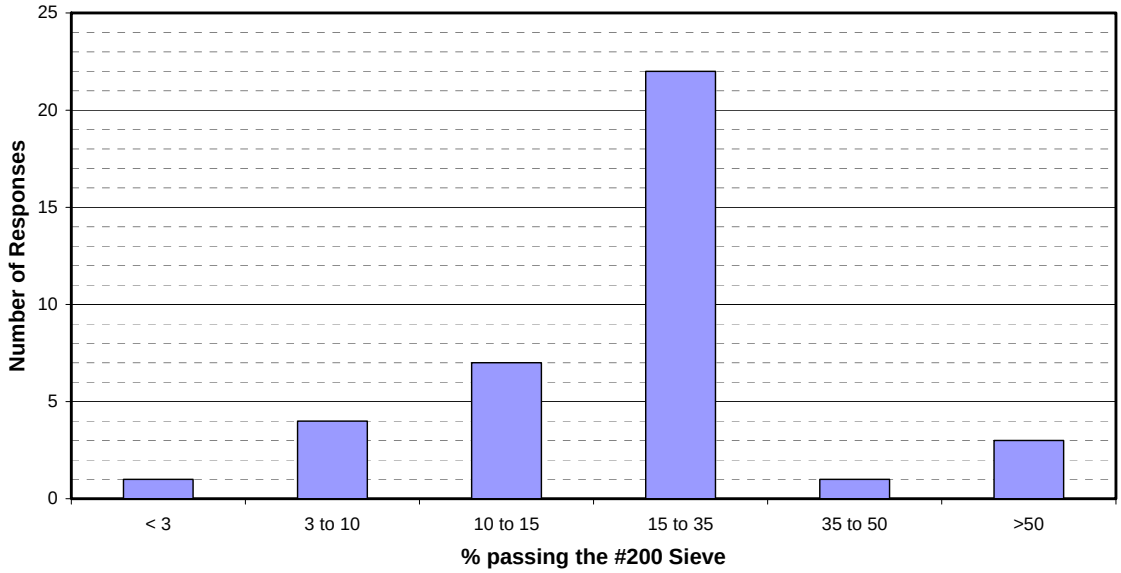

Figure 2-11 – Definition of "high fines" reinforced fill from State responses to survey

Most respondents consider a soil containing "fines" in excess of 15% as a "high fines" reinforced fill.

Question 3c - "High plasticity" Reinforced Fill. Thirty-five **(**35) state transportation agencies provided information as to their definition of a "high plasticity" reinforced fill. The responses are summarized in Figure 2-12.

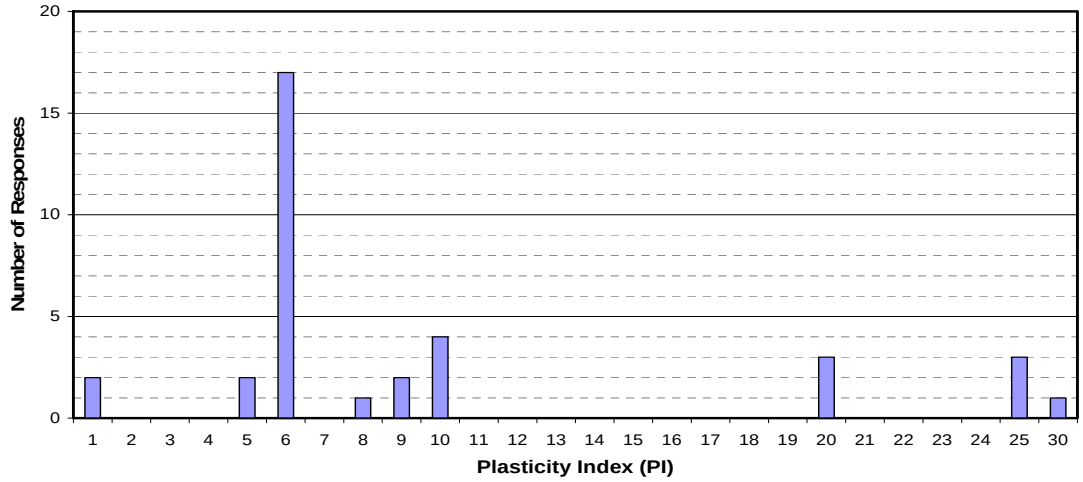

Figure 2-12 – Definition of "high plasticity" reinforced fill from State survey responses

Most respondents consider a soil with a plasticity index greater than 6 as exhibiting "high plasticity".

Question 3d – Do you allow "high fines and/or "high plasticity" reinforced fill?
Forty-five **(**45) state transportation agencies indicated that these materials are not permitted to be used in the reinforced zone of MSE walls. Thus, as instructed in the survey questionnaire, these respondents did not answer Questions 4. through 7., and proceeded to Question 8 (Additional comments).

Only two agencies (Arkansas (AHTD) and California (CALTRANS)) indicated that they allow the use of high fines and/or high plasticity soils in the reinforced zone of MSE walls.

Economic Implications of Using "High fines/High plasticity" Reinforced Fill

Neither AHTD nor CALTRANS provided additional information on the economic implications of using such soils.

Unsatisfactory Performance Using "High fines/High plasticity" Reinforced Fill

Both AHTD and CALTRANS indicated the following types of unsatisfactory MSE wall performance, in some cases, where "high fines" reinforced fill was used:

- Lateral deformation of wall
- Vertical settlement of reinforced fill
- Facing problems
    - Movement/cracking
    - Aesthetics/Staining

Neither agency provided additional information.

Enhancement of "high fines/high plasticity" reinforced fill

Neither AHTD nor CALTRANS provided information on any measures that might be used to enhance the performance of "high fines/high plasticity" reinforced fill.

Special Drainage Provision with "high fines/high plasticity" reinforced fill

AHTD indicated that internal drainage is provided when "high fines/high plasticity" reinforced fill is used. Details were not provided.

Additional Comments

Many state transportation agencies provided additional comments concerning MSE walls and the use of "high fines/ high plasticity" reinforced fill. These comments are included in their entirety in Appendix D. Most of the comments support the continuing use of "low fines/non-plastic" reinforced fill.

2.2.2 U.S. Industry Responses

The following industry responses represent those of the National Concrete Masonry Association (NCMA) as an organization and three individual member firms of that organization (2 independent consultants (IC), 1 MSE wall vendor (WV)).

MSE Reinforced fill Type

Question 1a - MSE Reinforced fill Specification. Three respondents follow AASHTO specifications for MSE walls with metallic reinforcement and NCMA specifications for MSE walls with geosynthetic reinforcement. The fourth respondent (IC) stated that they use their own project specific specifications.

Question 1b - Soil Classification System Used. All industry respondents indicated that they use the USCS classification system.

Question 1c - Special Reinforced fill requirements within 3 feet of the wall. The WV and an IC indicated that a vertical, granular drainage zone is provided immediately behind the wall face. NCMA and an IC indicated that light compaction equipment is used within 3 feet of the back of the wall face.

Question 1d – Placement and Compaction Requirements for MSE Reinforced fill. Industry respondents indicated the following:

Respondent	% Maximum Density/ Test Method	Placement Moisture Content (% +/- of OMC)	Lift Thickness
NCMA	95/ASTM D698	-2 to +2	6 to 8 in.comp.
WV	95/ASTM D698	-2 to +2	8 in. comp.
IC(a)	95/ASTM D698	-3 to +4	8 in. loose
IC(b)	95/ASTM D698	NA	8 in. comp.

Question 1e - QA/QC during placement of the reinforced fill. NCMA does not specify the frequency of field QA/QC testing. The WV and an IC indicated that QA/QC testing is based on Owner requirements. Interestingly, the other IC stated specifically that QA/QC testing should be performed on every other lift of reinforced fill, increasing to every lift on large projects.

MSE Reinforced fill Properties

Question 2a - Particle Size Distribution. Only NCMA and an IC provided a response. Figure 2.13 presents the acceptable upper limit (i.e. material cannot be finer) of the gradation for materials acceptable for use as reinforced fill by the two respondents. The NCMA specification allows up to 35% < 0.075 mm (i.e. passing the #200 sieve). The IC respondent restricts the amount finer than 0.075 mm to a maximum of 10%. Both recommendations are for geosynthetic reinforcement only. For comparison purposes, the upper limit allowed by AASHTO is depicted by the bold-dashed line in the figure.

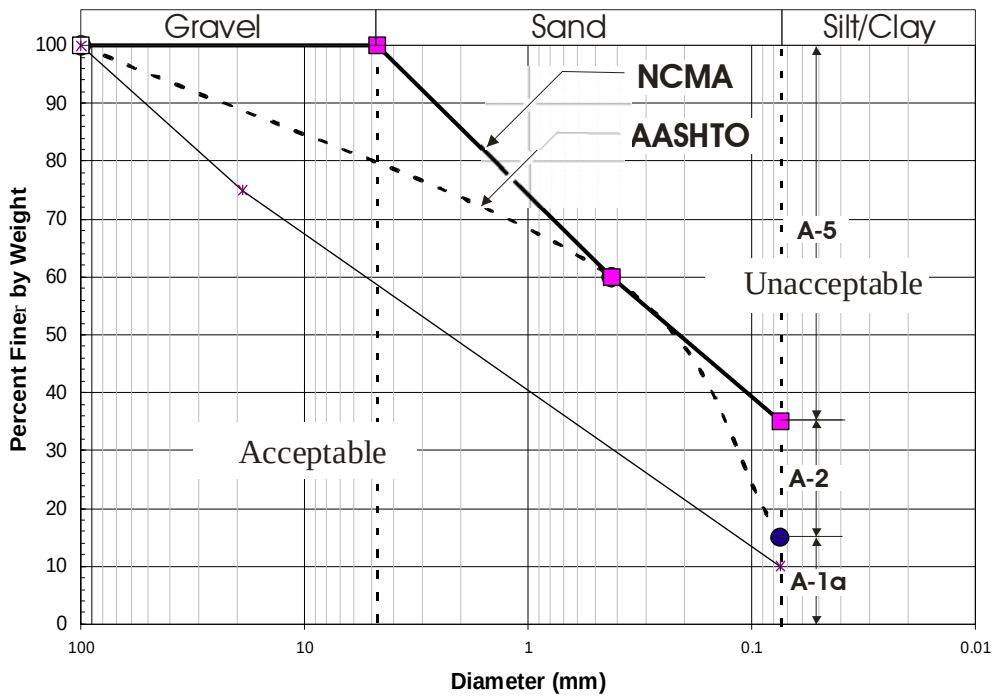

Figure 2-13 – Summary of particle size distribution in the reinforced zone from survey responses

Question 2a - Maximum Plasticity Index. The NCMA recommends allowing soils with a plasticity index of up to 20% to be used in a reinforced fill. One IC agreed with this recommendation, while the other IC would restrict the maximum acceptable plasticity index to 10%. The WV did not provide a response.

Question 2b – Electro-chemical. The NCMA and an IC responded only for geosynthetic reinforcements, and indicated pH limits of 3.0 to 9.0 and 8.2 to 12.0, respectively.

Question 2c - Mechanical Properties. The ICs provided the following information relative to the mechanical properties of the reinforced fill:

- Friction Angle 26 degrees, IC(a)
- Friction Angle 34 degrees, IC(b)

NCMA does not make recommendations relative to the mechanical properties of reinforced fill.

No responses were provided on the use of cohesion in design.

Use of "High Fines" and/or "High Plasticity" Reinforced fill

Question 3a – Definition of "Standard" Reinforced Fill. The responses from private industry regarding the definition of "standard reinforced fill" are extremely varied:

- IC(a) Reinforced fill type is selected on a project by project basis. Clay reinforced fill is often used, but select granular is also used frequently.
- IC(b) Plasticity Index less than 25% and Liquid Limit less than 45.
- NCMA Maximum 35% < 0.075 mm.

Question 3b – Definition of "High fines" Reinforced Fill. The private industry respondents classify "high fines" reinforced soil as follows:

- IC(a) 3 to 10% percent passing the #200 Sieve
- IC(b) 15 to 35% percent passing the #200 Sieve
- NCMA 35 to 55% percent passing the #200 Sieve

Question 3c - "High plasticity" Reinforced Fill. The private industry respondents classify "high plasticity" reinforced soil as follows:

- IC(a) PI > 20%
- IC(b) PI > 25%
- NCMA PI > 20%

Question 3d – Do you allow "high fines and/or "high plasticity" reinforced fill? The private industry respondents indicated the following:

- IC(a) "High fines" soils only.
- IC(b) Both "high fines" and "high plasticity" soils.
- NCMA No. NCMA noted the following: "NCMA guidelines recommend 35% fines; however, NCMA is aware of structures designed with NCMA guidelines using high fines soil."

Economic Implications of using "high fines/plasticity" reinforced fill

No information was provided by respondents on the economic implications of using high fines and/or high plasticity soils for MSE reinforced fill.

Question 4b - What additional testing (during construction) and/or restrictions do you impose when using "high fines" and/or "high plasticity" reinforced fill? NCMA stated the following: "NCMA recommends that a geotechnical engineer be retained to evaluate the time-dependent nature of the proposed reinforced fill, and that additional consideration be made for the inclusion of subsurface drainage collection (i.e. chimney drains at rear of reinforced fill zone, blanket drain at foundation/reinforced fill interface). NCMA recommends a plasticity index < 20 to ensure material classifies as SC, ML or CL per USCS."

Unsatisfactory Performance "high fines/plasticity" reinforced fill

Two respondents indicated the following relative to MSE walls that performed unsatisfactorily with either "high fines" or "high plasticity" soils used for the reinforced fill:

	IC(a)		IC(b)	
Problem	# of Projects with problem	# Projects Built	# of Projects with problem	# Projects Built
Lateral deformation of wall	3	200	5	500
Collapse of wall			1	500
Facing problems:				
Movement/Cracking	1	200	5	500
Aesthetics/Staining	1	20		

NCMA indicated that they are aware of poor MSE wall performance related issues with "high fines" reinforced fill, but they do not maintain statistical records.

Question 5g - Identify Index, Electro-chemical & Mechanical properties of reinforced fill for walls that experienced problems. NCMA indicated that the following factors have contributed to unsatisfactory performance of MSE walls: (1) reinforced fill materials with fines content at or greater than 50% fines, (2) poor or no surface or subsurface water control, resulting in excess poor pressures not considered in the design, (3) instances of inappropriate construction practices, (4) inadequate compaction, (5) improper reinforcement placement, and (6) improper control of soil moisture contents.

Enhancements of "high fines/plasticity" reinforced fill

NCMA recommends that "high fines" soils used for reinforced fill be placed and compacted wet of Optimum Moisture Content.

Special Drainage Provisions with "high fines/plasticity" reinforced fill

Questions 7a thru 7c. The private industry respondents indicated the following relative to special drainage provisions for MSE walls with "high fines/high plasticity" reinforced fill:

	NCMA	IC(a)	IC(b)
External Drainage:			
Back drainage	Yes	No	No
Base drainage	Yes	No	Yes
Internal drainage	Yes	No	Yes
Impervious barrier layer	Yes	No	No

Our interpretation of the responses is as follows. NCMA endorses all forms of drainage control for MSE walls, when "high fines/high plasticity" soils are used for reinforced fill. In fact, they have published a guidance manual (Segmental Retaining Wall Drainage Manual; NCMA Publication Number TR 204), which provides general guidelines for incorporating drainage details and systems into MSE wall design and construction. A "No" response from the ICs indicates that they do not incorporate the drainage feature in their wall designs, while a "Yes" response indicates that the ICs do incorporate the drainage feature in their wall designs.

Additional Comments

The Independent Consultants who responded to the survey questionnaire provided additional comments, which are included in Appendix D.

2.2.3 Summary and Conclusions - Survey Responses

The public sector response to the survey was excellent, with 49 of 52 U.S. transportation agencies submitting completed survey questionnaires. With only a few exceptions, U.S. transportation agencies specify a "low fines", non-plastic material for the reinforced fill zone of MSE walls. Most agencies specify a material that meets the requirements of AASHTO Class A-1a material.

Private industry response was from the National Concrete Masonry Association [NCMA] and several of its individual members. NCMA guidelines allow a higher percentage of fines in the reinforced fill (35% < 0.075 mm), and materials that are classified as SC, ML or CL (Unified Soil Classification System, USCS). NCMA also allows soils with a Plasticity Index as great as 20%.

The following pertinent conclusions can be drawn from the survey responses:

- Most, if not all, U.S. transportation agencies conform to AASHTO requirements regarding material type and properties of reinforced fill for MSE walls.

- The few transportation agencies that have allowed the use of "high fines" soils for reinforced fill have had mixed results (i.e. both acceptable and unacceptable MSE wall performance).

- The level of QA/QC provided during construction of MSE walls on public transportation projects varies considerably agency by agency.

- It is clear that, for U.S. transportation agencies to adopt the use of "higher" fines soils in reinforced fills in the future, the properties of "high fines" reinforced soils and associated design/construction controls that give acceptable performance must be demonstrated and clearly defined.

- The use of "high fines" and/or "high plasticity" soils for reinforced fill is much more common in the private sector. However, presently, there is no rational basis (other than cost savings) for including such soils in the reinforced fill zone of MSE walls.

2.3 REINFORCED FILL PROPERTIES THAT IMPACT DESIGN AND PERFORMANCE OF MSE WALLS

2.3.1 Introduction

The results of the literature search (Task 1) and the survey (Task 2) indicate that MSE walls on transportation projects are generally conservatively designed, with "low fines" reinforced soils. Private MSE walls are less conservatively designed, and use a variety of reinforced soils (NCMA allows for 35% < 0.075mm).

It is clear from the literature (Tables T-4 and T-5) that reinforced soil consisting of fine-grained soils (either "high" fines or "high" plasticity) and pore pressure resulting from lack of drainage in the reinforced zone were the principle reasons for serviceability problems (excessive deformation) or failure (collapse). As indicated later, the use of fine-grained reinforced fill soil will invariably result in reduced MSE wall costs. However, it can also lead to excessive wall deformations and/or failure.

We believe that, with proper design and selection of soil parameters, "high fines" soils can be safely used as reinforced fill and decrease the cost of MSE wall construction. Experience indicates that clay soils used as reinforced fill cause too many construction difficulties, thus, offsetting any economy of their use. For example, it can be extremely difficult to control placement moisture content in order to achieve required in-place density and desired engineering properties. The need to moisture condition these soils can impact construction schedules , and the effects of even brief periods of inclement weather can lead to further construction delays.

Therefore, the following section of this report will focus on "high fines" soil, and examine the issues that must be addressed in designing MSE walls using such soils.

2.3.2 Important Parameters for Reinforced Fill When Using "High Fines" Soils

When considering the use of "high fines" reinforced fill, the following material properties have an important effect on the design and performance of MSE walls:

Hydraulic Properties

Permeability of the reinforced fill is an important operational property. As the percentage of fines of the reinforced fill increases, its permeability decreases. Table 2-4 illustrates the substantial decrease in permeability with only a modest increase in fines. With as little as 7% passing the No. 100 Sieve, the permeability of the original washed sand decreased by a factor of 100 to 400.

Table 2-4 – Effect of Fines on Permeability of a Washed Filter Aggregate
(ref. Cedergren, H.R., Seepage, Drainage and Flow Nets, 1989)

% Passing No. 100 Sieve (<0.15mm)	Coef. of Permeability or Hydraulic Conductivity (cm/sec)
0	0.11 to 0.03
2	0.04 to 0.004
4	0.02 to 0.0007
6	0.007 to 0.0002
7	0.001 to 0.00007

Wetting of "high fines" MSE reinforced fill from infiltrating groundwater, rainfall or other sources of water (e.g. snow melt, etc.) can allow pore water pressures to develop within the reinforced fill zone. Surface water drainage and drainage from the retained soil zone are also of concern with respect to development of pore water pressures behind or within the reinforced fill zone. Positive pore water pressures affect the stability of a MSE wall in two important ways. Positive pore water pressures produce a horizontal seepage force on the reinforced fill that decreases stability. Positive pore water pressure also reduces the shear resistance of the reinforced fill soil.

To illustrate this point, we refer to the work of one of our team members, Dr. Robert M. Koerner (Koerner and Soong, GRI Report #24, 1999). Figure 10 of the Koerner report is reproduced as Figure 2-14.

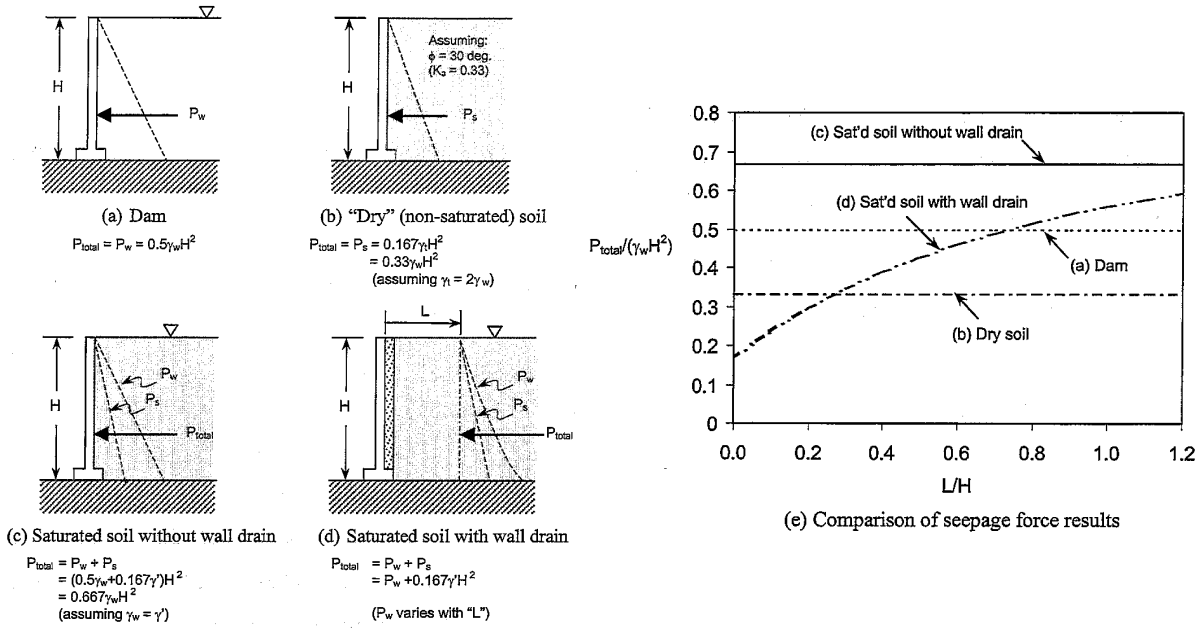

Figure 2-14 Relative significance of seepage pressures leading to seepage forces (Koerner et al. 1999)

Several different conditions are depicted in Figure 2-14 to demonstrate the relative magnitude of seepage pressures versus soil pressures:

(a) Hypothetical wall serving as a dam – The horizontal force on the wall is a direct function of the water force, and is constant when plotted against L/H (where L is the distance behind the back of the wall.

(b) Same hypothetical wall resisting a soil force without water forces (e.g. wall with a "drainable" reinforced soil zone) – Assuming a cohesionless reinforced fill soil of $\phi = 30$ degrees and the total dry weight is twice the unit weight of water, the non-dimensionalized total force is plotted as the "dry soil" line on Figure 2-14(e). This is the usual assumption made in designing MSE walls.

(c) Saturated reinforced fill condition (soil and water pressure exist behind wall face) – As noted on Figure 2-14(e), the total force is twice that of dry soil alone.

(d) Saturated soil with wall drain – As Koerner's work demonstrates, superimposing the water and soil forces results in the curved line in Figure 2-14(e) – See Appendix C for a further explanation. At L/H = 0.7, the total pressure is the same as the hydraulic dam condition. A value of L/H equal to 0.7 is the minimum reinforcement length specified by AASHTO for MSE walls.

As can be seen in Fig. 2-14(e), without drainage, the total force against the wall can be twice that of a properly drained reinforced fill soil.

Further, Koerner presents seepage studies for MSE walls to illustrate the mobilization of seepage forces in "high fines" (low permeability) reinforced fill. This work is included as Appendix C, with the permission of Dr. Koerner. As indicated, the effect on horizontal seepage force of the following parameters was investigated:

- location behind wall face,
- backslope inclination,
- surface cracking (this will be discussed later under "Environmental Effects"),
- permeability ratio (i.e. ratio of reinforced fill permeability to retained soil permeability),
- aniosotropic permeability (Reinforced fill zone),
- tapered reinforcement length, and
- inclination within the reinforced zone.

Of particular interest is the effect of permeability of the reinforced fill zone. Figure 2-15 depicts the effect of permeability ratio ($k_{rein\ (reinforced\ fill)}/k_{ret\ (retained\ soil)}$) on the non-dimensionalized seepage force ($P_w/\gamma_w H^{2)}$. Also shown is the effect of the backslope inclination, since it is also of significance.

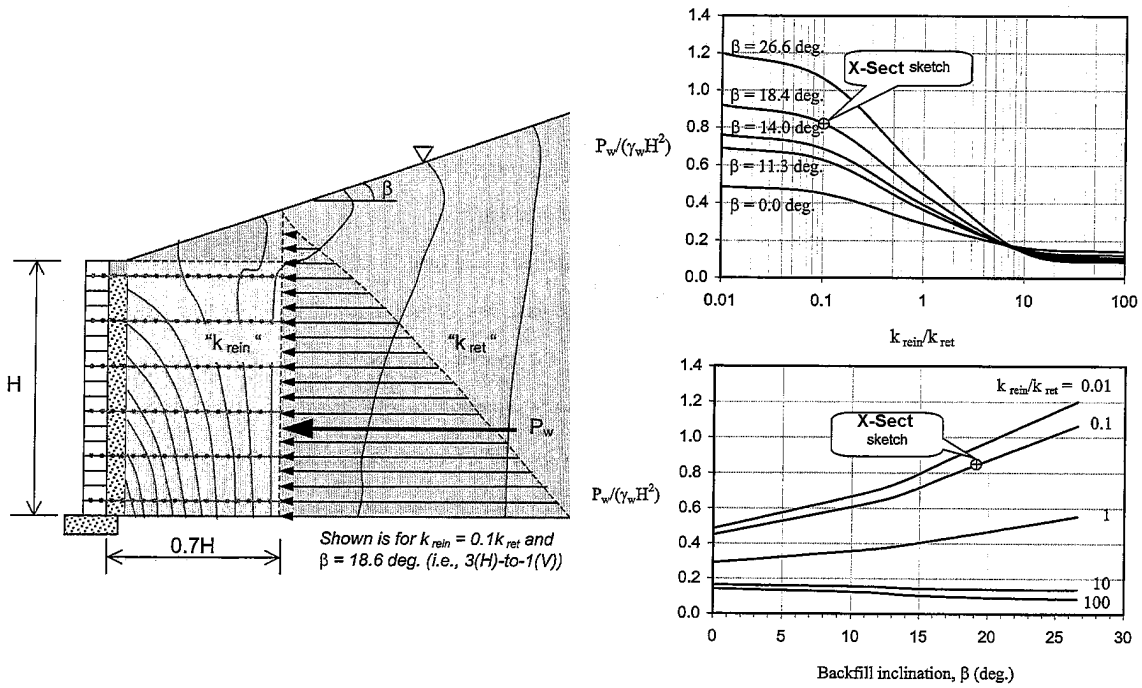

Figure 2-15 Effect of permeability ratio on seepage force (Koerner et al. 1999)

The trends in the curves indicate the following:

- As the permeability of the reinforced fill zone decreases, the ratio k_{rein}/k_{ret} decreases, and the seepage force increases.
- For high permeability of the reinforced fill zone, the ratio k_{rein}/k_{ret} increases, and the horizontal seepage force decreases. The horizontal seepage force becomes essentially constant at a k_{rein}/k_{ret} ratio greater than about 10.
- As backslope inclination increases, the trends increase dramatically, especially when the permeability of the reinforced fill zone is very low with respect to the retained soil zone.

In summary, when using "high fines" (low permeability) soil as reinforced fill, it is imperative that either any possible water be kept out of the reinforced zone by collecting, and discharging it away from the reinforced zone, or else pore pressures must be included in the analysis and design of MSE walls.

Mechanical Properties (φ)

The internal frictional strength of the reinforced fill is an important property influencing the maximum tensile force in the reinforcing layers; although the maximum tensile force is also related to the type of reinforcement in the MSE mass (i.e. extensible or inextensible). Figure 2-16 shows the variation in normalized reinforcement tensile force versus angle of internal friction (φ) of the reinforced fill for a MSE wall with a horizontal reinforced fill surface, for both extensible and inextensible reinforcement. For every one degree reduction in frictional strength of the reinforced fill, there is an approximately 5 percent increase in the reinforcement tensile force. Thus, with "high fines" soils, the required reinforcement strength will typically be greater than for a "low fines", granular soil.

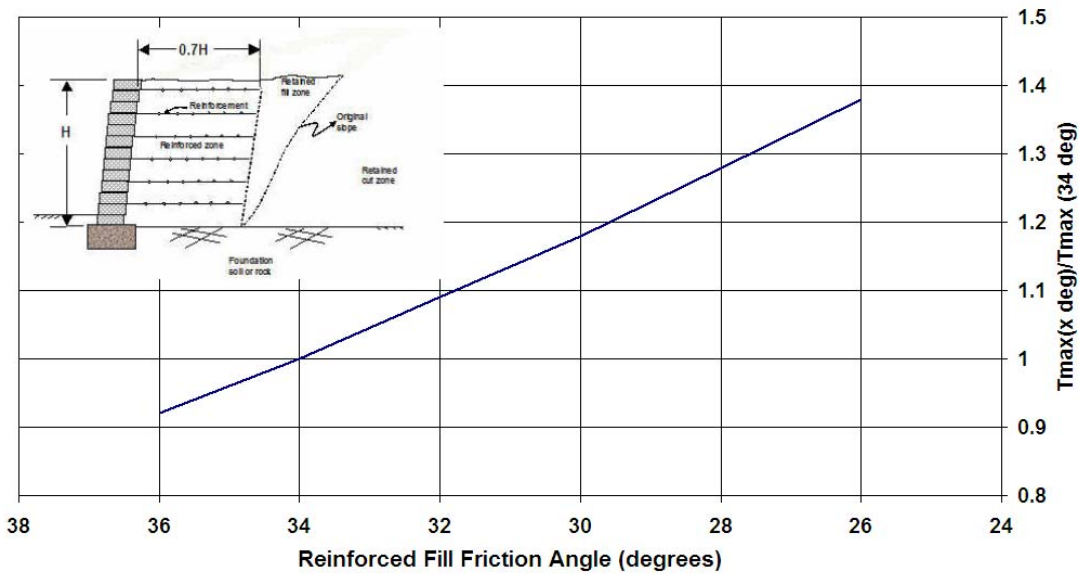

Figure 2-16 Strength of Reinforced Fill (ϕ) versus Normalized Reinforcement Tensile Force

Deformation Properties

One of the more serious issues with "high fines" reinforced fill is the anticipated increase in vertical and horizontal deformation, both during and after construction. During construction, the elastic modulus of the MSE reinforced fill is an important property value. "High fines" soils tend to deform more than clean, granular soils, and the deformation may be time dependent. Therefore, the compressibility characteristics of these soils may have to be evaluated, depending upon the nature of the MSE structure.

Increased deformation creates several issues that must be addressed in design including:

- Maintaining wall alignment during and after construction.
- Potential deformation of supported structures and utilities.
- Downdrag on the back of facing units and facing connections.
- Increased potential for tension cracks.

With regard to wall alignment, greater care will be required during construction to meet line and grade requirements. The magnitude of post construction movement should be estimated and provided to the designers of supported facilities. Where tolerances do not allow for post construction deformation (i.e., bridge abutments), high fines reinforced fill should not be used.

The surface of MSE structures using high fines fill should be monitored after construction to confirm that deformation has subsided before construction of supported structures. Downdrag issues may require greater overfilling at connections, beveled and/or rounded edges on the back of modular blocks and stress relief mechanisms in rigid connections.

The most serious issue is related to post construction tension cracks. The relatively brittle nature of compacted "high fines" soil makes it prone to tension cracks that tend to form at the back of the reinforcement zone, as settlement occurs in the reinforced fill. The corresponding low permeability of high fines fill allows for pore pressure to develop from surface water entering the crack(s). A number of wall failures have been attributed to this development of pore pressure in tension cracks. To preclude the development of tension cracks, upper level reinforcements may have to be extended. A chimney drain at the back of the reinforced soil mass could be used to provide pore pressure relief, and the ground surface should be sloped away from the wall face, or otherwise treated, to reduce the availability of surface water to the reinforced zone.

Environmental Effects

Environmental effects become an important consideration relative to the performance of MSE walls containing "high fines" reinforced fill. These include shrink/swell potential, frost susceptibility, hydro-compaction potential, and susceptibility to surface tension cracks.

Alternate wetting and drying of "high fines" fill can cause shrink/swell to occur as well as the formation of micro-cracks. Under alternate cycles of wetting and drying, these micro-cracks begin to grow and spread throughout the reinforced mass. Pore water pressure can result from water infiltration into the reinforced zone.

Frost susceptibility of soils increases with increasing "fines" content. In cold climates, freeze-thaw effects can cause a volume increase (i.e., increase in lateral movement) during freeze and strength reduction during thaw. Gravels and sands with greater than 20% to 15% fines and silts are categorized as high frost susceptible soils by the US Army Corps of Engineers.

Wetting of "high fines" soil compacted dry of optimum moisture from infiltration of groundwater or surface water can cause swelling and strength reduction, which in turn can result in increased movements/deformations (hydro-compaction). This is particularly true for soils that are placed and compacted well dry of Optimum Moisture Content.

An increase in electro chemical properties is also associated with increased fines in the soil. The fines often contain salts and, thus an increase in fines often corresponds to a higher potential for corrosion. A careful evaluation of sodium and chloride content, along with resistivity, is required.

2.4 CHARACTERIZATION OF REINFORCED FILL

2.4.1 Introduction

Design of MSE structures to AASHTO standards requires type A-1-a materials. These free draining materials typically have a relatively high friction angle for strength, high permeability and high stiffness. Strength of A-1-a materials are generally well above 34 degrees. Strength of this value or higher has little practical influence on the design of a MSE structure. These frictional materials also provide relatively high friction between the soil and the reinforcing elements. Hence there is little incentive to test the A-1-a materials for strength.

MSE wall design methodology assumes drained conditions, and the field conditions generally provide drained conditions. Thus, permeability generally need not be measured, as long as the material is Type A-1-a. Current design methodologies are also based on the earth pressure method or the limit equilibrium method, neither of which requires a value for stiffness. A-1-a materials generally compact well in all types of weather conditions. Only moderate compactive effort is required to obtain acceptable performance. Consequently, the only justifiable testing for the current AASHTO standard for MSE structures is that necessary to ensure that the reinforced fill material is type A-1-a and that adequate compactive effort is applied to the reinforced fill to prevent loose zones.

The type can be determined with a few sieve analyses for the entire project, together with visual inspection of the reinforced fill materials in the field. The aggregate supplier may provide the sieve analyses from their standard quality control program. Adequate compaction, to prevent loose zones, can be achieved by observing that controlled lifts of reinforced fill material receive a minimum number of passes with appropriate compaction equipment. Typically the required number of passes is determined in the field by an experienced person gauging the degree of compaction obtained with the equipment being used. Consequently, testing requirements for reinforced fill in current MSE structures are minimal.

Marginal reinforced fill soils with up to 35% fines and a PI of up to 10% have lower strength, lower permeability and lower stiffness than A-1-a materials. These lower values play a significant role in the design of MSE structures. Lower values of strength will increase the amount of required reinforcement . Lower values of permeability will necessitate additional drainage provisions. Lower stiffness may result in unacceptable deformations of the wall face or excessive forces on the reinforcing connections to the wall facing system. These values also depend on the degree of compaction achieved during construction, which will be affected by the moisture content of the soil. Moisture content is highly dependent on weather conditions, nature of the borrow materials and placement procedures used by the contractor. More laboratory and field testing will generally be required for all marginal reinforced fill soils, as compared to that required for A-1-a materials.

2.4.2 Potential Testing for Marginal Reinforced Fill Soils

Design practice for MSE structures using marginal reinforced fill soils will most likely employ drained parameters for the reinforced fill, and the incorporation of drainage features to ensure that positive pore water pressures do not develop in the reinforced fill. Consequently parameters for strength, stiffness and permeability should be for drained conditions and stress conditions that cover the range of stresses within the reinforced fill. Usually design practice will typically not require a specific value for the reinforced fill stiffness, only that the reinforced fill be sufficiently compacted to minimize deformations.

We anticipate the design methodology for MSE structures with marginal reinforced fill soils will require the permeability of the reinforced fill to be at least 100 times larger than that of the retained soil, when using a single drainage pipe at the back of the lowest reinforcement layer. If the reinforced fill soil is less than 100 times the permeability of the retained soil, an aggregate or geocomposite back drain will be required at the back of all reinforcement layers. Consequently we anticipate the need to determine the permeability of the marginal reinforced fill and of the most permeable portions of the retained soil.

Classification tests to determine percent fines (AASHTO T88 or T311; ASTM D422) and Plasticity Index (AASHTO T90; ASTM D 4318) should be completed on several representative

samples of the potential source material prior to using it as reinforced fill. The numbers and locations of tests should be determined by someone qualified to classify soils for engineering purposes using AASHTO M145; ASTM D2487).

Drained strength for marginal reinforced fill soils can be obtained from direct shear tests using AASHTO T236 (ASTM D3080). This test has been shown to produce a reliable measurement of friction angle of the soil, when performed in accordance with the standard. Each soil sample should be tested at three different normal stresses, approximately equal to ¼, ½ and 1 times the vertical stress at the elevation of the lowest reinforcement layer. Samples for direct shear testing must consist of the proposed reinforced fill material with the highest fines content and highest PI. Each source should be tested with a minimum of one direct shear test series.

Interface or pull out shear for the reinforcements will depend on the type of reinforcement and type of reinforced fill soil. Past practice has usually been to use a conservative fraction of the reinforced fill's friction angle as the available friction along the reinforcement interface. The standard values recommended for this approach may not apply to marginal reinforced fill soils. Recommendations for determining an appropriate shear strength for the reinforced fill/reinforcing interface will be determined in Phase II of this study, following the completion of laboratory testing on representative samples of the marginal reinforced fill soil. In addition, research on this issue has been and is being conducted by others. The results of Phase II testing will compared with information from other researchers.

It may be possible for approximate values of permeability of marginal reinforced fill soils to be estimated using the soil classification, where the reinforced fill is at least 1000 times the permeability of the retained soil. For smaller differences in permeability, some laboratory testing may be necessary, to determine whether the design requirement of $k_{rein}/k_{ret} > 100$ for no back drainage is met.

Testing to measure stiffness of marginal reinforced fill soils will likely not be required, except for special cases where advanced numerical analysis is being used. For such cases, the designer making the analysis must be sufficiently knowledgeable of soil behavior to recommend the most appropriate test method and test procedures.

2.5 GUIDELINES FOR MSE REINFORCED FILL

The survey of current practice reviewed in Chapter 2.2 indicates that a vast majority of agencies are currently using low fines (< 15% finer than 0.075 mm) reinforced fill, and have reported very good success. However, the geosynthetics industry routinely permits higher fines (routinely up to 35 % finer than 0.075 mm) for private sector projects, reporting a number of successful projects. The steel reinforcement industry allows higher fines in the manufacturers' specifications for private sector work (routinely up to 25% fines). However, the industry did report a greater number of problems with walls using "high fines" reinforced fill than state agencies. Also, the literature review of monitored structures, discussed in Section 2.1, indicated a number of problems with structures using high fines reinforced fill. However, each instance of poor performance could be traced to the design property issues noted in Section 2.3. In a majority of the cases, poor performance could be attributed to excessive deformation and/or uncontrolled seepage.

On the basis of this review, it appears that a higher quantity of fines (but fines without high plasticity) could be safely allowed in the reinforced fill, provided the properties of the materials are well defined and controls are established to address the design issues. This, of course, assumes that a strong economic incentive is provided, as discussed in the next section.

2.6 ECONOMIC INCENTIVE FOR RELAXING CURRENT MSE WALL REINFORCED FILL SPECIFICATIONS

A twenty foot high mechanically stabilized wall typically costs $25-$35 per square foot of face to construct with reinforced fill materials meeting AASHTO's standards. The higher cost applies to MSE walls with metallic reinforcement and the lower cost applies to MSE walls with geosynthetic reinforcement. The reinforced fill section represents 35 to 40 % of the total cost of the wall (FHWA BHI-00-043, 2001).

Reinforced fill materials meeting AASHTO requirements cost $10-11 per cubic yard delivered to the site from within a 10-mile radius. Less select soil with fines often reduces that cost by more than 50%. Optimally, if on site soil could be used, excavation and placement typically only costs $1 to $2 per cubic yard. However, at present, where onsite materials exist that could be used as marginal reinforced fills, they must be removed from the work area at an added cost of $2-3 per cubic yard.

A twenty-foot high wall requires approximately 0.6 cubic yards of reinforced fill within the reinforced zone per square foot of wall face. This gives a materials cost for reinforced fill of approximately $6 per square foot of wall face. The cost to remove existing materials that could be marginal reinforced fills is approximately $1.50 per square foot of wall face.

If we can use the onsite soils as reinforced fill, we avoid the $6 cost of offsite granular materials and the $1.50 cost of removing existing materials. However marginal soils will require more upfront design effort, more testing and more onsite screening of materials. They will also require drainage provisions to prevent buildup of pore pressure within the reinforced zone and hydrostatic pressure behind the reinforced fill zone.

The estimated cost of these differences is $2 per square foot of wall face for the wall with marginal reinforced fill. The net change to costs by changing from granular reinforced fill to marginal reinforced fills is estimated to be $5.50 per square foot of wall face. This amount represents a potential saving from using marginal reinforced fill of 15 to 20 percent. Considering the approximate nature of this calculation, the potential savings from replacing AASHTO reinforced fill materials with marginal reinforced fill materials could be in the range of 20 to 30% of current MSE wall costs.

This result assumes that all other costs (i.e. reinforcing, site preparation, mobilization, wall facing) are not changed by switching reinforced fill materials. The result is also directly related to the cost of granular reinforced fill materials. In areas with a shortage of suitable granular materials, the savings could be significantly higher. This result also assumes a comparable level of effort to place and compact the reinforced fill to an acceptable condition, whether granular or marginal. This is a reasonable assumption for marginal reinforced fills that are not too plastic (PI less than 10%). Plastic materials cost more to place and maintain at an acceptable moisture content. Work progress becomes much more dependent on the weather. Plastic reinforced fills also increase the risk of unacceptable performance.

Cost savings for plastic reinforced fills may be much less than the 20 to 30% described above. It is even possible that a MSE wall constructed with a clay reinforced fill could cost more than an A-1-a reinforced fill, if poor weather conditions and bad construction practices prevail.

CHAPTER **3**

NUMERICAL ANALYSIS METHODS USED FOR MSE WALL DESIGN

A wide range of analysis methods has been used since the 1970's to predict the performance of MSE walls. Some of these methods are used in routine design of such walls. The methods can be classified into three categories:

- Methods based on lateral earth pressure theories (e.g., AASHTO uses Rankine and NCMA uses Coulomb).
- Methods based on limit equilibrium theories (e.g., slope stability approach extended to reinforced walls).
- Methods based on continuum mechanics (e.g., finite element method, as in the computer code Plaxis, or finite difference method, as in the computer code FLAC).

3.1 NUMERICAL ANALYSES AND THEIR SUCCESS AT PREDICTING MSE WALL PERFORMANCE

Earth Pressure Theory

The magnitude of lateral earth pressure depends on the horizontal strain that develops within the reinforced soil. In flexible MSE walls (e.g., geosynthetic reinforcement), the movement occurring during construction is sufficient to develop an active state of stress. In stiff MSE walls (e.g., metallic reinforcement), the movement is limited, especially near the top of the wall. Consequently, the reinforced soil does not mobilize its full strength, resulting in higher lateral earth pressures near the top of the wall (1.5 to 2.5 times larger than the active stress) and gradually reaching near active state of stress at about 6 m below the top of the wall.

Once the lateral earth pressures along the height of the wall are known, the reactive force in each reinforcement layer is calculated by multiplying this pressure by the assumed tributary area for each layer. Knowledge of the force in the reinforcement enables one to calculate the length required to resist pullout (and hence allow this force to develop in the reinforcement), as well as the load at the connection with the wall face. These three aspects of stability, assessed using the lateral earth pressure approach, are commonly termed internal stability, enabling one to determine the required long-term strength of the reinforcement, the required connection strength to the facing, and the required embedment length to ensure sufficient resistance to pullout.

External stability in the lateral earth pressure approach considers the reinforced soil as a coherent mass, that behaves like a gravity wall. The reinforcement has to be long enough (i.e., the coherent mass has to be wide enough) to resist direct sliding, have sufficient resistance to overturning, and have an adequate factor of safety against a bearing capacity failure. In these calculations, the resultant of lateral earth pressure exerted by the retained soil is considered.

The final layout and strength of the reinforcement in the lateral earth pressure approach is a synergy of six different analyses, all related by the lateral earth pressure used. This approach is semi-empirical, since the lateral earth pressure coefficients for internal stability have been 'calibrated' by field tests and back calculations. The number of such field tests is limited, and generally have been restricted to simple geometries without pore water pressures. Experience gained in recent years shows that for granular soil and geosynthetic reinforcement, the lateral

earth pressure approach could be overly conservative (i.e., the maximum reinforcement force can be over-predicted by as much as a factor of two). In fact, the lateral earth pressure approach indicates that the required strength increases linearly with depth (for uniform reinforcement spacing), whereas measured field data shows nearly uniform mobilization of reinforcement strength with depth. The required total reinforcement strength is in good agreement; however, the distribution in reality is close to uniform whereas the predicted distribution is linear with depth. Overall, the lateral earth pressure approach produces safe structures, albeit conservative, in terms of reinforcement strength for MSE structures on firm foundations and reinforced fills with no positive pore pressures.

Seepage forces are not included and adequate drainage is implicitly assumed in this method.

Limit Equilibrium Theory

Limit equilibrium analysis has been used successfully to design reinforced steep slopes and embankments with base reinforcement. The stabilizing forces contributed by the reinforcement layers are incorporated into the limiting equilibrium equations, to determine the factor of safety of the reinforced mass. In the case of geosynthetic reinforcement, the predicted force in the reinforcement layers is reasonably conservative. Limit equilibrium can be applied to a wide range of problems (e.g., multi-tiered reinforced walls) and, in general, to non-homogeneous soils. It can include the effects of pore water pressure on stability. The limit equilibrium method with reinforcement is a natural extension of a method widely used in geotechnical engineering. Compared with Earth Pressure Theory, it does not require extensive field "calibration" (i.e. only limited confirmation for the specific type and configuration of reinforcement), and, thus, it can be used to design complex problems.

In limit equilibrium analysis, the common assumption is that all reinforcement layers are equally mobilized. This analysis looks at the overall stability of a potential sliding mass, targeting an acceptable factor of safety. It means that along portions of the failure surface the local safety factor might be smaller while at others it might be larger. Lower local safety factor means that some layers need to contribute more than predicted; i.e., be mobilized more than other layers. While such possibility exists, structures designed based on FHWA guidelines for reinforced soil masses show that this potential problem does not exist. In fact, comparison with sophisticated numerical methods indicates that this assumption results in safe structures, though less conservative that those based on Earth Pressure Theory.

The FHWA considers a reinforced structure to be a slope when the angle is less than 70 degrees from the horizontal; at 70 degrees or higher the structure is considered a wall. This is a somewhat arbitrary definition enabling simple calculations (Method A) to be used in walls, thus avoiding the need for computer-aided design which is essential when using slope stability analysis. The trade-off is, therefore, employing an empirical design method for walls with limitations (e.g., limited to simple geometries, not calibrated to cases where pore water exists).

Proper application of the limit equilibrium approach can address the required connection strength between the reinforcement and facia, as is commonly considered in walls. The other aspects of design used in walls (pullout length, bearing capacity, direct sliding, eccentricity/overturning) may all be analyzed as part of a limit equilibrium analysis. With limited modifications of current limit equilibrium analysis, an MSE wall may be designed safely and economically in one framework without the need for synergistic approach.

Continuum Mechanics

Analysis based on continuum mechanics considers the full constitutive relationships of all materials involved (i.e., the stress-strain-time relationships). It satisfies boundary conditions, produces displacements (unlike Earth Pressure Theory and Limit Equilibrium), and considers local conditions. Compatibility between dissimilar materials is analytically assured. It can represent a problem in the most realistic fashion, and prediction of performance can be quite accurate. Typically it requires a computational effort by a trained analyst. Most importantly, it requires quality input data that are frequently not available in common practice (although not that difficult to obtain). This analysis has been a good research and forensic tool, and is becoming more accepted in design practice.

3.2 LIMITATIONS OF NUMERICAL METHODS

Each of the numerical analysis methods has limitations, which are described below.

Earth Pressure Theory.

- Overly conservative for "geosynthetic" MSE walls.
- Empirically-based, thus is limited to relatively simple geometries against which field "calibration" has been performed.
- Difficult to extrapolate to complex geometries, such as multi-tiered walls.
- Limited to uniform granular soil; difficult to extrapolate to non-ideal reinforced fill soils.
- Cannot consider pore water pressure in the reinforced fill in a straightforward manner.
- Does not determine global stability.
- Downdrag at connections is not evaluated.
- Does not evaluate deformation of the wall structure or soil.

Limit Equilibrium Theory.

- Currently limited by FHWA recommendations to slope inclinations less than 70 degrees; however, this limitation is arbitrary and there is no theoretical reason why it could not be extended to vertical structures.
- Needs modifications so that aspects such as connection load can be assessed inside the limit equilibrium analysis (although this also can be done in an external analysis using the reinforcement forces).
- Needs further verification that uniform mobilization of all reinforcement layers is a valid assumption; otherwise, a refinement of this assumption is needed through field measurements.
- For most problems it can be applied only when using computerized analysis.
- Requires calibration with respect to partial coverage reinforcement (i.e. strips).
- Downdrag at connections is not evaluated.
- Does not consider deformation of the wall structure or soil.

Continuum Mechanics.

- Requires comprehensive characterization of strength and compressibility for all soils, reinforcement, and facing.
- Computationally intensive; however, available computer programs have significantly simplified the analysis.

- Predictions can be non-conservative, requiring careful evaluation of the reliability of input values and appropriate safety and/or resistance factors (unlike Methods A or B, both of which contain inherent conservatism).

3.3 ILLUSTRATIVE EXAMPLE

In order to compare the various numerical analysis methods by way of example, a model MSE wall cross-section was evaluated (See Figure 3-1). The example assumes a modular block face and geosynthetic reinforcement (geogrid) spaced vertically every other block (1.34 ft.). The length of reinforcement was selected as 0.7 times the height of the wall, in accordance with AASHTO requirements. A surface surcharge load of 240 psf (typically assumed in transportation projects) was selected. An ASSHTO A-1-a reinforced fill material was assumed, and assumed engineering properties for the retained soil and foundation soil are as noted. The focus of the illustrative analyses will be on the tensile force in the reinforcement (internal stability) and global stability.

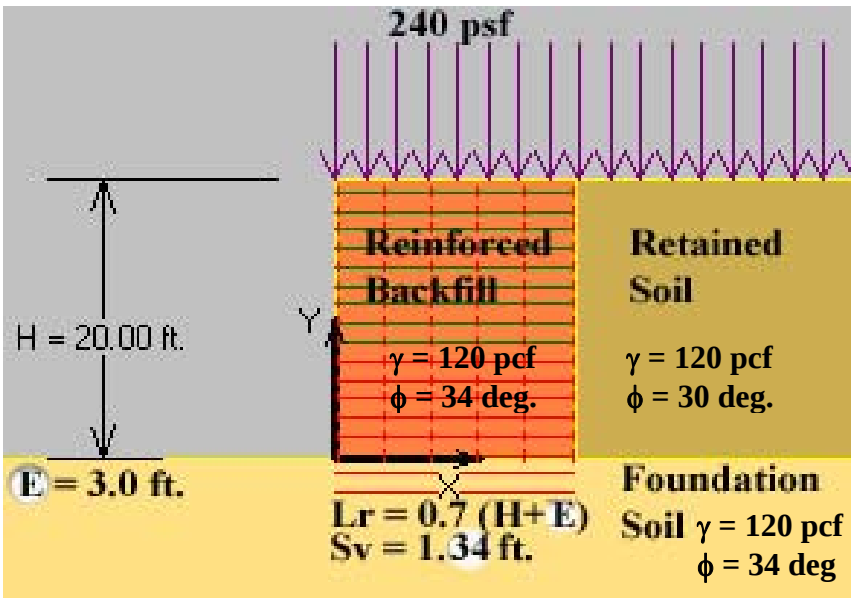

Figure 3-1 Example model for comparison of Earth Pressure, Limit Equilibrium and Numerical analysis methods

3.3.1 Internal Stability

With the lateral earth pressure approach, the maximum tensile force in each reinforcement layer is computed based on conventional lateral earth pressure theory. For the purpose of this example, the AASHTO approach was used (i.e. the Rankine Method). Utilizing the computer code MSEW (ADAMA Engineering, Inc.), developed for FHWA on the basis of FHWA-SA-96-071, the calculated distribution of maximum tensile force with depth is presented on Figure 3-2, and is linear with depth.

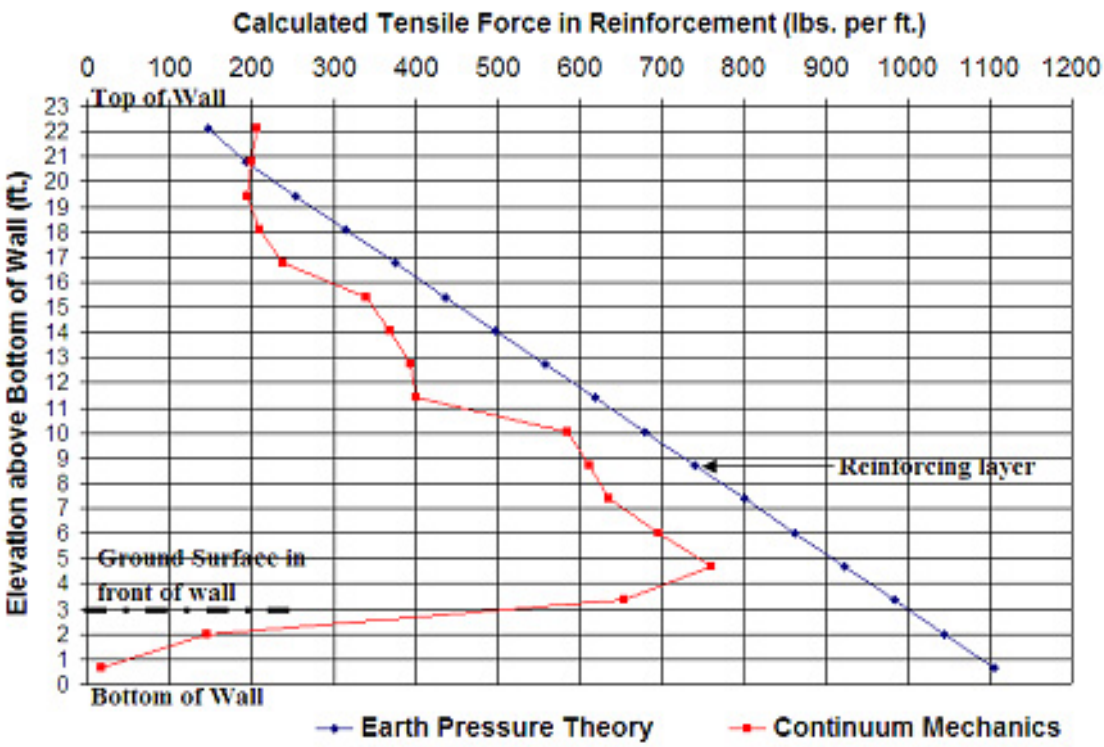

Figure 3-2 Comparison of calculated reinforcement tensile force from numerical analysis methods

Based on the maximum tensile forces computed from earth pressure theory, one economical reinforcement design to satisfy internal stability with respect to breakage of the reinforcement results in two zones of geogrid reinforcement (a lower zone that consists of reinforcing layers with an ultimate wide width strength of 6,000 lbs. per ft., and an upper zone having reinforcing layers with an ultimate wide width strength of 4,000 lbs. per ft.). This assumes a cumulative reduction factor of approximately 4.0 on the ultimate wide width strength of the geogrid reinforcement, and a factor of safety for design uncertainties of 1.5. This design also satisfies all other AASHTO internal and external stability requirements.

Utilizing the reinforcement design derived from earth pressure theory, a continuum mechanics-based numerical analysis was performed. The computer code PLAXIS was used for this analysis. The soil was modeled using an elastic-plastic Mohr-Coulomb model. The elastic parameters assumed are E=3,000 psf and v=0.2. The soil strength parameters are shown in figure 3-1. The geogrid layers were modeled using elements that can only sustain tensile forces and no compression. The stiffness of the grids was set at 60,000 lbs/ft per foot width for the bottom seven layers and 40,000 lbs/ft. per foot width for the upper layers. A reduced shear strength of 2/3 the soil strength was used to transfer forces between the geogrid layers and the soil layers. To simplify the analysis, the facing was modeled as a flexural beam element with a low bending stiffness.

Tensile force in the reinforcement layers was calculated in the PLAXIS analysis, and the results are presented in Figure 3-2. The results confirm observations from previous instrumented test walls (i.e. Earth Pressure theory over-estimates reinforcement loads in geosynthetic reinforced soil walls). The reduction in reinforcement tensile force at the bottom of the wall predicted in the PLAXIS analysis is a result of: a) the passive resistance against the embedded

portion of the wall, and b) friction between the base of the wall and the foundation soil. These two factors act to restrain the base of the wall from moving.

3.3.2 Global Stability

The results of the PLAXIS deformation analysis are presented in Figure 3-3. Of interest are the identified overall shear zone and shear strains occurring at the back of the reinforced fill zone. As noted, the predicted shear strains are for a factor of safety of 1.49. It should be noted that the definition of factor of safety in PLAXIS is the "Actual Strength/Reduced Strength" at failure, where failure .is the point at which large deformations occur without a large reduction in strength. Thus, Figure 3-3 represents predicted shear strains at a state of deformation with strengths reduced by a factor of 1.49. This reduction in strength is just about at failure (i.e. FS = 1.0). The amount of strength reduction required to reach this condition is, by definition, the same factor of safety as in the limit equilibrium analysis. This will be demonstrated later.

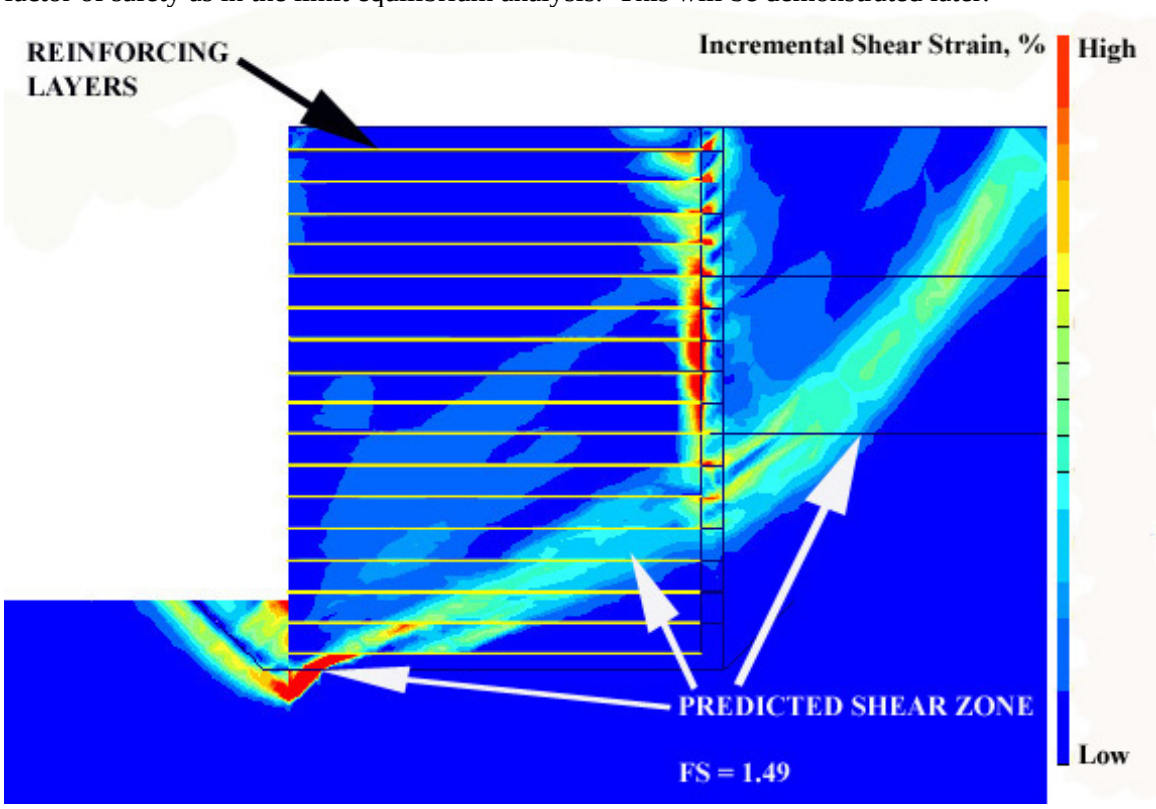

Figure 3-3 Predicted in-service shear strains (Continuum Mechanics, Finite Element Method [PLAXIS])

The predicted location and magnitude of shear strains at the back of the reinforced fill zone tend to support field observations relative to the development of surface tension cracks at the back of the reinforced fill zone of MSE walls when "high fines" and/or "high plasticity" reinforced fill is used.

Note that the continuum mechanics approach identified a compound failure surface (i.e. passing through both the unreinforced and reinforced zones) as the most critical failure mode for the example problem. AASHTO requires that compound failures be considered using limit equilibrium analyses.

A limit equilibrium analysis was performed using the computer program ReSSA 2.0 (ADAMA, 2003). For the example problem, Spencer's method was used. The results of the analysis are presented in Figure 3-4.

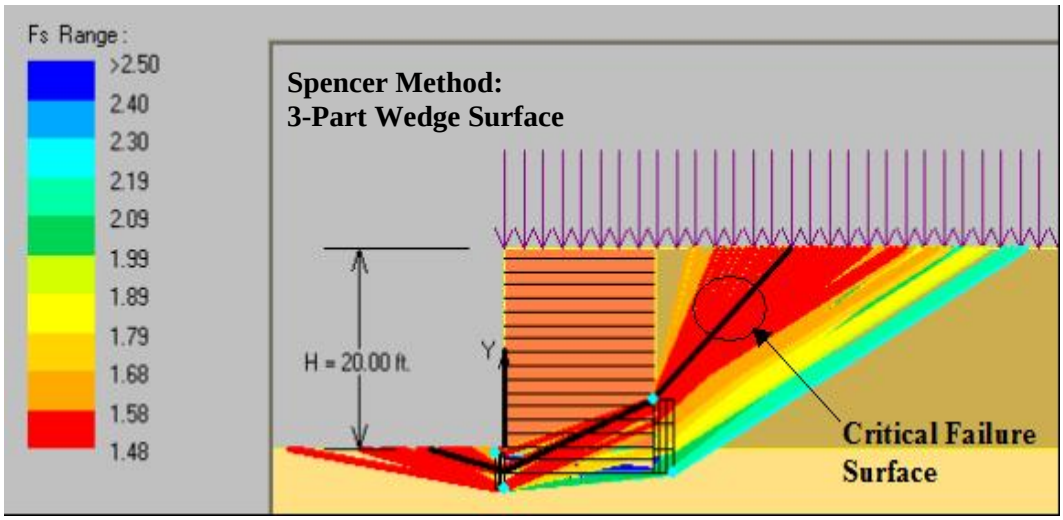

Figure 3-4 Spatial distribution of safety factors (Limit Equilibrium, Compound Failure)

The predicted critical failure surface and computed factor of safety are nearly identical to those predicted from the continuum mechanics analysis (i.e. PLAXIS). In limit equilibrium, the combination of critical failure surface and associated factor of safety constitutes the solution. It should be noted that the limit equilibrium approach is much easier to apply in a conventional design analysis than the continuum mechanics approach.

These numerical analysis methods were used in the design of the full-scale test walls constructed for this study, as discussed in Appendix B.

CHAPTER **4**

CONCLUSIONS

The focus of Project NCHRP 24-22, "Selecting Backfill Materials for MSE Retaining Walls", is to develop selection guidelines, soil parameters, testing methods, and construction specifications that will allow the use of a wider range of backfill materials within the reinforced zone of mechanically stabilized earth (MSE) retaining walls. This report describes Phase I of the project, which includes a survey of current practice, developing parameters for lowest-quality backfill soils that give acceptable performance, and preparing a work plan for conducting a full-scale, instrumented field test.

The results of the literature search and survey indicate that MSE walls on transportation projects are generally conservatively designed, with "low fines" reinforced soils. Private MSE walls are less conservatively designed, and use a variety of reinforced soils (NCMA allows for 35% < 0.075mm). It is also clear from the literature that reinforced soil consisting of fine-grained soils (either "high" fines or "high" plasticity) and pore pressure resulting from lack of drainage in the reinforced zone were the principle reasons for serviceability problems (excessive deformation) or failure (collapse) of MSE walls.

Material properties of "high fines" reinforced fill have been identified that have an important effect on the design and performance of MSE walls. It has been concluded that a higher quantity of fines could be safely allowed in the reinforced fill, provided the properties of the materials are well defined and controls are established to address the design issues. The potential savings from replacing AASHTO backfill materials with marginal backfill materials could be in the range of 20 to 30% of current MSE wall costs.

A full-scale field test was recommended for Phase II of the evaluation. The field test included provisions to demonstrate the role of pore water pressure in the backfill and the importance of including a positive drainage system to obtaining good wall performance. Details and results of the full-scale field test are presented in Appendix B.

Results from the field test allowed guidelines and specifications to be developed for suitable reinforced fills for MSE retaining walls, as presented in this report.

CHAPTER **5**

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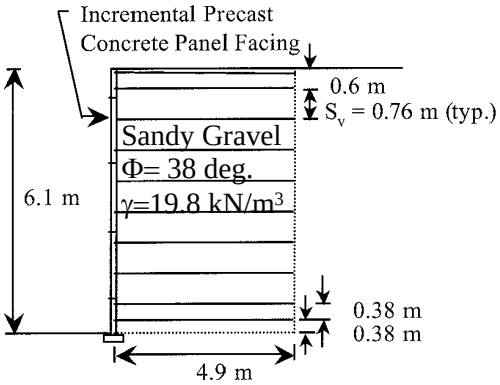
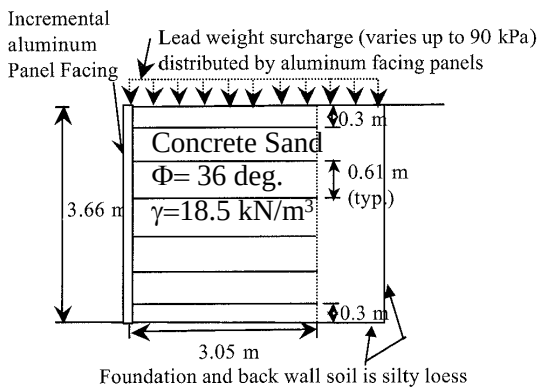
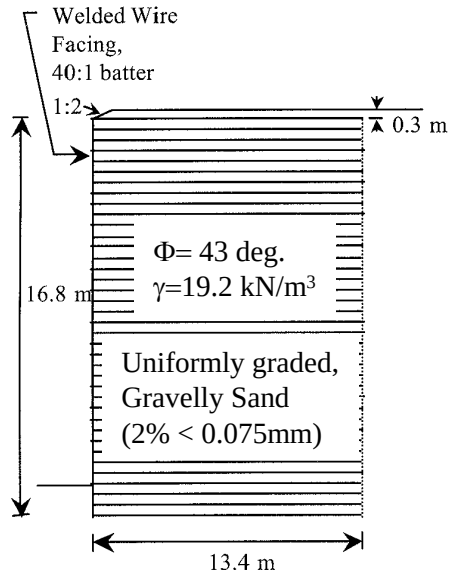
ATTACHMENT A - Figures - Instrumented Case Histories



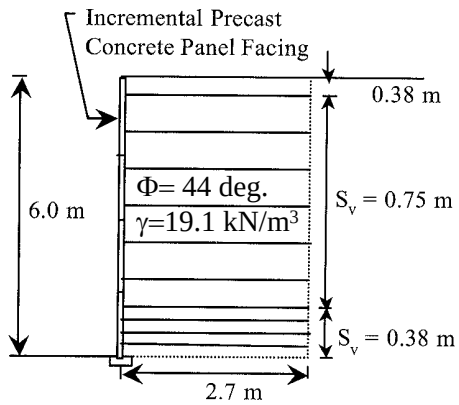
1974 - UCLA steel strip test wall



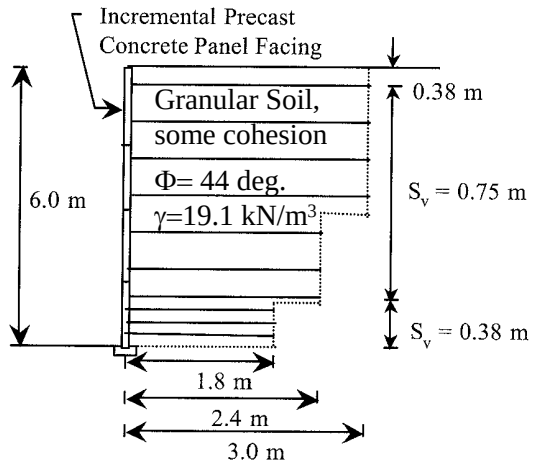
1976 - WES steel strip test wall



1985 - Rainier Avenue welded wire wall
USA (Washington)



(a) Rectangular section



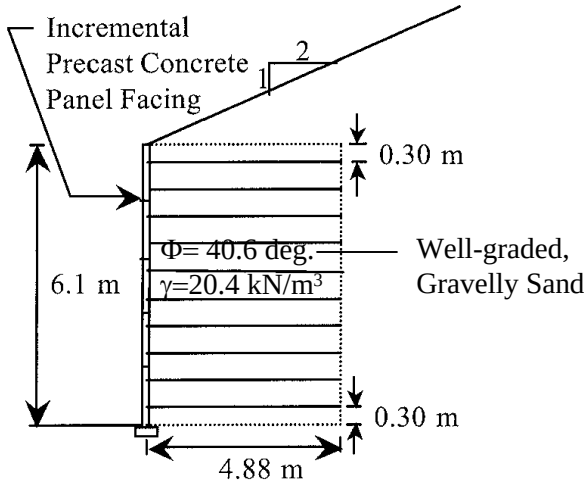
(b) Trapezoidal section

1983 - Millville, West Virginia, steel strip MSE walls

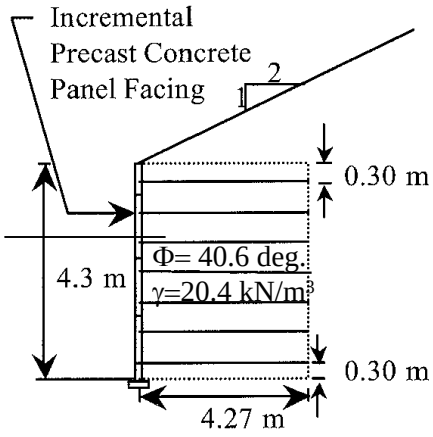
From Allen, Christopher, Elias & DiMaggio, "Development of the Simplified Method For Internal Stability Design of Mechanically Stabilizes Earth Walls", June 2001

Instrumented MSE Walls
Metallic Reinforcement
(Domestic Cases)

Figure F-1

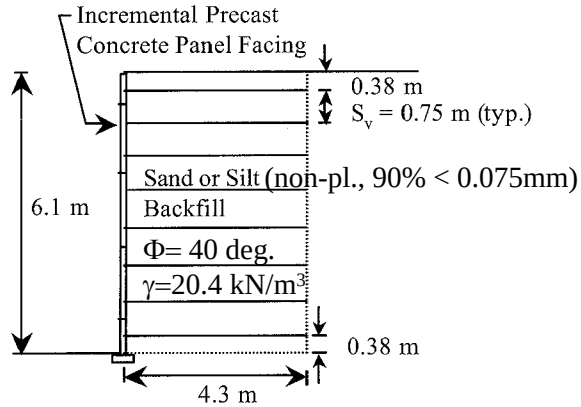


(a) Section 1

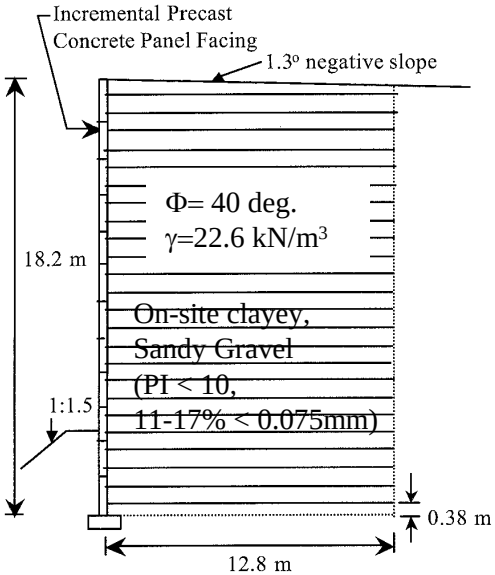


(b) Section 2

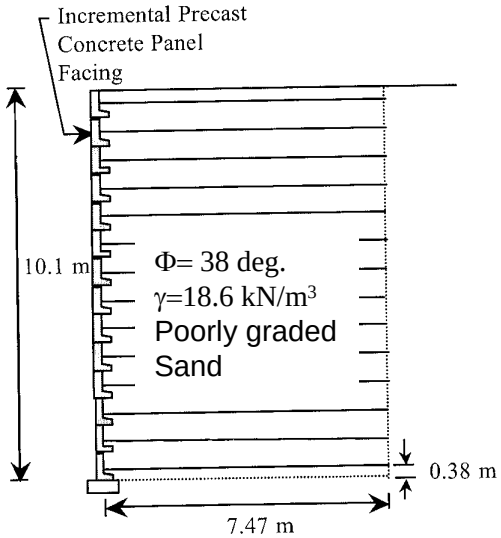
1981 - Hayward bar mat walls
USA (California)



1988 - Algonquin steel strip and bar mat MSE wall
USA (Illinois)



1988 - Cloverdale, California, bar mat wall

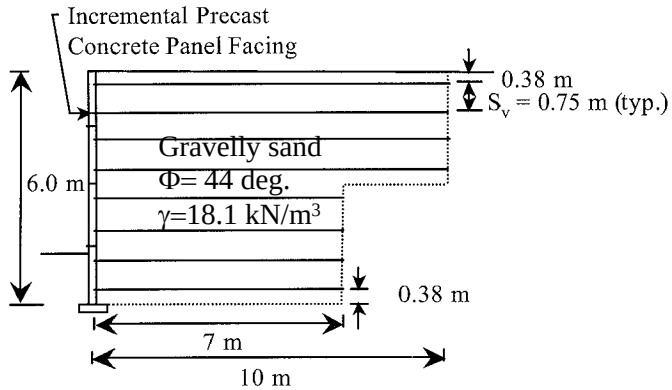


1991 - Houston, Texas, welded wire wall

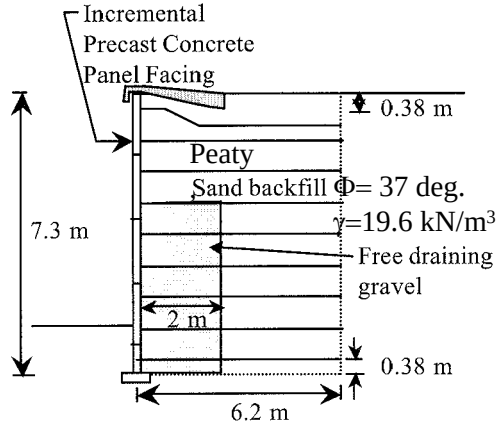
From Allen, Christopher, Elias & DiMaggio, "Development of the Simplified Method For Internal Stability Design of Mechanically Stabilizes Earth Walls", June 2001

Instrumented MSE Walls
Metallic Reinforcement
(Domestic Cases)

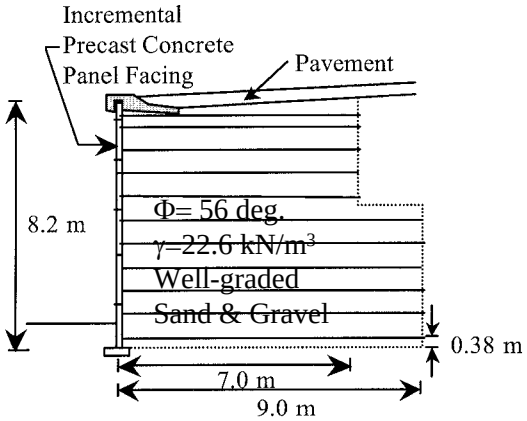
Figure F-2



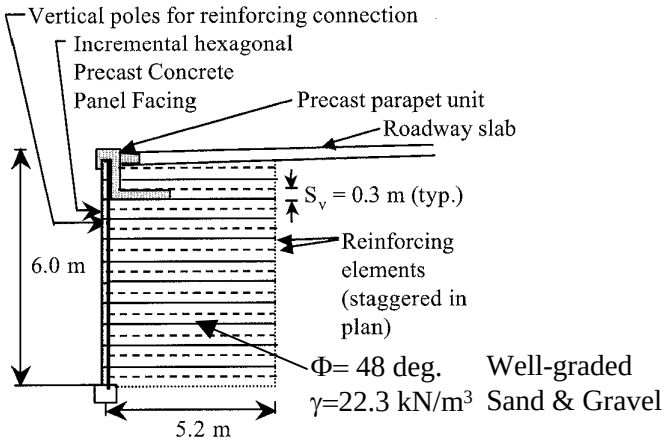
1972 - Lille, France, steel strip test wall



1980 - Fremersdorf steel strip MSE wall.
Germany



1981 - Waltham Cross steel strip MSE wall
United Kingdom

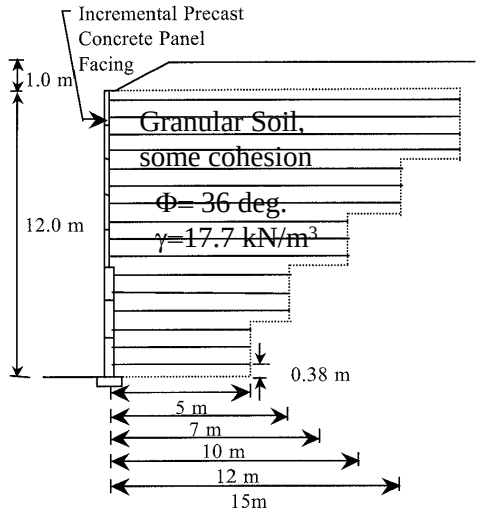


1981- Guildford Bypass steel strip reinforced MSE wall
United Kingdom

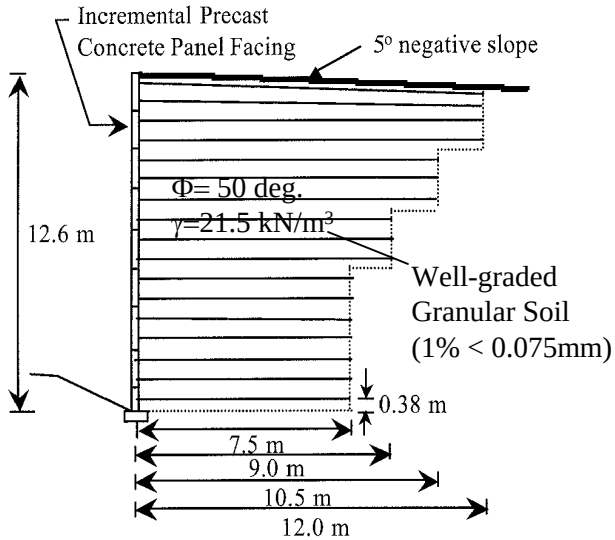
Instrumented MSE Walls
Metallic Reinforcement
(International Cases)

Figure F-3

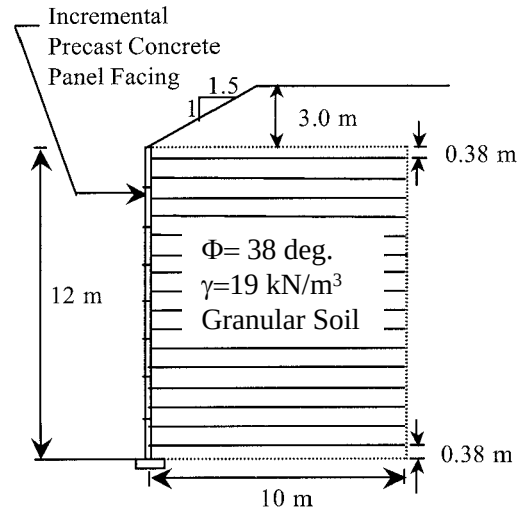
From Allen, Christopher, Elias & DiMaggio, "Development of the Simplified Method For Internal Stability Design of Mechanically Stabilizes Earth Walls", June 2001



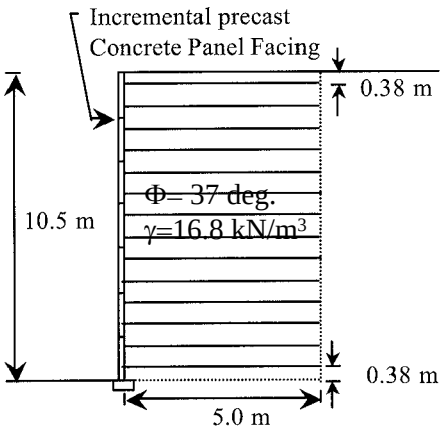
1982 - Asahigaoka, Japan, steel strip MSE wall



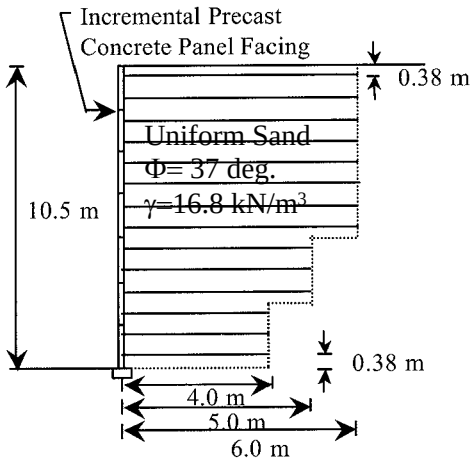
1985 - Ngauranga, New Zealand, steel strip MSE wall



1990 - Gjovik, Norway, steel strip MSE wall



(a) Rectangular section



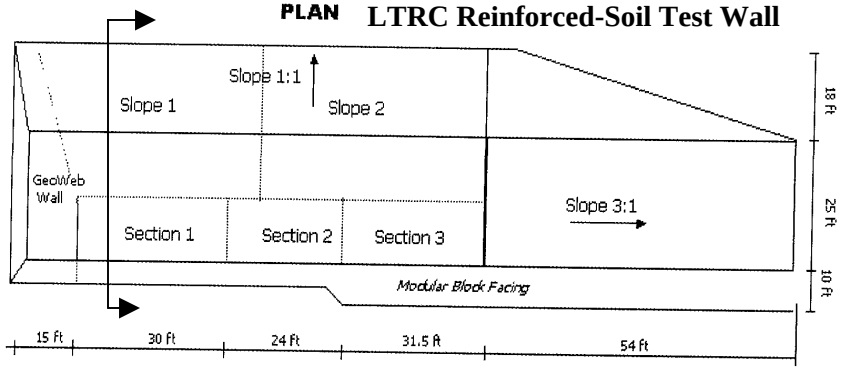
(b) Trapezoidal section

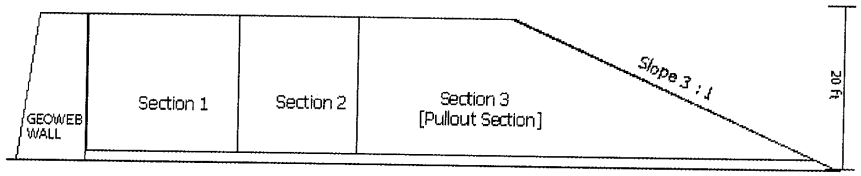
1993 - Bourron Marlotte steel strip MSE test walls
France

From Allen, Christopher, Elias & DiMaggio, "Development of the Simplified Method For Internal Stability Design of Mechanically Stabilizes Earth Walls", June 2001

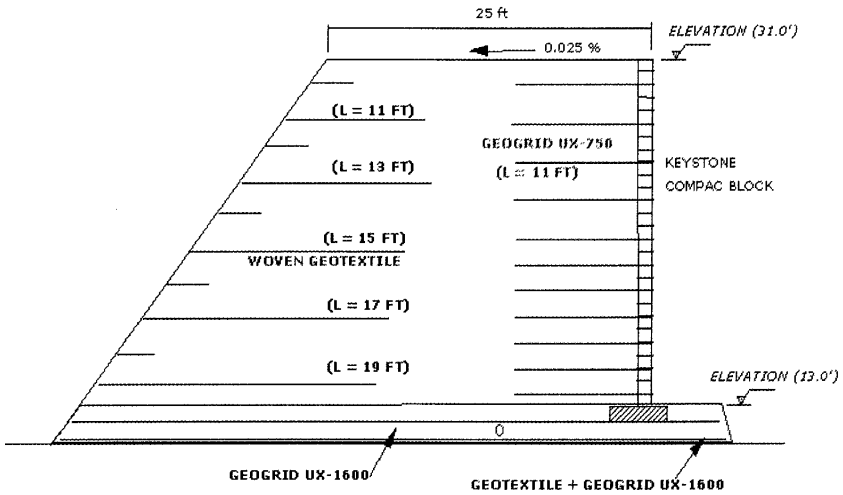
Instrumented MSE Walls
Metallic Reinforcement
(International Cases)

Figure F-4

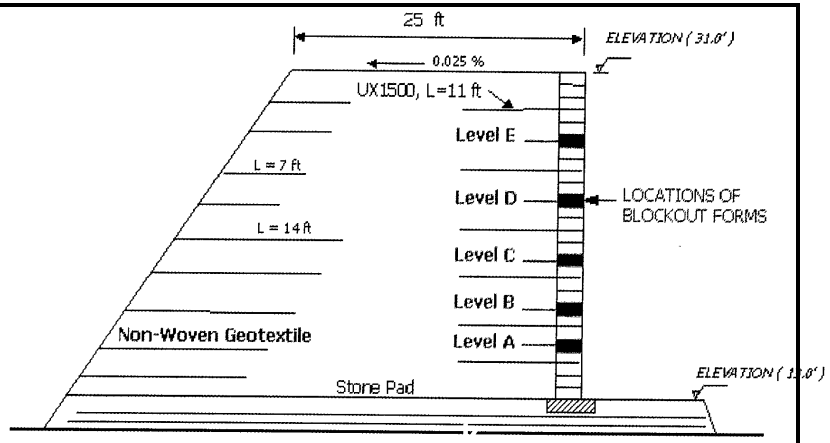




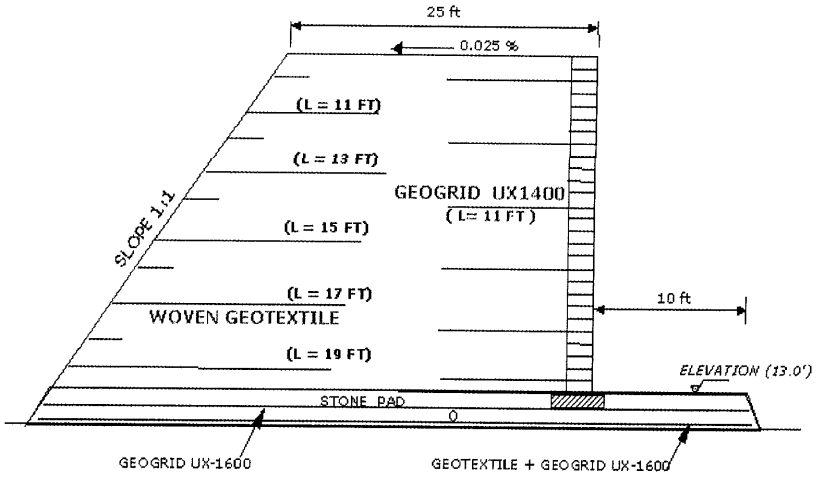
ELEVATION



Section 1



Section 3



Section 2

Properties of the Reinforced Fill

% Silt	72
% Clay	19
PI	15%
Φ (deg.)	24
Cohesion, C (psf)	30
OMC (%)	18.5
γdry max (pcf)	105

Plan & Cross-Sections
Instrumented MSE Test Wall
with Marginal Reinforced Fill
Louisiana Transportation Research Center

Figure F-5

From Louisiana Department of Transportation and Development, Louisiana Transportation Research Center (LTRC) – Farrag et al, 2004

ATTACHMENT B - Summary Tables – Task 1 Literature Search & Task 2 Survey

Table T-1 Full-scale MSE Walls constructed using "poorly draining" reinforced soil (Ref. Mitchell et al. 1995)

Name	Location	Date	Structure	Height (m)	Reinforcement Method	Reinforced Soil	Facing	Construction	Comments	Reference
Yokahama Residential Complex	Japan	1978	MSE Wall	8.7	Metal strips	Volcanic clay	Facing panels	Reinforcement tension was monitored	Final settlements of up to 91 cm were observed	Hashimoto 1979
Virginia Wall	USA VA	1978-79	MSE Wall	up to 7.0	Ribbed steel strips	Residual low-plasticity Silt	Concrete panels	Tilting 250 to 300 mm out of plumb occurred after construction	Areas of backfill with more than 25% fines were excavated and replaced with select backfill	Elias and Swanson 1983
Interstate 80	USA Baxter, CA	1982	4 Embankment Walls	5	Bar mat	Silt (49%<0.075mm)	Prefabricated concrete facing	Construction forced to stop due to rainfalls	Extensive instrumentation showed no significant wall movements	Hannon & Forsythe 1984
Interstate 580	USA Hayward, CA	1982	Vertical-faced Wall	1.8 to 9.I	Welded wire mesh	Sandy Clay with potential expasibility	Facing panels	There was poor drainage of surface water	Wall showed excessive movement and cracking	Mitchell and Villet 1987
Paulsgrove Experimental Wall	Hampshire UK	1985	Experimental wall	5.6	Steel strips	Three types of local chalk	Concrete facing panels	Negative pore water pressures were generated during constr.	Horizontal wall movements were up to 15 mm 3 mos. after construction. No later movement.	Temporal et al. 1989
AIT Experimental Wall	Thailand	?	Experimental wall	5.7	Welded wire mats	Clayey Sand, lateritic soil, weathered clay	Vertical wire mesh	Wall was stable. Large settlements & lateral movements occurred.	Subsoil movements greatly influenced vertical pressure beneath the wall and reinforcement tensions	Bergado et al. 1991

Table T-2 (a) – General information for the domestic wall and slope cases

Wall												
Case No.	Reference (Author)	Completed Year	Facing Type	Wall Geometry						Type of Wall		Structure above Wall
				Site Location	Dimension (H/L, area)	Slope		Style Tiered /Set back	Wall Batter	Cut	Fill	
						Backfill	Front slope					
1	R.J. Bathurst	1990	modular concrete facing unit	California	18m/76m	N/A		two tiers: 14.3m for lower wall, 3.7m for upper one	vertical	x		N/A
2	R.J. Bathurst	1993	segmental wall block unit	University of Wisconsin-Platteville, Wisconsin	3.5m/37m	N/A		setback	1H:8V	N/A		N/A
3	J. Paulson	1999	segmental wall block unit	southeastern USA	7.8m high	N/A		setback	1H:12V	x		traffic surcharge
4	J.K. Mitchell	1983	wrap around	Newport, Oregon	9m high	N/A		setback	1H:5.7V	N/A		N/A
5	K.L. Fishman	N/A	full height precast panel	Tucson, Arizona	N/A	no slope		no	vertical	N/A		road
6	U. Eliahu	N/A	masonry module	San Bruno, California	5.5m/124m	2H:1V	1H:1V	N/A	N/A	x		N/A
7	J.L. King	1991	wire baskets with shotcrete facing	Kingwood, Texas	0.9-3.36m/26.9m	no slope		no	N/A	x		N/A
8	S.Kemp	1992 (primary wall)	modular Block	Charleston, South Carolina	4.7m/541m	no slope	3H:1V	no	N/A	N/A		N/A
9	O.A. Moreno	1991	modular Block	San Antonio, Texas	1-3.14m/41-267m	no slope	N/A	N/A	N/A	x		N/A
10	R.R. Berg	1992	segmental block	Eau Claire, Wisconsin	max. 6.7m/42.7m	N/A	N/A	no	N/A	N/A		N/A
11	M.R. Simac	1994	PVC-coated gabion basket	between Tellico Plains, Tennessee and Robbinsville, North Carolina	10.5m/60m	1.5H:1V	1.6H-2.5H:1V	no	1H:12V	x		N/A
12	D. Greenway	1974	sand-cement mortar sprayed directly on geotextile	northwest Oregon	3m/20m	N/A	N/A	no	1H:8V	x		N/A

Table T-2 (a) – Continued

Wall												
Case No.	Reference (Author)	Completed Year	Facing Type	Wall Geometry						Type of Wall		Structure above Wall
				Site Location	Dimension (H/L, area)	Slope		Style Tiered /Set back	Wall Batter	Cut	Fill	
						Backfill	Front slope					
13	R.A.Reid	1996-1997	wrap-around	Higyway 73, and Interstate 29, South Dakota	N/A	no slope	no slope	no	vertical	N/A		bridge
14	D. Chandler	1989	rap-around and shotcrete facing for UV protection	Everett, Washington	approximately 2.1m/190m	N/A	N/A	no	1H:6V	N/A		road
15	R. D. Maynard	Dec., 1987	wire basket made of galvanized No.5 wire welded into a 4" square grid pattern	Snohomish County, Washington	typically 3.3 to 5.7m high	N/A	N/A	no	1H:6V	N/A		road
16	M. R. Simac	1990	modular block facing	Charlotte, North Carolina	N/A	no slope	N/A	no	1H:20V	N/A		N/A
17	M. R. Simac	1990	modular block facing	Exton, Pennsylvania	5.4m/255m	no slope	N/A	setback of 1.27cm to 0.64 cm per course	N/A	N/A		N/A
18	M. R. Simac	1990	modular masonry concrete block	Deptford, New Jersy	2.55m/60m	no slope	no slope	no	N/A	N/A		N/A
19	R. B. Anderson	Nov. 1989	modular block facing	South wall, Atlanta, Geogia	0.6-7.8m/157.5m	no slope	no slope	no	vertical	N/A		N/A
20	R. B. Anderson	Jun., 1990	modular block facing	North wall, Atlanta, Geogia	up to 12m high	no slope	N/A	no	vertical	N/A		parking lot
21	D. J. Yovaish	Jun., 1987	concrete block	Seminole County, Florida	1.8-5.4m/112.5m	N/A	4H:1V berm	N/A	1H:96V	N/A		N/A
22	J. C. Kliethermes	Jun., 1988	modular concrete masonry unit	Bloomington, Minnesota	about 9m high	N/A	N/A	three tiers: 4.8m horizontal distance between lower wall and middle wall, and 2.4m horizonatl distance between middle wall and upper wall.	1H:4V	N/A		sidewalk

Table T-2(a) – Continued

Wall												
Case No.	Reference (Author)	Completed Year	Facing Type	Wall Geometry						Type of Wall		Structure above Wall
				Site Location	Dimension (H/L, area)	Slope		Style Tiered /Set back	Wall Batter	Cut	Fill	
						Backfill	Front slope					
23	T.M. Allen	1975	wrapped-face	Shelton, Washington	5.6m hight	no slope	N/A	no	1H:6V	N/A		traffic surcharge
24	T.M. Allen	1982	wrapped-face	Central Oregon coast, Oregon	8.8m high	no slope	N/A	no	1H:6V	N/A		traffic surcharge
25	T.M. Allen	1984	full-height precast concrete panel	Tucson, Arizona	4.65m high	N/A	N/A	no	no	N/A		traffic surcharge
26	T.M. Allen	1985	precast concrete panel	Lithonia, Georgia	6.1m high	N/A	N/A	no	no	N/A		traffic surcharge
27	T.M. Allen	1988	precast concrete panel	Algonquin, Illinois	6.1m/10m	N/A	1.5H:1V slope	no	no	N/A		2.1m soil surcharge
28	T.M. Allen	1988	masonry block facing unit	Algonquin, Illinois	6.1m/15m	N/A	1.5H:1V slope	no	1H:20V	N/A		2.1m soil surcharge
29	T.M. Allen	1988	wrapped-face	Algonquin, Illinois	5.9m high	N/A	N/A	no	no	N/A		N/A
30	T.M. Allen	1989	wrapped-face	Seattle, Washington	11.85m high	N/A	1.8H:1V	no	1H:20V	N/A		5.3m soil surcharge
31	N.Abu-Hejleh	1999	concrete block (part of Mesa System)	N/A	typically 4.5m-5.9m high	N/A	N/A	no	no	N/A		bridge
32	R.J. Fannin	1987	steel mesh	N/A	4.8m/20m	N/A	N/A	N/A	1H:2V	N/A		N/A

Table T-2(a) – Continued

Wall												
Case No.	Reference (Author)	Completed Year	Facing Type	Wall Geometry						Type of Wall		Structure above Wall
				Site Location	Dimension (H/L, area)	Slope		Style Tiered /Set back	Wall Batter	Cut	Fill	
						Backfill	Front slope					
33	G.N. Richardson	1987	timber	Raleigh, North Carolina	max. 3.9m, aver. 3m high, 102m long	N/A		N/A	N/A	x		N/A
34	G.A. Leonards	1990	Keystone block facing	Glasgow, Kentucky	3-6m high	bewtween 1.7H:1V and 2H:1V	N/A	no	no	x		soil slope
35	R.M.Koerner	1994	segmental block	N/A								
36	R.M.Koerner	1996	segmental block	N/A								
37	R.M.Koerner	1997	segmental block	N/A								
38	R.M.Koerner	1998	segmental block	N/A								
39	R.M.Koerner	1998	segmental block	N/A								
40	R.M.Koerner	1998	segmental block	N/A								
41	R.M.Koerner	1990	wrap-around	N/A								
42	R.M.Koerner	1995	segmetal block	N/A								
43	R.M.Koerner	1995	precast panel	N/A								
44	R.M.Koerner	1998	segmetal block	N/A								

Table T-2(b) – Reinforcement information for the domestic wall and slope cases

Reinforcement						
Case No.	Reinforcement			Strength	Spacing	Length
	GG	GT	Style			
1	x		GG (PET)	long term design strength 59KN/m	23 layers for 18m	N/A
2	x		GG (HDPE)	15.3KN/m	0.5-0.75m	2.12m
	x		woven GG (PET)	15.3KN/m	0.5-0.75m	2.12m
		x	woven GT (PET)	13.1KN/m	0.5-0.75m	2.12m
3	x		flexible PET GG	N/A	0.4-0.6m	4-4.5m
4	x		Tensar SR2 (HDPE)	N/A	min. 0.3m	4.9m
5	x		Tensar SR2 (HDPE)	max. tensile strength is 99kN/m; and 2% modulus 1094kN/m	N/A	N/A
6	x		UX1600, UX1500	N/A	N/A	up to 7.2m
7	x	used only for separation	N/A	17.78kN/m	N/A	max. 1.86m
8	x		Miragrid 5T, 7T,10T (PET)	5T:38kN/m; 7T:52.5kN/m; 10T: 93.4kN/m.	N/A	N/A
9	x		N/A	long term design strength: 6.13kN/m	0.4-0.7m	2-2.99m
10	x		uniaxial geogrid (HDPE)	long term allowable strength: 17.7kN/m	approximately 0.5m	3.65m
11	x		Miragrid geogrid	type 1: tensile strength (MD) 370kN/m; long term allowable strength 86.5kN/m.	N/A	11-17m
				type 2: tensile strength (MD) 259kN/m; long term allowable strength 60.5kN/m.		
				type 3: tensile strength (MD) 102kN/m; long term allowable strength 20.4kN/m.		
12		x	nonwoven, needle-punched, continuous filament geotextile (PP)	grab strength: 1.1kN	N/A	3m

Table T-2(b) – Continued

Reinforcement						
Case No.	Reinforcement			Strength	Spacing	Length
	GG	GT	Style			
13		x	woven GT	N/A	30.5cm for the first 3 layers, varying for the other layers	N/A
14		x	N/A	allowable tensile strength 11.7kN/m	0.3m spacing for the to depth of 1.8m and 0.23m for the depth between 1.8m and 2.7m	N/A
15	x		N/A	for walls less than 2.4m high use allowable long term design strength of 14.8kN/m; for those larger than2.4m, use 29.6kN/m	typically 0.6m	3.3m-5.1m
16	x		high tenacity, continuous multi-filament polyester geogrid	15.3kN/m in the upper part; 39.4kN/m in the lower part	varying	4.35m
17	x		large aperture, high strength, continuous multi-filament polyester geogrid	39.4kN/m	varying	3.9m
18	x		large aperture, high strength, continuous multi-filament polyester geogrid	39.4kN/m	varying	6m
19	x		Tensar UX1400, UX1500, UX1600	long term design strength: UX 1400 17.8kN/m; UX1500 32.6 kN/m; UX1600 44.5kN/m	varying for efficiency	varying with wall height, and length to wall height ratio is about 0.6
20	x		Tensar UX1400, UX1500, UX1600	long term design strength: UX 1400 17.8kN/m; UX1500 32.6 kN/m; UX1600 44.5kN/m	varying for efficiency	varying with wall height, and length to wall height ratio is between 0.45 and 0.5
21	x		Tensar SR2	long term allowable strength 29.6kN/m	N/A	the first 3 layers are 6.6m, other layers varied from 2.4m to 3.2m
22	x		geogrid (HDPE) with apertures of 13.7cm long in machine direction and 1.7cm wide in cross machine direction	long term design strength 44.5kN/m	0.6m	4.8m in lower wall; 3.6m in middle wall; 1.8m in upper wall.

Table T-2(b) – Continued

Reinforcement						
Case No.	Reinforcement			Strength	Spacing	Length
	GG	GT	Style			
23		x	nonwoven needle-punched geotextile (PP or PET)	ultimate strength for PET 14.4-24.7kN/m, and 15.8-22.1kN/m for PP	0.23-0.3m	3.7m
24	x		Tensar SR-2 Extruded geogrid (HDPE)	73 kN/m	in upper 2.2m, 0.9m spacing; in middle 3.8m, 0.6m spacing; in lower 2.8m, 0.3m spacing	4.9m
25	x		Tensar SR-2 Extruded geogrid (HDPE)	73 kN/m	varying from 0.3 to 0.9m	3.7m
26	x		Tensar SR-2 Extruded geogrid (HDPE)	79kN/m	in upper 3.6m, 0.6m spacing; in lower 2.1m, 0.3m spacing	3.7m
27	x		Tensar SR-2 Extruded geogrid (HDPE)	67.8kN/m	typically 0.75m	4.3m
28	x		Miragid 5T geogrid (PET)	39.2kN/m	0.6-1m	4.3m
29	x		Quline160 geogrid (PET)	19.3kN/m	typically 0.75m	4.3m
30		x	Exxon GTF200 (PP), GTF375 (PP), GTF500 (PET), GTF1225T (PET)	31, 63, 92, and 186kN/m, respectively	typically 0.38m	9.75m
31	x		Tensar UX6 geogrid	ultimate strength is157.3 kN/m	typically 0.4m	about 7.8m
32	x		uniaxial geogrid for primary reinforcement (HDPE); biaxial geogrid for intermediate geogrid (PP)	N/A	less than 0.6m for primary reinforcement	1.5m-3m

Table T-2(b) – Continued

Reinforcement						
Case No.	Reinforcement			Strength	Spacing	Length
	GG	GT	Style			
33		x	N/A	N/A	0.6m	N/A
34	x		Tensar geogrid	N/A	varying	varying with wall height
35	x		N/A			
36	x		N/A			
37	x		N/A			
38	x		N/A			
39	x		N/A			
40	x		N/A			
41	x		N/A			
42	x		N/A			
43	x		N/A			
44	x		N/A			

Table T-2(c) – Soil information for the domestic wall and slope cases

Soil																
Case No.	Reinforced Zone (Filled soil)							Retained Filled zone (Filled Soil)							Retained Cut zone (In-situ Soil)	
	Type	Fine (%) (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	% Fine (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	Strength
			c	Φ	H_2O%	Density				c	Φ	H_2O %	Density			
1	sand	N/A						sand	N/A						N/A	
2	washed crushed limestone	N/A	0	38°	N/A			N/A							N/A	
3	silty sand (local fill)	N/A	N/A	30°	N/A	20.4kN/m^3	soil compacted on wet side of optimum	same as reinforced zone							N/A	
4	crushed basalt	N/A	0	40°	N/A	22kN/m^3	N/A	N/A							N/A	
5	N/A	N/A	N/A	34°	N/A	19.6kN/m^3	N/A	N/A			30°	N/A	18kN/m^3	N/A	N/A	
6	granular fill	5	N/A	22°	N/A		compacted to a min. of 90% max. dry density at min. 3% above optimum moisture	N/A							silty sand, lean clay	N/A
7	cement stabilized sand	N/A					15.2 cm lifts, compacted using a hand operated vibratory compactor	N/A							N/A	
8	cohesionless soil	15	N/A	35°	N/A	18.8kN/m^3	N/A	same as reinforced zone							N/A	
9	clean limestone	N/A					using 20.3cm lifts	N/A							N/A	
10	sand	N/A	N/A	32°	N/A	18.8kN/m^3	N/A	same as reinforced zone							N/A	allowable bearing capacity 239kN/m^2
11	N/A	N/A	N/A	34°	N/A	19.4kN/m^3	N/A	same as reinforced zone							N/A	c=4kPa,Φ=40°, γ=19.4kN/m^3
12	N/A	N/A	N/A	30°	N/A	19.6kN/m^3	30cm lift in the top portion, and 23cm lift in the lower portion	N/A							N/A	

Table T-2(c) – Continued

Soil																	
Case No.	Reinforced Zone (Filled soil)							Retained Filled zone (Filled Soil)							Retained Cut zone (In-situ Soil)	
	Type	Fine (%) (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	% Fine (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	Strength
			c	Φ	H_2O%	Density				c	Φ	H_2O %	Density			
13	selective granular material	N/A						N/A							N/A	
14	a free drainig sand used as backfill for all areas within 0.46m of excavation face, and the on-site materials utilized for remainder of the backfill	N/A					at least 95% of the Modified Procotor max. dry density	N/A							N/A	
15	gravel with some fines	N/A					95% of its max. dry density	N/A							N/A	
16	silty fine to medium sand (SP-SM) (There is a drainage strip along the facing)	N/A	0	30°	N/A	moisture density 21.4kN/m^3	N/A	N/A							N/A	
17	clayey silt (ML) and silty sand (SP-SM)	N/A	0	for ML: total 30°, effective 32°; for SP-SM: total 32°, effective 34°;	N/A	moisture density for ML is 20.7kN/m^3; for SP-SM is 21.6 kN/m^3	N/A	N/A							N/A	
18	sand	N/A	0	32°	N/A		N/A	N/A	N/A	0	20°	N/A	18.1kN/m	N/A	N/A	
19	silty sand with some clay	N/A	N/A	32°	N/A	20.6 kN/m^3	a hand operated hamper used to compacted the crushed stone and common fill within 3ft of wall face; a vibratory sheep's foot used to compact the fill behind this 3ft zone to a 95% Standard Proctor	N/A							N/A	
20	silty sand with some clay	N/A	N/A	32°	N/A	20.6 kN/m^3	N/A	N/A							N/A	
21	sand	<5	N/A	N/A	N/A	N/A	compacted to a min. density equivalent to 95% max. Modified Proctor density	N/A							N/A	
22	poorly graded fine sand (SP)	N/A	0	32°	N/A	moisture density 19.8 kN/m^3	N/A	same as reinforced zone							same as reinforced zone	

Table T-2(c) – Continued

Soil																
Case No.	Reinforced Zone (Filled soil)							Retained Filled zone (Filled Soil)							Retained Cut zone (In-situ Soil)	
	Type	Fine (%) (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	% Fine (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	Strength
			c	Φ	H_2O%	Density				c	Φ	H_2O %	Density			
23	crushed rock	N/A	N/A	40°	N/A	18.1 kN/m^3	N/A	N/A							N/A	
24	crushed basalt	10	N/A	40°	N/A	22 kN/m^3	N/A	N/A							N/A	
25	clean well graded gravelly sand	3	N/A	34°	N/A	19.6 kN/m^3	N/A	N/A							N/A	
26	well graded sandy gravel	N/A	N/A	40°	N/A	21.2 kN/m^3	N/A	N/A							N/A	
27	clean well graded gravelly sand	0	N/A	40°	N/A	20.4 kN/m^3	N/A	N/A							N/A	
28	Cohesionless well-graded gravelly sand	0	N/A	40°	N/A	20.4 kN/m^3	compacted to 95% max. Standard Proctor density	N/A							N/A	
29	clean well graded gravelly sand	0	N/A	40°	N/A	20.4 kN/m^3	N/A	N/A							N/A	
30	well graded gravelly sand	N/A	N/A	36°	N/A	21.1 kN/m^3	N/A	N/A							N/A	
31	SW-SM(well-graded silty sand)	N/A	0	34°	N/A	21 kN/m^3	95% of maximum dry unit weight	N/A							N/A	
32	medium-fine uniformly graded sand with a trace of silt	N/A					placed at a water content between 5 and 10%, compacted to a dry denstiy greater than 92% max. dry denstiy	N/A							N/A	

Table T-2(c) – Continued

Soil																
Case No.	Reinforced Zone (Filled soil)							Retained Filled zone (Filled Soil)							Retained Cut zone (In-situ Soil)	
	Type	Fine (%) (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	% Fine (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	Strength
			c	Φ	H_2O%	Density				c	Φ	H_2O %	Density			
33	quarry crusher fines	N/A														
34	clay	N/A													silty clay to clayey silt limestone residual soil	N/A
35	poor drainage soil	N/A														
36	clay	N/A														
37	clay	N/A														
38	clay	N/A														
39	silty clay	N/A														
40	clayey silt	N/A														
41	sandy gravel	N/A														
42	clay	N/A														
43	clayey silt	N/A														
44	silty clay	N/A														

Table T-2(d) – Performance of the domestic wall and slope cases

Performance									
Case No.	Acceptibility	Deformation			Collapse				Design
		Description	Reason	Retrofit	Description	Reason	service life	Repair Method	
1	yes	outward movement at the crest during construction. Max. crest movement was appr. 1.3% of the height of the wall	construction activity	N/A	N/A	N/A	N/A	N/A	AASHTO and FHWA guidelines
2	yes	outward movement of about 0.2% of the height of the wall within the lower half of of the wall	construction activity	N/A	N/A	N/A	N/A	N/A	NCMA guidelines with long-term strength
3	no	several months after completion, vertical settlement of a section of wall, amounting to 1-2 inches; a bulge of about 8 feet in diameter extending about 4 inches out from the plane of the wall.	the movements were related to settlement caused by compression of the soil placed in the undercut, and the lateral distortion was a result of surface water overtopping the curb and the drop structure and running behind the block, forcing the block outward.	in late summer of 1999 (9 months after the wall was originally completed), destack and restack the wall	a small washout, consisting of a section of wall about 4 feet in depth and 10 feet wide, occurred adjacent to an access manhole.	due to improper grading, surface runoff overtopped the curb and infiltrated the backfill around the manhole and into the granular zone behind the back of the block causing soil particle washout and block face distortion	shortly after the wall was completed	rebuilt using granular fill and the original blocks	NCMA design software that included the block type, geogrid type
4	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	Bell et al.(1975), and Whitcomb and Bell (1979)
5	yes	displacement indicates nonlinear behavior	interaction effects such as stress transfer, relative motions, and arching	N/A	N/A	N/A	N/A	N/A	available reinforced soil mothodology (Guidelines, 1984)
6	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
7	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	AASHTO Interim Guidelines and AASHTO-AGC-ARTBA Task Force 27 Guidelines
8	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	stability was assessed by two commercially available computer programs: "MGWALL" (Bsthurst) and "STABL6" (Humphrey, Holtz)
9	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10	no	N/A	N/A	N/A	first there was a surface crack in the fill soil, 7 weeks later it failed suddenly and catastrophically	global stability shows a critical failure surface with a FOS of 1.2; compound failure planes show FOS equal to 1.13 and 1.12 for the preliminary and final wall designs, respectively	on August 18, 1992 (7 weeks after construction)	replaced in 1995 with a combination of MSEW and sheetpile wall	a computer program developed by the geogrid manufacturer
11	yes	some movement of the walls during the construction of the upper fill slope	N/A	N/A	N/A	N/A	N/A	N/A	some commercially available design and analysis software
12	yes	only a few hairline shrinkage cracks in the mortar facing after 24 years	N/A	N/A	N/A	N/A	N/A	N/A	Bell, Stilley, and Vandre, 1975; Bell and Steward, 1977

Table T-2(d) – Continued

Performance									
Case No.	Acceptibility	Deformation			Collapse				Design
		Description	Reason	Retrofit	Description	Reason	service life	Repair Method	
13	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	based on a system used by Wyoming DOT
14	yes	settlement and resulted localized cracking of the overlying pavement	initial placement of the geotextile too loosely, and/or possible poor compaction at the start of construction	cracks were repaired by the contractor	N/A	N/A	N/A	N/A	design methods described by Koerner (1996)
15	yes	bulging amounting to 6-7 inches was noted at the face of the wall in the lower lifts after 3 or 4 lifts had been completed.	a combination of details and materials used at the wall face, from which the vertical load resulted in a flexural failure of the wire in the baskets	N/A	N/A	N/A	N/A	N/A	N/A
16	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	"MGRSW" program
17	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	"MGRSW" program
18	yes	differential settlement is less than 2 inches	N/A	N/A	N/A	N/A	N/A	N/A	"MGRSW" program
19	yes	the highest section was about 6 inches lower in the middle that at each end during contruction, which was expected, and a few stress cracks in the untis	N/A	N/A	N/A	N/A	N/A	N/A	"Tenswal" program, which is based on Mohr-Coulomb failure theory and the assumption that the reinforced and retained fill develop a Rankine active state of stress
20	yes	slight bulges and undulations, and a few stress cracks in the units	N/A	N/A	N/A	N/A	N/A	N/A	"Tenswal" program, which is based on Mohr-Coulomb failure theory and the assumption that the reinforced and retained fill develop a Rankine active state of stress
21	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
22	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	procedures set forth in a Tensar Technical Note

Table T-2(d) – Continued

Performance									
Case No.	Acceptibility	Deformation			Collapse				Design
		Description	Reason	Retrofit	Description	Reason	service life	Repair Method	
23	yes	lateral movements of 25 to 50 mm which occurred during the first 1.5 years of service and within 1 m of the front of the wall. 18 years later, the movements ceased based on survey	this movement is attributed partly to downhill creep of the foundation that was seated on a relatively steep slope	N/A	N/A	N/A	N/A	N/A	N/A
24	yes	little wall movement	N/A	N/A	N/A	N/A	N/A	N/A	N/A
25	yes	the maximum strains at end of construction were appr. 0.3% located at the connections between panel facing and reinforcement layers. 8 years later, maximum geogrid strains were 0.5%, maximum deformation of the face at the top of the wall was appr. 65 mm.	N/A	N/A	N/A	N/A	N/A	N/A	N/A
26	yes	maximum lateral movement of the wall face during construction and the first year of service was 100 to 150 mm	N/A	N/A	N/A	N/A	N/A	N/A	N/A
27	yes	post-construction outward wall movement generated over the bottom half of the wall	a small percentage of the wall units have cracks in the face. This is attributed to manufacturing flaws and not wall movements	N/A	N/A	N/A	N/A	N/A	Christopher et al. (1989), AASHTO (1990)
28	yes	the measured maximum lateral movement was appr. 150 mm, the majority of which occurred during construction; and maximum total strains appr. 1.2%. Outward movement on the bottom half of the wall.	N/A	N/A	N/A	N/A	N/A	N/A	N/A
29	no	total lateral wall deformation was appr. 150mm together with maximum reinforcement strains of 2 to 3% at the end of construction. Following release of external support, the wall face deflected an additional 450mm at the crest. The soil exhibitd signs of failure (i.e., tension cracks in the backfill surface behind the wall face)	purposely underdesigned to produce an internal stability failure	N/A	N/A	N/A	N/A	N/A	N/A
30	yes	maximum lateral movement of the instrumented section was 140mm during construction and the wall movements appeared to be stable after one year	N/A	N/A	N/A	N/A	N/A	N/A	N/A
31	yes	most of the outward displacements at the wall facing occurred during the initial stages of backfill placement and compaction. In particular, placement of the bridge suprestructure induced a maximum outward displacement of front wall equal to 0.17% of wall height and a settlement of 0.29% wall height. post-construction outward displacments were small and with a decreasing rate with time. After 28 months, maximum outward displacement was 0.22% and maximum settlement was 0.18% of wall height, respectively	N/A	N/A	N/A	N/A	N/A	N/A	Colorado DOT and AASHTO design procedures
32	yes	time-dependent increase in lateral deformation	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table T-2(d) – Continued

Performance									
Case No.	Acceptibility	Deformation			Collapse				Design
		Description	Reason	Retrofit	Description	Reason	service life	Repair Method	
33	no	nearing completion, a bulge in the lower third of the wall was observed	N/A	N/A	a section of the timber facing rotated and fell from the face of the wall; the upper layers of geotextile reinforcement was pulled out without disturbing the overlying fill	the lower timber for attachment of the facing to the geotextile was not used and no compaction was performed on the backfill	shortly after completion	entirely rebuild the wall including the fabric inclusions and backfilling; the spacing was reduced to 0.3m	based on methods proposed by Bell and discussions at June 1987 NATO workshop
34	no	relatively shallow slumping of the sloped backfill followed by significantly slumping shortly after the initiation of seasonally heavy rainfall; various amounts of lateral wall movements	movement at the top wall due to the lack of stability of the backfill slope, which was not accounted for in the reinforcement design; movement at the base of the wall due to departures of geogrid placements from the construction drawings and uncompacted slide debris behind the geogrids	N/A	increased slumping of the sloped backfill proceded the collapse of the wall facing	in-service saturation and weakening of poorly compacted clay in the backfill slope combined with the omission of the upper layer of geogrid reinforcement	6 months	N/A	N/A
35	no	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	N/A	N/A	N/A
36	no	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	12 months	N/A	N/A
37	no	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	12 months	N/A	N/A
38	no	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	8 months	N/A	N/A
39	no	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	8 months	N/A	N/A
40	no	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	12 months	N/A	N/A
41	no	bulge at bottom	design deficiency	N/A	N/A	N/A	N/A	N/A	N/A
42	no	deformed throughout	design deficiency	N/A	N/A	N/A	N/A	N/A	N/A
43	no	bulge at top	design deficiency	N/A	N/A	N/A	N/A	N/A	N/A
44	no	bulge at top	design deficiency	N/A	N/A	N/A	N/A	N/A	N/A

Table T-2(e) – Literature references for the domestic wall and slope cases

Reference			
Case No.	Author	Source	Article
1	R.J.Bathurst, M.R. Simac	Proceedings of the 5th International Conference on Geotextiles, Geomembranes and Related Products, Singapore, Special Lecture & Keynote Lectures (Preprint),1994, pp. 29-54.	Geosynthetic Reinforced Segmental Retaining Wall Structures in North America
2	R.J.Bathurst, M.R. Simac	Proceedings of the 5th International Conference on Geotextiles, Geomembranes and Related Products, Singapore, Special Lecture & Keynote Lectures (Preprint),1994, pp. 29-54.	Geosynthetic Reinforced Segmental Retaining Wall Structures in North America
3	J. Paulson, W. Groff	Proceedings Geosynthetics Conference 2001, Portland, Oregon, February 12-14, IFAI Publ., Roseville, MN, pp. 399-408.	Case History: Destacking and Restacking a Geogrid Reinforced Segmental Retaining Wall
4	J.K. Mitchell, W.C.B. Villet	W.C.B. NCHRP Program Report 290, Transportation Research Board, National Research Council. 1987, pp.232-233	Reinforcement of earth slopes and Embankments
5	K.L Fishman, C.S. Desaio	Journal of Geotechnical Engineering, Vol.119, No.8, Aug.1993, pp.1293-1307	Field Behavior of Instrumented Geogrid Soil Reinforced Wall
6	U. Eliahu, S. Watt	Geotechnical Fabric Report, Vol 9, No.2, 1991, pp.8-13	Geogrid Reinforced wall withstands earthquake
7	J.L. King, W.J. Harper, E.D. Carlson	Proceedings of Geosynthetics, 1993 Conference, Vancouver, pp.137-152	Shotcrete-Faced Retaining Walls: Application in Flood-Control Channels-Blackland Gully, Kingwood, Texas
8	S.Kemp, J.S. Martin, A.T. Stadler	Proceedings of Geosynthetics, 1993 Conference, Vancouver,pp. 153-166	The Design and Construction of Geogrid-Reinfoced Retaining Walls at the South Carolina Port Authority's Wando Terminals
9	O.A. Moreno, J.L. King, R.A. MacDonald	Proceedings of Geosynthetics, 1993 Conference, Vancouver, pp. 167-180	From a Contractor's Viewpoint: Construction Technicalities of a Tiered Modular Block Wall-The Fiesta, Texas Case
10	R.R. Berg, M.S. Meyers	Proceedings of Geosynthetics, 1997, Conference, Long Beach, California, pp.85-104	Analysis of the Collapse of a 6.7 m High Geosynthetic-reinforced wall structure
11	M.R. Simac, R.J. Bathurst, T.W.Fennessey	Proceedings of Geosynthetics, 1997, Conference, Long Beach, California, pp. 105-120	Design of Gabion-Geosynthetic Retaining Walls on the Tellico Plains to Robbinsville Highway
12	D. Greenway, J.R. Bell, B. Vandre	Proceedings of Geosynthetics, 1999, Conference, Boston, Massachusetts, pp.905-920	Snailback Wall-First Fabric Wall Revised at 25-year Milestone

Table T-2(e) – Continued

Reference			
Case No.	Author	Source	Article
13	R.A.Reid, S.P.Soupir, V.R.Schaefer	Sixth International Conference on Geosynthetics Proceedings, March 25-29, 1998, Atlanta, Geogia, pp.573-576	Use of Fabric Reinforced Soil Walls for Intergral Bridge End Treatment
14	D. Chandler, T.Kirkland	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.755-764	Design and Construction of Geotextile Wall
15	R.D.Maynard, S.E.Thomsen, P.W.Grand	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.765-780	Geogrid Reinforced Soil Wall Bridge Replacement at Picnic Point, Snohomish County, Washington
16	M.R. Simac, R.J. Bathurst, R.A. Goodrum	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.781-798	Design and Analysis of Three Reinforced Soil Retaining Walls
17	M.R. Simac, R.J. Bathurst, R.A. Goodrum	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.781-798	Design and Analysis of Three Reinforced Soil Retaining Walls
18	M.R. Simac, R.J. Bathurst, R.A. Goodrum	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.781-798	Design and Analysis of Three Reinforced Soil Retaining Walls
19	R.B. Anderson, F.N. Boyd, L.Shaw	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.889-902	Modular Block Faced Polymer Geogrid Reinforced Soil Walls U.S. Postal Service Combined Carrier Facility
20	R.B. Anderson, F.N. Boyd, L.Shaw	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.889-902	Modular Block Faced Polymer Geogrid Reinforced Soil Walls U.S. Postal Service Combined Carrier Facility
21	D.J. Yovaish, S.W.Berry	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.903-916	Design and Construction of an 18 Foot High Geogrid Stayed Concrete Block Wall "A Case History"
22	J.C.Kliethermes, K.Butty, E.McCullough, R.Wetzel	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,Geogia, pp.951-964	Modular Concrete Retaining Wall and Geogrid Reinforcement Performance and Laboratory Modeling

Table T-2(e) – Continued

Reference			
Case No.	Author	Source	Article
23	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
24	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
25	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
26	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
27	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
28	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
29	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
30	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
31	N.Abu-Hejleh, J.G. Zornberg, T.Wang, J.Watcharamonthein	Geosynthetics International, 2002, Vol.9, No. 1, pp.71-79	Monitored Displacements of Unique Geosynthetic-Reinforced Soil Bridge Abutments
32	R.J. Fannin	Geosynthetics International, 2000, Vol.8, No. 1, pp.81-96	Long-term Variations of Force and Strain in a steep geogrid-reinforced Soil Slope

Table T-2(e) – Continued

Reference			
Case No.	Author	Source	Article
33	G.N.Richardson, L.H. Behr Jr.	GFR 6(4), pp14-18	Geotextile Reinforced Walls: Failure and Remedy
34	G.A. Leonards, J.D. Frost, J.D. Bray	Journal of Performance of Constructed Facilities, 1994 8(4), pp 274-292	Collapse of Geogrid Reinforced Retaining Structure
35	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
36	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
37	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
38	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
39	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
40	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
41	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
42	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
43	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls
44	R.M.Koerner, T.Y. Soong	Geotextiles and Geomembranes, Vol. 19, No. 6, Aug. 2001, pp359-385	Geosynthetic Reinforced Segmental Retaining Walls

Table T-3 (a) – General information for the international wall and slope cases

Wall												
Case No.	Reference (Author)	Completed Year	Facing Type	Wall Geometry						Type of Wall		Structure above Wall
				Site Location	Dimension (H/L, area)	Slope		Style	Wall Batter	Cut	Fill	
						Backfill	Front slope	Tiered or Set back				
1	M. Bernard	Jul., 1992	precast concrete unit	Quebec, Canada	"L" shaped, Max. Hight 5.3m	no slope		no	N/A	x		no
2	Tenax co.	N/A	vegetation	TaiChung, Taiwan	35m high	N/A		5m high 2V:1H slope with 2.5m wide berms	1H:1.73V	x		N/A
3	Tenax co.	N/A	vegetation	Iserlohn, Germany	17m high	N/A		N/A	1H:5.7V	N/A		N/A
4	M.A. Knight	1990	precast concrete panel	New Brunswick, Canada	6.1m/389.5m	no slope		no	1H:28.6V	x		no
5	R.J. Bathurst	1989	2.4m wide, 0.15m thick full height reinforced concrrete panal	Ontario, Canada	1.25-7.1m/125m	N/A		no	N/A	N/A		N/A
6	J.P. Gourc	1982	wrap-around woven geotextile with projected bituminous emulsion	Propoutel, France	2-10m/170m	N/A		no	1H:3.7V	N/A		N/A
7	J.P. Gourc	1984	wrap-around nonwoven needle-punched reinforcement geotextile	Pellafol, France	4.2m/22.5m	N/A		no	1H:11.4V	N/A		Road
8	J.P. Gourc	1986	wrap-around woven geotextile with projected bituminous emulsion	La Houpette, France	1.4m/350m	3H:2V	N/A	no	vertical	N/A		embankment
9	J.P. Gourc	1984	geotextile framework cubic container	Trouville, France	6m/60m	N/A	N/A	no	no	N/A		Road
10	J.P. Gourc	1983	old concrete wall	Langres, France	4m/150m	N/A	N/A	no	no	N/A		N/A
11	J.P. Gourc	1985	light concrete scale	Lixing, France	3-4.5m/120m	N/A		no	vertical	N/A		landslide zone
12	J.P. Gourc	1987	prefabricated cantilever wall	Luchon, France	5m/60m	N/A		no	vertical	N/A		mountain road on slope
13	J.P. Gourc	1986	geogrid and top soil facing	La Galaure, France	2.7m high	N/A		no	1H:1.2V	N/A		N/A
14	J.P. Gourc	1989	geogrid and top soil facing	Vienne,France	10.5m high	N/A		no	1H:1.2V	N/A		motorway embankment
15	J.P. Gourc	1990	precast concrete block	La Valentine, France	4m/25m	N/A		N/A	vertical	N/A		N/A
16	J.P. Gourc	1989	used tyre	Aigueblanche, France	7m/1000m	N/A		N/A	1H:1.73V	N/A		no

Table T-3 (a) – Continued

Wall											
Case No.	Reference (Author)	Completed Year	Facing Type	Wall Geometry						Type of Wall		Structure above Wall
				Site Location	Dimension (H/L, area)	Slope		Style	Wall Batter	Cut	Fill	
						Backfill	Front slope	Tiered or Set back				
17	J.P. Gourc	1982	geotextile with metal plate	Laragne, France	3.2m high	N/A		N/A	vertical	N/A		N/A
18	J.P. Gourc	1987-1988	grassed and planted	St.Laurent de Chamousset, France	8m high	N/A		N/A	1H:1.4V	N/A		road embankment
19	F.W.Gassner	1993	30cm deep dry stack concrete block	Gauteng Province, South Africa	3.6m high	no slope	N/A	no	1H:2.75V	N/A		N/A
20	F.W.Gassner	1993	concrete block	Gauteng Province, South Africa	up to 4.9m high	N/A		no	1H:2.75V	N/A		N/A
21	G.Sembenlli	Started in 1994 summer, stopped from 1994 winter through 1996 April, then resumed and completed by end of 1996	0.5-0.6m high, L shaped forms made by welded wire steel mesh, the surface finally hydroseeded and grassed	Italian Alps, Italy	27.5m/150m	N/A		stepped to create 5m wide sloping berm	1H:1.73V (30°) for each tier	N/A		10m high unreinforced Embankment
22	J.H.Dixon	1996	geogrid wrap-around (bodkin joint) with geotextile filter	Lake1, Bluewater, London, UK	typically 10m high in north side	N/A		no	1H:2.75V	N/A		road embankment
23	J.H.Dixon	1997	2.7m long, 0.6m high, 0.3m wide, site-cast concrete block (stepped vertical face)	Lake 2, Bluewater, London, UK	N/A	N/A		stepped	vertical	N/A		N/A
24	W.J.Burwash	1984	steel H-piles, timber lagging	Calgary, Alberta, Canada	north wall and northwest wall have a total lenth of 59.4m and up to 9m high.	N/A		no	vertical	N/A		asphalt surfaced parking lot
25	T.M. Allen	1987	welded wire facing unit	Olso, Norway	7.8m high	N/A		two tiers with a setback of 0.5m	1H:2V	N/A		N/A
26	T.M. Allen	1998	welded wire facing unit	Vicenza, Italy	7.5m high	N/A		two tiers with a setback of 0.5m	1H:12V	N/A		N/A
27	J.P. Gourc	N/A	segmental block	Bridge les Bains, France	upper part 8.4m high; lower part 6.7m high	N/A		two tiers	upper part vertical; lower part 1H:2V	N/A		N/A
28	C.C. Huang	1992	wrap-around	Taiwan	10m high	N/A		no	N/A	N/A		no
29	C.C. Huang	1992	wrap-around	Taiwan	8m high	N/A		no	N/A	N/A		highway
30	C.C. Huang	1992	segmental concrete panal	Taiwan	10m high	N/A		yes	N/A	N/A		N/A
31	R.P. Stulgis	N/A	modular block (Tensar Mesa wall)	Panama	7.6m high	N/A		no	N/A	N/A		bridge

Table T-3 (b) – Reinforcement information for the international wall and slope cases

Reinforcement						
Case No.	Reinforcement			Strength	Spacing	Length
	GG	GT	Style			
1	used in lower part	used in upper part	N/A			
2	x		Tenax TT 601 SAMP(HDPE)	N/A		
3	x		Tenax TT 701 SAMP(HDPE); Tenax LBO 220 SAMP(PP)	N/A	TT701: 0.45m in the lower part, 0.9m in the upper part; LBO220: used as secondary reinforcement	N/A
4	x		Tensar SR2 (HDPE); Tensar SR1 (HDPE)	SR2: 16kN/m; SR1: 8kN/m;	1.22m	3.4m
5	x		Tensar uniaxial (UX1600)	N/A	N/A	N/A
6		x	woven (PET)	200kN/m	1.2m	4-5m
7		x	woven (PET)	50kN/m	0.4m	4.5m
8		x	woven (PET)	120kN/m	0.7m	3.5m and 4m
9		x	woven (PET)	96kN/m	0.8m	4m
10		x	woven (PET)	80kN/m	0.4-0.6m	3m
11		x	woven (PET)	100kN/m	0.75m	3-5m
12		x	woven (PET)	217kN/m	0.8m	3-4m
13		x	woven (PET)	N/A	0.65m	3.5m
14		x	woven (PET)	100kN/m	0.25-0.6m	5m-7.5m
15		x	woven (PET)	150kN/m	N/A	N/A
16		x	woven (PET)	80kN/m	0.7m	4m

Table T-3 (b) – Continued

Reinforcement						
Case No.	Reinforcement			Strength	Spacing	Length
	GG	GT	Style			
17		x	woven (PP)	70kN/m	0.8m	N/A
18		x	waste plastics (PE, PP)	5-200kN/m	0.4m	N/A
19		x	woven (PET)	25kN/m	varying	1.6m
20		x	woven (PET)	upper part 25kN/m; lower part 50kN/m	varying	2.4m
21		x	woven (PET)	three grades of nominal tensile strength: 40, 100, 120 kN/m	0.3m in lower part, 0.5m in upper part	N/A
22	x		woven (PET)	40, and 80 kN/m, respectively	typically 0.6m in upper part	typically 20m
23	x		woven (PET)	80 kN/m	N/A	N/A
24	x		woven (PET)	peak tensile strength 29kN/m; long term design strength 29.2kN/m	up to 0.8m	up to 6.8m
25	x		woven (PET)	SR-55: 47kN/m; SS1: 19.6kN/m	typically 0.6m	1.5-3m
26	x		woven (PET)	58, and 23.7kN/m, respectively	typically 0.5m	2.2 m for PP georide wall; 2 m for HDPE wall
27	x		woven (PET)	short term strength 150kN/m	0.6m in upper wall; 0.3-0.6m in lower wall	6-8m in upper wall; 9m in lower wall
28	x		N/A			
29	x		N/A			
30	x		N/A	ultimate strength 20-30 kN/m	N/A	
31	x		Tensar geogrid	N/A	N/A	

Table T-3 (c) – Soil information for the international wall and slope cases

Soil																
Case No.	Reinforced Zone (Filled soil)							Retained Filled zone (Filled Soil)							Retained Cut zone (In-situ Soil)	
	Type	Fine (%) (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	% Fine (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	Strength
			c	ϕ	H_2O%	Density				c	ϕ	H_2O %	Density			
1	well graded sand (SW)	5	N/A				N/A	N/A							granular fill, bedrock	N/A
2	N/A	N/A	N/A				N/A	N/A							N/A	
3	N/A	N/A	N/A				N/A	N/A							N/A	
4	sand and gravel	12	0	35°	N/A		compacted to 90% of Modified Proctor max. dry density (ASTM D1557); 30cm lifts	same as reinforced zone							N/A	
5	N/A	N/A	N/A	30°-40°	N/A		N/A	N/A							N/A	
6	N/A	N/A	N/A	N/A	close to the liquidity limit	N/A	N/A	N/A							N/A	
7	gravel	N/A	N/A	N/A	N/A		N/A	N/A							N/A	
8	marly-limestone rip-rap	N/A	N/A	N/A	N/A		N/A	N/A							N/A	
9	dredged sand	N/A	0	30°	N/A		N/A	N/A							N/A	
10	crushed limestone	N/A	0	40°	N/A		N/A	N/A							N/A	
11	gritty sand	N/A	N/A	36°	N/A		N/A	N/A							N/A	
12	schist	N/A	10 kPa	35°	N/A		N/A	N/A							N/A	
13	gravel	N/A	N/A	41°	N/A		N/A	N/A							N/A	
14	alluvium	N/A	N/A	35°	N/A		N/A	N/A							N/A	
15	gravel	N/A	N/A	N/A	N/A		N/A	N/A							N/A	
16	scree material	N/A	N/A	N/A	N/A		N/A	N/A							N/A	

Table T-3 (c) – Continued

Soil															
Case No.	Reinforced Zone (Filled soil)							Retained Filled zone (Filled Soil)						Retained Cut zone (In-situ Soil)	
	Type	Fine (%) (< 0.075 mm)	Backfill soil properties				Compaction Method	Type	% Fine (< 0.075 mm)	Backfill soil properties			Compaction Method	Type	Strength
			c	ϕ	H_2O%	Density				c	ϕ	H_2O %	Density		
17	site clay	N/A	N/A	N/A	N/A		N/A	N/A						N/A	
18	N/A	N/A	N/A	N/A	N/A		N/A	N/A						N/A	
19	silty sand	N/A	6kPa	26.3°	PI varies between 24 and 32, and LL is above 43.	N/A	N/A	N/A						N/A	
20	clayey sand to gravelly silt	N/A	similar to Case 19			N/A	N/A	N/A						N/A	
21	well graded crushed rock with a relatively large sand and silt fractions	N/A	0	35°	N/A	21kN/m^3 (assumed in design)	N/A	N/A						N/A	
22	silty fine sand	N/A	0	31°	N/A	γ_{opt}=19.3kN/m^3 γ_{sat}=20.2kN/m^3	compacted in lifts to a depth of 60cm	N/A						N/A	
23	silty fine sand	N/A	0	31°	N/A	γ_{opt}=19.3kN/m^3 γ_{sat}=20.2kN/m^3	compacted in lifts to a depth of 60cm	N/A						N/A	
24	low plastic clay till (typical liquid limit of 30 and plastic limit of 15 and 25% sand, 50% silt and 25% clay sizes)	25	N/A	N/A	10.5% at placement, which is 4% dry of the optimum of 14.5%	N/A	compacted to a minimum of 95% standard Proctor dry density(there is 600mm wide zone of granular fill adjacent to the timber lagging for drainage)	N/A						N/A	
25	uniformly graded fine to medium sand	3	N/A	33°	N/A	17 kN/m^3	N/A	N/A						N/A	
26	clayey, sandy gravel	10	N/A	40°	N/A	21.1 kN/m^3	N/A	N/A						N/A	
27	cohesionless	N/A	0	35°	N/A	N/A	N/A	N/A						N/A	
28	clayey silt	N/A													
29	clayey silt	N/A													
30	clayey silt	N/A													
31	fine-grained soil	appr. 0-25%	N/A		PI bewteen 10 and 15	N/A									

Table T-3 (d) – Performance of the international wall and slope cases

Performance									
Case No.	Acceptibility	Deformation			Collapse				Design
		Description	Reason	Retrofit	Description	Reason	service life	Repair Method	
1	no	N/A	N/A	N/A	partial collapse along the northern face	combination effect of rain/snowmelt, lateral pressure, weathered bedrock, and design	less than 1 year	use concrete plug between the back of concrete unit and bedrock to eliminate the potential for washout and eliminate the lateral earth pressure; use 600mm high drainage stone at the base of the wall to capture any water filtration from the bedrock	N/A
2	yes	after 5 years, only minimal base and face movements were recorded	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	according DIN V 1054.100 (partial
4	yes	strains of geogrid are less than 1 % during construction	N/A	N/A	N/A	N/A	N/A	N/A	N/A
5	yes	maximum out-of-alignment movement of wall sections is about 1.2% of wall height after 1.5 years	mailnly due to quality of construction	N/A	N/A	N/A	N/A	N/A	FHWA guidelines (Christopher et al., 1989)
6	yes	considerable facing deformation	less compaction close to the facing	N/A	N/A	N/A	N/A	N/A	N/A
7	yes	sheet deformation of 2% after 9 months	N/A	N/A	N/A	N/A	N/A	N/A	N/A
8	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
9	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
10	yes	sheet strain about 1-1.5%	N/A	N/A	N/A	N/A	N/A	N/A	N/A
11	yes	sheet strain value less than 2.6% when structure was completed	N/A	N/A	N/A	N/A	N/A	N/A	N/A
12	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
13	yes	relative displacement of 20mm between geogrid facing and geotextile reinforcment	soil cannot be compacted properly close to the facing	N/A	N/A	N/A	N/A	N/A	N/A
14	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
15	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
16	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table T-3 (d) – Continued

Performance									
Case No.	Acceptibility	Deformation			Collapse				Design
		Description	Reason	Retrofit	Description	Reason	service life	Repair Method	
20	no	N/A	N/A	N/A	the facing of wall next to the field light mast collapsed over a 20m section. A 7m long section moved horizontally and opened up a wide tension crack near the top of the surcharge slope. Outward bulge in mid-height of the wall.	wrong spacing between the reinforcing layers was used	3 years	the low wall that had not deflected excessively was reinforced with either one, two, or three rows of 6m long soil nails, spaced at 1.6m horizontally. These soil nails consisted of 20mm diameter galvanised high tenile steel bar encased in grout. The callapsed wall sections and the sections where the backfill and wall had deflected excessively were rebuilt.	N/A
21	yes	horizontal displacement less than 100mm	N/A	N/A	N/A	N/A	N/A	N/A	N/A
22	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
23	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
24	no	The wall performed satisfactorily for 16 months when signs of settlement were first observed in the fill behind the wall. Conditions gradually deteriorated and over the next 22 months settlement of the backfill approached 0.9m in one area. The top of the retaining wall rotated outwards about the toe and a deflection of 310mm was recorded with a slope indicator over a 17 month period.	1. settlement of foundation soils and backfill; 2. internal erosion; 3. saturation of backfill	In June, 1987 approximately 3 years after commpletion of construction, the upper 6m of wall was replaced with a free standing 2H:1V slope.	N/A	N/A	N/A	N/A	N/A
25	yes	strain less than 1% and largely occurred during construction	N/A	N/A	N/A	N/A	N/A	N/A	N/A
26	yes	maximum strain in the reinforcement ranged up to 1.6% for HDPE and up to 4.2% for PP geogrid	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table T-3 (d) – Continued

Performance									
Case No.	Acceptibility	Deformation			Collapse				Design
		Description	Reason	Retrofit	Description	Reason	service life	Repair Method	
27	yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	Ultimate Limit States and Serviceability Limit States calculations associated partial factors (prepared for future standard in France)
28	no	N/A	N/A	N/A	global stability (outward sliding)	triggered by the reduction of strength of soil due to saturation and high water pressure within the soil mass after a 6-day heavy rainfall	N/A	N/A	N/A
29	no	N/A	N/A	N/A	internal instability	triggered by the pull-out of reinforcement near the toe of the wall due to the decrease of the strength and the increase of self-weight of the soil during rainfall period	N/A	N/A	N/A
30	no	N/A	N/A	N/A	geogrids break	strength reduction of the soil at high water content and the subsequent high lateral earth pressure, and low strength grid for steep sloping reinforcement	N/A	N/A	N/A
31	no	cracking at the base of the wall	hydraulic pressure	N/A	geogrids break at grid/block connection area and wall collapse at the top of the wall	excess porewater pressure buildup in the backfill as a result of heavy rains and improper drainage control	N/A	only recommendations are provided so far, which are: 1) tear down and rebuild the whole wall; 2) analyze all remaining walls and take measures to prevent future problems	N/A

Table T-3 (e) – Literature references for the international wall and slope cases

Reference			
Case No.	Author	Source	Article
1	M. Bernard	Proceeding Of GRI 12, 1998, pp.58-65	Necessity of proper site assessment using SRWs as a case history
2	not specified	www.tenax.com case history	A 35m high reinforced slope in Taichung City
3	not specified	www.tenax.com case history	17m high wall with an inclination of 80° and green facing
4	M.A. Knight, A.J. Valsangkar	Proceedings of Geosynthetics, 1993 Conference, Vancouver, pp.123-136	Instrumentation and Performance of a Tilt-Up Panel Wall
5	R.J. Bathurst	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.159-166	Case Study of a Monitored Propped Panel Wall
6	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
7	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
8	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
9	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
10	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
11	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
12	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
13	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
14	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
15	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
16	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures

Table T-3 (e) – Continued

Reference			
Case No.	Author	Source	Article
17	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
18	J.P. Gourc, Y.Matichard	Geosynthetic-reinforced Soil Retaining Walls, 1992, pp.131-152	Development of Geotextile Reinforcement Techniques in France - Application to Retaining Structures
19	F.W. Gassner, G.M.James	Sixth International Conference on Geosynthetics Proceedings, March 25-29, 1998, Atlanta, Geogia, pp.559-564	Failure of Two Fabric Reinforced Segmental Block Walls in South Africa
20	F.W. Gassner, G.M.James	Sixth International Conference on Geosynthetics Proceedings, March 25-29, 1998, Atlanta, Geogia, pp.559-564	Failure of Two Fabric Reinforced Segmental Block Walls in South Africa
21	G.Sembenelli, P.Sembenelli	Sixth International Conference on Geosynthetics Proceedings, March 25-29, 1998, Atlanta, Geogia, pp.619-624	The Verrand High Reinforced-Soil Structure
22	J.H.Dixon	Sixth International Conference on Geosynthetics Proceedings, March 25-29, 1998, Atlanta, Geogia, pp.637-642	Case Study. Bluewater Retail & Leisure Destination-Major reinfoced soil slopes to form steep sided new lakes
23	J.H.Dixon	Sixth International Conference on Geosynthetics Proceedings, March 25-29, 1998, Atlanta, Geogia, pp.637-642	Case Study. Bluewater Retail & Leisure Destination-Major reinfoced soil slopes to form steep sided new lakes
24	W.J. Burwash, J.D.Frost	Geosynthetics'91 Conference Proceedings, Feb. 1991, Atlanta,GA, pp.485-494	Case History of a 9m High Geogrid Reinforced Wall Backfilled with Cohesive Soil
25	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
26	T.M Allen, R.J. Bathurst, R.R. Berg	Geosynthetics International, 2002, Vol.9, No. 5-6, pp.395-450	Global Level of Safety and Performance of Geosynthetic Walls: An Historical Perspective
27	J.P.Gourc, R.Arab, H.Giraud	Geosynthetics International, 2001, Vol.8, No. 2, pp.163-191	Calibration and Validation of Design Methods for Geosynthetic-reinforced Retaining Structures Using Partial Factors
28	C.C. Huang	Recent Case Histories of permanent Geosynthetic-Reinforced Soil Retaining Walls, Tatsuoka & Leshchinsky, Balkema, Rotterdam, 1994, pp219-222	Report on Three Unsuccessful Reinforced Walls
29	C.C. Huang	Recent Case Histories of permanent Geosynthetic-Reinforced Soil Retaining Walls, Tatsuoka & Leshchinsky, Balkema, Rotterdam, 1994, pp219-222	Report on Three Unsuccessful Reinforced Walls
30	C.C. Huang	Recent Case Histories of permanent Geosynthetic-Reinforced Soil Retaining Walls, Tatsuoka & Leshchinsky, Balkema, Rotterdam, 1994, pp219-222	Report on Three Unsuccessful Reinforced Walls
31	R.P. Stulgis	Project from Geocomp Consulting	N/A

Table T-4 – Details of wall cases with unacceptable performance

Case no.	Reinforced zone soil			Deformation			Collapse		
	Soil type	Soil property	Compaction	Description	Reason	Retrofit	Description	Reason	Retrofit
D-#3	silty sand	$\Phi=30^{o}$, $\gamma=20.4kN/m^3$	soil compacted on wet side of optimum	several months after completion, vertical settlement of a section of wall, amounting to 1-2 inches; a bulge of about 8 feet in diameter extending about 4 inches out from the plane of the wall.	the movements were related to settlement caused by compression of the soil placed in the undercut, and the lateral distortion was a result of surface water overtopping the curb and the drop structure and running behind the block, forcing the block outward.	destack and restack the wall	a small washout, consisting of a section of wall about 4 feet in depth and 10 feet wide, occurred adjacent to an access manhole.	due to improper grading, surface runoff overtopped the curb and infiltrated the backfill around the manhole and into the granular zone behind the back of the block causing soil particle washout and block face distortion	rebuild using granular fill and the original blocks
D-#10	sand	$\Phi=32^{o}$, $\gamma=18.8kN/m^3$	N/A	N/A	N/A	N/A	first there was a surface crack in the fill soil, 7 weeks later it failed suddenly and catastrophically	global stability shows a critical failure surface with a FOS of 1.2; compound failure planes show FOS equal to 1.13 and 1.12 for the preliminary and final wall designs, respectively	replaced in 1995 with a combination of MSEW and sheetpile wall
D-#29	clean well graded gravelly sand	no finer less than 0.075mm, $\Phi=40^{o}$, $\gamma=20.4kN/m^3$	N/A	total lateral wall deformation was appr. 150mm together with maximum reinforcement strains of 2 to 3% at the end of construction. Following release of external support, the wall face deflected an additional 450mm at the crest. The soil exhibitd signs of failure (i.e., tension cracks in the backfill surface behind the wall face)	purposely underdesigned to produce an internal stability failure	N/A	N/A	N/A	N/A
D-#33	quarry crusher fines	N/A	N/A	nearing completion, a bulge in the lower third of the wall was observed	N/A	N/A	a section of the timber facing rotated and fell from the face of the wall; the upper layers of geotextile reinforcement was pulled out without disturbing the overlying fill	the lower timber for attachment of the facing to the geotextile was not used and no compaction was performed on the backfill	entirely rebuild the wall including the fabric inclusions and backfilling; the spacing was reduced to 0.3m

Table T-4 – Continued

Case no.	Reinforced zone soil			Deformation			Collapse		
	Soil type	Soil property	Compaction	Description	Reason	Retrofit	Description	Reason	Retrofit
D-#34	clay	N/A	N/A	relatively shallow slumping of the sloped backfill followed by significantly slumping shortly after the initiation of seasonally heavy rainfall; various amounts of lateral wall movements	movement at the top wall due to the lack of stability of the backfill slope, which was not accounted for in the reinforcement design; movement at the base of the wall due to departures of geogrid placements from the construction drawings and uncompacted slide debris behind the geogrids	N/A	increased slumping of the sloped backfill proceded the collapse of the wall facing	in-service saturation and weakening of poorly compacted clay in the backfill slope combined with the omission of the upper layer of geogrid reinforcement	N/A
D-#35	poor drainage soil	N/A	N/A	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	N/A
D-#36	clay	N/A	N/A	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	N/A
D-#37	clay	N/A	N/A	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	N/A
D-#38	clay	N/A	N/A	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	N/A
D-#39	silty clay	N/A	N/A	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	N/A
D-#40	clayey silt	N/A	N/A	N/A	N/A	N/A	Collapse	hydrostatic not considered enough in design	N/A
D-#41	sandy gravel	N/A	N/A	bulge at bottom	design deficiency	N/A	N/A	N/A	N/A
D-#42	clay	N/A	N/A	deformed throughout	design deficiency	N/A	N/A	N/A	N/A
D-#43	clayey silt	N/A	N/A	bulge at top	design deficiency	N/A	N/A	N/A	N/A
D-#44	silty clay	N/A	N/A	bulge at top	design deficiency	N/A	N/A	N/A	N/A

Table T-4 – Continued

Case no.	Reinforced zone soil			Deformation			Collapse		
	Soil type	Soil property	Compaction	Description	Reason	Retrofit	Description	Reason	Retrofit
I-#1	well graded sand (SW)	5% finer than 0.075mm	N/A	N/A	N/A	N/A	partial collapse along the northern face	combination effect of rain/snowmelt, lateral pressure, weathered bedrock, and design	use concrete plug between the back of concrete unit and bedrock to eliminate the potential for washout and eliminate the lateral earth pressure; use 600mm high drainage stone at the base of the wall to capture any water filtration from the bedrock
I-#19	silty sand	c=6kPa,Φ=26.3^{o}, PI varies between 24 and 32, and LL is above 43.	N/A	N/A	N/A	N/A	facing blocks failed as a result of the lower blocks being sheared by horizontal deformation of wall and backfill.	heavy rain induced hydraustic pressure and missing some of the reinforced geotextile layers.	the wall was rebuilt using an imported granular fill. The fill was reinforced with 5 layers of a woven polyester geotextile with a tensile strength of 50kN/m at a uniform depth of 2.5m into the fill.
I-#20	clayey sand to gravelly silt	similar to above	N/A	N/A	N/A	N/A	the facing of wall next to the field light mast collapsed over a 20m section. A 7m long section moved horizontally and opened up a wide tension crack near the top of the surcharge slope. Outward bulge in mid-height of the wall.	wrong spacing between the reinforcing layers was used	the low wall that had not deflected excessively was reinforced with either one, two, or three rows of 6m long soil nails, spaced at 1.6m horizontally. These soil nails consisted of 20mm diameter galvanised high tenile steel bar encased in grout. The callapsed wall sections and the sections where the backfill and wall had deflected excessively were rebuilt.

Table T-4 – Continued

Case no.	Reinforced zone soil			Deformation			Collapse		
	Soil type	Soil property	Compaction	Description	Reason	Retrofit	Description	Reason	Retrofit
I-#24	low plastic clay till	25% fines < 0.075mm	water content of 10.5% at placement, which is 4% dry of the optimum of 14.5%; compacted to a minimum of 95% standard Proctor dry density(there is 600mm wide zone of granular fill adjacent to the timber lagging for drainage)	The wall performed satisfactorily for 16 months when signs of settlement were first observed in the fill behind the wall. Conditions gradually deteriorated and over the next 22 months settlement of the backfill approached 0.9m in one area. The top of the retaining wall rotated outwards about the toe and a deflection of 310mm was recorded with a slope indicator over a 17 month period.	1. settlement of foundation soils and backfill; 2. internal erosion; 3. saturation of backfill	In June, 1987 approximately 3 years after commpletion of construction, the upper 6m of wall was replaced with a free standing 2H:1V slope.	N/A	N/A	N/A
I-#28	clayey silt	N/A	N/A	N/A	N/A	N/A	global stability (outward sliding)	triggered by the reduction of strength of soil due to saturation and high water pressure within the soil mass after a 6-day heavy rainfall	N/A
I-#29	clayey silt	N/A	N/A	N/A	N/A	N/A	internal instability	triggered by the pull-out of reinforcement near the toe of the wall due to the decrease of the strength and the increase of self-weight of the soil during rainfall period	N/A
I-#30	clayey silt	N/A	N/A	N/A	N/A	N/A	geogrids break	strength reduction of the soil at high water content and the subsequent high lateral earth pressure, and low strength grid for steep sloping reinforcement	N/A
I-#31	fine-grained soil	appr. 0-25% finer than 0.075mm; PI between 10 and 15	N/A	cracking at the base of the wall	hydraulic pressure	N/A	geogrids break at grid/block connection area and wall collapse at the top of the wall	excess porewater pressure buildup in the backfill as a result of heavy rains and improper drainage control	only recommendations are provided so far, which are: 1) tear down and rebuild the whole wall; 2) analyze all remaining walls and take measures to prevent future problems

Table T-5 – Summary of reinforced backfill properties for walls with unacceptable performance

Case no.	Failure	Soil in reinforcing zone	% passing 0.075 mm	Cohesion (kPa)	Friction angle, Φ
Domestic Cases					
D-#3	collapse	silty sand	N/A	N/A	30°
D-#10	collapse	sand	N/A	N/A	32°
D-#29	deformation	clean well graded gravelly sand	0	N/A	40°
D-#33	collapse	quarry crusher fines	N/A	N/A	N/A
D-#34	collapse	clay	N/A	N/A	N/A
D-#35	collapse	poor drainage soil	N/A	N/A	N/A
D-#36	collapse	clay	N/A	N/A	N/A
D-#37	collapse	clay	N/A	N/A	N/A
D-#38	collapse	clay	N/A	N/A	N/A
D-#39	collapse	silty clay	N/A	N/A	N/A
D-#40	collapse	clayey silt	N/A	N/A	N/A
D-#41	deformation	sandy gravel	N/A	N/A	N/A
D-#42	deformation	clay	N/A	N/A	N/A
D-#43	deformation	clayey silt	N/A	N/A	N/A
D-#44	deformation	silty clay	N/A	N/A	N/A
International cases					
I-#1	collapse	well graded sand	5	N/A	N/A
I-#19	collapse	silty sand	N/A	6	26.3^o
I-#20	collapse	clayey sand to gravelly silt	N/A	similar to above	
I-#24	deformation	low plastic clay till	25	N/A	N/A
I-#28	collapse	clayey silt	N/A	N/A	N/A
I-#29	collapse	clayey silt	N/A	N/A	N/A
I-#30	collapse	clayey silt	N/A	N/A	N/A
I-#31	collapse	fine-grained soil	appr. 0-25	N/A	N/A

Table T-6 : Index Properties for MSE Backfill from U.S. State Transportation Agency Survey Responses

		Percent passing by weight on specified Sieve																			Plasticity Index
REGION	RESPONDER	6 in. 150 mm	4 in. 100 mm	3.0 in. 75 mm	2.0 in 50 mm	1.5 in. 37.5 mm	1.0 in 25 mm	3/4 in. 19 mm	1/2 in. 12.7 mm	3/8 in. 9.5 mm	1/4 in. 6.3 mm	No. 4 4.75 mm	No. 8 2.36 mm	No. 10 2.0 mm	N0. 16 1.19 mm	No. 30 0.595 mm	No. 40 0.425 mm	No. 50 0.295 mm	No. 100 0.149 mm	No. 200 0.075 mm	
EASTERN	CT (ConnDOT)	---	100	---	---	55 to 95	---	---	---	---	25 to 60	---	---	15 to 45	---	---	5 to 25	---	10 to 10	0 to 5	0
	DC (DDOT)	---	100	---	---	---	---	60 to 95	---	---	---	40 to 75	---	65	---	---	10 to 45	---	---	2 to 15	6
	DE (DelDOT)	---	100	---	---	---	---	20 to 60	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 10	6
	MA (MHD)	---	100	---	---	100	100	100	---	---	---	40 to 75	---	---	---	---	8 to 28	---	---	0 to 10	20
	MD (MDOT)	---	100	---	---	0 to 100	100	0 to 95	---	---	---	0 to 10	---	0 to 5	---	---	0	---	---	0	0
	ME (MaineDOT)	---	100	---	---	0 to 100	100	0 to 100	---	---	70	---	---	---	---	---	---	---	---	0 to 10	6
	NH (NHDOT)	---	100	---	---	---	---	---	---	---	---	25 to 60	---	---	---	---	---	---	---	0 to 12	6
	NJ (NJDOT)	---	100	100	---	---	---	---	---	---	---	30 to 80	---	---	---	---	0 to 25	---	---	0 to 10	---
	NY (NYSDOT)	---	100	100	---	100	100	100	100	100	30 to 100	---	---	---	---	---	0 to 60	---	---	0 to 15	5
	PA (PENNDOT)	---	100	100	---	100	100	20 to 100	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 5	---
	RI (RIDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	7
	VA (VDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	6
	VT (VTRANS)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 12	20
	WV (WVDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	6
MIDWESTERN	IA (Iowa DOT)	---	100	---	---	100	100	100	100	100	100	100	20 to 100	---	---	---	---	---	---	0 to 10	---
	IL (IDOT)	---	100	100	---	100	100	100	100	100	100	94 to 100	60 to 100	---	35 to 65	---	---	30	17	0 to 8	---
	IN (INDOT)	---	100	---	---	70 to 100	---	45 to 90	35 to 85	---	---	20 to 65	10 to 50	---	---	---	---	---	---	0 to 8	---
	KS (KDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 5	6
	MI (MDOT)	---	100	---	---	100	60 to 100	---	---	---	---	---	---	---	---	---	---	---	0 to 30	0 to 10	6
	MN (Mn/DOT)	---	---	---	---	---	83.3	---	---	---	---	---	---	---	---	---	---	---	---	0 to 10	6
	MO (MoDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 10	6
	NE (NDOR)	---	100	---	---	100	100	100	---	---	---	---	---	---	---	---	---	---	---	0 to 15	6
	OH (ODOT)	---	100	100	---	100	100	100	100	100	100	60 to 100	---	---	---	---	---	---	---	0 to 7	6
	WI (WisDOT)	---	100	100	---	100	100	100	100	100	100	45 to 100	---	---	---	---	10 to 50	---	---	0 to 8	10
SOUTHERN	AL (ALDOT)	---	---	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 10	10
	FL (FDOT)	---	100	---	---	---	---	70 to 100	---	---	---	30 to 100	---	---	---	---	15 to 100	---	---	0 to 15	6
	GA (GDOT)	---	100	---	---	100	---	---	---	---	---	---	---	20 to 90	---	---	---	---	---	0 to 12	---
	KY (KYTC)	---	100	---	---	---	---	---	---	---	---	10 to ---	---	---	---	---	10	---	---	0 to 5	---
	LA (DOTD)	---	100	---	---	100	100	100	100	100	100	20 to 100	---	15 to 85	---	---	10 to 35	---	---	0 to 5	0
	MS (MDOT)	---	100	---	---	100	75 to 100	100	100	100	100	20 to 100	---	---	---	---	0 to 60	---	---	0 to 15	6
	NC (NCDOT)	---	100	100	---	100	100	95 to 100	25 to 45	---	---	---	---	0	---	---	0	---	---	0	---
	NM (NMDOT)	---	100	---	---	---	---	---	---	---	---	60	---	---	---	---	---	---	---	0 to 15	6
	SC (SCDOT)	---	100	---	---	---	---	---	---	---	---	60	---	---	---	---	---	---	0 to 30	0 to 15	6
	TN (TDOT)	---	100	---	---	---	---	---	---	---	---	0 to 60	---	---	---	---	---	---	---	0 to 15	6
	TX (TxDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	---
WESTERN	AZ (ADOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	6
	CA (CALTRANS)	100	---	78 to 100	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	---	0 to 25	10
	CO (CDOT)	---	100	---	---	---	---	---	---	---	---	30 to 100	---	---	---	---	---	10 to 60	---	5 to 10	6
	HI (HDOT)	---	100	---	---	---	---	---	---	---	---	0 to 60	---	---	---	---	---	---	---	0 to 9	---
	ID (ITD)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	6
	ND (NDDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	6
	NV (NDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	6
	OR (ODOT)	---	100	100	---	100	100	50 to 100	100	100	100	20 to 100	100	15 to 100	---	---	10 to 60	---	---	0 to 15	6
	SD (SD DOT)	---	100	100	---	100	---	---	---	0 to 15	---	---	---	---	---	---	---	---	---	0 to 10	2
	UT (UDOT)	---	100	---	---	---	---	---	---	---	---	---	---	---	---	---	0 to 60	---	---	0 to 15	6
	WA (WSDOT)	---	100	---	---	---	---	---	---	---	---	50 to 80	---	---	---	---	0 to 30	---	---	0 to 7	---

Table T-7 :Electro-Chemical Properties for MSE Backfill from U.S. State Transportation Agency Survey Responses

Region	State	pH Metallic Reinforcement Hi	pH Metallic Reinforcement Low	pH Geosynthetic Reinforcement High	pH Geosynthetic Reinforcement low	Chlorides Maximum (ppm)	Sulfates Maximum (ppm)	Resistivity Maximum (ohm-cm)	Organic Content Maximum (%)
EASTERN	Delaware (DelDOT)	10	5	10	5	100	200	3000	0
	Maine (MaineDOT)	10	5	9	4.5	100	200	3000	1
	Maryland (MDOT)	9	3	9	3	100	200	3000	1
	Massachusetts (MHD)	10	5	9	3	100	200	3000	1
	New Hampshire (NHDOT)	10	5	---	---	100	200	3000	---
	New Jersey (NJDOT)	10	5	---	---	100	200	3000	1
	New York (NYSDOT)	10	5	---	---	100	200	3000	---
	Pennsylvania (PENNDOT)	10	6	---	---	100	200	---	---
	Rhode Island (RIDOT)	10	5	9	3	100	200	3000	1
	Vermont (VTRANS)	10	5	9	3	100	200	3000	1
	Virginia (VDOT)	10	---	---	---	100	200	3000	---
	West Virginia (WVDOT)	10	5	10	5	100	200	3000	1
MIDWESTERN	Illinois (IDOT)	10	5	9	4.5	100	200	3000	1
	Indiana (INDOT)	10	5	---	---	100	200	3000	1
	Kansas (KDOT)	10	5	10	5	100	200	5000	1
	Michigan (MDOT)	10	5	10	5	100	200	3000	1
	Minnesota (Mn/DOT)	10	5	---	---	100	200	3000	1
	Missouri (MoDOT)	9	5	9	4.5	100	200	3000	1
	Nebraska (NDOR)	---	---	10	5	100	200	3000	---
	Ohio (ODOT)	9	4.5	9	4.5	100	200	3000	---
	Wisconsin (WisDOT)	10	4.5	10	4.5	100	200	3000	---
SOUTHERN	Florida (FDOT)	10	5	10	3	100	200	30	2
	Georgia (GDOT)	9.5	6	---	---	100	200	3000	0
	Kentucky (KYTC)	10	5	10	5	200	1000	3000	---
	Louisiana (DOTD)	10	4.5	10	4.5	100	200	3000	0.5
	Mississippi (MDOT)	10	5	10	5	100	200	3000	1
	New Mexico (NMDOT)	10	5	---	---	100	200	2500	1
	North Carolina (NCDOT)	9.5	4.5	9.5	4.5	---	---	5000	0.1
	South Carolina (SCDOT)	9	4.5	9	4.5	100	200	3000	1
	Tennessee (TDOT)	10	5	10	5	100	200	3000	1
	Texas (TxDOT)	10	5.5	10	5.5	100	200	3000	0
WESTERN	California (CALTRANS)	10	5.5	10	5.5	500	2000	1500	0
	Hawaii (HDOT)	9	5.5	---	---	100	200	3000	1
	Idaho (ITD)	9.5	4.5	9	3	100	200	3000	1
	Nevada (NDOT)	10	5	10	5	100	200	3000	1
	Oregon (ODOT)	9.5	4.5	9.5	4.5	100	200	5000	2
	South Dakota (SD DOT)	10	5	---	---	100	200	3000	0
	Washington (WSDOT)	10	5	9	4.5	100	200	3000	0

Table T-8: Mechanical Properties for MSE Backfill from U.S. State Transportation Agency Survey Responses

Region	State	Friction Angle (degrees)	Cohesion (psf)	Test Method
EASTERN	CT (ConnDOT)	34	---	See specifications
	DE (DelDOT)	34	0	AASHTO T236
	MA (MHD)	33	0	Direct Shear
	MD (MDOT)	34	---	Direct Shear
	ME (MaineDOT)	34	---	ASTM D30-80 AASHTO T236 on %<#10 sieve
	NH (NHDOT)	34	0	---
	NJ (NJDOT)	34	---	---
	PA (PENNDOT)	34	0	Direct Shear
	RI (RIDOT)	30	---	SPT-N correlations
	VA (VDOT)	34	0	AASHTO T236
	VT (VTRANS)	34	0	AASHTO T236
	WV (WVDOT)	34	0	Direct Shear
MIDWESTERN	IA (Iowa DOT)	32	0	---
	IL (IDOT)	34	---	AASHTO T 236 or T 296
	IN (INDOT)	34	---	AASHTO T236
	KS (KDOT)	34	---	AASHTO T236
	MI (MDOT)	34	---	AASHTO T236
	MN (Mn/DOT)	34	0	AASHTO T236
	MO (MoDOT)	34	---	AASHTO T236/T248
	NE (NDOR)	34	0	AASHTO T236
	OH (ODOT)	34	---	---
	WI (WisDOT)	30	---	Not Specified
SOUTHERN	AR (AHTD)	28	0	---
	KY (KYTC)	34	---	AASHTO T236
	LA (DOTD)	30	---	AASHTO T236
	MS (MDOT)	34	0	AASHTO T296
	NM (NMDOT)	34	0	AASHTO T236
	SC (SCDOT)	32	0	T-239 or T-234
	TN (TDOT)	34	0	AASHTO T236
	TX (TxDOT)	---	---	None
WESTERN	CA (CALTRANS	34	---	---
	CO (CDOT)	34	0	Direct Shear
	HI (HDOT)	34	---	AASHTO T236
	ID (ITD)	34	---	AASHTO T236
	ND (NDDOT)	34	0	AASHTO T236
	NV (NDOT)	34	0	AASHTO T236
	OR (ODOT)	34	0	not spec'd
	SD (SD DOT)	34	0	AASHTO T236
	UT (UDOT)	34	0	AASHTO T236

ATTACHMENT C: TASK 2 - SURVEY QUESTIONAIRE

SURVEY QUESTIONNAIRE **Jan. 2004**
SELECTING BACKFILL MATERIALS for MSE[a1] RETAINING WALLS
NCHRP Project 24-22

Name	
Title	
Agency/Company	

All questions relate to the earth backfill materials within the reinforced zone.

1. MSE Backfill Type

a. What specification are you using for material comprising the reinforced backfill zone?

	Specification			
Reinforcement Type	**FHWA/ AASHTO[a2]**	**NCMA[a3]**	**State/ Province**	**Other (Describe)**
Metallic				
Geosynthetic				

b. Soil Classification System, (i.e. AASHTO, USCS[a4], Other), if you answered State/Province or Other in 1.a. [ ]

c. Are there special requirements within 3 ft. of the face of the wall?

[ ]

d. What compaction requirements have you established for the reinforced backfill zone?

Percentage of Maximum Density, % [ ] by which Test Method: [ ] [ ]

Moisture Content (%+/- of Optimum Moisture Content): [ ]

Lift Thickness:

Loose Measure, in. (mm) [ ]
Max. compacted, in. (mm) [ ]

e. What level of QC/QA is required during placement of the reinforced backfill?

[ ]

Test Method for Compaction Conformance (i.e. nuclear, sand cone, method spec., other)	
Within 3 ft. of the Face	Outside 3 ft. of the Face

[a1] "Mechanically Stabilized Earth"
[a2] U.S. Federal Highway Administration/American Association of State Highway Officials
[a3] National Concrete Masonry Association
[a4] Unified Soil Classification System

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Attachment C: TASK 2 - SURVEY QUESTIONAIRE

2. MSE Backfill Properties

a. Index:

Reinforcement Type	Particle Size: 100 mm (4 in.) % passing	(3/4 in.) % passing	4.75 mm (#4 sieve) % passing	(#10 sieve) % passing	0.425 mm (#40 sieve) % passing	0.075 mm (#200 sieve) % passing	Plasticity Index (PI)
Metallic							
Geosynthetic							

Are different index properties specified, if the reinforced backfill will be placed in below freezing temperatures (i.e. winter conditions)?

If so, please indicate:

Reinforcement Type	Particle Size: 100 mm (4 in.) % passing	(3/4 in.) % passing	4.75 mm (#4 sieve) % passing	(#10 sieve) % passing	0.425 mm (#40 sieve) % passing	0.075 mm (#200 sieve) % passing	Plasticity Index (PI)
Metallic							
Geosynthetic							

b. Electro-chemical:

Reinforcement Type	pH	Chlorides (ppm)	Sulfates (ppm)	Resistivity (ohm-cm)	Organic Content (%)
Metallic	≤	≤	≤	≥	≤
	≥				
Geosynthetic	≤	≤	≤	≥	≤
	≥				
Test Method					
ASTM					
AASHTO					
Other					

Comments:

c. Mechanical:

Friction Angle (phi): (deg.) Test Method:

Cohesion: kN/m^2 (psf) Test Method:

If cohesion is not zero, how do you incorporate into MSE Wall design?

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ATTACHMENT C: TASK 2 - SURVEY QUESTIONAIRE

3. Use of "High Fines" and/or "High Plasticity" MSE Backfill, as opposed to "Standard" Backfill

a. What is your definition of "standard" backfill?

Index Properties As Listed in Response 2.a.		Other
Yes	No	

b. What is your definition of "high fines" backfill?

> 50% passing #200 sieve	
35% < passing #200 sieve < 50%	
15% < passing #200 sieve < 35%	
10% < passing #200 sieve <15%	
3% < passing #200 sieve <10%	
3% < passing #200 sieve	
Other	

c. What is your definition of "high plasticity" backfill?

Plasticity Index (PI) >

d. Do you allow "high fines" and/or "high plasticity" backfill to be used in the reinforced backfill zone of MSE Walls?

YES	NO

If NO, then proceed to Question #8.

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ATTACHMENT C: TASK 2 - SURVEY QUESTIONAIRE

4. Economic Implications of Using "High Fines" and/or "High Plasticity" Backfill

a. Specific cases of significant savings using "high fines" and/or "high plasticity" backfill?

	$/Volume		
	$/cu.yd.	$/cu. M	$/ton
Cost of "standard" reinforced backfill:			
Cost of "high fines" reinforced backfill			
Cost of "high plasticity" reinforced backfill			

b. What additional testing (during construction) and/or restrictions do you impose when using "high fines" and/or "high plasticity" backfill?

5. Unsatisfactory Performance Using "High Fines" and/or "High Plasticity" Reinforced Backfill

		Yes	No	# of Projects w/ Problem	# Projects Built
a.	Lateral deformation of wall				
b.	Vertical settlement of reinforced backfill				
c.	Collapse of wall				
d.	Facing Problems:				
	Movement/Cracking				
	Aesthetics/Staining				

e. Reinforcement damage (corrosion, etc.)
Explain:

f. Other:

g. Identify Index, Electro-chemical & Mechanical properties of reinforced backfill for walls that experienced problems. Identify reinforcement type.

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ATTACHMENT C: TASK 2 - SURVEY QUESTIONAIRE

6. Enhancement of "High Fines" and/or "High Plasticity" MSE Backfill

a. Density/moisture content properties?

b. Lime or cement treatment?

Yes		%	
No			

c. Blending with granular soil?

Yes		%	
No			

d. Other? Describe:

7. Special Drainage Provisions with "High Fines" and/or "High Plasticity" MSE Backfill

a. ~~External drainage:~~

	Yes	No
Back Drainage?		
Base Drainage?		

Provide Details!

b. Internal drainage?

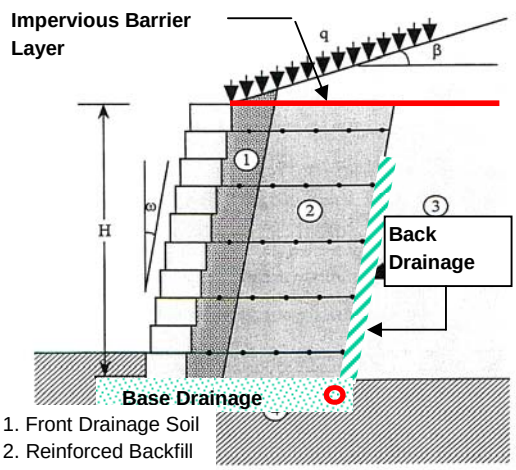
Yes	No

Provide Details!

c. Impervious barrier layer?

Yes	No

Provide Details!



1. Front Drainage Soil
2. Reinforced Backfill
3. Retained Soil

ATTACHMENT C: TASK 2 - SURVEY QUESTIONAIRE

8. Additional Comments

Please provide any additional comments or information on the use of "high fines" and/or "high plasticity" soils as backfill materials in the reinforced zone of MSE walls that you feel will be of benefit to this survey:

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ATTACHMENT - D

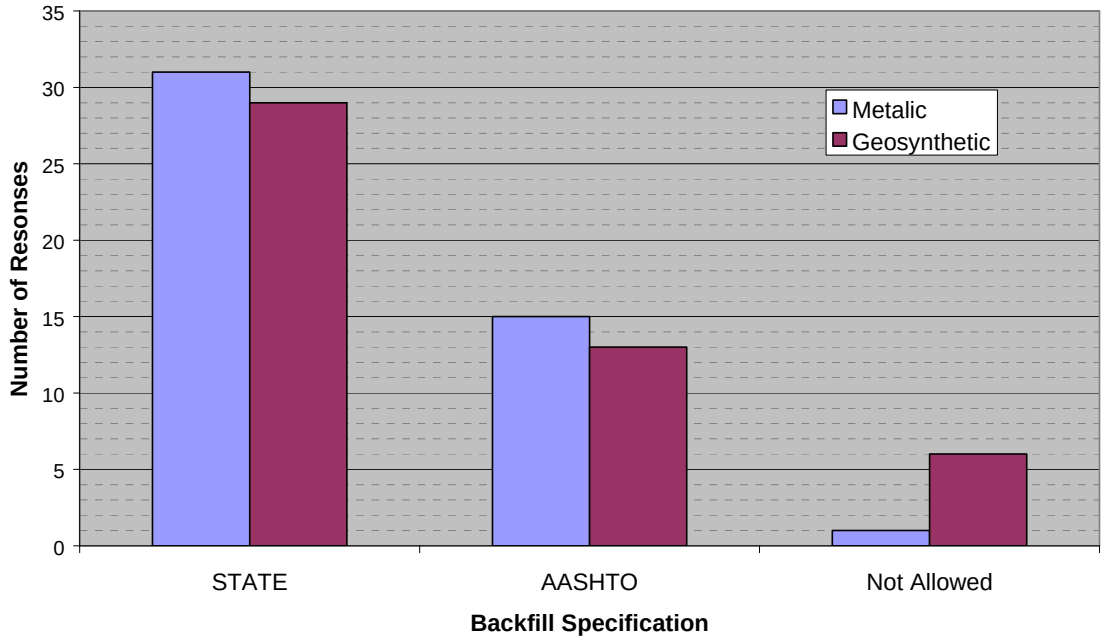
TASK 2 - SURVEY SUMMARY OF RESPONSES

This appendix presents summarizes excerpts from all the survey responses. Each section of the appendix corresponds to a section in the survey. The question from the survey is presented followed by either a graphic or tabulated summary of the survey responses.

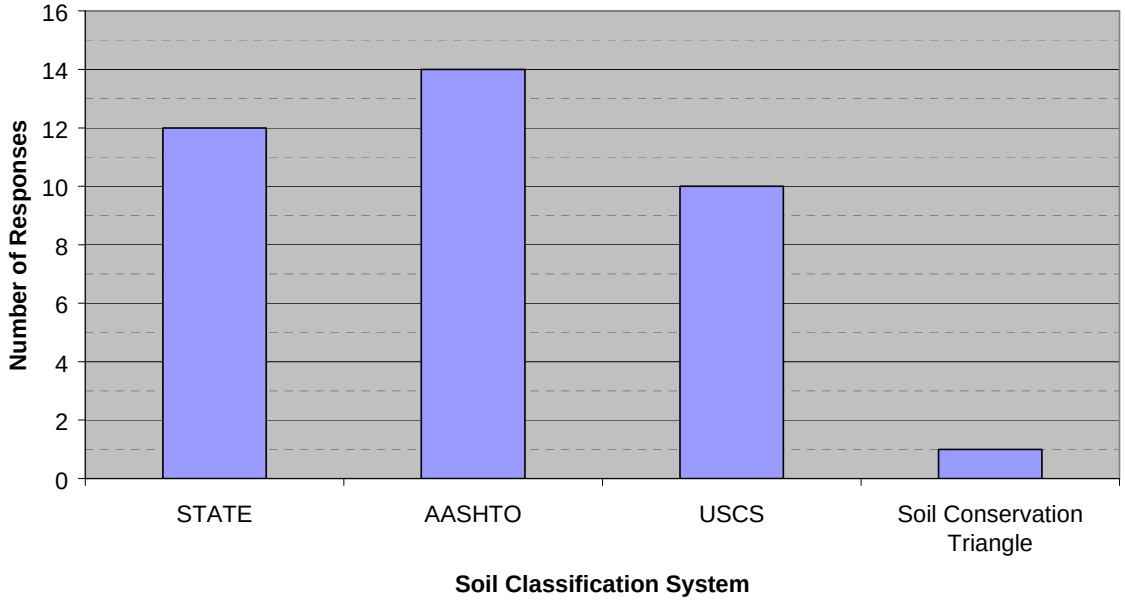
The information contained in this appendix is summarized and discussed in Chapter 2 of Appendix A.

ATTACHMENT D: Survey Response - Section 1 - MSE Backfill Type

1. MSE Backfill Type
a. What specification are you using for material comprising the reinforced backfill zone?



1. MSE Backfill Type
b. Soil Classification System, (i.e. AASHTO, USCS(a4), Other), if you answered State/Province or Other in 1.a.



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1c. Are there special requirements within 3 feet of the face of the wall?

As directed per manufacturer. Our MSE Walls are approved on a complete system basis.
12-inch wide drainage zone is required between the back of the wall face and the reinforcement zone material. This material shall meet AHTD Class 3 mineral aggregate specifications (crushed or uncrushed gravel, or crushed stone; 100% pass 1/2-inch, 80-100% pass 3/8-inch, 0-15% pass #10 sieve, 0-3% pass #16 sieve)
Yes, the material directly behind the face panel shall be a Class 1, Type B Permeable Material that shall only be compacted with hand-held or hand-guided compacting equipment. This material was specified to reduce the compactive effort required to place the material which reduces the amount of face panel distortion.
Granular material with filter fabric or pea gravel
See specifications
3 passes with a lightweight tamper or roller
No
no equipment heavier than 8 tons within 3 feet of the wall
Use light mechanical tampers (non-motorized) and compact the backfill without causing the panels to move outwards.
Use lightweight hand-operated compaction equipment.
No. Except for MSE segmental block walls that sometimes require a layer of drain rock within 0.3 to 0.6 m of the wall face.
The compaction of the select fill shall be achieved using a minimum of 3 passes of a light weight mechanical tamper, roller or vibratory system. Outside of this area the large roller that you would find on a bituminous paving operation or earth moving project may be used.
Achieved using a minimum of 3 passes of a light weight mechanical tamper, roller or vibratory system.
Compaction by light equipment with 3 feet, Max loose lift thickness 5" with 3 feet.
Compaction is to be done with light mechanical tampers. Compaction in this area is not subject to density testing.
Light Compaction Equipment Only
Yes, only hand operated compactors within 3' of face of wall. No less than 3 passes of a lightweight mechanical tamper, roller, or vibratory system.
Use of lightweight Mechanical Tampers only.
For geosynthetic reinforced backfills, a Under drain Backfill Material is specified for a distance of 1 foot from the back of the facing units. 95-100 passing 1 in, 75-100 passing 1/2 in, 50-100 passing #4, 15-80 passing #20, 0-15 passing #50, 0-5% passing #200. No special requirements for metallic reinforcing.
Compaction criteria, use of the light weight roller and placement of reinforcement fill material at the bottom initial 3 feet. Spreading the reinforced fill near the face of the wall.
On most projects, a free-draining gravel borrow is required
No
Compaction within 1 m (3 feet) of the back face of the concrete panels shall be achieved by means of a minimum of 3 passes with a lightweight mechanical tamper, roller, or vibratory system.
Unit Infill or Drainage Fill is required. Gradation Requirements define properties.
Compaction within 3 feet of the backfill face shall be achieved by at least 3 passes of a lightweight mechanical tamper, roller or vibratory system.
No use of heavy compaction equipment.
None

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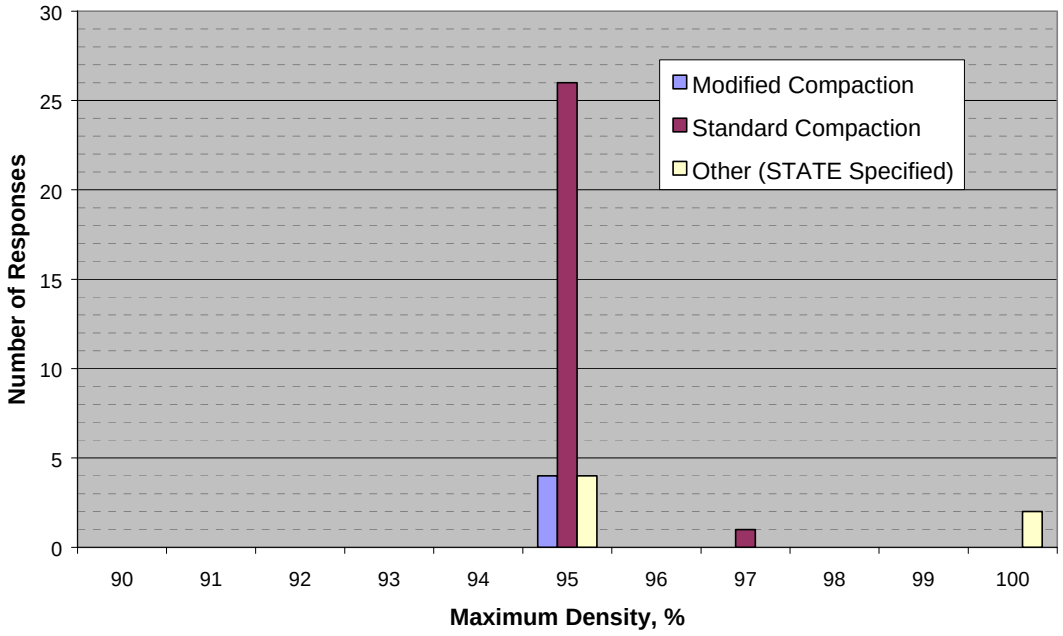
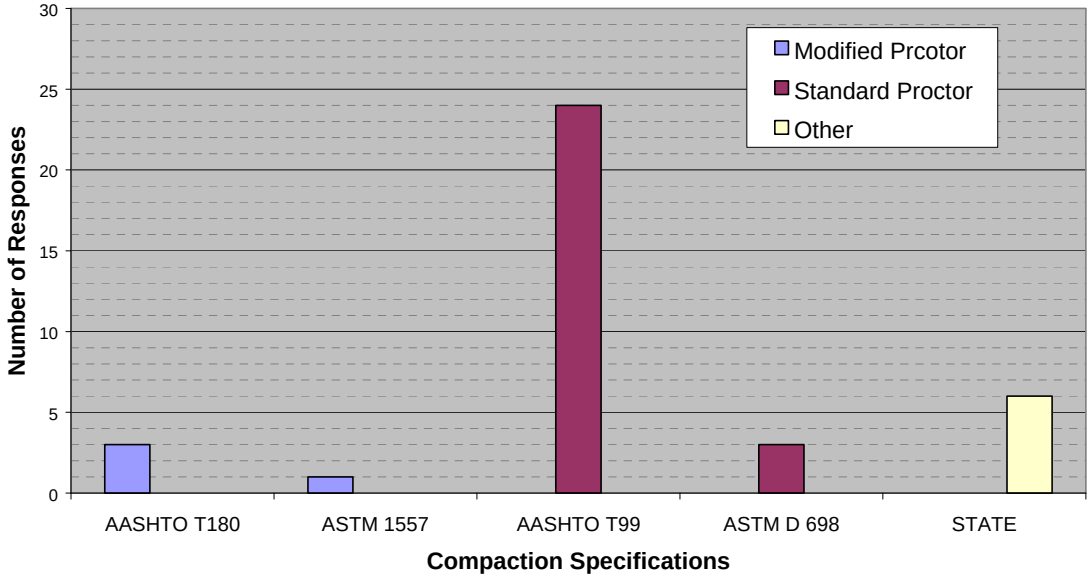
Same material and density requirements throughout the reinforced zone. Contractor is required to use lightweight compaction equipment w/in 3 feet of the wall face.
NJDOT specs require that within 5 feet of the wall small single or double drum, hand operated, wal-behind vibratory rollers or wall-behind vibrating plate shall be used, and as least three passes shall be made
90% Standard Proctor Density
None
No, not for the backfill
Compaction within 3 feet of the face of the wall shall be achieved by a minimum of three passes with a lightweight mechanical tamper or a vibratory plate compactor. All vehicles shall be prohibited from being within 3 feet of the face of the wall until the retaining wall is complete.
Compact the reinforced zone of fill without disturbing or distorting the soil reinforcement and facing panels. Place the embankment material within 3 feet of the backside of the panels in 6 to 8 inch thick lifts and compact this area by at least 3 passes of a light mechanical tamper. This area does not have to satisfy density test requirements.
A "low mass" mechanical tamper, roller or vibrator is specified. No further definition of low mass is made.
No
Hand operated compaction equipment and lower (4-in) lift thickness.
Modular concrete block facings require one foot of free draining aggregate behind the wall facing. Method specification. In order to determine the number of passes needed to compact the area within 3 feet of the back face of the wall to 95 percent of the maximum dry density, the Contractor shall establish a test strip area (3 feet from wall facing) measuring a minimum of 3 feet by 5 feet within the reinforced backfill and compact it with lightweight walk behind vibratory plate or roller.
None Three passes of a lightweight mechanical tamper system
The use of a "lightweight mechanical tamper, roller, or vibratory system" is required
Limit the compaction equipment to hand-operated or walk-behind in the 3' wide strip adjacent to wall panels.
Special compaction procedures
Compaction is to be achieved with a lightweight mechanical tamper, roller or vibratory system. Light thickness is adjusted as warranted by the type of compaction equipment
No Material requirements within 3 feet of wall, must use mechanical hand tampers
In most cases, no.
MSE Panel-faced: None MSE Modular block-face: No tracked or wheeled equipment may operate on the backfill within 3 feet from the back face of the modular blocks.
Only if recommended by the supplier. Our Special Provision restricts the use of only hand operated compaction equipment within 2.0 feet from the wall face.

ATTACHMENT D: Survey Response - Section 1 - MSE Backfill Type

1. MSE Backfill Type
d. What compaction requirements have you established for the reinforced backfill zone? Percentage of Maximum Density, %



1. MSE Backfill Type
d. What compaction requirements have you established for the reinforced backfill zone? by which Test Method:



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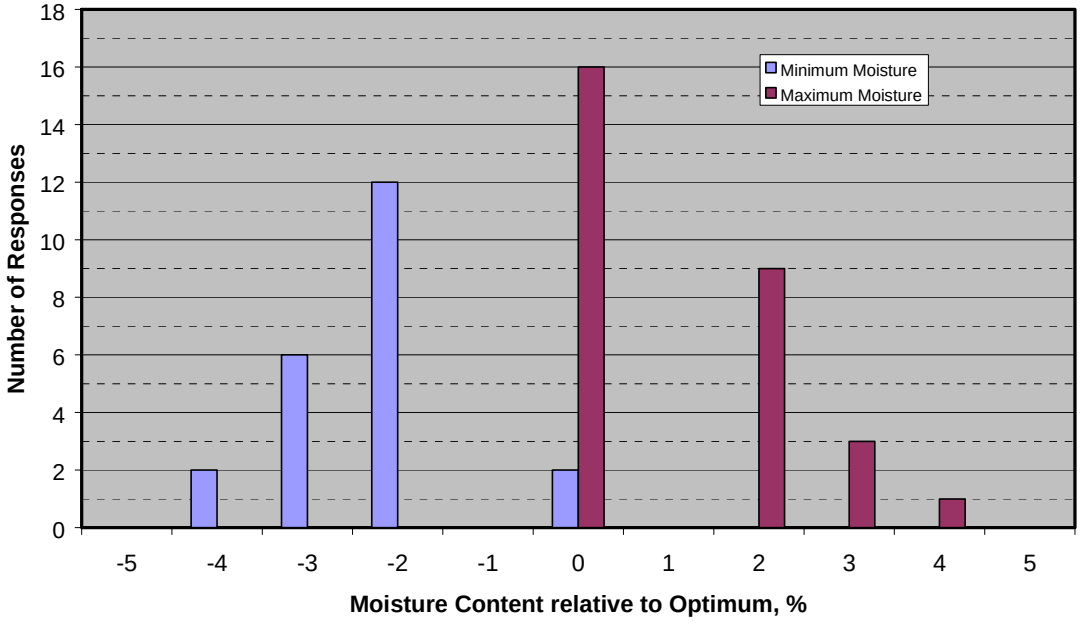
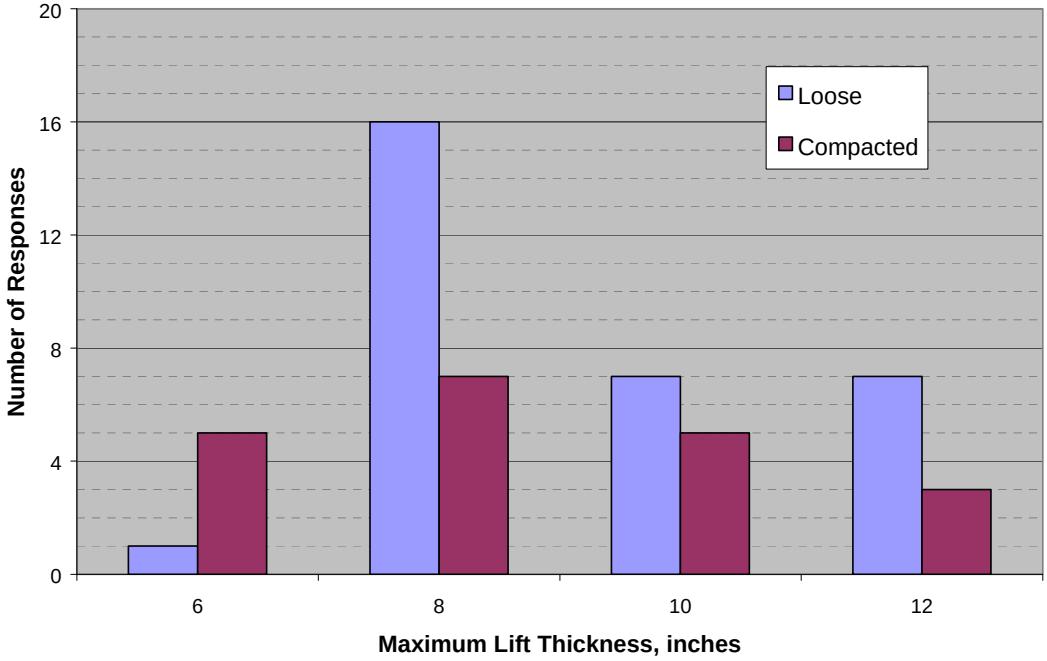
ATTACHMENT D: Survey Response - Section 1 - MSE Backfill Type

1. MSE Backfill Type
What compaction requirements have you established for the reinforced backfill zone?
Moisture Content (%+/- of Optimum Moisture Content):



1. MSE Backfill Type
d. What compaction requirements have you established for the reinforced backfill zone? Lift Thickness



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ATTACHMENT D: Survey Response - Section 1 - MSE Backfill Type

1e. What level of QC/QA is required during placement of the reinforced backfill?

Each row in the following table presents a response.

As Directed per manufacture
The Contractor shall perform a minimum frequency of acceptance testing shall be one test for density and moisture content for each 3000 cubic yards of backfill material placed and at least one set of tests shall be performed on each layer of backfill.
Yes, the material directly behind the face panel shall be a Class 1, Type B Permeable Material that shall only be compacted with hand-held or hand-guided compacting equipment. This material was specified to reduce the compactive effort required to place the material which reduces the amount of face panel distortion.
For on-site material consists more than 30% retained on 3/4" sieve, a method of measuring compaction consisting of conventional heavy vibratory roller starting with minimum 5 passes shall be used to establish the number of passes required that exceeds 95% T 180.
See specifications
Contractor performs QC; state performs QA
One Set of density tests (i.e. one test within 3 feet of wall and one test more than 3 feet from wall) per LOT (a single lift of finished embankment not to exceed 500 feet)
Inspection by project engineer and random compaction checks by testing management section.
One compaction test per lift per 500 sq. ft.
One density test per 2000 cu.meter of fill material placed
None
Select fill is treated as embankment which requires one (QA) density test per 20,000 cu yd for a continuous operation. In confined areas, 1 test per 3 ft of lift and not less than 1 test per fill area.
We use a "Materials method" specification for the placement of the backfill. We have no QA/QC program for this purpose. We require density tests on each lift.
QC each lift; QA assurance testing as determined by DOT field Materials staff.
density testing every lift
If applicable, inspector will measure moisture and density of backfill material. Otherwise, backfill will be accepted by visual observation.
One test per every 5' of wall height for every 100' of wall (compaction)Gradation and pH every 1000 cu yds.Friction angle, organic content, resistivity, sulfates and chloride content - every 5000 cu yds
One compaction test every other lift
Certify the source and random sampling to verify the compliances with Specification and the certification criteria. Passing of No. compaction machine. Because of the use of the No. 57 stone no compaction control is specified for the placement of the reinforced fill material
Compaction method and Compaction test results
one test per 300 cu yd
Field sampling and testing procedures shall be in accordance with Mn/DOT sampling/ testing procedures found in the Mn/DOT Grading and Base Manual and in the Schedule of Materials Control. Laboratory procedures shall be in accordance with Mn/DOT Laboratory Testing Manual.
Testing of Backfill is done based on a lot basis. Lot size is based on the Contractor's hourly production.
As directed by the Engineer.
Testing frequency as directed by Construction Project Engineer

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ATTACHMENT D: Survey Response - Section 1 - MSE Backfill Type

Refer to attached spec, Sec.520.03.O and Section 203.10 Density Control Method
FHWA Guidelines for testing frequencies.
Backfill is stockpiled, sampled and tested for gradation, soundness and chemical properties. Inspectors verify lift thicknesses. Density is checked every 350-400 cubic meters. One sample is taken from each structure location and tested in the lab to verify the properties match those determined during the stockpile evaluation process.
Method Spec. Up to Engineer to determine desired degree of compaction based on rutting or displacement from compaction equipment.
Wall supplier will provide a company representative to monitor the precast operation and ensure that the requirements of the specifications are met. Independent soils consultant hired to ensure placement and compaction of select granular backfill is in compliance with requirements of the specifications. The soil consultant provides the Engineer with 2 copies of all inspection reports signed by an Ohio registered PE. The Engineer will inspect the material delivered to the site; review certified test data; monitor erection of the structure and placement of the select granular material along with consulting with the soils consultant and wall supplier as necessary for acceptance to the requirements of the relevant specifications.
1 density test per every 100 cubic meters of backfill
PA Test Method (PTM) 112 (sand cone) or 402 (nuclear), essentially same as AASHTO/ASTM standards. 100% in top 3 feet
Field density tests by RIDOT Materials section. Certificates of Compliance for materials/fill from Contractor, and copies of all test results of tests performed by the Contractor.
Compaction control testing of the reinforced backfill shall be performed with a minimum frequency of one density test per every two lifts for every 25 feet of wall at bridge abutments (including the first 100 feet of wall parallel to the roadway) and every 100 feet of wall along roadways (more than 100 feet away from bridge abutments). Compaction within 3 feet of the back face of the wall shall be compacted by a Method Specification which shall be achieved by at least three passes of a lightweight walk behind vibratory plate or roller.
The engineer has the option of decreasing lift thickness depending on Contractor's operation and compaction equipment. Lift thicknesses for large panel walls are a max. of 12 inches and for modular block walls is 8 inches.
Currently under revision
Gradation, Resistivity and pH every 5000 cy., Soundness as directed by the engineer, In-place Density one per backfill lift. For walls greater than 500' in length, perform one in-place density test per lift for every 500' in length.
State will take at least one density test on every lift for every 30 m of retaining wall
Compaction testing is conducted on every third lift as a minimum.
MSE walls less than 100 linear feet, a minimum of one test every other lift. The testing will be performed a minimum of 8 feet away from the backface of the wall, to within three feet of the back edge of the zone of select fill area. Stagger the test sites throughout the length of the wall to obtain uniform coverage. Testing will begin after the first two lifts of select ill have been placed and compacted. Walls more than 100 linear feet, a minimum of two test every other lift not to exceed 200 linear feet.
Quality control of the select granular material is the responsibility of the contractor as specified in the WVDOH standard specifications. The contractor shall maintain equipment and qualified personnel to perform all sampling and testing necessary to comply with the Specifications. Compaction is achieved by vibratory roller. Select granular material shall be compacted to 95% of the maximum dry density, moisture content is not a requirement.

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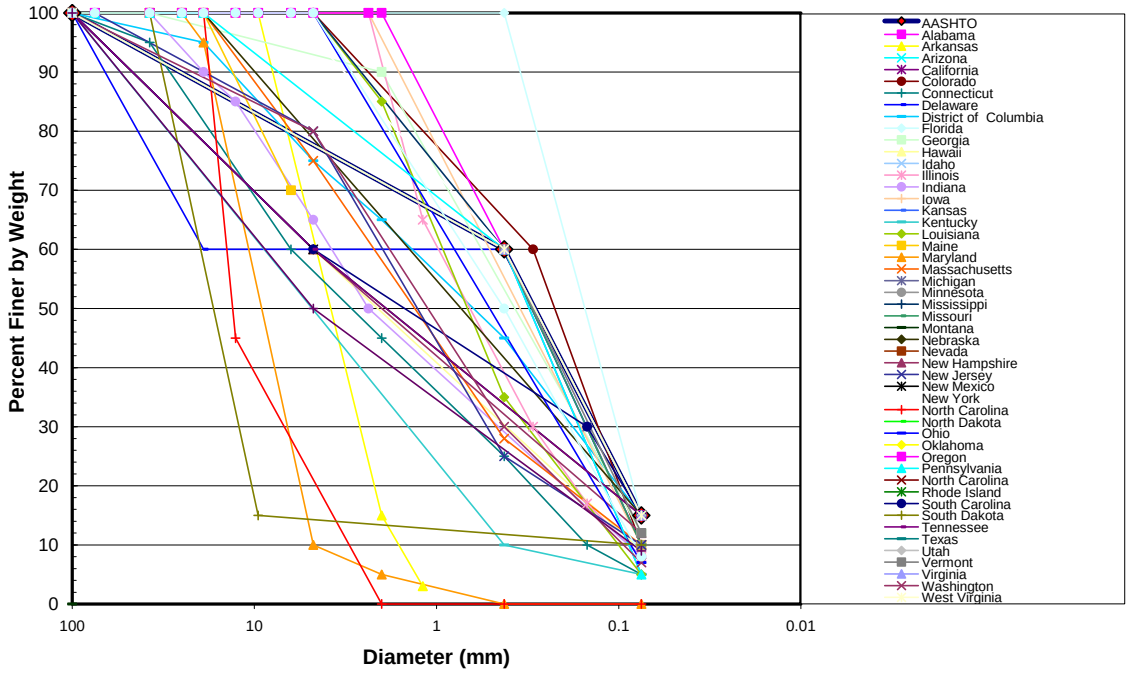
ATTACHMENT D: Survey Response - Section 1 - MSE Backfill Type

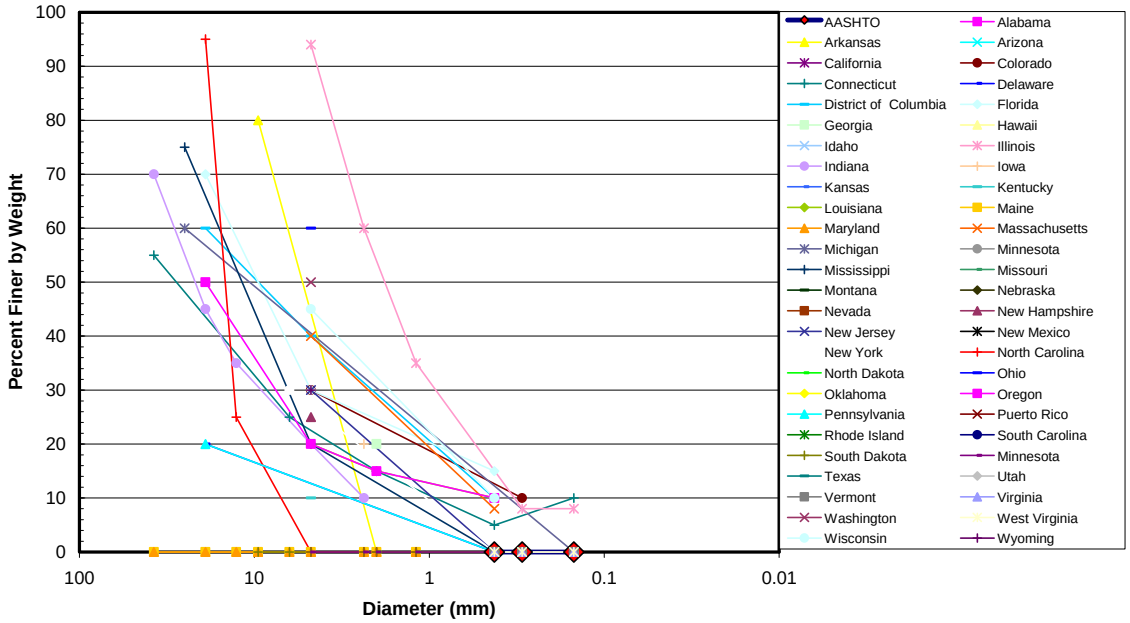
Compaction performed by the contractor (QC) is a minimum frequency of 1 test per 2 foot layer per 200 feet of wall. Departmental QA is minimal.
If 60% or less retained on the 4.75 mm Sieve. If greater than 60% retained on the 4.75 mm Sieve then have a method spec taking relative density counts with a nuclear density gauge to determine number of passes to get maximum density.

ATTACHMENT D: Survey Response - Section 2 MSE Backfill Properties

2a. MSE Backfill Properties
Index-maximum percent passing



2a. MSE Backfill Properties
Index - minimum percent passing



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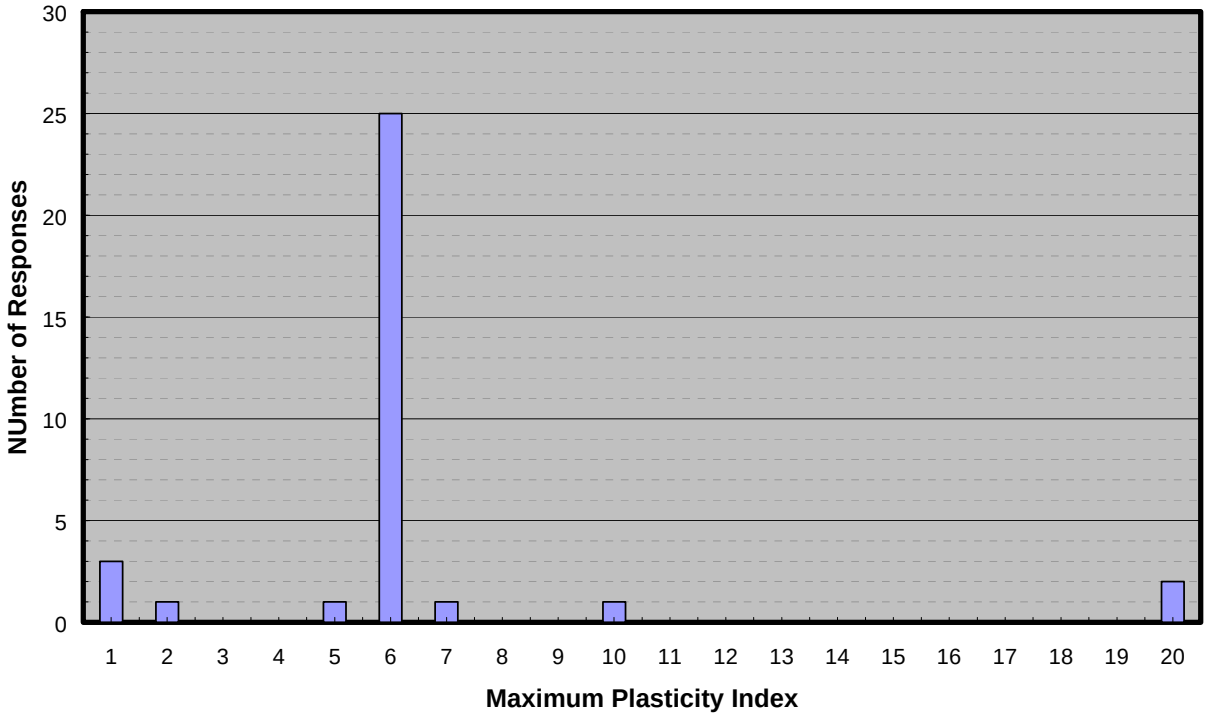
ATTACHMENT D: Survey Response - Section 2 MSE Backfill Properties

2a. Maximum Plasticity Index



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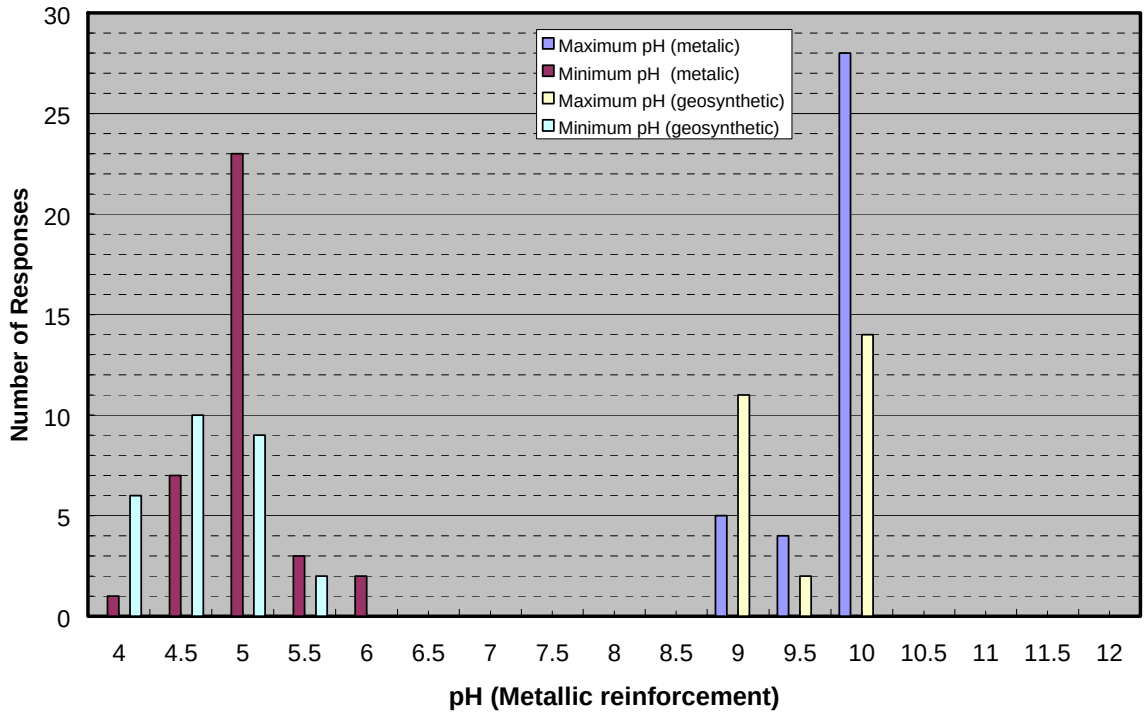
2b. Electro-chemical - pH



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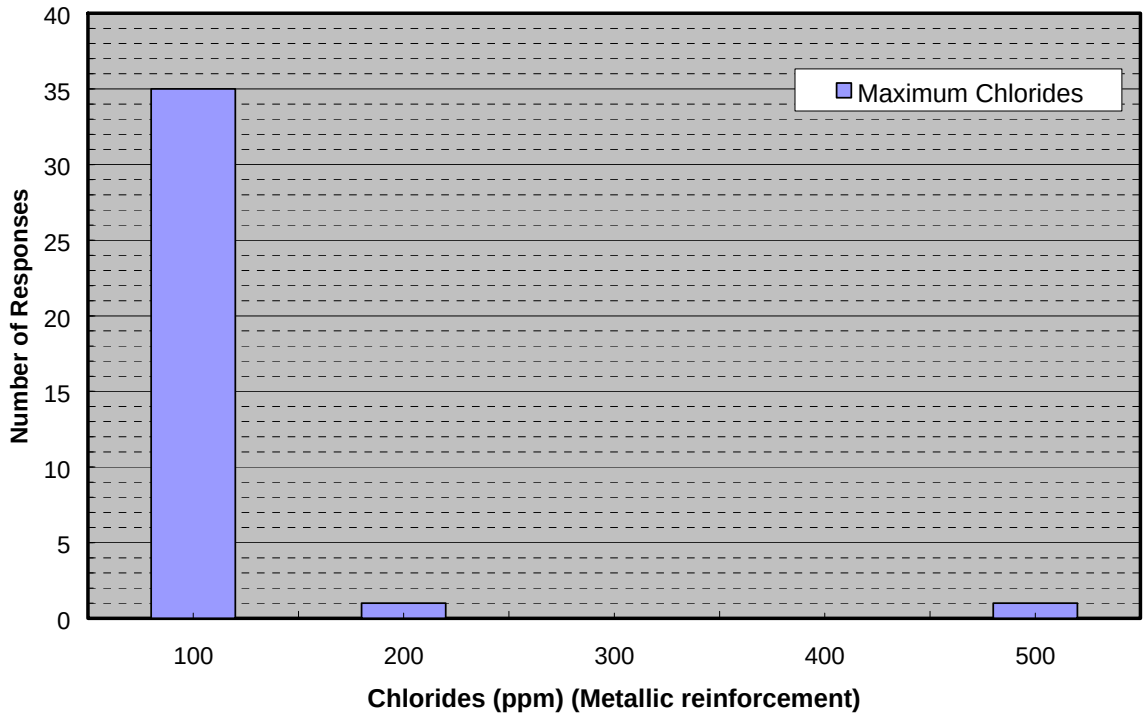
2b. Electro-chemical - Chlorides



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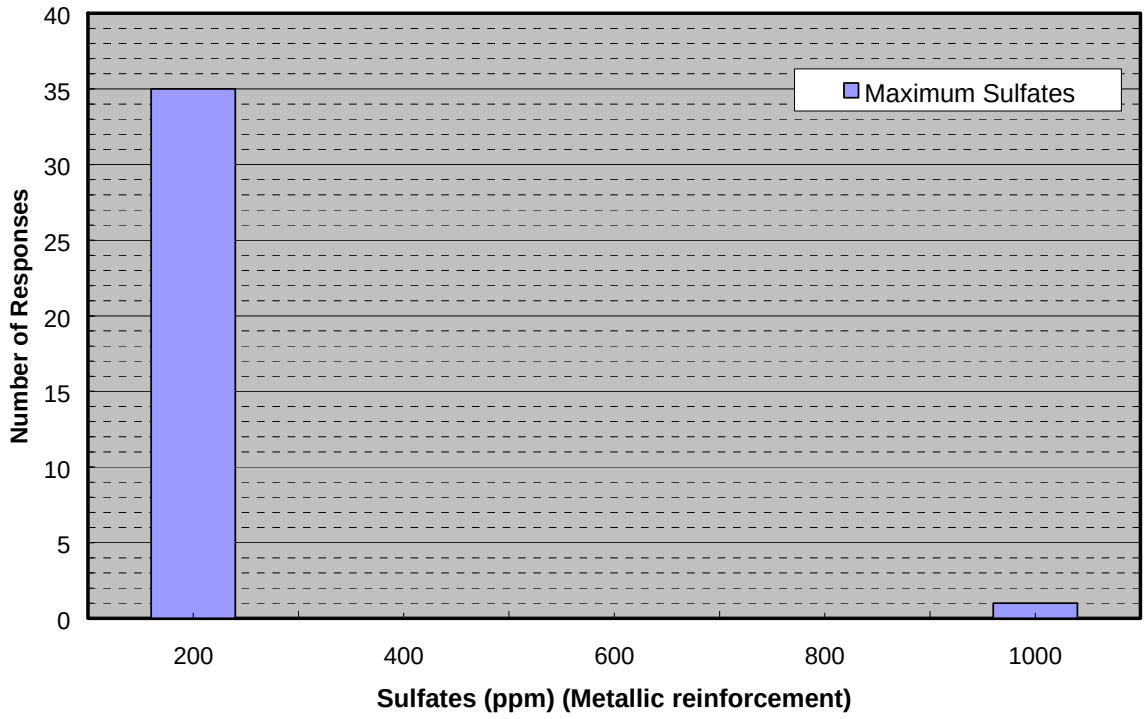
2b. Electro-chemical - Sulfates



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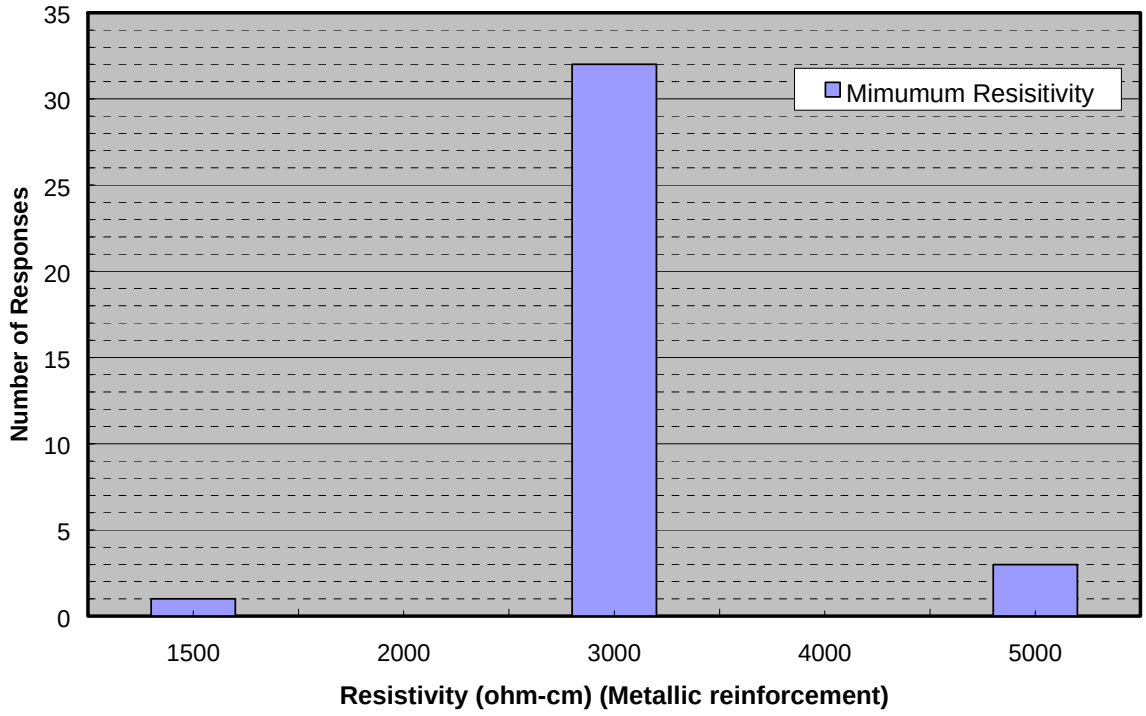
2b. Electro-chemical - Resistivity (ohm-cm)



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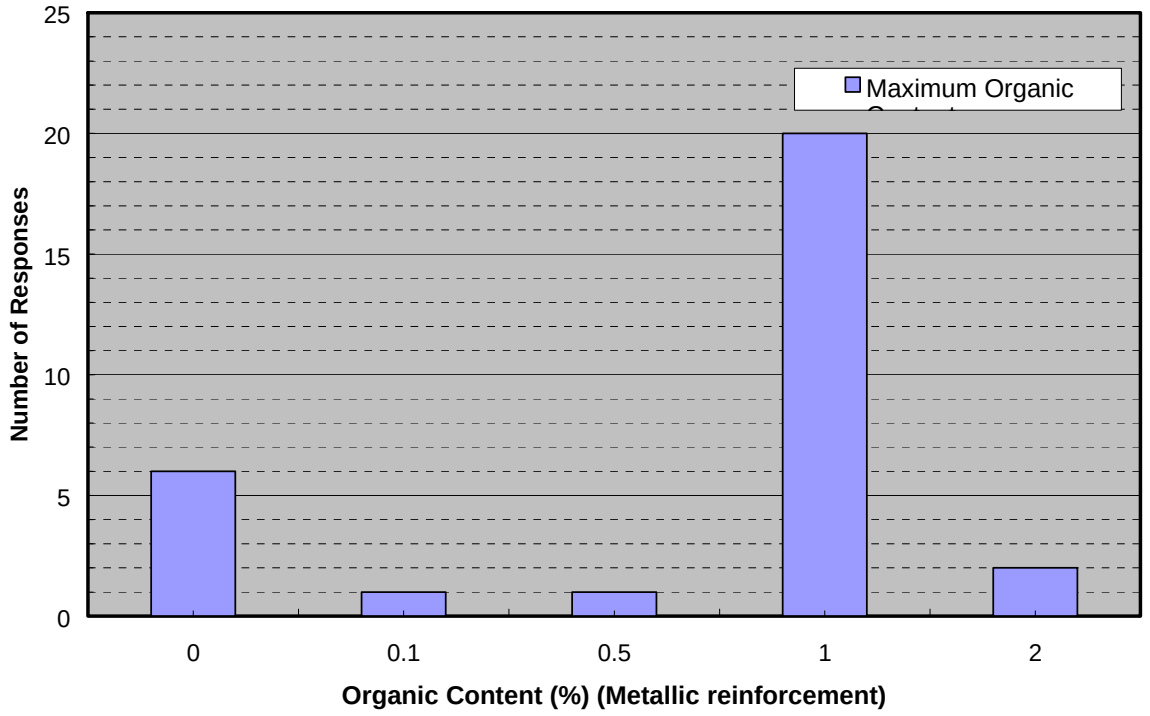
2b. Electro-chemical - Organic Content (%)



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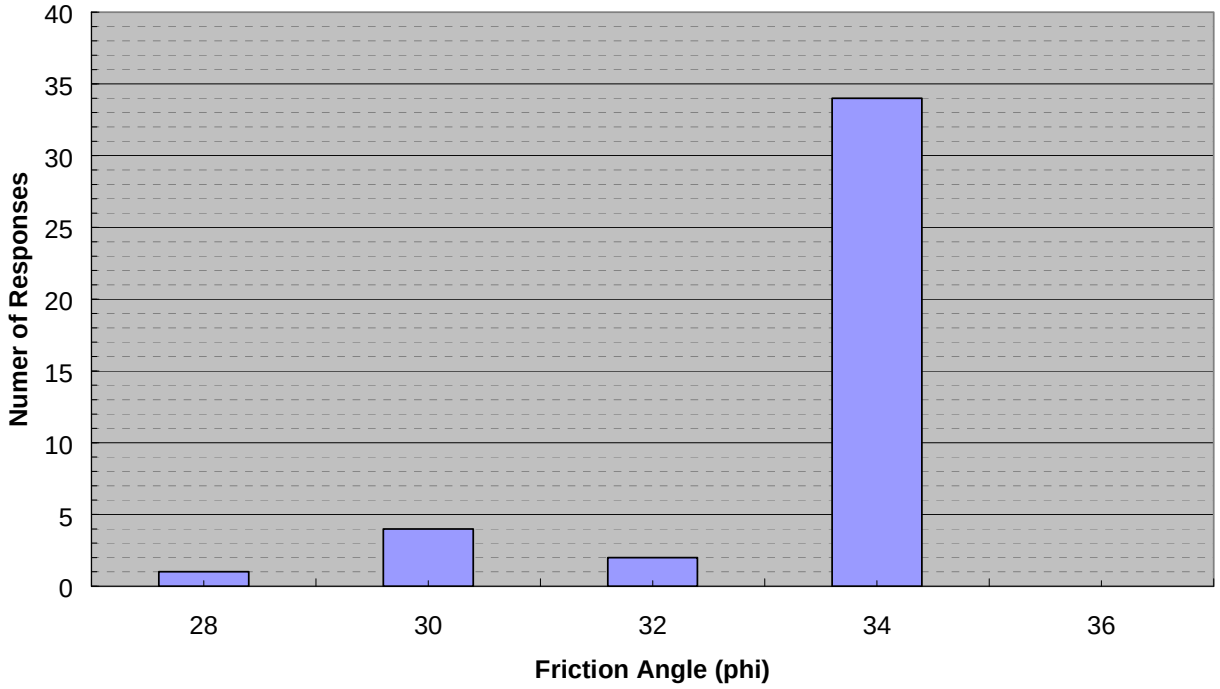
2c. Friction Angle (phi)



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2. MSE Backfill Properties - Comments

Each line in the following table presents one response.

none
Specified By Manufacturers Recommendations.
none
none
There is no electro-chemical requirement for borrowing (Colorado selected clean granular material).
none
none
Not required
none
Chemical tests are performed in accordance with GDT-98 test methods.
No electro-chemical properties specified for backfill with geosynthetic reinforcement.
* For walls with geosynthetic reinforcement, maximum grain size of backfill is 50mm
none
We do not allow geosynthetic reinforcement. If resistivity exceeds 500 ohm/cm, testing for chlorides and sulfates may be waived. Resisitivity tested at 100% saturation.
No electrochemical requirements are set by the Iowa DOT.
none
Material is to be reasonably free of shale, organic and other deleterious materials.
If resisitivity > 5000 ohm/cm - Chloride and Sulfate requirements are waived. AASHTO 7.3.6.3
1) For geosynthetics, if backfill resistivity is >5000 ohm-cm, the chloride and sulfate requirements are waived. For metallic reinforcement in conjunction with crushed stone MSE backfill (for winter work) chloride and sulfate requirements are waived in Resistivity >5000 ohm-cm. 2). for both geosythetic and metallic reinforcement, there a soundness requirements - magnesium sulfate soundness loss <30% after 4 cycles per AASHTO T104 (ASTM C88)
none
none
none
If the resistivity is greater than or equal to 5000 ohm-cm, the chloride and sulfate requirements may be waived.
none
If the resistivity is greater than or equal to 5000 ohm-cm, the chloride and sulfate requirements may be waived.
#N/A
Reinforcing strip phys. and mech. properties shall conform to either ASTM A 36/A 36M or ASTM A 572/A 572M Grade 65 or equal. Galvinization shall be applied after the mesh is fabricated and conform to ASTM A 153 or ASTM A 123
none
none
none
Geosynthetics not allowed
Chemical requirements apply for metal reinforcing only. Material failing to meet resistivity may be tested for sufate and chlorides. Material meeting criteria for sulfates and chlorides AND having resistivity greater than 1,000 ohm-cm is acceptable.

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ATTACHMENT D: Survey Response - Section 2 MSE Backfill Properties

Resistance to abrasion: maximum percentage wear of 55% AASHTO T96
none
For section a (above), there are multiple gradations allowed for reinforced backfill. One of the gradations is listed above. The others are similar.
none
Yes
Resistivity >5000, no Cl or SO4 test reqd. Resistivity 2000-5000, do Cl and SO4 tests. Provide randomly selected backfill samples for testing, as directed by the Engineer. Obtain approval for backfill material, prior to use. (For gradation also.)
none
Typically applies to metallic reinforcement.
The chloride and sulfate testing is waived if the resistivity is greater than or equal to 5000 ohm-cm
Modular block walls utilize geosynthetics which we state using the manufacturer's recommendations.
If the resistivity is greater than 5000 ohm-cm, the chlorides and sulates requirement may be waived.
none
Per AASHTO
Geosynthetic pH shown above is for PET. pH >3 for PP and HDPE
none
Yes
Organic Content: Loss On Ignition (LOI) has been modified on Bottom Ash Material due to the inclusion of unburned coal fragments that affect the organics.
See attached specifications.

ATTACHMENT D: Survey Response - Section 3 Use of “High Fines” and/or “High Plasticity” MSE Backfill, as opposed to “Standard” Backfill

3a. What is your definition of “Standard” backfill?

Each line in the following table presents one response

Index Properties as listed in Response 2a	Other
---	Non-cohesive Granular Material with a minimum phi of 28 degrees.
---	ADOT does not allow or recommend MSE Wall with "High Fines" and "High Plasticity" MSE Backfill. As such, we do not have information on specifications and performances of "Low Quality" MSE Wall Backfills, as solicited in the survey questionnaire.
YES	
YES	
NO	
YES	
YES	
YES	
YES	
YES	
YES	
YES	
YES	
NO	"Granular borrow" - for portion passing 3 in sieve, 0-70% passes #400 sieve, and 0-20 passes #200.
YES	Based on our past experience gained with the use of granular material, we are currently only using the No. 57 stone as the reinforcement material. The use of No. 57 stone have provided lots of positive feed back from contractor and the designer because of the ease of compaction and the interaction with the reinforcing elements.
YES	
YES	
YES	Select Granular Fill: Backfill material used in the reinforced zone shall comply with Mn/DOT 3149.2B2
YES	
YES	
NO	Cohesive or granular material used outside the reinforced zone of the MSE Wall.
YES	
YES	
---	Standard backfill is defined in the Specs Section 520.02.6
YES	---
YES	---
YES	---
YES	---
YES	---
YES	---
NO	Standard backfill would be our "common borrow", which is limited to <=17% passing #200 and <=9-inches or 3/4 lift thickness.
YES	Coefficient of uniformity >= 4
YES	---
YES	---
YES	---

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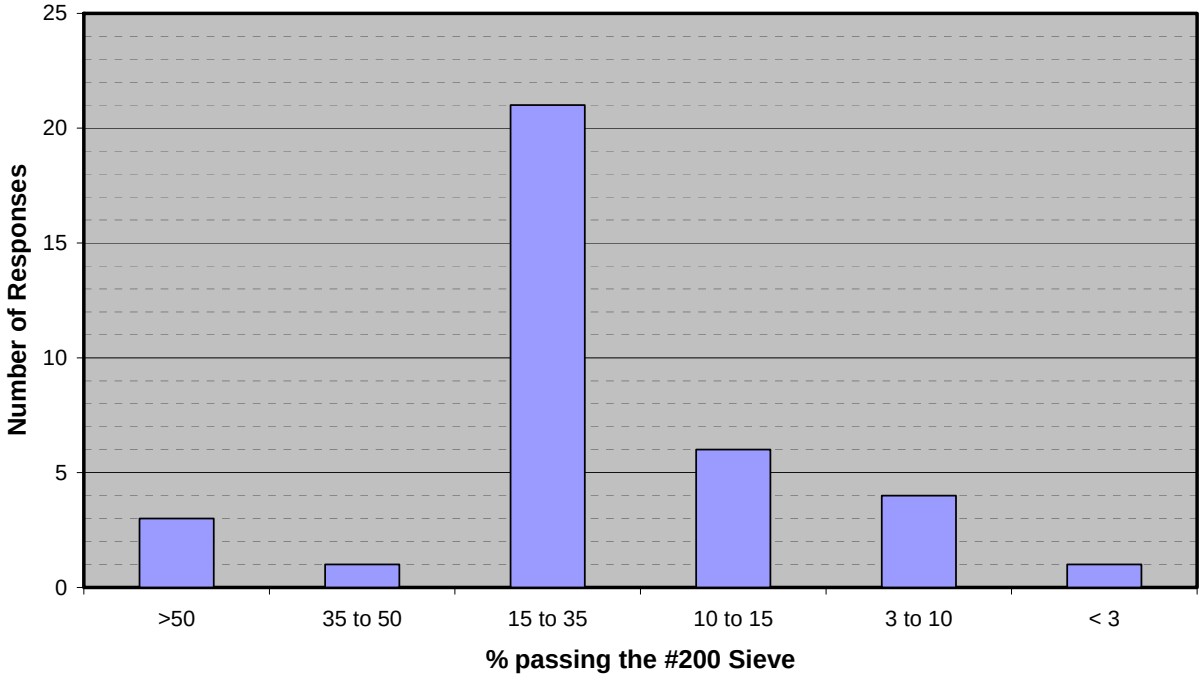
ATTACHMENT D: Survey Response - Section 3 Use of “High Fines” and/or “High Plasticity” MSE Backfill, as opposed to “Standard” Backfill

YES	Select Aggregate
YES	
---	conforming to the requirements of AASHTO M57
YES	
YES	
YES	

ATTACHMENT D: Survey Response - Section 3 Use of “High Fines” and/ or “High Plasticity” MSE Backfill, as opposed to “Standard” Backfill

3b. What is your definition of "high fines" backfill?



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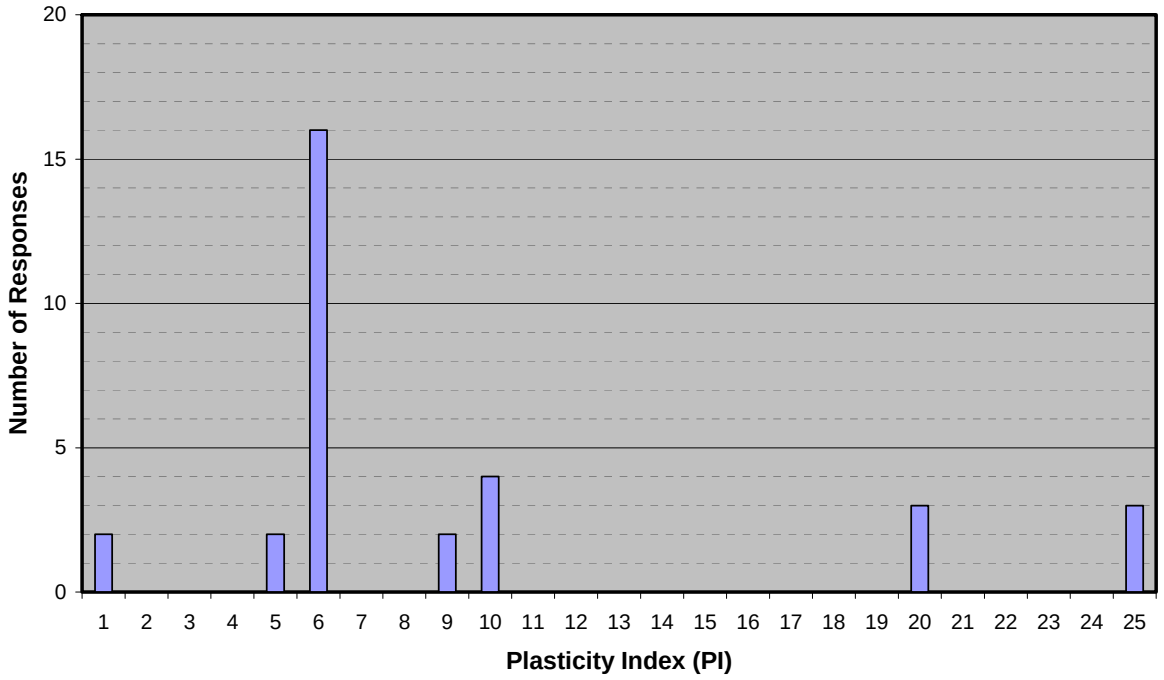
ATTACHMENT D: Survey Response - Section 3 Use of “High Fines” and/ or “High Plasticity” MSE Backfill, as opposed to “Standard” Backfill

3C. What is your definition of "high plasticity" backfill?



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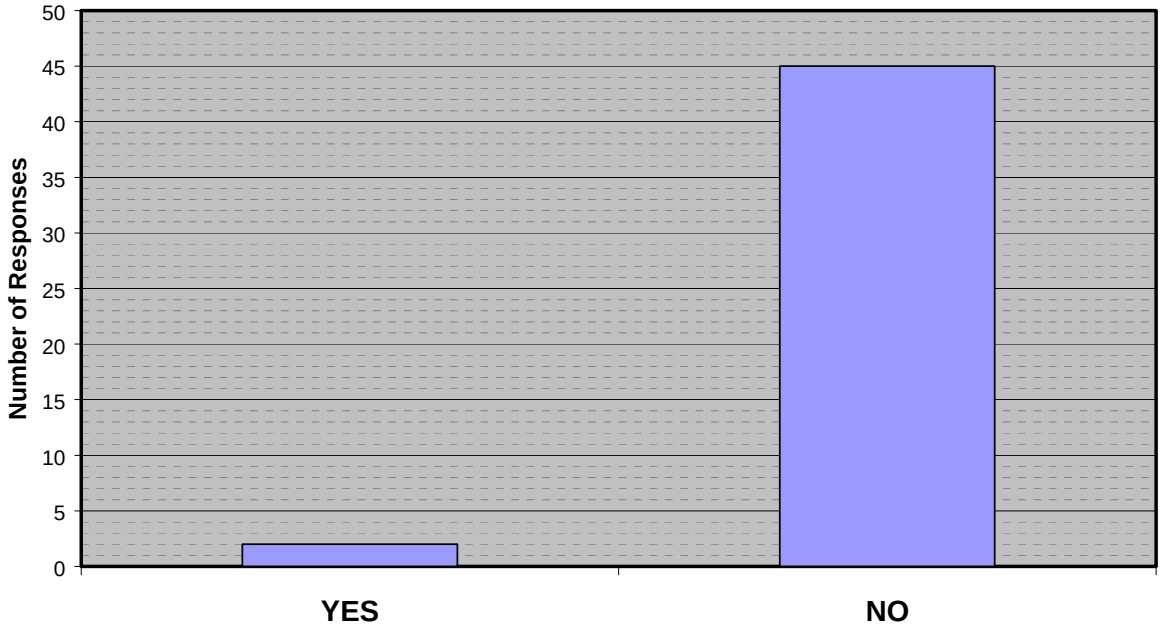
ATTACHMENT D: Survey Response - Section 3 Use of “High Fines” and/ or “High Plasticity” MSE Backfill, as opposed to “Standard” Backfill

3d. Do you allow "high fines" and/or "high plastcity" backfill to be sued in the reinforced backfill zone of MSE Walls?



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ATTACHMENT D: Survey Response - Sections 4, 5, 6, and 7

Question 3c was followed by the following instructions,

If NO, then proceed to Question #8.

Only 2 states responded that they allow the use of high fines or high plasticity MSE backfill. The responses from these 2 states is summarized in Section XX of the report.

ATTACHMENT D: Survey Response - Section 8 Additional Comments

Each line in the following table presents one survey response

We changed our spec in 2002 to the following requirements: Concrete sand meeting Section 802, F.M. waived, Section 801 course aggregate crushed material smaller than #467 with 10% or less passing the #200, or Crusher run material with 100% passing a 2-inch sieve with 10% or less passing the #200.
The Department is experiencing an increasing number of construction related problems associated with the backfill that is allowed by our specifications. The major suppliers in out state are expressing similar concerns regarding the high fines and high PI that is allowed. These problems range from deflection and/or distortion of the face panels to movement of the entire mass vertically and laterally. The problems are occurring predominantly during construction of the wall but some settlement and deflection problems are becoming more pronounced after placement of the barrier slab and the roadway section. The particular numbers of problem walls vs. total walls constructed are not available but qualitatively they are significant enough to make changes. We are currently working on several specification changes including 0-15% passing the #200 and PI less than or equal to 6. In addition, we are also working on several changes for construction including a vertical and horizontal tolerance and a manufacturers r
The fine material meeting a max. 15% passing No. 200 US sieve has been used for the construction of the MSE walls on a few occasions. Based on our record we have distressed condition on the walls and the lesson learned by our bridge department is not to allow soil as the fill material.
Standard MSE backfill readily available within NH at reasonable cost. Use of poorer quality materials would likely result in greater density of reinforcement (which could offset the cost benefit of lower cost backfill) or longer reinforcement zone that could affect other project issues (e.g., adjacent lateral support systems, right of way).
We use good quality backfill for our MSE walls because we want good performance. The suppliers agree with this philosophy, as our specification requirements match theirs. Suppliers do not want their products to perform poorly, as it results in their wall products not being selected for future use. In addition, we usually specify MSE structures for applications that cannot tolerate significant post-construction movement, such as support for stub bridge abutments. We find that MSE walls give excellent performance for their cost, and we feel that it is not worth trading performance for what we consider a minor cost savings. New York State has an abundance of good granular materials and so does not need to compromise on backfill quality.
We generally follow AASHTO and FHWA criteria and don't use what is traditionally considered marginal or "high fines" backfill. All designs are typically on a case-by-case basis.
Our MSE Walls are approved on a complete system basis.
Arkansas DOT has allowed the use of materials with a high Fine content (greater than 25% passing the #200 sieve but generally less than 35% passing the #200sieve), but this material does produce a high internal angle of friction.. We do not allow a material with high plasticity. It is preferred that a non-plastic material be utilized. Also, a 12 inch drainage layer is required behind the face of the wall with underdrain pipe at its base.
High fines and/or high plasticity backfill not allowed on Georgia DOT MSE walls.

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Erodible or unstable materials are not allowed to be used as backfill material for MSE walls. The following materials are considered erodible or unstable. 1) Friable Sandstone, 2) Crushed or uncrushed Gravel any size, 3) Crushed coarse aggregate smaller than size #57, 4) Any material with 50% or more passing the #4 sieve. All of this was taken from section 805.12 of our standard specifications with can be found at the following: http://www.kytc.state.ky.us/construction/spec/2004/2004_Division800-A.pdf. A copy of our Draft MSE wall note is available upon request.
Some research done by LTRC (Louisiana Transportation Research Center) with soils of less quality than those specified in reinforced backfill specifications. Contact Mark Morvant at LTRC (225) 767-9124 for information on past research.
Using sand let alone high fines soil is currently not possible in North Carolina.
High fine soils/High plasticity soils as defined above are typically used only in temporary MSE walls. In permanent MSE walls we use granular or stone backfill. The need for drainage in permanent MSE walls is determined on a job by job basis.
These type materials are subject to high variability with less control of properties and would require a much higher level of testing and inspection (which would be unlikely to occur) to produce confidence in an acceptable final product.
We have, on occasion, had walls built with backfill materials that did not meet our requirements for backfill as outlined in our retaining wall specification. These were a result of limited testing and poor project oversight. We discovered these instances when problems arose in construction or during the service life of the wall. In these instances, no special provisions were made in the wall design to take into account the poor quality backfill. Repair of these walls has been very costly, has impacted traffic, and in most cases has resulted in complete reconstruction of the retaining walls. We are currently in the process of re-writing our Standard Specification Items. The responses included in this survey reflect the Specification re-write. You can find copies of our current and our proposed specifications for Item 423 at http://www.dot.state.tx.us/cst/2003SpecProj/index.htm
Iowa DOT alllows no greater than 10% passing the 0.075 mm (#200 sieve) in Granular Backfill. This places an effective limit on use of "high fines" backfill. Typical Granular Backfill material is sand/gravel or crushed stone. If crushed stone is used, 100% must pass the 25.4 mm sieve. The Iowa DOT specifications for Granular Backfill placement and materials are included as file attachments to the e-mail reply to the NCHRP
The questions above were answered with our concrete faced walls in mind. The special provision for this wall does not allow for anything but a coarse or fine aggregate. The question regarding the gradations of the material of our select backfill was not filled in because we allow various gradations from what IDOT calls a CA-6 through a CA-16 and on the Fine aggregate side an FA-1, FA-2 or FA-20. If you want these gradations please let me know and I can email or fax them to you.
Only free draining granular material is allowed in the reinforced volume
MODOT does not allow high fines or high plasticity backfill. During one project we had a temporary wall with wire face and metallic reinforcement. The contractor backfilled the reinforced zones with soils found near the project, even though the temporary wall called for select granular fill. The temporary wall had lateral deformation with possible pullout of the metallic reinforcement.
Wisconsin has several standard wall specifications for different types of MSE walls. We have attached two: one for panel-faced MSE walls and another for Modular Block MSE walls. These can be consulted for further details and are representative of all of our MSE wall specifications.

ATTACHMENT D: Survey Response - Section 8 Additional Comments

ADOT Retaining Wall Policy and MSE Wall system approval process follows AASHTO specifications for MSE/Structure Backfill, as provided in the AASHTO Standard Specifications for Highway Bridges, Division II, Construction. ADOT does not allow or recommend MSE Wall with "High Fines" and "High Plasticity" MSE Backfill. As such, we do not have information on specifications and performances of "Low Quality" MSE Wall Backfills, as solicited in the survey questionnaire.
CDOT does not allow high fine and high plasticity soil at this time. However we do allow to relax the 2" maximum size (CDOT Class 1) to 4" (on-site granular material) at location that good borrowing material is not available. For avoiding roadway de-ice salt, we require impervious membrane and collect drain for wall supporting roadway. The construction damage is not a problem for those 4" maximum on-site material, because it is considered without taking the credit of higher friction angle.
Dr. Richard Bathurst of Royal Military College in Kingston, Canada will study the use of low quality backfill for MSE walls as part of his research project on MSE walls that has been going on in the last few years.
To date, we have not developed MDT general guidelines or special provisions governing the design and construction of MSE walls. We have referred to either AASHTO or FHWA guidelines in the specials that we have prepared. We plan to develop a special governing MSE wall design and construction. In short, we are not in a good position to fill out the questionnaire since we have no general guidelines
While we are open to the idea of using high fines/pasticity soils as backfill, ability to place and compact the soil, depending on the wetness of the climate and the timing of the contruction project (i.e., wet winter months versus drier summer months), our climate does limit our ability to do this. In general, I would be very reluctant to use anything poorer in quality than an ML, and with ML's, we would need to carefully consider the timing of construction and the specific drainage features needed.

ATTACHMENT E - Koerner, R.M. and Soong, T.-Y., 1999, “Geosynthetic Reinforced and Geocomposite Drained Retaining Walls Utilizing Low Permeability Backfill Soils, GRI Report #24, Geostnthetic Institute, Folsom, PA, 119 pg.

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4. Determination of Seepage Pressure Behind Low-Permeability Backfill Soils

This section presents various seepage pressure* scenarios on low permeability reinforced zone backfill soils when adequate drainage is not available. It is important to note that when mobilized, these seepage pressures must be added to the lateral soil pressures presented earlier. Such superposition of the seepage pressures and geostatic pressures is indeed valid since the entire analysis is based on limit equilibrium methods.

4.1 Significance of Seepage Pressures

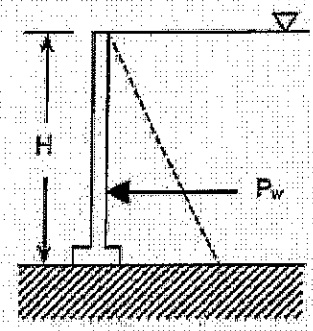
In order to gain a perspective of the relative magnitude of seepage pressures versus soil pressures, the following contrasting situations are offered. Figure 10a represents a hypothetical wall serving as a dam containing water of depth "H". The resulting magnitude of water pressure is as follows:

$$P_{total} = P_w = 0.5\gamma_w H^2 \qquad (1)$$

When nondimensionalized against $\gamma_w H^2$, the force is constant as plotted against L/H, where *"L"* is the distance behind the back of the wall. See Figure 10e for this response.

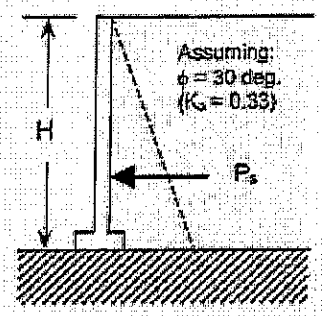
Figure 10b represents the same hypothetical wall serving to resist a soil force without seepage forces, e.g., a wall with a drainable reinforced zone backfill soil. This is the usual assumption that one makes in designing SRWs (and most other walls) and the one which was used in the earlier design section of the report. Assuming a cohesionless soil backfill of $\phi = 30$ deg., the resulting active earth pressure coefficient is $K_a = 0.33$. Also, assuming that the total dry unit weight is twice the unit weight of water, we have the following:

*the terms seepage pressure and seepage stress will be used interchangeably in this report



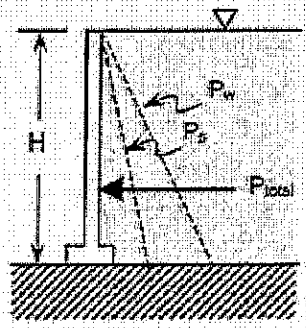
(a) Dam

$P_{total} = P_w = 0.5\gamma_w H^2$



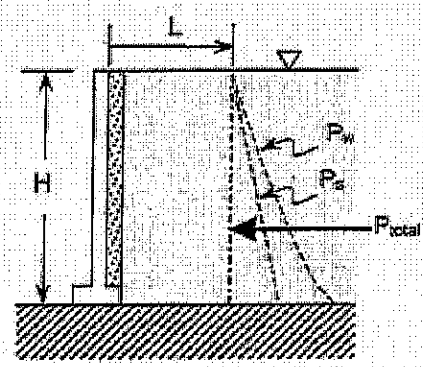
(b) "Dry" (non-saturated) soil

$P_{total} = P_s = 0.167\gamma_l H^2$
$= 0.33\gamma_w H^2$
(assuming $\gamma_l = 2\gamma_w$)



(c) Saturated soil without wall drain

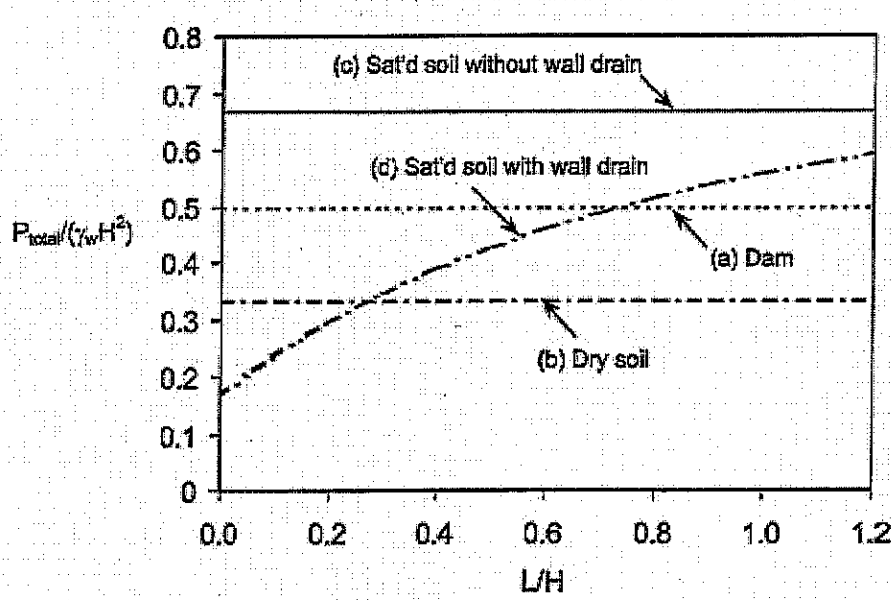
$P_{total} = P_w + P_s$
$= (0.5\gamma_w + 0.167\gamma')H^2$
$= 0.667\gamma_w H^2$
(assuming $\gamma_w = \gamma'$)



(d) Saturated soil with wall drain

$P_{total} = P_w + P_s$
$= P_w + 0.167\gamma' H^2$

(P_w varies with "L")



(e) Comparison of seepage force results

Figure 10 – Relative significance of seepage pressures leading to seepage forces

$$P_{total} = P_s = 0.5K_a(\gamma_s H^2) \qquad (2)$$
$$= 0.167(\gamma_s H^2)$$
$$P_{total} = 0.33(\gamma_w H^2)$$

Again nondimensionalizing this total force with $\gamma_w H^2$ and plotting on Figure 10e, the horizontal line is labeled as "dry soil".

Figure 10c represents the combined situation of soil with its voids in the saturated condition. Thus, both soil pressure and water pressure exists behind the wall face. The same hypothetical wall is used under the same assumed conditions, i.e., $\phi = 30$ deg. Also, $\gamma_s = 2\gamma_w$ and $\gamma' = \gamma_w$. For this situation of saturated soil, we have the following:

$$P_{total} = P_w + P_s$$
$$= (0.5\gamma_w + 0.167\gamma')H^2 \qquad (3)$$
$$P_{total} = 0.667(\gamma_w H^2)$$

When the above total force is nondimensionalized with $\gamma_w H^2$ and plotted on Figure 10e, the horizontal line is seen to be twice the magnitude of the dry soil by itself. <u>Thus without drainage, the total force against the wall can be double that of a properly drained backfill soil</u>.

Walls of the type described earlier (including SRWs), however, generally do have a drainage zone directly behind them, recall Figure 7. Unfortunately, if the reinforced zone consists of a low permeability soil and it is relatively nondraining, the seepage pressure moves back within the reinforced zone until finally it acts against the back of the reinforced zone soil mass as shown in Figure 10d. To determine the magnitude of the total force, superposition of the water and soil forces are illustrated as follows:

$$P_{total} = P_w + P_s$$
$$= P_w + 0.167\gamma H^2 \qquad (4)$$

The value of "P_w" varies with the distance "L" as will be seen later in this section. The "P_{total}" response is shown as the curved line (without justification for the time being) in Figure 10e. Interestingly, at L/H = 0.7 the total pressure is the same as with a hydraulic dam. This value of L/H = 0.7 is commonly used for the reinforcement length of SRWs.

The fundamental message of Figure 10e is that seepage forces can be extremely large and when not recognized, or accounted for, can completely overcome a traditional factor-of-safety value of 1.5. As was seen in Section 3.0, twenty of the serviceability problems and actual collapse situations (out of a total of 26-case histories) had silts, clays and related fine-grained soil mixtures as backfill in the reinforced zone. Not one of these walls had any drainage accommodation to relieve seepage pressures. Clearly, seepage pressure was involved in the deformations and/or failures in these 20-case histories.

The remainder of this section develops the methodology of how seepage pressures (and its related seepage forces) are calculated.

4.2 Fundamentals of Seepage in Soils

The flow of water within the voids of a soil mass follows the same basic laws as steady-state heat flow, electrostatics, and current flow in conductors. For water flow in soils, the following assumptions are necessary:

- the soil is homogeneous,
- the voids are completely filled with water under steady-state conditions,
- no compression or expansion of the soil takes place,
- soil particles and water are incompressible, and
- flow is laminar and follows Darcy's equation.

Under the above assumptions it can then be shown (Cedergren, 1989) that the quantity of water entering an element of soil must equal that leaving, and the equation of continuity takes the following form:

$$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z}=0 \tag{5}$$

The terms u, v, and w are discharge velocity components in directions x, y, and z directions, in cartesian coordinates.

According to Darcy's equation $(v_d = ki)$, the components of the discharge velocity are

$$u=-k\frac{\partial h}{\partial x},\quad v=-k\frac{\partial h}{\partial y},\quad w=-k\frac{\partial h}{\partial z} \tag{6}$$

Substituting in Eq. 5 we obtain the following:

$$\partial\frac{-k(\partial h/\partial x)}{\partial x}+\partial\frac{-k(\partial h/\partial y)}{\partial y}+\partial\frac{-k(\partial h/\partial z)}{\partial z}=0 \tag{7}$$

If the soil mass is isotopic in its permeability, i.e., if $k_x = k_y = k_z = k$, the value "k" can be factored out of the left side of the equation and cancelled, resulting in the following equation:

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}+\frac{\partial^2 h}{\partial z^2}=0 \tag{8}$$

This equation is the common form of Laplace's equation for three dimensional flow of water through isotropic homogeneous soils. In two dimensions, the equation has the form:

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=0 \tag{9}$$

If the soil mass is anisotropic in its permeability, i.e., if $k_x \neq k_y \neq k_z$, the equations become somewhat unwieldy. They result in the following two equations in three dimensional and two dimensional forms, respectively.

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} + k_z \frac{\partial^2 h}{\partial z^2} = 0 \tag{10}$$

and

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} = 0 \tag{11}$$

Eqs. 10 and 11 (for anisotropic soils) are not Laplacian and must be adjusted in order to be solved by the methods to be described. Although difficult to accomplish, results for anisotropic soils will be handled, although the isotropic soil condition will be the general focus of the report.

Laplace's equation in two dimensions, i.e., Eq. 9, can be represented by two families of curves that intersect at right angles to form a pattern of curvelinear square figures known as a "flow net". One set of curves is called the streamlines or flow lines, the other set of curves is called the equipotentials. The adjacent flow lines represent channels within which water can flow. The equipotential lines are lines of equal energy level expressed as total hydraulic head.

Although mathematical solutions have been developed for a number of cases of relatively idealistic flow, the solutions are cumbersome and tedious to obtain. Leliavsky (1955) writes, "... the analytical method, although rigorously precise, is not universally applicable because the number of known conditions on which it depends is limited. Moreover, except in a few elementary cases, the analytical method lies beyond the mathematics of practicing design engineers."

As a result of the analytic complications of solving practical problems, the method of flow nets was developed in the early 1900's. Apparently, Professor Forcheimer of the Technical University of Graz, in Austria, developed the technique in which the following requirements must be met to result in a proper solution of Laplace's equation, i.e., Eq. 9.

- Flow lines and equipotential lines must intersect one another at right angles to form areas that are called "curvelinear" squares.
- Specific entrance and exit requirements (i.e., boundary condition) must be met.
- A basic flow line deflection rule must be followed in passing from a soil of one permeability to a soil of a different permeability.
- Adjacent equipotentials have equal head losses
- The same quantity of seepage flows between adjacent pairs of flow lines.

The last two items are fundamental requirements indirectly entering into the construction of flow nets. An additional assumption should also be stated in that the quantity of seepage flowing through a section must be constant throughout the section unless additional water enters or some is removed by drains. Flow nets for commonly encountered geotechnical engineering structures are found in all geotechnical engineering texts, e.g., Cedergren, 1989.

Clearly, the hand sketching of flow nets leads to an enormous appreciation of the flow of water in soils and an appreciation of the information that can be obtained, e.g., seepage pressures, seepage forces and seepage flow rates. At this point in time, however, the computer has allowed for the avoidance of hand sketching. It also increases the accuracy enormously. In a review of numerical methods and computer solutions in soil engineering, Christian (1987) points out that there are a number of computer programs available to the general user for solving problems of fluid flow in soils. As long as non-linear properties or phreatic surfaces are not involved, general purpose programs can be used. In this regard, one has the choice of either the finite element, or finite difference methods. The authors have selected the finite difference method for the purpose of this report. The essentials of the method follow, with a complete appendix to this report provided for details in the form of a "user's manual".

4.3 Overview of Finite Difference Code Used in This Study

The finite difference method is a numerical approach for solving partial differential equations such as those governing the two dimensional steady-state seepage flow in soils (i.e., Eq. 9). As shown in Figure 11, the two dimensional space behind a retaining wall (the back of the wall being represented by the y-axis) is discretized with a grid of points (called nodes or nodal points); the coordinates of which are denoted by *i* and *j*. Note that curved boundaries have to be approximated with straight line segments, the accuracy of which is enhanced by the closeness of the grid spacing.

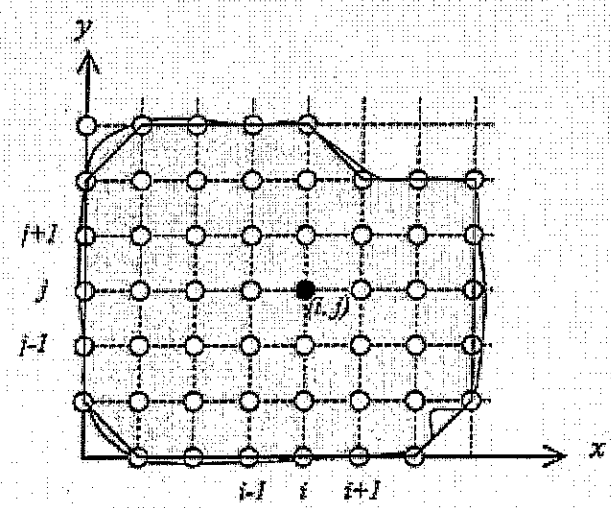


Figure 11 – Discrete representation of a two dimensional region.

If Δx and Δy are the nodal intervals in the x and y directions, respectively, the discretized form of Equation (9) at point *(i,j)* can be written as follows (for the complete derivation, see Bardet (1997)):

$$\frac{k_x}{\Delta x^2}\left(h_{i+1,j}+h_{i-1,j}-2h_{i,j}\right)+\frac{k_y}{\Delta y^2}\left(h_{i,j+1}+h_{i,j-1}-2h_{i,j}\right)=0 \qquad (12)$$

As shown in Eq. (12) and Figure 12, only the values of the head in the immediate vicinity of the selected node "*i,j*" contribute to the solution of the head of the selected node.

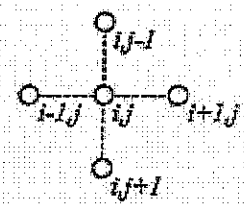


Figure 12 – Nodes contributing to Equation (12)

When $\Delta x = \Delta y$, Eq. (12) becomes the following:

$$h_{i,j} = \frac{1}{2(1+\alpha)}\left(\alpha h_{i+1,j} + \alpha h_{i-1,j} + h_{i,j+1} + h_{i,j-1}\right) \tag{13}$$

where $\alpha = k_x/k_y$. Furthermore, when $\Delta x = \Delta y$ and $k_x = k_y$ *(i.e.,* $\alpha = 1$*)*, Eq. (12) reduces to the following:

$$h_{i,j} = \frac{1}{4}\left(h_{i+1,j} + h_{i-1,j} + h_{i,j+1} + h_{i,j-1}\right) \tag{14}$$

Eqs. (13) and (14), along with the prescribed boundary conditions, can be used to determine the total head distribution over the considered domain. Table 15 lists the finite difference algorithms of treating several special situations. For detailed derivations see Bardet (1997).

The values of the total head at the various grid points may be found by an interactive method, which can be implemented to spreadsheet programs*. In this method, appropriate equations for calculating the total head at different nodes are first implanted into various cells that represent different nodes. This step ensures that all the nodal points, including points along the boundaries and interfaces, are appropriately related to their neighboring points. Arbitrary initial values are then assigned to these unknowns (i.e., to the total head at the grid points). Subsequently, new values are calculated from old ones in an interactive manner, using an established spreadsheet. Final values are eventually reached within a specified error tolerance. The entire procedure is illustrated by the following example.

*This report offers the reader such a solution in the form of MS Excel® which we hope will be a major contribution to design and analysis of seepage designs for retaining walls and related soil problems such as slope stability.

Table 15 - Finite Difference Methods of Calculating Total Head at Impervious Boundary and Interfaces

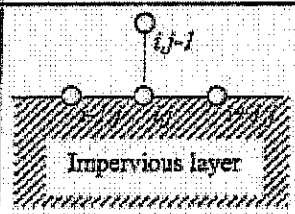
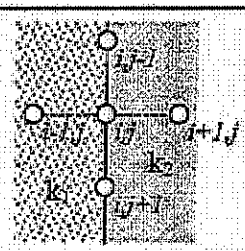
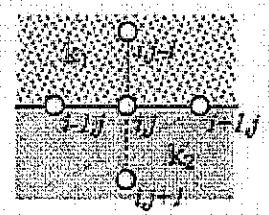
Situation	Contributing nodes	Total head at point *(i,j)*
Horizontal impervious boundary		For anisotropic soil ($k_x \neq k_y$): $$h_{i,j} = \frac{1}{2(1+\alpha)}\left(\alpha h_{i+1,j} + \alpha h_{i-1,j} + 2h_{i,j-1}\right) \quad (15)$$ For isotropic soil ($k_x = k_y$): $$h_{i,j} = \frac{1}{4}\left(h_{i+1,j} + h_{i-1,j} + 2h_{i,j-1}\right) \quad (16)$$
Vertical interface		$$h_{i,j} = \frac{k_1}{k_1+k_2} h_{i-1,j} + \frac{k_2}{k_1+k_2} h_{i+1,j} \quad (17)$$
Horizontal interface		$$h_{i,j} = \frac{k_1}{k_1+k_2} h_{i,j-1} + \frac{k_2}{k_1+k_2} h_{i,j+1} \quad (18)$$

Figure 13 presents a seepage problem associated with a SRW. A free-draining gravel in the drainage zone is assumed. The reinforced zone is assumed to be of lower permeability and is saturated with a water table stationary at the ground surface. The retained zone is also of low permeability and assumed to be of the same permeability as the reinforced zone. Note that the soil mass of consideration (i.e., the influence domain of calculation) extends twice the wall height behind the wall drain. This is based on the assumption that the wall drain will have practically no effect on soil beyond that distance.

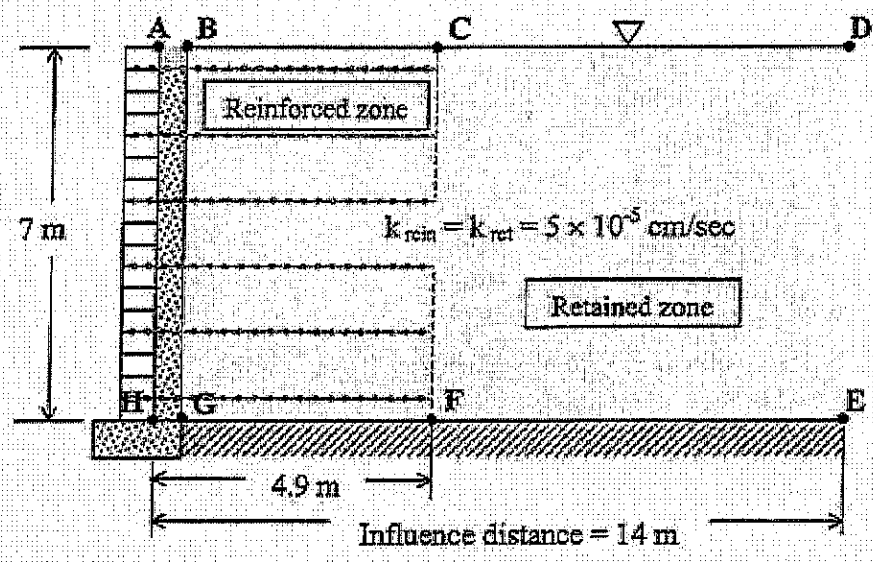


Figure 13 – Node Identification for the example seepage problem.

The spreadsheet developed for solving the seepage problem of Figure 13 is shown in Table 16. The entire soil mass, including both reinforced and retained soils, is divided into 10 divisions in height (from row "4" to row "14") and 20 division in width (from column "B" to column "V"). In other words, the domain consists of 231 (11×21) nodal points regardless of the actual wall height. Higher accuracy (i.e., smoother seepage pressure distribution curves) may be achieved with more nodal points, however, the number used in this study (231 nodes) seems to provide the desirable degree of accuracy within reasonable computing time.

The equations utilized for calculating the total head at different points are also shown in Table 16. The principles of selecting appropriate equations and grid layout are described as follows:

- The interface between the back of the wall and the wall drain (Line *AH*) is simulated by column "B" in Table 16, along which the total heads are the corresponding (and known) elevation heads.

Table 16 - Spreadsheet for solving the example seepage problem

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V
1		k_{rein} =	5.0E-07	m/sec					k_{ret} =	5.0E-07	m/sec											
2	Wall height, H =		7.0	m																		
3	Y																					
4	0	=$C$2	=$C$2	=$C$2	=$C$2			=$C$2	=$C$2	=$C$2											=$C$2	=$C$2
5	1	=B4-$C$2/10	=B5	=(C5+D6+E6+D4)/4	=(D5+E6+F5+E4)/4			=(B5+H6+I5+H4)/4	=H5*$C$1/(C$1+$I$1)+I5*$I$1/(C$1+$I$1)	=(J5+K6+L5+J4)/4											=$C$2	=$C$2
6	2	=B5-$C$2/10	=B6	=(C6+D7+E6+D5)/4	=(D6+E7+F6+E5)/4			=(C6+I7+I6+I5)/4	=I6*$C$1/(C$1+$I$1)+I7*$I$1/(C$1+$I$1)	=(K5+L6+M5+L4)/4											=(T6+U7+V6+U5)/4	=$C$2
7	3	=B6-$C$2/10	=B7	=(C7+D8+E7+D6)/4	=(D7+E8+F7+E6)/4			=(C7+H8+I7+H6)/4	=H7*$C$1/(C$1+$I$1)+I7*$I$1/(C$1+$I$1)	=(K6+L7+M6+L5)/4											=(T7+U8+V7+U6)/4	=$C$2
8	4	=B7-$C$2/10	=B8	=(C8+D9+E8+D7)/4	=(D8+E9+F8+E7)/4			=(C8+H9+I8+H7)/4	=H8*$C$1/(C$1+$I$1)+I8*$I$1/(C$1+$I$1)	=(K7+L8+M7+L6)/4											=(T8+U9+V8+U7)/4	=$C$2
9	5	=B8-$C$2/10	=B9	=(C9+D10+E9+D8)/4	=(D9+E10+F9+E8)/4			=(C9+H10+I9+H8)/4	=H9*$C$1/(C$1+$I$1)+I9*$I$1/(C$1+$I$1)	=(K8+L9+M8+L7)/4											=(T9+U10+V9+U8)/4	=$C$2
10	6	=B9-$C$2/10	=B10	=(C10+D11+E10+D9)/4	=(D10+E11+F10+E9)/4			=(C10+H11+I10+H9)/4	=H10*$C$1/(C$1+$I$1)+I10*$I$1/(C$1+$I$1)	=(K9+L10+M9+L8)/4											=(T10+U11+V10+U9)/4	=$C$2
11	7	=B10-$C$2/10	=B11	=(C11+D12+E11+D10)/4	=(D11+E12+F11+E10)/4			=(C11+H12+I11+H10)/4	=H11*$C$1/(C$1+$I$1)+I11*$I$1/(C$1+$I$1)	=(K10+L11+M10+L9)/4											=(T11+U12+V11+U10)/4	=$C$2
12	8	=B11-$C$2/10	=B12	=(C12+D13+E12+D11)/4	=(D12+E13+F12+E11)/4			=(C12+H13+I12+H11)/4	=H12*$C$1/(C$1+$I$1)+I12*$I$1/(C$1+$I$1)	=(K11+L12+M11+L10)/4											=(T12+U13+V12+U11)/4	=$C$2
13	9	=B12-$C$2/10	=B13	=(C13+D14+E13+D12)/4	=(D13+E14+F13+E12)/4			=(C13+H14+I13+H12)/4	=H13*$C$1/(C$1+$I$1)+I13*$I$1/(C$1+$I$1)	=(K12+L13+M12+L11)/4											=(T13+U14+V13+U12)/4	=$C$2
14	10	=B13-$C$2/10	=B14	=(C14+2*D13+E14)/4	=(D14+2*E13+F14)/4			=(C14+2*H13+I14)/4	=H14+2*I13+I14)/4	=(K13+L14+M13+L12)/4											=(T14+2*U13+V14)/4	=$C$2
15	X	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
16																						
17																						
18																						
19																						
20																						

Wall face/ wall drain interface

Wall drain/ reinforced soil interface

Reinforced soil/ retained soil interface

Reinforced soil with k_{rein} (cell "D1")

Retained soil with k_{ret} (cell "J1")

- The interface between the wall drain and the reinforced soil (Line *BG*) is simulated by column "C" in Table 16, along which the total heads are also the corresponding (and known) elevation heads.
- The interface represented by Line BG is always located at 0.1H behind the back of the wall. This assumes that the drainage layer thickness is of a comparable width.
- The total head along the interface between the reinforced soil and the retained soil (Line *CF*) is calculated using Eq. 17 as shown in Column "T" in Table 16. Note that this procedure is still valid when the same soil is used in the two different zones, as for this example problem.
- The total head along the furthest distance (Line *DE*) is assumed to be the height of wall, see column "V" in Table 16.
- The total head along the ground surface behind the wall (Line *AD*) is also assumed to be the height of wall, as illustrated in Row "4" in Table 16.
- The foundation soil is conservatively assumed to be impervious, thus the total head along the foundation surface (Line *HE*) is calculated using Eq. 16. This is illustrated in Row "14" in Table 16.

Modifications of boundary conditions to handle special cases such as base drains, back drains, and geomembranes on the ground surface can be implemented into the spreadsheet program. Details will be described in the Appendix. Conversely, it is difficult to include a wall batter, i.e., a value of $\omega \neq 0$. Typical SRW batters of 3, 6 or 12 deg. can be accommodated only by using a very fine grid spacing which necessitates many more nodal points than utilized herein.

The spreadsheet program of MS Excel 97® was used in this study. The interactive process is initiated by choosing Calculations in the Options box of the Tools menu. The results

of the calculations after 300 iterations are shown in Table 17. The error after 300 iterations is less than 0.001 m. Excel® has a two dimensional capability to represent the distribution of the calculated total head values. Figure 14 shows such a two-dimensional contour plot for the example problem. Similar results can be used to determine seepage pressure distributions and the corresponding seepage force along any line (or curve) of interest behind the wall for either external or internal stability considerations. The program can also be formatted to obtain the quantity of seepage which will eventually be necessary for flow rate design of drainage systems.

4.4 Selected Seepage Studies with Respect to Geosynthetic Reinforced Retaining Walls

The finite difference code described in the previous subsection, with complete operational details in the appendix, allows for innumerable studies to be conducted. Clearly, the site-specific situation should be the targeted problem. However, in its absence, we have selected a number of parametric evaluations to illustrate the importance of the seepage issue with respect to the mobilization of seepage forces on low permeability backfill soils in the reinforced zone. As noted previously, these seepage forces are to be added to the force of the soil mass. In all cases, the general cross section of the segmental retaining wall is as shown in Figure 13, although the computer code has applicability to the entire spectrum of retaining walls, and steep soil slopes as well.

4.4.1 Effect of D/H Ratio

The initial step in the design of a SRW is to assume a length of reinforcement, which is usually expressed as a ratio of the wall height. AASHTO recommends using L/H = 0.7, and this value will be used in most of the subsequent evaluations. NCMA recommends either L/H = 0.5 or 0.6. Conceivably, other values (either longer or shorter) could also be selected.

Table 17 - Values of total head for the example problem after 300 interations

0	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000	7.000
1	6.300	6.300	6.369	6.435	6.499	6.558	6.611	6.660	6.703	6.745	6.783	6.817	6.847	6.873	6.896	6.917	6.936	6.953	6.969	6.985
2	5.600	5.600	5.739	5.874	6.002	6.121	6.228	6.325	6.410	6.496	6.571	6.638	6.696	6.748	6.795	6.836	6.873	6.907	6.940	6.970
3	4.900	4.900	5.113	5.319	5.514	5.694	5.857	6.001	6.128	6.255	6.367	6.466	6.553	6.630	6.698	6.758	6.813	6.864	6.911	6.956
4	4.200	4.200	4.493	4.776	5.042	5.285	5.502	5.695	5.862	6.030	6.177	6.306	6.420	6.519	6.608	6.687	6.758	6.824	6.885	6.943
5	3.500	3.500	3.883	4.251	4.592	4.900	5.174	5.412	5.618	5.825	6.005	6.162	6.300	6.421	6.527	6.623	6.709	6.788	6.861	6.931
6	2.800	2.800	3.288	3.752	4.175	4.551	4.879	5.162	5.404	5.645	5.855	6.037	6.196	6.335	6.458	6.568	6.666	6.757	6.841	6.922
7	2.100	2.100	2.718	3.293	3.805	4.249	4.630	4.952	5.225	5.497	5.732	5.935	6.112	6.266	6.402	6.523	6.632	6.732	6.825	6.914
8	1.400	1.400	2.191	2.898	3.502	4.011	4.437	4.793	5.090	5.387	5.641	5.860	6.050	6.215	6.361	6.490	6.607	6.714	6.813	6.908
9	0.700	0.700	1.748	2.606	3.295	3.855	4.314	4.693	5.006	5.318	5.585	5.814	6.012	6.184	6.336	6.470	6.592	6.702	6.806	6.904
10	0.000	0.000	1.494	2.481	3.218	3.800	4.272	4.659	4.977	5.295	5.565	5.798	5.999	6.174	6.327	6.464	6.586	6.699	6.803	6.903
	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

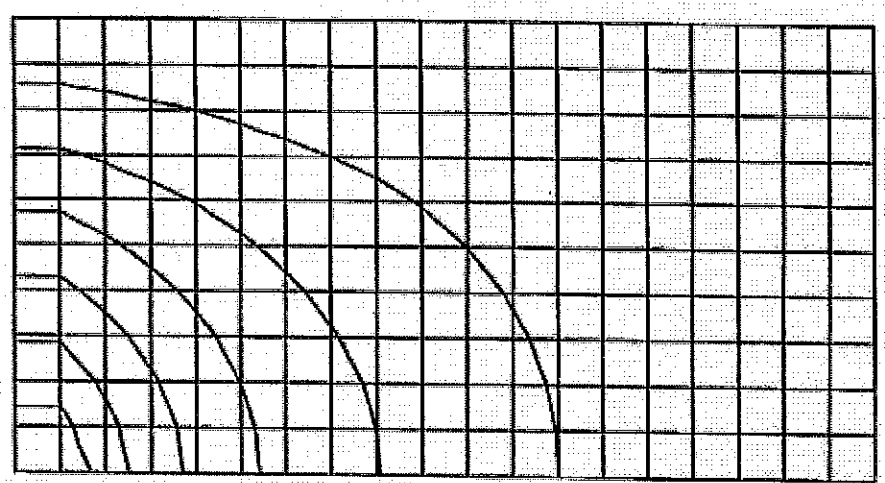


Figure 14- Total head distribution for the example problem.

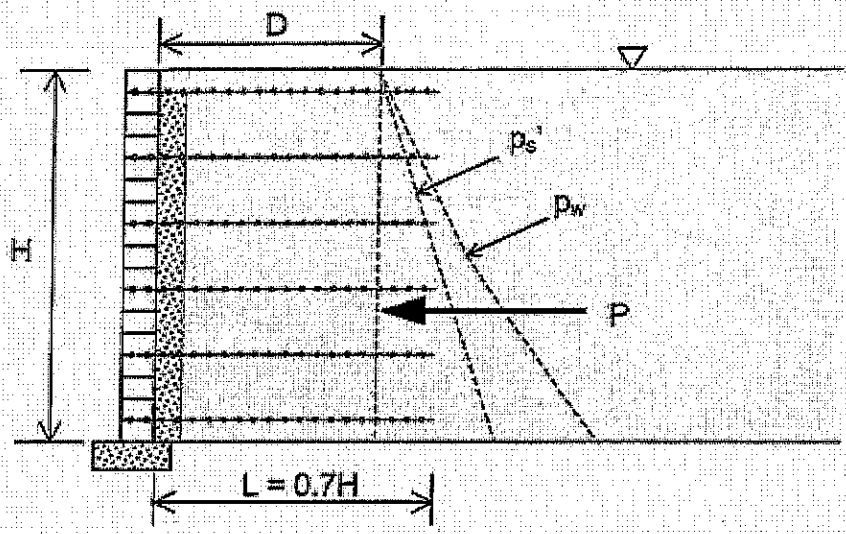
Using the wall cross section shown in Figure 15, soil and seepage pressures were generated for various D/H ratios using the finite difference code in the appendix to this report. Note that "D" is the distance in back of the wall facing which defines an imaginary vertical plane at which the seepage pressures are to be calculated. It is not the length of the reinforcement which is held constant at a value of L = 0.7H. The resulting values were then integrated over the wall height to result in a seepage force "P_w". This value was then added to the soil force based on effective unit weight to arrive at a value of $(P_s' + P_w)$. The value was then nondimensionalized using the soil force based on dry unit weight, i.e., P_s. Thus, by plotting $(P_s' + P_w)/P_s$, the relative influence of the seepage force to the soil force can be assessed.

In the lower portion of Figure 15, these values are plotted against various D/H ratios. For values of D/H $\leq$ 0.4, the drainage zone behind the block wall facing dominates the behavior. However, this D/H value is not a realistic situation for reinforcement length, i.e., it is far too short. For values of D/H > 0.4 the effect of seepage force gradually increases over that of the soil alone. At a value of D/H = 0.7 (which is the length of the reinforcement in this case), the total force is approximately 40% greater than for the soil by itself. In the limit, the total force is seen to be twice that of the soil by itself. The results substantiate the behavior shown previously in Figure 10e.

4.4.2 Effect of Backslope Inclination

It is very common to have a wall whose height leaves the backslope at an angle "β" as shown in Figure 16. The inclination of the backslope is expressed by either its horizontal-to-vertical dimensions, or by a specific backslope angle, shown as "β".

Using a reinforcement length of L = 0.7H, the computer code generated seepage pressures which were integrated over the wall height to result in a seepage force. This was added



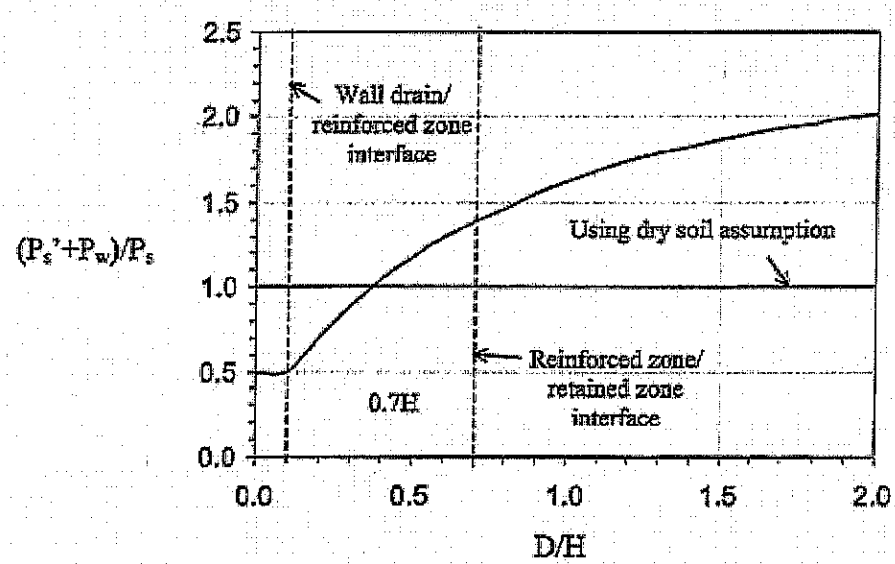


Figure 15 – Effect of location behind wall facing on seepage force

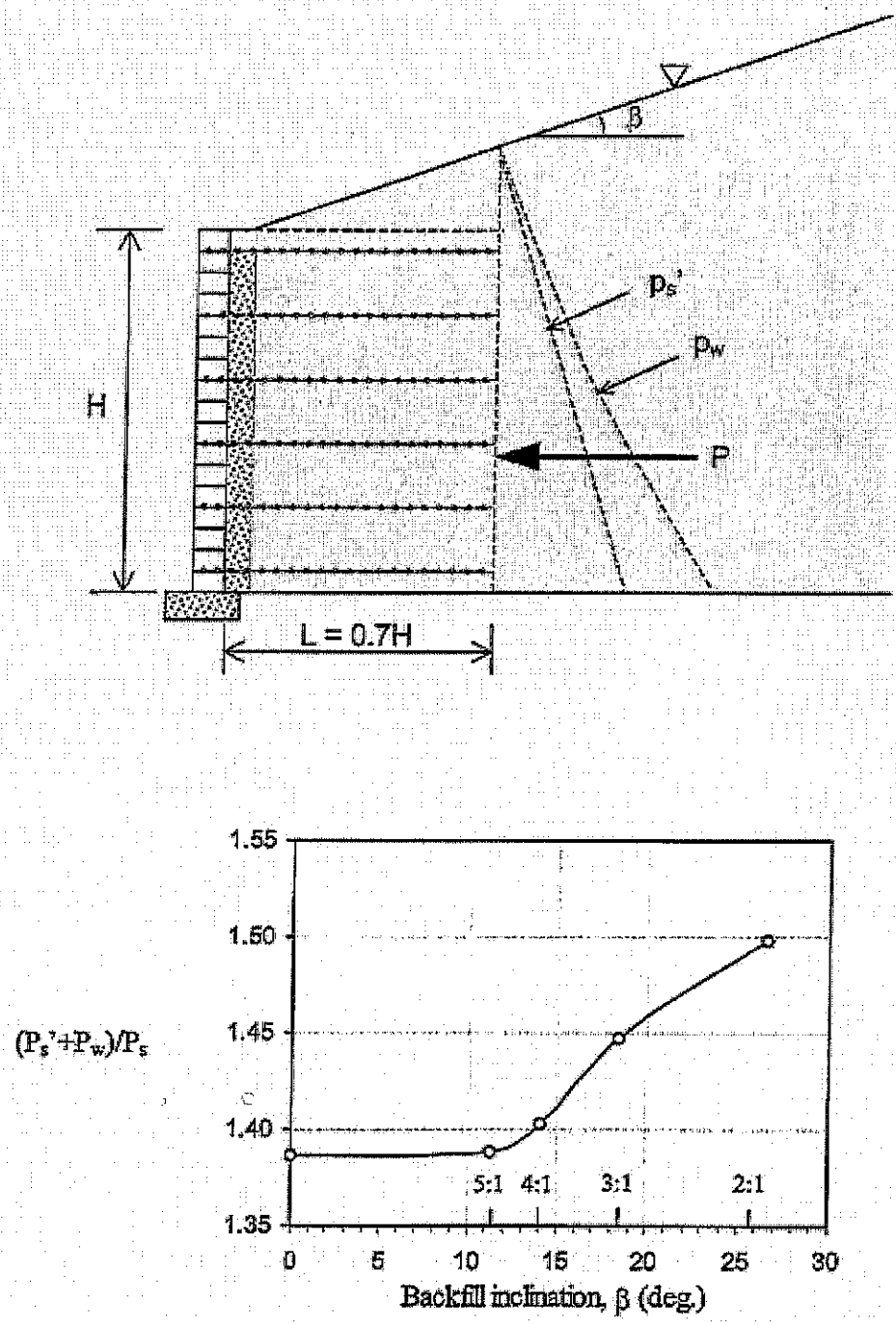


Figure 16 – Effect of backslope inclination on seepage force

to the soil force based on effective stresses, and then nondimensionalized by the force from the soil in its dry state. The resulting value of $(P_s' + P_w)/P_s$ is plotted against inclination angle in Figure 16.

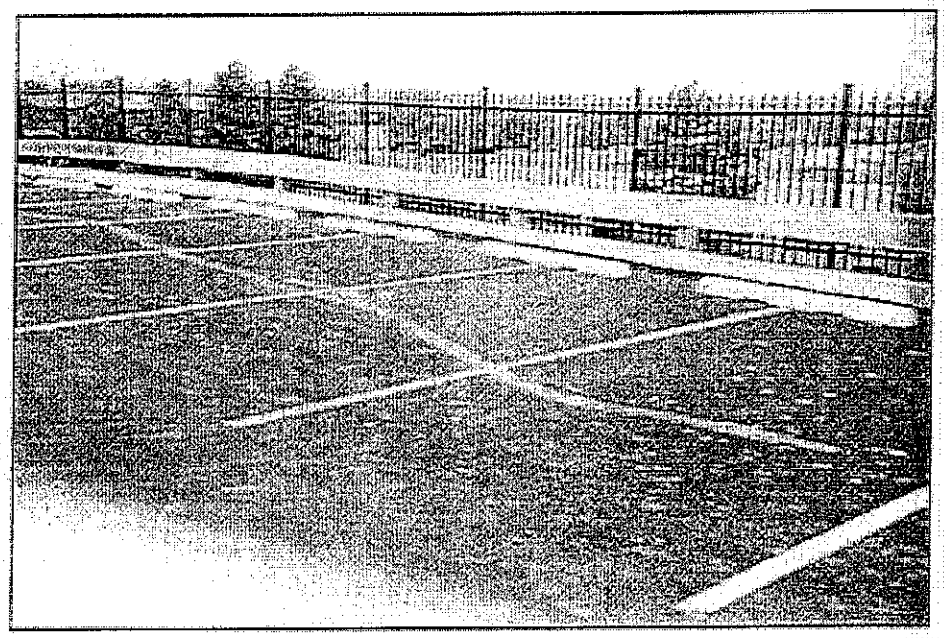
In Figure 16 it is seen that the saturated state increase over the dry state is significant and is approximately constant at 38% for relatively flat backslopes. After approximately 12 deg., i. e., 5(H)-to-1(V), however, the percentage increases rapidly reaching 50% at approximately 26 deg., i.e., 2(H)-to-1(V). The graph was terminated at this point since this is as high as soil can support itself.

Clearly shown by the two exercises of Sections 4.4.1 and 4.4.2 is that saturated soil forces exceed dry soil forces by significant amounts (e.g., the total force can increase by 100%). The response curves of Figures 15 and 16 indicate the actual behavior of these increases.

4.4.3 Effect of Surface Cracking

It is not uncommon to observe surface cracking within the backslope behind SRWs at the end of the reinforced zone. Figure 17 shows photographs of parking lots built on the horizontal backslope of two different SRWs. The end of the reinforcement zone is located exactly where the patched and unpatched cracks are easily visible. Both of the walls eventually failed. These two walls are identified in Table 12 as Walls F13 and F12, respectively. This type of surface cracking was observed in at least two additional wall failures included in Table 12.

In such cases, surface water enters the crack, and, if the soils in the reinforced zone are not sufficiently permeable or are without a drainage system, hydrostatic pressure results. The magnitude of the hydrostatic pressure is a function of the crack depth, "d". Figure 18 illustrates the situation. Note that this hydrostatic pressure will be in addition to the saturated soil's seepage



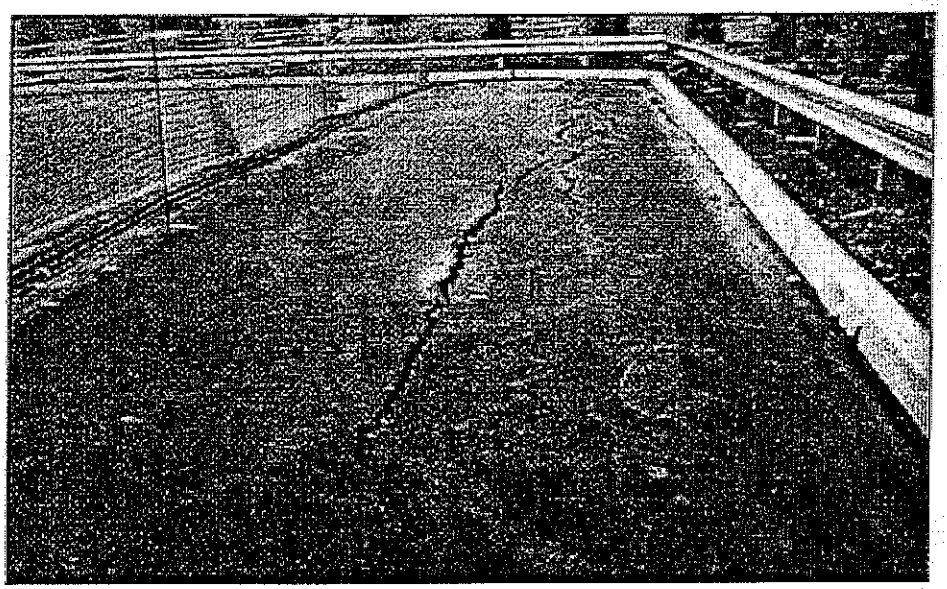


Figure 17 – Examples of small and large surface cracks at end of SRW reinforced zone

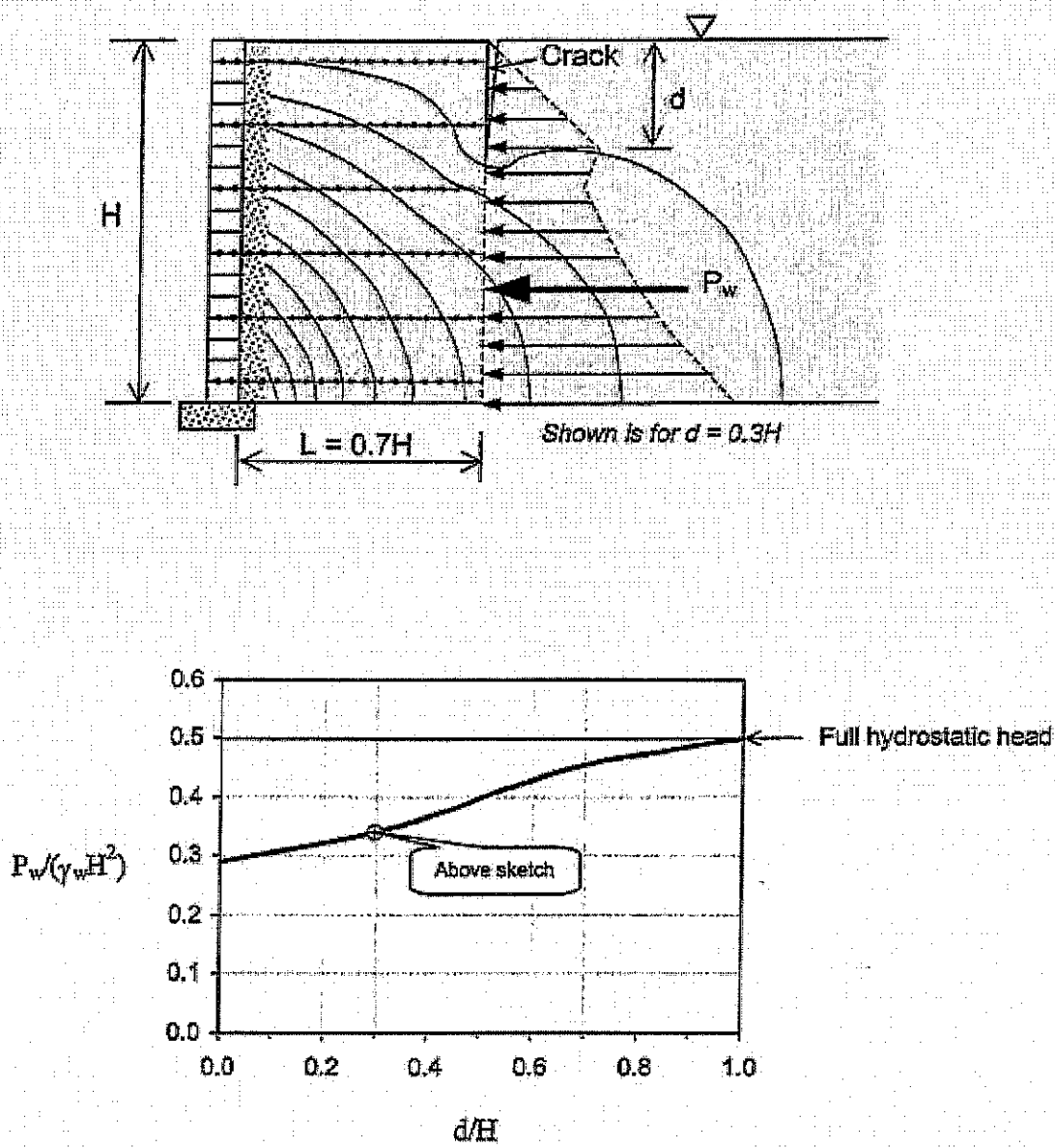


Figure 18 – Effect of surface cracking on seepage force

pressure, which extends from the top of the crack downward. Thus, it represents an additional force to that calculated previously.

For this parametric study and for all others to follow in this section and the next section as well, the seepage force will be used in-isolation, i.e., without comparison to the effective or total soil forces. The specific term used will be "P_w", the seepage force as obtained by integration of the seepage pressure over the height along which it acts. This value of P_w will be nondimensionalized by $(\gamma_w H^2)$, resulting in the term: "$P_w/(\gamma_w H^2)$".

The finite difference code was used to analyze the crack depth variation for a SRW of L/H = 0.7 under varying d/H-ratios. The results are plotted in Figure 18 against the nondimensionalized seepage force $P_w/\gamma_w H^2$. For the case of no surface cracking, the value is 0.29. As the depth of crack increases, the seepage force increases until at its full depth it results in a value of 0.50.

This situation represents the worst case scenario where full seepage force via the relatively impermeable reinforced zone backfill soil is mobilized and, in addition, the hydrostatic force from water in the crack at the back of the reinforced zone is superimposed. The wall system (now consisting of facing, drainage zone and reinforced zone) is acting as a dam in the same manner as was illustrated in Figure 10a and shown in Figure 10e.

4.4.4 Effect of Permeability Ratios

The issue of relative permeability of reinforced zone soil to the retained zone soil (in the form of a ratio "k_{rein}/k_{ret}") is at issue in this subsection. The ratio will be viewed from the perspective of the reinforced zone soil permeability, i.e., high or low, with respect to the retained zone soil permeability.

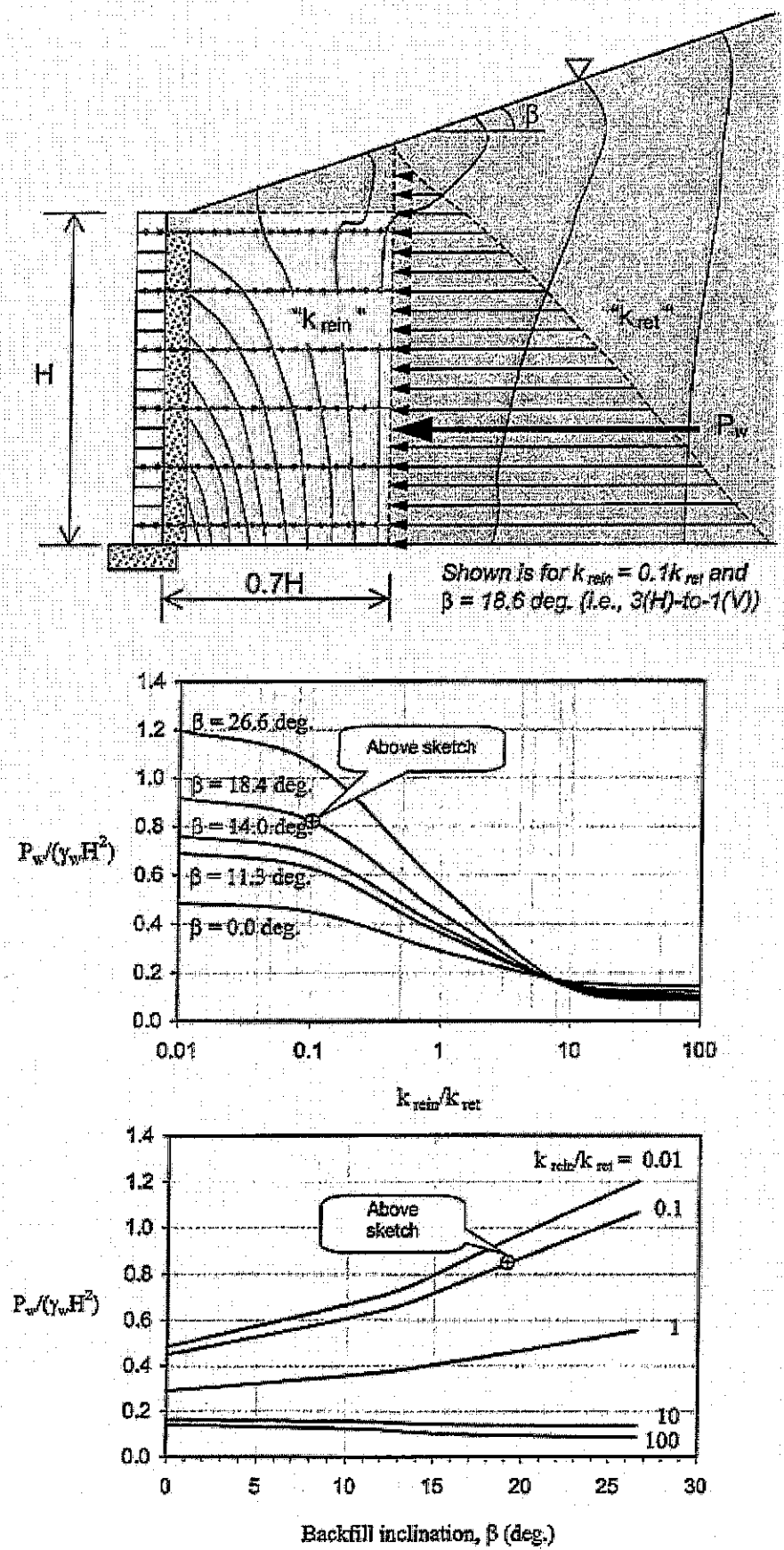


Figure 19 – Effect of permeability ratio on seepage force

Values of "k_{rein}/k_{ret}" were varied from 0.01 to 100 in units of ten. The computer code generated seepage pressures for a SRW of L/H = 0.7 as seen in Figure 19. The results are given as the nondimensionalized seepage force $P_w/(\gamma_w H^2)$. As seen in Figure 19, the backslope inclination was also investigated since it was seen to be significant. The trends in the curves are of interest.

- As the permeability of the reinforced zone soil decreases, the ratio k_{rein}/k_{ret} decreases, and the seepage forces increase. The respective values become essentially constant for values lower than $k_{rein}/k_{ret} \simeq 0.1$.
- For high permeability of the reinforced zone soil, the ratio k_{rein}/k_{ret} increases, and the seepage force decreases, becoming essentially constant higher than $k_{rein}/k_{ret} \simeq 10$.
- The trend between the above two extremes of high-vs-low relative permeability of the reinforced zone soil is very significant and somewhat nonlinear.
- As the backslope inclination increases, the above trends are exacerbated particularly when the permeability of the reinforced zone soil is very low with respect to the retained zone soil.
- The lower set of graphs shown in Figure 19 is interchangeable with the center set, the only difference being the change in x-axis with the plotted data.

4.4.5 Effect of Anisotropic Permeability

During the discrete lift placement of soil backfill in the reinforced zone it is possible that the horizontal permeability might increase over the vertical permeability. This possibility is further enhanced by virtue of the presence of the reinforcement layers which are placed horizontally and represent potentially higher permeability paths.* As such, this subsection

*This latter feature will be capitalized upon in the next section where high transmissivity geotextiles are purposely used to enhance horizontal drainage through the reinforced zone.

evaluates anisotropic permeability ratio effects in the form of "k_h/k_v" varying from 1 to 100. Variation of the k_h/k_v ratio in the upper sketches of Figure 20 is readily seen to cause a "spreading out" of the equipotential lines. The left sketch is for $k_h/k_v = 1.0$, and the right sketch is for $k_h/k_v = 100$. The seepage pressure diagrams were integrated over the wall height to give a seepage force, which was nondimensionalized in the form of $P_w/(\gamma_w H^2)$. Also included in this subsection is the effect of the distance behind the back of the wall via a D/H ratio.

The results are shown in the two lower graphs of Figure 20, where the only difference is the transposition of the x-axes between D/H ratios and k_h/k_v ratios. The curves clearly indicate that the seepage force is a maximum for the isotropic case of $k_h/k_v = 1.0$. As the ratio increases, i.e., as the soil becomes higher in its horizontal permeability over vertical permeability, the seepage force decreases. Hence, the reinforced zone soil becomes more drainable resulting in lower seepage forces. Also seen in the curves is that lower D/H ratios result in lower seepage forces confirming the similar finding shown in Figure 15.

4.4.6 Effect of Tapered Reinforcement Length

For relatively high walls where base sliding stability is not an issue, it may prove economical to use a tapered length for the various layers of reinforcement, see Figure 21. With low permeability backfill soil in the reinforced zone, seepage pressures can obviously occur, except now the orientation is perpendicular to the potential failure plane as shown. It is interesting to note that this exact problem (although without the geosynthetic reinforcement) was worked out by-hand by Gray in 1958.

The situation that was selected for analysis is a failure plane inclination at 63.5 deg. which corresponds to a ϕ-angle of 37 deg.* Also varied is the permeability ratio k_{rein}/k_{ref}. The

*A single value was selected due to the restrictiveness of the grid pattern selected for this study. Finer grid patterns will allow greater flexibility in this regard.

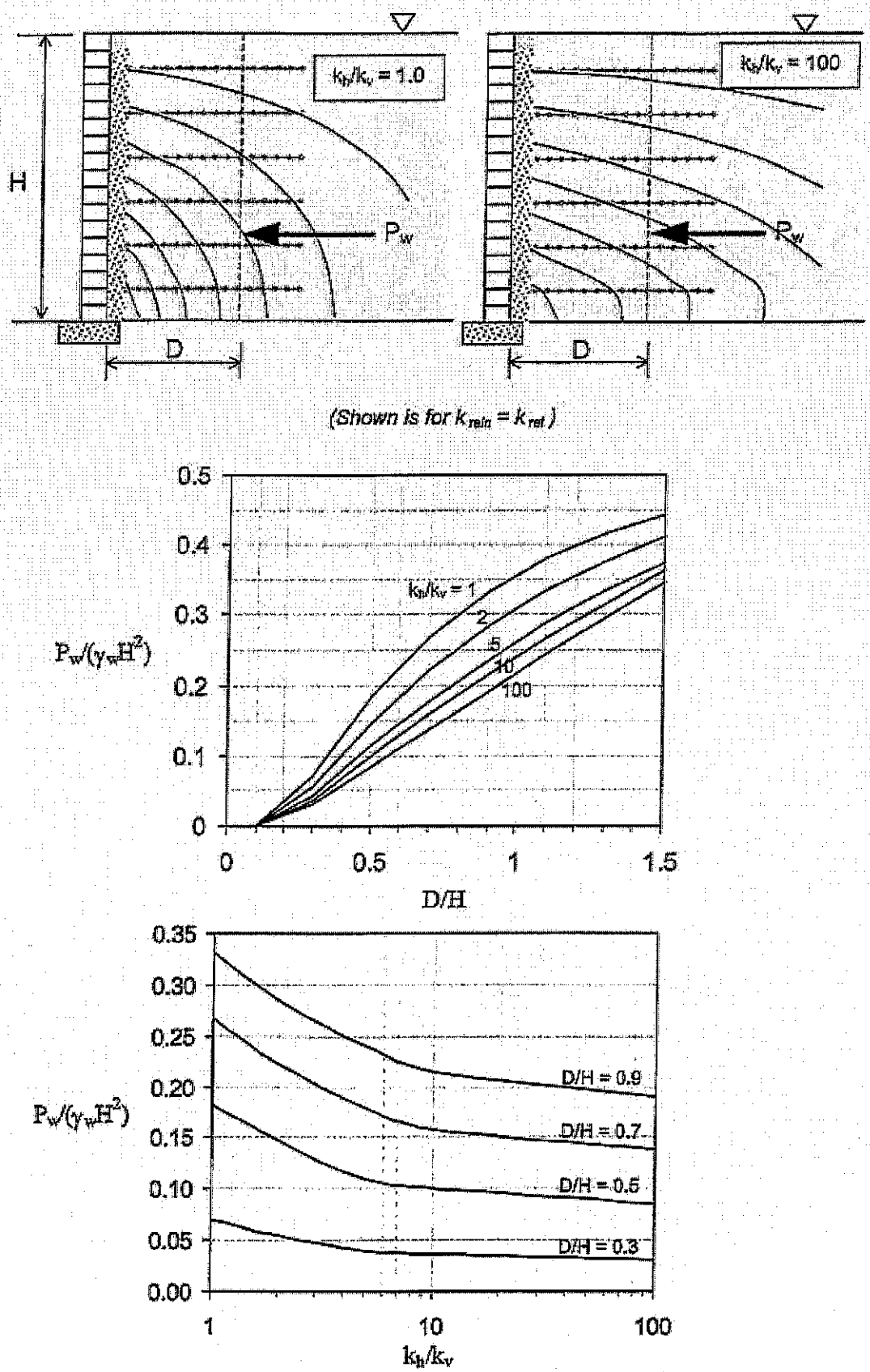
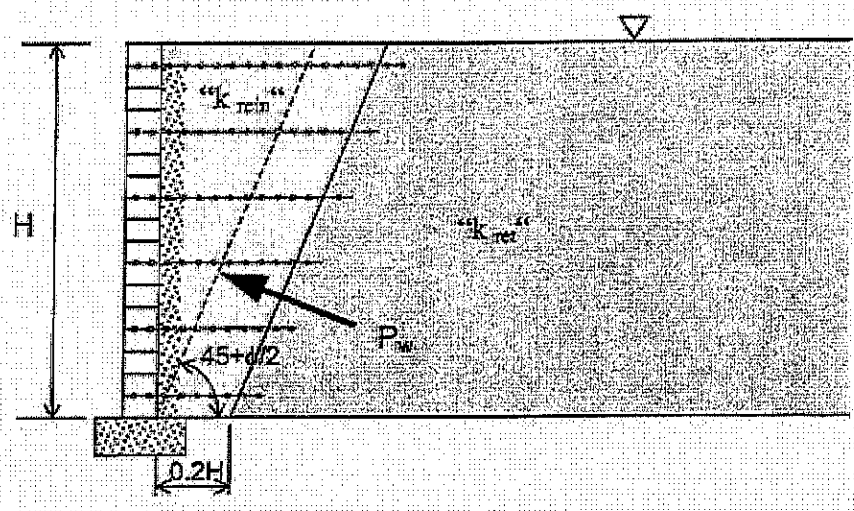
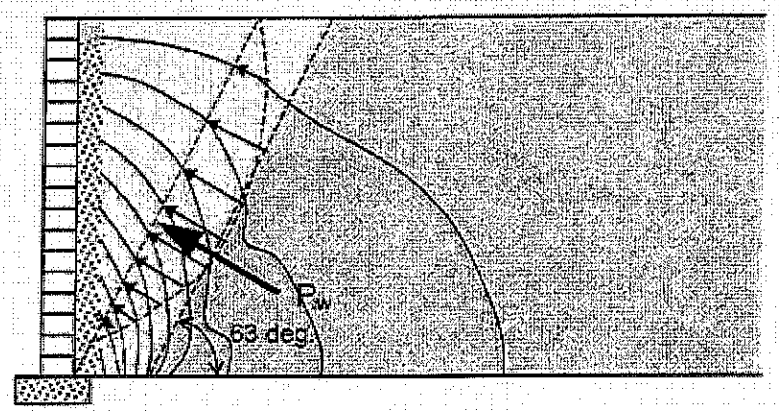


Figure 20 – Effect of anisotropic permeability on seepage force





(Shown is for $k_{rein} = 0.1k_{ret}$)

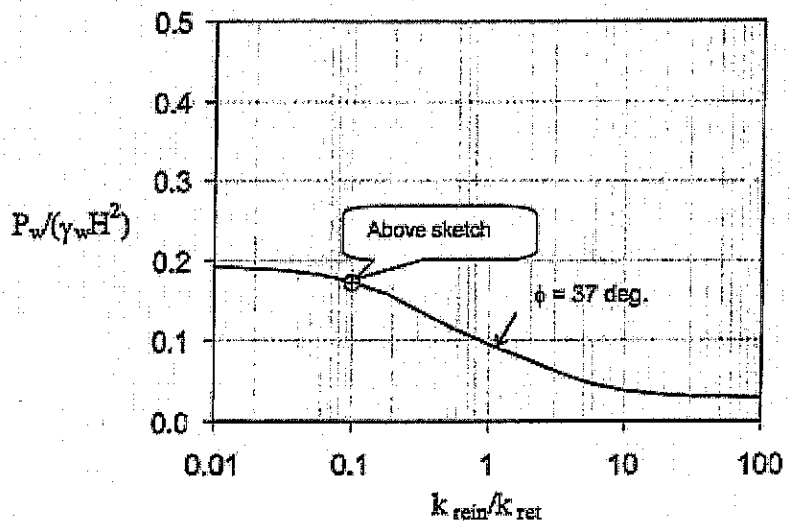


Figure 21 – Effect of tapered reinforcement length on seepage force

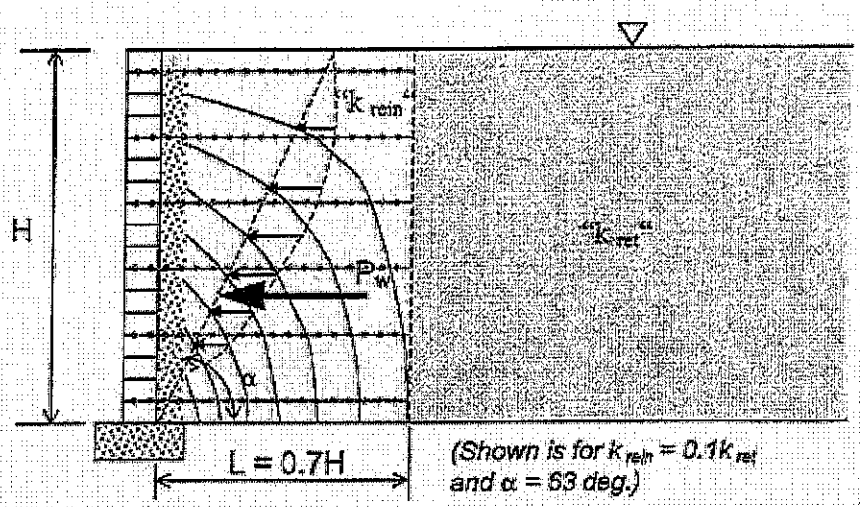
response curve of Figure 21 is in terms of the nondimensionalized seepage force, i.e., $P_w/(\gamma_w H^2)$. Here it is seen that the seepage force is the highest for low permeability ratios (such as low permeability reinforced zone soil) and almost nonexistent for high permeability ratios (such as high permeability reinforced zone soil). The response is also seen to be quite nonlinear.

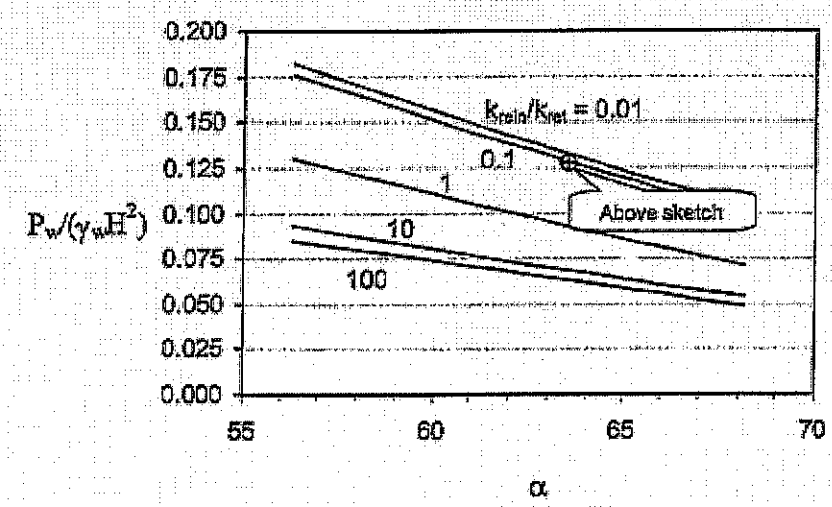
4.4.7 Effects of Inclination Within the Reinforced Zone

It may be of interest to investigate the seepage pressure within the reinforced soil zone. This can be done by using a inclined plane rising at an angle of "α" from the inside toe of the wall as shown in Figure 22. The ultimate purpose of such an exercise is for internal stability calculations regarding the required tensile stress in the reinforcement layers. The angles selected are 56, 63 and 68 deg., which correspond to soil ϕ-angles of 22, 36 and 46 deg., respectively.

The wall selected has a reinforcement length of 0.7H, a horizontal backslope, and permeability ratios (k_{rein}/k_{ret}) of 0.01 to 100, in increments of ten. Results are reported in units of the nondimensionalized seepage force, $P_w/(\gamma_w H^2)$.

The results are shown in the two sets of curves of Figure 22, where are only difference is that their x-axes have been transposed between α values and k_{rein}/k_{ret} values. For high values of "α", the seepage forces decrease since the drain behind the wall is having a relatively greater influence than it has for low values of "α". The trend is quite linear in this regard. Also note that the seepage force is highest for low permeability ratios (such as low permeability reinforced zone soil) and very low for high permeability ratios (such as high permeability reinforced zone soil). This same trend was also observed in Figures 19 and 21.





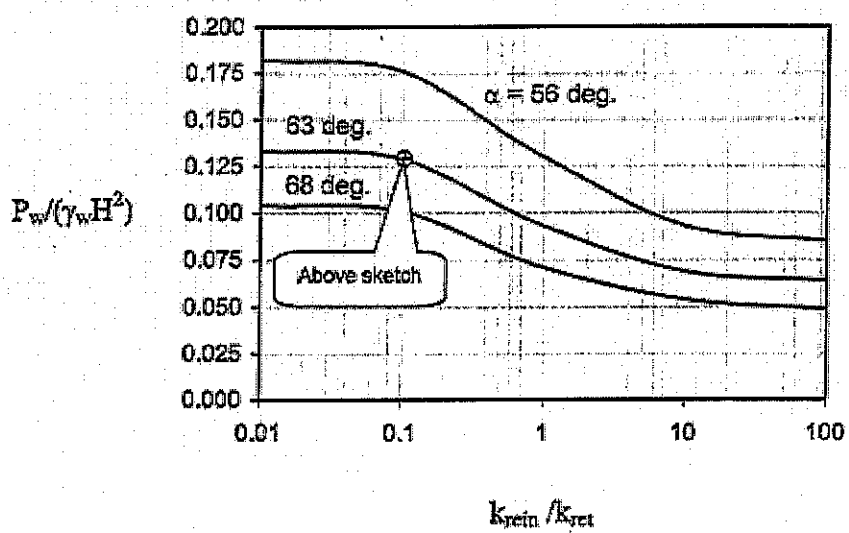


Figure 22 – Effect of seepage force within reinforced zone

APPENDIX B

FULL-SCALE FIELD TEST

CONTENTS - APPENDIX B

FIGURES

TABLES

ATTACHMENTS

ATTACHMENT B

Attachment B cont.

Attachment B cont.

EXECUTIVE SUMMARY

The National Cooperative Highway Research Project 24-22 (NCHRP 24-22) aims to develop material selection guidelines, soil parameters, testing methods, and construction specifications that will allow the use of more fine-grained backfill materials within the reinforced zone of mechanically stabilized earth (MSE) retaining walls. The objective of the research is to provide results that allow the use of more fine-grained soils as backfills for MSE structures, thereby lowering their construction cost. The estimated potential savings from replacing AASHTO A-1-a reinforced fill materials with "higher fines" reinforced fill materials could be in the range of 20 to 30% of current MSE wall costs, especially in areas where relatively clean granular soils are not readily available.

The project work consisted of three phases: survey current practice and design full-scale field test; conduct field test and draft design aids for backfill materials; and prepare final report materials for the work. GeoTesting Express, Inc. of Boxborough, MA was awarded the contract for this work and began work in August 2003. This report provides a summary of the research findings, guidelines for selecting MSE backfills, representative soil parameters, and appropriate testing methods and construction specifications for a wide variety of backfill materials. Appendix A summarizes the results of the survey of current practice of MSE retaining wall backfill design. This appendix, B, documents the field tests, the measured performance and our evaluation of that performance.

Four full-scale field tests were conducted, in order to establish performance for "high fines" reinforced soils and associated design controls that give acceptable MSE wall performance. The walls were constructed with welded wire basket face elements and polyester geogrid reinforcement, except that one wall used geotextile reinforcement. Each wall was 20 ft. high and sufficiently long to remove end condition effects. The field tests included provisions to demonstrate the role of pore water pressure in the reinforced fill and the importance of including a positive drainage system in the backfill to obtaining good wall performance. Each test section was fully instrumented to obtain performance data with which to address a number of technical questions. Instrumentation consisted of strain gages mounted on the geosynthetic reinforcement; piezometers, thermistors, multiple position horizontal extensometers and vertical extensometers positioned throughout the reinforced fill; vertical inclinometers; and an array of high precision prisms mounted on the face of the test walls on which optical survey readings were obtained using Automated Robotic Total Station technology. The monitoring system worked very well and produced an enormous amount of detail about the performance of each wall. The Automated Total Monitoring Stations were particularly valuable at providing detailed information on the vertical and horizontal movements of numerous points located on the face of the walls and settlement of plates buried in the reinforced fill.

The tests showed that:

- Mechanically stabilized earth retaining walls can withstand positive pore water pressures, provided they are designed to do so. It was observed that increasing the pore pressure in the reinforced fill causes lateral and vertical deformations of the wall that are as significant as those resulting from adding surcharge loads.

- Soils with an AASHTO A-1-b classification (e.g. soils with as much as 25% fines and a Plasticity Index below 6%) can be successfully used as backfill materials in MSE structures, provided the design uses the appropriate material properties and takes into

consideration any positive pore pressures that may develop in the backfill over the life of the structure.

Soils with high fines and some plasticity have lower permeability which creates more difficulty placing the materials to the desired condition. Moisture and density control become more important and can be more difficult to achieve. Construction with these materials is more weather dependent. Consequently, a higher degree of quality control is necessary to make sure the materials are placed and appropriately compacted to minimize future performance issues. The lower permeability of soils with high fines and some plasticity also creates the possibility for unintended pore pressures to build up in the reinforced fill and cause adverse performance, if not explicitly considered in the design. This result was clearly demonstrated when the test site experienced 15 inches of rain over a one month period. Large lateral movements near the face of the walls occurred, which we believe to be the result of a buildup in pore pressure near the face of the walls of sufficient magnitude to cause the wire basket facing elements to slide over the geosynthetic reinforcement. Due to the potential for such unplanned behavior, it appears that backfills with permeabilities less than 10^{-3} cm/sec should be designed with positive drainage measures at the back of the reinforcing and provisions to prevent surface water from entering the reinforced fill material.

Due to the significant effects of pore pressure build up in the reinforced zone on stability and deformations and the complications and uncertainties introduced by the added forces from pore pressure increase, we conclude that the best practice for using more fine-grained soils in MSE structures is to adopt measures the minimize the potential of pore pressure build up in the reinforced fill. By preventing pore pressure build up, the more fine-grained soils have adequate strength and stiffness characteristics to provide a wall that functions safely and with acceptable deformation. All potential sources of pore pressure build up in the reinforcing zone should be considered in the design, including horizontal flow from the retained soil, downward flow from surface water and upward flow from the foundation. Where potential sources exist or may develop during the life of the facility, the design should include specific measures to prevent flow of water into the reinforced fill in the form of a water barrier, a drain or both.

The research clearly demonstrates that MSE structures with higher fines soils in the reinforced fill than currently allowed by AASHTO will provide excellent performance subject to the following considerations:

- Soils with less than 25% fines and a Plasticity Index not greater than 6% are used. All potential sources of pore pressure build up in the reinforcing zone should be considered in the design.
- Good practices are used to control the quality of material selection and placement, as the performance of higher fines soils is more susceptible to as-placed density and moisture content than free draining materials.

The report proper contains more guidance on implementation of these recommendations.

CHAPTER 1

INTRODUCTION AND WORK PLAN

The National Cooperative Highway Research Project 24-22 (NCHRP 24-22) aims to develop selection guidelines, soil parameters, testing methods, and construction specifications that will allow the use of a wider range of backfill materials within the reinforced zone of mechanically stabilized earth (MSE) retaining walls. The objective of the research is to provide results that allow the use of a wider range of soils as backfills for MSE structures, thereby lowering their construction cost. The work consists of three phases: survey current practice and design full-scale field test; conduct field test and draft design aids for backfill materials; and prepare final report materials for the work. Table 1-1 provides a summary of the project work plan. GeoTesting Express, Inc. of Boxborough, MA (GTX) was awarded the contract and began work on in August 2003.

The results of the research are documented in this report, which provides a summary of the research findings, guidelines for selecting MSE backfills, representative soil parameters, and appropriate testing methods and construction specifications for a wide variety of backfill materials. Appendix A summarizes the results of the survey of current practice of MSE retaining wall backfill design. Appendix B documents the field tests, the measured performance and our evaluation of that performance.

Table 1-1: WORK PLAN FOR MSE BACKFILLS

PHASE I: Current Practice

- *Task 1*: Synthesize current practice for design, specification and construction of MSE backfill from literature.
- *Task 2*: Synthesize current practice for design, specification and construction of MSE backfill from survey of manufacturers and transportation agencies in US, Canada, Europe and Asia.
- *Task 3*: Develop parameters for lowest-quality backfill soils that give acceptable performance and work plan for full-scale field test.
- *Task 4*: Provide interim report and meet with NCHRP.

PHASE II: Full-scale Field Tests

- *Task 5*: Test backfill materials and make numerical analyses for field test wall systems.
- *Task 6*: Construct and run field tests.
- *Task 7*: Draft guidelines and specifications for suitable backfills for MSE retaining walls and meet with NCHRP.

PHASE III: New Guidelines and Specifications

- *Task 8*: Finalize guidelines and specifications for suitable backfills for MSE retaining walls.
- *Task 9*: Prepare final report.

1.1 PURPOSE OF FULL-SCALE FIELD TEST

The survey of the literature provided in Appendix A of this research indicates that many walls with “high fines” reinforced soil performed satisfactorily. However, there are cases, for both metallic and geosynthetic reinforced MSE walls, where the use of “high fines” and/or “high plasticity” soils in the reinforced zone contributed to the problems in stability for the walls that

resulted in unacceptable performance. Water pressure in the reinforced zone was the major cause for either excessive deformation or collapse of those walls. Based on this review, it appears that a higher quantity of fines could be safely allowed in the reinforced fill, provided: (1) the properties of the materials are well defined, and (2) controls are established to address the performance issues and limit the development of positive pore water pressures in the reinforced fill.

In order to establish properties for "high fines" reinforced soils and associated design controls that give acceptable performance, the NCHRP Review Panel approved the conduct of a full-scale field test. The field test included provisions to demonstrate the role of pore water pressure in the reinforced fill and the importance of including a positive drainage system behind the reinforced fill zone to obtaining good wall performance. Based on the survey of the literature, only a few full-scale test or experimental MSE walls have been constructed, but none of those have rigorously evaluated the seepage issue associated with the use of "high fines" reinforced fill.

The project originally included testing of three full-scale sections as illustrated in Figure 1-1(a). At a September 2004 review meeting, the Panel voted to request continuation funds from NCHRP for construction of a fourth test section as part of the full-scale field test. The research team subsequently provided a budget estimate to the NCHRP 24-22 Program Officer for this additional work. In April 2005, the AASHTO Standing Committee on Research approved additional funds for the fourth test section. Thus, the revised work plan (Figure 1-1(b)) includes four test sections.

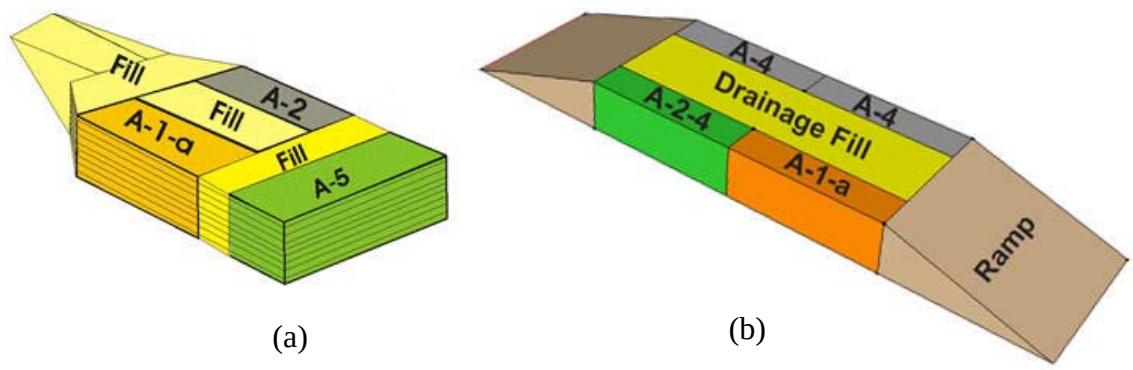


Figure 1-1: (a) Original Proposed and (b) Revised layout of Field Test for MSE Retaining Walls

1.2 SITE LOCATION

The test site was located at a private site presently occupied by an active sand and gravel borrow and rock quarry operation, located in Massachusetts, just east of Leominster; it is approximately 15 miles from the GTX headquarters in Boxborough, MA and about 45 miles northwest of Boston. A relatively flat location in a "played out" area of the gravel pit, away from any active operations, was provided by the P.J. Keating Co.

Figure 1-2: Location of Test Site

GTX personnel supervised the construction. GTX retained the services of the gravel pit operator to provide earth moving equipment, borrow materials and construction labor. Instrumentation technicians from GTX's sister company, Geocomp Corporation, provided and installed the instrumentation and monitoring system. As many of the instruments as possible were electronically connected to a real-time WEB-based data acquisition system, to allow us to detect any problems at an early stage and to facilitate the dissemination of results among our team members in an efficient and effective way.

1.3 Work Plan

1.3.1 Layout and Design of Field Test Sections

Figure 1-3 illustrates the field test layout that fit within the site constraints and met the project objectives. This layout permitted the simultaneous testing of four test sections. Each test section was 20 ft. high and 60 ft. long. The wall sections were designed so that they demonstrate acceptable performance for the normal design conditions, but to show distress when subjected to extreme conditions of high water pressures in the reinforced fill and a surcharge load. The width of fill behind the reinforced fill was established by analysis to minimize boundary condition effects on the test results.

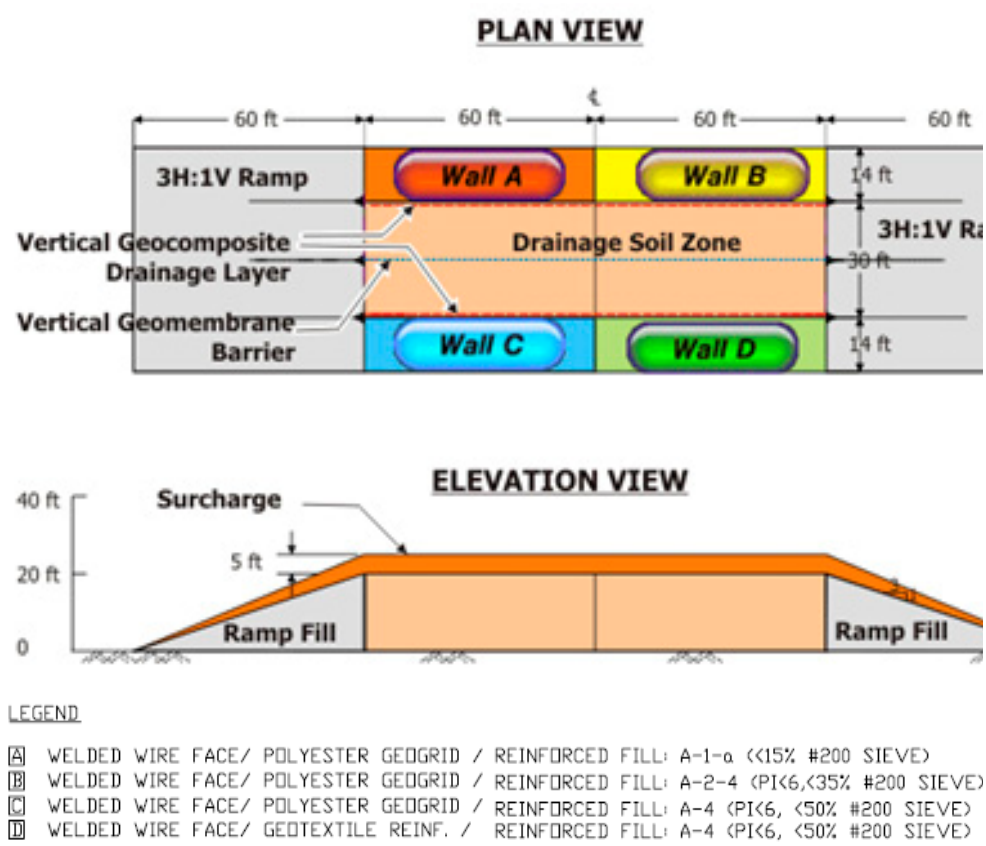


Figure 1-3: Plan/Elevation of Field Test for MSE Retaining Walls

Table 1-2 Summary - Wall Details

Wall	Reinforced Fill	Reinforcement	Comment
A	AASHTO A-1-a (granular material with 15% or less passing the 0.075mm/#200 sieve and Plasticity Index less than or equal to 6%), Reinforced fill with approximately 13% silt content	Polyester Geogrid	Provides a baseline of performance for current AASHTO standards
B	AASHTO A-2-4 (a granular material with 35% or less passing the 0.075mm/#200 sieve and Plasticity Index less than or equal to 10%), Reinforced fill with approximately 25% silt content	Polyester Geogrid	Evaluate performance of non-plastic, silty sand materials with up to 35% fines (of no plasticity)
C	AASHTO A-4 (a silt-clay soil with more than 35% passing the 0.075mm/#200 sieve and a Plasticity Index less than or equal to 10%), Reinforced fill that behaves like a silt.	Polyester Geogrid	Evaluate high-fines silty materials
D	AASHTO A-4 (a silt-clay soil with more than 35% passing the 0.075 mm sieve and a Plasticity Index less than or equal to 10%), Reinforced fill that behaves like a silt.	Non-woven geotextile	Evaluate high-fines silty materials

The drainage zone in the center of the test wall structure was locally sourced sand. Specific properties of the soils actually used for the test sections are given in Chapter 2 of this report volume.

It is worth noting that a wall comprised of clay reinforced fill was not tested in this program. Available information in the literature and the response summary from the Task 2 survey presented Volume II of this research indicate that appropriate high-fines reinforced fills were largely considered to be non-plastic and having between 15 and 35% of material passing the #200 sieve. Clay soils as reinforced fill have the potential to cause too many construction difficulties which may negate the economic benefit of using on-site clay soils. Furthermore, the literature review summarized in Volume II of this research demonstrates that clay soils have routinely contributed to excessive MSE wall deformations and wall collapses.

1.3.2 Groundwater/Rainfall Simulation and Monitoring Pore water Pressure Effects

An essential component of an MSE retaining wall that uses reinforced fill with "high fines" soil is aggressive drainage to prevent the buildup of pore water pressure in the reinforced zone. Pore pressures produce an additional outward force that the wall must resist, and reduce the strength of the soil that holds the wall in place. Our review of case studies summarized in Volume II of this research indicates that pore water pressures in the reinforced soil zone invariably play a major role for wall serviceability problems. In some cases complete collapse of walls occurred.

The field test section walls included provisions to demonstrate the role of pore water pressure on the performance of the reinforced fill, and the importance of including a positive drainage system to obtaining good wall performance. Figure 1-4 shows how this was accomplished. A geocomposite drainage material was placed at the back of the reinforcement zone in each test section. The drainage material was wrapped around a slotted drain pipe at the bottom of the reinforced fill that facilitated the removal of water. By controlling the pressure in the water supply pipe with the slotted drain pipe open, the effect of rising groundwater level on the performance of the constructed wall can be simulated. We would expect little if any effect on the test sections as long as the geocomposite drains function as designed. This phase of the test was intended to demonstrate that various reinforced fill materials will provide suitable performance, even in areas with high groundwater conditions, as long as they are properly drained.

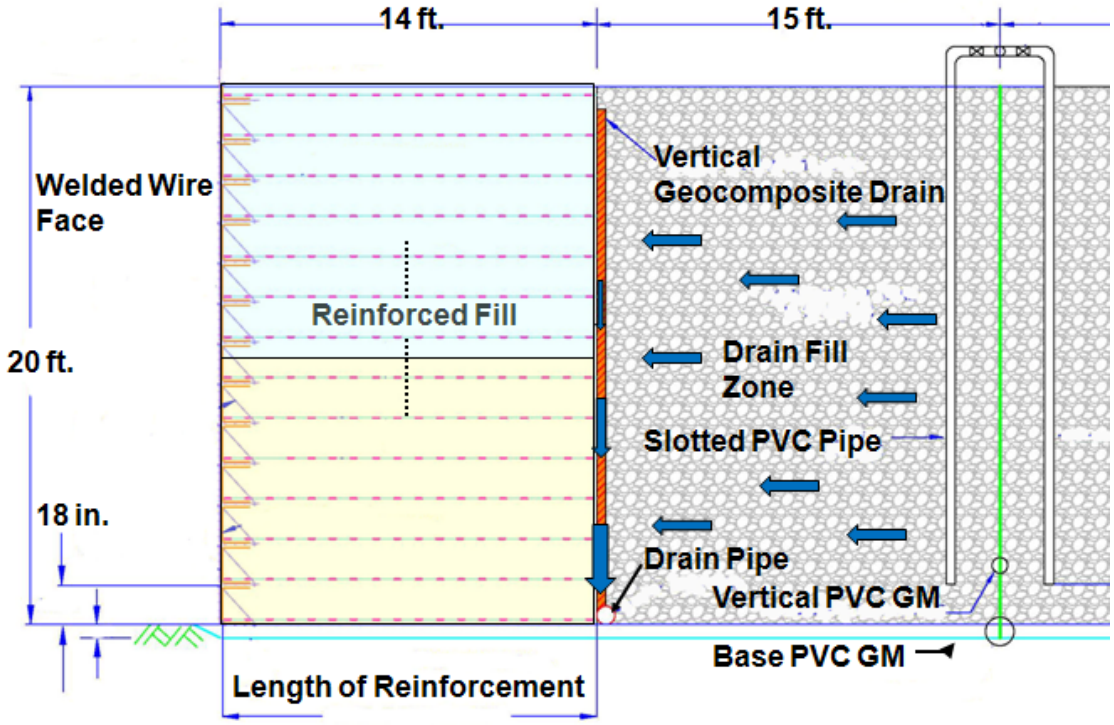


Figure 1-4: Simulation of Groundwater and Rainfall

By closing a valve on the drain pipe and spraying water on top of the reinforced fill zone, the effects of poor drainage and heavy rainfall can be simulated. Keeping the valve on the drain pipe closed the pore pressures in the reinforced fill can be further increased until the wall experiences noticeable distress. This phase provided valuable information to evaluate the ability of the numerical models to consider the effects of pore pressure.

The walls are designed so that they should experience considerable distress when subjected to additional surcharge loading and high pore pressures. To evaluate the failure conditions for each test section wall, the test sections were drained, and a surcharge was added, and the test sequence to measure the effects of groundwater and rainfall was repeated. This phase of the program provided the opportunity to check the ability of numerical models to predict a factor of safety at the only place we can measure it, i.e. at a value of 1.0 (also called incipient failure).

Figure 1-5 illustrates the test sequence that was conducted. It consisted of the following steps:

- Construct the MSE wall to 20 ft. height with a geocomposite drain located at the back of the reinforcing elements.
- Monitor the wall through the winter season with soil at its natural (aka, low) in-situ moisture content, to measure effects of freeze-thaw on wall performance and reinforcing elements.

- In the spring, raise water levels in the fill behind the wall to within 1 ft. of ground surface with the geocomposite drain open and functioning. This is to demonstrate that the design will work for high groundwater conditions, if proper drainage is in place and working.
- Close off the geocomposite drain and let pore pressure rise in the reinforced fill until some distress is observed in wall or reinforcing elements.
- Drain reinforced fill and monitor response of wall under capillary heads.
- Monitor the wall through the winter season with soil at high in-situ moisture content.
- Add surcharge.
- Raise water level in fill behind the wall to within 1 ft. of ground surface with geocomposite drain open and functioning. Wall is designed to support a 5 ft. surcharge under this groundwater condition without unacceptable distress.
- Close off geocomposite drain and let pore pressure rise in reinforced fill until failure of wall occurs. This will provide an important calibration of the ability of the numerical models to predict factor of safety for a value of 1.0.

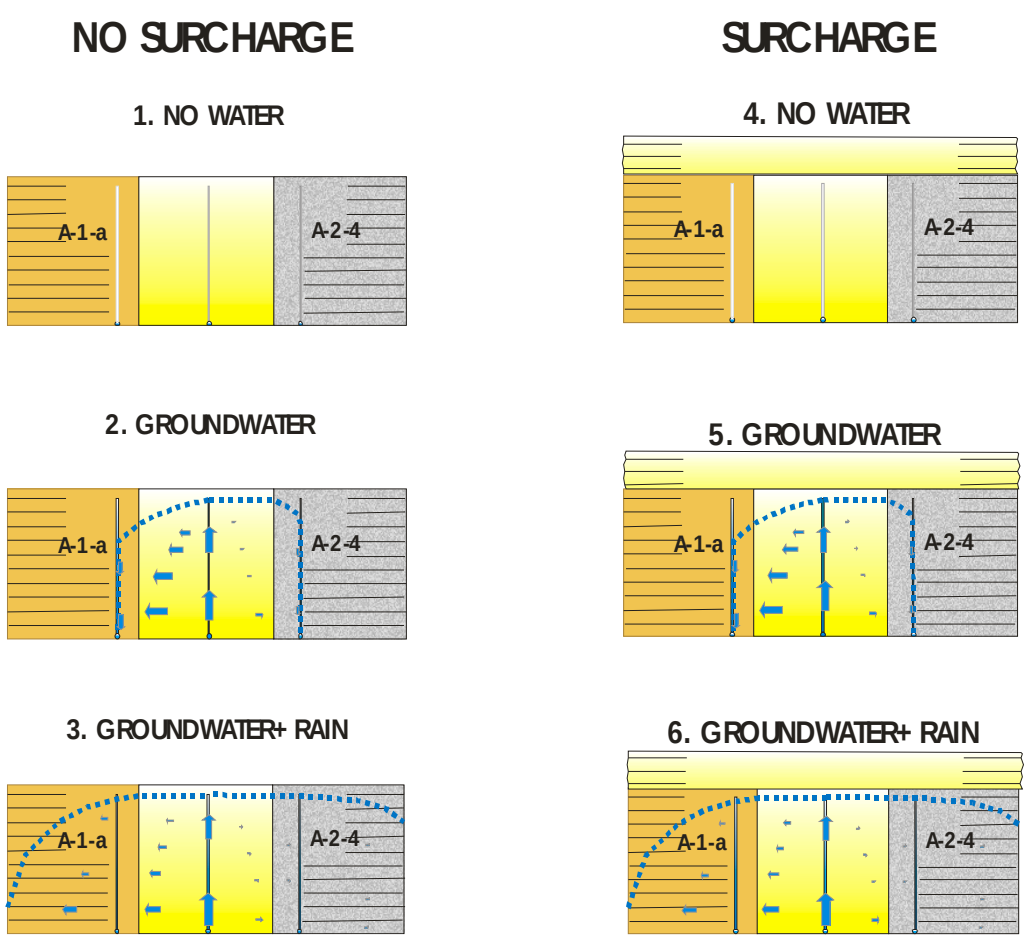


Figure 1-5: Test Sequence for Full-scale Test Walls

1.3.3 Seasonal Effects

The long duration project time scale provided the opportunity to subject the test section walls to two seasons of winter conditions. Since high-fines reinforced soils may be more susceptible to freeze-thaw conditions, evaluating this performance was an important aspect of the field test.

1.3.4 Instrumentation Program

Table 1-3 summarizes the potential questions or concerns that were addressed in the full-scale field test. For each question, the technical reasoning for the validity of the question and a proposed monitoring solution as part of the full-scale field test are provided.

Table 1-3 Instrumentation Program – Full-scale Field Test

Technical Question	Discussion	Monitoring Approach
What is the distribution of pore pressures in the reinforced fill mass?	Excess pore pressures can produce an additional outward force that the wall must resist Water also reduces the strength of the reinforced fill that holds the wall in place.	Install multiple piezometers (e.g. Vibrating wire) at selected positions throughout the reinforced fill to evaluated seepage pressures. Monitor sensors often, using automated system to capture short-term and long-term changes/trends.
What loads are being carried by the reinforcing elements and where is the location of the failure surface?	Measuring the loads in the reinforcement will help in the assessment of the numerical models used during design, and to predict and develop the means to induce wall failure. It is important to measure loads locally and over larger reinforcement elements. Local loads may give misleading results, due to imperfections in reinforcing material or hard/soft spots in the reinforced fill. Measuring average loads over longer elements will miss peak strains along failure surfaces.	Since load cannot be easily measured in the reinforcement elements, strains are measured and equivalent loads calculated based on the reinforcement material's physical properties. Use strain gages to measure localized strain on the reinforcing material. Use horizontal rod extensometers to measure strain over a larger gage distance. These two independent measurements of strain provide cross-checks of the measurements and redundancy in the strain measuring system. Monitor sensors often, using automated logging system to capture short-term and long-term changes/trends.

Technical Question	Discussion	Monitoring Approach
What are the lateral deflections at/near the face of the wall during construction, loading and failure?	Excess pore pressures, surcharges and combination of the two will cause outward forces that lead to bulging in the wall face and possibly failure.	Install inclinometers along the inside face of the outer wall deep into the foundation material. Manual reading may also be performed after each test phase, to validate automated system data.
What are the 3-dimensional wall deflections during construction, loading and testing?	Deflections during construction may affect the verticality and stability of the wall and wall face. Loading may result in differential settlement between the wall elements - may lead to cracking, separation, seepage.	Install reflective surveying prisms/targets at various elevations along the wall face. Targets will also be affixed near the top of the inclinometer casings, where visible. Targets will be surveyed relative to independent benchmarks during construction and testing. Targets will be manually read, unless automated systems become a more cost-effective option.
Is there any slip between the reinforcement and the retained soil?	Pull-out failure may occur, if the resistive shear strength of the soil-reinforcement interface is exceeded. This may occur during wall construction (most-likely in the upper layers of reinforcements where confining stresses are low) or as a result of soil-softening during wetting of the material after construction.	Horizontal rod extensometers will extend beyond the limits of the reinforcement, to record any differential movements at the tail of the reinforcing element.
What are the horizontal stresses acting on the back of the wall face?	The reinforcement design is based on the predicted horizontal earth pressures on the wall facing. These stresses will depend on the compaction effort when placing the retained soil, and will change with water presence (notably if the soil swells when wetted) and due to seasonal changes (freeze/thaw cycles).	Total lateral earth pressure cells positioned behind wall face can measure horizontal stresses that develop during wall construction and testing. However, such measurements are not accurate due to compliance issues within soil pressure gages and difficulty in obtaining accurate calibration. Therefore, these instruments were not used for the wire faced walls.

Technical Question	Discussion	Monitoring Approach
Are there adverse seasonal effects on the reinforced fill near the wall face and at the top of the wall?	Reinforced fill containing fines may be susceptible to damage from freezing conditions. Soils with fines swell due to frost lenses and other freeze/thaw features. This may increase the forces in the reinforcing elements and can cause the wall facing to move outwards.	Install thermistors at selected locations and offsets back from the wall face, to monitor temperatures in the reinforced fill. Install weather station to monitor and record ambient conditions, including temperature, rainfall, relative humidity, and wind speed/direction. Monitor sensors often, using automated system to capture short-term and long-term changes/trends.
What role does drainage play on the wall stability and response?	Studies have shown that most wall problems (serviceability issues and collapse) are caused by poor drainage behind the reinforced zone. This problem is particularly persistent when using materials with significant fines.	Measure flow from drainage system at outlet, using flow meter or by manual means to correlate water in-flow and out-flow. Use pore pressures from piezometers to demonstrate connection of poor drainage to poor performance.

All instruments except the manual inclinometers were electronic and connected to automatic data logging equipment using the iSiteCentral™ system. This system was programmed for each instrument to have a warning level at which an electronic notice is sent to key personnel indicating that some activity is occurring at that instrument. Instruments were read multiple times each day and stored in the on-site data loggers. The data loggers are connected by cell phone-modem to our web server, which regularly contacted the site and updated its database with the latest readings on all instruments. The database was accessible with a WEB browser and provided any of our team with up-to-date process readings plotted in engineering units at any time from any location with WEB access. With iSite™ and iSiteCentral™, we spent very little effort collecting and processing data from the instrumentation. Instead, our computers performed this task and performed it frequently. This allowed us to carry out the field tests with far more extensive monitoring than typically possible. The benefit of this more extensive monitoring was to identify the effects of environmental changes, such as temperature and rainfall on the performance of the wall to a degree of detail not previously possible.

CHAPTER 2

DESCRIPTION OF THE FULL-SCALE TEST WALLS

2.1 Wall Facing/Reinforcement System

MSE retaining wall systems can consist of a number of different combinations of facing types and reinforcing materials, as noted in Table 2-1.

Table 2-1 Available MSE Wall Facing Types and Reinforcing Materials

FACING	REINFORCEMENT		
Modular Block	Metallic	Geosynthetic	
Segmental Precast Concrete Panels (SPCP)	Strips	Geogrid (Extruded, woven and welded)	Geotextile (woven and nonwoven)
Full Height Concrete Panels (FHCP)	Bar Mats	Polyethylene (HDPE)	Polyester
Welded Wire	Welded Wire Mesh	Polyester (PET)	Polypropylene
Gabion Baskets		Polypropylene (PP)	
Other			

Many public sector MSE walls have been constructed using metallic reinforcement and SPCP and FHCP facings. Welded wire facing is becoming attractive for public sector projects where large foundation settlements are anticipated. Many private sector MSE walls have been constructed using geosynthetic reinforcement and modular block facing. However, as indicated from the literature study described in Volume II of this research, it is clear that when using “high fines” soil in the reinforced zone that the performance of MSE walls is most affected by a) the properties of these “high fines” soils and b) the pore pressure in the reinforced zone, regardless of the wall facing/reinforcement type.

A significant focus of the field test for this demonstration project was on the role of pore water pressure in “high fines” reinforced soil and the importance of including a positive drainage system to obtaining good wall performance. Thus, in our opinion, facing and reinforcement type, although important, were a secondary consideration for this study. With the concurrence of the Panel, flexible welded wire mesh facing was selected for the four test walls. Polyester geogrid was selected for the primary reinforcement in Walls A, B and C. A nonwoven geotextile was used as the primary reinforcement in Wall D at the request of the Panel. The reinforcement was not connected to the welded wire face forms, as is typical in private practice.

There are issues with reinforcement selection and their use in “high fines” soils that are important to the design and serviceability of MSE structures. These include different interaction properties of metallic and geosynthetic materials, and electro-chemical effects (i.e. corrosion) associated with metallic reinforcement in “high fines” reinforced soil. However, these and other reinforcement issues have been and are being studied by other researchers and are not in the work scope of this project.

2.2 Design of the Walls

The walls were designed with a very low factor of safety at the extreme loading condition of surcharge with full water pressure in the reinforced fill. The long term internal factor of safety selected was less than 1.1, without surcharge loading, in order to maximize strains during the field test. The intent was to promote development of a failure plane and maximize load in each reinforcement layer under the extreme load condition (saturation of the reinforced fill combined with soil surcharge load). Designing at low factors of safety requires accurate determination of all input parameters.

2.2.1 Reinforced Fill Characteristics

Three (3) different soil types with varying degrees of fines were selected for the reinforced fill zones of the test walls in order to meet the work plan goals in Chapter 1. Each soil type was sampled and tested by GeoTesting Express for classification, strength, compressibility and permeability. Attachment A includes the test results. The gradation characteristics of the Fill soils are shown in Figure 2-1, along with representations of the target soils: A-1-a (blue), A-1-b (red) and A-2-4 (orange). The criterion used for comparing the reinforced fills with the target soils is the percent passing the #200 (0.075mm) sieve.

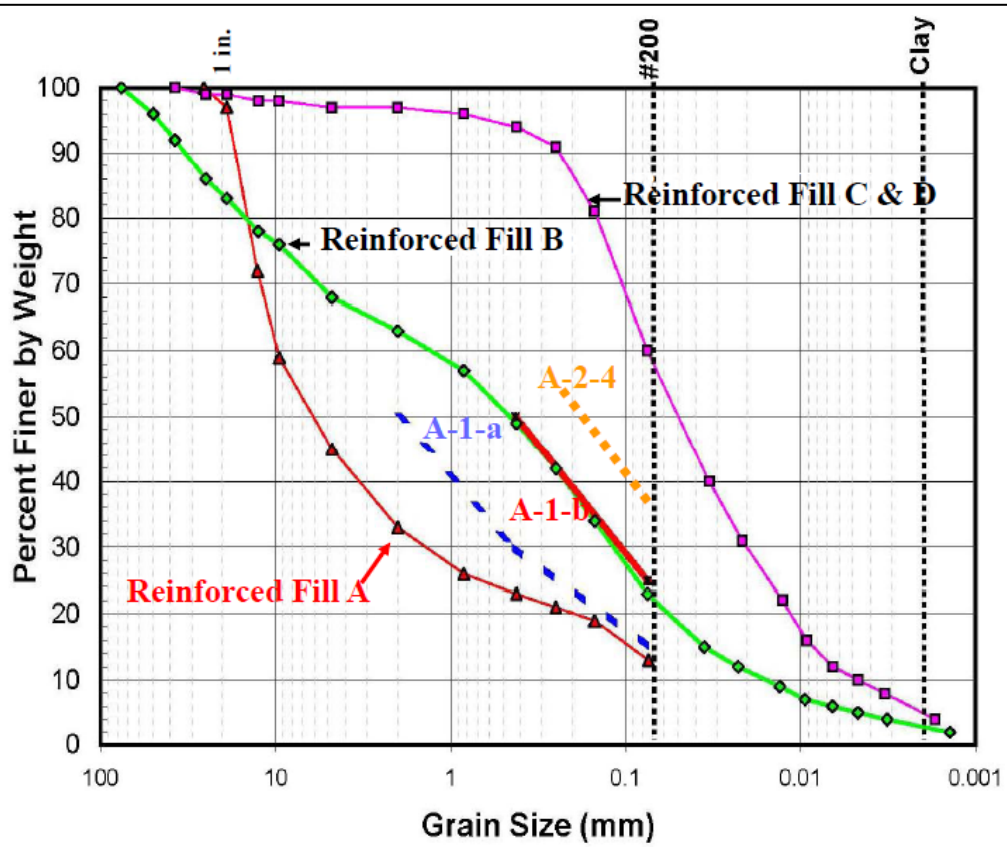


Figure 2-1: Grain Size Distribution of Reinforced Fill

Reinforced Fill A is the A-1-a wall control section and was matched to a blended soil produced by mixing a processed, relatively clean granular soil readily available (identified by the site operator as “Keating Graded Base") with on-site silty screenings at a ratio of 4:1. The result was a soil with a maximum fines content of about 13 percent (See Figure 2-1) which closely

represented the “highest fines” material possible while meeting the AASHTO A-1-a requirements.

Reinforced Fill B was matched to an on-site glacial till. With maximum fines passing the #200 sieve of 23 percent, the till classified as A-1-b (i.e. 25% maximum allowable passing #200 sieve). While our research team had considered using a reinforced fill soil for Wall B that contained fines up to 35 percent, our goal for this test section was to select a "higher fines" soil representative of a material that realistically DOTs would be comfortable moving to. Our survey indicated that all DOTs consider material with a fines content of 25% as a "high fines" material. Therefore, the till was considered a good choice for Wall B (and AASHTO A-1-b). From a constructability standpoint, the in-situ moisture content of the till was very close to the D 698 Optimum Moisture Content, which would allow the till to be placed and compacted directly from the pit without the need for moisture conditioning.

The two remaining soils – Reinforced Fills C and D - were chosen to meet the AASHTO A-4 soil with more than 35% passing the #200 sieve. As a refinement, the Panel desired that the A-4 soil contain less than 50% fines and have a PI < 6. During the early development of the project, on-site screenings (identified by the site operator as “Keating #1 Washings") were made available. Sieve analysis data provided by Keating in 2004 indicated that the fines content for this material was around 40% and the material was non-plastic. This screening material was therefore selected for use as the reinforced fill soil in Walls C and D. Laboratory tests performed by the research team in 2005 immediately prior to construction indicated that the fines content of the screenings was as high as 60%. Since there was no other viable option to source the soil for these walls, the project proceeded using these on-site screenings to construct Walls C and D.

Pertinent engineering properties of the reinforced fill soils were determined by laboratory testing and are indicated in Table 2-2.

Table 2-2: Pertinent Engineering Properties of the Reinforced Fill & Drainage Soil

Material	Max. Dry Density $\gamma_{d\,max}$ (pcf)	Optimum Moisture Content (%)	Friction Angle ϕ_{PEAK} (deg.)	Permeability[5] k (cm/sec)
Reinf. Fill A (A-1-a; Keating 4:1 Blend)	134.5 [1] 138 [2]	8.0 [1] 6.0 [2]	50 [3], 37 [4]	2.7×10^{-2}
Reinf. Fill B (A-1-b/A-2-4; Glacial Til)	134.5 [1] 139.5 [2]	6.0 [1] 4.5 [2]	33 [3], 35 [4]	9.3×10^{-5}
Reinf. Fill C & D (A-4; Keating #1 Washings)	112 [1] 119 [2]	13.5 [1] 12.0 [2]	39 [3], 33 [4]	6.1×10^{-5}
Drainage Soil (Local Sand Source)	121.5 [2]	10.5 [2]		7.0×10^{-3}

[1] ASTM D 698 [2] ASTM D 1557 [3] ASTM D 3080 [4] ASTM D 4767 [5] ASTM D 2434 or D 5084

The foundation conditions beneath the test walls were evaluated by test borings, and consisted of very dense natural inorganic soils overlying bedrock, which occurs at relatively shallow depths. Groundwater was not encountered in the test borings.

The compaction measurements taken as part of the QA/QC of the fill placement are summarized in Attachment B, Figures B-6 through B-9. A discussion of the fill placement and a description of the compaction measurements and results are presented in Section 3.1 of this report volume.

2.2.2 Reinforcement Design

The philosophy adopted for the reinforcement design of the test walls was as follows:

1. Walls should "fail" under the combined test conditions of surcharge load and water seepage/surface water infiltration. Ideally, overstress the reinforcement.

2. Use uniform and much weaker reinforcement than usual design practice.

3. Determine required geogrid strength assuming lateral earth pressure equals ½ $K_{A\ RANKINE}$.

AASHTO uses the Rankine earth pressure theory and NCMA uses the Coulomb earth pressure theory. The magnitude of lateral earth pressure depends on the horizontal strain that develops within the reinforced soil. In flexible MSE walls (e.g., geosynthetic reinforcement), the movement occurring during construction is sufficient to develop an active state of stress. Experience gained in recent years shows that for granular soil and geosynthetic reinforcement, the lateral earth pressure approach could be overly conservative (i.e., the maximum reinforcement force can be over-predicted by as much as a factor of two). In fact, the lateral earth pressure approach indicates that the required strength increases linearly with depth (for uniform reinforcement spacing), whereas measured field data shows nearly uniform mobilization of reinforcement strength with depth. The required total reinforcement strength is in good agreement; however, the distribution in reality is close to uniform whereas the predicted distribution is linear with depth.

In addition to lateral earth pressure theory, reinforcement loads were also computed using a new working stress methodology, termed the K-Stiffness Method, proposed by Allen and Bathurst (2003). The past performance of MSE walls has provided strong evidence that current design methodologies for internal stability, in particular the prediction of reinforcement loads, are very conservative. Allen et al (2003) indicate that the proposed "K-Stiffness" method has been calibrated against measurements of strain and load in monitored, full-scale walls reported in the literature. The design methodology incorporates the following key factors, which influence the magnitude of maximum reinforcement load, T_{max}:

- Height of the wall and any surcharge loads,
- Global and local stiffness of the soil reinforcement,
- Resistance to lateral movement caused by the stiffness of the facing and restraint at the wall toe,
- Face batter,
- Shear strength and modulus of the soil,
- Unit weight of the soil,
- Vertical spacing of the reinforcement.

It should be noted that the K-Stiffness Method has not been specifically developed to handle the effects of water saturation within the wall backfill, since a saturated wall backfill is not a normal design situation for public infrastructure projects. Due to the semi-empirical nature of the K-Stiffness Method, incorporating water into the design to estimate reinforcement loads and strains is not straight-forward. Thus, two design cases were evaluated. One in which it was attempted to incorporate the water effects directly into the K-Stiffness model, and a worst case scenario in which superposition was used to add the water loads directly to Tmax, as would be done for the AASHTO Simplified Method.

The reinforcement design process consisted of the following steps for each test section:

1. Calculate maximum reinforcement load based on lateral earth pressure theory (assume $K = K_a$ and $K = 0.5\ K_a$) and the K-Stiffness method. The analyses were performed for the end of construction condition ("dry") and test stages 4 (dry, 5 ft. surcharge) and 6 (wet, 5 ft. surcharge).

2. Calculate the factor of safety against reinforcement tensile failure for candidate geogrid strengths for all test stages from Step 1.

   Assume:
   - Uniform vertical spacing of reinforcement (s_v = 18 in.).
   - Polyester geogrid, using lowest possible reduction factors for geogrid strength (e.g. $RF_{Creep} = 1.45$, $RF_{Durability} = 1.0$, $RF_{InstallationDamage} = 1.05$, $RF_{Total} = 1.52$).
   - For the vertical spacing of reinforcement selected, use two different strength geogrids for the upper and lower half of the walls.
   - Nonwoven geotextile reinforcement, using lowest possible reduction factors for reinforcement strength (e.g. $RF_{Creep} = 2.85$, $RF_{Durability} = 1.0$, $RF_{InstallationDamage} = 1.05$, $RF_{Total} = 3.0$).

3. Check reinforcement strength using limit equilibrium theory (e.g., slope stability approach extended to reinforced walls), to quantify the margin of safety of the selected reinforcement system design against a state of imminent failure. The limit equilibrium method with reinforcement is a natural extension of a method widely used in geotechnical engineering. Compared with Earth Pressure Theory, it does not require extensive field "calibration" (i.e. only limited confirmation for the specific type and configuration of reinforcement), and, thus, it can be used to design complex problems. In limit equilibrium analysis, the common assumption is that all reinforcement layers are equally mobilized. This analysis looks at the overall stability of a potential sliding mass, targeting an acceptable factor of safety. Comparison with sophisticated numerical methods indicates that this assumption results in safe structures, though less conservative that those based on Earth Pressure Theory. The computer code ReSSA was used to perform the analyses.

   Assume uniform length of reinforcement (L = 0.7H, in accordance with AASHTO).

Figure B-1 (See Attachment B) presents a comparison, for Wall A, of the calculated maximum reinforcement load versus wall height, based on lateral earth pressure theory and the

K-Stiffness method. The analyses were performed for the end of construction condition ("dry") and test stages 4 (dry, 5 ft. surcharge) and 6 (wet, 5 ft. surcharge).

Figure 2-2 presents the calculated factor of safety against tensile failure of the reinforcement versus wall height for the test stages analyzed. Based on the results of the T_{MAX} analyses performed in Step 1 (Figure B-1), ultimate reinforcement strengths of 800 and 1,500 lbs. per ft. were selected for the reinforcing layers in the upper half and lower half of the wall, respectively. For $T_{MAX\ K=0.5Ka}$ the calculated factor of safety in the lower half of the wall is at or less than 1.0 for test stage 6.

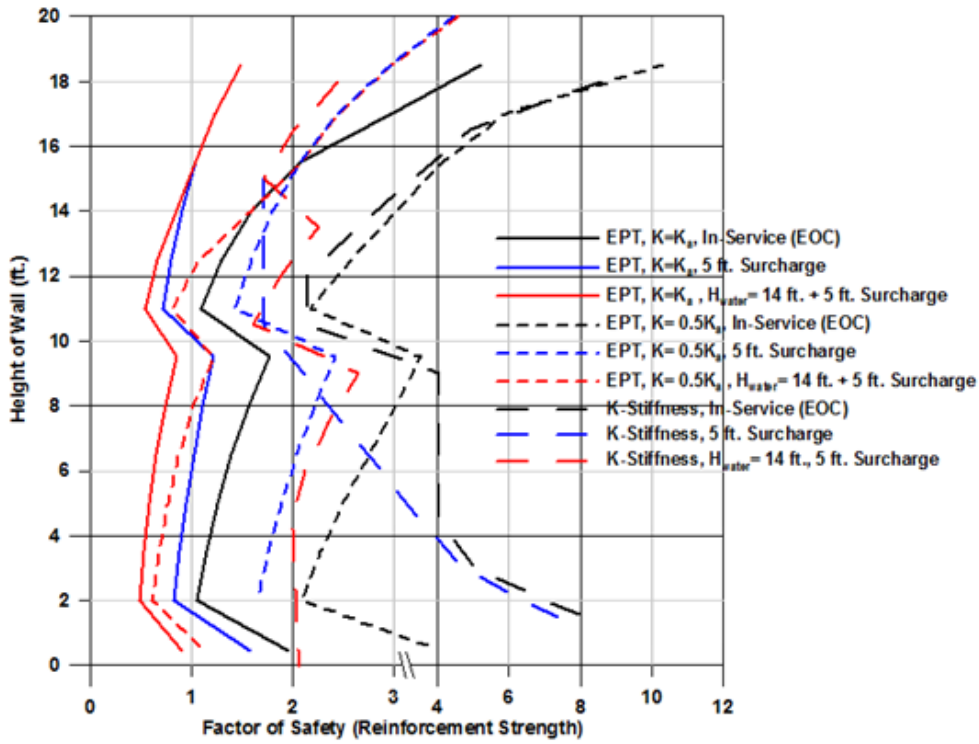


Figure 2-2: Factor of Safety (Reinforcement Strength) vs. Wall Height, Wall A

Step 3 of the reinforcement design consisted of performing limit equilibrium analyses. A Factor of Safety = 1.1 was selected as the benchmark where significant signs of distress should appear (e.g., increase in rate of wall deformations). While FS = 1.0 generally implies collapse, MSE walls with extensible reinforcement are just deforming; thus, making the definition of collapse difficult to define. The results of limit equilibrium analyses are presented in Figures B-2 (Walls A, B, C) and B-3 (Wall D). The analyses were performed for the "end-of-construction" condition and test stages 4 (dry, 5 ft. surcharge) and 6 (wet, 5 ft. surcharge). The friction angle of the reinforced fill was varied to evaluate the effect on the internal stability factor of safety. Circular and non-circular failure surfaces were analyzed.

The results indicate that, for the proposed reinforcement design and most probable values of the friction angle of the selected reinforced fill soils (Wall A, 37 deg.; Wall B, 35 deg.; Walls C and D, 33 deg.) , that the internal factor of safety will be less than 1.0 for Stage 6 loading (wet, 5 ft. surcharge).

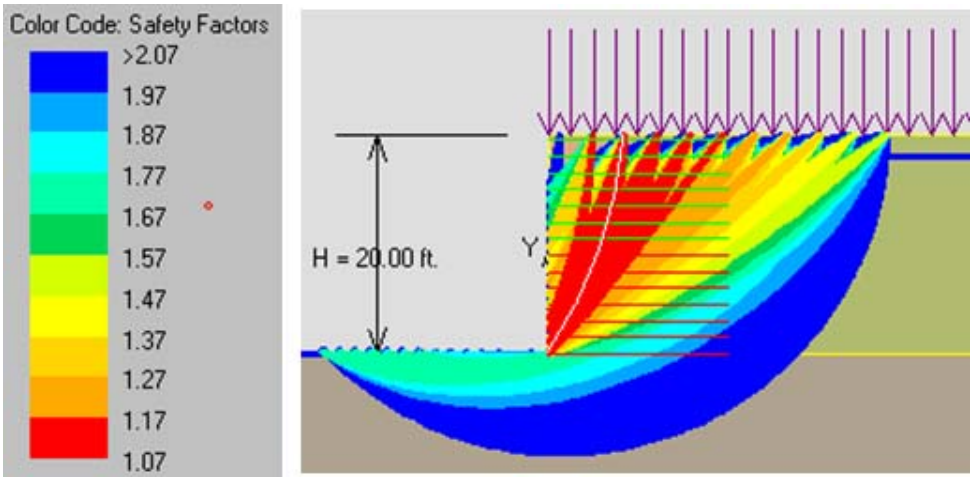


Figure 2-3: Example "Safety Map" from Limit Equilibrium Analysis

Figure 2-3 presents the "safety map" for the most probable friction angle value of Wall A under Stage 6 loading. The calculated factor of safety from the Bishop analysis is shown in increments of 0.1. This is a helpful tool because it identifies suspect zones where most deformation is likely to occur.

For Wall D, the vertical reinforcement spacing of the nonwoven geotextile was reduced to 9 in from 18 in. This was done because the total reduction factor applied to the ultimate reinforcement strength of the nonwoven geotextile (RF_{Total}) was taken as 3, approximately twice that of the RF_{Total} = 1.52 for the PET geogrid used in Walls A, B and C.

Although not part of the study scope, finite element analyses were performed using Plaxis (ver. 8.6) to predict wall deformations and strains in the reinforcement for each test wall. Plaxis is a two-dimensional finite element program for stability and deformation analysis in geotechnical engineering. The program is intended for the use in a wide variety of geotechnical analyses. The soil is modeled using constitutive stress-strain behavior and Mohr-Coulomb failure criteria.

The finite element modeling performed was truly a Class A prediction of the response of the walls. Modeling did not include the environmental effects related to large rainfall events, seasonal changes, freeze/thaw cycles or creep. These factors had significant impacts on the actual wall performance, and make comparisons between the predicted and observed responses for the entire project impractical.

The FEA results are discussed later in this report.

2.2.3 Reinforcement Materials

A description of the specific reinforcements selected for the project and strength properties used in the analyses are listed in Table 2-.3. The reinforcement structural property used in the wall designs was the ultimate strength.

Reinforcement	Wall	Product	Material	Strength Characteristics T_{ULT} (lbs/ft)
Polyester (PET) Geogrid	A,B&C Upper half of wall (Layers 8-14)	FORTRAC 20/13-20 (PET)	High tenacity polyester yarn encapsulated in PVC	800 (XMD)[1]
	A,B&C Lower half of wall (Layers 1-7)	Miragrid 3XT (modified)[2]	High tenacity polyester yarn encapsulated in PVC	1,500
Nonwoven Geotextile	D Upper half of wall (Layers 15-28)	Propex 4551	6 oz./yd^2 Polypropylene nonwoven needle punched fabric	800
	D Lower half of wall (Layers 1-14)	Propex 4553[3]	8 oz./yd^2 Polypropylene nonwoven needle punched fabric	1,500

Table 2-3: Description of Reinforcements

[1] The cross machine direction (XMD) ultimate strength of the FORTRAC geogrid matched the desired ultimate reinforcement strength in the upper portion of the walls. Additionally, the roll width (14 ft.) matched the required reinforcement length, allowing the geogrid to be placed with the XMD perpendicular to the wall face without splicing.
[2] TenCate Geosynthetics manufactured a "modified" 3XT geogrid specifically to meet the strength requirements of the NCHRP wall design (every other rib in the machine direction was removed).
[3] Transmissivity = 0.001 gpm/ft. (2 x 10^{-7} m^2/sec)

Wide width strength tests (ASTM D6637-01) were performed on samples of the polyester geogrids to determine stress-strain characteristics for use in interpreting reinforcement loads from strain gage readings. The results are presented below in Figures 2-4 and 2-5.

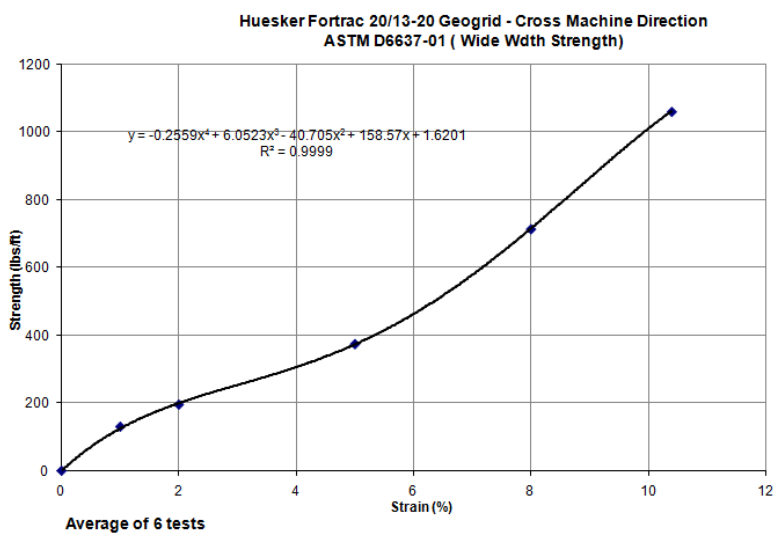


Figure 2-4: Wide Width Strength Test (Fortrac 20/13-20 Geogrid)

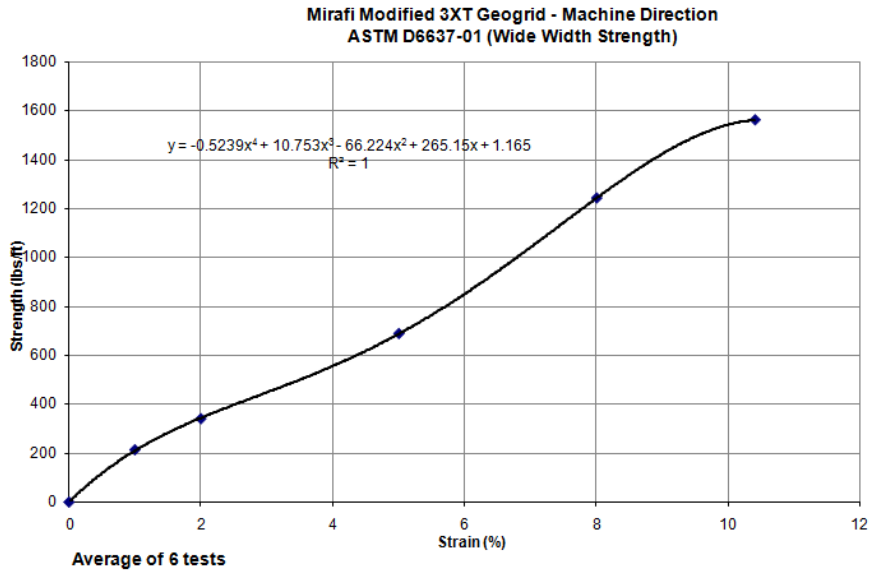


Figure 2-5: Wide Width Strength Test (Mirafi Modified 3XT Geogrid)

Both wide width strength (ASTM D4595) and confined stress strain (Christopher, Holtz and Bell, 1986) tests were performed on the Wall D nonwoven geotextile. The results are presented below in Figure 2-6. A design strength value of 400 lbs./ft. at 10% strain was selected for the 8 oz./yd^2 NWGT, based on the confined stress strain tests.

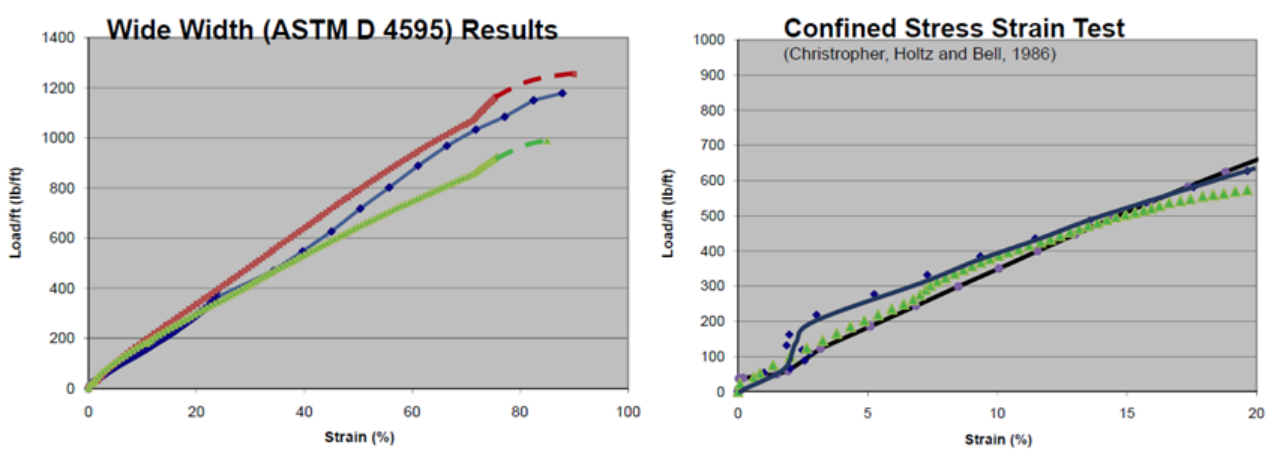


Figure 2-6: Wide Width Strength & Confined Stress-Strain Test Results (NWGT)

CHAPTER 3

CONSTRUCTION OF THE FULL-SCALE TEST WALLS

3.1 GENERAL

Construction of the four full-scale test walls commenced in early August 2005, and was completed in approximately two months. The construction sequence consisted of the following:

1. Prepare sub-grade and place base PVC geomembrane (GM).
2. Seam vertical GM (See Figure B-4).
3. Place and compact reinforced fill (See Figure B-5) and install and tension primary soil reinforcement.
4. Coordinate installation of the hydraulic elements of the wall design with construction of the drainage fill zone and reinforced fill sections.
5. Coordinate installation of instrumentation with construction of the reinforced fill sections.

Inclusion of the vertical PVC GM barrier to hydraulically separate individual wall test sections became problematic from a construction perspective. The NCHRP Review Panel recommended that this be added to hydraulically separate each test wall. Its placement greatly slowed the progress of construction and impeded the compaction of the soils immediately adjacent to it. The field staff were so concerned with puncturing the GM that compaction of the soil immediately adjacent to the GM suffered. In retrospect, it resulted in more problems than the benefits it might have provided.

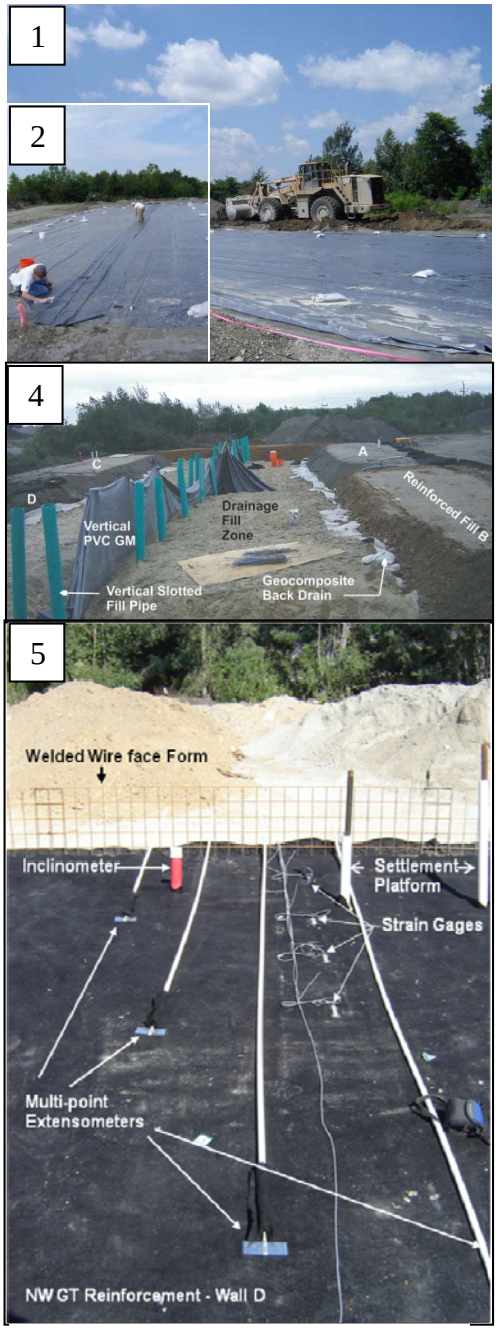


Figure 3-1: Construction Photos

During construction, field density tests were performed at four locations per compacted lift of reinforced fill. The reinforced fill materials were placed in maximum 9-in. loose lifts prior to compaction. The target minimum density was 95 percent of the maximum density determined by ASTM D 698 (Standard Proctor).

Figures B-6 through B-9 present the results of the field density tests performed at each wall. In general, the target density was achieved for all walls with the exception of limited zones at the face and close to the geomembrane barriers where compactive effort was applied using hand-guided equipment. Much of the reinforced fill placement occurred in August at a very dry and hot time of the year in New England. As a result, the reinforced fill materials for Walls A, C and D were placed at moisture contents 2% to 4% below optimum moisture content. The Wall B reinforced fill was placed at moisture contents wet of optimum (maximum 2%).

Figure 3-2 shows the test walls nearing 80% completion in September 2005.

Figure 3-2: Construction Progress (September 2005)

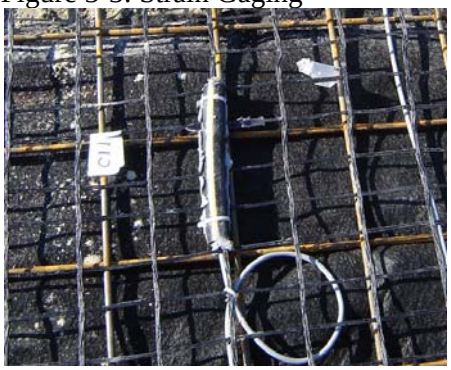
3.2 INSTRUMENTATION OF THE TEST WALLS

Chapter 1 of this appendix described the potential questions or concerns that were to be addressed in the full-scale field test. For each question, the technical reasoning for the validity of the question and a proposed monitoring solution was provided as part of the full-scale field test. Figure B-10 presents details of the instrumentation which were incorporated into each test wall to achieve the test goals. Figure B-11 depicts a typical as-installed instrumentation layout at each of 4 instrumentation levels within a wall. Figure B-12 shows the instrumentation readout cluster for the test walls. Note the data logger which provided 24/7 real-time instrumentation readings to the iSiteCentral server.

3.2.1 Measurement of Reinforcement and Global Strains

Strains on the geogrid/geotextile reinforcement were monitored using strain gages and horizontal rod extensometers. The strain gages are used to measure a direct local strain of the reinforcement at the gage location; the rod extensometers provide an average strain over the spacing of the rod anchors. Both systems were installed at several different levels in each wall section. A typical instrument layout (plan and section) is shown in Figure B-11.

Figure 3-3: Strain Gaging

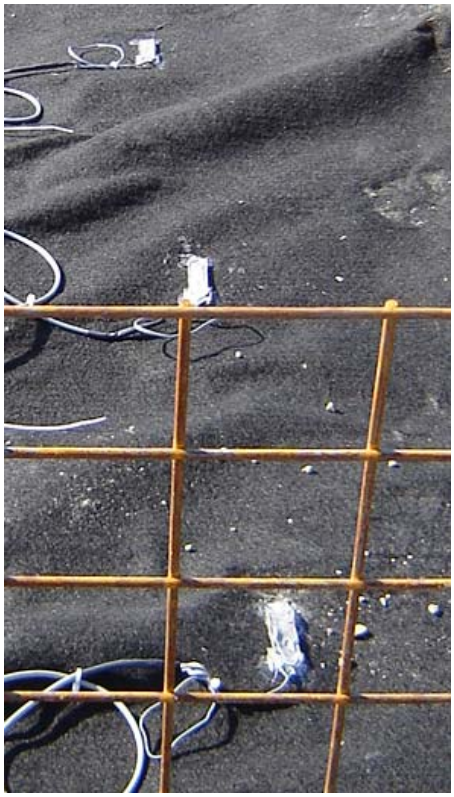


Strain gage on geogrid showing split-tube protection filled with RTV silicone

Strain gages: For each test wall a total of 21 gages were installed – typically 5 to 6 gage locations installed on 4 separate reinforcement layers, spaced vertically through the wall height (See Figure B-10).

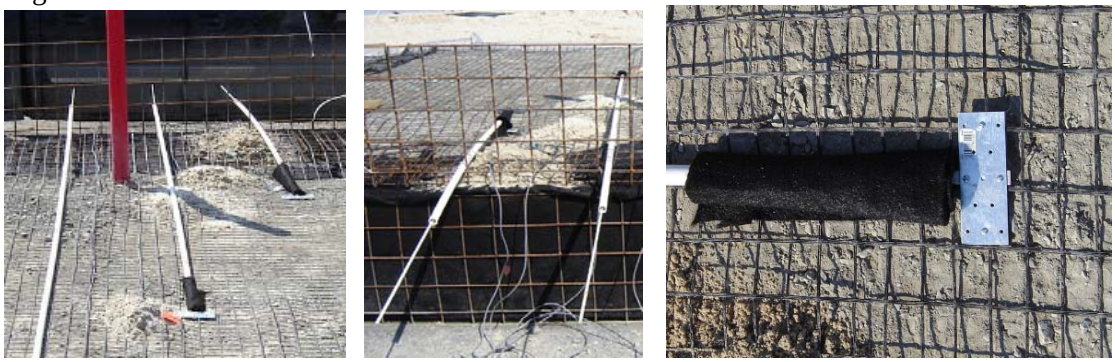
For the geogrids (Walls A, B and C), the gages used were Vishay foil resistance gages, approximately ½ in. long. At each location 2 gages were used – one on the top and one of the bottom of the reinforcement. For the geogrid, strands were scraped and cleaned and then impregnated with a flexible 2-part epoxy suitable for high elongation applications (>10%). Pre-wired strain gages were pressed into the epoxy on each side of the geogrid, and sandwiched under light even pressure until the adhesive set. The gages were then waterproofed with RTV Silicone and then further protected by using a section of split flexible tubing that provided a shield over each of the top and bottom gages.

For the non-woven geotextile (Wall D), Vishay foil strain gages approximately 2 in. long were first affixed to narrow flexible rubber sheets that were attached at either end with a small metal tab. These tabs were pinched and epoxied onto the geotextile. The gages were waterproofed with RTV silicone, and were protected during installation and the following soil compaction using carefully placed sand pockets above and below each gage position. Each gage pair was wired into an iSite data logger for remote automated monitoring.



Gages on top-side of non-woven geotextile; welded wire face form in foreground.

Figure 3-4: Rod Extensometer



(a) (b) (c)

(a) rods and anchors on geogrid, (b) rods extending through wire face basket, and (c) close-up of rod anchor plates and sleeved rod (wrapped with geotextile to prevent soil ingress)

Multi-point rod extensometers: rod extensometers were used on the same reinforcement layer as the strain gages, and were each comprised of a rod anchor attached to the reinforcement, and a sleeved ¼ in. diameter fiberglass rod that extended horizontally through the fill and out the wall face (see Figures 3-4 and B-11). The installation on each layer was typically 4 rods anchored at 4 ft. intervals. The anchors were made up of metal plates sandwiched with adhesive on to the reinforcement layer using low-profile screws/nuts. At the exposed end of the extensometer rod a reflective prism target was affixed with epoxy. These targets were monitored using the robotic total station (described below).

3.2.2 Measurement of Pore Water Pressure

Pore water pressure measurements in the test walls were obtained using Geokon model 4500S vibrating wire piezometers with integrated thermistors for recording ground temperatures. Prior to installation, the piezometers were first saturated and then attached to the reinforcement layers. The fill was placed around each location and the piezometers were finally covered by a small sand pocket to improve drainage locally to the sensor. Each piezometer cable was connected to an iSite data logger for remote automated monitoring.

3.2.3 Measurement of Wall Face Movement

Measurements of the wall face movements and the rod extensometers were obtained using a Leica Model TCA1201 Automated Motorized Total Station (AMTS). A local reference system of prismatic reference targets was established using prisms rigidly attached to stiff steel poles set into 3-4 ft. deep concrete pillars in the ground. Three to four of these reference targets were installed for each of the two AMTS locations. Each AMTS instrument was also mounted on a 3 ft. x 3ft. x 3ft. concrete pad/block and protected within an enclosure. The TCA1201 instrument is a 1 arc-second instrument capable of resolving 1ppm on distance measurements. Typical sights distances for the AMTS instruments were less than 100 ft.

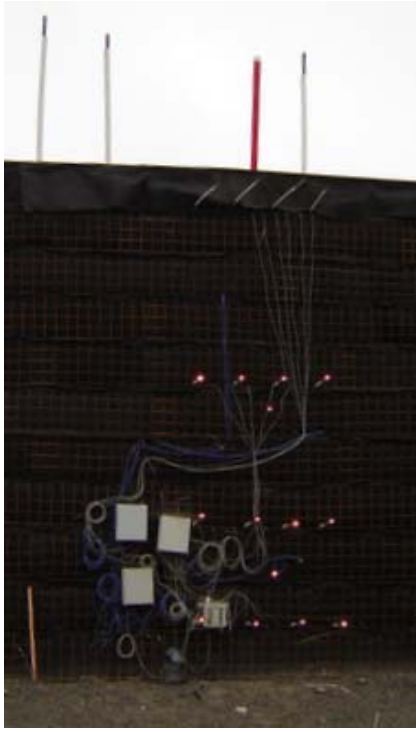


Figure 3-5: Wall face showing reflective prism survey targets (glowing). Note red inclinometer casing and settlement plate pipes rising above top of wall.

3.2.4 Measurement of Reinforced Fill Settlement

Settlement of the fill was measured using settlement plates that consisted of 2 ft. by 2 ft. marine grade plywood. Iron pipe flanges were bolted to the plywood square and 1inch iron pipe inside 1.5 inch PVC sleeve was extended up through the reinforced fill materials. Reflective prism targets were attached near the top of each pipe and the prisms were surveyed using the AMTS instrument described above.

3.2.5 Measurement of Lateral Movement of Reinforced Fill

Lateral movements of the reinforced fill near the test wall face were recorded using a Slope Indicator biaxial inclinometer probe, cable and Datamate readout. Slope Indicator inclinometer casing was 2.75 in diameter. The inclinometer casing was pre-drilled to between 10 and 15 ft. below the existing ground level prior to starting wall construction and was extended upward as the walls were constructed.

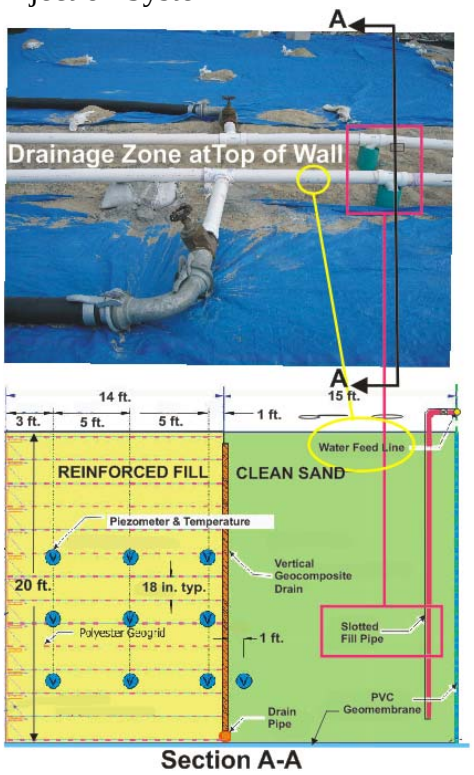
3.2.6 Measurement of Temperature within Reinforced Fill

Temperature within the reinforced fill was recorded at each of the vibrating wire sensor locations. Three additional Geokon thermistors were directly buried in the fill at 3 ft. depth below the top of the wall. All temperature gages were connected to iSite data loggers for remote automated monitoring.

3.3 HYDRAULIC SYSTEM

The plan view and details of the system installed to introduce water into the reinforced fill, via the drainage fill zone, during the hydrotest phases of the full-scale field test are shown on Figures B-13 and B-14. Figure 3-6 presents additional details of the water injection system.

Figure 3-6: Details of Water Injection System



Each test wall was provided with a separate header system, so that water could be introduced independently into the reinforced fill zone of each wall. The header system was connected to six (6) 6-in. diameter perforated PVC pipes that were installed vertically on 10-ft. centers during placement of the drainage fill.

Water was pumped from a storage tank (20,000 gallon capacity) into the header system via a primary pump (max. 400 gpm pumping rate). A secondary pump was connected to the wall drain system and intermittently returned captured water to the storage tank. In order to allow continuous pumping once the tests began, water in the storage tank had to be replenished at about 7 hr. intervals. Water built up over the top of the fill during the water test so there was no need to install the sprinkler system originally envisioned to simulate prolonged rainfall.

CHAPTER 4

PERFORMANCE OF THE FULL-SCALE TEST WALLS

4.1 INTRODUCTION

The instrumented test walls were monitored 24/7 over a period of approximately 23 months starting in September 2005. The test duration is divided into 6 periods/events as noted in Table 4.1 below.

Stage	Description	Duration/Period
1	Fall 2005 Rainfall Events	Start of monitoring – November 15th 2005
2	Winter 2005 – Spring 2006	November 16th 2005 – June 1st 2006
3	Hydrotest #1	August 3rd – August 19th 2006
4	Fall 2006 – Spring 2007	August 20th 2006 –April 1st 2007
5	Surcharge fill loading	April 23rd – May 1st 2007
6	Hydrotest #2	August 27th – September 1st 2007

The total monitoring period allowed for the observation of seasonal effects on wall performance over a two-year period.

This chapter of the report presents the instrumentation results for the Fall 2005 Rainfall Events and for the engineering-induced test events: Hydrotest #1, surcharge fill loading, and Hydrotest #2. There is also a section discussing the long-term thermal measurements in the reinforced zone over the two year period.

4.2 Fall 2005 Rainfall Events

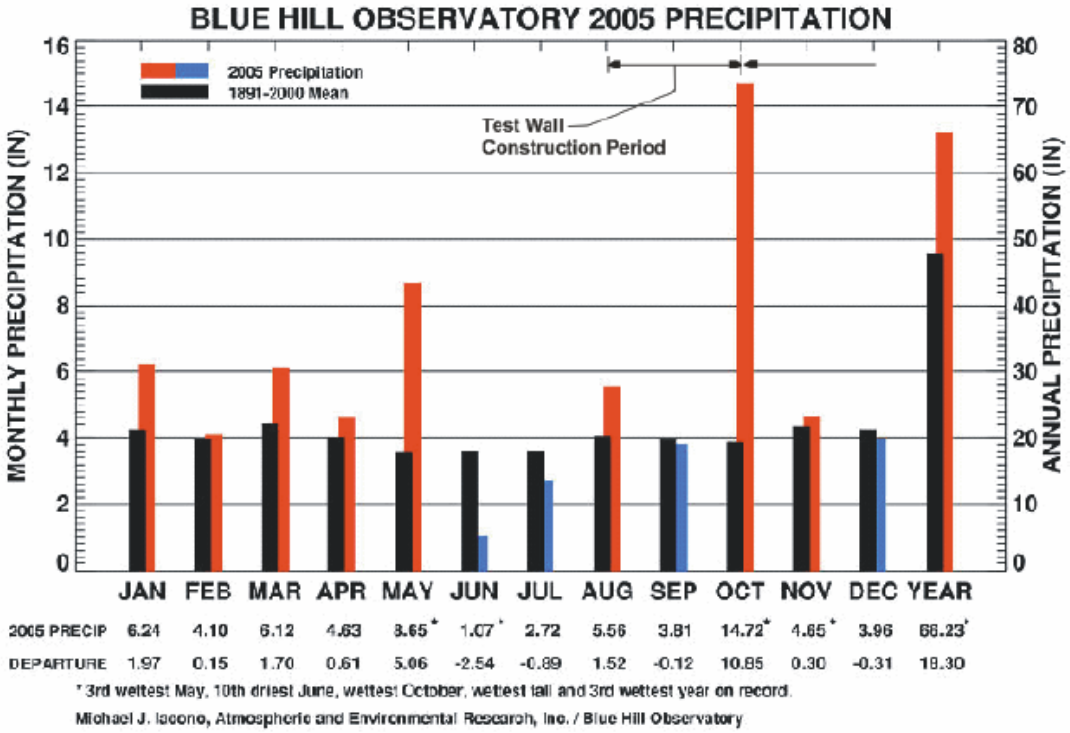


Figure 4-1: 2005 Precipitation at the Test Site

While the principle focus of the field test was the behavior of the reinforced fill during the three scheduled test events (i.e. two hydrotest phases and surcharge fill phase), a significant environmental event also occurred. Specifically, the site experienced record rainfall events in fall 2005 immediately after completion of construction of the test walls. As indicated in Figure 4-1, the rainfall that occurred at the site in October 2005 was approximately 15 in. (almost four times greater than the mean monthly precipitation during the period 1891 – 2000). These rainfall events actually simulated test conditions planned for later in the field test, as part of Stages 3 and 6. The observed wall performance during this period played an important role in the development of some of the recommendations presented later in the report.

4.2.1 Observed Wall Performance

Figures B-15 through B-17 present some of the recorded visual observations made of the effects of surface water infiltration, as a result of the Fall 2005 record rainfall events. Figure B-15 shows tension cracks that developed parallel to and within several feet of the wall face for Walls A and B. In addition there was a limited trough of subsidence observed at the ground surface.

In contrast, larger deformations were observed at Walls C and D. Figure B-16 shows the location and pattern of observed surface tension cracking. Note the observed progression of tension crack development between early October and November 2005 (i.e. the appearance of tension cracks further back from the wall face towards the end of the reinforced fill section). The largest deformations were centered about the vertical PVC geomembrane separating Walls C and D. As previously noted, the vertical geomembrane posed an impediment to adequately compacting the fill materials located in the zone immediately adjacent to either side. As a result, it is likely that larger deformations observed are a result of a lower fill density achieved with less compaction. Figure B-17 clearly indicates vertical wall face deformations increasing in the vicinity of the vertical PVC geomembrane.

4.2.2 Instrumentation Measurements

Figure B-18 presents a comparison of the results of inclinometer readings for all walls. While all walls experienced horizontal movements in the frontal zone due to the Fall 2005 rainfall events, Walls C and D experienced significantly more movements than Walls A and B. At Walls A and B the maximum horizontal movement recorded was approximately 2.0 in., while those recorded at Walls D and C were between 4.0 and 6.0 inches respectively.

Settlement plate readings versus time are presented on Figure B-19. The data are arranged comparing settlement plate behavior at similar levels within each of the test walls. Little to no vertical movement was observed in Walls A and B at the different levels, while vertical movement of about 0.5 in. was recorded at Walls C and D.

Strain gage readings versus time are presented on Figures B-20 through B-23. The data are arranged to compare the strain gage behavior at similar levels within each of the test walls. In general, the following observations are made:

- Only very small strains were observed on the bottom level of gaging (level 1) for all walls
- Strain increases in the reinforcement for all walls were primarily observed in the frontal zone of the reinforced soil (i.e. within 4 ft. of the face) and in the upper level (Level 4). Wall D also showed some reinforcement strain on Level 3.

The results broadly support visual observations discussed previously of the tension cracks at the tops of the walls, and the settlement of the wire baskets near the wall face.

It should be noted that the strain gage monitoring for Levels 1- 3 of Wall C was compromised during the rainfall event by a faulty data logger component. The failure was not obvious and the results were questionable. To avoid misinterpretation, the data has been omitted from consideration here.

Rod extensometer readings versus time are presented on Figures B-24 through B-27 and show measured lateral displacement at different locations back from the wall face. These data are also arranged to compare extensometer movements at similar levels within each of the test walls. In general, the following observations are made:

- Lateral movements during the Fall 2005 rainfall events increase with proximity to the wall face
- Movements were less 0.5in during the rainfall events period, with consistently larger movements detected at Walls C and D

The monitoring for Walls C and D extensometers was started later than other locations and appears to have missed some of the movements attributed to the rain events.

The data from the rod extensometers is also presented on Figures B-28 through B-31 as global strain, calculated between adjacent rod anchors on each layer. When reviewed in context with the strain gages, similar strain behavior patterns as for the strain gage observations are seen. These calculations of strain provide redundancy for the strain gage measuring system.

Optical survey readings of the wall faces are shown on Figures B-32 through B-40: Figures B-32 to B-36 present horizontal/lateral movements versus time and Figures B-37 to B-40 show vertical movements/settlement. The data are again arranged to compare the observed movements at similar positions on the face of each of the test walls.

In summary, the following observations can be made concerning the impact of the Fall 2005 rainfall events:

- Lateral face movement of Walls A and B was generally less than about 1.0 in
- Larger movements occur in the upper 1/3 of the wall height.
- Maximum lateral face movement of Walls C and D approached 3.0 in. and are generally uniform over approximately the upper 1/2 of the wall height.
- Vertical face movements of approximately 1.0 in. were recorded for Walls A and B in the upper 1/3 of the wall. Little to no vertical movement was observed in the lower 1/2 of the walls.
- Large vertical face movements ranging from about 3.0 to 4.0 inches were recorded for Walls D and C, respectively, in the upper 1/3 of the wall. Up to 4.0 in. of vertical movement was observed in the lower 1/2 of the walls.

Most instruments showed small creep movements of the reinforced fill during Winter 2005 - Spring 2006. Deformation of the wall faces, as shown on Figures B-32 to B-36, continued after the Fall 2005 rainfall events through the Winter 2005/Spring 2006. As shown in Figure 4-2, Walls A and B experienced additional lateral movement less than 0.75 in. Walls C and D showed larger additional lateral deflection, up to 1.5in for Wall D. Additional vertical deformations are on the order of 1 to 1.5 in for Walls A and B, but between 4 and 5 in. for Walls C and D in places.

Walls C and D experienced continuing wall face movements (outward and downward) during this time period (See Figures B-34, B-40, etc.), including heavy rain events that occurred in May 2006, and resulted in notable strains and movements in the upper frontal zones of the walls.

During the Winter 2005 - Spring 2006 time period some strain gages failed. It was noted that all gages for all walls at Level 4 (top layer) Position 2 (behind the face baskets) failed. We believe these failures could be the result of water entering the strain gage along the reinforcement, or due to strain on the cables locally pulling on the gage.

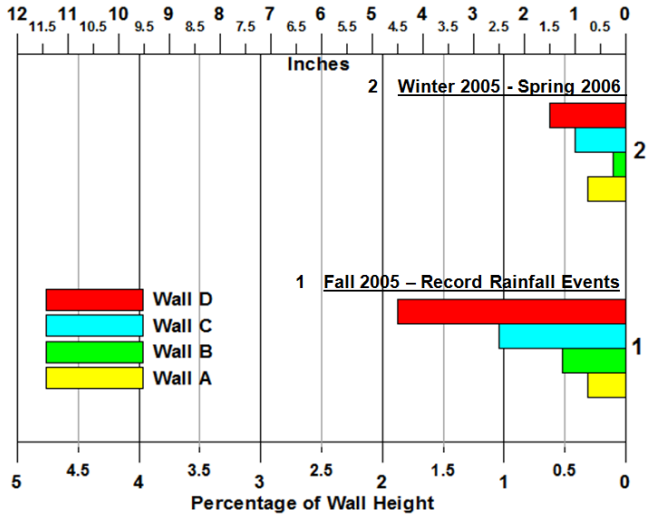


Figure 4-2: Maximum Horizontal Face Movement At Center-Line of Wall

The inclinometers recorded continued movements at all walls after the fall rains, as shown on Figure B-18. The rate of movement accelerated somewhat for all walls after the middle of December 2005, and the movements continued through the winter months.

4.3 Hydrotest #1 (August 2006)

The first hydrotest was conducted between August 9th and August 20th, 2006. The maximum head achieved during the test for each wall is shown in Figure 4-3. The test was performed in two stages. During Stage 1, the outlet pipe at the base of the geocomposite back drain was left open. Head was built up in the drainage fill zone, and water flow was captured by the geocomposite drain (i.e. head was not allowed to build up in the reinforced fill zone.). In Stage 2, the outlet pipe was sealed (packers inserted in the slotted PVC outlet pipe and end capped) allowing seepage into the reinforced fill and the head buildup noted in Figure 4-3 to occur. The piezometer responses against time during the first hydrotest are presented on Figure B-41 for all walls.

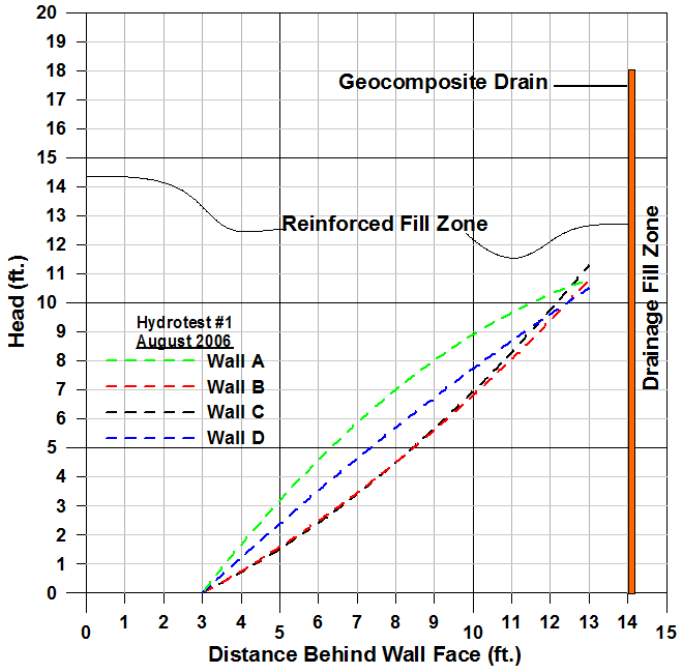


Figure 4-3: Maximum Achieved Head – Hydrotest #1

At the maximum pumping rate into the drainage fill zone, the maximum head achieved in the reinforced fill was generally about 11.0 ft. The inability to generate greater head buildup was attributed to zones of concentrated flow occurring through the reinforced fill around the contact zone with the vertical PVC geomembrane between adjacent walls; this flow was observed to daylight at the face of the walls. Incorporating a vertical geomembrane barrier between test wall sections for hydraulic containment purposes appeared to be reasonable in design, but in retrospect it introduced significant construction difficulties, not the least of which was the reluctance of the field construction team to adequately compact the soil in the zone immediately adjacent to the geomembrane for fear of damaging the barrier. This inadequate compaction effort may have left zones of loose, and more permeability soil that eroded and discharged significant quantities of water at the vertical PVC geomembrane. Based on field observations and the measured results of the first hydrotest, a significant effort was undertaken in spring/early summer 2007 to seal the primary leakage zones prior to conducting the second hydrotest.

4.3.1 Instrumentation Measurements

The inclinometers were read approximately one month after completion of the hydrotest. The data for all walls presented on Figure B-18 showed additional lateral deflection, with larger changes observed at Walls A and D. This inclinometer reading was the last performed, as the access to the instrument locations had become unsafe for our personnel. The strain gage responses shown on Figures B-19 through B-22 generally showed little to no response. Some exceptions were:

- Wall A (gages A23 and A24)
- Wall D (gages D32, D34, D43 and D45) showed larger changes during the hydrotest.

The Rod extensometers Figures B23-26 showed some lateral movement in the frontal zone of Walls A and C (about 0.5 in.). Extensometer movements greater than 1.0 in. were recorded throughout the reinforced fill section of Wall D. Very little to no extensometer movements were recorded in Wall B.

During Hydrotest #1 the maximum lateral face movements were generally less than 0.5 in. for Walls A, B and C (Figures B-32 to B-36). Wall D experienced maximum lateral movements of up to 1.0 in. Vertical face movements near the wall center were generally less than 0.25in for Walls B and C, while Walls A and D demonstrated relatively larger vertical face movements with maximums between 1.25 and 2 inches respectively. It is noted that there were some larger settlements recorded for targets near the vertical PVC geomembrane at Wall A/B.

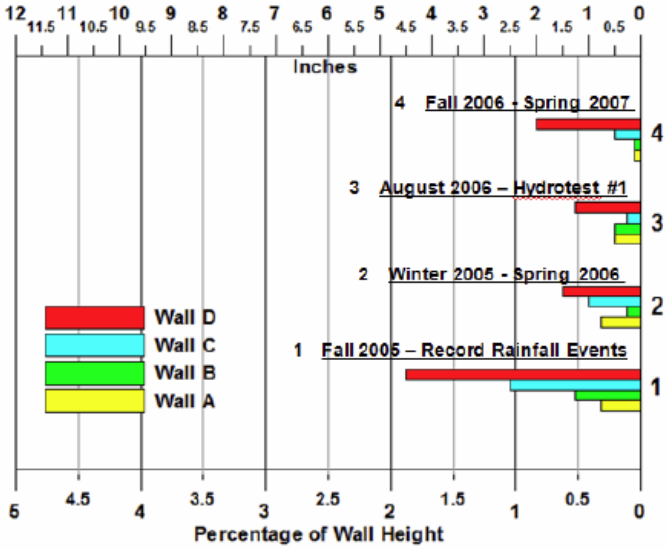


Figure 4-4: Maximum Horizontal Face Movement

During the period Fall 2006 through Spring 2007, very little horizontal face movement occurred at Walls A and B. About 0.5 in. of additional horizontal movement was recorded at Wall C and approximately 2.0 in. was experienced at Wall D (See Figure 4-4). Vertical movements observed at the faces of all walls were generally less than 0.5 in. Shortly after

completion of the Hydrotest #1 in October/November 2007, strain gages A23, A32 and C32 became inoperable.

4.4 Surcharge Fill Placement (April 2007)

This phase of the test program consisted of placing an earth surcharge fill over the top of the test walls. A 5.0 ft. high surcharge was constructed at the end of April 2007 using available on-site silt screenings. No effort was made to place and compact the surcharge fill in controlled lifts. At the face of the walls, the surcharge was sloped at 1H: 1V.

4.4.1 Instrumentation Measurements

Figure 4-5: As-Constructed Surcharge Fill

Figure B-19 presents the vertical movements recorded at the settlement plates with time in all walls. These plots show a marked response under the surcharge load at all levels and all walls. The increment of change due solely to the surcharge fill loading is represented on Figure 4-6 below. The data are presented as a column for each wall with depth, and the diameter of each circle represents the magnitude of settlement observed. Walls A and C each experienced about 1 in. of settlement; Wall B about 0.5 in. of settlement; and Wall D approximately 3 in. of settlement.

Figures B-20 through B-23 of Appendix B show increases in tensile strain observed in most of the strain gages during surcharge placement. At Walls A, B, and C small tensile strain increases less than 0.25% were recorded. At Wall D relatively larger tensile strain changes were observed, exceeding 1.2%. Strain gages D32 and D43 failed during the surcharge period.

The increment of strain for the surcharge fill load was also contoured for each wall on Figure B-42. The plot demonstrates the difference in strain magnitudes and wall response for the different reinforced fill types.

The rod extensometer data on Figures B-24 and B-27 show that the upper Level extensometers (located at 5 ft. and 10 ft. below the top of the wall) experienced horizontal movement of about 0.5 in. in all walls.

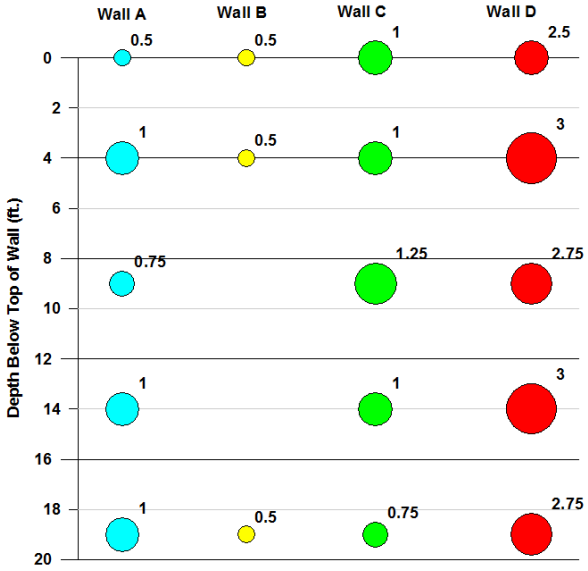


Figure 4-6: Vertical Movement due to Surcharge Fill (in.) – Settlement Plates

The wall face movements recorded by the AMTS system are shown on Figures B-32 to B-40 of Appendix B. Horizontal movements of the wall faces caused by the surcharge loading were on the order of 0.75 in. for Walls A and B; less horizontal movement was recorded at Walls C and D. Vertical wall face movements were about 1 in. for Walls A; 0.5 in. for Wall B; 1 in. for Wall C, and about 3 in. for Wall D. These magnitudes of vertical face movement correlate well with the settlement plate readings.

It should be noted that there were some larger local deformations recorded near the ends of the test walls near the vertical PVC geomembrane (e.g. B41, C45); the response at those locations are not considered representative of the general wall response to surcharge loading.

After placement of the surcharge fill, it became unsafe to access and read the inclinometers. Further inclinometer readings were, therefore, not made for the remainder of the field test for personnel safety considerations.

4.5 Hydrotest #2 (August 2007)

During Hydrotest #1 (August 2006) only moderate pressure heads were generated in the reinforced fill zones. In advance of Hydrotest #2, a program was implemented in July 2007 to increase the hydraulic efficiency of the system during the second hydrotest. Specifically, a significant effort was undertaken to seal the observed primary leakage regions in the zone immediately adjacent to the vertical geomembrane separating the different wall sections. A drill rig was used to inject chemical grout into the suspect leakage zones. Figure 4-7 shows the maximum head achieved during Hydrotest #2 (solid lines), in comparison with the heads achieved in Hydrotest #1 (dashed lines) for each wall. A 30% increase in head was achieved during Hydrotest #2 and a greater portion of the reinforced fill zone of the test walls was saturated. The time to achieve steady state flow is ultimately controlled by the permeability of the reinforced fill. Therefore, it was not possible to fully saturate the reinforced fill zone in any of the walls during the short term pump test time period.

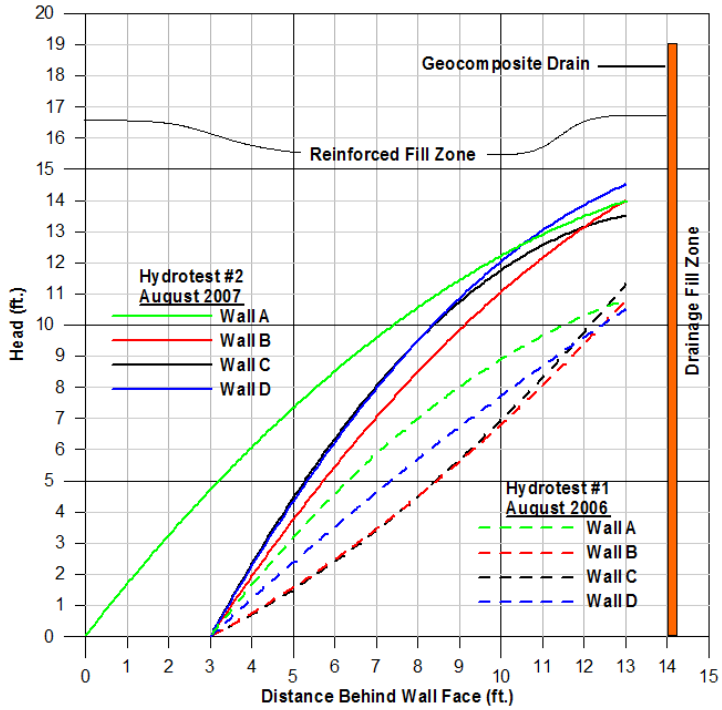


Figure 4-7: Comparison of Maximum Achieved Head (Hydrotest #1& #2)

The piezometer responses observed during Hydrotest #2 are presented graphically against time on Figure B-43 for all walls. Contours of the maximum total pressure head measured are presented in Figure B-44. The contours for Walls B, C, and D appear very similar in pattern. Wall A is similar for the upper zone of measurement, but showed somewhat greater water pressure heads at the lower level near the wall face.

4.5.1 Instrumentation Measurements

The vertical movement recorded at the settlement plates are shown on Figure B-19. Walls A, B and C all showed settlement of about 0.5in at all plate levels. Wall D showed larger movements of about 1in at all plate levels.

Figures B-20 through B-23 of Attachment B show the strain gage responses for all walls at all levels. Generally there were negligible increases in tensile strain on the upper layers (3 & 4) of reinforcement. Increases typically of 0.25% were observed in the lower two instrumented levels (1 & 2) and for all walls. These strains were back from the wall face in the reinforced zone. Wall D did have one gage that showed nearly 2.5% strain. Multiple gages in Wall D malfunctioned prior to or during the test (including D12, D22, D24, D34 and D45).

The rod extensometer data on Figures B-24 and B-27 show that small lateral movements of about 0.5in were recorded in the frontal zone near the faces of Walls A, B and C. Lateral movements as large as 2.0 in. were recorded in the reinforced fill of Wall D.

The wall face movements recorded by the AMTS system are shown on Figures B-32 to B-40. Horizontal movements recorded during Hydrotest #2 were about 0.75 in. maximum for Walls A, B and C. Wall D experienced larger maximum lateral movements of nearly 1.75 in. Maximum vertical movements of 0.5 in. were recorded on Walls A, B and C, while at Wall D maximum settlement was nearly 1.5 in.

4.6 Thermal Performance

The project duration allowed for observations of the walls through two winter seasons. The results give some insight into the effects of freeze-thaw conditions on the performance of different backfills. The backfills contain more fines than usual, and may be more susceptible to damage from freezing.

Figures B-45 through B-48 present temperature versus time graphs for each test wall through the duration of the field test. The temperatures are shown for the readings recorded at a point 3 ft. behind the wall face and at different vertical positions below the top of the wall. The average daily ambient temperature is also shown. It is worth noting that the faces of Walls A and B were south facing and Walls C and D north facing.

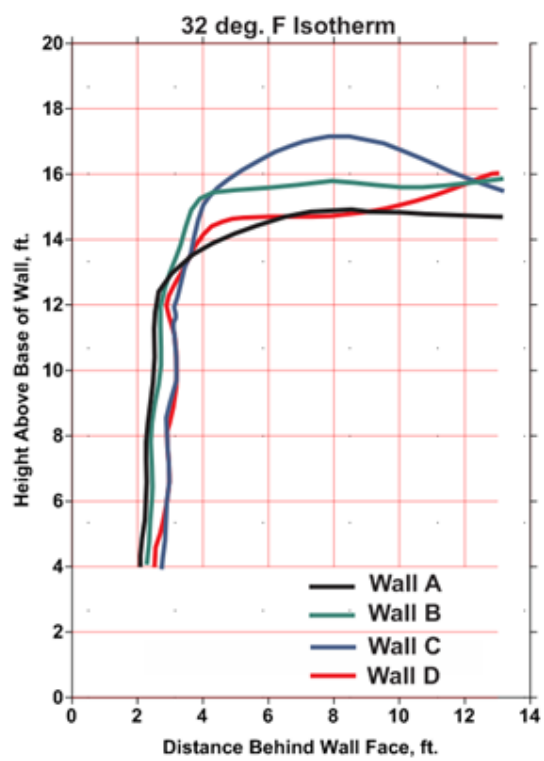


Figure 4-8: Mid-Winter 2007

Contours of temperature within the reinforced fill zone were plotted for the coldest period of the field test (mid-winter 2007), and are presented in Figure B-49 for all walls. The 32 degree Fahrenheit isotherms noted on the figures are reproduced in Figure 4-8. The penetration of cold is 'deepest' in Walls A (least fines) and D (greatest fines). Of note is the limited extent of the freezing zone perpendicular to the wall face, particularly below about mid-wall height.

The response of strain gages near the wall face and AMTS face targets were evaluated during the spring period of 2006 and 2007 to discern any freeze/thaw effects. Walls A and B exhibited almost no movement that might be attributed to freeze thaw. Wall C experienced minor lateral face movements (less than 0.5 in.) that may be attributed to freeze/thaw. Wall D, however, experienced lateral face movements approaching 2.0 in. and strain increases in the reinforcement of about one percent. While Walls C and D were constructed of the same reinforced fill material, it is likely that the NWGT reinforcement in Wall D provided a pathway for moisture to migrate from the face of the wall. This would permit buildup of ice lenses in the frontal zone, which could then provide additional free water during thawing, resulting in a greater reduction in shear strength and larger face movements.

4.8 Summary of Instrumentation Results

All walls survived the surcharge loading and two hydrotests without structural failure. However, because of excessive leakage near the vertical geomembrane the water levels during hydrotests only reached 14 ft. of total head, whereas 18 ft. of total head was used for the design. Nevertheless, the walls behaved as anticipated in the design. In addition, the monitoring systems largely worked well and produced an enormous amount of detail about the performance of each wall.

Figure B-50 summarizes the performance history of the strain gages for all walls during the test period with respect to mechanical failure of the gages. Twenty-two strain gage locations were monitored on the reinforcement in each wall. Nineteen (19) of eighty-eight gage locations malfunctioned, eleven (11) of which failed on the non-woven geotextile at Wall D. This result correlates with the difficulty gaging the non-woven geotextile. The gaging did not allow for direct connection of the gage to the geotextile, as stiffening of the material (i.e. with adhesives and water-proofing) resulted in the strains bridging around the gage location. Instead the gages were installed on a rubber membrane that was attached between two small plates that were pinched onto the geotextile. The gage strained consistently with the strain between the pinch points of the plates. The mounting, however, was delicate and was probably the reason that so many gages failed on this wall.

The absence of strain gage data through the entirety of the test for Wall D makes contouring strains at completion of the program problematic. Instead, Figure B-42 provides plots of incremental contours of strain measured during the surcharge loading for each of the walls. At this stage of the test only a few of the gages had malfunctioned. Figure B-42 demonstrates that the patterns of strain observed are similar with the largest increments of strain seen halfway back from the wall face in the reinforced zone. A mode of deformation/failure can also be inferred. The magnitudes of response to the surcharge loading are small for walls A, B and C, but Wall D shows maximum strain at nearly 4 times the strain level observed at the other walls.

Figure B-51 shows contour plots of horizontal movement of the face of the walls at the end of the test. There are clearly variations due to the end effects near the vertical geomembrane. Generally, the centerline at Wall A displaced outwards by about 3.5 inches, Wall B moved outwards by about 3 inches, Wall C moved out about 5½ inches and Wall D by up to 12 inches.

Despite high deformation levels at the wall face, the walls continued to perform well in terms of structural stability.

The Automated Total Monitoring Station was particularly valuable at providing detailed information on the vertical and horizontal movements of numerous points located on the face of the wall and settlement of plates buried in the reinforced fill. Figures B-32 through B-40 show the measured horizontal and vertical displacement of surveying prisms located on the face of the wall at the center of the loaded area. These movements are continuous from the time the prism could be installed, which was right after placement of the lift of fill in which the prism was anchored. To our knowledge this is the first application of automated total stations to monitor the long term performance of mechanically stabilized earth retaining walls. The system worked very well. The data clearly show the displacements of the wall occurring at the times when additional shear stress was imposed on the soil and reinforcement.

Figure B-52 summarizes settlement plate movements recorded in each wall throughout the duration of the field test. Each plot shows the observed response for each event/period at each level. The size of the circle represents the magnitude of observed change. The portion of the total reinforced fill settlement attributed to both environmental events and the test events is noted. The maximum measured settlements were: Wall A: 3.25 in., Wall B: 2.0 in., Wall C: 2.5 in. and Wall D: 9.25 in.

Figures B-53 and B-54 summarize maximum horizontal and vertical face movement at the center of the walls for both environmental and test events. Maximum total measured horizontal/vertical face movement at the center of the wall was: Wall A: 3.75 in./6.0 in., Wall B: 3.5 in./3.25 in., Wall C: 5.75 in./8.0 in. and Wall D: 11.0 in./12.75 in.

The percentage distribution of maximum horizontal and vertical face movements for each wall, as a result of environmental factors and test events, is indicated in Figure 4-9. It is interesting to note that for horizontal face movement 60% of the movement for Walls A and B (lower "fines") occurred during the "test events" and 40% of the movement occurred as a result of "environmental factors". Conversely, for Walls C and D (higher "fines") greater horizontal movement (70%) occurred due to "environmental factors" and 30% occurred during the "test events".

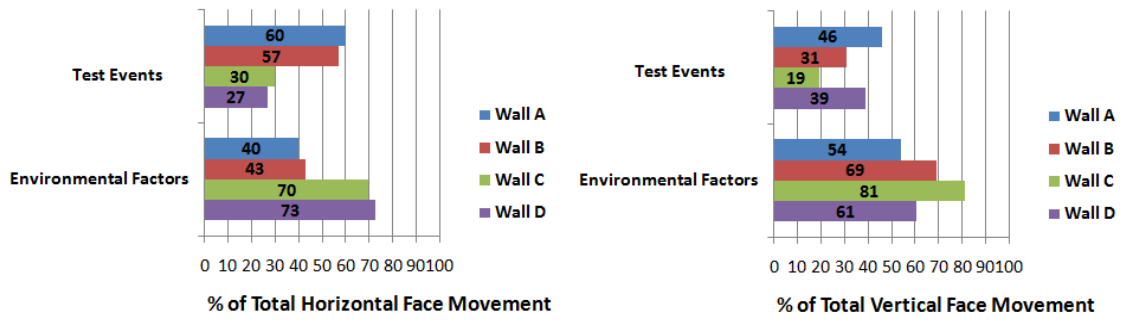


Figure 4-9: Wall Face Movement Summary

Figure B-55 presents a summary of rod extensometer movements for all walls at the end of the field test. Clearly there were greater horizontal movements at Wall D in the reinforced zone than for the other walls. The slope of the lines for each layer represents the global strain occurring in the reinforced fill – the steeper the slope the larger the strain. Calculations of global strain between each rod anchor were also calculated and shown on Figures B-28 to B31. At the

end of the field test, the data shows that the maximum global strains in the reinforced fills at Walls A, B and C were less than 2%; generally strains were less than 1% for most calculations. For Wall D the maximum global strains were 5.75%. For all walls, the largest strains are seen near the wall faces.

4.9 Evaluation of Wall Performance

Our overall observations indicate that Walls A and B performed much better than Wall C and Wall D. Vertical and horizontal face movements were typically 1.5 to 3 times greater in Walls C and D than Walls A and B during each event and test stage (including construction and surcharge). This observation indicates some concerns for construction alignment control and down drag effects (even if water is controlled) when more fine-grained soils with higher plasticity are used.

The strain gage measurements indicate that the strains did not generally exceed 1% at Walls A and B for the entire field test. This is borne out by both the strain gage and rod extensometer readings. Wall C showed somewhat larger local strains from the strain gages, but the global strains were similar to Walls A and B. Many gages failed on Wall D so the strain field for the entire testing period is not well defined, but the global strains from the rod extensometers indicate a significantly higher strain level (between 3% and 5.75%) was recorded in Wall D within the frontal zone.

All four of the walls withstood extreme loading from the five foot surcharge and a buildup of pore pressure within the reinforced fill during the hydrotests without experiencing any observed failure of the reinforcing elements. This happened even though the reinforcing was designed to fail during the last load event. It should be noted, however, that the hydrotests did not achieve the full pore pressure build up assumed in the reinforcement design analyses. In any event, this outcome suggests that our methods of calculating the forces in the reinforcement for MSE walls may predict higher forces than actually develop and that the actual soils used in the test walls were stronger than considered in the design of the walls. This outcome needs further consideration in future work.

Figures B-51 and B-53 show that Walls A (A-1-a soil) and B (A-1-b soil) performed similarly with respect to overall displacement. Wall C (A-4 soil) deformed vertically and horizontally about 60% more than Walls A and B. This is an expected result because a wall constructed with a higher fines reinforced backfill is expected to deform more. Wall D (A-4 soil) - constructed with the same reinforced backfill soil as Wall C - deformed 60 to 90% more than Wall C. This also is an expected result because of the lower stiffness of the non-woven geotextile reinforcement, when compared to the geogrid reinforcement.

Wall D (constructed with an A-4 soil (60% fines) and the needle punched nonwoven geotextile reinforcement) experienced about one foot of vertical and horizontal deformation or roughly 5% of the wall height. Although it remained structurally competent, the face/exterior had the look and feel of a failed wall. Considered from the perspective of a deformation limit state, this wall - for all practical purposes - failed. The Wall D design anticipated lateral transmissivity of water through the geotextile reinforcement to allow for some drainage and dissipation of pore water pressure; the intent was to improve the performance of a marginal soil (e.g., Koerner and Soong, 1999 and Zornberg, Christopher and Mitchell, 1998). However the piezometric measurements made during the hydrotests (e.g., Figure B-41 and B-43) indicate that the pore water pressure in Wall D was approximately the same as the other walls and the pressures did not appear to dissipate significantly faster in comparison with the other walls. The transmissivity of the selected geotextile was relatively low (an order of magnitude less than similar nonwoven,

needle punched geotextiles) and the strength was 30% less than anticipated. As a result, Wall D is more or less an evaluation of a weaker geosynthetic, lower modulus reinforcement. It demonstrates the nature of the performance anticipated for an under-designed MSE wall system using marginal reinforced fill soil (i.e., significant lateral and vertical movements, face distortions, and tension cracks in the surface of the structure).

Although not part of the study scope, finite element analyses were performed to predict wall deformations and strains in the reinforcement for each test wall. To illustrate both the success and limitations of the FEA analysis for this project study, we have limited our discussion to the modeling of surcharge loading in comparison with the field observations on Walls A and B only.

The predicted displacements of Wall A are compared with the observed response at the wall face in Figure 4-10. The finite element modeling tended to slightly over-predict the movements at the wall toe and under-predict the observed movements at/near the wall top, but the magnitudes are very similar.

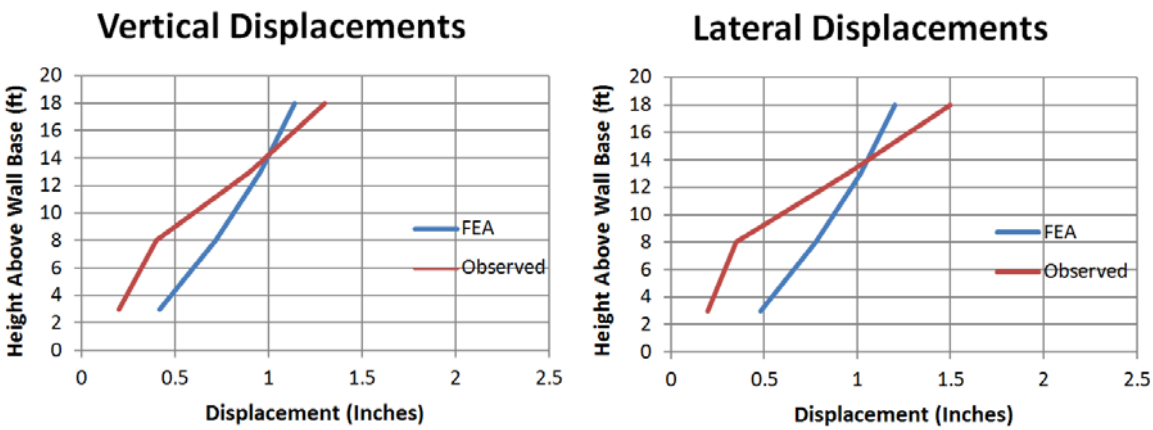


Figure 4-10: Predicted and Observed Wall Face Displacements at Wall A Surcharge Load

The predicted strains in Figure 4-11 are also larger near the toe than the strains interpreted from the strain gage measurements and shown as strain contours for Wall A. The incremental strain magnitudes for the surcharge, however, are similar.

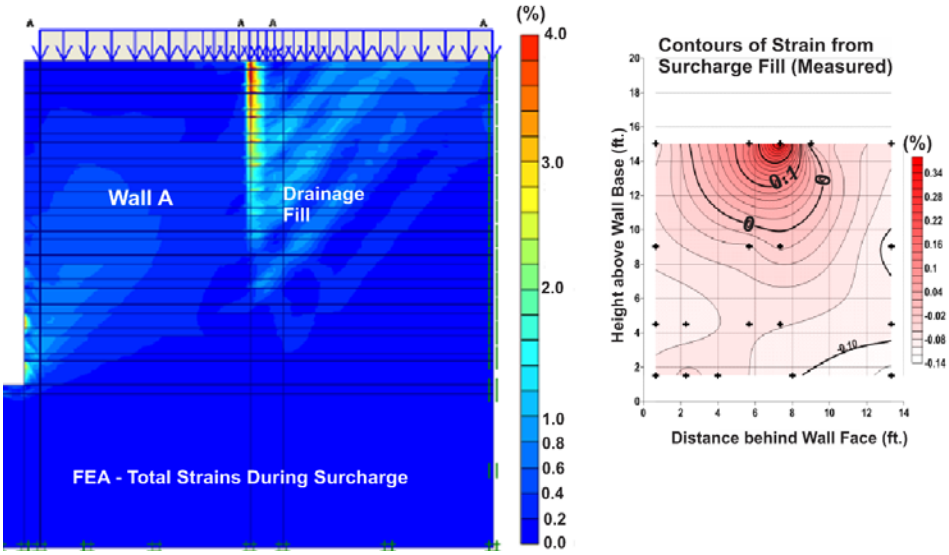


Figure 4-11: Predicted and Observed Strains – Wall A Surcharge Load

For Wall B, predicted displacements are compared with the observed and are shown in Figure 4-11. The finite element modeling over-predicted the observed face movements by a factor of about 6 for both lateral and vertical deformations. The reason for this result has not been determined, and requires further evaluation.

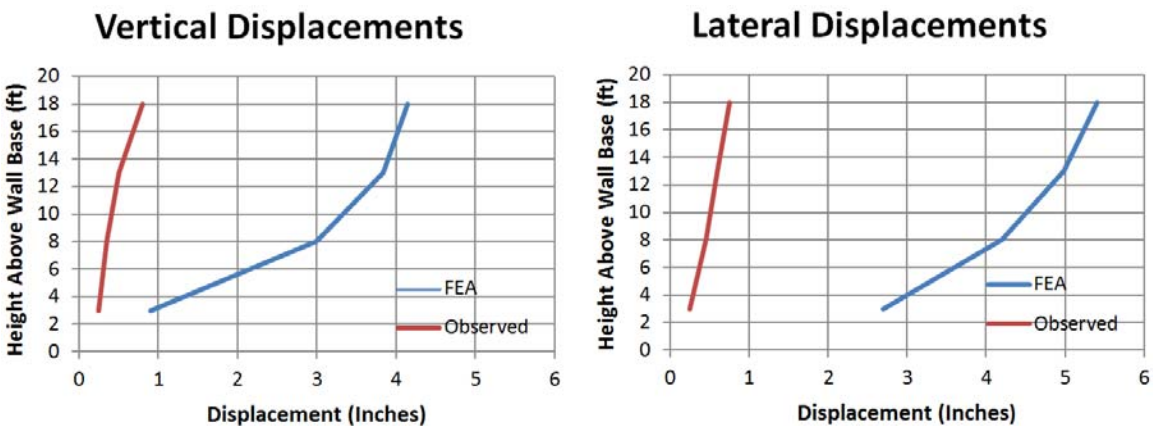


Figure 4-12 – Predicted and Observed Wall Face Displacements at Wall B

Finite modeling analyses for Walls C and D produced results similar to those for Wall B and were so different from the observed behavior that the FEA were not really useful.

CHAPTER 5

CONCLUSIONS

The test program and monitoring results show that mechanically stabilized earth retaining walls can withstand positive pore water pressures, provided they are designed to do so. The tests also show that increasing the pore pressure in the reinforced fill causes lateral and vertical deformations of the wall that are as significant as those resulting from adding surcharge loads.

Based on the performance of Wall B, Soils with an AASHTO A-1-b classification (e.g. soils with as much as 25% fines and a Plasticity Index below 6%) can be successfully used as backfill materials in MSE structures, provided the design uses the appropriate material properties and takes into consideration any positive pore pressures that may develop in the reinforced fill over the life of the structure.

Soils with high fines and some plasticity have lower permeability which creates more difficulty placing the materials to the desired condition. Moisture and density control become more important and can be more difficult to achieve. Also, settlement will occur during placement and compaction of such soils, which will induce down drag at the connections for rigid faced walls, such as modular block. Construction with these materials is more weather dependent. Consequently, a higher degree of quality control is necessary to make sure the materials are placed and appropriately compacted to minimize future performance issues.

The lower permeability of soils with high fines and some plasticity also creates the possibility for unintended pore pressures to build up in the reinforced fill and cause adverse performance, if not explicitly considered in the design. This result was clearly demonstrated when the test site experienced 15 inches of rain over a one month period. Large lateral movements near the face of the wall occurred, which we believe to be the result of a buildup in pore pressure near the face of the wall of sufficient magnitude to cause the wire basket facing elements to slide over the geosynthetic reinforcement. This observation appears to support Terzaghi's 1943 conclusion that resultant pressures on walls can significantly increase during storm events when the soil has permeability less than 0.002 cm/sec. Due to the potential for such unplanned behavior, it appears that reinforced fills with permeability less than 10^{-3} cm/sec should be designed with positive drainage measures at the base and back of the reinforcing and provisions to prevent surface water from entering the reinforced fill material.

Due to the significant effects of pore water pressure build up in the reinforced zone on stability and deformations and the complications and uncertainties introduced by the added forces from pore pressure increase, we conclude that the best practice for using higher fines soils in MSE structures is to adopt measures that minimize the potential of pore pressure build up in the reinforced fill. By preventing pore pressure build up, the higher fines soils have adequate strength and stiffness characteristics to provide a wall that functions safely and with acceptable deformation. All potential sources of pore pressure build up in the reinforcing zone should be considered in the design, including horizontal flow from the retained soil, downward flow from surface water and upward flow from the foundation. Where potential sources exist or may develop during the life of the facility, the design should include specific measures to prevent flow of water into the backfill in the form of a water barrier, a drain or both.

The research clearly demonstrates that MSE structures with a greater percentage of fine-grained soils in the reinforced fill than currently allowed by AASHTO will provide acceptable performance subject to the following considerations:

- Soils with less than 25% fines and a Plasticity Index not greater than 6% are used.
- All potential sources of pore pressure build up in the reinforcing zone should be considered in the design.
- Good practices are used to control the quality of material selection and placement as the performance of marginal soils is more susceptible to as-place density and moisture content than free draining materials.

The main report contains more guidance on implementation of these recommendations.

ATTACHMENT A

GeoTesting express

1145 Massachusetts Avenue
Boxborough, MA 01719
978 635 0424 Tel
978 635 0266 Fax

Geotechnical Test Report

June 15, 2005

NCHRP 24-22 Project

Prepared for:

Geocomp consulting

GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Composite	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	05/21/05	Checked By:	n/a
Depth :	---	Sample Id:	30624		
Test Comment:	---				
Sample Description:	Moist, gray silty gravel with sand				
Sample Comment:	---				

Moisture Content of Soil - ASTM D 2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
Composite	Proposed A-1-a (4:1 Mix)	---	Moist, gray silty gravel with sand	6

Notes: Temperature of Drying : 110° Celsius

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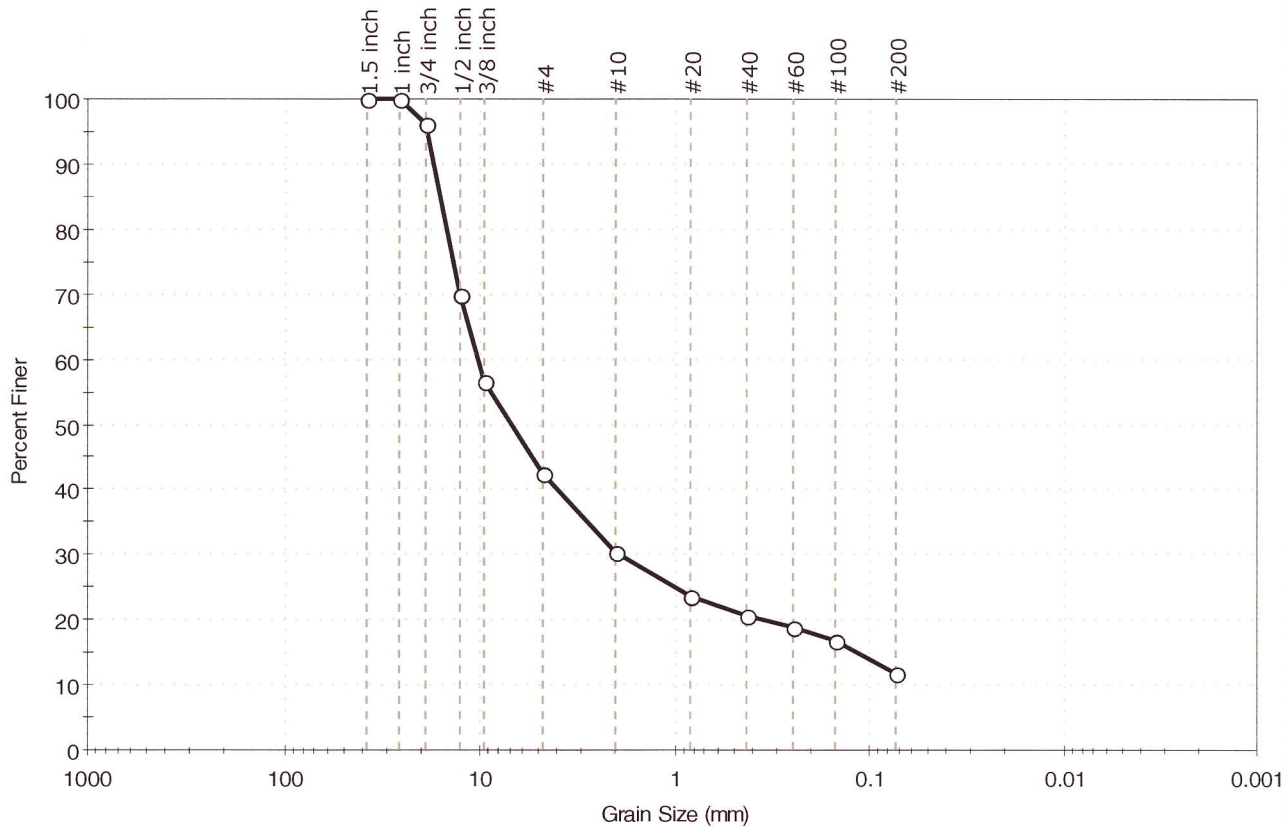
GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	No. 1	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	06/14/05	Checked By:	jdt
Depth :	Bucket 8	Test Id:	71168		
Test Comment:	---				
Sample Description:	Moist, gray gravel with silt and sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	57.6	30.6	11.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 inch	38.10	100		
1 inch	25.70	100		
3/4 inch	19.00	96		
1/2 inch	12.70	70		
3/8 inch	9.51	57		
#4	4.75	42		
#10	2.00	30		
#20	0.84	24		
#40	0.42	21		
#60	0.25	19		
#100	0.15	17		
#200	0.074	12		

Coefficients

D_{85} = 15.9960 mm D_{30} = 1.9367 mm

D_{60} = 10.2256 mm D_{15} = 0.1172 mm

D_{50} = 6.8769 mm D_{10} = 0.0566 mm

C_u = 180.664 C_c = 6.481

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

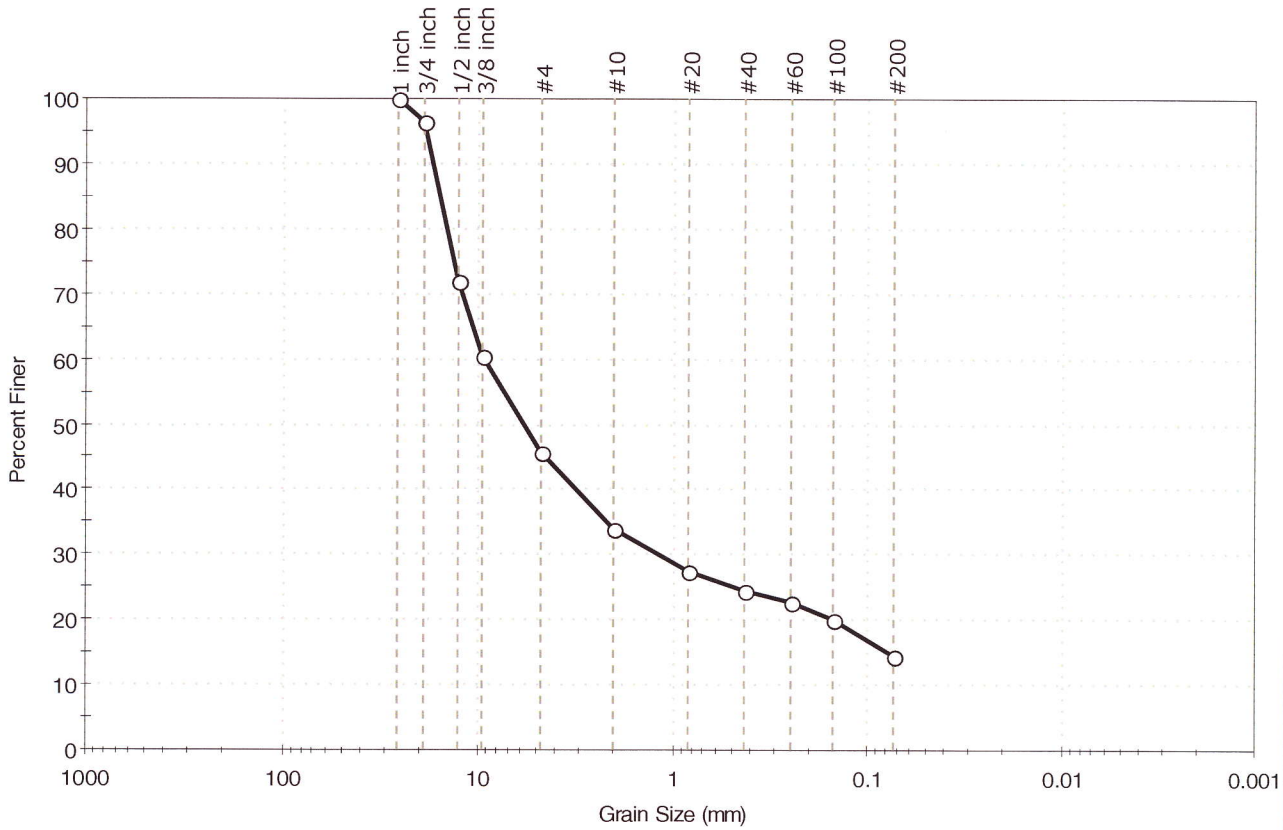
Sand/Gravel Hardness : HARD

GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	No. 2	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	06/15/05	Checked By:	jdt
Depth :	Bucket 1	Test Id:	71169		

Test Comment: ---
Sample Description: Moist, gray silty gravel with sand
Sample Comment: ---

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	54.2	31.5	14.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.70	100		
3/4 inch	19.00	96		
1/2 inch	12.70	72		
3/8 inch	9.51	60		
#4	4.75	46		
#10	2.00	34		
#20	0.84	27		
#40	0.42	24		
#60	0.25	23		
#100	0.15	20		
#200	0.074	14		

Coefficients

D_{85} = 15.7502 mm D_{30} = 1.1895 mm
D_{60} = 9.3572 mm D_{15} = 0.0803 mm
D_{50} = 5.8110 mm D_{10} = 0.0441 mm
C_u = 212.181 C_c = 3.429

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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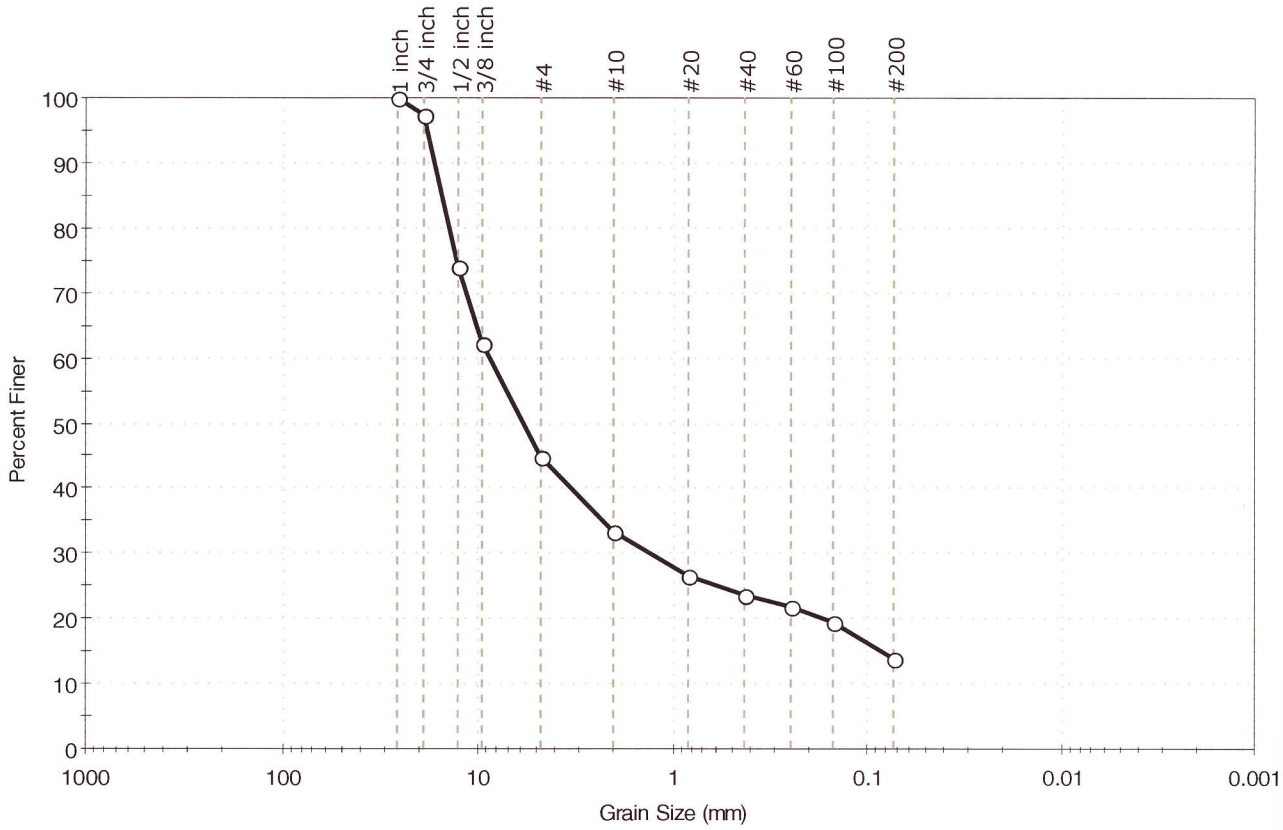
GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	No. 3	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	06/15/05	Checked By:	jdt
Depth :	Bucket 17	Test Id:	71170		
Test Comment:	---				
Sample Description:	Moist, gray silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	55.1	31.1	13.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.70	100		
3/4 inch	19.00	97		
1/2 inch	12.70	74		
3/8 inch	9.51	62		
#4	4.75	45		
#10	2.00	33		
#20	0.84	27		
#40	0.42	24		
#60	0.25	22		
#100	0.15	19		
#200	0.074	14		

Coefficients

D_{85} = 15.3226 mm D_{30} = 1.3209 mm

D_{60} = 8.7284 mm D_{15} = 0.0860 mm

D_{50} = 5.8350 mm D_{10} = 0.0455 mm

C_u = 191.833 C_c = 4.393

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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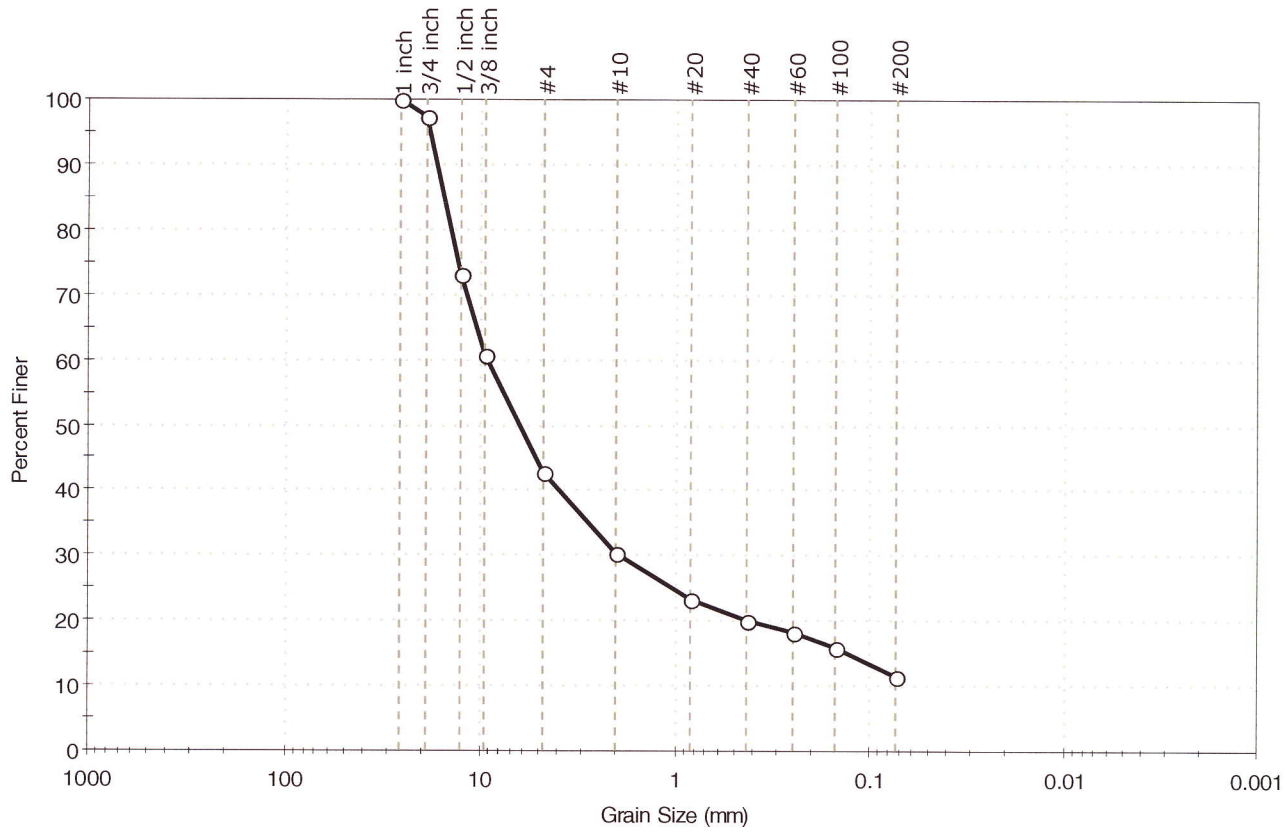
GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	No. 4	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	06/15/05	Checked By:	jdt
Depth :	Bucket 6	Test Id:	71171		
Test Comment:	---				
Sample Description:	Moist, gray gravel with silt and sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	57.3	31.3	11.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.70	100		
3/4 inch	19.00	97		
1/2 inch	12.70	73		
3/8 inch	9.51	61		
#4	4.75	43		
#10	2.00	30		
#20	0.84	23		
#40	0.42	20		
#60	0.25	18		
#100	0.15	16		
#200	0.074	11		

Coefficients

D_{85} = 15.4712 mm D_{30} = 1.8982 mm

D_{60} = 9.2552 mm D_{15} = 0.1286 mm

D_{50} = 6.2927 mm D_{10} = 0.0599 mm

C_u = 154.511 C_c = 6.499

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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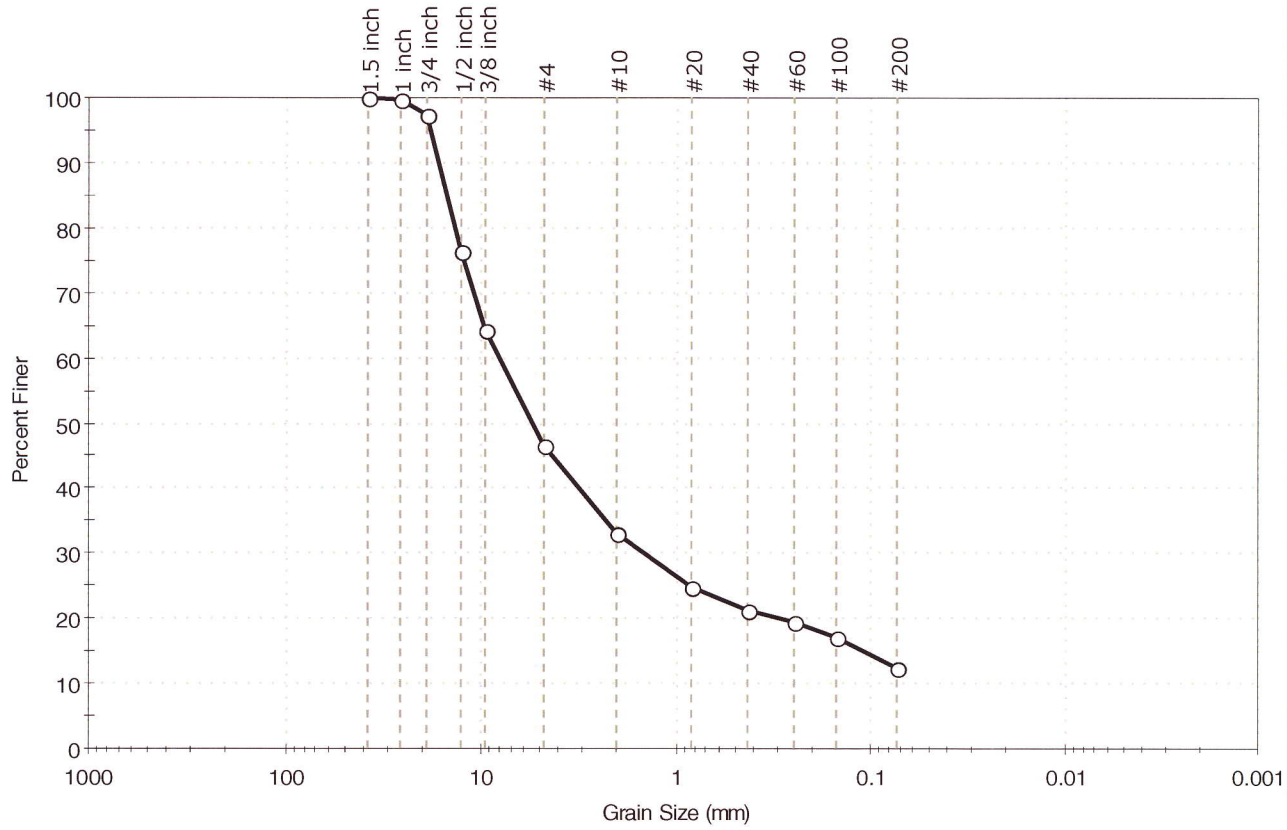
GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	No. 5	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	06/15/05	Checked By:	jdt
Depth :	Bucket 2	Test Id:	71172		
Test Comment:	---				
Sample Description:	Moist, gray silty gravel with sand				
Sample Comment:	2 of 17				

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	53.4	34.2	12.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 inch	38.10	100		
1 inch	25.70	100		
3/4 inch	19.00	97		
1/2 inch	12.70	76		
3/8 inch	9.51	64		
#4	4.75	47		
#10	2.00	33		
#20	0.84	25		
#40	0.42	21		
#60	0.25	20		
#100	0.15	17		
#200	0.074	12		

Coefficients

D_{85} = 14.9758 mm D_{30} = 1.4578 mm

D_{60} = 8.0578 mm D_{15} = 0.1083 mm

D_{50} = 5.4325 mm D_{10} = 0.0514 mm

C_u = 156.767 C_c = 5.131

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

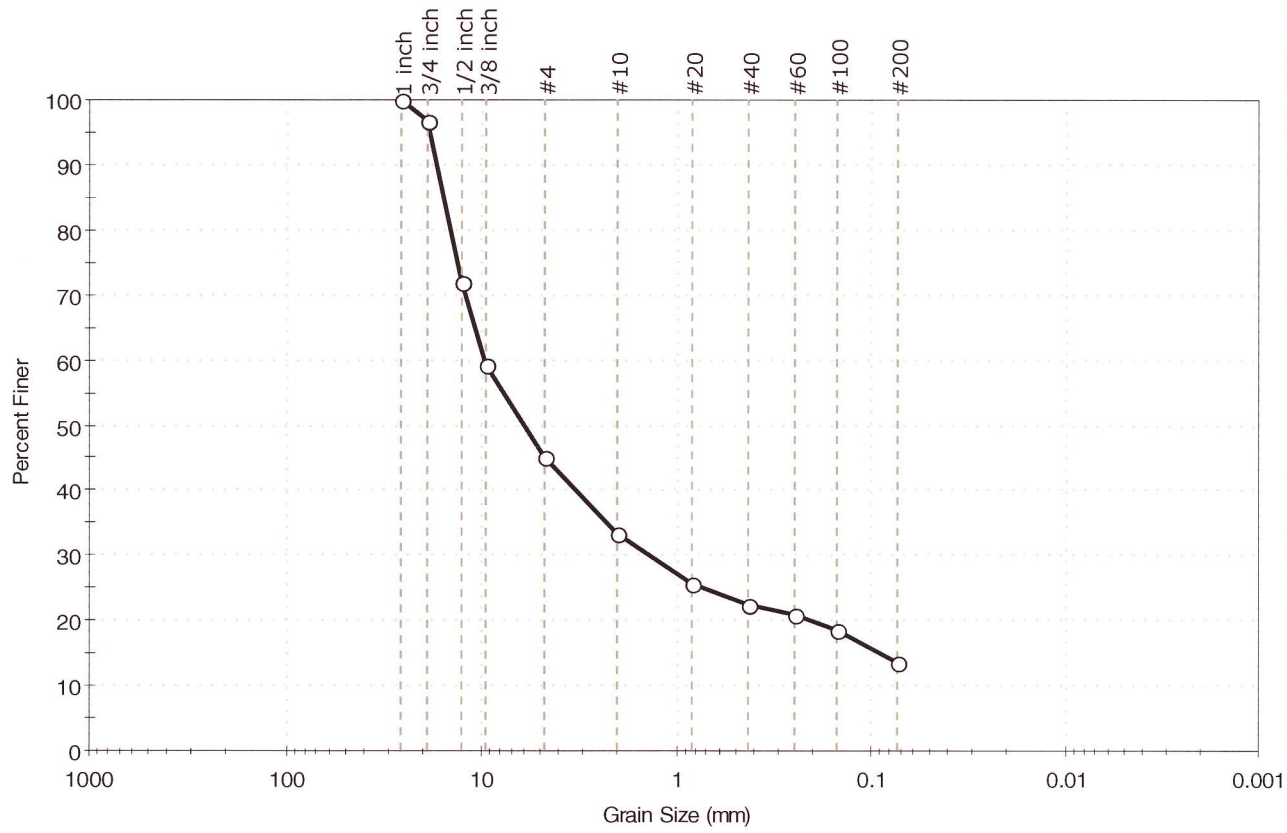
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	No. 6	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	06/14/05	Checked By:	jdt
Depth :	Bucket 16	Test Id:	71173		
Test Comment:	---				
Sample Description:	Moist, gray silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	54.7	31.8	13.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.70	100		
3/4 inch	19.00	97		
1/2 inch	12.70	72		
3/8 inch	9.51	59		
#4	4.75	45		
#10	2.00	33		
#20	0.84	26		
#40	0.42	23		
#60	0.25	21		
#100	0.15	19		
#200	0.074	13		

Coefficients

D_{85} = 15.7224 mm D_{30} = 1.3771 mm

D_{60} = 9.6737 mm D_{15} = 0.0910 mm

D_{50} = 6.0069 mm D_{10} = 0.0464 mm

C_u = 208.485 C_c = 4.225

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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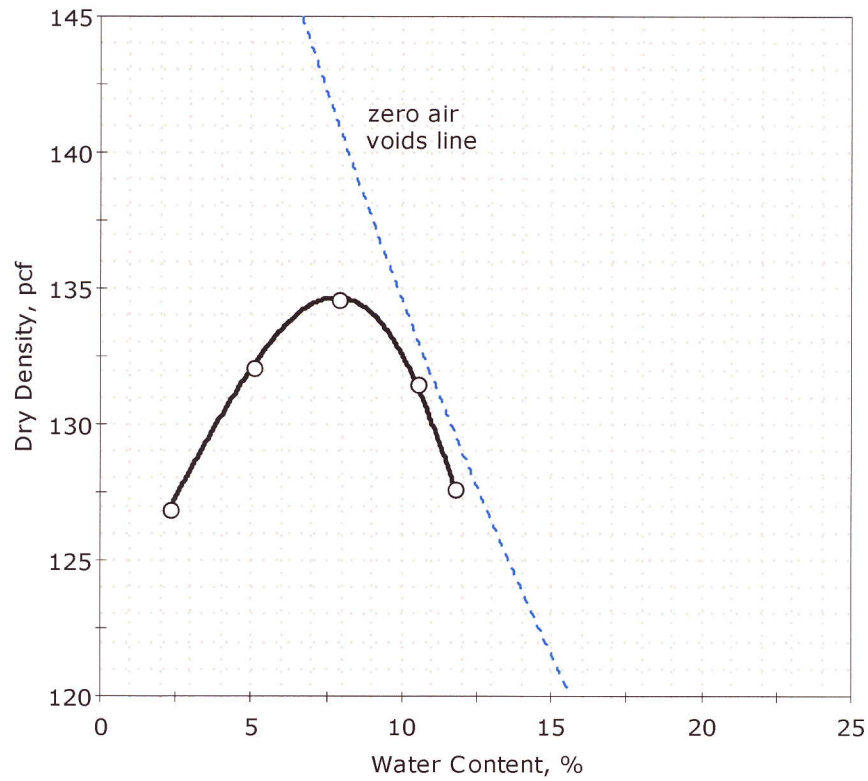
Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Composite	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	06/17/05	Checked By:	jdt
Depth :	---	Test Id:	71177		
Test Comment:	---				
Sample Description:	Moist, gray silty gravel with sand				
Sample Comment:	---				

Compaction Report - ASTM D 698



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5
Dry density, pcf	126.9	132.1	134.6	131.5	127.7
Moisture Content, %	2.3	5.1	7.9	10.5	11.8

Method : C

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.75

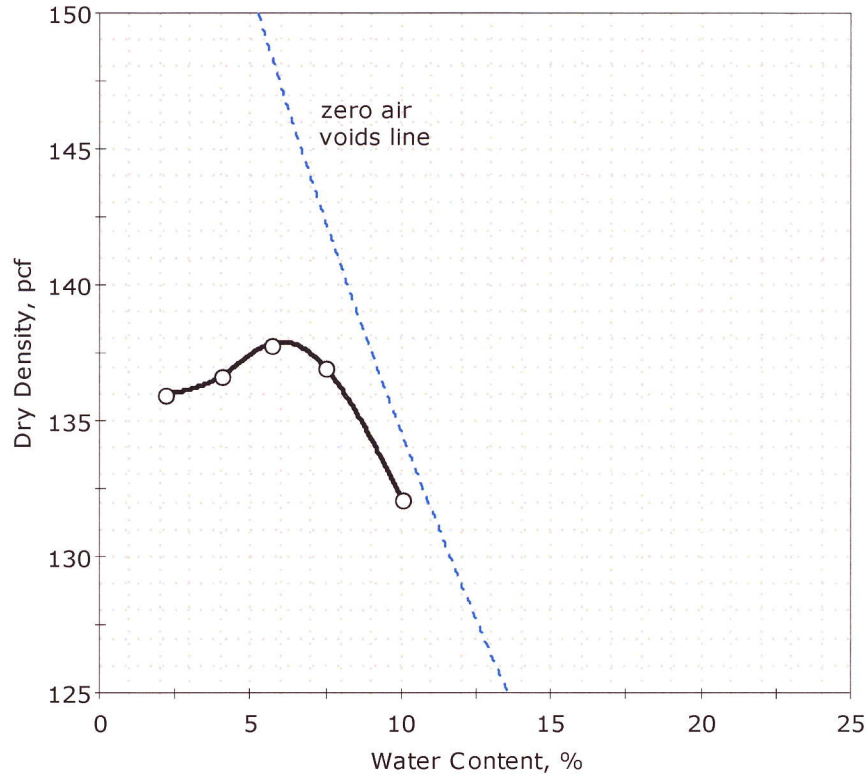
Maximum Dry Density= 134.5 pcf

Optimum Moisture= 8.0 %

printed 6/21/2005 9:27:55 AM

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Composite	Sample Type:	bucket	Tested By:	pcs
Sample ID:	Proposed A-1-a (4:1 Mix)	Test Date:	06/15/05	Checked By:	jdt
Depth :	---	Test Id:	71176		
Test Comment:	---				
Sample Description:	Moist, gray silty gravel with sand				
Sample Comment:	---				

Compaction Report - ASTM D 1557



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5
Dry density, pcf	136.0	136.6	137.8	136.9	132.1
Moisture Content, %	2.1	4.0	5.7	7.5	10.0

Method : C

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.75

Maximum Dry Density= 138.0 pcf

Optimum Moisture= 6.0 %

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GeoTesting express

Client:	Geocomp Consulting		
Project Name:	NCHRP 24-22		
Project Location:	Lunenburg MA		
GTX #:	5504		
Start Date:	06/20/05	Tested By:	pcs
End Date:	06/20/05	Checked By:	jdt
Boring #:	Composite		
Sample #:	Proposed A-1-a (4:1 Mix)		
Depth:	---		
Visual Description:	Moist, gray silty gravel with sand		

Permeability of Granular Soils (Constant Head) by ASTM D 2434

Sample Type: Remolded

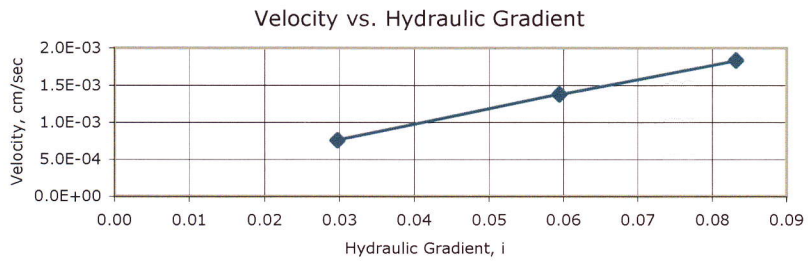
Sample Information:

Maximum Dry Density:	138.0 pcf
Optimum Moisture Content:	6.0 %
Compaction Test Method:	ASTM D 1557
Classification (ASTM D 2487):	---
Assumed Specific Gravity:	2.70

Sample Preparation / Test Setup: Compacted to 95% of Maximum Dry Density at air-dried moisture content; >1 inch material screened out of sample prior to testing (<1% of sample). 5.27 lb surcharge

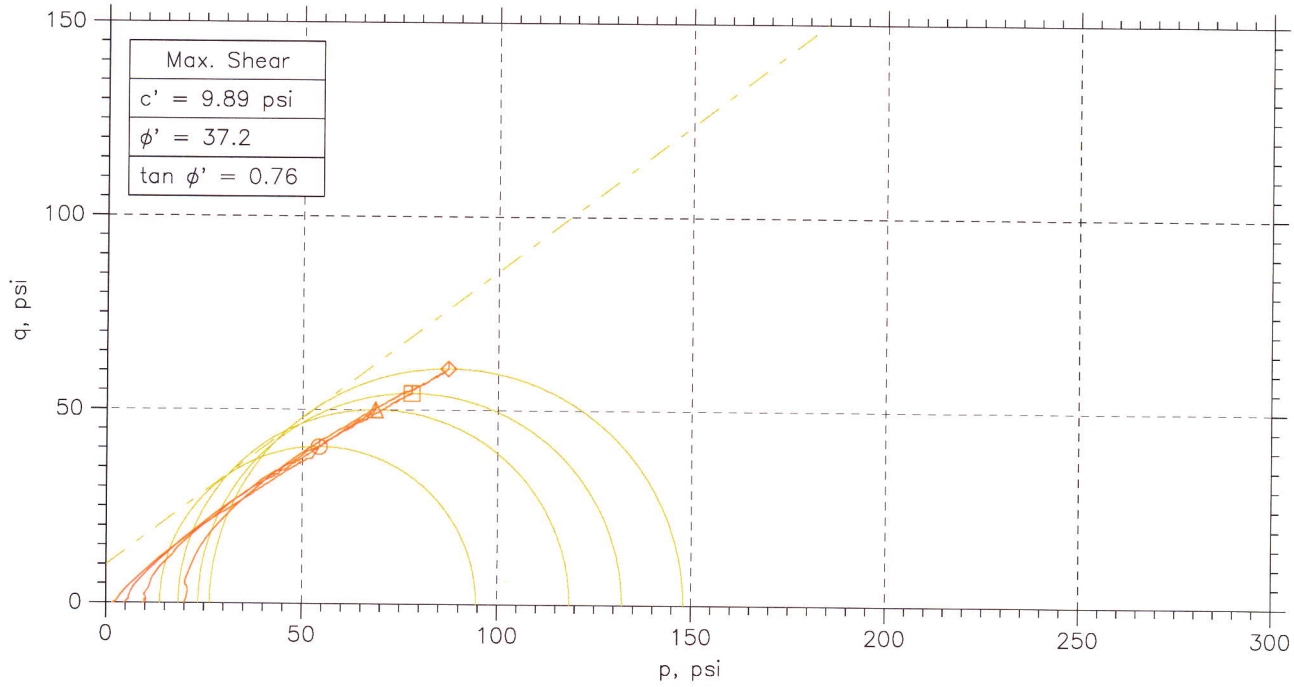
Parameter	Initial	Final
Height, in	5.05	5.05
Diameter, in	9.50	9.50
Area, in^2	70.9	70.9
Volume, in^3	358	358
Mass, g	12322	13670
Bulk Density, pcf	131	145
Moisture Content, %	0.4	10
Dry Density, pcf	131	132
Degree of Saturation, %	---	98.3
Void Ratio, e	---	0.27

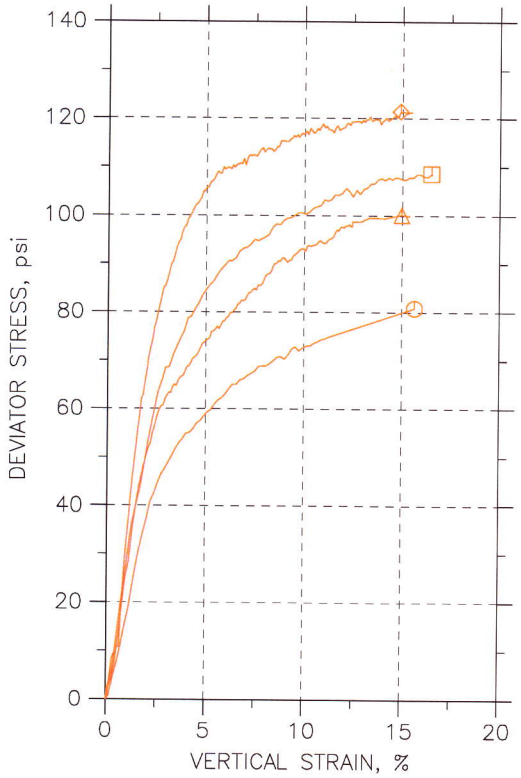
Date	Reading #	Volume of Flow, cc	Time of Flow, sec	Flow Rate, cc/sec	Gradient	Permeability, cm/sec	Temp., °C	Correction Factor	Permeability @ 20 °C, cm/sec
06/20	1	3.45	10	0.35	0.03	2.5E-02	15.0	1.135	2.9E-02
06/20	2	3.47	10	0.35	0.03	2.6E-02	15.0	1.135	2.9E-02
06/20	3	3.49	10	0.35	0.03	2.6E-02	15.0	1.135	2.9E-02
06/20	4	6.33	10	0.63	0.06	2.3E-02	15.0	1.135	2.6E-02
06/20	5	6.29	10	0.63	0.06	2.3E-02	15.0	1.135	2.6E-02
06/20	6	6.29	10	0.63	0.06	2.3E-02	15.0	1.135	2.6E-02
06/20	7	8.39	10	0.84	0.08	2.2E-02	15.0	1.135	2.5E-02
06/20	8	8.40	10	0.84	0.08	2.2E-02	15.0	1.135	2.5E-02
06/20	9	8.43	10	0.84	0.08	2.2E-02	15.0	1.135	2.5E-02



PERMEABILITY @ 20 °C = 2.7×10^{-2} cm/sec

CONSOLIDATED UNDRAINED TRIAXIAL TEST

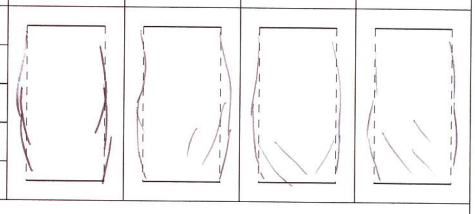




Symbol		⌀	△	□	◇
Sample No.		A-1-A	A-1-A	A-1-A	A-1-A
Test No.		CU21	CU22	CU23	CU24
Depth		---	---	---	---
Initial	Diameter, in	6	6	6	6
	Height, in	12	12	12	12
	Water Content, %	5.8	5.9	5.8	5.8
	Dry Density, pcf	131.3	131.2	131.3	131.3
	Saturation, %	55.4	56.3	55.6	55.4
	Void Ratio	0.283	0.285	0.284	0.283
Before Shear	Water Content, %	9.6	9.2	8.6	10.2
	Dry Density, pcf	133.9	134.9	136.8	132.1
	Saturation*, %	100.0	100.0	100.0	100.0
	Void Ratio	0.259	0.249	0.232	0.276
	Back Press., psi	102	102.7	102.2	102.6
Ver. Eff. Cons. Stress, psi		1.724	4.818	9.979	19.98
Shear Strength, psi		40.41	50.06	54.3	60.75
Strain at Failure, %		15.6	15	16.5	14.9
Strain Rate, %/min		0.05	0.05	0.05	0.05
B-Value		0.95	0.95	0.95	0.95
Estimated Specific Gravity		2.7	2.7	2.7	2.7
Liquid Limit		---	---	---	---
Plastic Limit		---	---	---	---

GeoTesting express

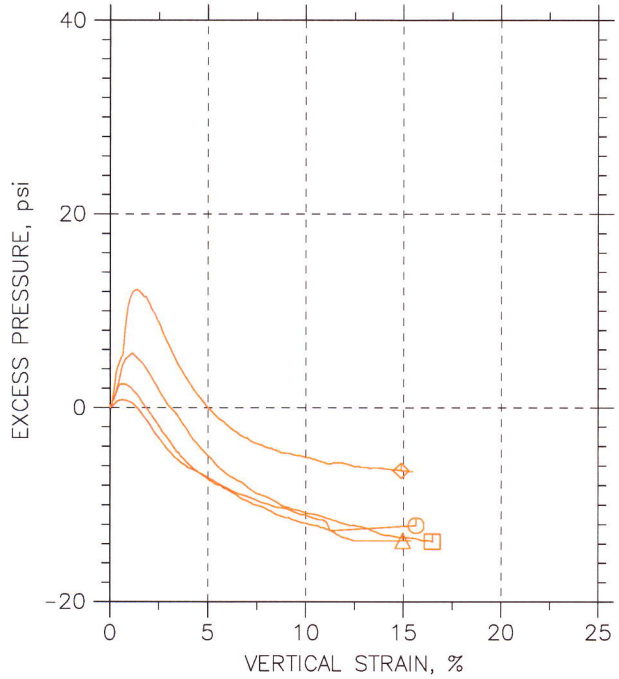
Project: Geocomp Consulting
Location: ---
Project No.: GTX-5504
Boring No.: 4:1 mix
Sample Type: Compacted
Description: Moist, gray silty gravel with sand
Remarks: Specimens compacted to 95% of Max Dry Density (138 pcf) at the Opt. Moisture (6.0%).

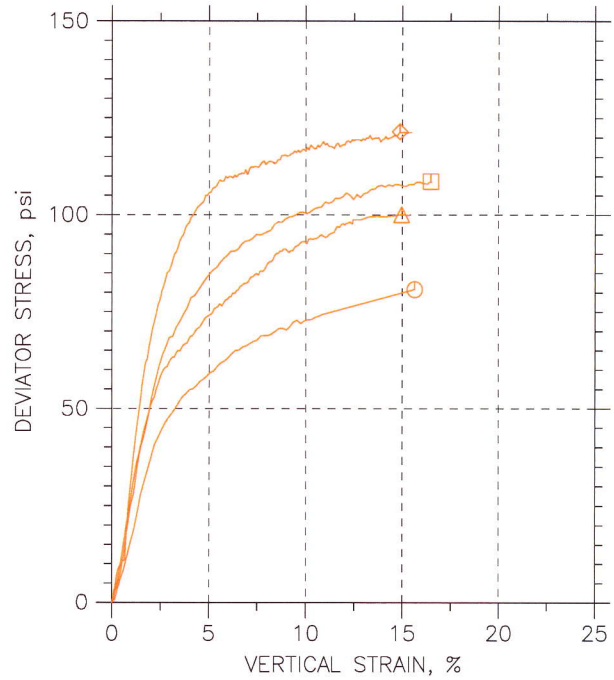


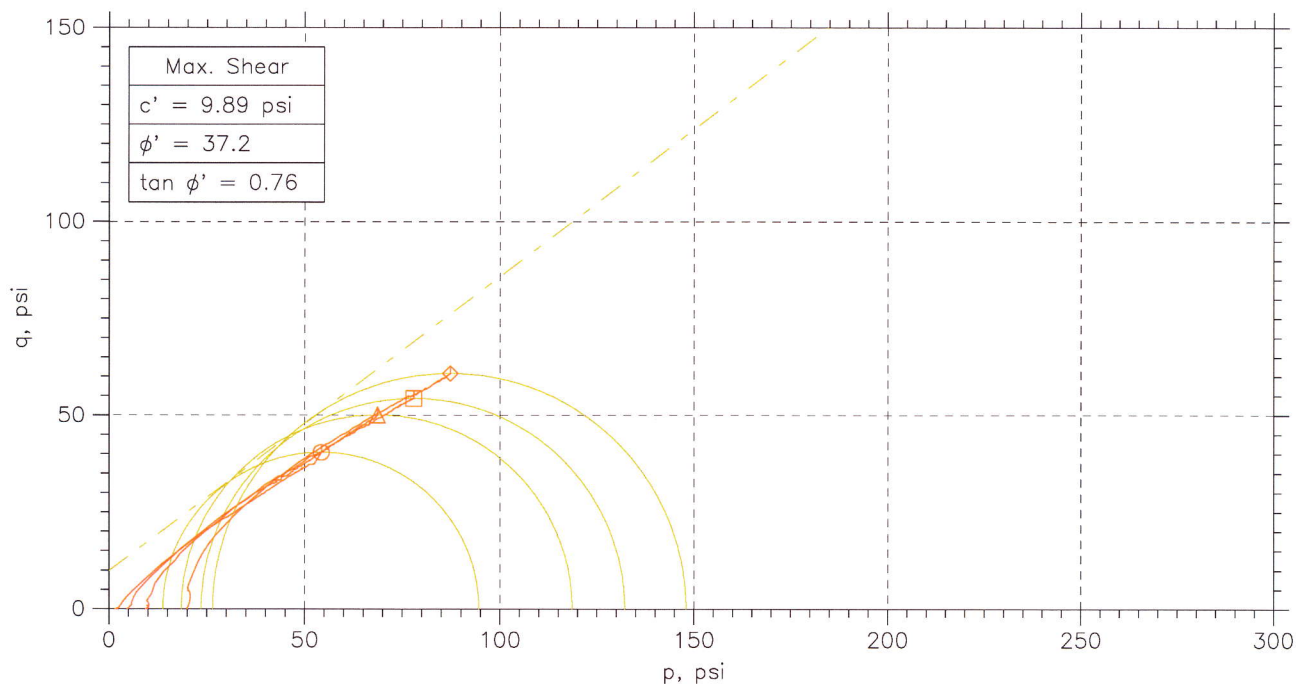
Thu, 07-JUL-2005 14:34:15

Phase calculations based on start and end of test.
* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST







	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⊙	A-1-A	CU21	---	fy	06/23/05	jdt		5504cu21d.dat
△	A-1-A	CU22	---	fy	06/28/05	jdt		5504cu22b.dat
□	A-1-A	CU23	---	fy	06/29/05	jdt		5504cu23b.dat
◇	A-1-A	CU24	---	fy	07/01/05	jdt		5504cu24d.dat

GeoTesting express

Project: Geocomp Consulting	Location: ---	Project No.: GTX-5504
Boring No.: 4:1 mix	Sample Type: Compacted	
Description: Moist, gray silty gravel with sand		
Remarks: Specimens compacted to 95% of Max Dry Density (138 pcf) at the Opt. Moisture (6.0%).		

Thu, 07-JUL-2005 14:34:46

Client:	Geocomp Consulting		
Project Name:	NCHRP 24-22		
Project Location:	Lunenburg, MA		
GTX #:	5504		
Start Date:	06/20/05	Tested By:	bdf/rmt
End Date:	06/21/05	Checked By:	jdt
Soil ID:	Proposed A-1-a (4:1 mix)		
Soil Description:	Moist, gray silty gravel with sand		

Direct Shear Test Series by ASTM D 3080

Soil Preparation:	Test points compacted to 95% of the Maximum Dry Density at the Optimum Moisture Content. All material greater than 1-inch (<1%) screened out of sample.		
Compaction Characteristics:	Corrected Maximum Dry Density	138.0	pcf
	Corrected Optimum Moisture Content	6.0	%
	Compaction Test Method	ASTM D 1557	
Test Equipment:	Top box = 12 in x 12 in; Bottom box = 16 in x 12 in; Load cells and LVDTs connected to data acquisition system for shear force, normal load and horizontal displacement readings; Flat plate clamping device; surface area = 144 in^2		
Horizontal Displacement, in/min:	0.02	Test Condition:	inundated

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6
Initial Moisture Content, %	6	6	6	---	---	---
Initial Dry Density, pcf	130	130	130	---	---	---
Percent Compaction, %	94.5	94.5	94.5	---	---	---
Normal Compressive Stress, psf	600	1200	2400	---	---	---
Peak Shear Stress, psf	1040	1892	3230	---	---	---
Post Peak Shear Stress, psf	---	---	---	---	---	---
Final Moisture Content, %	11	10	10	---	---	---

NOTES:
Points 1,2,3: Gap between boxes = ~1.0 inch
Latex membrane place along sides of sample to prevent soil from spilling out of the boxes.

Peak Friction Angle:	**50**	**degrees**
Peak Cohesion:	**371**	**psf**
Post Peak Friction Angle:	**---**	**degrees**
Post Peak Cohesion:	**---**	**psf**

Figure a. Shear Force vs. Horizontal Displacement

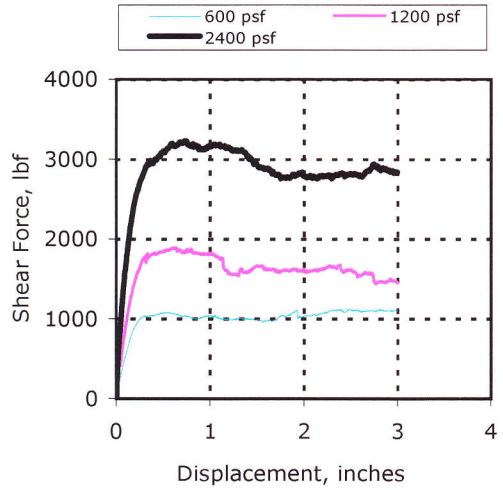
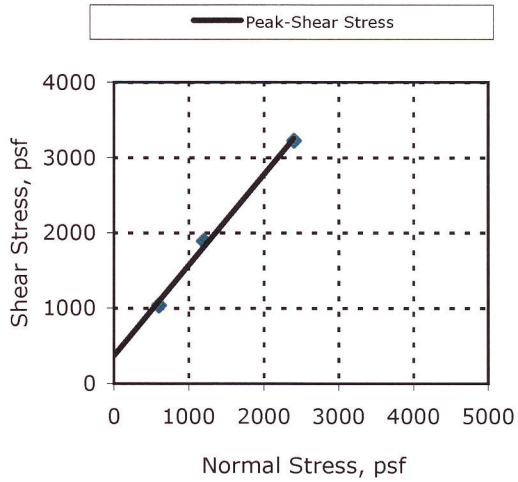


Figure b. Shear Stress vs. Normal Stress



GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	---	Sample Type:	---	Tested By:	njh
Sample ID:	---	Test Date:	10/17/05	Checked By:	jdt
Depth :	---	Sample Id:	---		

Moisture Content of Soil - ASTM D 2216

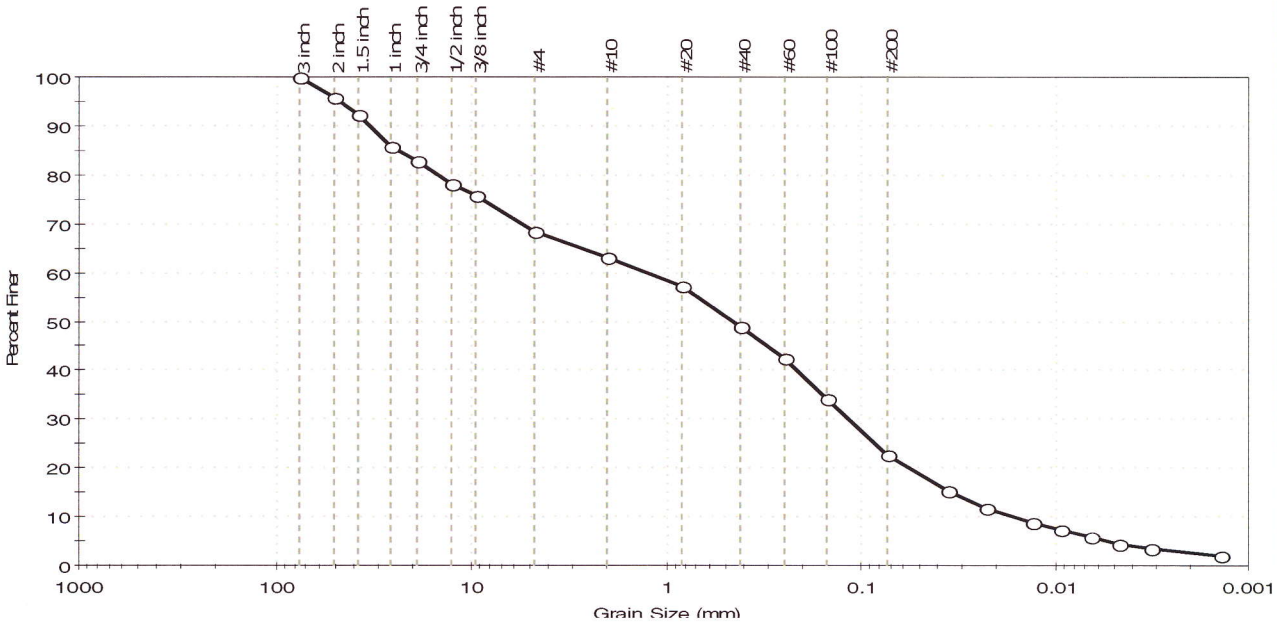
Boring ID	Sample ID	Depth	Description	Moisture Content,%
Proposed A-4	Keating #1 Washings #3	---	Moist, dark grayish brown sandy silt	23
Proposed A-4	Keating #1 Washings #4	---	Moist, olive gray sandy silt	22
Proposed A-2-4	Glacial Till #7	---	Moist, light olive brown silty sand with gravel	10
Proposed A-2-4	Glacial Till #8	---	Moist, light olive brown silty sand with gravel	6
Proposed A-2-4	Glacial Till #9	---	Moist, olive gray silty sand with gravel	5
Proposed A-2-4	Glacial Till #10	---	Moist, light olive brown silty sand with gravel	7

Notes: Temperature of Drying : 110° Celsius

GeoTesting express

Client:	Geocomp Consulting		
Project:	NCHRP 24-22		
Location:	Lunenburg MA	Project No:	GTX-5504
Boring ID: Proposed A-2-4	Sample Type: bag	Tested By:	njh
Sample ID: Glacial Till #7-#10	Test Date: 04/15/05	Checked By:	jdt
Depth : ---	Test Id: 68255		
Test Comment: ---			
Sample Description: Moist, olive silty sand with gravel			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	31.6	45.6	22.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3 inch	76.10	100		
2 inch	50.80	96		
1.5 inch	38.10	92		
1 inch	25.70	86		
3/4 inch	19.00	83		
1/2 inch	12.70	78		
3/8 inch	9.51	76		
#4	4.75	68		
#10	2.00	63		
#20	0.84	57		
#40	0.42	49		
#60	0.25	42		
#100	0.15	34		
#200	0.074	23		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0354	15		
---	0.0227	12		
---	0.0131	9		
---	0.0094	7		
---	0.0066	6		
---	0.0047	5		
---	0.0032	4		
---	0.0014	2		

Coefficients

D_{85} = 23.5999 mm	D_{30} = 0.1157 mm
D_{60} = 1.2646 mm	D_{15} = 0.0341 mm
D_{50} = 0.4613 mm	D_{10} = 0.0163 mm
C_u = 77.583	C_c = 0.649

Classification

ASTM Silty sand with gravel (SM)

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

GeoTesting express

Client:	Geocomp Consulting		
Project:	NCHRP 24-22		
Location:	Lunenburg MA	Project No:	GTX-5504
Boring ID: Proposed A-2-4	Sample Type: bag	Tested By:	njh
Sample ID: Glacial Till #7-#10	Test Date: 04/18/05	Checked By:	jdt
Depth : ---	Test Id: 68275		
Test Comment: ---			
Sample Description: Moist, olive silty sand with gravel			
Sample Comment: ---			

Atterberg Limits - ASTM D 4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	Glacial Till #7-#10	Proposed A-2-4	---	8	n/a	n/a	n/a	n/a	Silty sand with gravel (SM)

51% Retained on #40 Sieve

Dry Strength: LOW

Dilentancy: RAPID

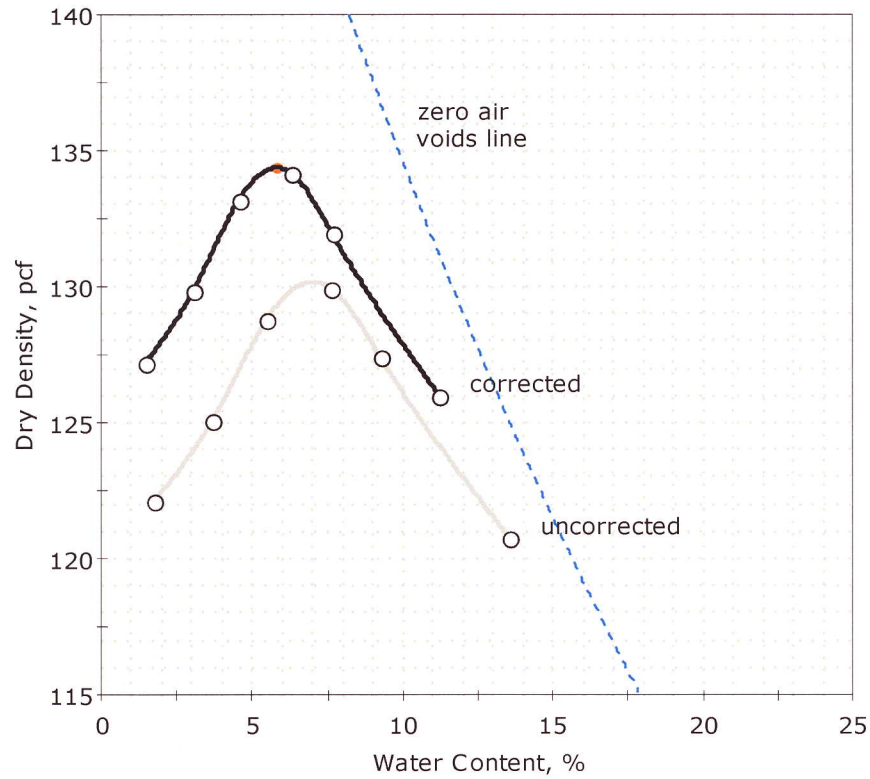
Toughness: n/a

The sample was determined to be Non-Plastic

GeoTesting express

Client:	Geocomp Consulting		
Project:	NCHRP 24-22		
Location:	Lunenburg MA	Project No:	GTX-5504
Boring ID: Proposed A-2-4	Sample Type: bag	Tested By:	jdt
Sample ID:Glacial Till #7-#10	Test Date: 04/19/05	Checked By:	jdt
Depth : ---	Test Id: 68267		
Test Comment:	---		
Sample Description:	Moist, olive silty sand with gravel		
Sample Comment:	---		

Compaction Report - ASTM D 698



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6
Dry density, pcf	122.1	125.1	128.8	129.9	127.4	120.8
Moisture Content, %	1.7	3.7	5.5	7.6	9.2	13.5

Method : C

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.75

Maximum Dry Density= 130.0 pcf
Optimum Moisture= 7.0 %

Oversize Correction (17.1% > 3/4 inch Sieve)
Corrected Maximum Dry Density= 134.5 pcf
Corrected Optimum Moisture= 6.0 %
Assumed Average Bulk Specific Gravity = 2.55

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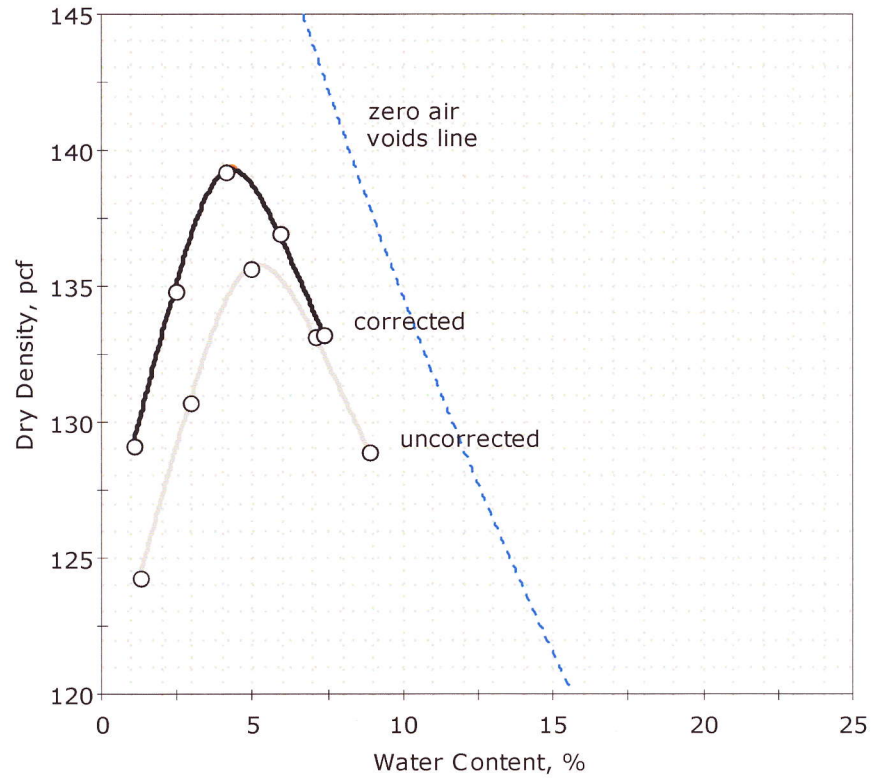
GeoTesting express

Client:	Geocomp Consulting		
Project:	NCHRP 24-22		
Location:	Lunenburg MA	Project No:	GTX-5504
Boring ID: Proposed A-2-4	Sample Type: bag	Tested By:	jdt
Sample ID: Glacial Till #7-#10	Test Date: 04/18/05	Checked By:	jdt
Depth : ---	Test Id: 68263		
Test Comment:	---		
Sample Description:	Moist, olive silty sand with gravel		
Sample Comment:	---		

Compaction Report - ASTM D 1557



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5
Dry density, pcf	124.3	130.7	135.7	133.2	128.9
Moisture Content, %	1.2	2.9	4.9	7.1	8.8

Method : C

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.75

Maximum Dry Density= 136.0 pcf
Optimum Moisture= 5.0 %

Oversize Correction (17.1% > 3/4 inch Sieve)
Corrected Maximum Dry Density= 139.5 pcf
Corrected Optimum Moisture= 4.5 %
Assumed Average Bulk Specific Gravity = 2.55

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GeoTesting express

Client:	Geocomp Consulting		
Project Name:	NCHRP 24-22		
Project Location:	Lunenburg, MA		
GTX #:	5504		
Start Date:	4/20/2005	Tested By:	rmt
End Date:	4/22/2005	Checked By:	jdt
Boring #:	Proposed A-2-4		
Sample #:	Glacial Till #7 - #10		
Depth:	---		
Visual Description:	Moist, olive silty sand with gravel		

Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter by ASTM D 5084

Constant Gradient

Sample Type:	remolded	Permeant Fluid:	de-aired tap water
Orientation:	Vertical	Cell #:	---

Sample Preparation: Compacted to 95% of Maximum Dry Density at Optimum Moisture Content.
Maximum Dry Density = 130.0 pcf; Optimum Moisture Content = 7.0% (ASTM D 698).
Trimmings moisture content = 7.2%.

Parameter	Initial	Final
Height, in	2.00	2.02
Diameter, in	2.87	2.91
Area, in^2	6.47	6.65
Volume, in^3	12.9	13.4
Mass, g	448	480
Bulk Density, pcf	132	136
Moisture Content, %	7.2	15
Dry Density, pcf	123	118
Degree of Saturation, %	---	99

B COEFFICIENT DETERMINATION

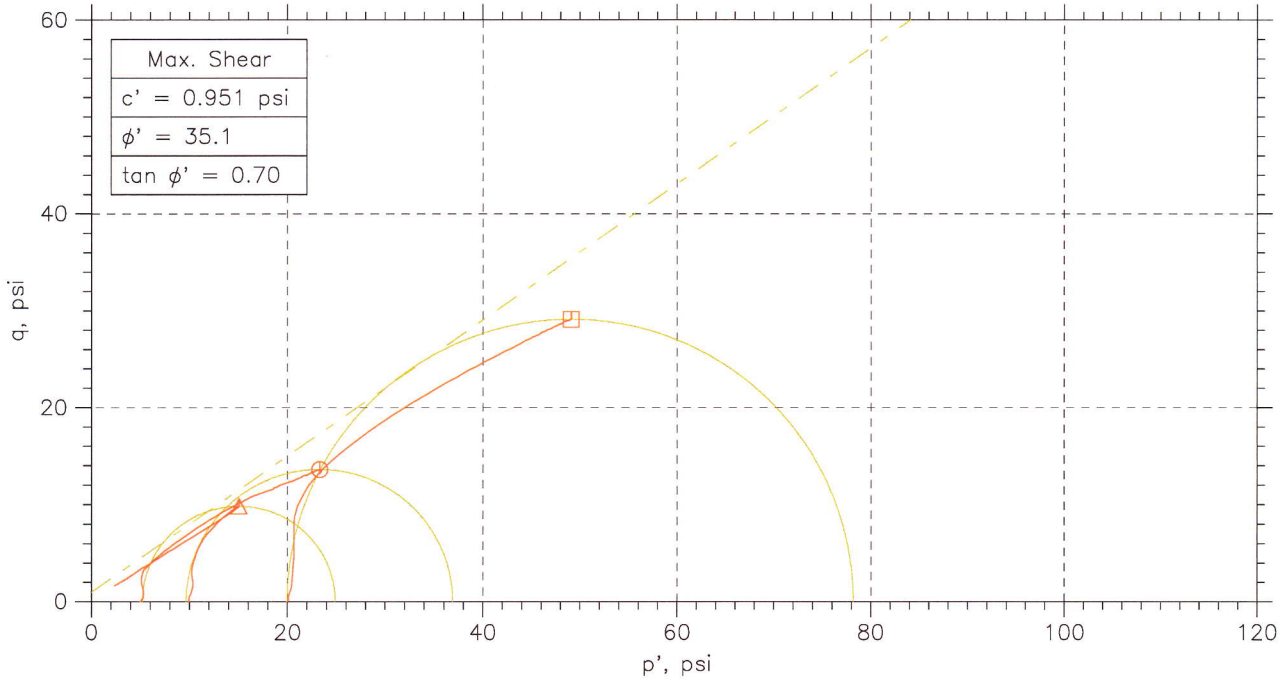
Cell Pressure, psi:	95	Pressure Increment, psi:	4.9
Sample Pressure, psi:	89.8	B Coefficient:	0.98

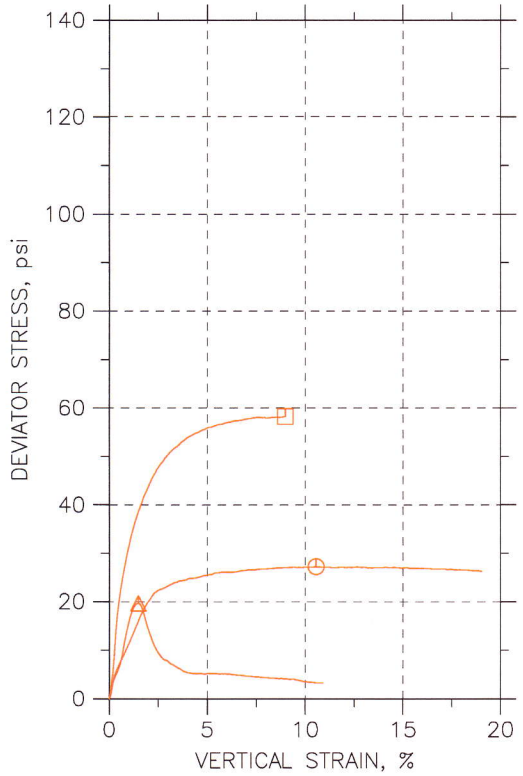
FLOW DATA

Date	Time, sec	Pressure, psi			Gradient	Flow Volume, cc				Temp, °C	R_t	Permeability K @ 20 °C, cm/sec
		Cell	Inlet	Outlet		In	Out	Δ In	Δ Out			
04/22	---	90.0	86.0	84.0	27.4	1.45	23.65	---	---	---	---	---
04/22	70	90.0	86.0	84.0	27.4	9.60	15.50	8.15	8.15	20	1.000	9.9E-05
04/22	---	90.0	86.0	84.0	27.4	9.60	15.50	---	---	---	---	---
04/22	34	90.0	86.0	84.0	27.4	13.50	11.55	3.90	3.95	20	1.000	9.8E-05
04/22	----	90.0	86.0	84.0	27.4	13.50	11.55	---	---	---	---	---
04/22	27	90.0	86.0	84.0	27.4	16.45	8.70	2.95	2.85	20	1.000	9.1E-05
04/22	----	90.0	86.0	84.0	27.4	16.45	8.70	---	---	---	---	---
04/22	75	90.0	86.0	84.0	27.4	23.80	1.30	7.35	7.40	20	1.000	8.4E-05

PERMEABILITY AT 20° C: 9.3 x 10^{-5} cm/sec (@ 5 psi effective stress)

CONSOLIDATED UNDRAINED TRIAXIAL TEST





Symbol		◷	△	⊡	
Sample No.		Till	Till	Till	
Test No.		CU202	CU204	CU200	
Depth		---	---	---	
Initial	Diameter, in	6	6	6	
	Height, in	12	12	12	
	Water Content, %	6.0	6.1	6.5	
	Dry Density, pcf	127.7	127.7	127.2	
	Saturation, %	51.1	51.2	53.8	
	Void Ratio	0.32	0.32	0.325	
Before Shear	Water Content, %	12.0	13.4	12.3	
	Dry Density, pcf	127.3	123.9	126.6	
	Saturation*, %	100.0	100.0	100.0	
	Void Ratio	0.324	0.361	0.332	
	Back Press., psi	118.	47.02	122.	
Ver. Eff. Cons. Stress, psi		4.994	9.892	19.96	
Shear Strength, psi		13.62	9.829	29.11	
Strain at Failure, %		10.6	1.47	8.98	
Strain Rate, %/min		0.04	0.04	0.04	
B-Value		0.71	0.71	0.71	
Estimated Specific Gravity		2.7	2.7	2.7	
Liquid Limit		NP	NP	NP	
Plastic Limit		NP	NP	NP	

GeoTesting express

Project: NCHRP

Location: ---

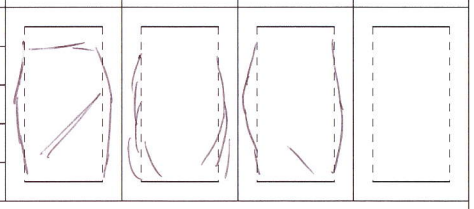
Project No.: GTX-5504

Boring No.: GlacialTill

Sample Type: Remolded

Description: Moist, olive silty sand with gravel

Remarks: ---



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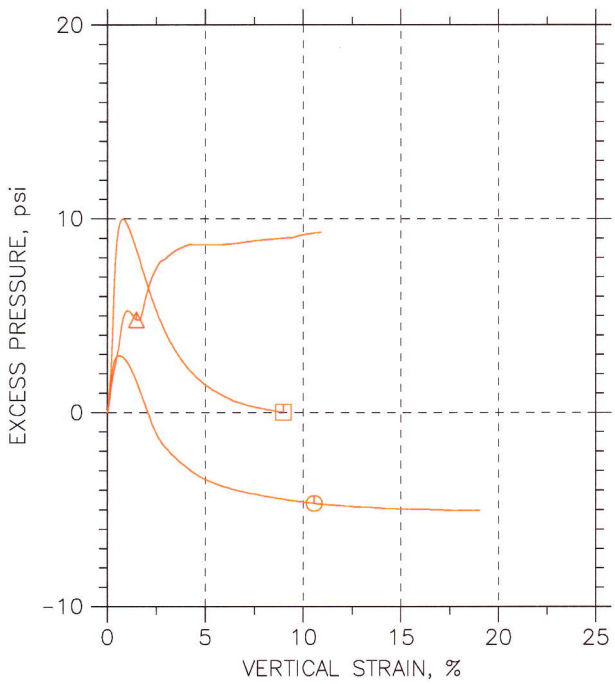
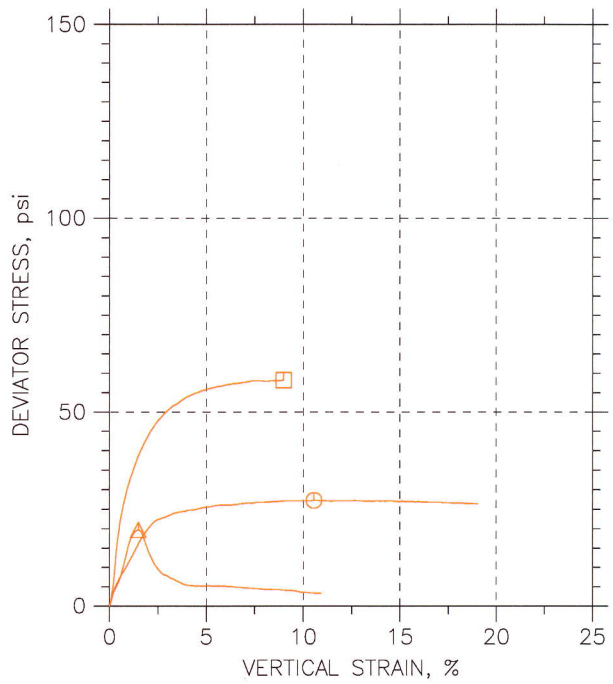
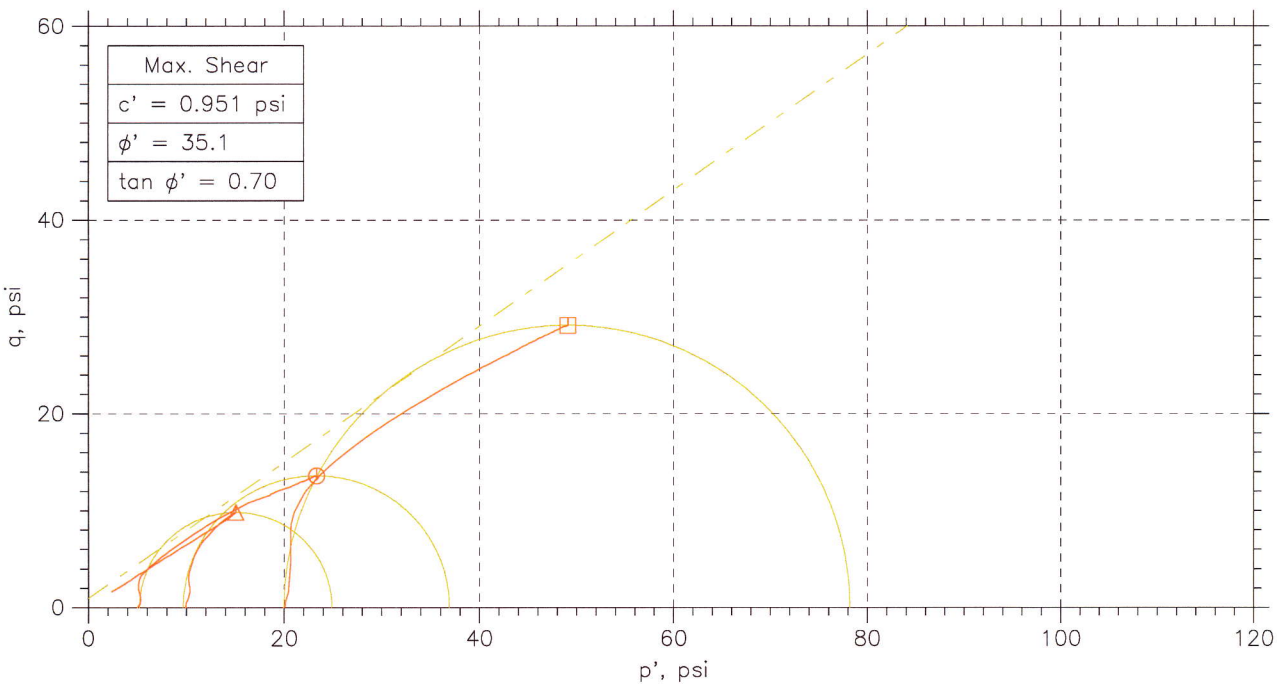
Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST







	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⊙	Till	CU202	---	njh	5/24/5	jdt		5504-cu202c.dat
△	Till	CU204	---	njh	6/1/5	jdt		5504-cu204a.dat
⊡	Till	CU200	---	njh	05/16/05	jdt		5504-cu200e 4.dat

GeoTesting express

Project: NCHRP	Location: ---	Project No.: GTX-5504
Boring No.: GlacialTill	Sample Type: Remolded	
Description: Moist, olive silty sand with gravel		
Remarks: ---		

Wed, 08-JUN-2005 08:49:36

Client:	Geocomp Consulting		
Project Name:	NCHRP 24-22		
Project Location:	Lunenburg, MA		
GTX #:	5504		
Start Date:	05/16/05	Tested By:	bdf/rmt
End Date:	05/18/05	Checked By:	jdt
Soil ID:	Glacial Till #7 - #10 (Proposed A-2-4)		
Soil Description:	Moist, olive silty sand with gravel		

Direct Shear Test Series by ASTM D 3080

Soil Preparation:	All material greater than 1-inch (14%) screened out of sample and replaced with No. 4 sieve to 1-inch material. Test points compacted to 95% of the Corrected Maximum Dry Density at the Corrected Optimum Moisture Content.		
Compaction Characteristics:	Corrected Maximum Dry Density	134.5	pcf
	Corrected Optimum Moisture Content	6.0	%
	Compaction Test Method	ASTM D 698	
Test Equipment:	Top box = 12 in x 12 in; Bottom box = 16 in x 12 in; Load cells and LVDTs connected to data acquisition system for shear force, normal load and horizontal displacement readings; Flat plate clamping device; surface area = 144 in^2		
Horizontal Displacement, in/min:	0.02	Test Condition:	inundated

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6
Initial Moisture Content, %	6	6	6	---	---	---
Initial Dry Density, pcf	127	127	127	---	---	---
Percent Compaction, %	94.8	94.8	94.8	---	---	---
Normal Compressive Stress, psf	600	1200	2400	---	---	---
Peak Shear Stress, psf	433	910	1618	---	---	---
Post Peak Shear Stress, psf	---	---	---	---	---	---
Final Moisture Content, %	10	10	10	---	---	---

NOTES:
Points 1,2,3: Gap between boxes = ~1.0 inch
Latex membrane place along sides of sample to prevent soil from spilling out of the boxes.

Peak Friction Angle:	**33**	**degrees**
Peak Cohesion:	**79**	**psf**
Post Peak Friction Angle:	**---**	**degrees**
Post Peak Cohesion:	**---**	**psf**

Figure a. Shear Force vs. Horizontal Displacement

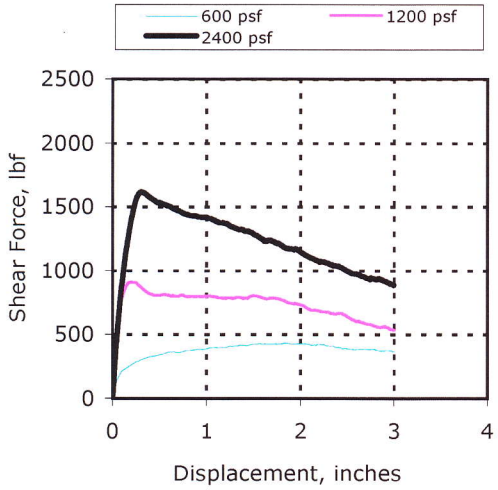
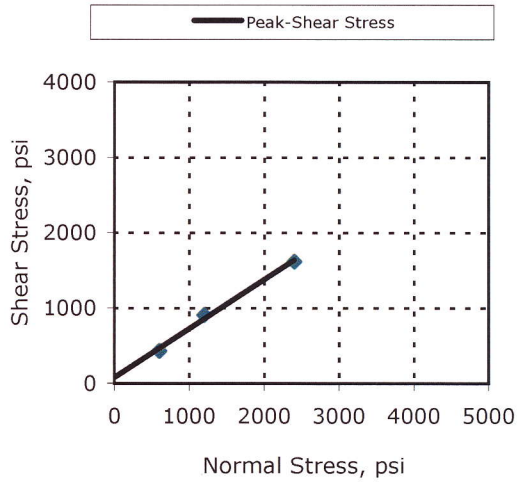


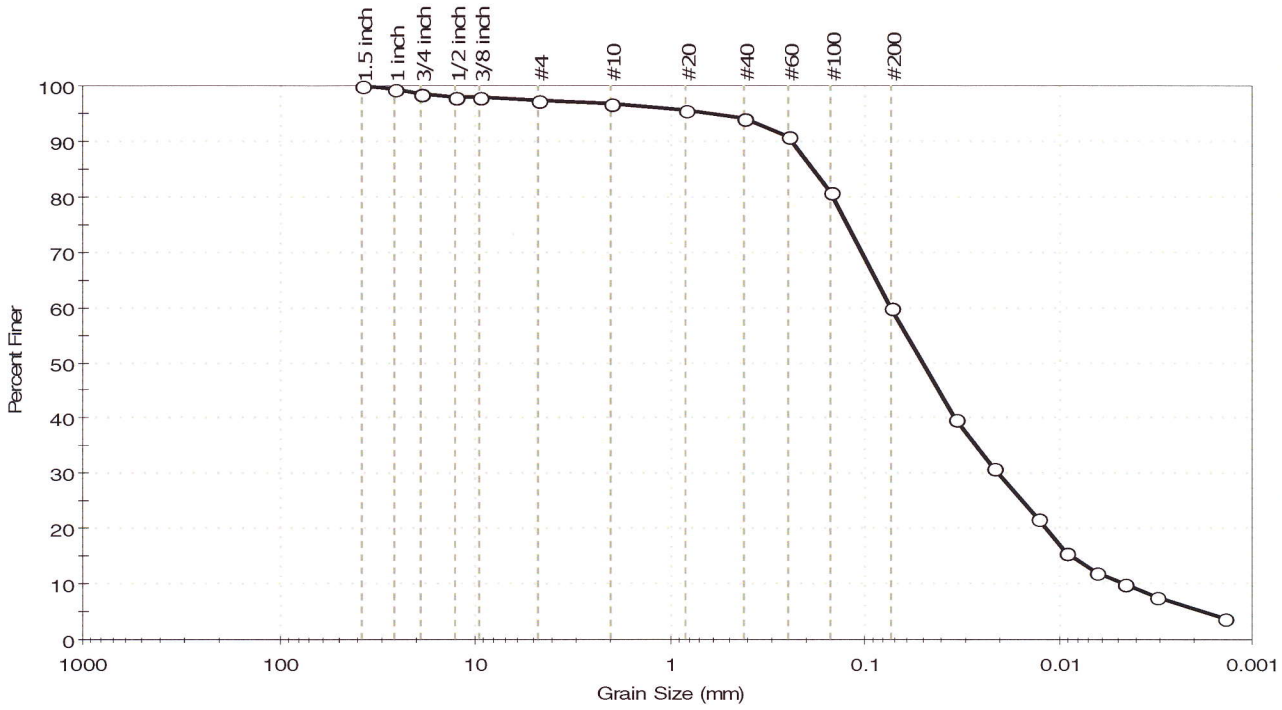
Figure b. Shear Stress vs. Normal Stress



GeoTesting express

Client:	Geocomp Consulting		
Project:	NCHRP 24-22		
Location:	Lunenburg MA	Project No:	GTX-5504
Boring ID: Proposed A-4	Sample Type: bucket	Tested By:	njh
Sample ID: Keating #1 Washings #3	Test Date: 04/14/05	Checked By:	jdt
Depth : ---	Test Id: 68253		
Test Comment:	---		
Sample Description:	Moist, dark grayish brown sandy silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	2.7	37.4	59.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 inch	38.10	100		
1 inch	25.70	99		
3/4 inch	19.00	99		
1/2 inch	12.70	98		
3/8 inch	9.51	98		
#4	4.75	97		
#10	2.00	97		
#20	0.84	96		
#40	0.42	94		
#60	0.25	91		
#100	0.15	81		
#200	0.074	60		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0338	40		
---	0.0217	31		
---	0.0129	22		
---	0.0093	16		
---	0.0065	12		
---	0.0046	10		
---	0.0031	8		
---	0.0014	4		

Coefficients

D_{85} = 0.1844 mm D_{30} = 0.0206 mm
D_{60} = 0.0744 mm D_{15} = 0.0087 mm
D_{50} = 0.0503 mm D_{10} = 0.0047 mm
C_u = 15.830 C_c = 1.214

Classification

ASTM Sandy silt (ML)

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Proposed A-4	Sample Type:	bucket	Tested By:	njh
Sample ID:	Keating #1 Washings #3	Test Date:	04/15/05	Checked By:	jdt
Depth :	---	Test Id:	68273		
Test Comment:	---				
Sample Description:	Moist, dark grayish brown sandy silt				
Sample Comment:	---				

Atterberg Limits - ASTM D 4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	Keating #1 Washings #3	Proposed A-4	---	23	n/a	n/a	n/a	n/a	Sandy silt (ML)

6% Retained on #40 Sieve

Dry Strength: LOW

Dilentancy: RAPID

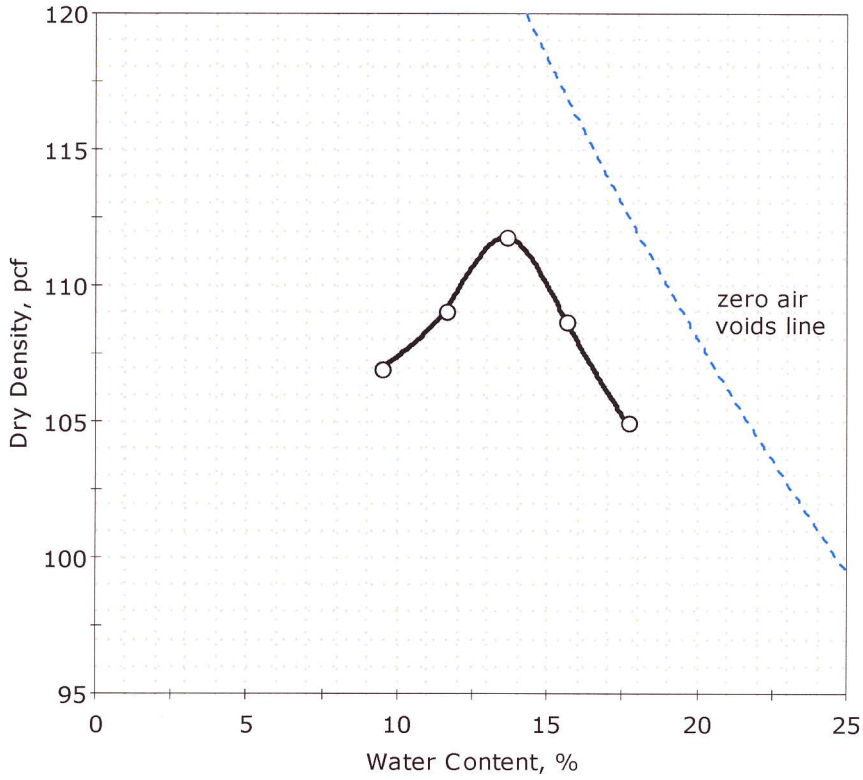
Toughness: n/a

The sample was determined to be Non-Plastic

GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Proposed A-4	Sample Type:	bucket	Tested By:	jdt
Sample ID:	Keating #1 Washings #3	Test Date:	04/15/05	Checked By:	jdt
Depth :	---	Test Id:	68264		
Test Comment:	---				
Sample Description:	Moist, dark grayish brown sandy silt				
Sample Comment:	---				

Compaction Report - ASTM D 698



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5
Dry density, pcf	107.0	109.1	111.8	108.7	105.0
Moisture Content, %	9.5	11.6	13.6	15.6	17.7

Method : A

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.65

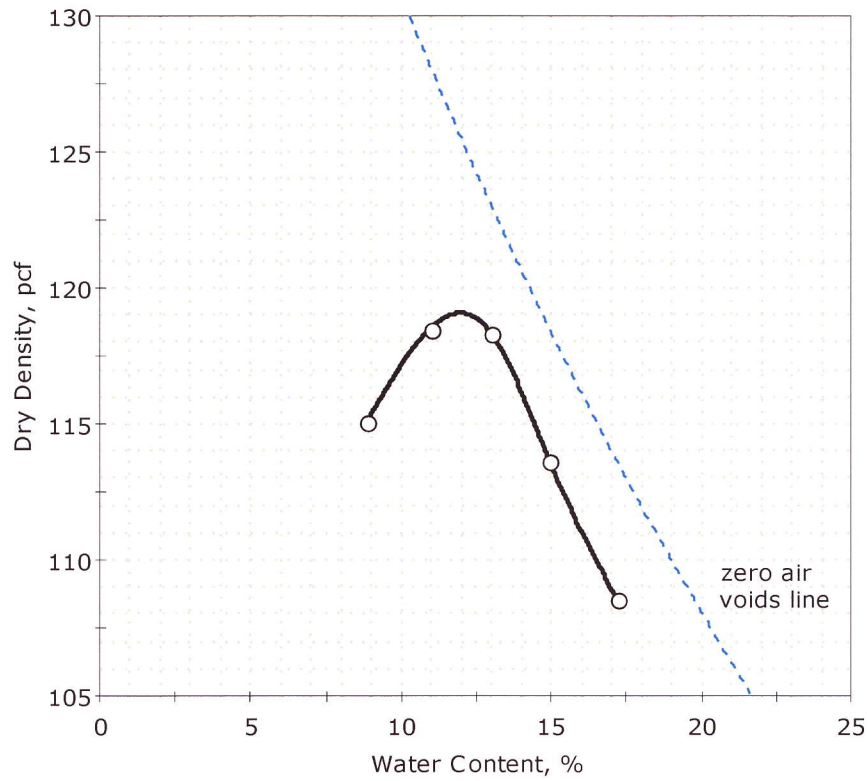
Maximum Dry Density= 112.0 pcf

Optimum Moisture= 13.5 %

GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Proposed A-4	Sample Type:	bucket	Tested By:	jdt
Sample ID:	Keating #1 Washings #3	Test Date:	04/15/05	Checked By:	jdt
Depth :	---	Test Id:	68258		
Test Comment:	---				
Sample Description:	Moist, dark grayish brown sandy silt				
Sample Comment:	---				

Compaction Report - ASTM D 1557



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5
Dry density, pcf	115.0	118.5	118.3	113.6	108.6
Moisture Content, %	8.8	11.0	13.0	14.9	17.2

Method : A

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.65

Maximum Dry Density= 119.0 pcf

Optimum Moisture= 12.0 %

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GeoTesting express

Client:	Geocomp Consulting		
Project Name:	NCHRP 24-22		
Project Location:	Lunenburg, MA		
GTX #:	5504		
Start Date:	4/15/2005	Tested By:	rmt
End Date:	4/22/2005	Checked By:	jdt
Boring #:	Proposed A-4		
Sample #:	Keating #1 Washings #3		
Depth:	---		
Visual Description:	Moist, dark grayish brown sandy silt		

Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter by ASTM D 5084

Constant Gradient

Sample Type:	remolded	Permeant Fluid:	de-aired tap water
Orientation:	Vertical	Cell #:	---

Sample Preparation: Compacted to 95% of Maximum Dry Density at Optimum Moisture Content.
Maximum Dry Density = 112.0 pcf; Optimum Moisture Content = 13.5% (ASTM D 698).
Trimmings moisture content = 13.5%.

Parameter	Initial	Final
Height, in	2.00	1.98
Diameter, in	2.87	2.90
Area, in^2	6.47	6.61
Volume, in^3	12.9	13.1
Mass, g	410	437
Bulk Density, pcf	120	127
Moisture Content, %	14	21
Dry Density, pcf	106	105
Degree of Saturation, %	---	97

B COEFFICIENT DETERMINATION

Cell Pressure, psi:	95	Pressure Increment, psi:	4.9
Sample Pressure, psi:	89.8	B Coefficient:	0.98

FLOW DATA

Date	Time, sec	Pressure, psi Cell	Inlet	Outlet	Gradient	Flow Volume, cc In	Out	Δ In	Δ Out	Temp, °C	R_t	Permeability K @ 20 °C, cm/sec
04/21	---	90.0	86.0	84.0	28.0	3.20	22.00	---	---	---	---	---
04/21	45	90.0	86.0	84.0	28.0	6.90	18.30	3.70	3.70	20	1.000	6.9E-05
04/21	---	90.0	86.0	84.0	28.0	6.90	18.30	---	---	---	---	---
04/21	52	90.0	86.0	84.0	28.0	10.85	14.30	3.95	4.00	20	1.000	6.4E-05
04/21	----	90.0	86.0	84.0	28.0	10.85	14.30	---	---	---	---	---
04/21	67	90.0	86.0	84.0	28.0	15.60	9.50	4.75	4.80	20	1.000	6.0E-05
04/21	----	90.0	86.0	84.0	28.0	15.60	9.50	---	---	---	---	---
04/21	105	90.0	86.0	84.0	28.0	22.20	2.90	6.60	6.60	20	1.000	5.3E-05

PERMEABILITY AT 20° C: 6.1 x 10^{-5} cm/sec (@ 5 psi effective stress)

GeoTesting express

Client:	Geocomp Consulting		
Project Name:	NCHRP 24-22		
Project Location:	Lunenburg, MA		
GTX #:	5504		
Start Date:	04/15/05	Tested By:	bdf/rmt
End Date:	04/18/05	Checked By:	jdt
Soil ID:	Keating #1 Washings #3 (Proposed A-4)		
Soil Description:	Moist, dark grayish brown sandy silt		

Direct Shear Test Series by ASTM D 3080

Soil Preparation: Compacted to 95% of the Maximum Dry Density at the Optimum Moisture Content.

Compaction Characteristics:

Maximum Dry Density	112.0	pcf
Optimum Moisture Content	12.5	%
Compaction Test Method	ASTM D 698	

Test Equipment: Top box = 12 in x 12 in; Bottom box = 16 in x 12 in; Load cells and LVDTs connected to data acquisition system for shear force, normal load and horizontal displacement readings; Flat plate clamping device; surface area = 144 in^2

Horizontal Displacement, in/min: 0.04 Test Condition: inundated

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6
Initial Moisture Content, %	14	14	14	---	---	---
Initial Dry Density, pcf	106	106	106	---	---	---
Percent Compaction, %	94.7	94.7	94.7	---	---	---
Normal Compressive Stress, psf	600	1200	2400	---	---	---
Peak Shear Stress, psf	495	1120	1962	---	---	---
Post Peak Shear Stress, psf	---	---	---	---	---	---
Final Moisture Content, %	21	22	22	---	---	---

NOTES:

Peak Friction Angle:	**39**	**degrees**
Peak Cohesion:	**74**	**psf**
Post Peak Friction Angle:	**---**	**degrees**
Post Peak Cohesion:	**---**	**psf**

Figure a. Shear Force vs. Horizontal Displacement

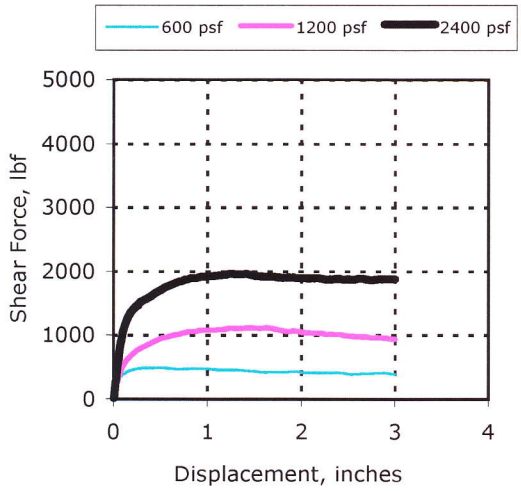
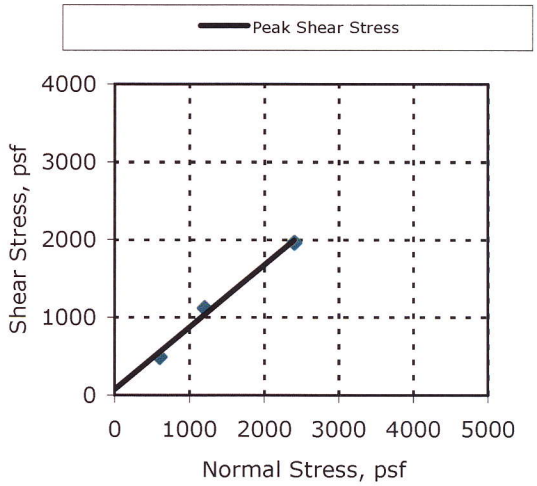
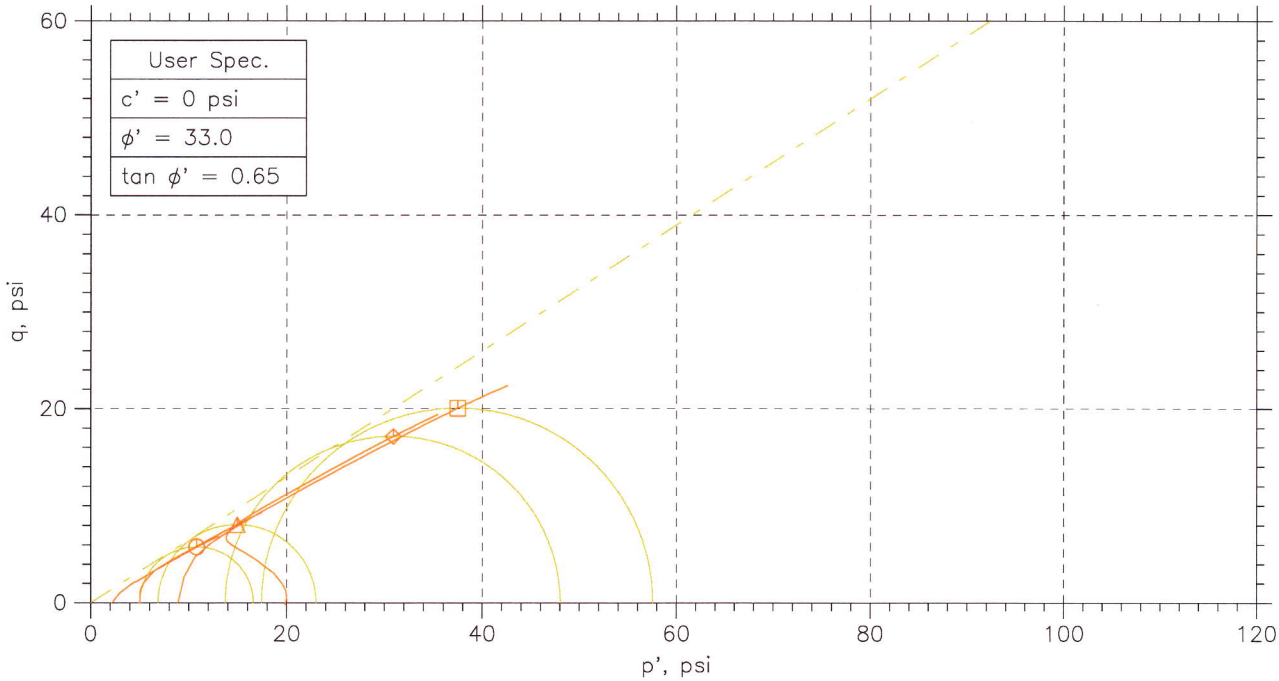
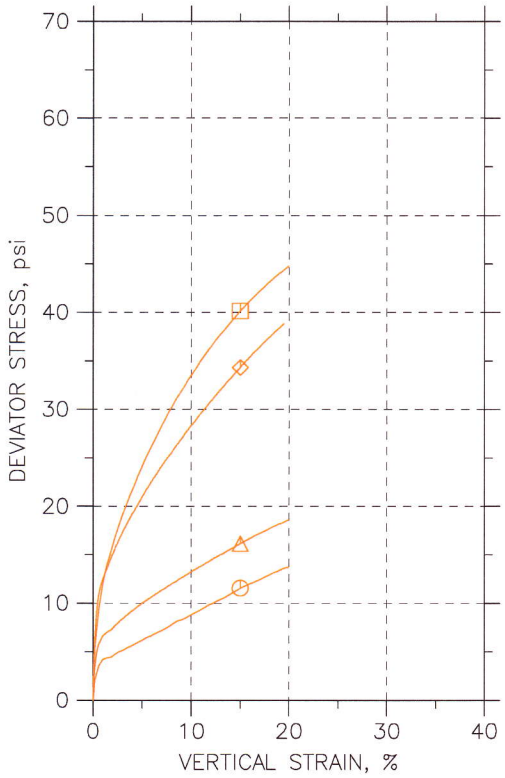


Figure b. Shear Stress vs. Normal Stress



CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⦶	△	⊡	◇
Sample No.		Keating	Keating	Keating	Keating
Test No.		CU9	CU10	CU11	CU12
Depth		---	---	---	---
Initial	Diameter, in	2.87	2.87	2.87	2.87
	Height, in	6	6	6	6
	Water Content, %	13.3	13.6	13.6	13.7
	Dry Density, pcf	106.6	106.4	106.4	106.3
	Saturation, %	61.8	62.7	62.7	63.1
	Void Ratio	0.58	0.585	0.584	0.586
Before Shear	Water Content, %	22.2	21.9	20.7	19.3
	Dry Density, pcf	105.4	106.	108.2	110.8
	Saturation*, %	100.0	100.0	100.0	100.0
	Void Ratio	0.599	0.59	0.558	0.522
	Back Press., psi	88.	109.	99.99	88.01
Ver. Eff. Cons. Stress, psi		2.483	4.999	10	20
Shear Strength, psi		5.773	8.065	20.07	17.15
Strain at Failure, %		15	15	15	15
Strain Rate, %/min		0.04	0.04	0.04	0.04
B-Value		0.70	0.78	0.70	0.71
Estimated Specific Gravity		2.7	2.7	2.7	2.7
Liquid Limit		NP	NP	NP	NP
Plastic Limit		NP	NP	NP	NP

GeoTesting express

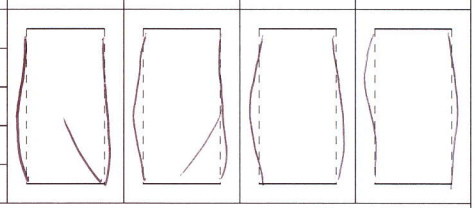
Project: NCHRP

Location: ---

Project No.: GTX-5504

Boring No.: Washing #3

Sample Type: Remolded



Description: Moist, dark grayish brown sandy silt

Remarks: Target Compaction: 95% of Max Dry Density (112.0 pcf) at Opt Moisture Content (13.5%).

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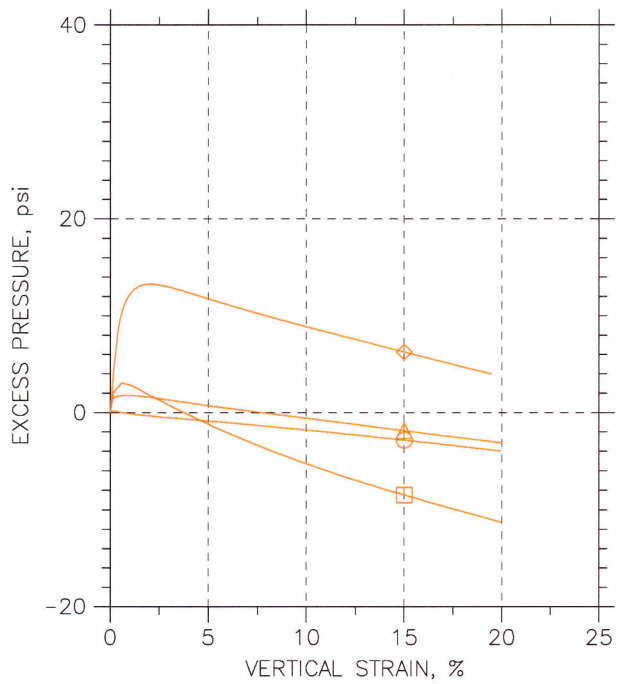
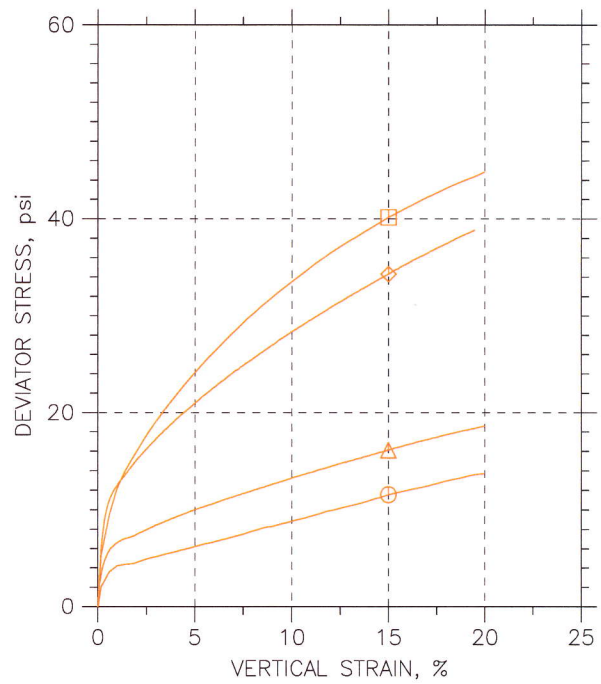
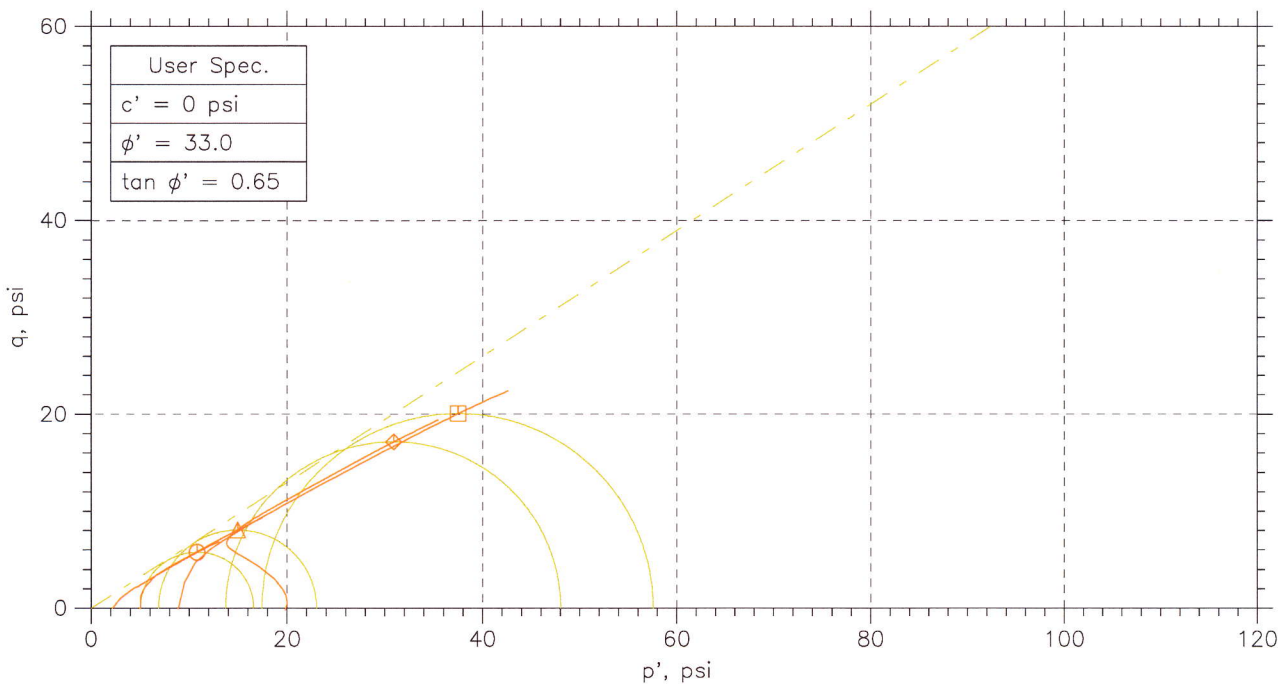
Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767







	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	Keating	CU9	---	fy	04/27/05	jdt		5504-CU9.dat
△	Keating	CU10	---	fy	04/27/05	jdt		5504-cu10.dat
□	Keating	CU11	---	fy	04/28/05	jdt		5504-CU11.dat
◇	Keating	CU12	---	njh	05/02/05	jdt		5504-cu12a.dat

GeoTesting express

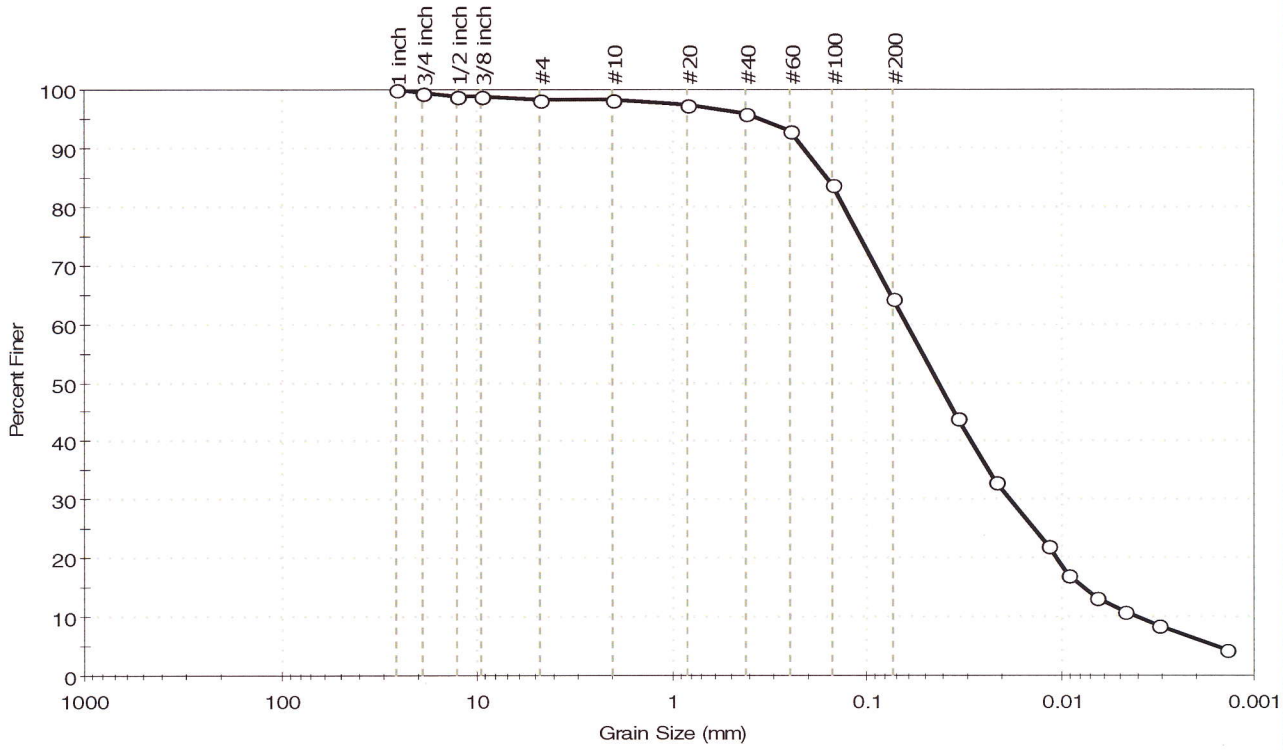
Project: NCHRP	Location: ---	Project No.: GTX-5504
Boring No.: Washing #3	Sample Type: Remolded	
Description: Moist, dark grayish brown sandy silt		
Remarks: Target Compaction: 95% of Max Dry Density (112.0 pcf) at Opt Moisture Content (13.5%).		

Wed, 04-MAY-2005 11:53:05

GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Proposed A-4	Sample Type:	bucket	Tested By:	njh
Sample ID:	Keating #1 Washings #4	Test Date:	04/14/05	Checked By:	jdt
Depth :	---	Test Id:	68254		
Test Comment:	---				
Sample Description:	Moist, olive gray sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	1.6	33.9	64.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.70	100		
3/4 inch	19.00	100		
1/2 inch	12.70	99		
3/8 inch	9.51	99		
#4	4.75	98		
#10	2.00	98		
#20	0.84	97		
#40	0.42	96		
#60	0.25	93		
#100	0.15	84		
#200	0.074	64		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0338	44		
---	0.0219	33		
---	0.0118	22		
---	0.0092	17		
---	0.0066	13		
---	0.0047	11		
---	0.0032	8		
---	0.0014	4		

Coefficients

D_{85} = 0.1600 mm D_{30} = 0.0185 mm

D_{60} = 0.0625 mm D_{15} = 0.0076 mm

D_{50} = 0.0428 mm D_{10} = 0.0041 mm

C_u = 15.244 C_c = 1.336

Classification

ASTM Sandy silt (ML)

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Proposed A-4	Sample Type:	bucket	Tested By:	njh
Sample ID:	Keating #1 Washings #4	Test Date:	04/15/05	Checked By:	jdt
Depth :	---	Test Id:	68274		
Test Comment:	---				
Sample Description:	Moist, olive gray sandy silt				
Sample Comment:	---				

Atterberg Limits - ASTM D 4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
*	Keating #1 Washings #4	Proposed A-4	---	22	n/a	n/a	n/a	n/a	Sandy silt (ML)

4% Retained on #40 Sieve

Dry Strength: LOW

Dilentancy: RAPID

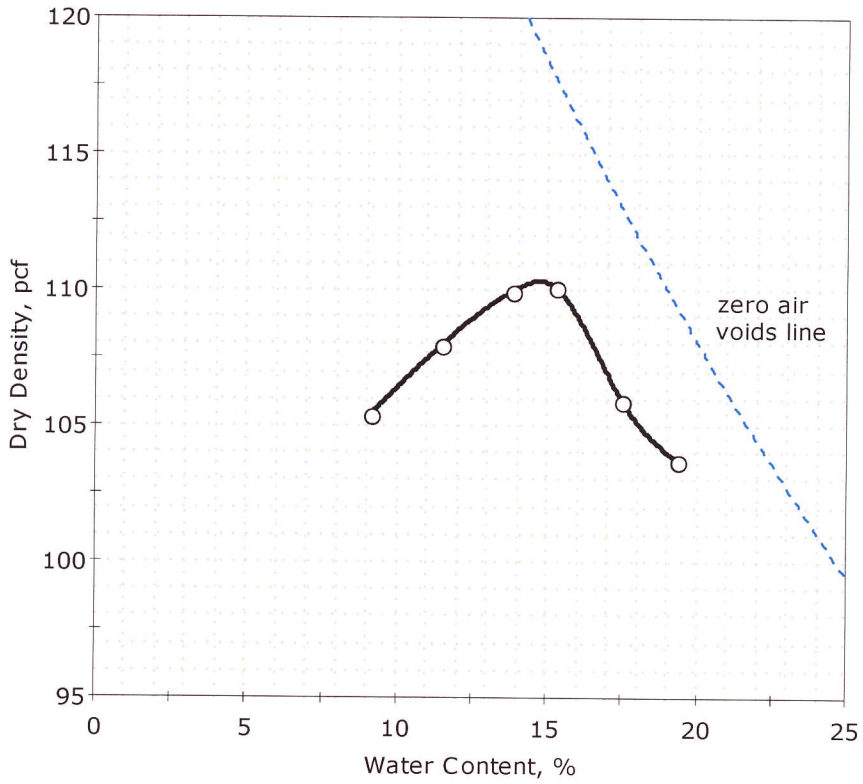
Toughness: n/a

The sample was determined to be Non-Plastic

GeoTesting express

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Proposed A-4	Sample Type:	bucket	Tested By:	jdt
Sample ID:	Keating #1 Washings #4	Test Date:	04/15/05	Checked By:	jdt
Depth :	---	Test Id:	68265		
Test Comment:	---				
Sample Description:	Moist, olive gray sandy silt				
Sample Comment:	---				

Compaction Report - ASTM D 698



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6
Dry density, pcf	105.4	108.0	109.9	110.1	105.9	103.7
Moisture Content, %	9.1	11.5	13.8	15.3	17.5	19.4

Method : A

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.65

Maximum Dry Density= 110.5 pcf

Optimum Moisture= 14.5 %

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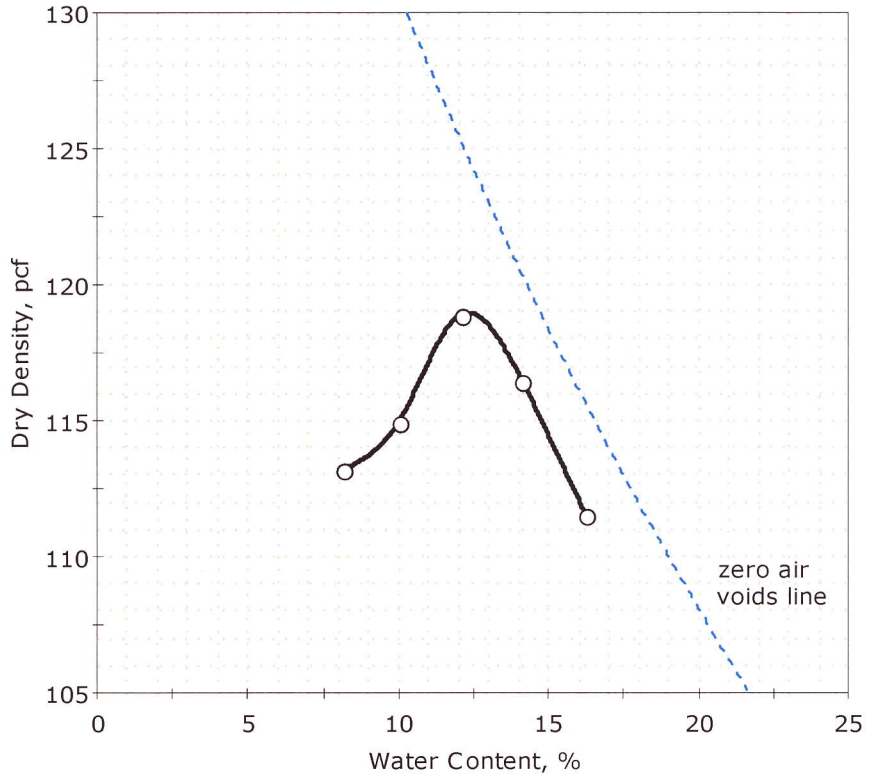
Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Proposed A-4	Sample Type:	bucket	Tested By:	jdt
Sample ID:	Keating #1 Washings #4	Test Date:	10/17/05	Checked By:	jdt
Depth :	---	Test Id:	68259		
Test Comment:	---				
Sample Description:	Moist, olive gray sandy silt				
Sample Comment:	---				

Compaction Report - ASTM D 1557



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5
Dry density, pcf	113.2	115.0	118.9	116.5	111.5
Moisture Content, %	8.2	10.0	12.1	14.1	16.2

Method : A

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.65

Maximum Dry Density= 119.0 pcf

Optimum Moisture= 12.5 %

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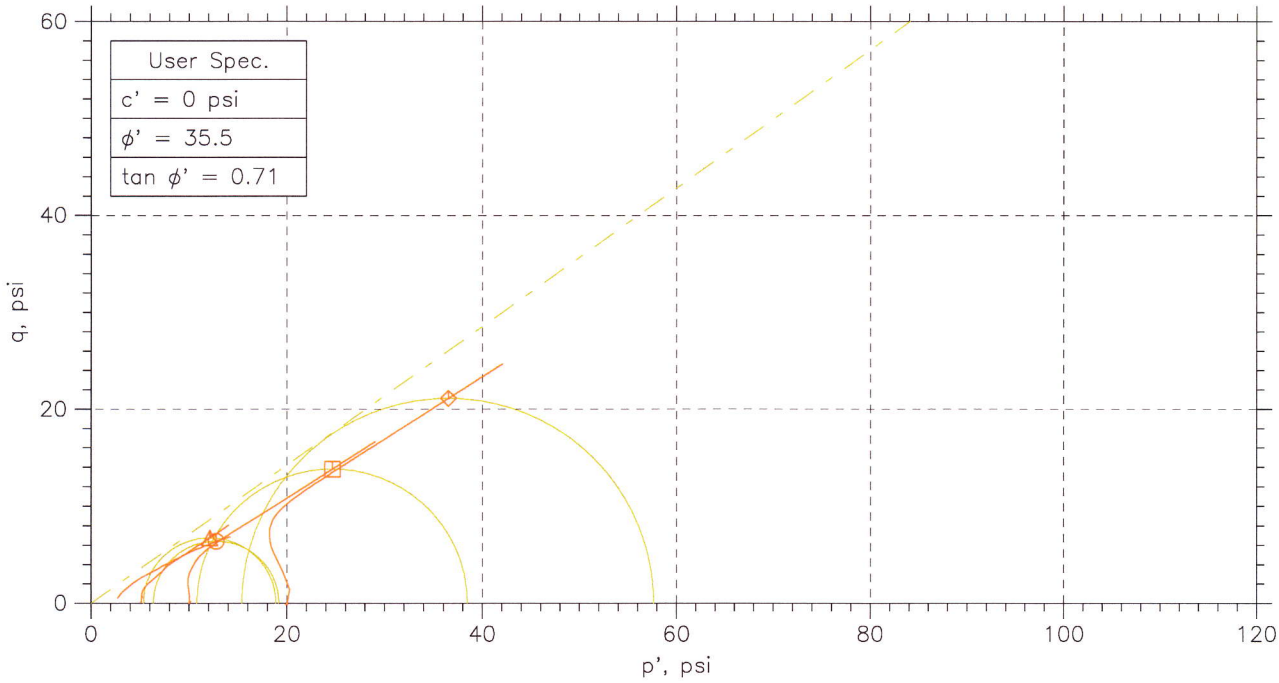
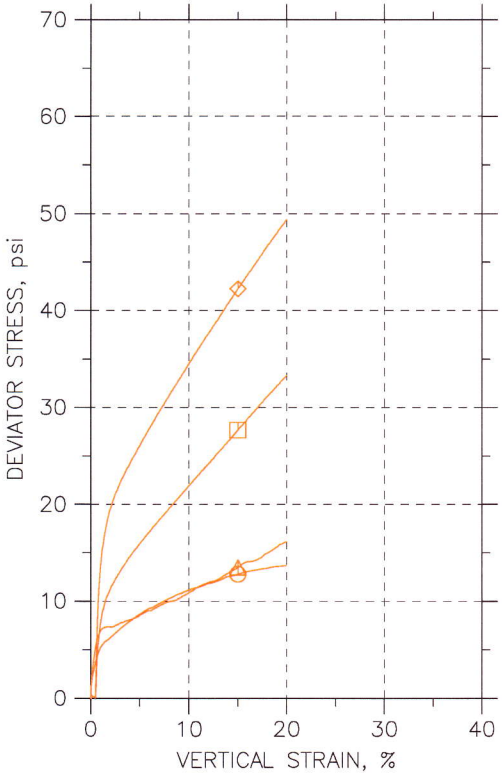
CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⊙	△	□	◇
Sample No.		Keating	Keating	Keating	Keating
Test No.		CU13	CU14	CU15	CU16
Depth		Keating	---	---	---
Initial	Diameter, in	2.87	2.87	2.87	2.87
	Height, in	6	6	6	6
	Water Content, %	14.7	14.7	14.7	14.7
	Dry Density, pcf	104.8	104.8	104.8	104.9
	Saturation, %	65.3	65.2	65.3	65.2
	Void Ratio	0.608	0.608	0.608	0.607
Before Shear	Water Content, %	22.2	23.0	22.0	21.2
	Dry Density, pcf	105.4	104.	105.7	107.2
	Saturation*, %	100.0	100.0	100.0	100.0
	Void Ratio	0.6	0.621	0.595	0.573
	Back Press., psi	82.	109.	97.	94.
Ver. Eff. Cons. Stress, psi		2.407	4.999	10	20
Shear Strength, psi		6.4	6.746	13.83	21.12
Strain at Failure, %		15	15	15	15
Strain Rate, %/min		0.04	0.04	0.04	0.04
B-Value		0.70	0.78	---	---
Estimated Specific Gravity		2.7	2.7	2.7	2.7
Liquid Limit		NP	NP	NP	NP
Plastic Limit		NP	NP	NP	NP

GeoTesting express

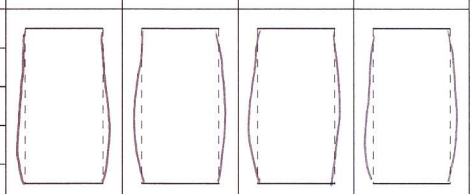
Project: NCHRP

Location: ---

Project No.: GTX-5504

Boring No.: Washings #4

Sample Type: Remolded



Description: Moist, olive gray sandy silt

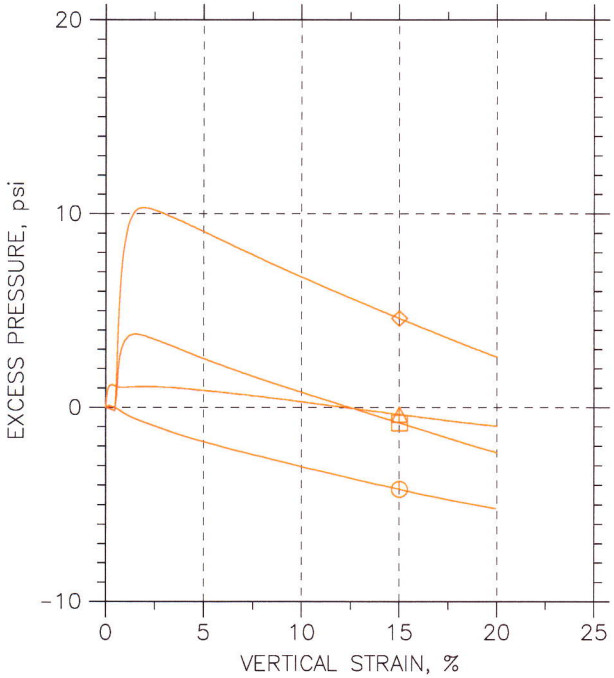
Remarks: Target Compaction: 95% of Max Dry Density (110.5 pcf) at Opt Moisture Content (14.5%).

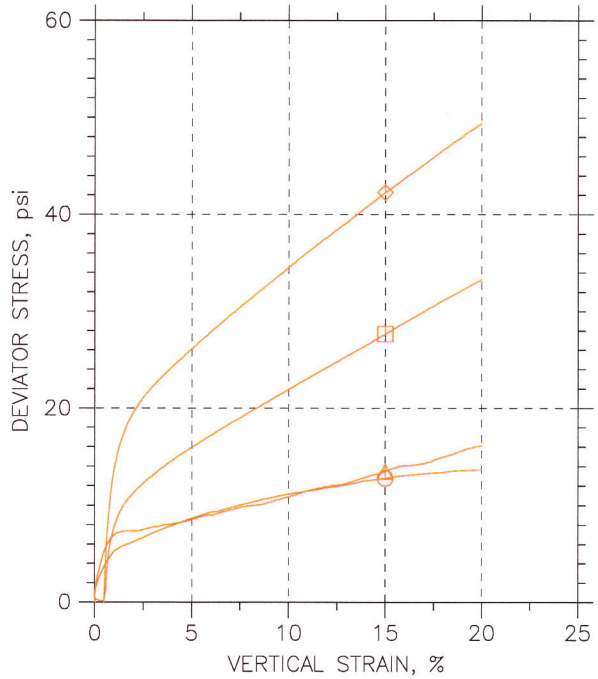
Tue, 03-MAY-2005 14:18:50

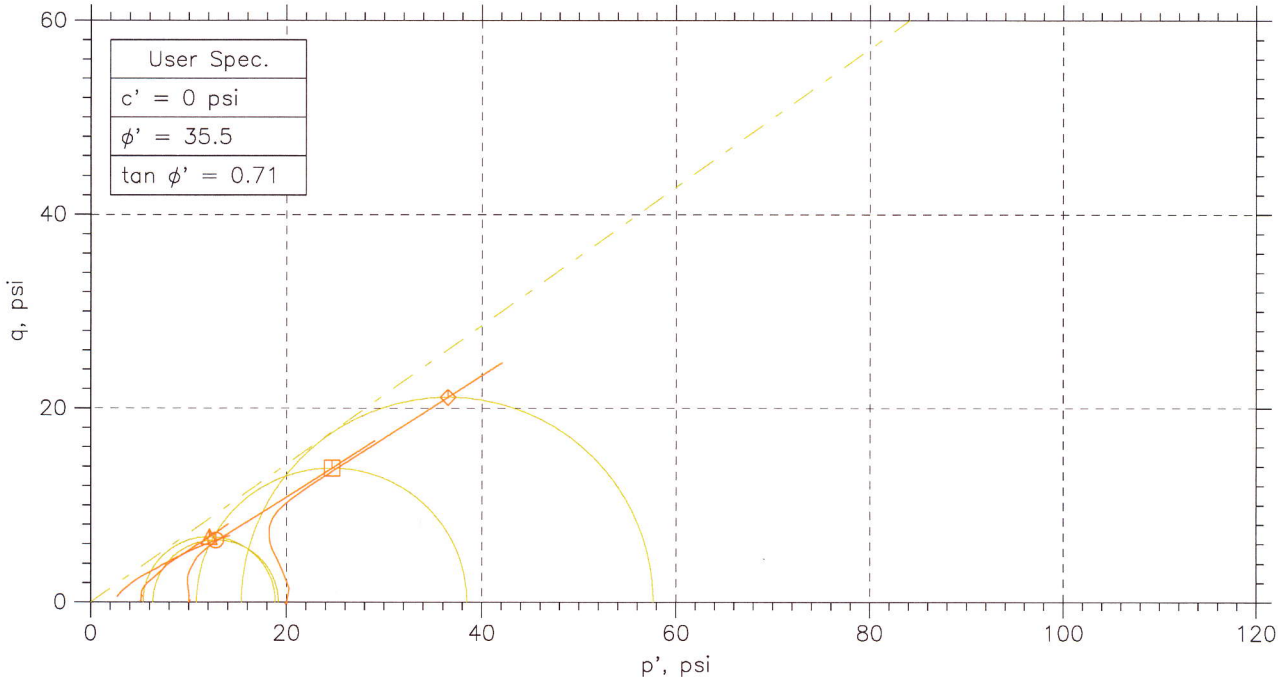
Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767







	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⊙	Keating	CU13	Keating	fy	04/28/05	jdt		5504-CU13.dat
△	Keating	CU14	---	fy	04/28/05	jdt		5504-cu14.dat
⊡	Keating	CU15	---	fy	04/29/05	jdt		5504-cu15.dat
◇	Keating	CU16	---	jdt	05/01/05	jdt		5504-cu16.dat

GeoTesting express

Project: NCHRP	Location: ---	Project No.: GTX-5504
Boring No.: Washings #4	Sample Type: Remolded	
Description: Moist, olive gray sandy silt		
Remarks: Target Compaction: 95% of Max Dry Density (110.5 pcf) at Opt Moisture Content (14.5%).		

Tue, 03-MAY-2005 14:19:09

Client:	Geocomp Consulting		
Project Name:	NCHRP 24-22		
Project Location:	Lunenburg, MA		
GTX #:	5504		
Start Date:	04/16/05	Tested By:	bdf/rmt
End Date:	04/19/05	Checked By:	jdt
Soil ID:	Keating #1 Washings #4 (Proposed A-4)		
Soil Description:	Moist, olive gray sandy silt		

Direct Shear Test Series by ASTM D 3080

Soil Preparation:	Compacted to 95% of the Maximum Dry Density at the Optimum Moisture Content.		
Compaction Characteristics:	Maximum Dry Density	110.5	pcf
	Optimum Moisture Content	14.5	%
	Compaction Test Method	ASTM D 698	
Test Equipment:	Top box = 12 in x 12 in; Bottom box = 16 in x 12 in; Load cells and LVDTs connected to data acquisition system for shear force, normal load and horizontal displacement readings; Flat plate clamping device; surface area = 144 in^2		
Horizontal Displacement, in/min:	0.04	Test Condition:	inundated

Parameter	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6
Initial Moisture Content, %	15	15	15	---	---	---
Initial Dry Density, pcf	105	105	105	---	---	---
Percent Compaction, %	94.7	94.7	94.7	---	---	---
Normal Compressive Stress, psf	600	1200	2400	---	---	---
Peak Shear Stress, psf	562	1281	2002	---	---	---
Post Peak Shear Stress, psf	---	---	---	---	---	---
Final Moisture Content, %	19	19	18	---	---	---

NOTES:

Peak Friction Angle:	**38**	**degrees**
Peak Cohesion:	**201**	**psf**
Post Peak Friction Angle:	**---**	**degrees**
Post Peak Cohesion:	**---**	**psf**

Figure a. Shear Force vs. Horizontal Displacement

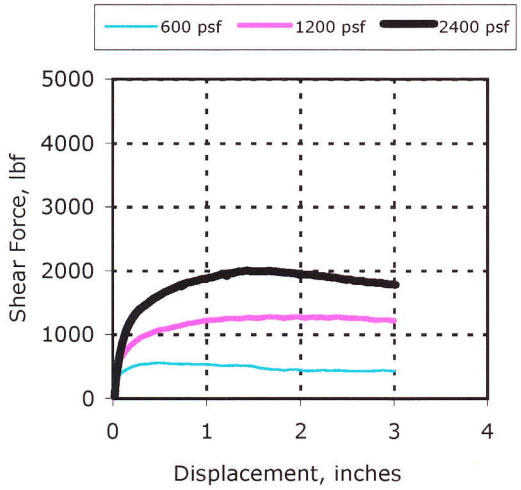
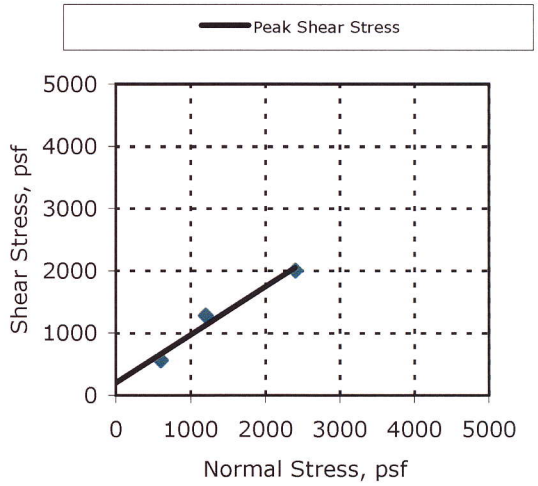


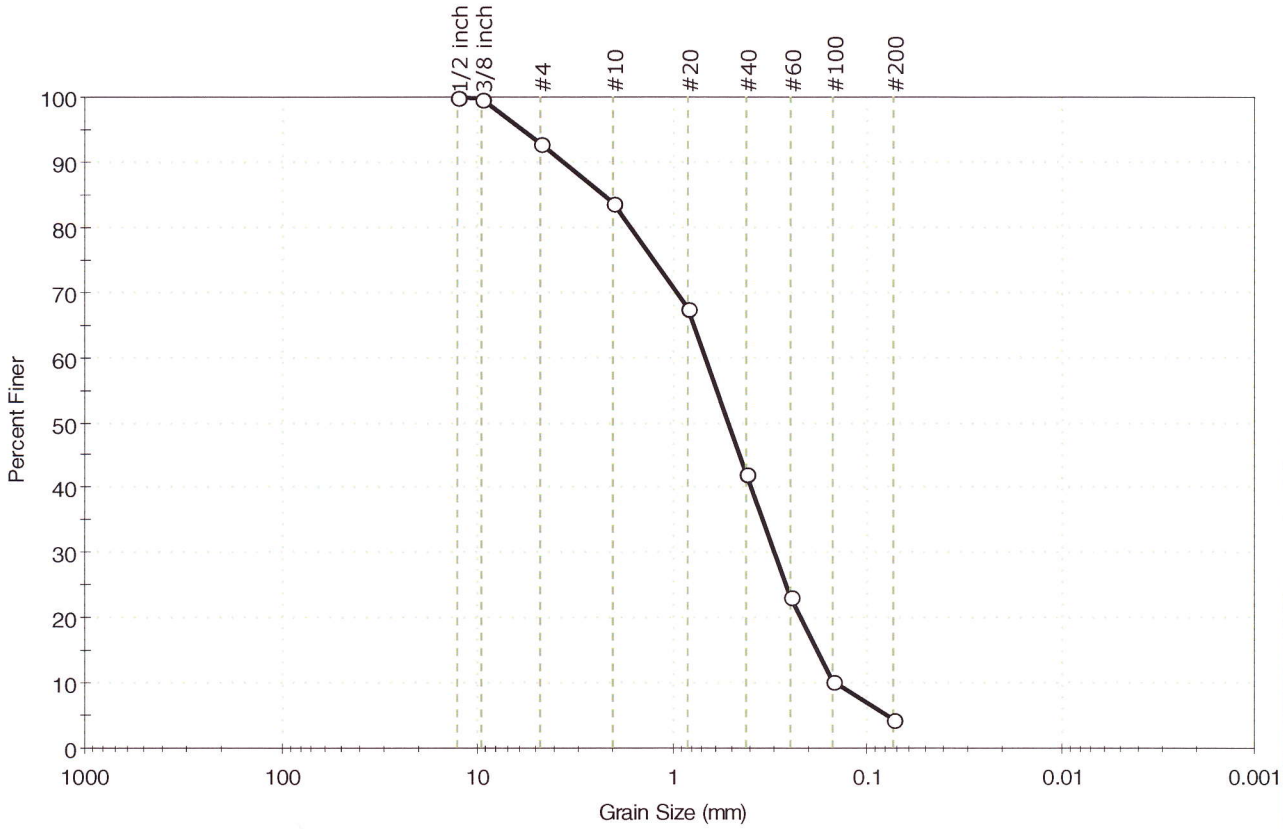
Figure b. Shear Stress vs. Normal Stress



GeoTesting express

Client:	Geocomp Consulting		
Project:	NCHRP 24-22		
Location:	Lunenburg MA	Project No:	GTX-5504
Boring ID: Drain Soil Alt.	Sample Type: bag	Tested By:	njh
Sample ID: Local Sand Source #5	Test Date: 04/18/05	Checked By:	jdt
Depth : ---	Test Id: 68250		
Test Comment: ---			
Sample Description: Moist, dark yellowish brown sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	7.0	88.4	4.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1/2 inch	12.70	100		
3/8 inch	9.51	100		
#4	4.75	93		
#10	2.00	84		
#20	0.84	68		
#40	0.42	42		
#60	0.25	23		
#100	0.15	10		
#200	0.074	5		

Coefficients

D_{85} = 2.2339 mm	D_{30} = 0.3014 mm
D_{60} = 0.6851 mm	D_{15} = 0.1798 mm
D_{50} = 0.5216 mm	D_{10} = 0.1432 mm
C_u = 4.785	C_c = 0.133

Classification

ASTM Poorly graded sand (SP)

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

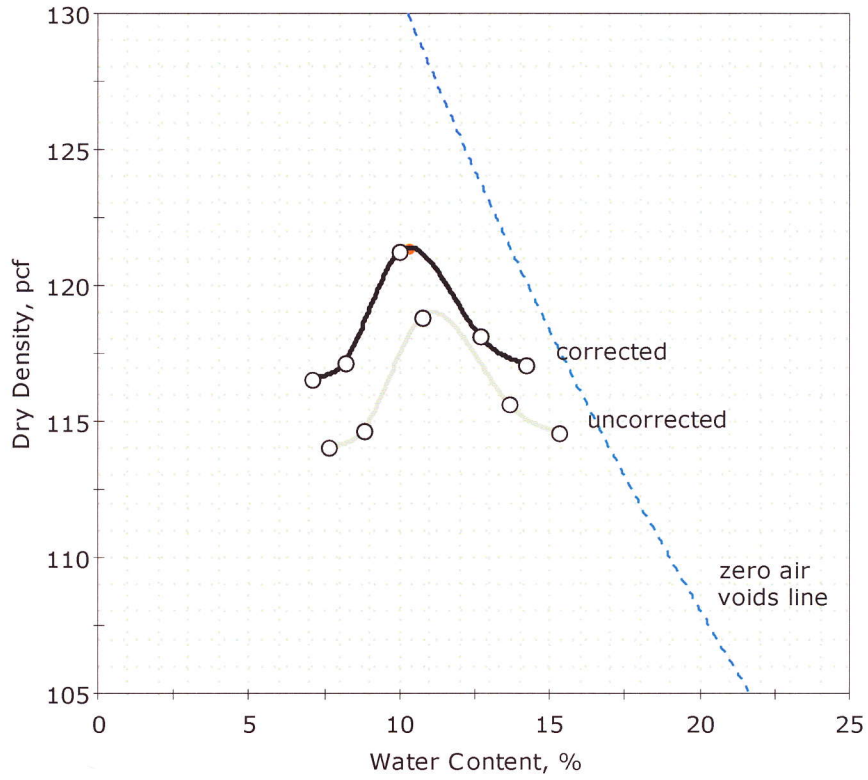
Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

Client:	Geocomp Consulting				
Project:	NCHRP 24-22				
Location:	Lunenburg MA			Project No:	GTX-5504
Boring ID:	Drain Soil Alt.	Sample Type:	bag	Tested By:	jdt
Sample ID:	Local Sand Source #5	Test Date:	04/18/05	Checked By:	jdt
Depth :	---	Test Id:	68260		
Test Comment:	---				
Sample Description:	Moist, dark yellowish brown sand				
Sample Comment:	---				

Compaction Report - ASTM D 1557



Data Points	Point 1	Point 2	Point 3	Point 4	Point 5
Dry density, pcf	114.1	114.7	118.9	115.7	114.6
Moisture Content, %	7.6	8.8	10.7	13.6	15.3

Method : A

Preparation : WET

As received Moisture :

Rammer : Manual

Zero voids line based on assumed specific gravity of 2.65

Maximum Dry Density=	119.0 pcf
Optimum Moisture=	11.0 %
Oversize Correction	(7.0% > #4 Sieve)
Corrected Maximum Dry Density=	121.5 pcf
Corrected Optimum Moisture=	10.5 %
Average Bulk Specific Gravity =	2.65

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GeoTesting express

Client:	Geocomp Consulting		
Project Name:	NCHRP 24-22		
Project Location:	Lunenburg MA		
GTX #:	5504		
Start Date:	04/21/05	Tested By:	pcs
End Date:	04/21/05	Checked By:	jdt
Boring #:	Drain Soil Alt.		
Sample #:	Local Sand Source #5		
Depth:	---		
Visual Description:	Dry, light yellowish brown sand		

Permeability of Granular Soils (Constant Head) by ASTM D 2434

Sample Type: Remolded

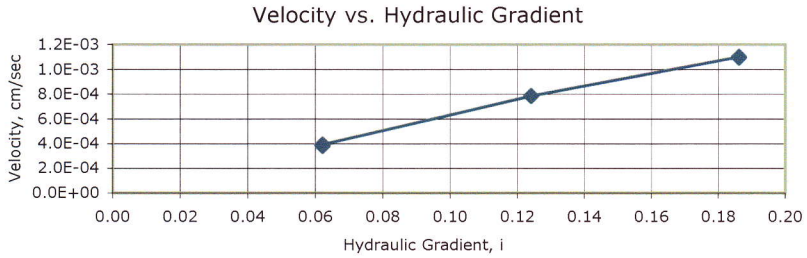
Sample Information:

Maximum Dry Density:	121.5	pcf
Optimum Moisture Content:	10.5	%
Compaction Test Method:	ASTM D 1557	
Classification (ASTM D 2487):	SP	
Assumed Specific Gravity:	2.65	

Sample Preparation / Test Setup: Compacted to 95% of Maximum Dry Density; >3/8 inch material screened out of sample prior to testing (>1% of sample). 5.27 lb surcharge

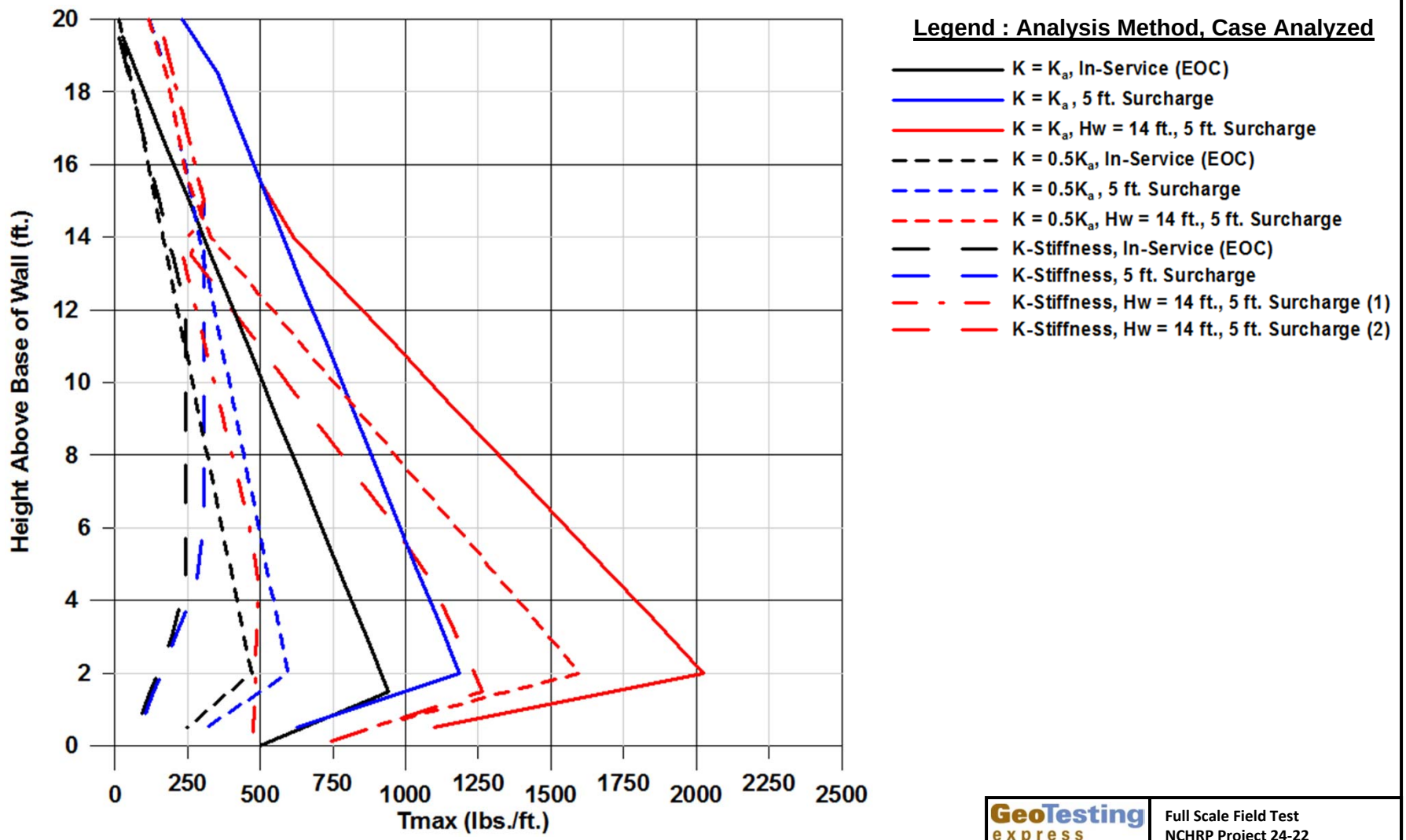
Parameter	Initial	Final
Height, in	4.03	4.03
Diameter, in	3.98	3.98
Area, in^2	12.4	12.4
Volume, in^3	50.1	50.1
Mass, g	1519	1761
Bulk Density, pcf	115	134
Moisture Content, %	0.3	16
Dry Density, pcf	115	115
Degree of Saturation, %	---	98.6
Void Ratio, e	---	0.44

Date	Reading #	Volume of Flow, cc	Time of Flow, sec	Flow Rate, cc/sec	Gradient	Permeability, cm/sec	Temp., °C	Correction Factor	Permeability @ 20 °C, cm/sec
04/21	1	0.61	20	0.03	0.06	6.1E-03	15.0	1.135	7.0E-03
04/21	2	0.61	20	0.03	0.06	6.1E-03	15.0	1.135	7.0E-03
04/21	3	0.63	20	0.03	0.06	6.3E-03	15.0	1.135	7.2E-03
04/21	4	1.26	20	0.06	0.12	6.3E-03	15.0	1.135	7.2E-03
04/21	5	1.26	20	0.06	0.12	6.3E-03	15.0	1.135	7.2E-03
04/21	6	1.26	20	0.06	0.12	6.3E-03	15.0	1.135	7.2E-03
04/21	7	1.77	20	0.09	0.19	5.9E-03	15.0	1.135	6.7E-03
04/21	8	1.76	20	0.09	0.19	5.9E-03	15.0	1.135	6.7E-03
04/21	9	1.76	20	0.09	0.19	5.9E-03	15.0	1.135	6.7E-03



PERMEABILITY @ 20 °C = 7.0×10^{-3} cm/sec

ATTACHMENT B



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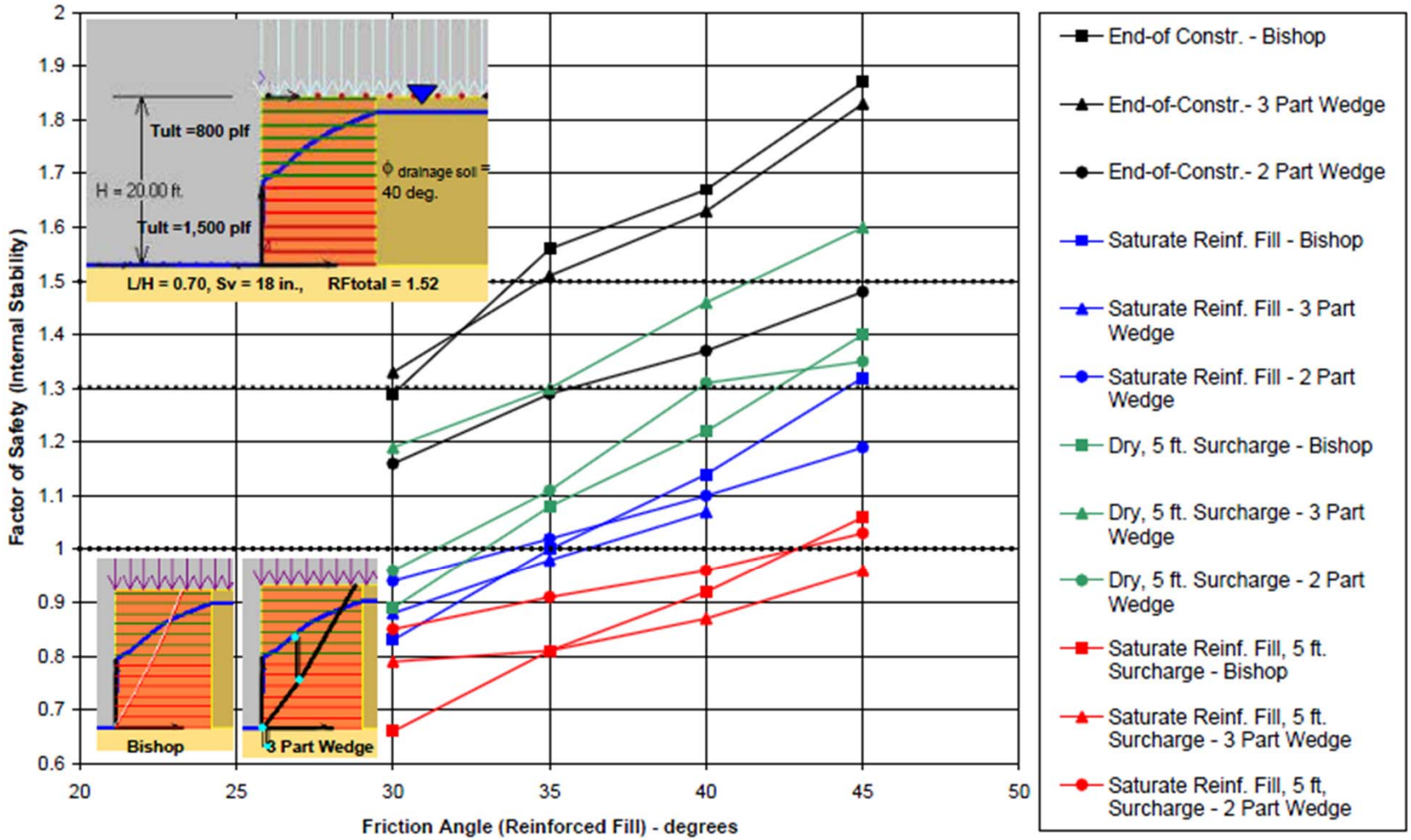
Full Scale Field Test
NCHRP Project 24-22

Calculated Maximum Reinforcement Load vs. Height of Test Wall, Wall A

Figure B-1



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Full Scale Field Test
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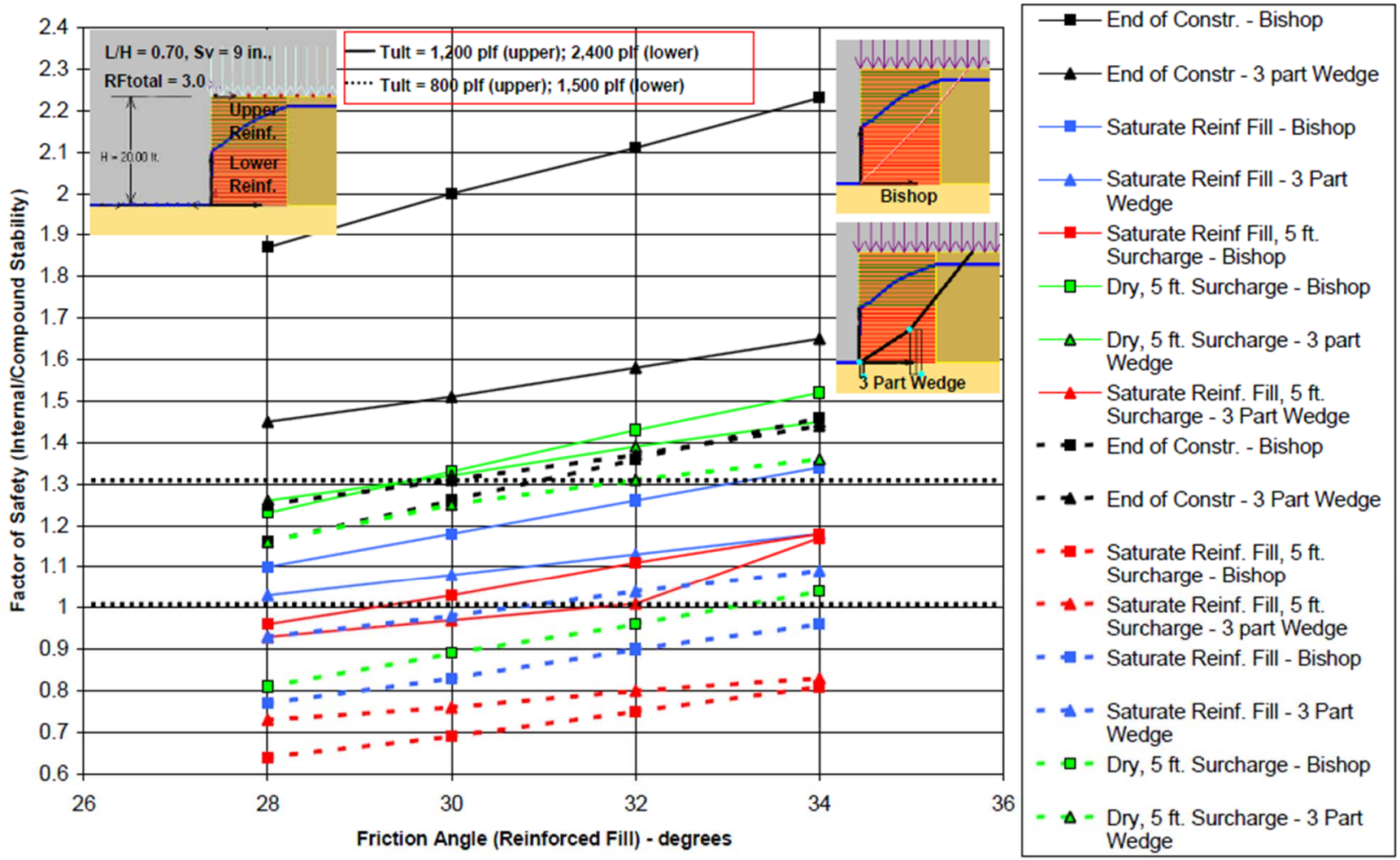
Summary of Limit Equilibrium Analyses Walls A,B,C (PET Geogrid Reinforcement)

Figure B-2



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Full Scale Field Test
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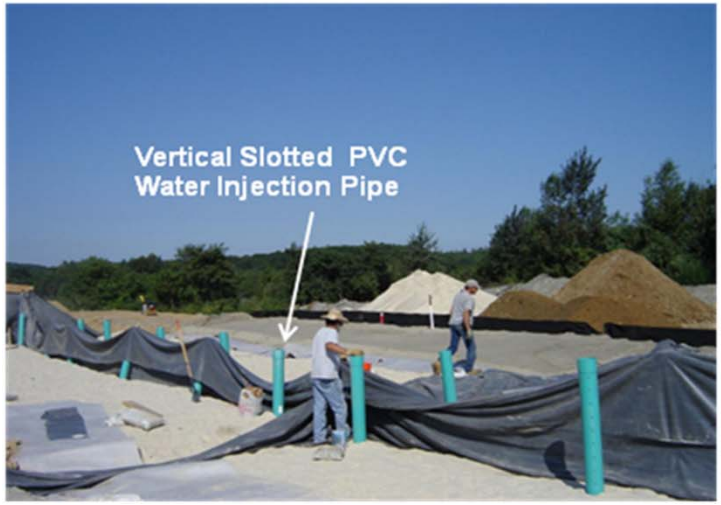
Summary of Limit Equilibrium Analyses Wall D (NWGT Reinforcement)

Figure B-3



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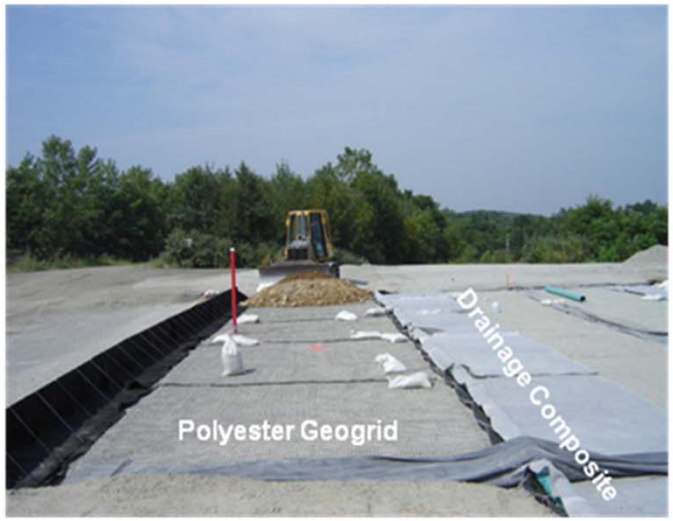
Seaming Vertical PVC Geomembrane

Figure B-4



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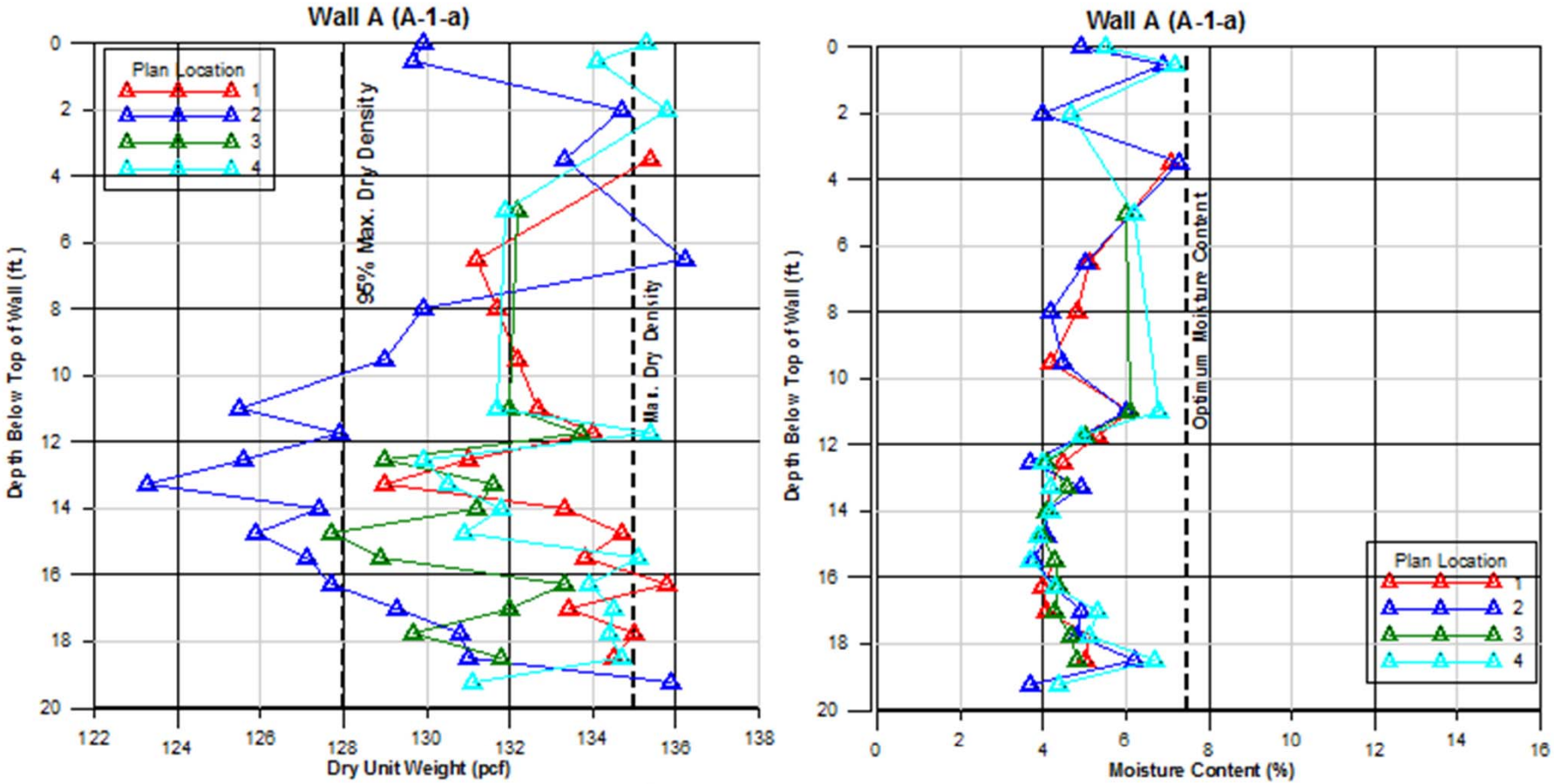
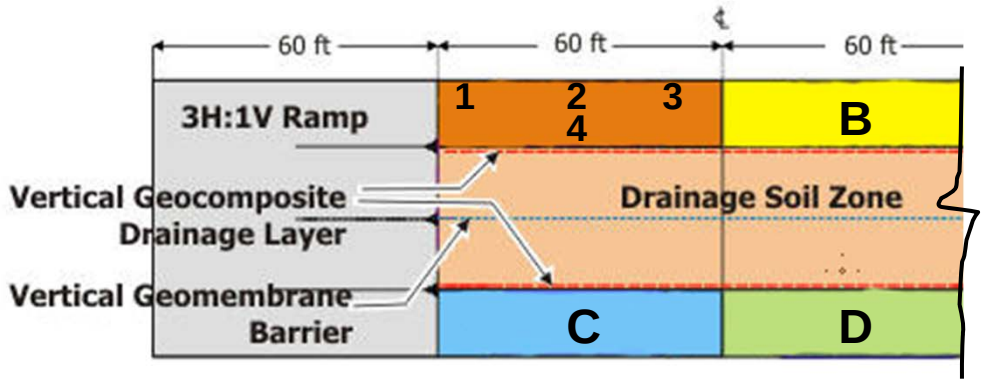
NCHRP Project 24-22

Placing and Compacting Reinforced Fill

Figure B-5





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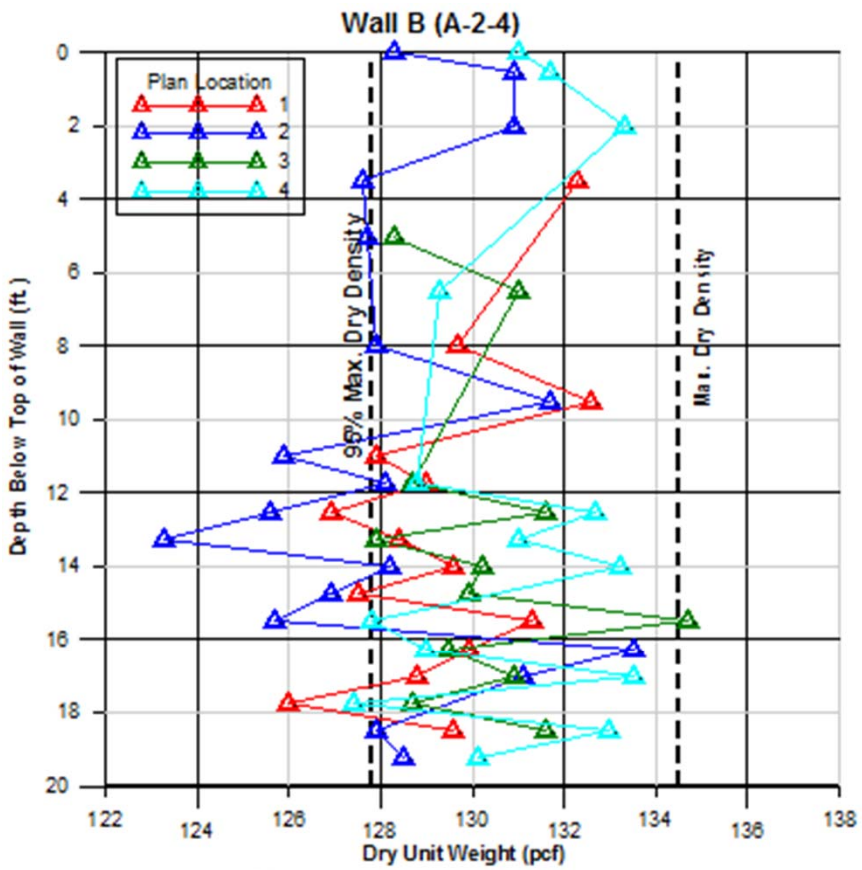
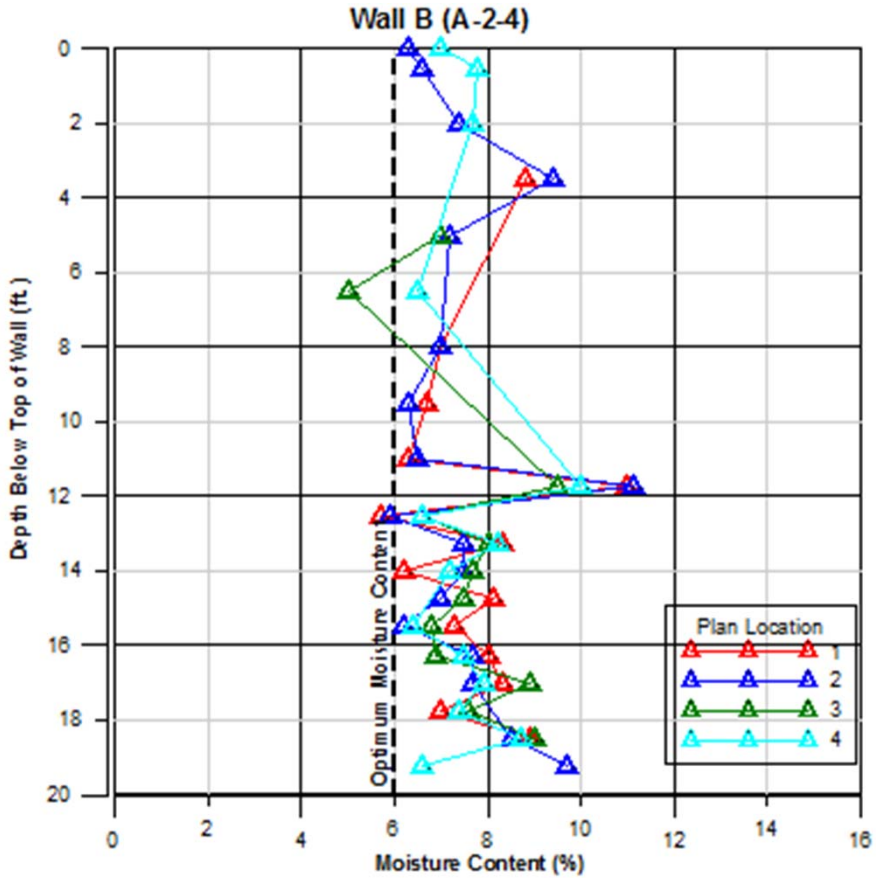
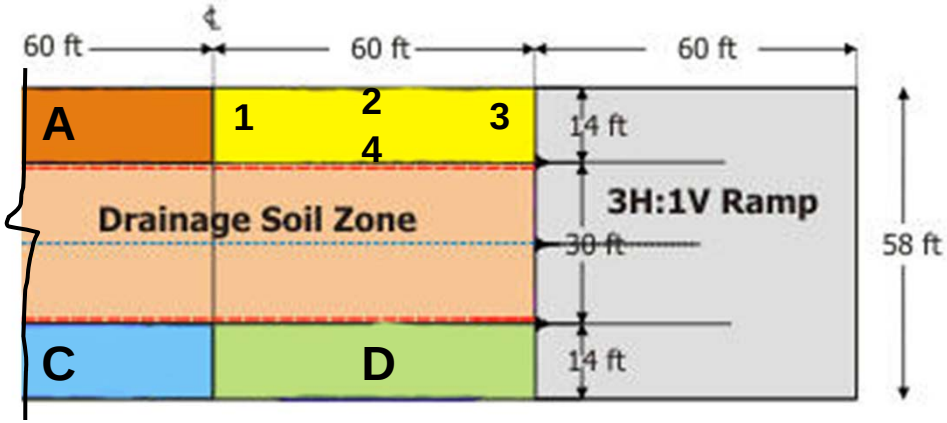
Full Scale Field Test
NCHRP Project 24-22

Reinforced Fill, As-Compacted Properties vs. Depth Below TOW Wall A

Figure B-6







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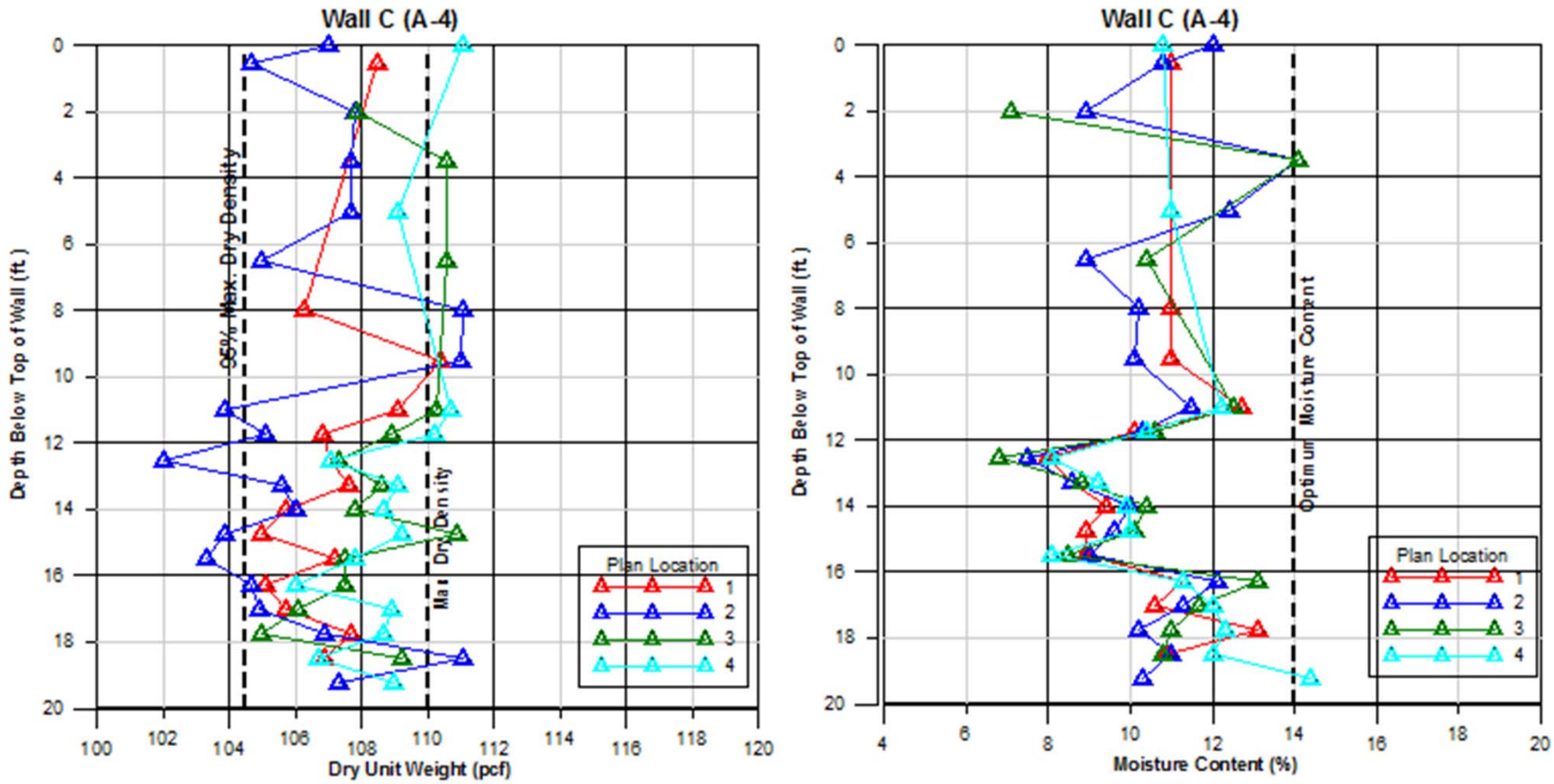
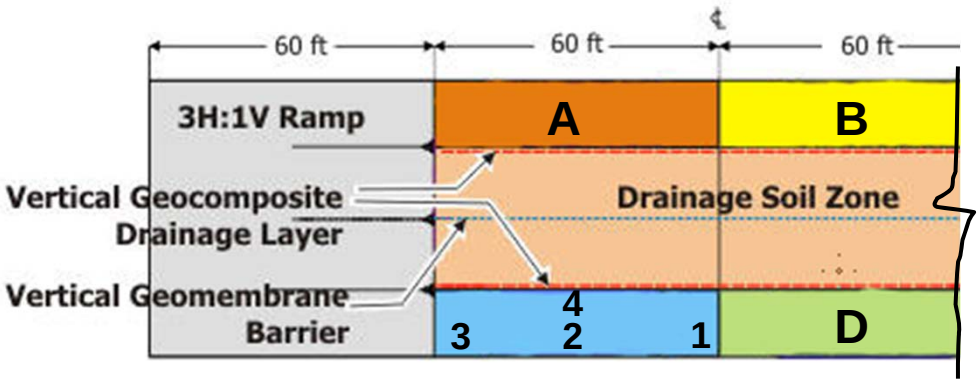
Full Scale Field Test
NCHRP Project 24-22

Reinforced Fill, As-Compacted Properties vs. Depth Below TOW Wall B

Figure B-7





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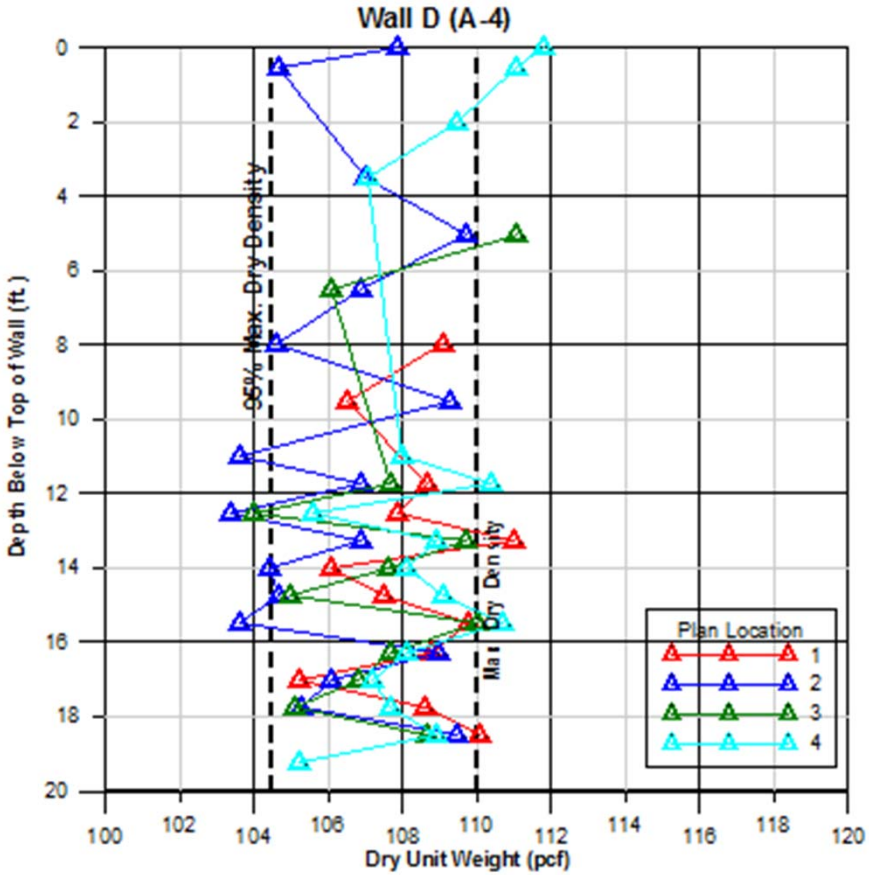
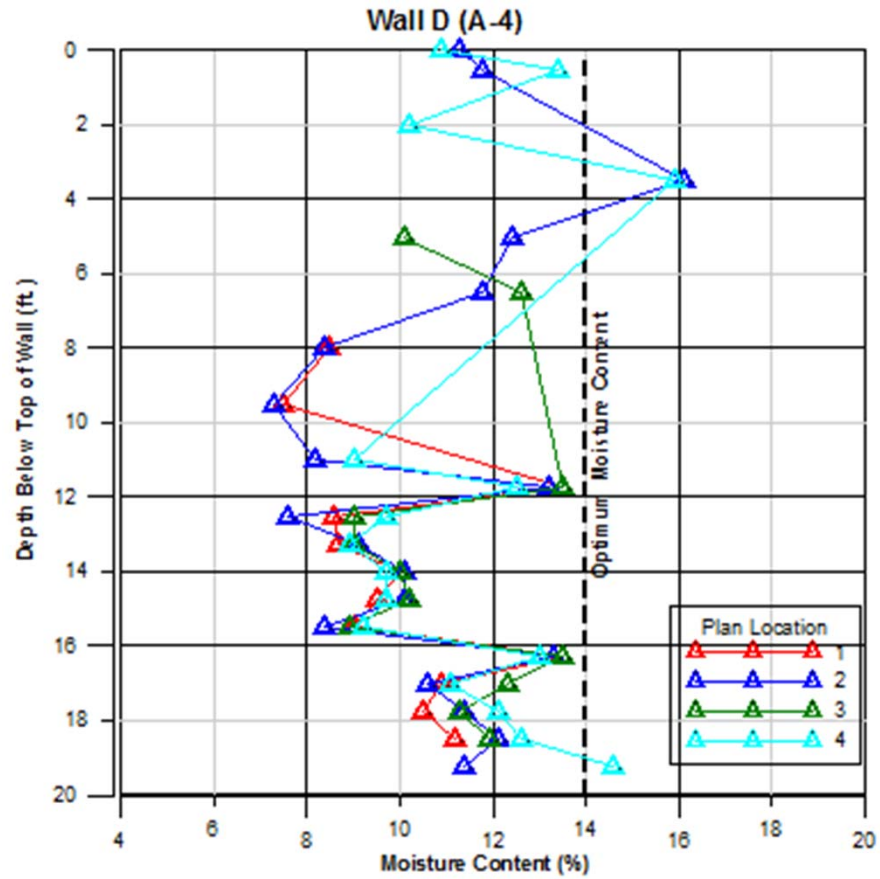
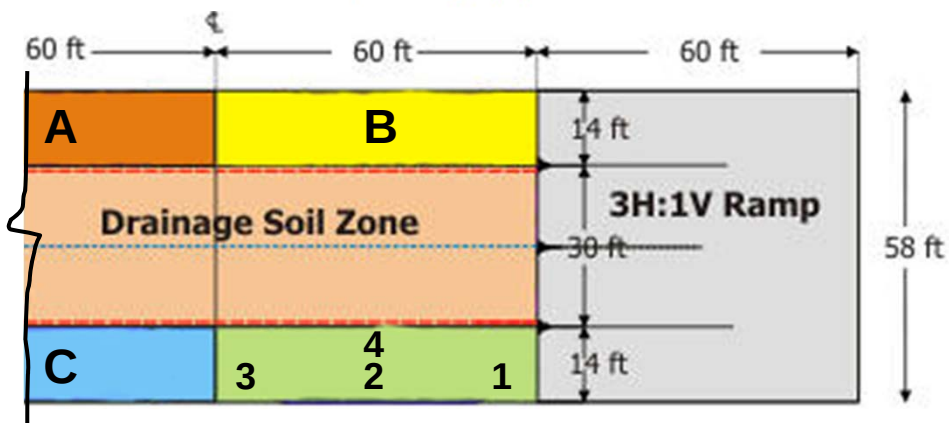
Full Scale Field Test
NCHRP Project 24-22

Reinforced Fill, As-Compacted Properties vs. Depth Below TOW Wall C

Figure B-8







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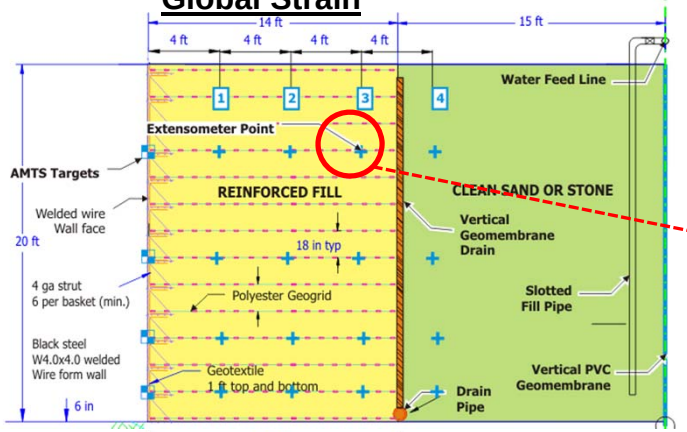
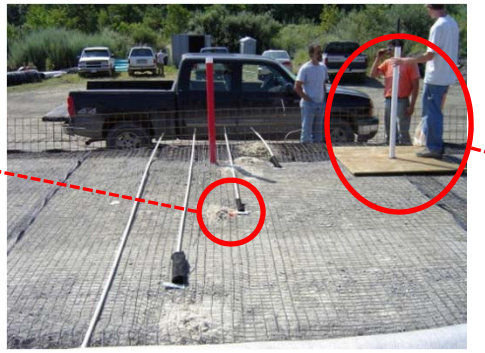
Full Scale Field Test
NCHRP Project 24-22

Reinforced Fill, As-Compacted Properties vs. Depth Below TOW Wall D

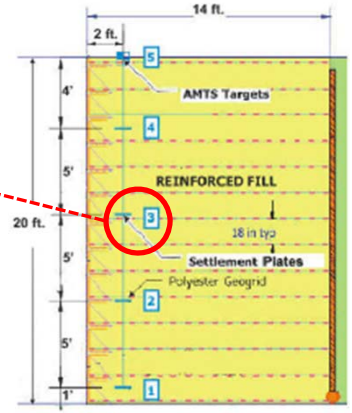
Figure B-9

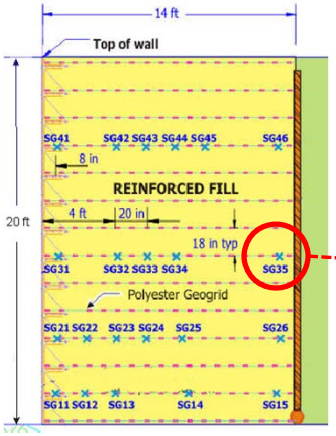
Global Strain



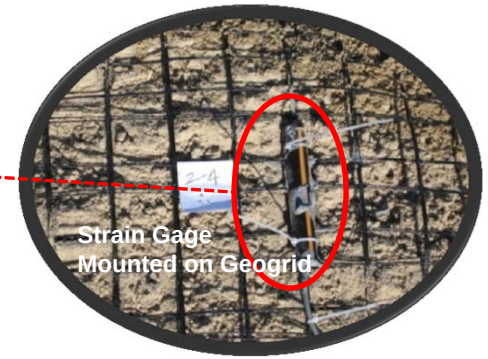


Settlement of Reinforced Fill

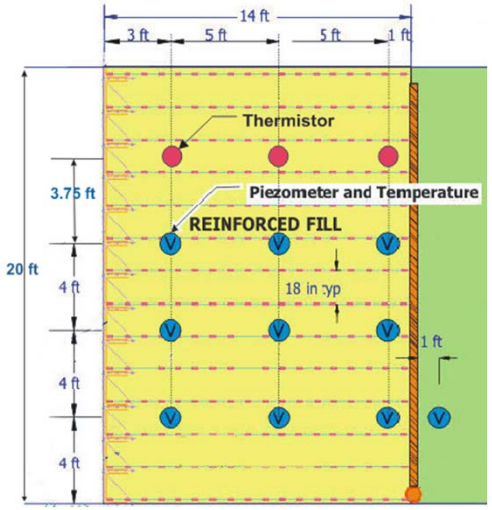




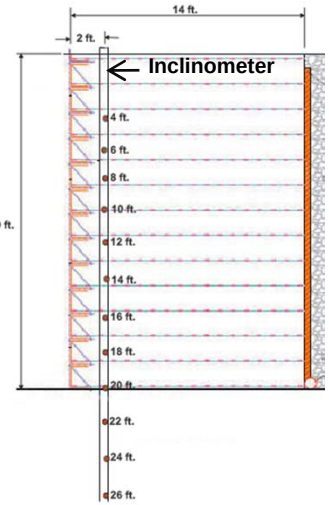
Reinforcement Strain/Load



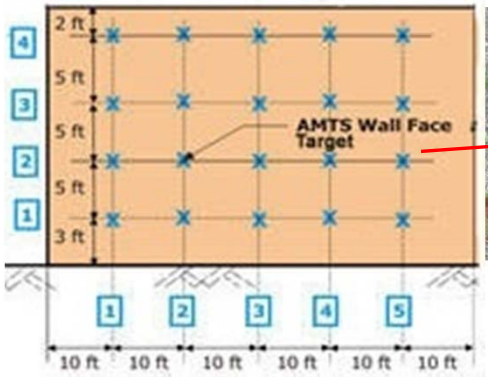
Porewater Pressure & Temperature



Lateral Movement



Face Movement



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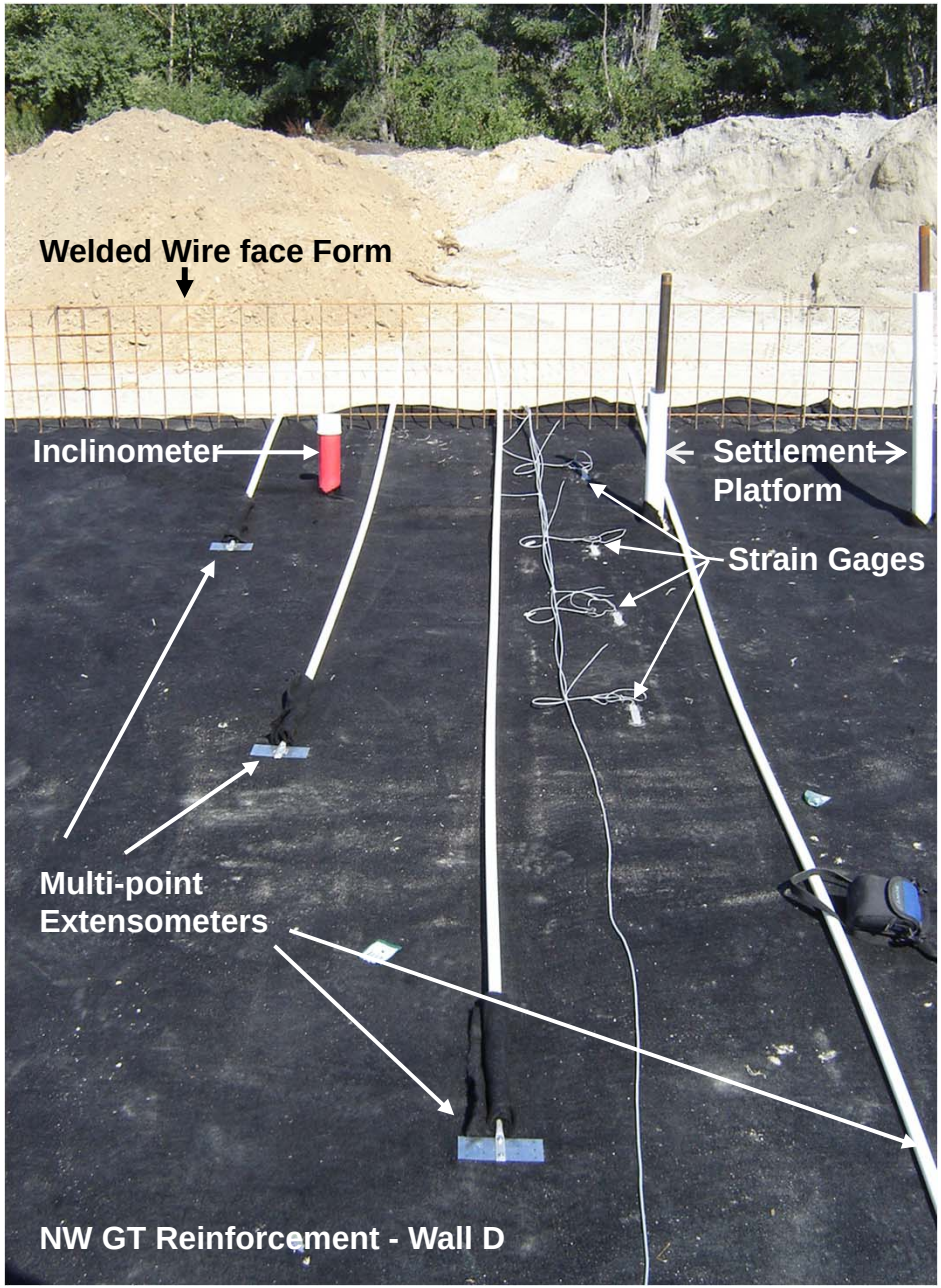
Full Scale Field Test
NCHRP Project 24-22

Instrumentation Details
All Walls

Figure B-10



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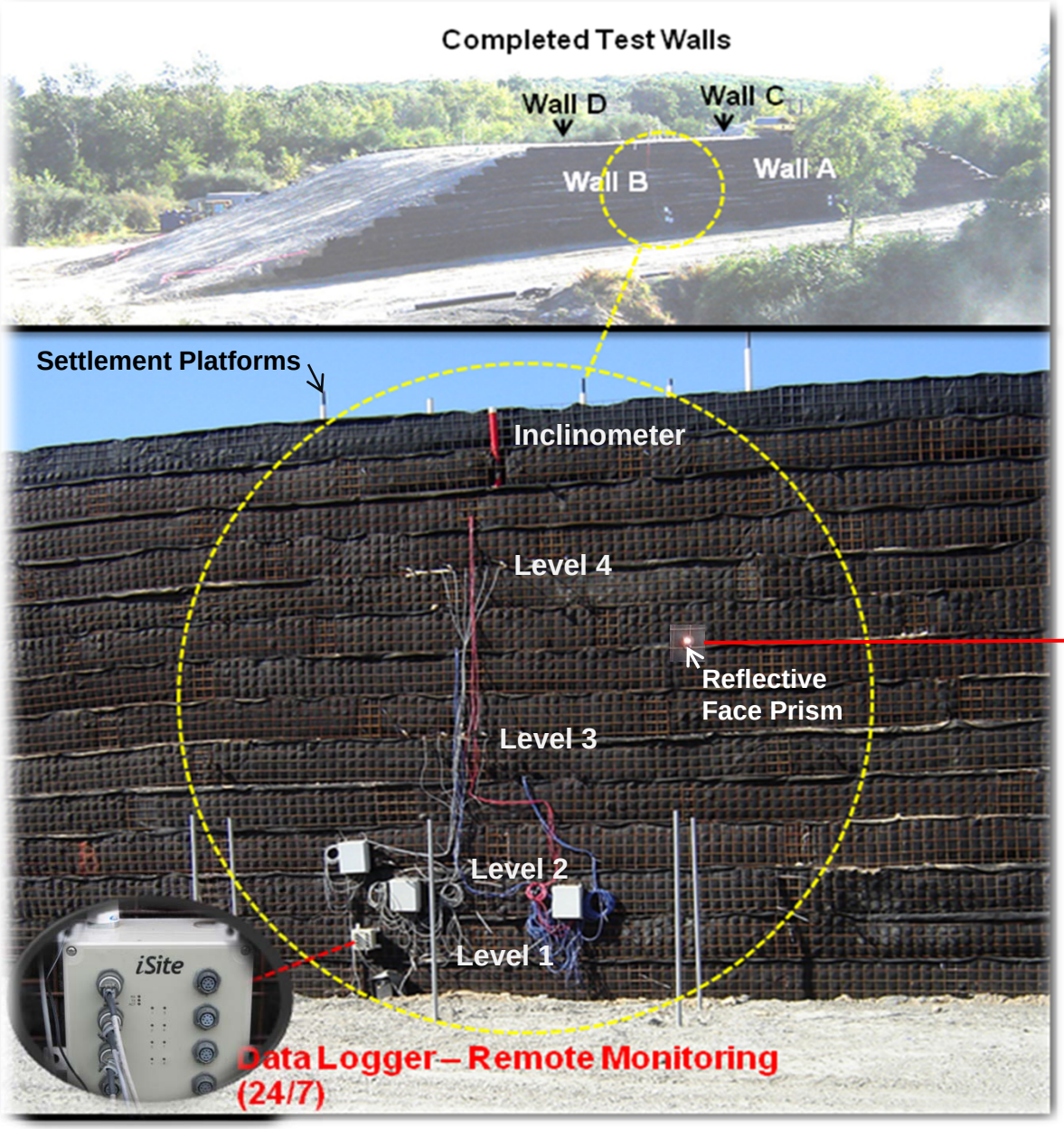
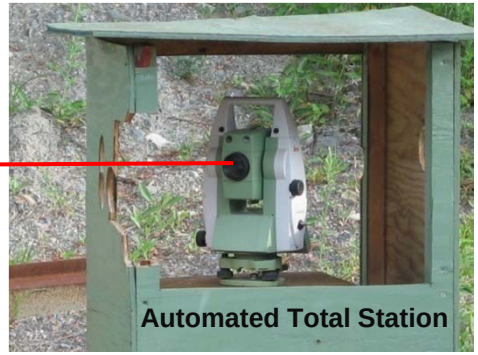
Full Scale Field Test
NCHRP Project 24-22

Typical As-Installed Instrumentation
All Walls

Figure B-11





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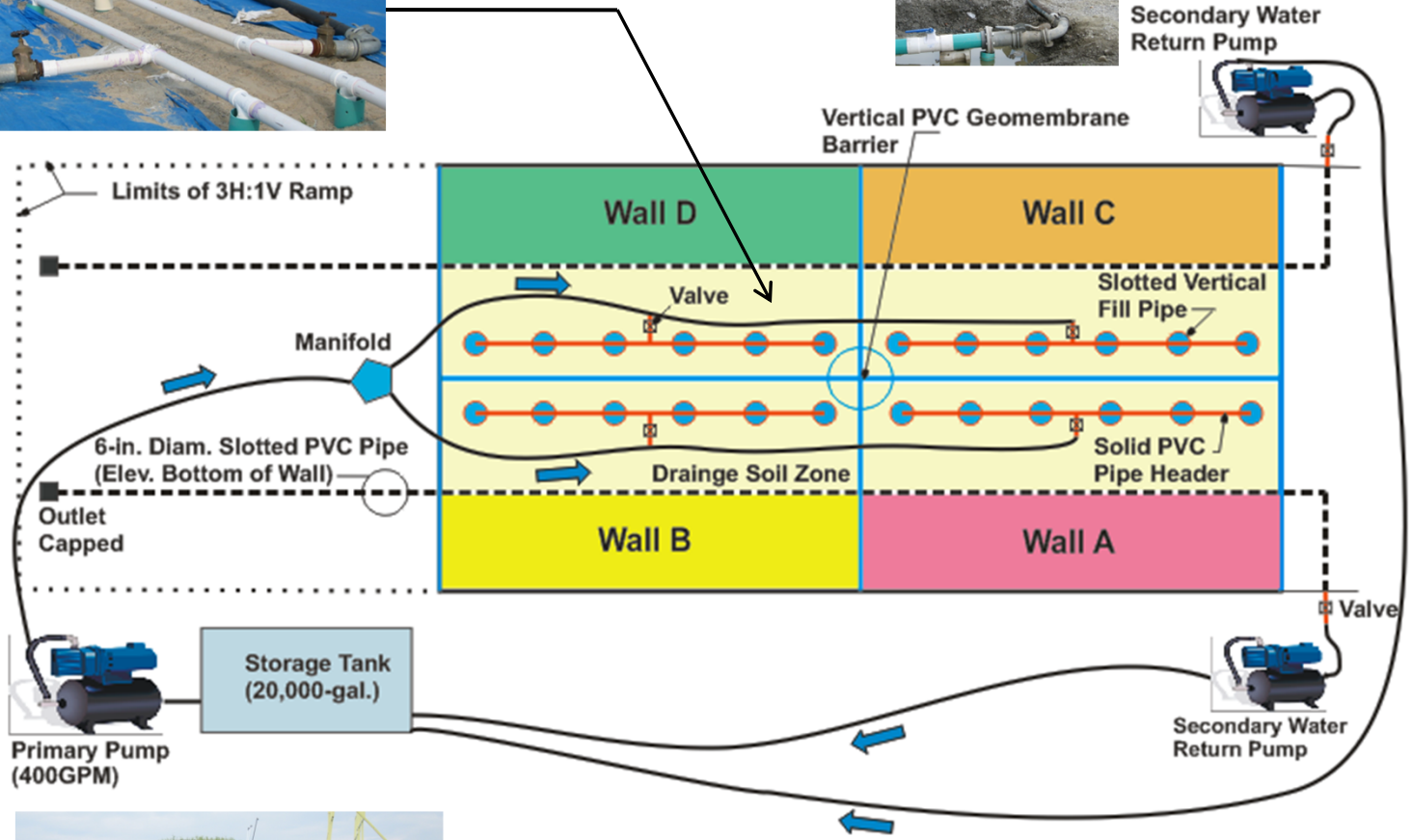
Full Scale Field Test
NCHRP Project 24-22

Typical Instrumentation Readout Cluster

Figure B-12



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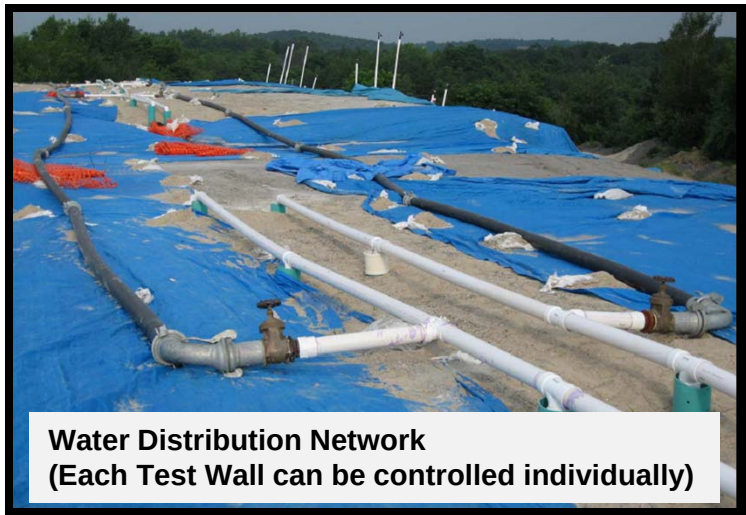
Full Scale Field Test
NCHRP Project 24-22

Plan View
Hydrotest Details

Figure B-13



Water Distribution Network
(Each Test Wall can be controlled individually)

Refilling Holding Tank

20,000 gal. Holding Tank & Primary Water Supply Pump

One of 2 Secondary Water Return Systems

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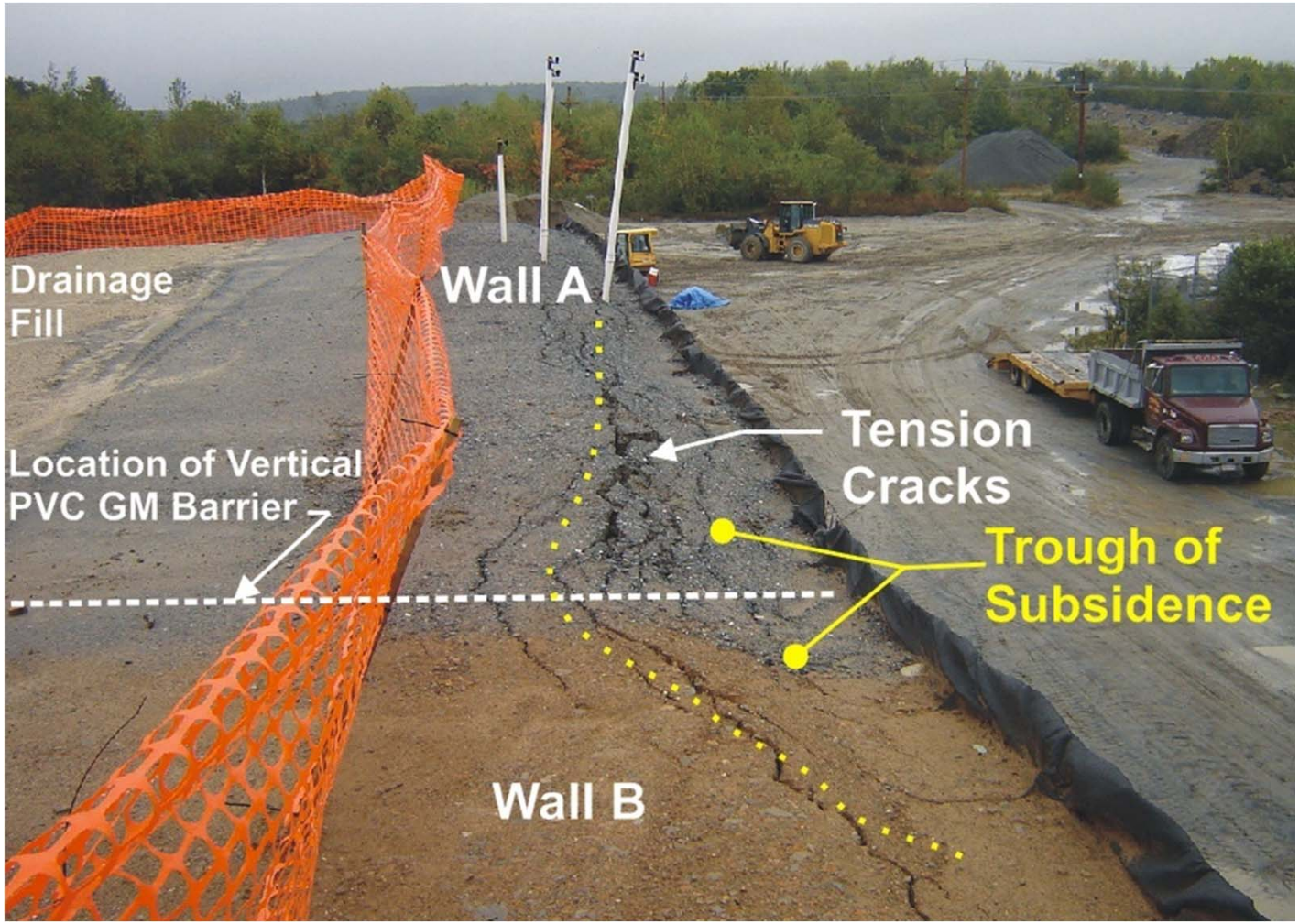
Full Scale Field Test
NCHRP Project 24-22

Test Stages 3 and 6
(Hydrotests Aug. '06 & '07)

Figure B-14



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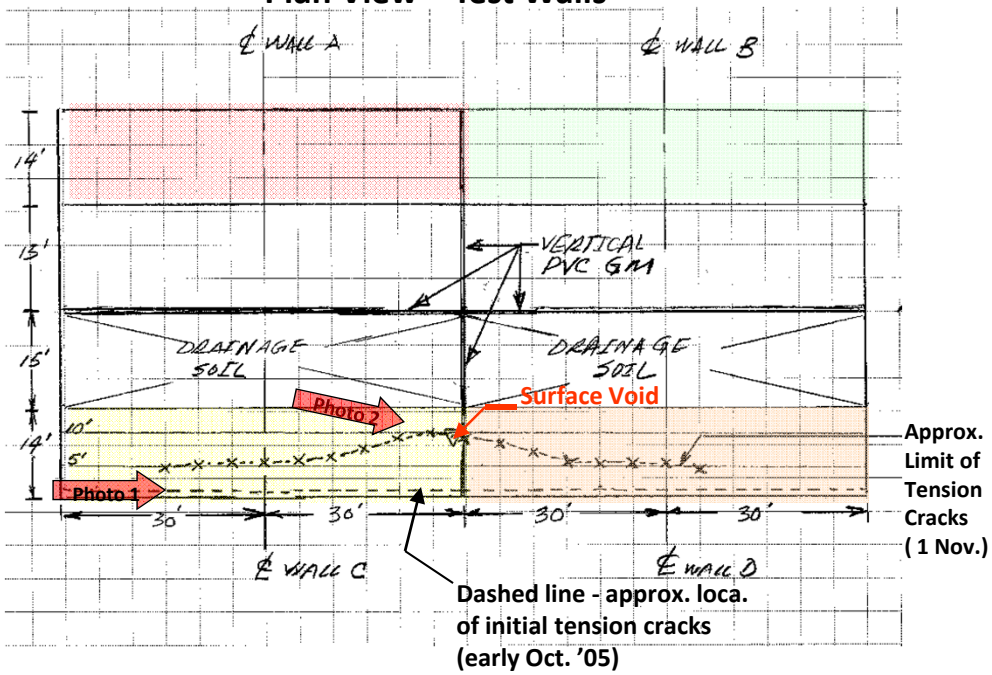
Full-Scale Field Test
NCHRP Project 24-22

Effects of Oct. 2005 Record Rainfall (Test Walls A and B)

Figure B-15

Plan View – Test Walls



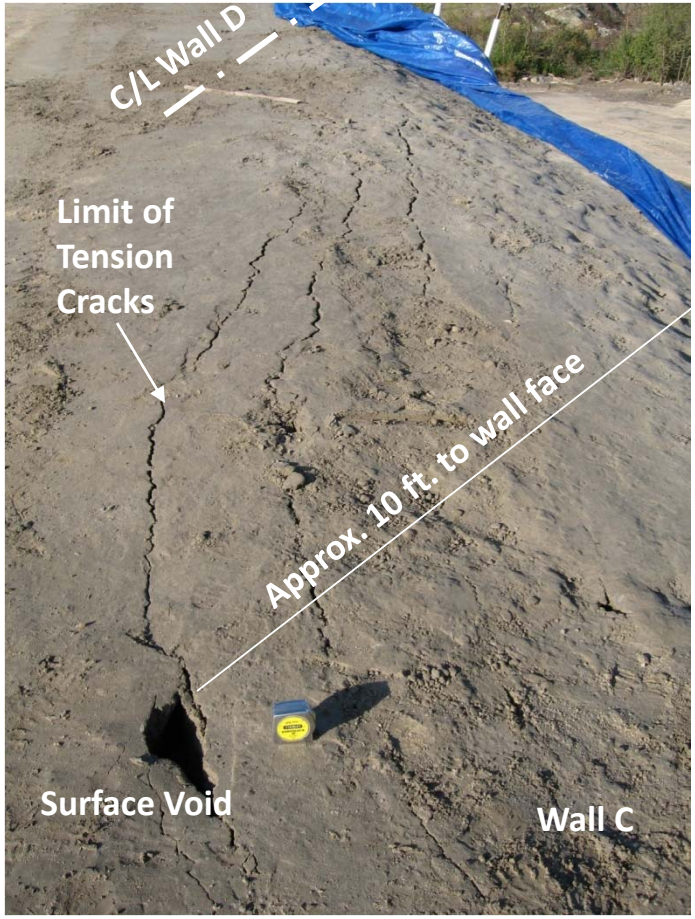


Photo 2 (1 Nov. '05)

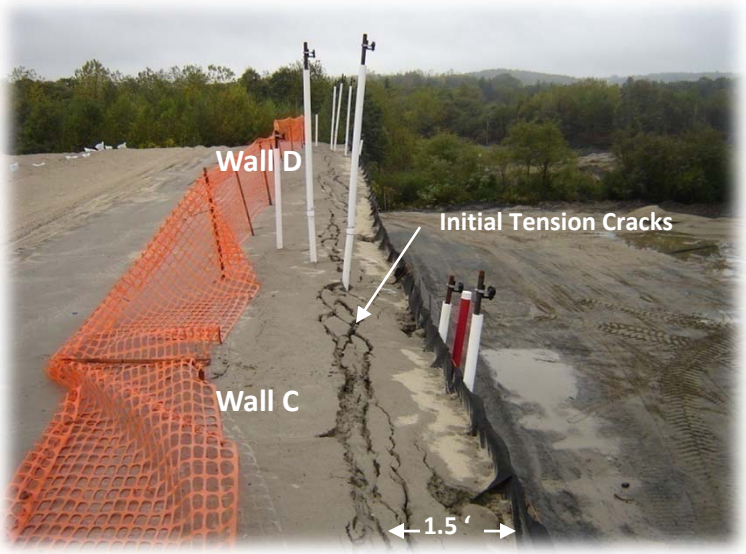


Photo 1 (Early Oct. '05)

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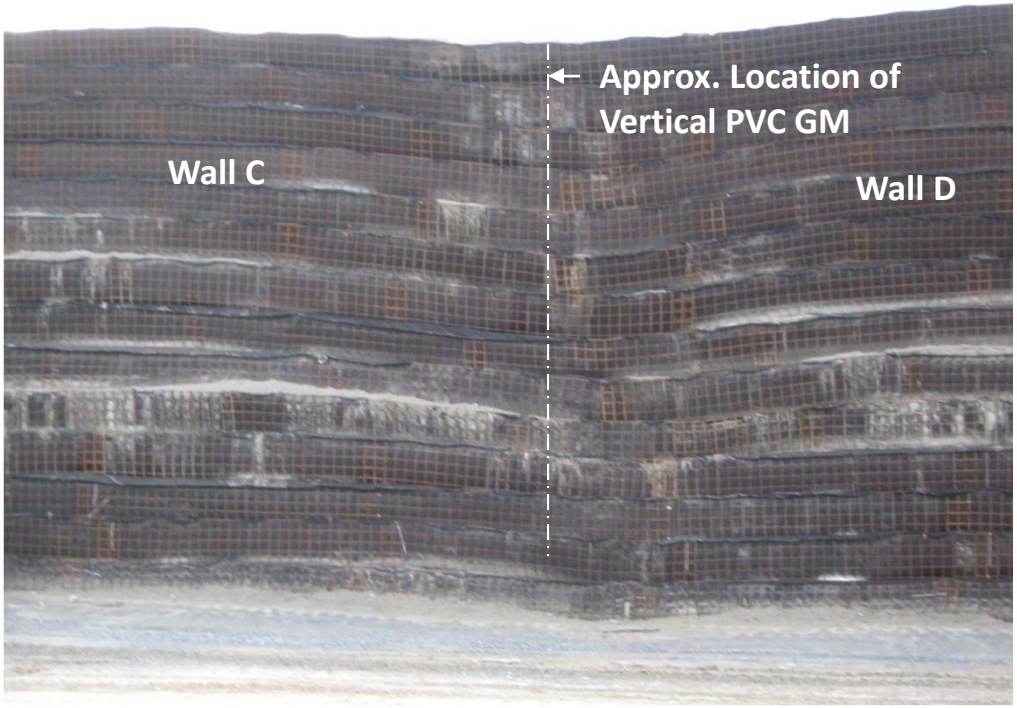
Full-Scale Field Test
NCHRP Project 24-22

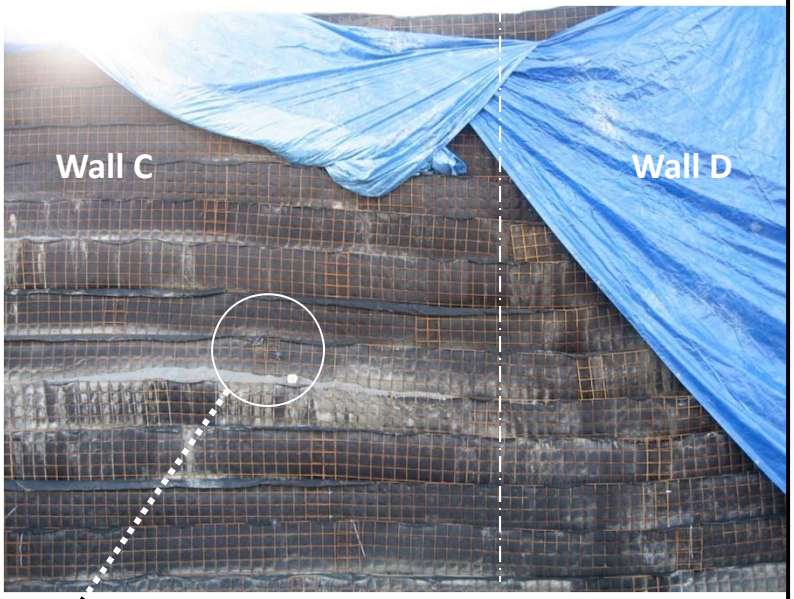
Effects of Oct. 2005 Record Rainfall (Test Walls C and D)

Figure B-16

18 Oct. '05

1 Nov. '05





Note Upward Rotation of
Face Prism Target

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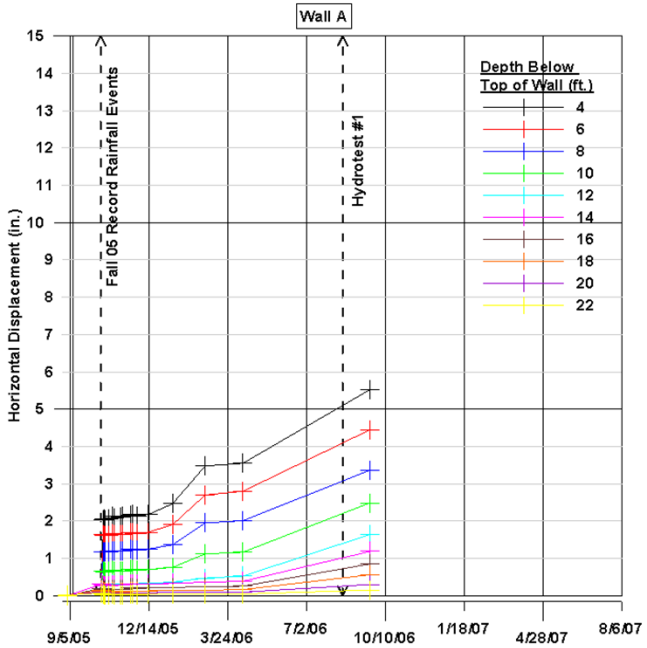
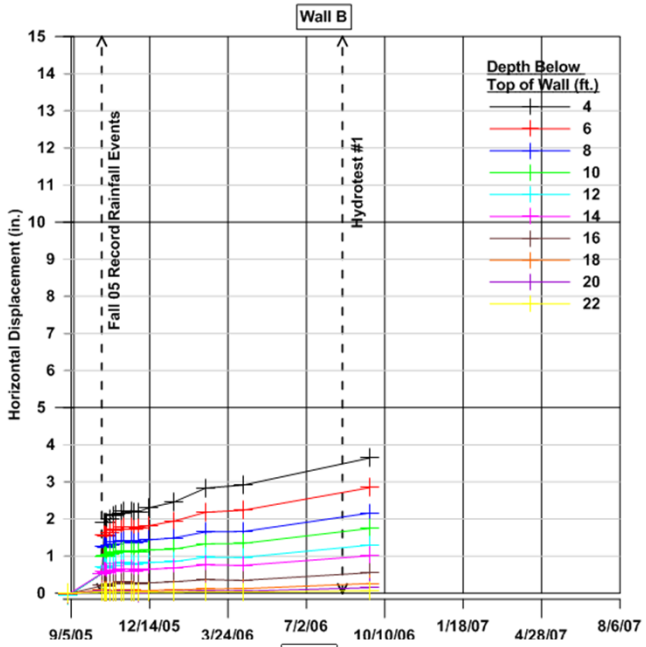
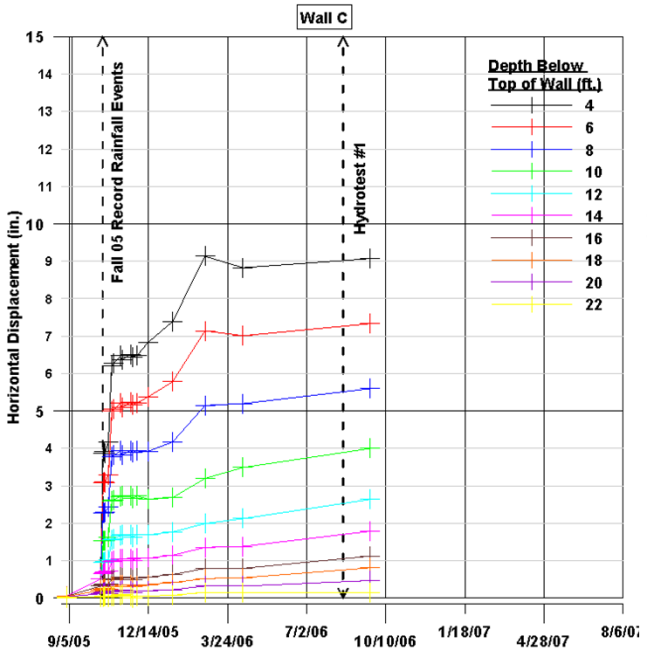
Full-Scale Field Test
NCHRP Project 24-22

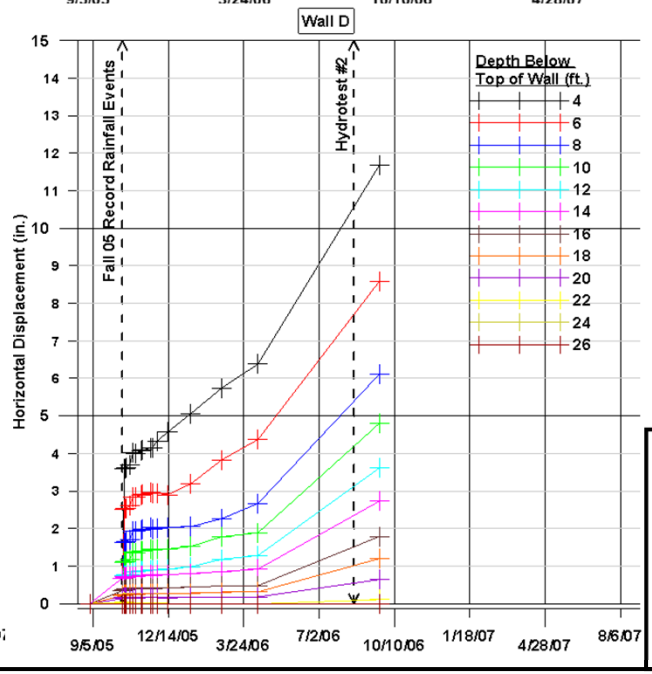
Effects of Oct. 2005
Record Rainfall
(Test Walls C and D)

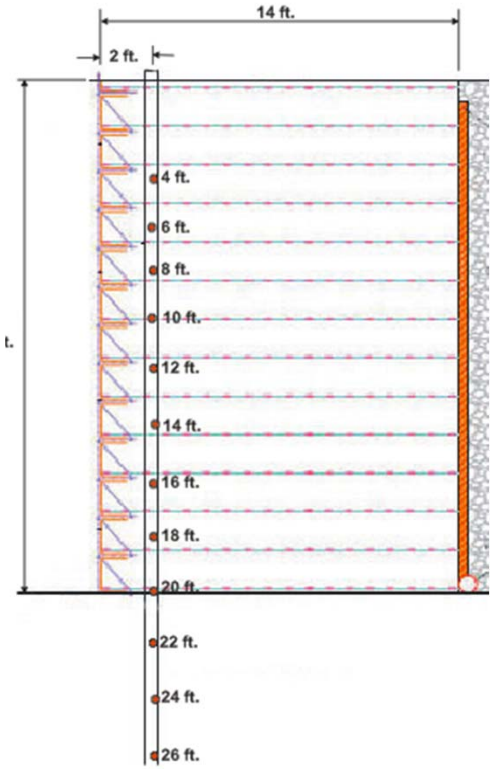
Figure B-17











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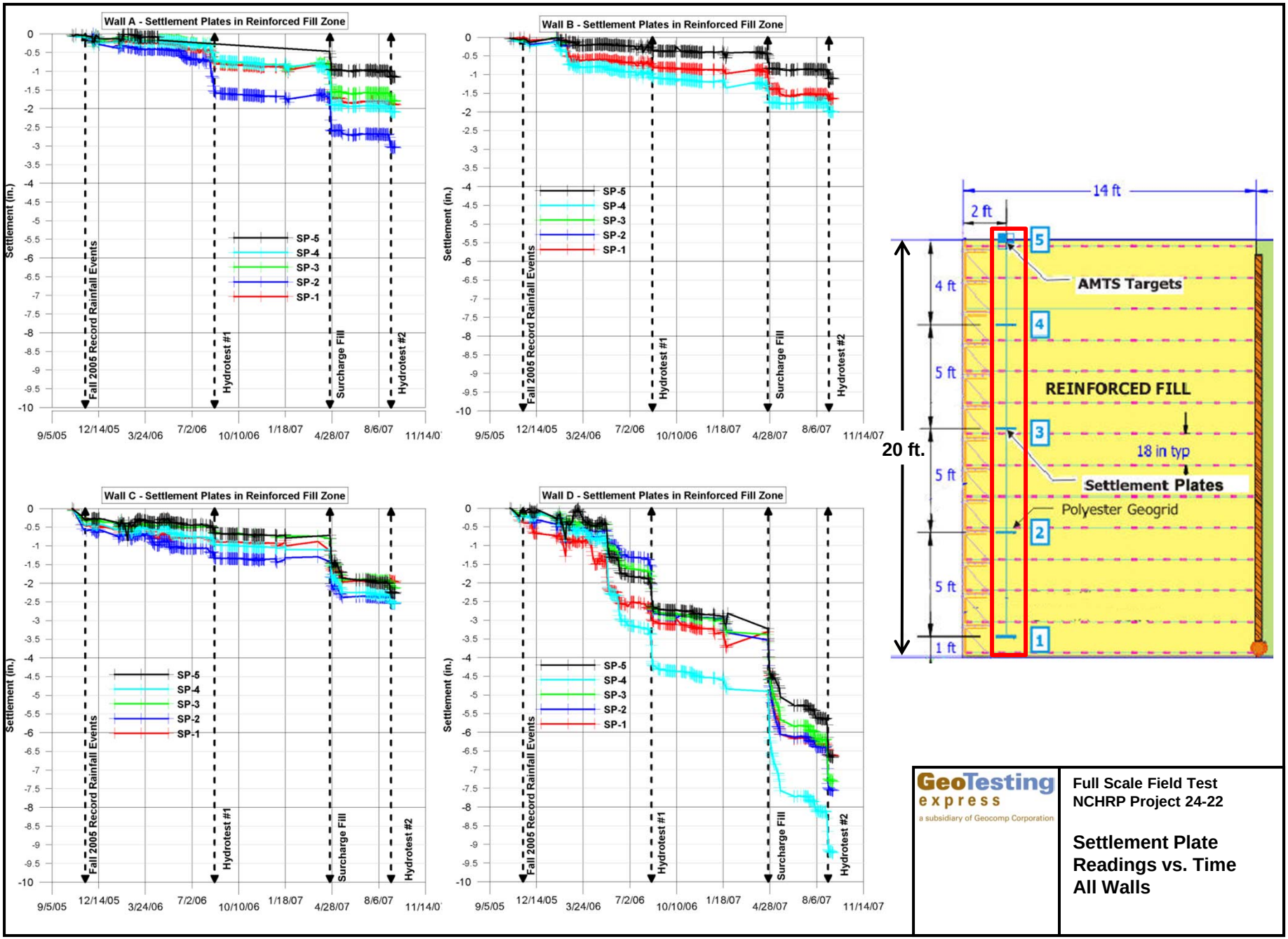
Full Scale Field Test
NCHRP Project 24-22

Inclinometer Readings vs. Time
All Walls

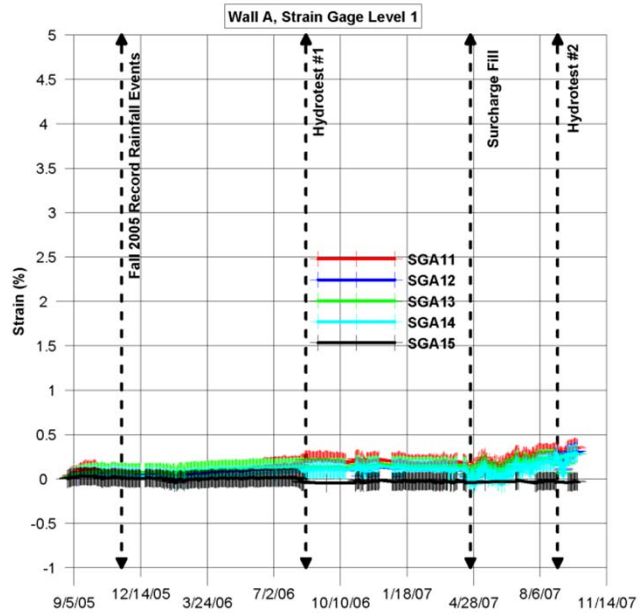
Figure B-18

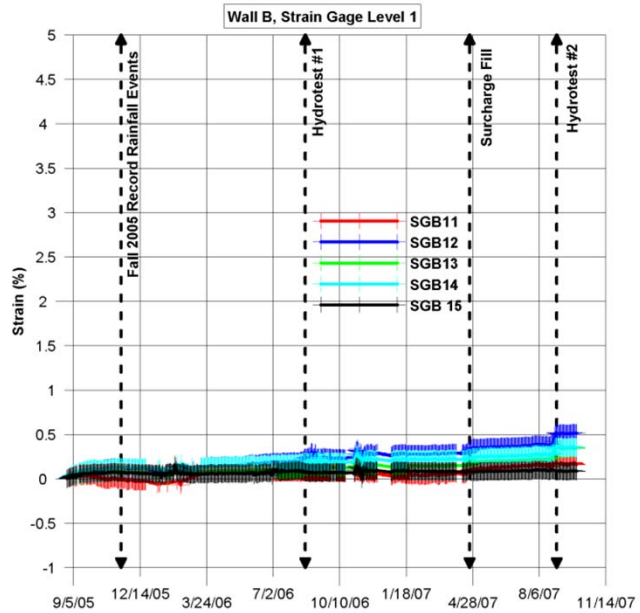


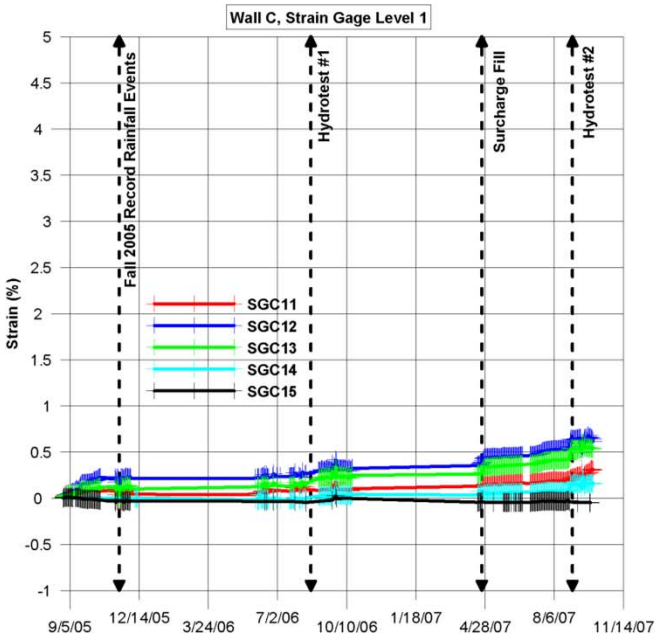
Full Scale Field Test
NCHRP Project 24-22

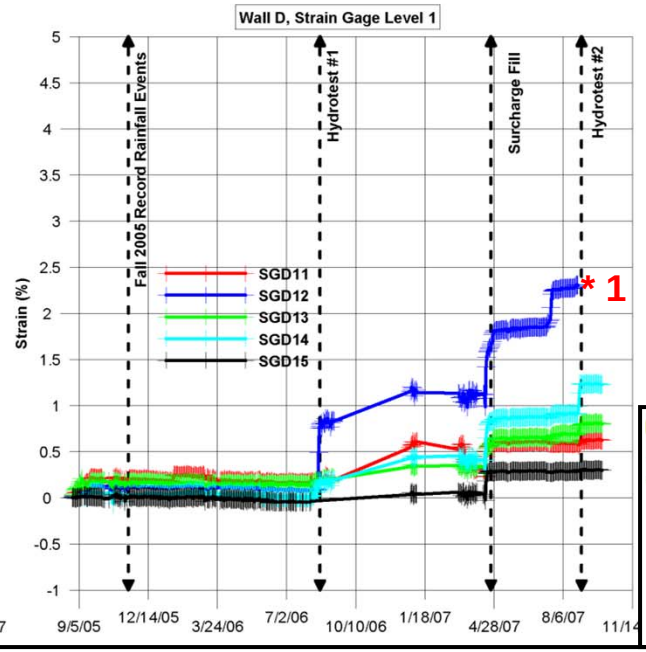
Settlement Plate Readings vs. Time All Walls

Figure B-19

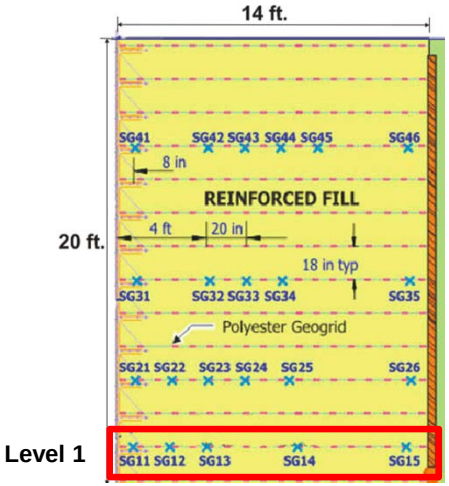




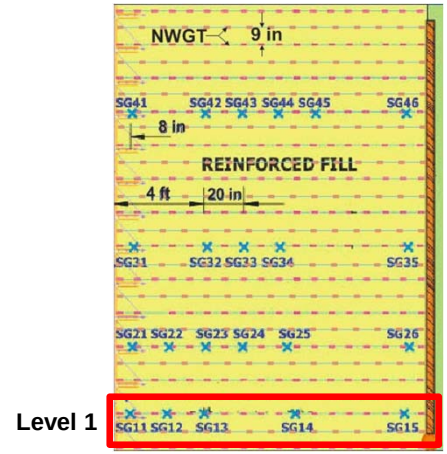




Walls A, B & C



Wall D



***1: SGD-12, 08/27/06 Gage Malfunction (See Plot)**

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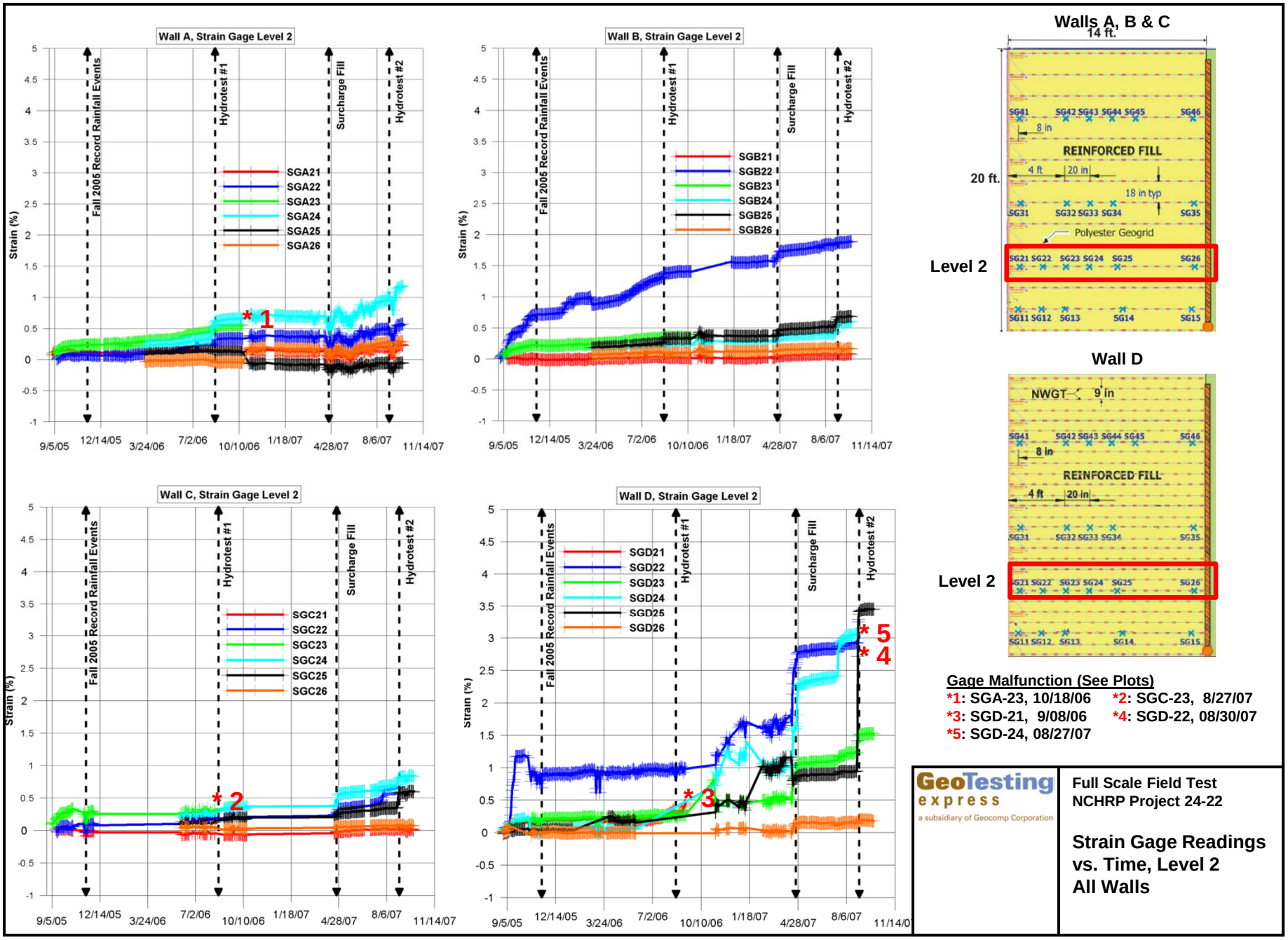
Full Scale Field Test
NCHRP Project 24-22

Strain Gage Readings vs. Time, Level 1 All Walls

Figure B-20


Wall A, Strain Gage Level 2
Wall B, Strain Gage Level 2
Wall C, Strain Gage Level 2
Wall D, Strain Gage Level 2
Strain (%)
Fall 2005 Record Rainfall Events
Hydrotest #1
Surcharge Fill
Hydrotest #2
SGA21
SGA22
SGA23
SGA24
SGA25
SGA26
SGB21
SGB22
SGB23
SGB24
SGB25
SGB26
SGC21
SGC22
SGC23
SGC24
SGC25
SGC26
SGD21
SGD22
SGD23
SGD24
SGD25
SGD26
* 1
* 2
* 3
* 4
* 5
9/5/05
12/14/05
3/24/06
7/2/06
10/10/06
1/18/07
4/28/07
8/6/07
11/14/07
Walls A, B & C
14 ft.
20 ft.
REINFORCED FILL
8 in
4 ft
20 in
18 in typ
Polyester Geogrid
Level 2
SG41 SG42 SG43 SG44 SG45 SG46
SG31 SG32 SG33 SG34 SG35
SG21 SG22 SG23 SG24 SG25 SG26
SG11 SG12 SG13 SG14 SG15
Wall D
NWGT
9 in
Level 2
Gage Malfunction (See Plots)
*1: SGA-23, 10/18/06
*2: SGC-23, 8/27/07
*3: SGD-21, 9/08/06
*4: SGD-22, 08/30/07
*5: SGD-24, 08/27/07
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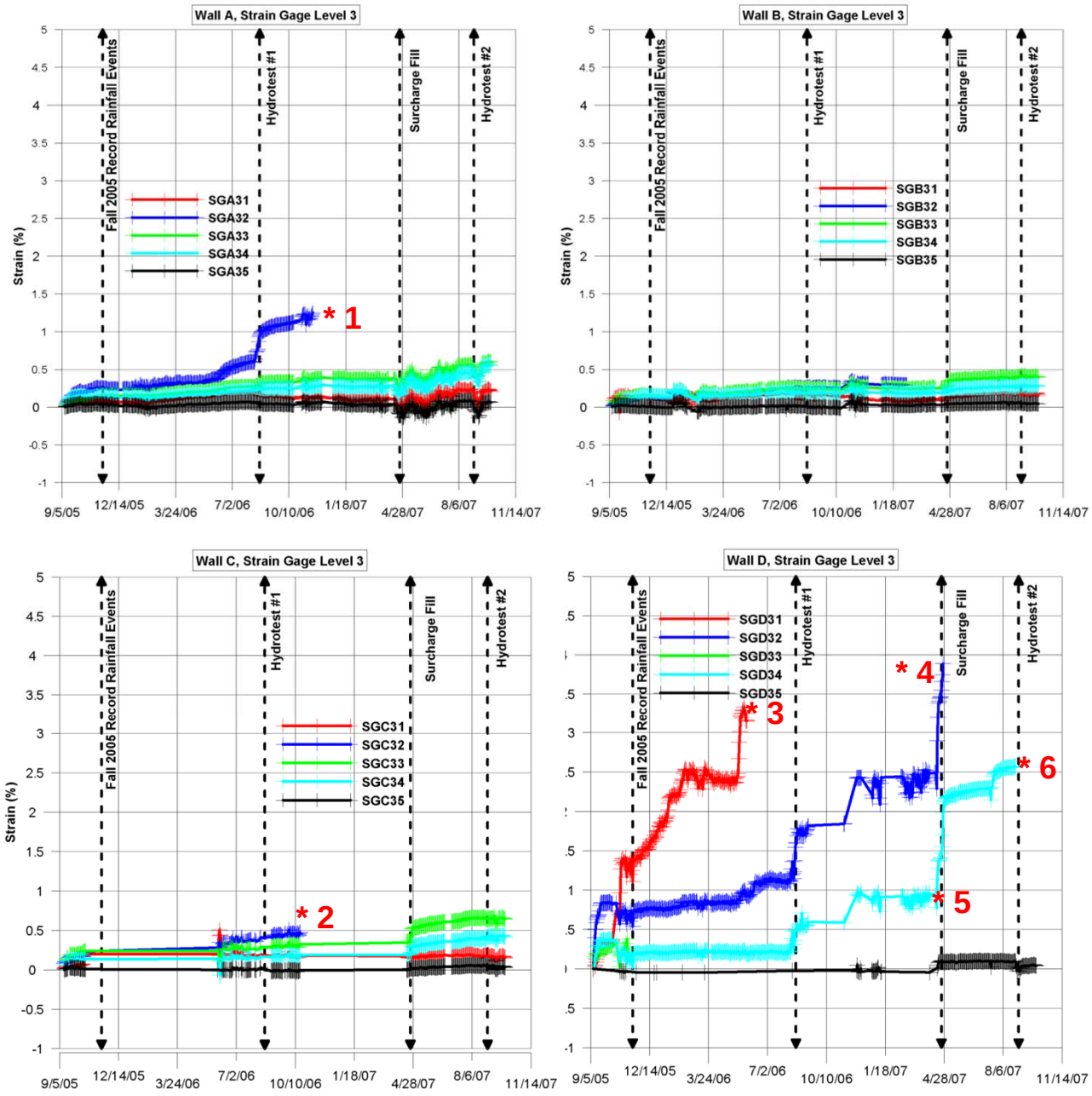
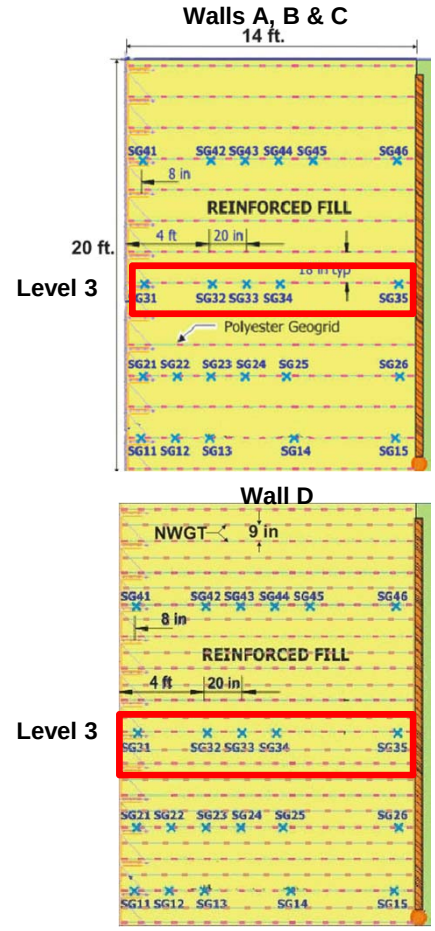
Full Scale Field Test
NCHRP Project 24-22
Strain Gage Readings vs. Time, Level 2 All Walls

Figure B-21





Gage Malfunction (See Plots)

*1: SGA-32, 10/18/06 *2: SGC-32, 10/18/06
*3: SGD-31, 5/28/06 *4: SGD-32, 4/26/07
*5: SGD-33, 4/26/07 *6: SGD-34, 08/27/07

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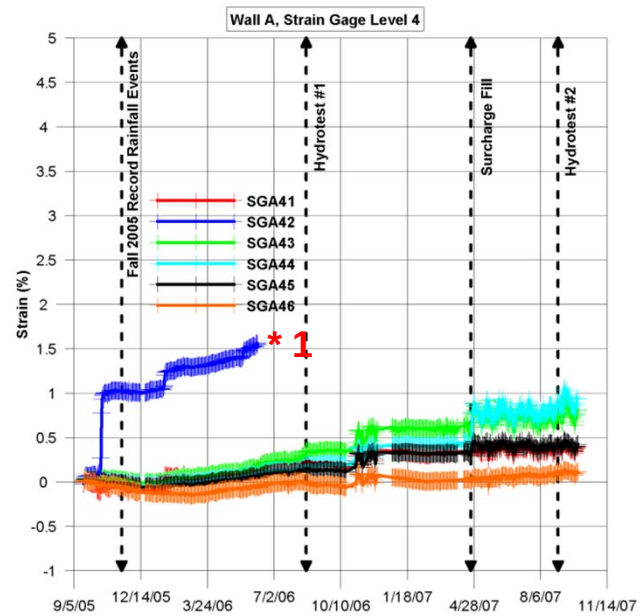
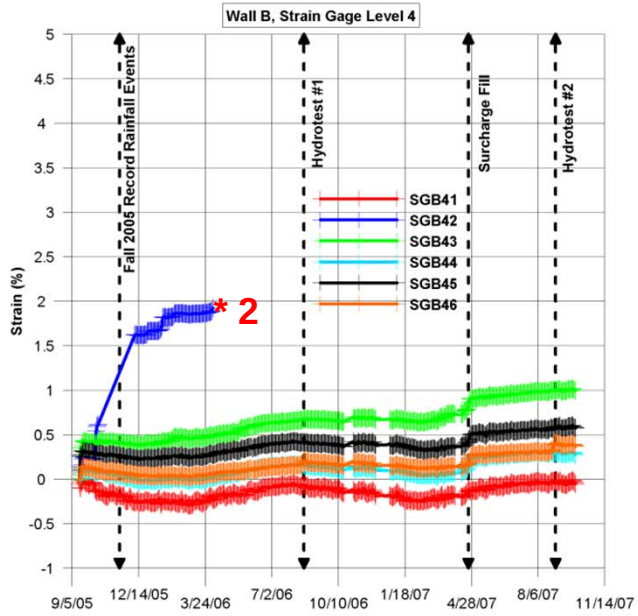
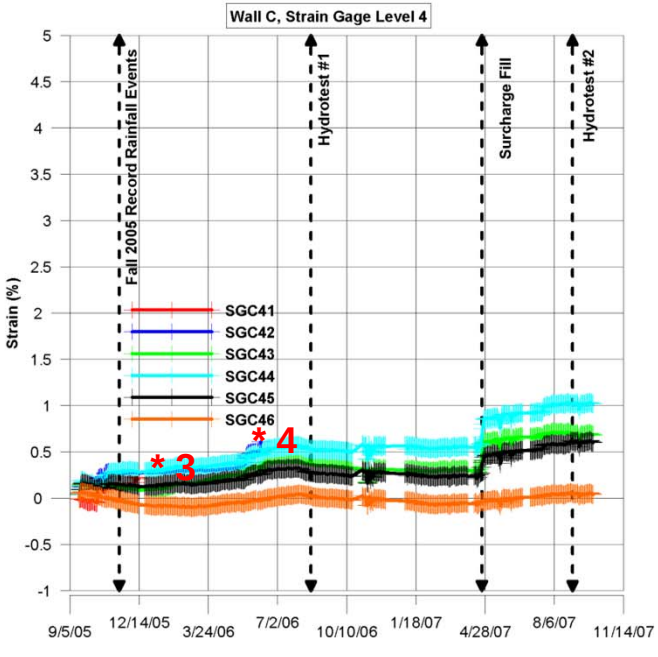
Full Scale Field Test
NCHRP Project 24-22

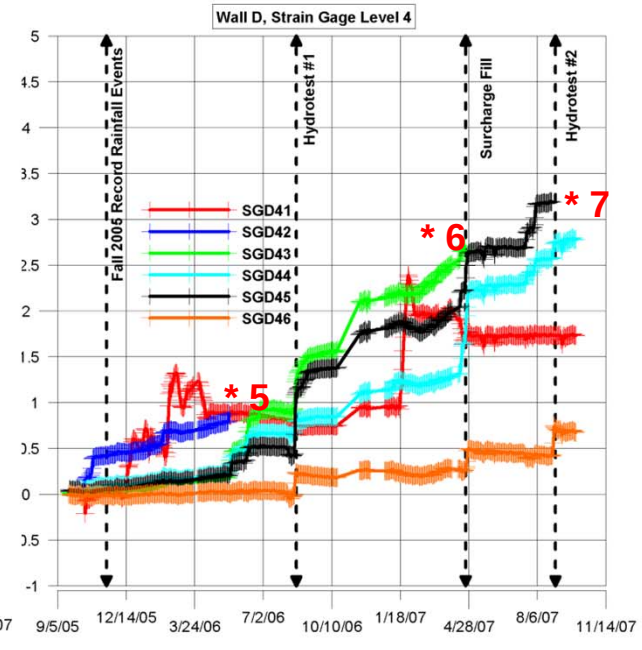
Strain Gage Readings vs. Time, Level 3 All Walls

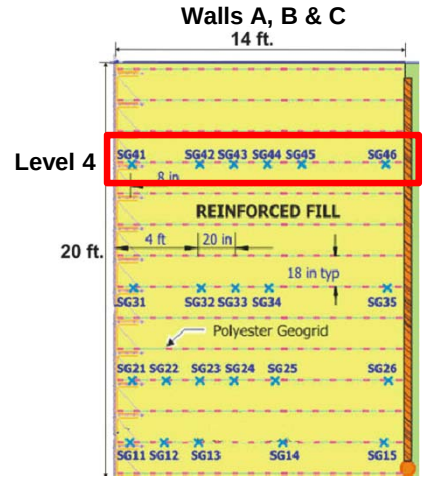
Figure B-22

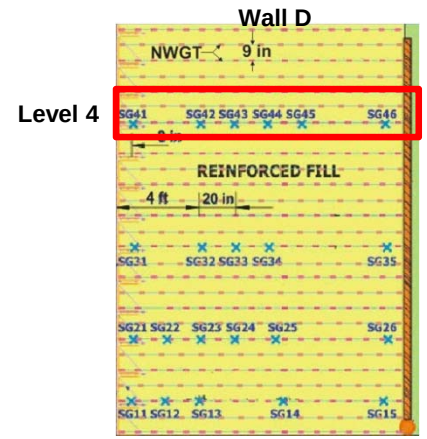












Gage Malfunction (See Plots)

*1: SGA-42, 6/07/06 *2: SGB-42, 4/04/06
*3: SGC-41, 12/31/05 *4: SGC-42, 6/07/06
*5: SGD-42, 5/13/06 *6: SGD-43, 4/22/07
*7: SGD-45, 8/27/07

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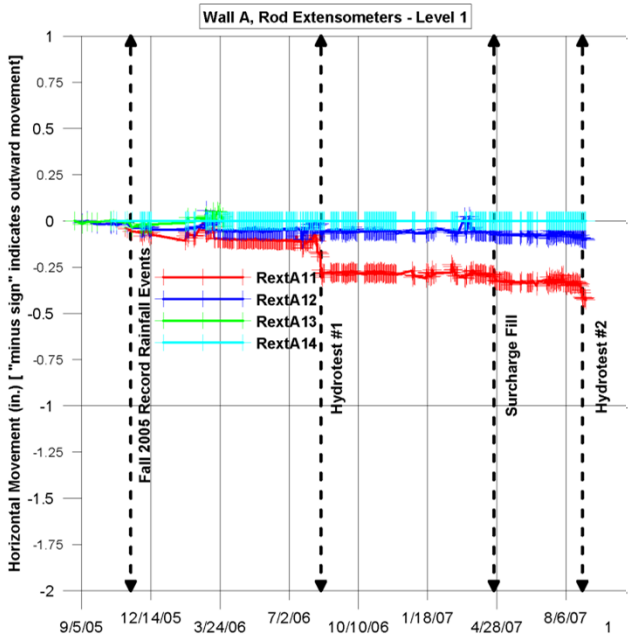
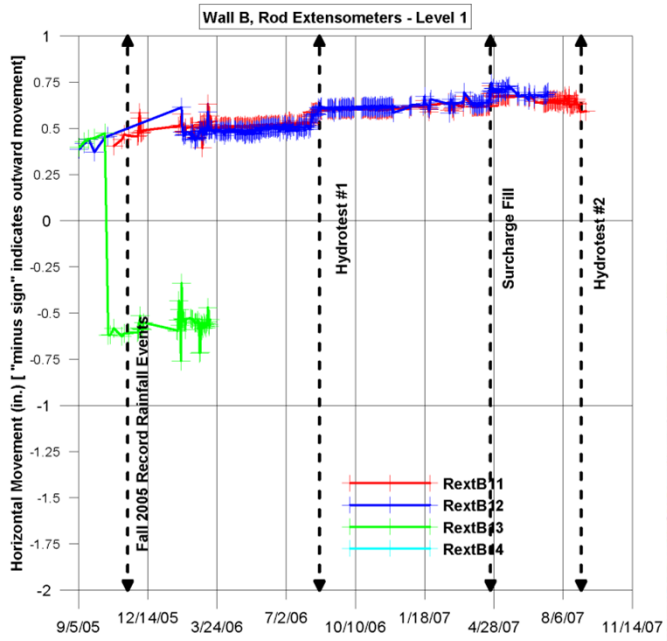
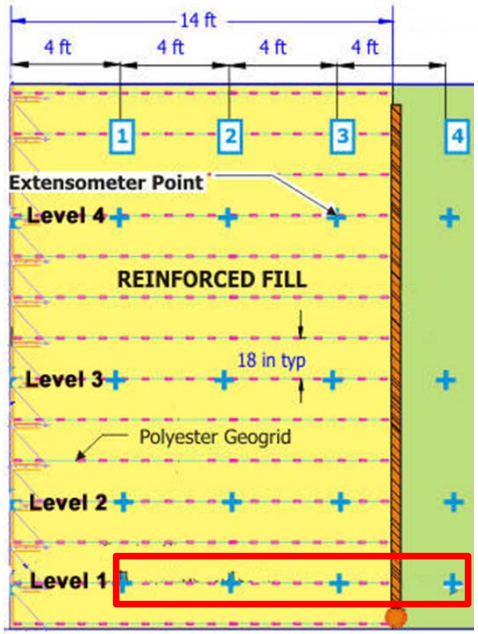
Full Scale Field Test
NCHRP Project 24-22

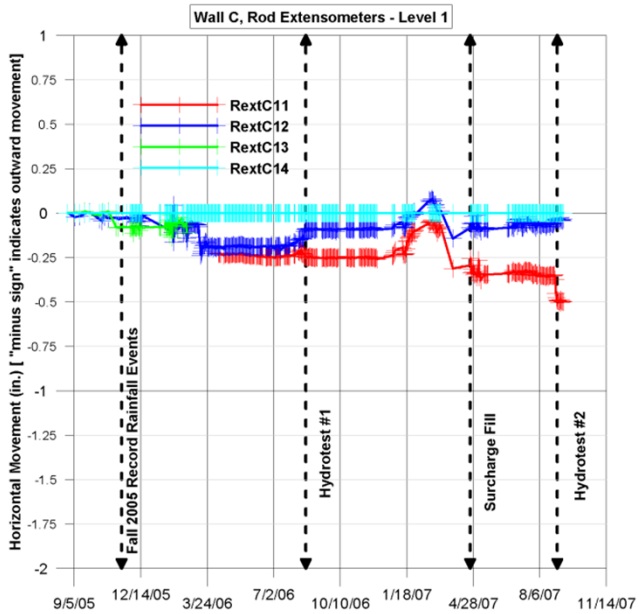
Strain Gage Readings vs. Time, Level 4 All Walls

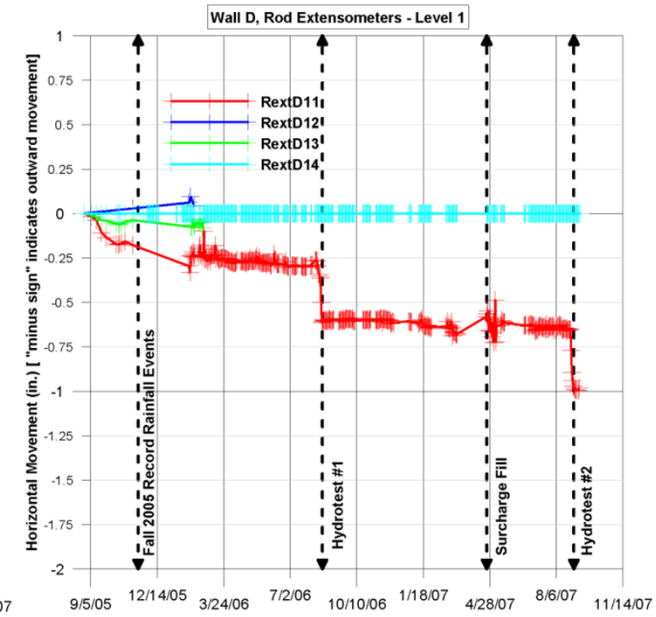
Figure B-23











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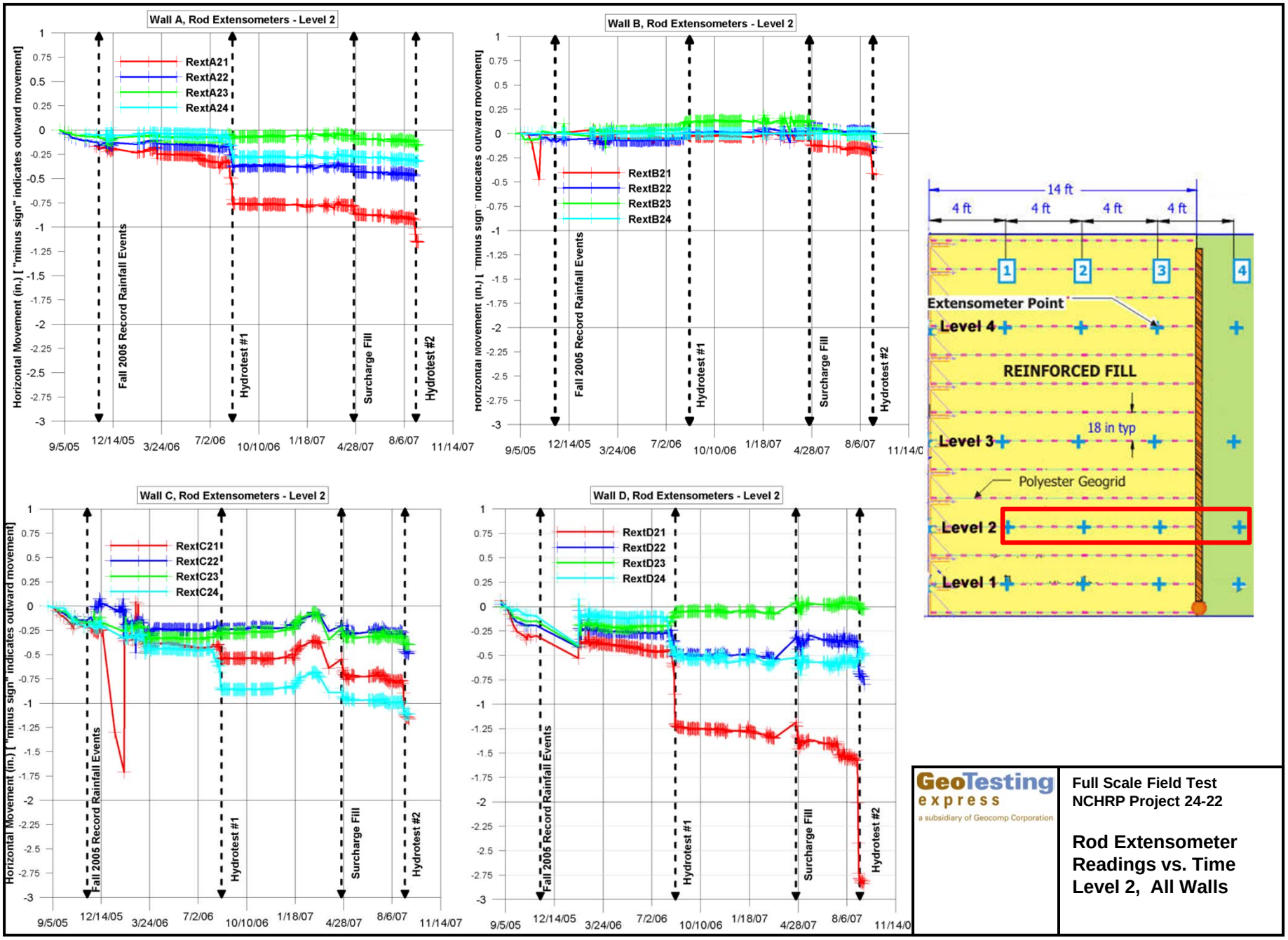
Full Scale Field Test
NCHRP Project 24-22

Rod Extensometer Readings vs. Time Level 1, All Walls

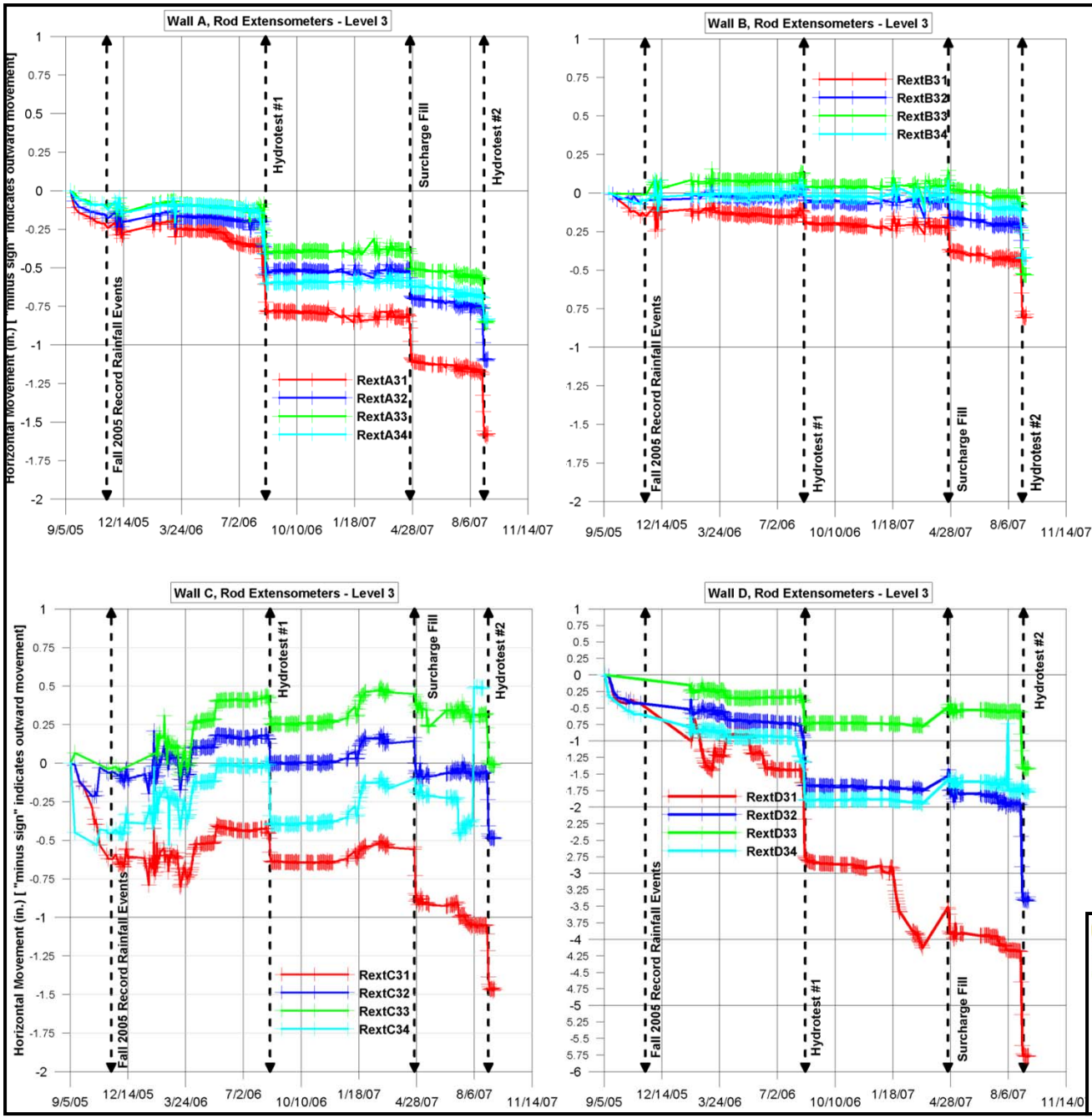
Figure B-24

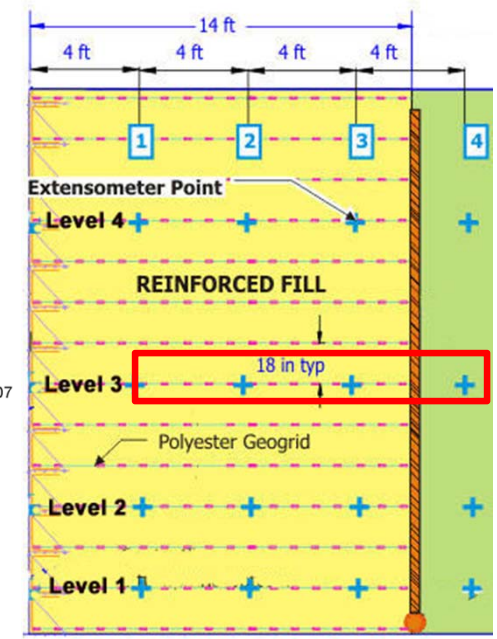


Full Scale Field Test
NCHRP Project 24-22

Rod Extensometer Readings vs. Time Level 2, All Walls

Figure B-25





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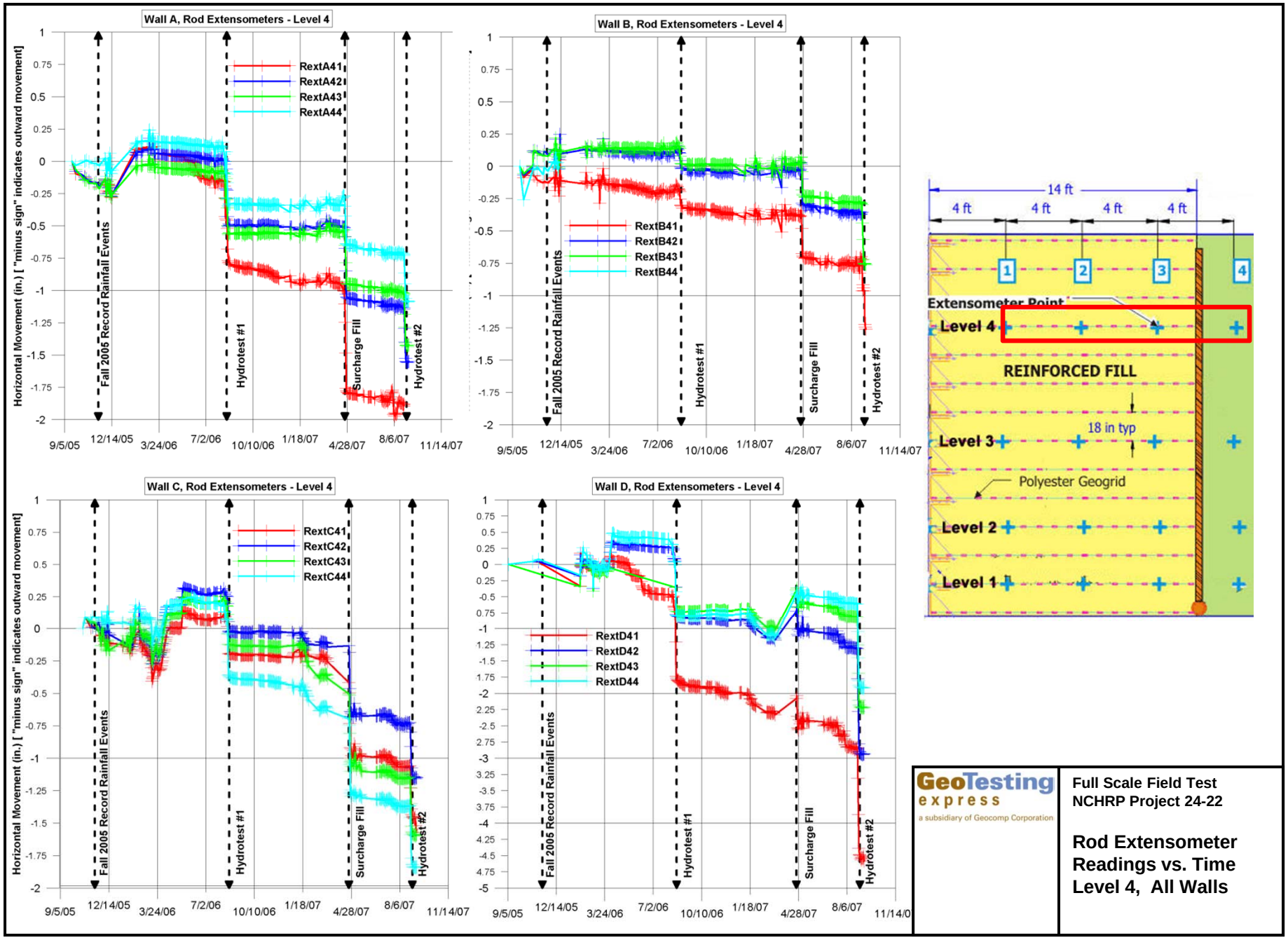
Full Scale Field Test
NCHRP Project 24-22

Rod Extensometer Readings vs. Time Level 3, All Walls

Figure B-26


Wall A, Rod Extensometers - Level 4
RextA41
RextA42
RextA43
RextA44
Wall B, Rod Extensometers - Level 4
RextB41
RextB42
RextB43
RextB44
Wall C, Rod Extensometers - Level 4
RextC41
RextC42
RextC43
RextC44
Wall D, Rod Extensometers - Level 4
RextD41
RextD42
RextD43
RextD44
Horizontal Movement (in.) ["minus sign" indicates outward movement]
Fall 2005 Record Rainfall Events
Hydrotest #1
Surcharge Fill
Hydrotest #2
9/5/05
12/14/05
3/24/06
7/2/06
10/10/06
1/18/07
4/28/07
8/6/07
11/14/07
14 ft
4 ft
Extensometer Point
Level 4
REINFORCED FILL
18 in typ
Level 3
Polyester Geogrid
Level 2
Level 1
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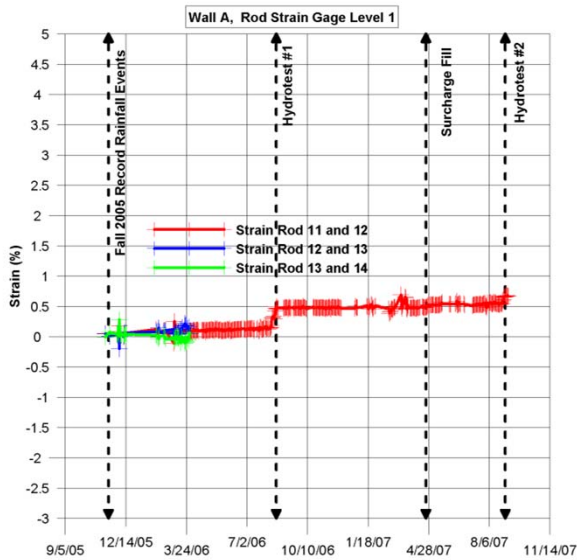
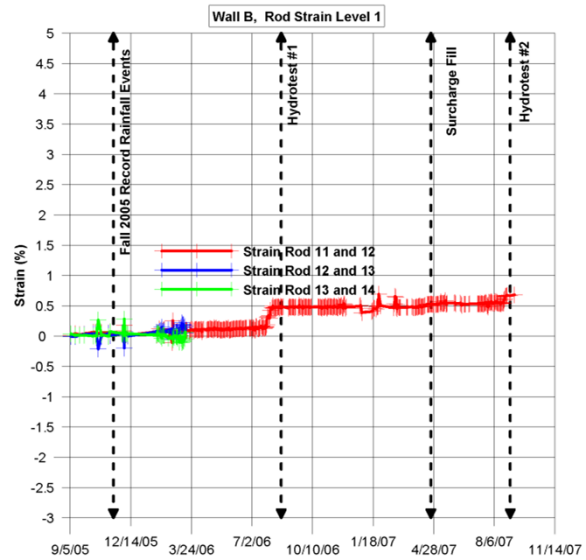
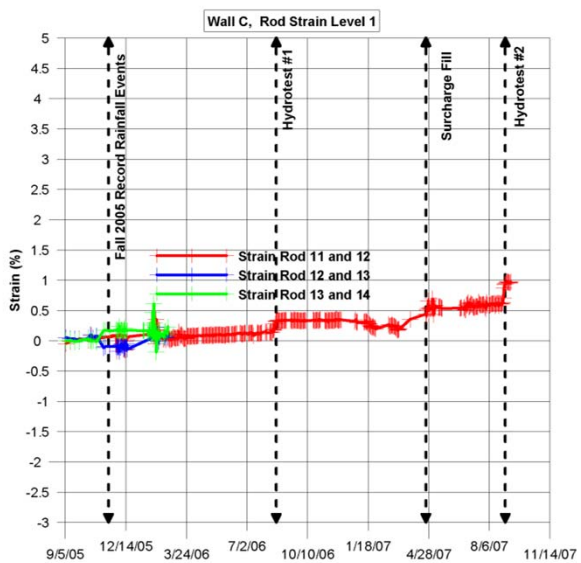
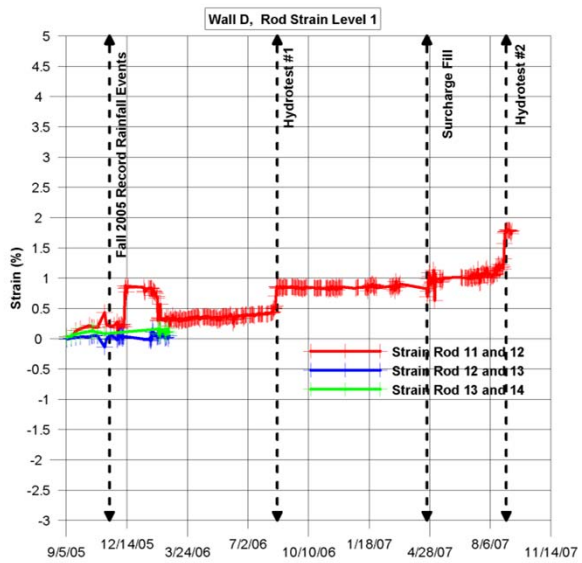
Full Scale Field Test
NCHRP Project 24-22
Rod Extensometer Readings vs. Time Level 4, All Walls

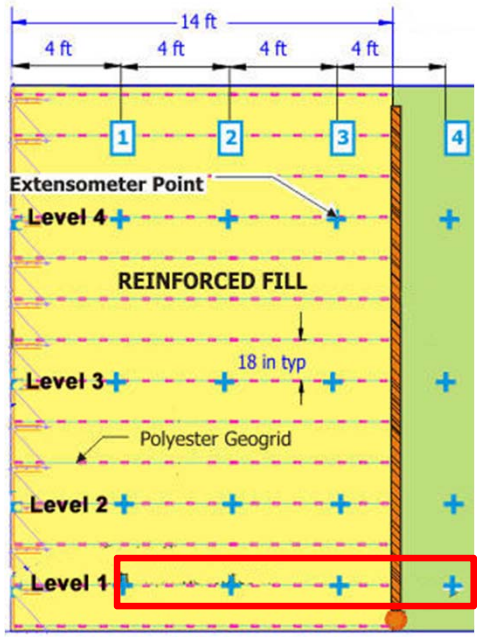
Figure B-27











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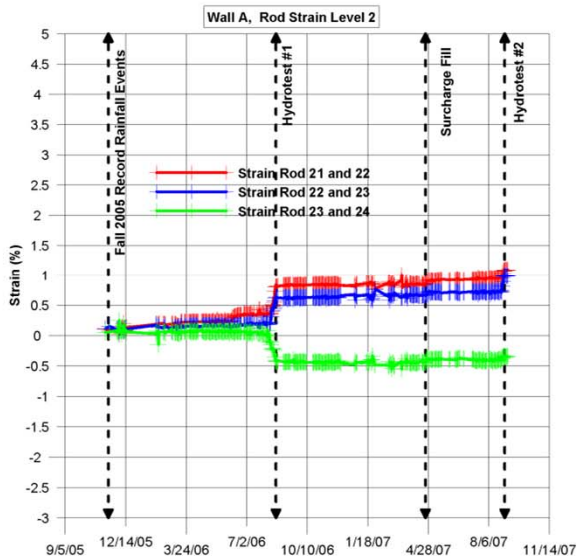
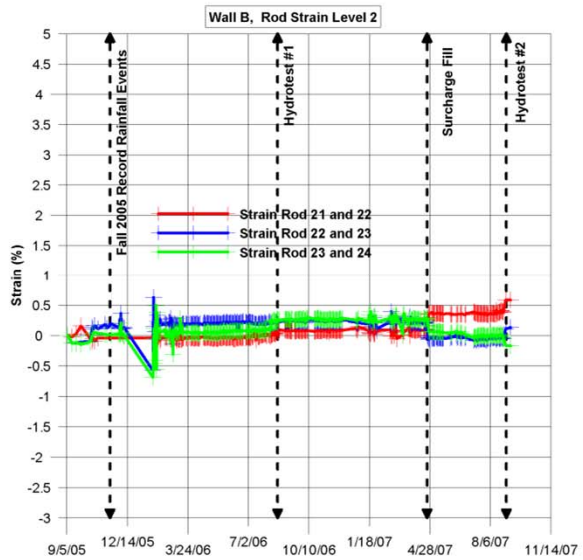
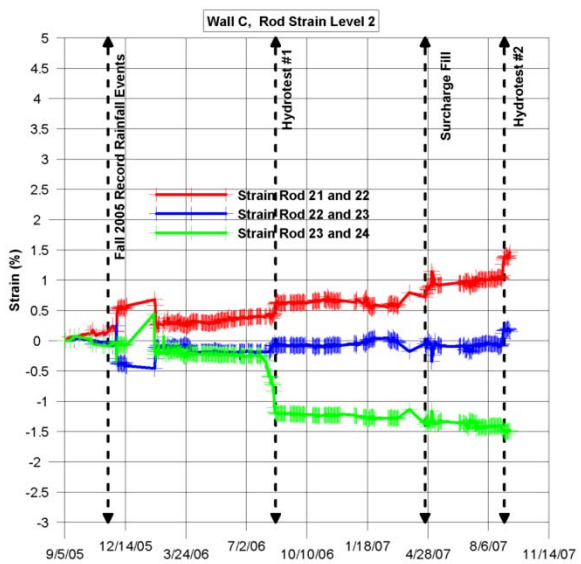
Full Scale Field Test
NCHRP Project 24-22

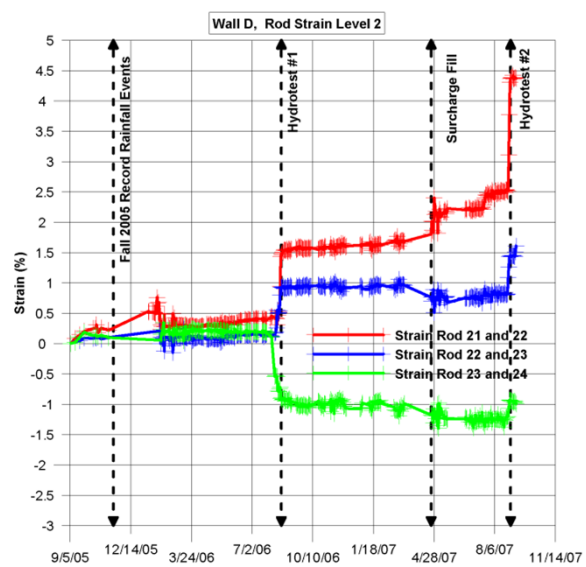
Rod Extensometer
Strain vs. Time
Level 1
All Walls

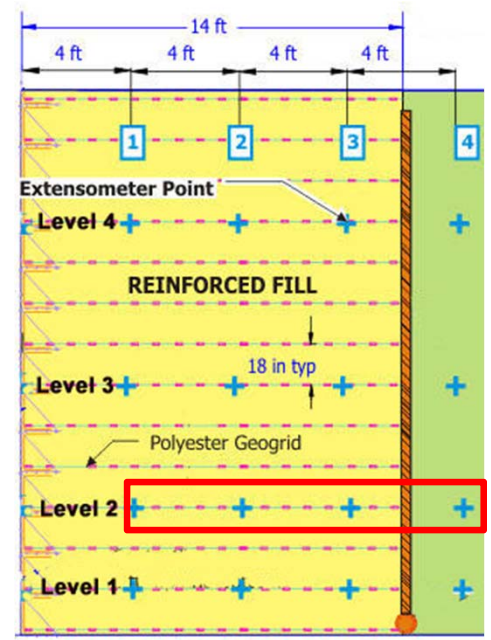
Figure B-28











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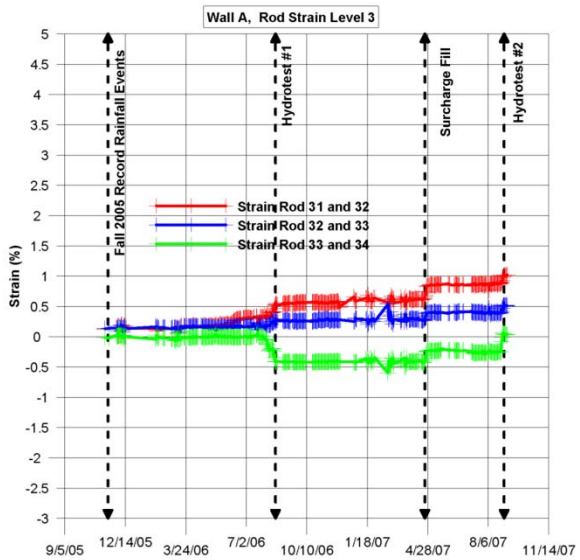
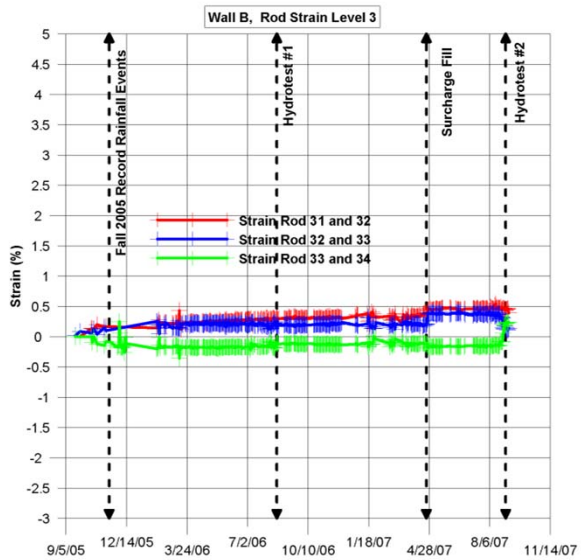
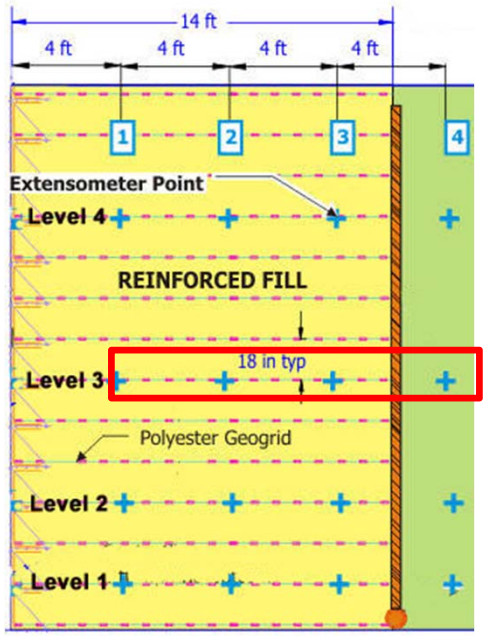
Full Scale Field Test
NCHRP Project 24-22

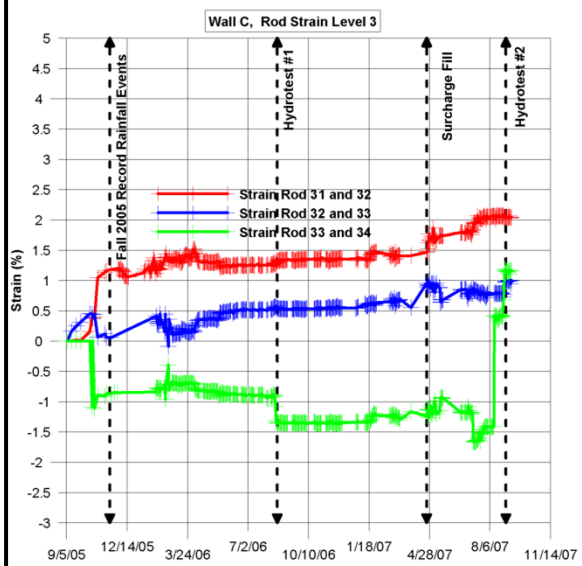
Rod Extensometer
Strain vs. Time
Level 2
All Walls

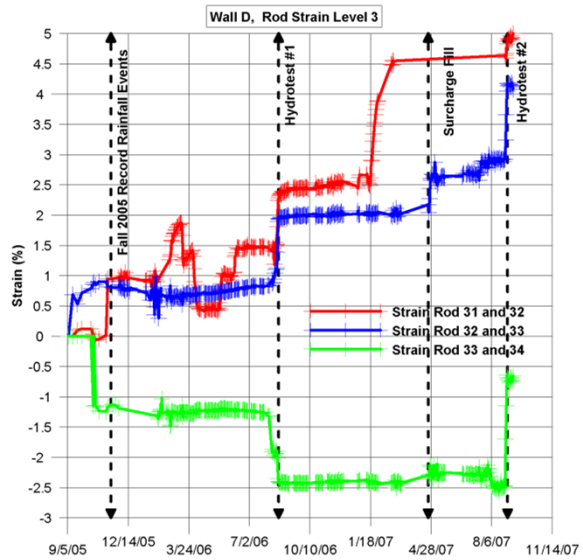
Figure B-29











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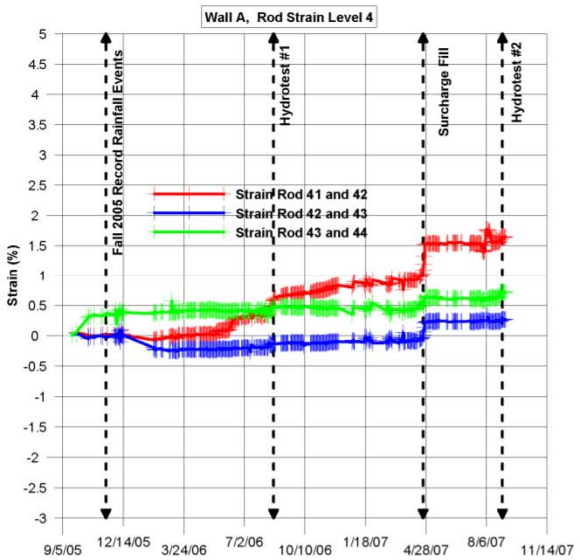
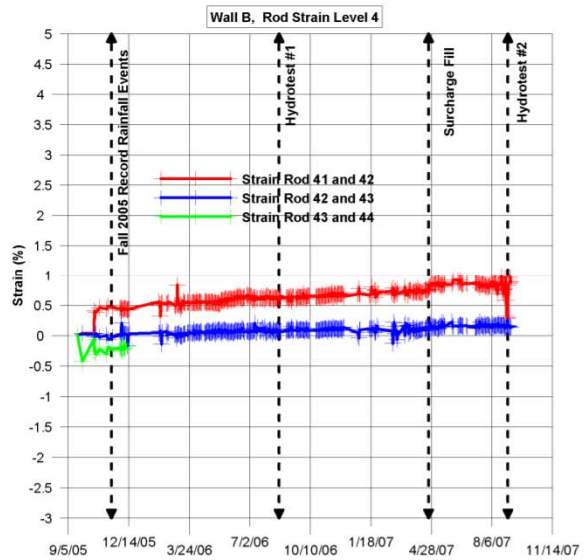
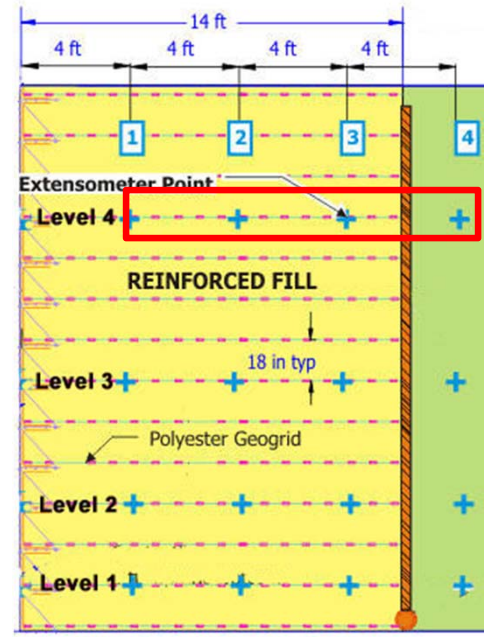
Full Scale Field Test
NCHRP Project 24-22

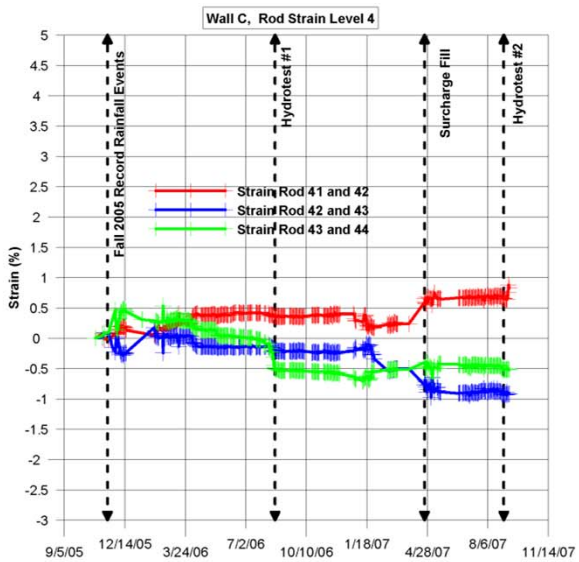
Rod Extensometer
Strain vs. Time
Level 3
All Walls

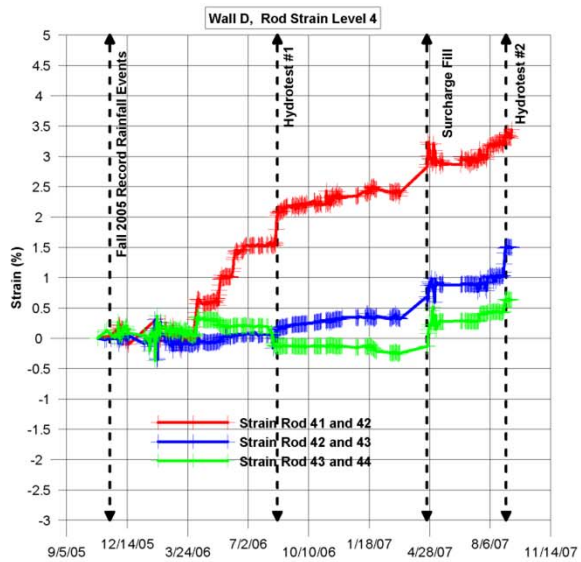
Figure B-30











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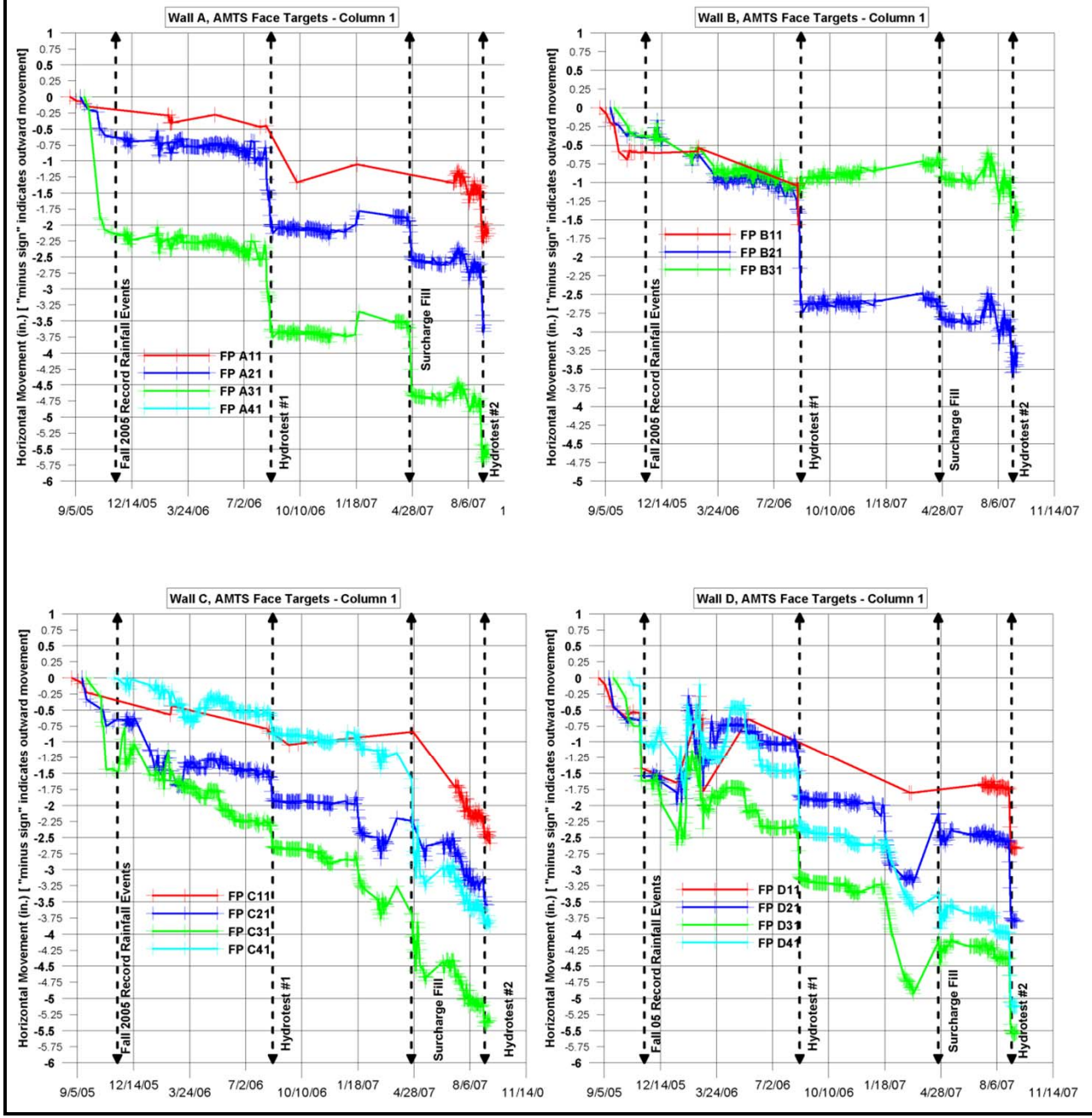
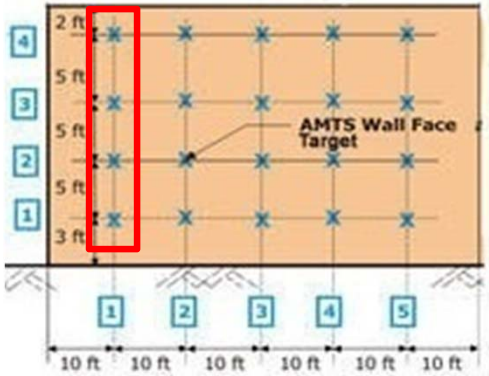
Full Scale Field Test
NCHRP Project 24-22

Rod Extensometer
Strain vs. Time
Level 4
All Walls

Figure B-31





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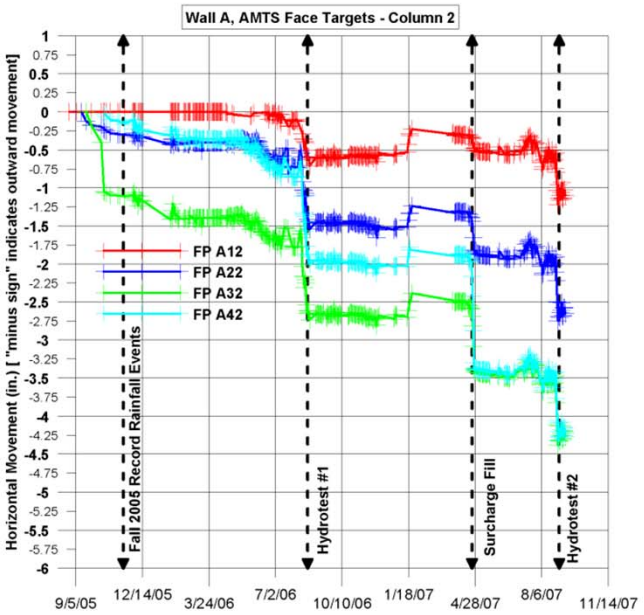
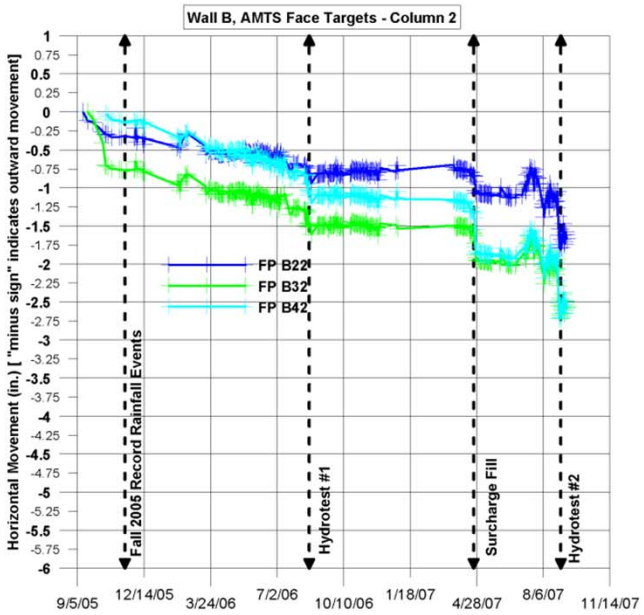
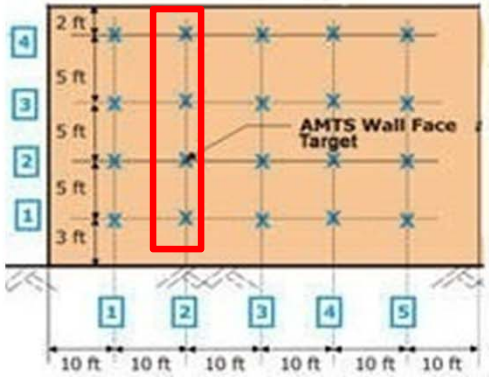
Full Scale Field Test
NCHRP Project 24-22

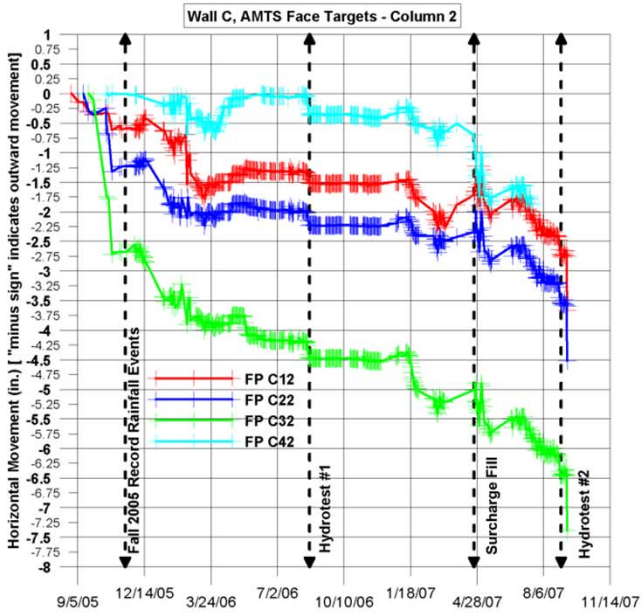
Lateral Face Movement vs. Time, Column 1 All Walls

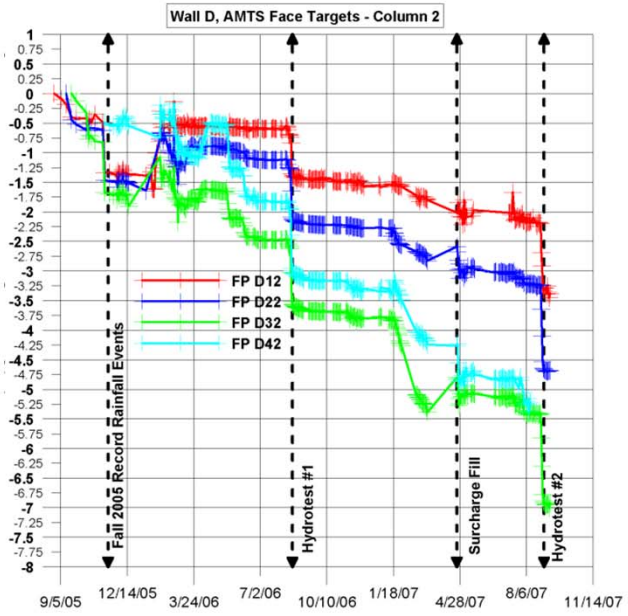
Figure B-32











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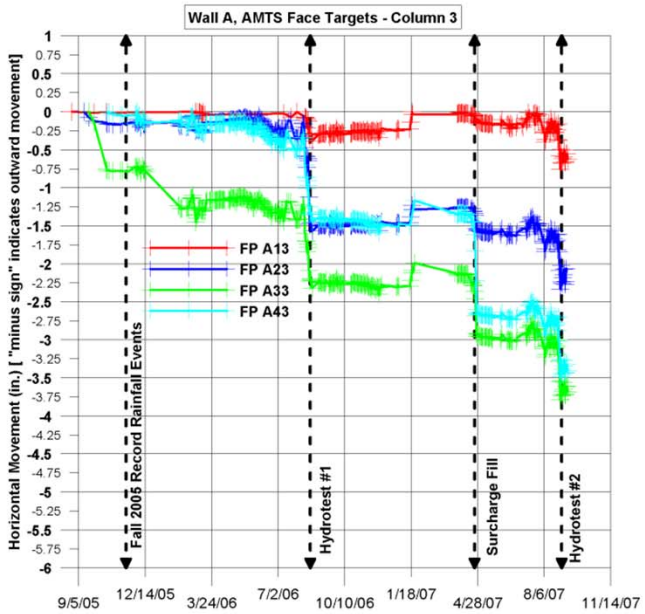
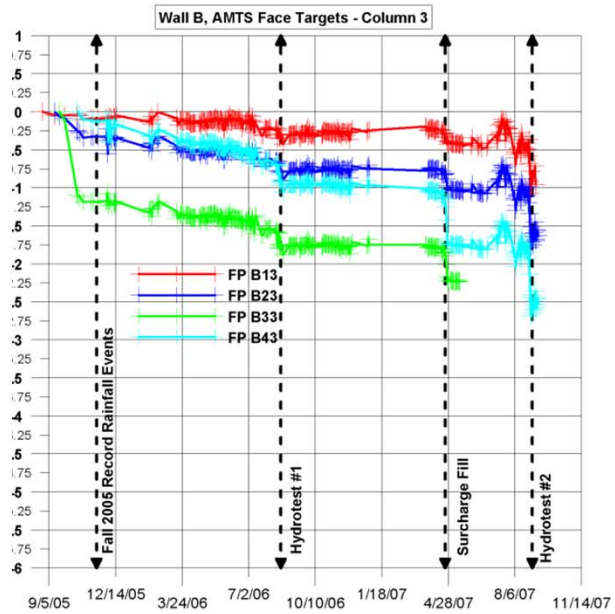
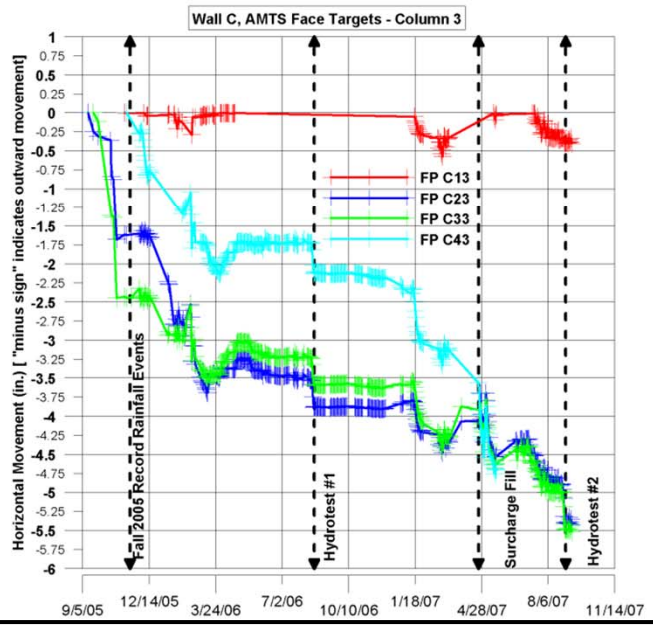
Full Scale Field Test
NCHRP Project 24-22

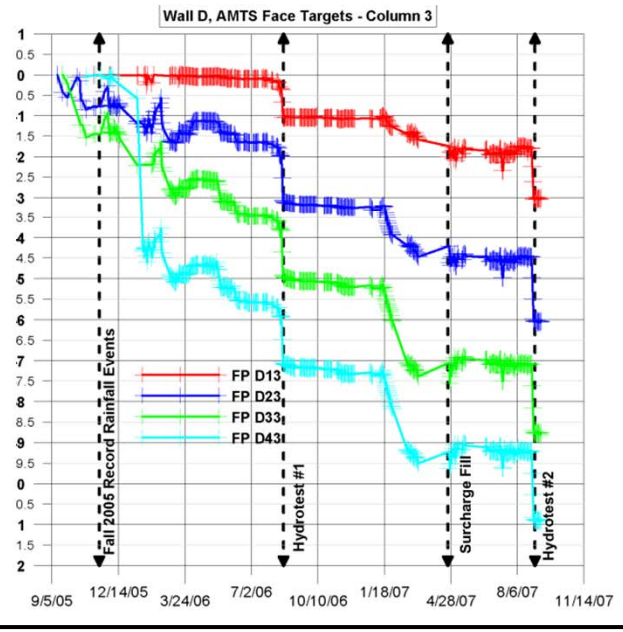
Lateral Face Movement vs. Time, Column 2 All Walls

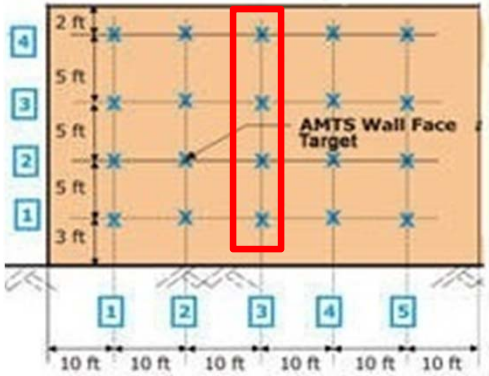
Figure B-33











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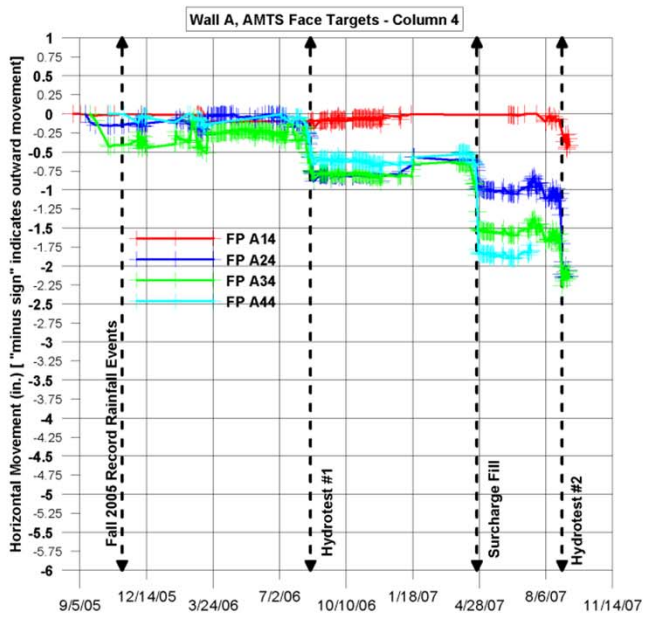
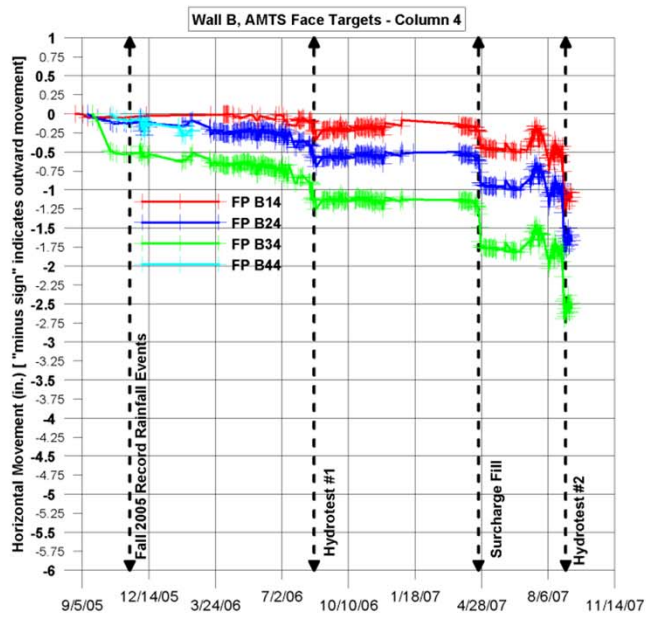
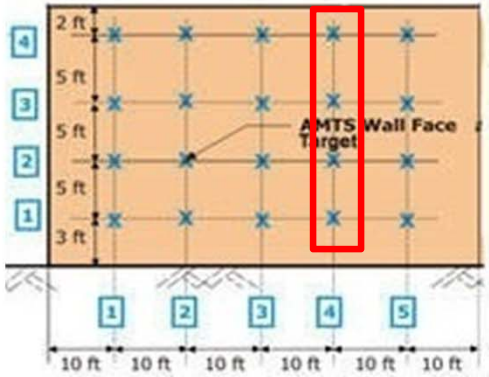
Full Scale Field Test
NCHRP Project 24-22

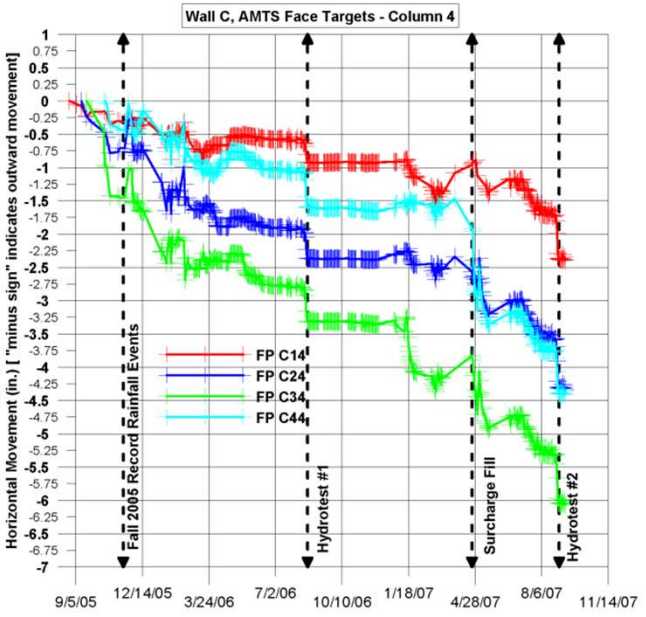
Lateral Face Movement vs. Time, Column 3 All Walls

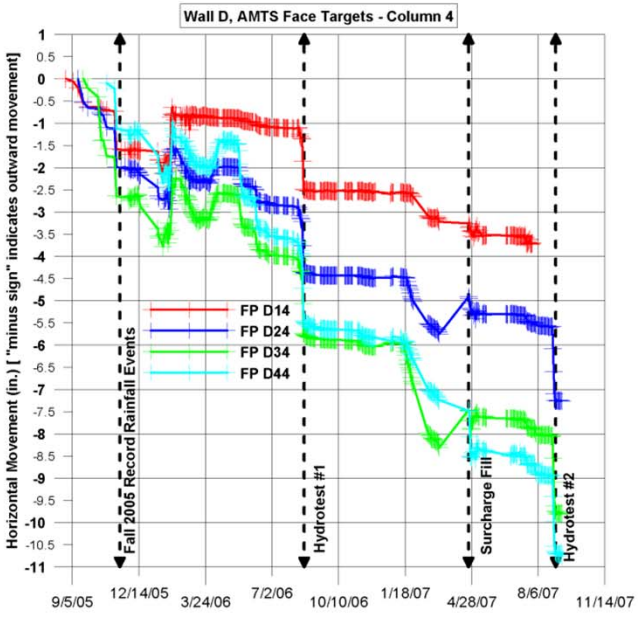
Figure B-34











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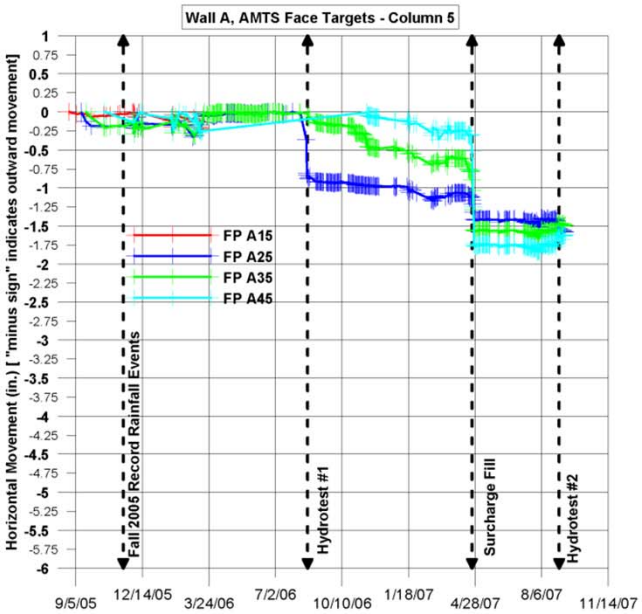
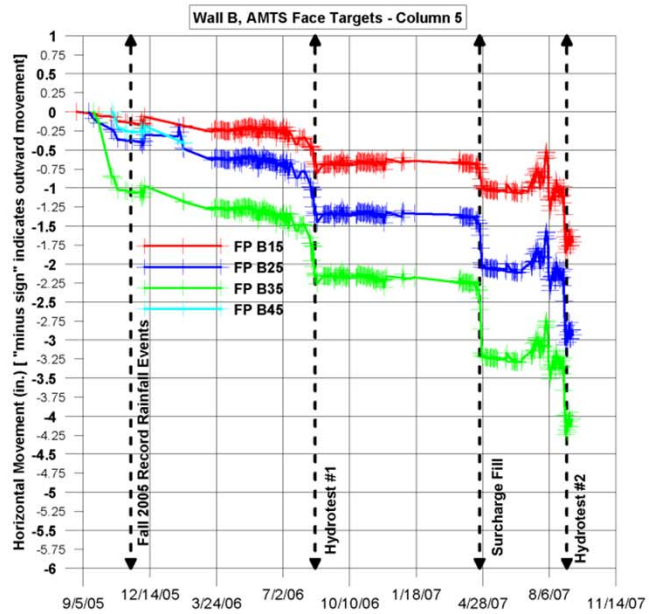
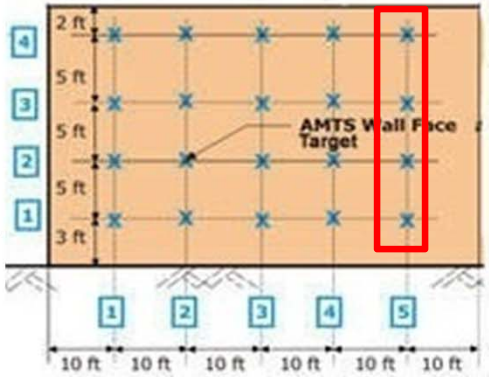
Full Scale Field Test
NCHRP Project 24-22

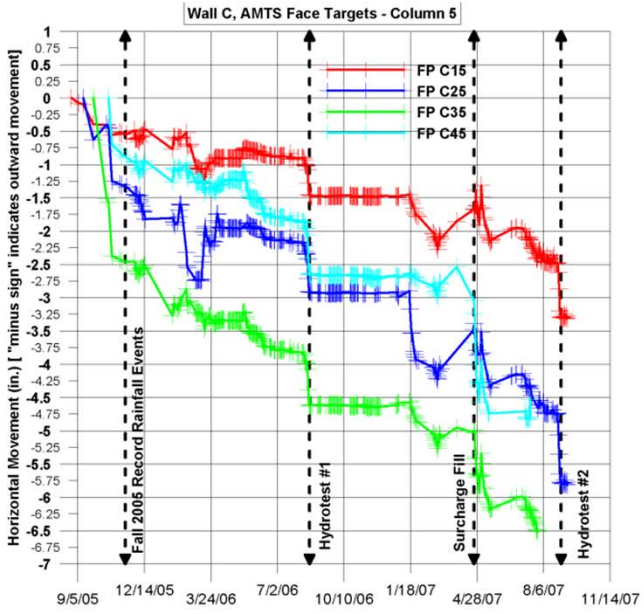
Lateral Face Movement vs. Time, Column 4 All Walls

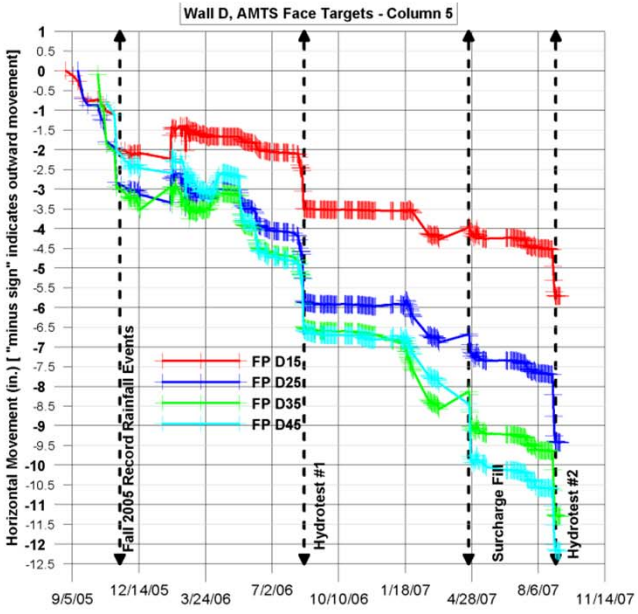
Figure B-35











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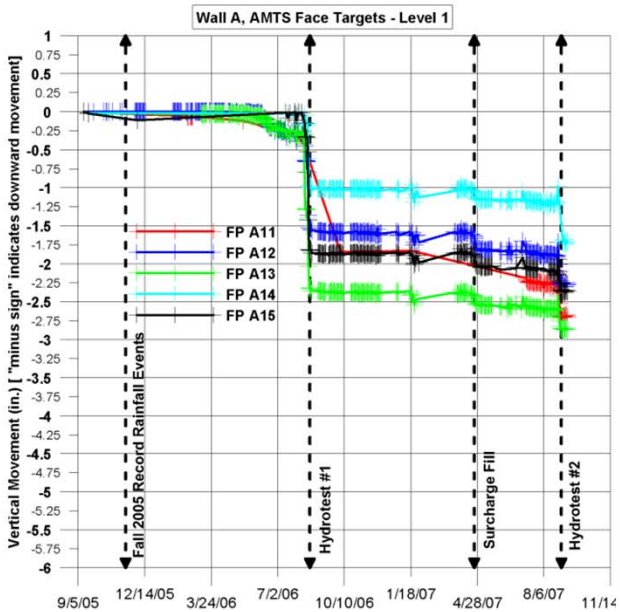
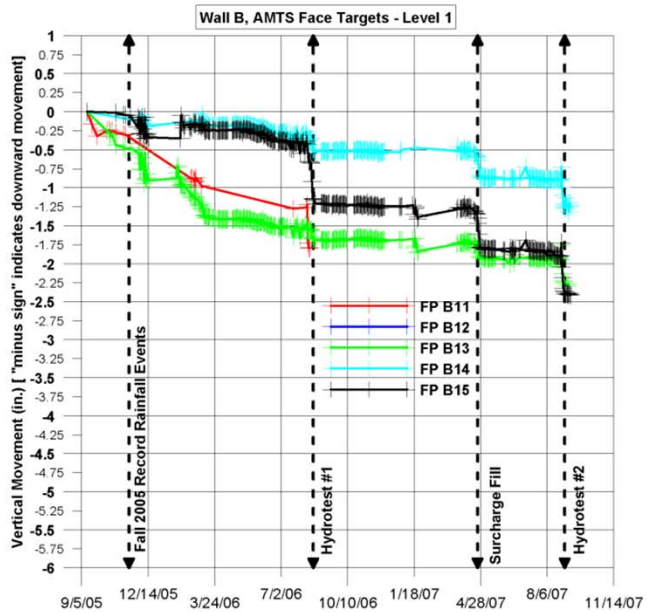
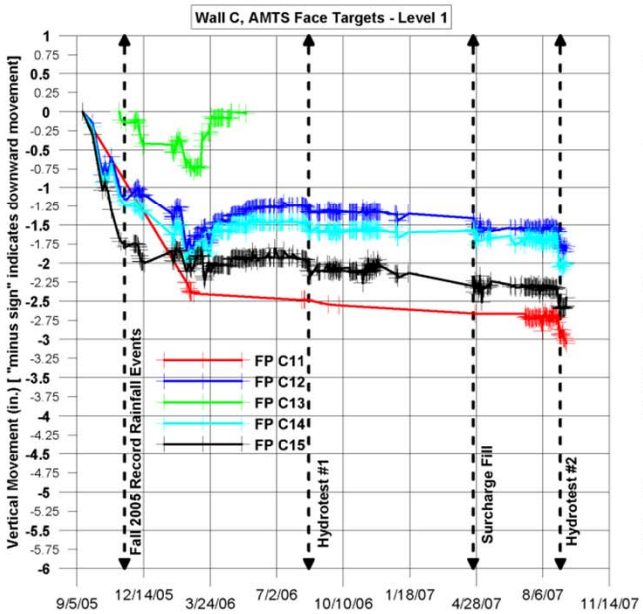
Full Scale Field Test
NCHRP Project 24-22

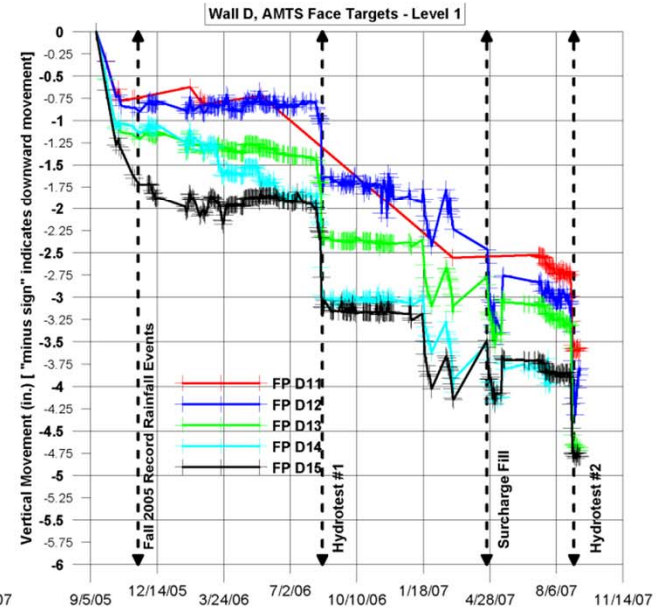
Lateral Face Movement vs. Time, Column 5 All Walls

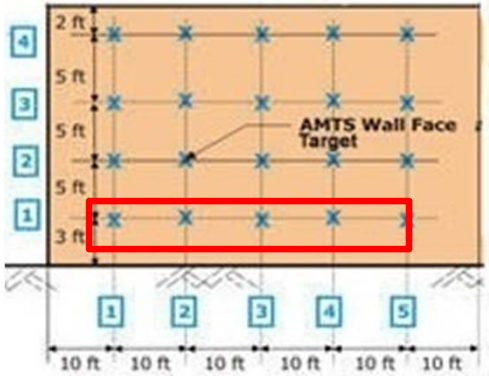
Figure B-36











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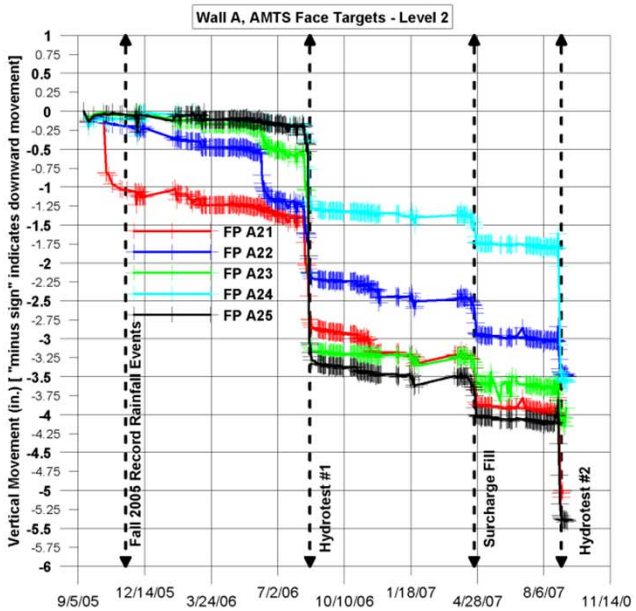
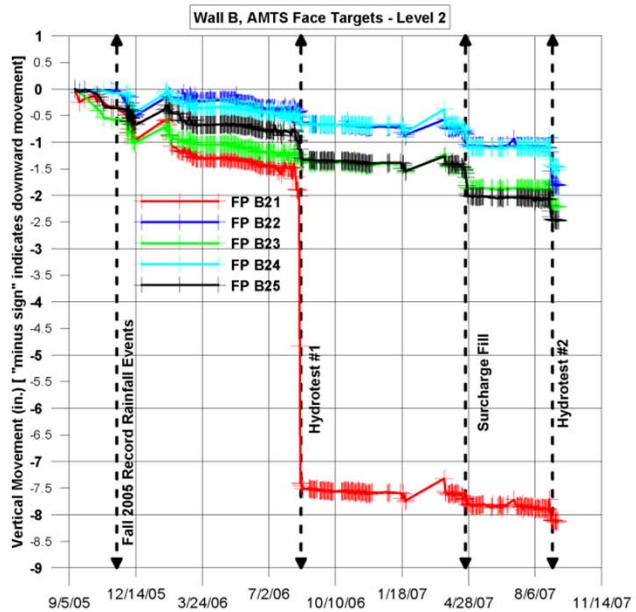
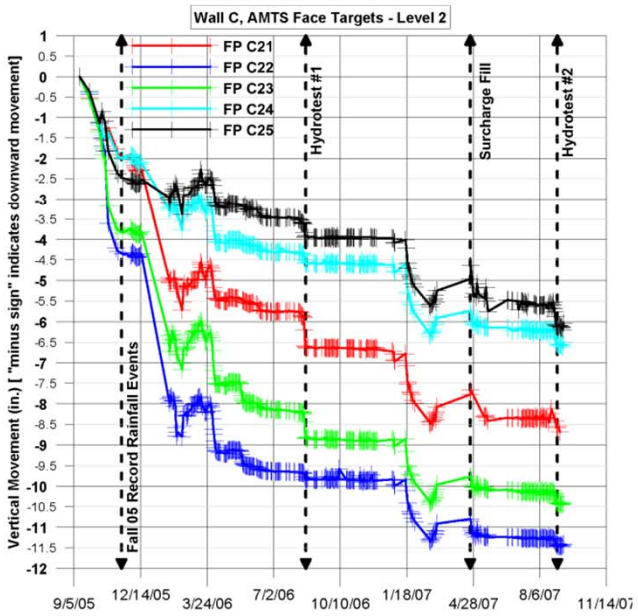
Full Scale Field Test
NCHRP Project 24-22

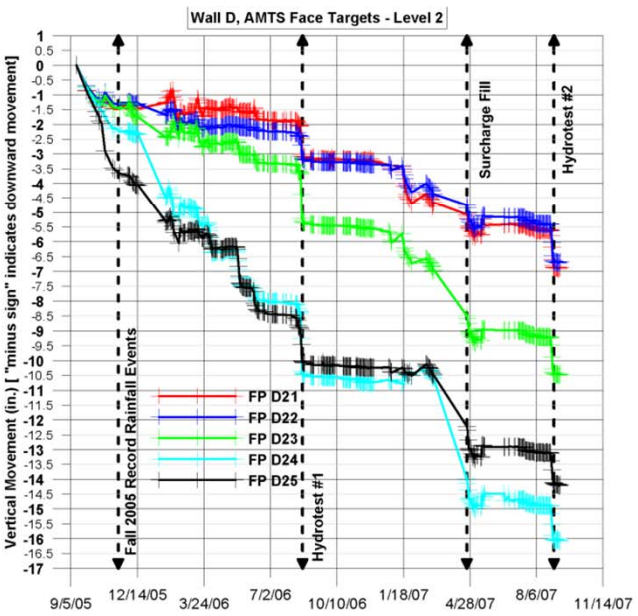
Vertical Face Movt. vs. Time, Level 1 All Walls

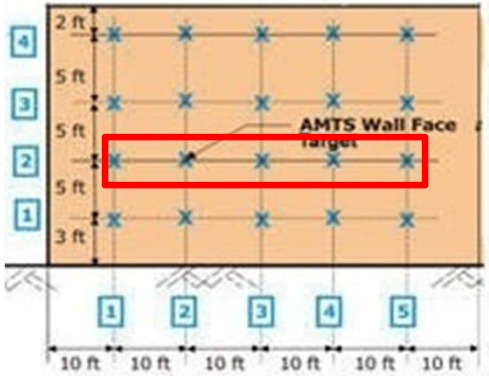
Figure B-37











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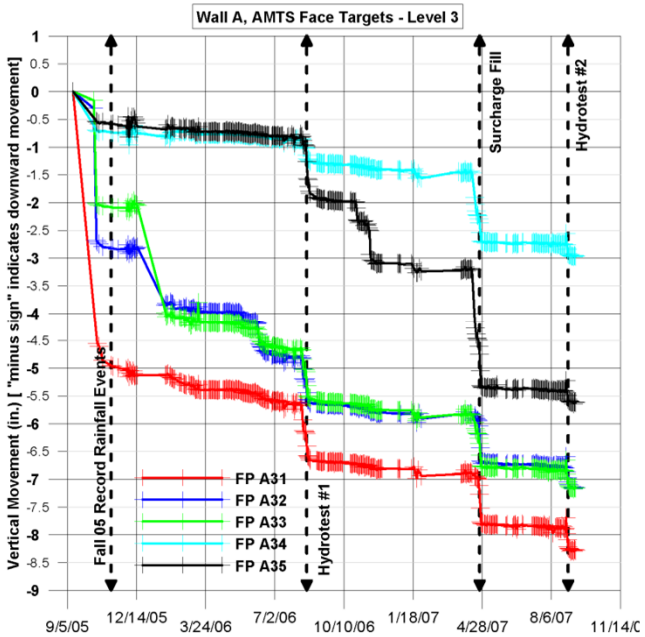
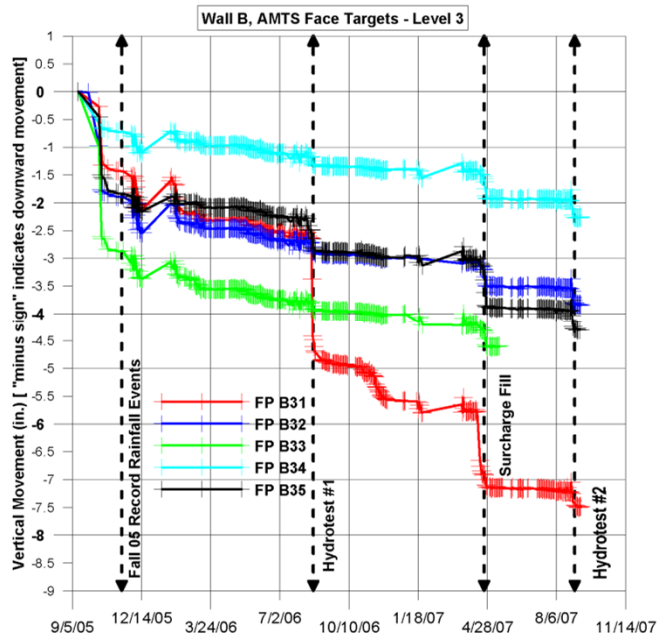
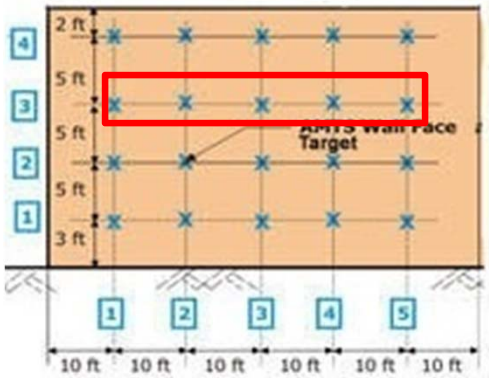
Full Scale Field Test
NCHRP Project 24-22

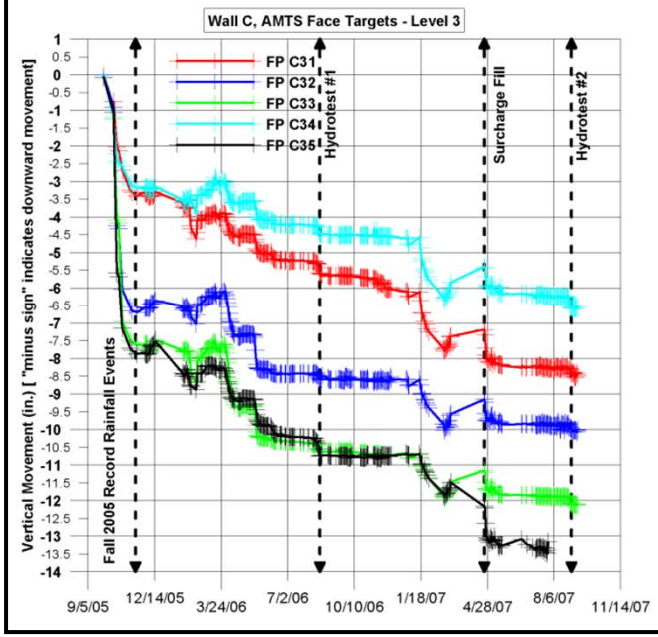
Vertical Face Movt. vs. Time, Level 2 All Walls

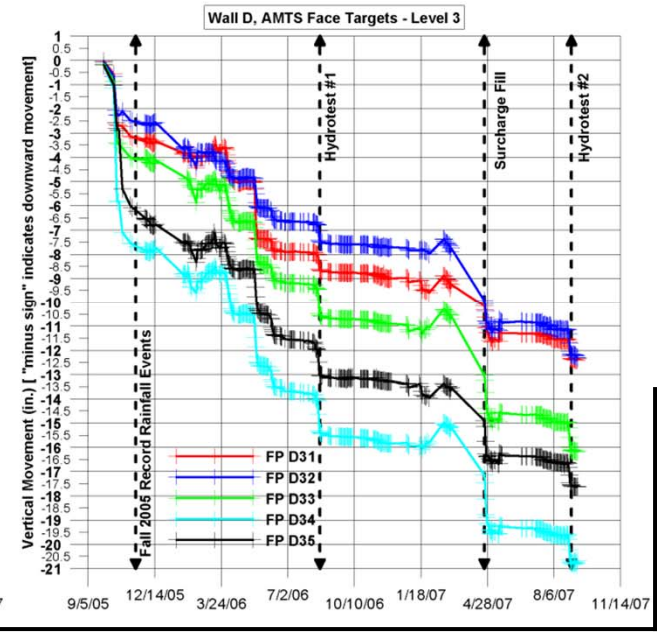
Figure B-38











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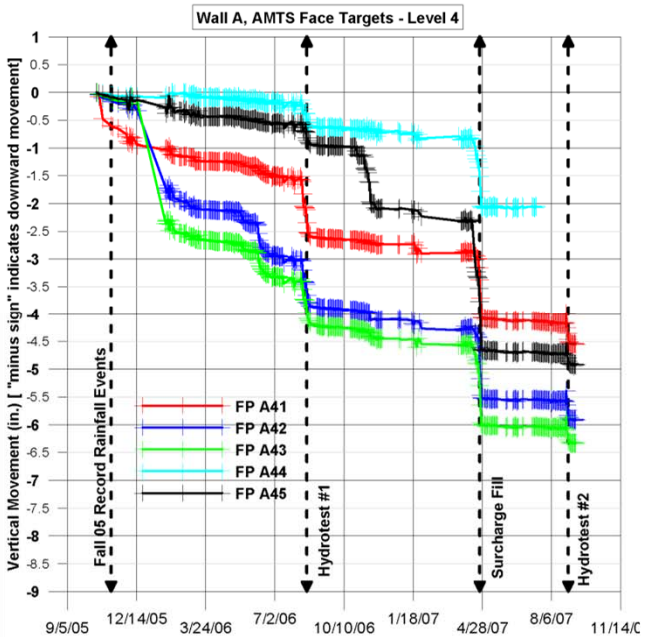
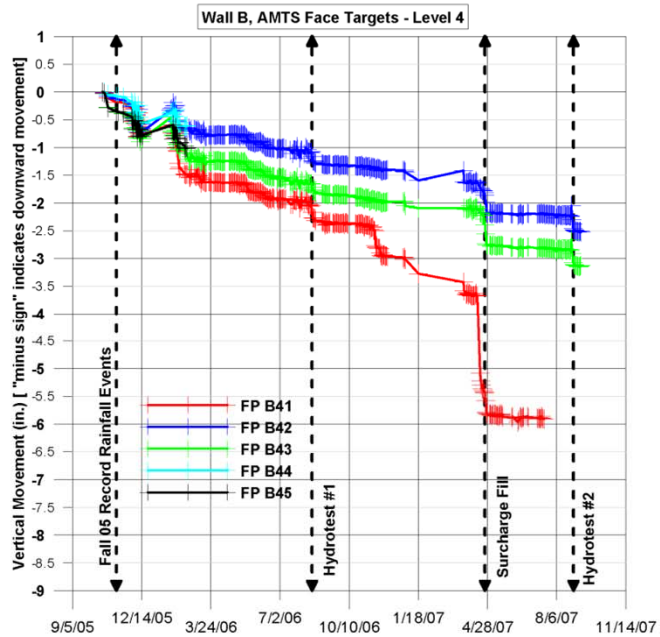
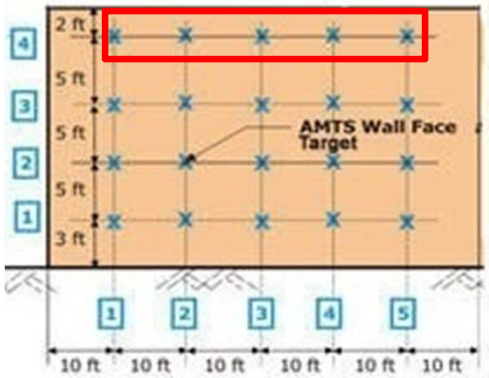
Full Scale Field Test
NCHRP Project 24-22

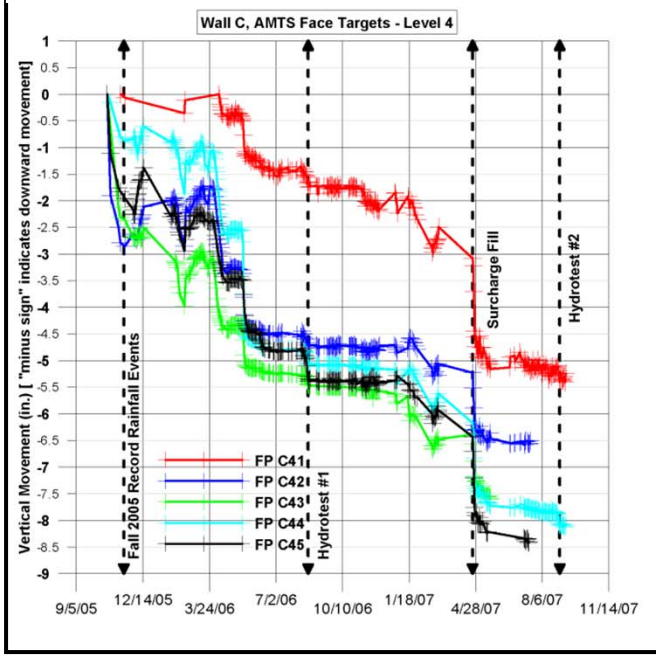
Vertical Face Movt. vs. Time, Level 3 All Walls

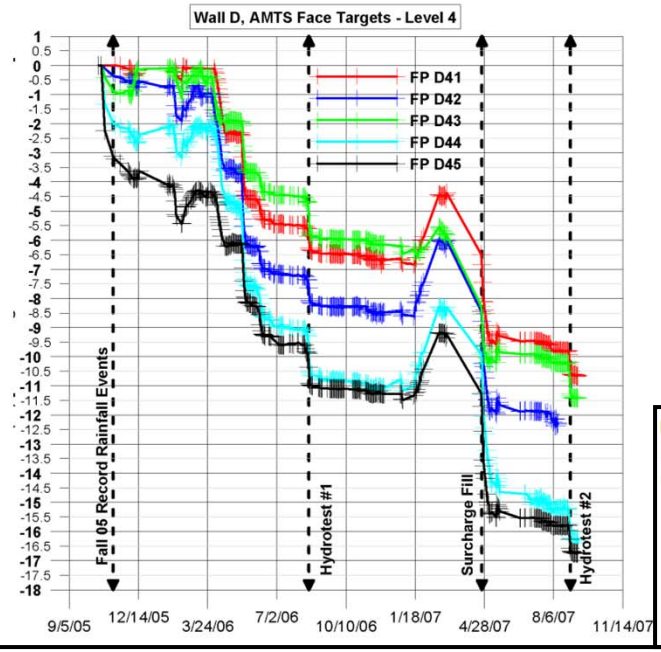
Figure B-39











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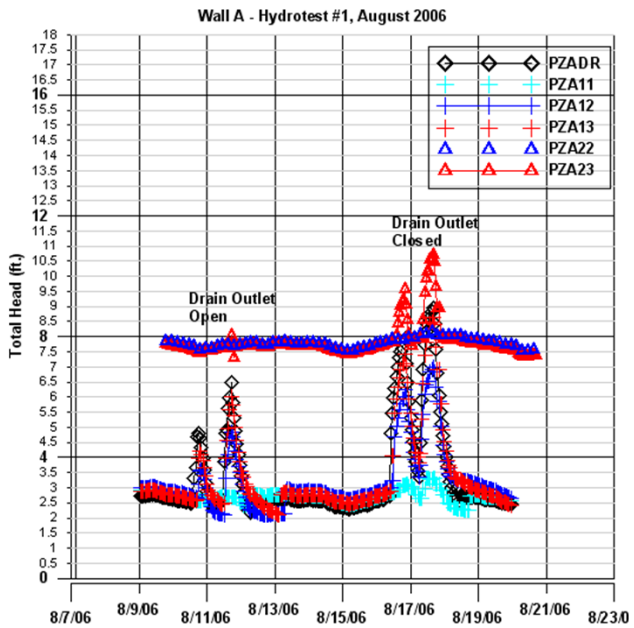
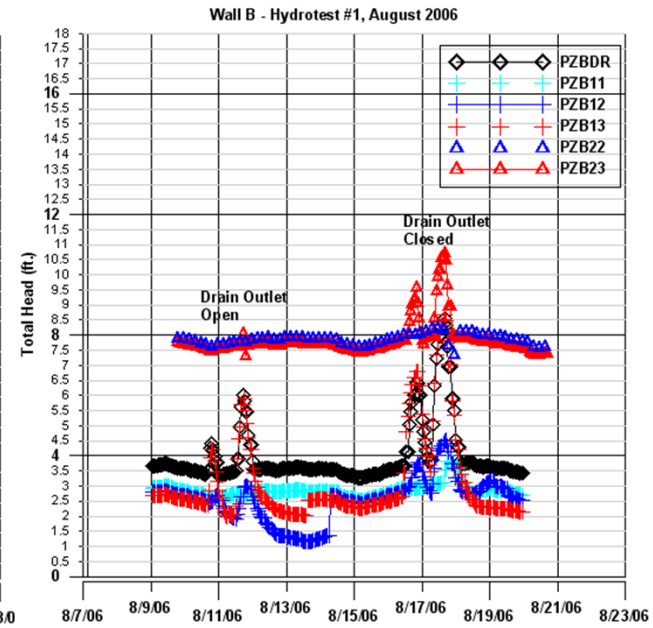
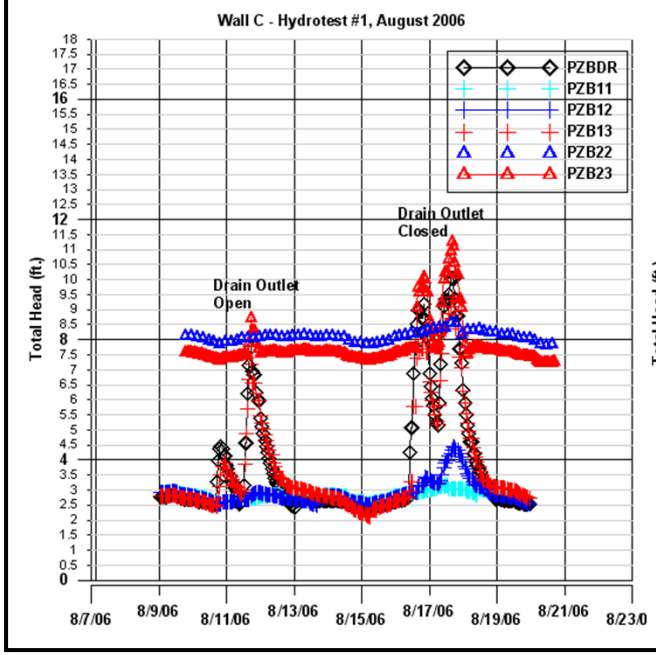
Full Scale Field Test
NCHRP Project 24-22

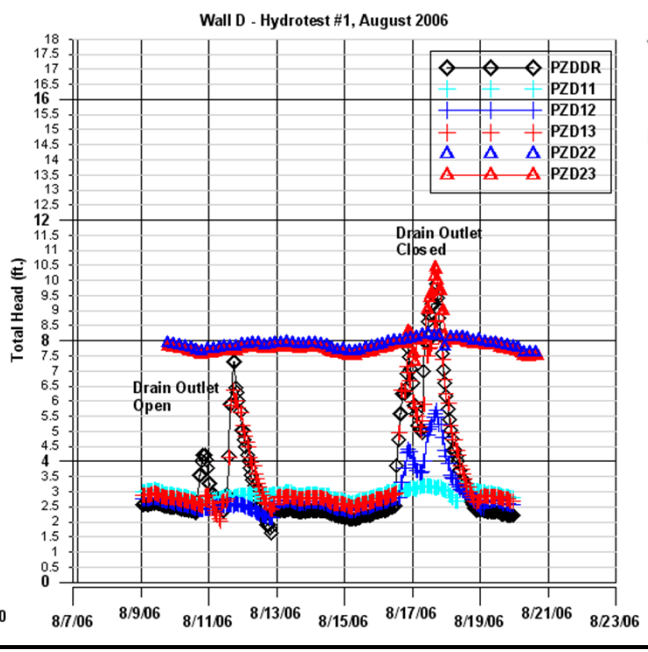
Vertical Face Movt. vs. Time, Level 4 All Walls

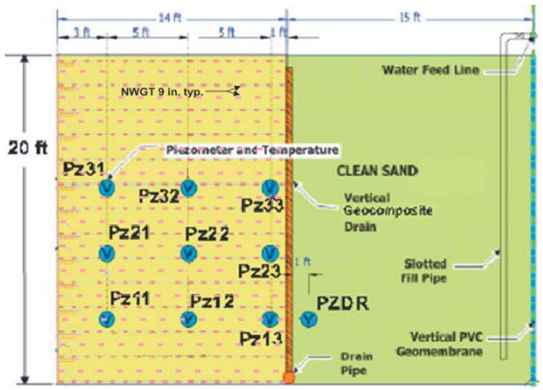
Figure B-40











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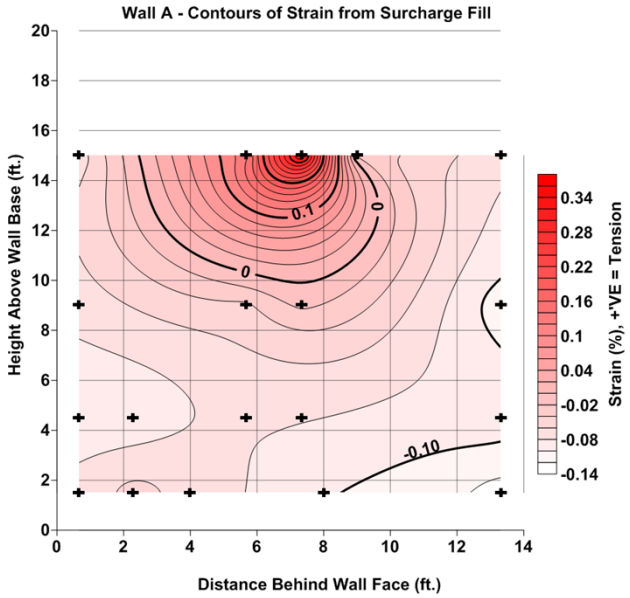
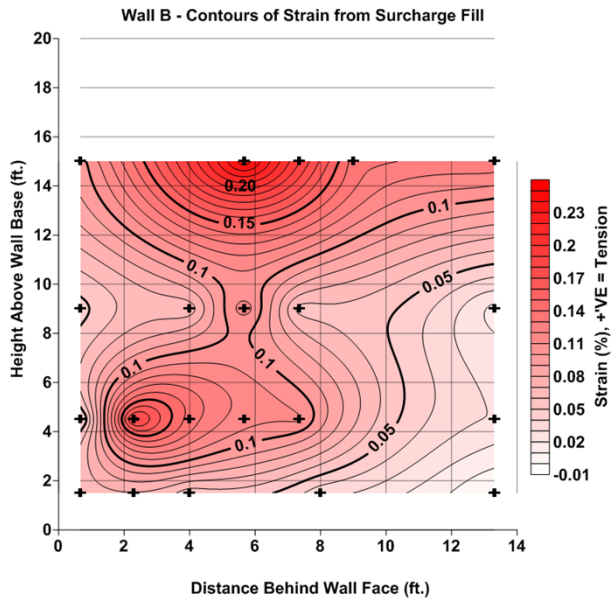
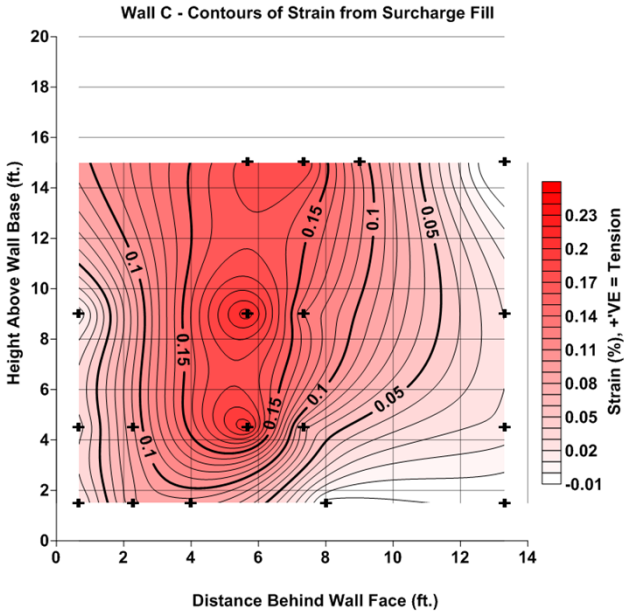
Full Scale Field Test
NCHRP Project 24-22

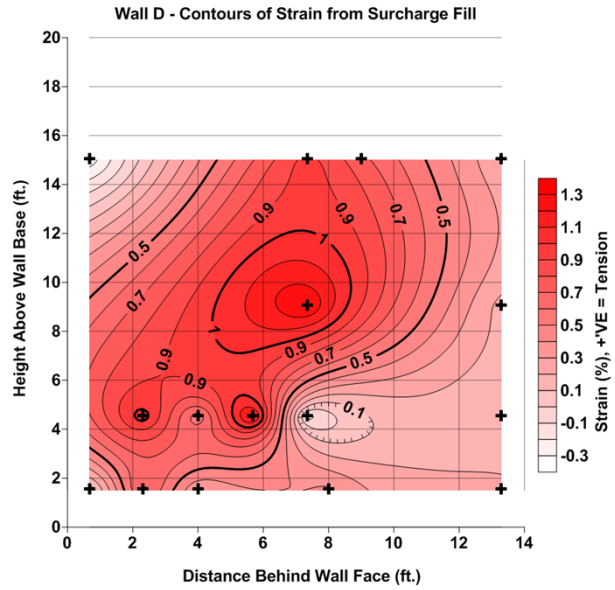
Hydrotest # 1
Piezometer Readings
All Walls

Figure B-41

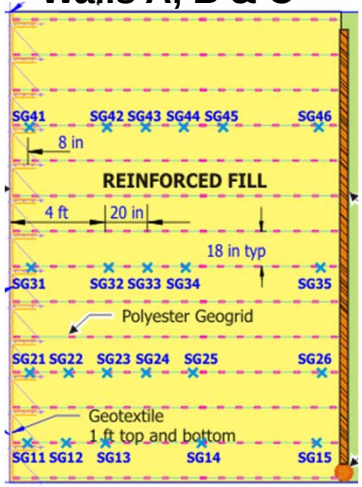




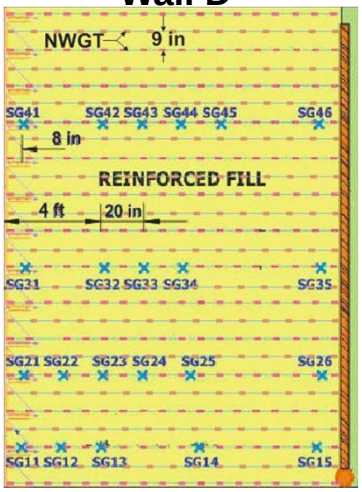




Walls A, B & C



Wall D



No data plotted for following gages: Wall A: SGA-42, 32 and 23
Wall B: SGB-42
Wall C: SGC-41,42 & 23.
Wall D: SGD-42,43,31,32,33 & 21.

Wall C Strain Increment (4/20/07 – 5/20/07)
Wall A, B and D Strain Increment (3/1/07 – 5/20/07)

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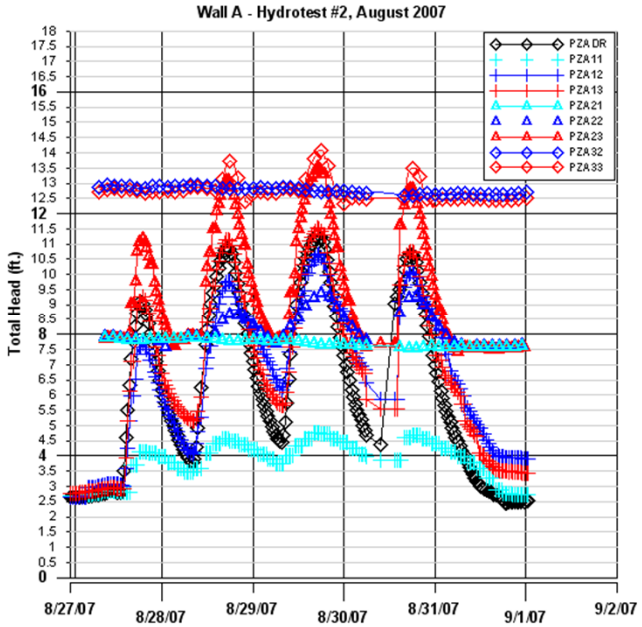
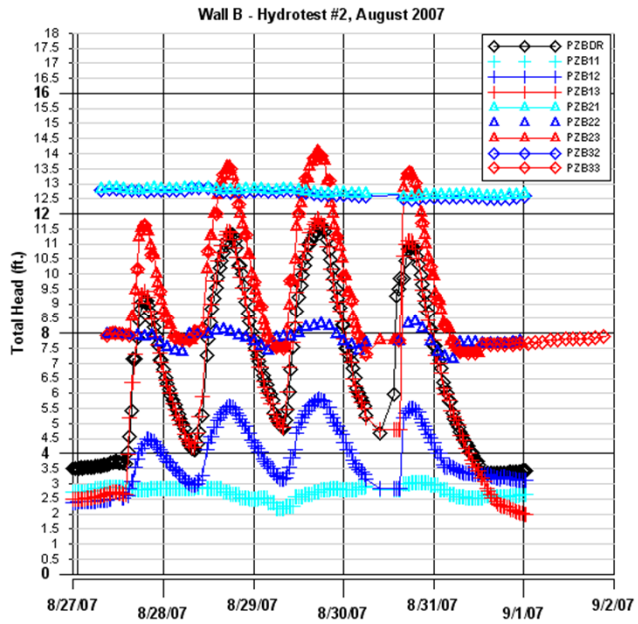
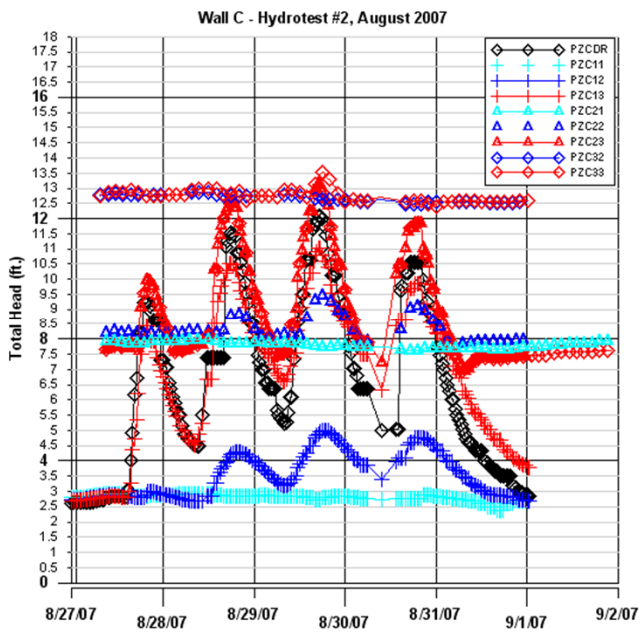
Full Scale Field Test
NCHRP Project 24-22

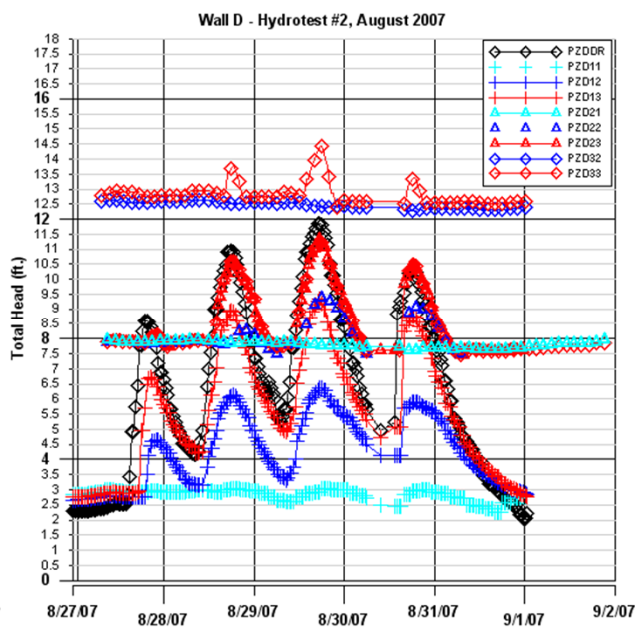
Contours of Strain Increment for Surcharge Fill at All Walls

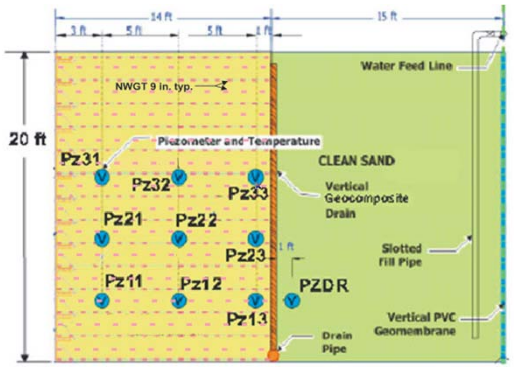
Figure B-42











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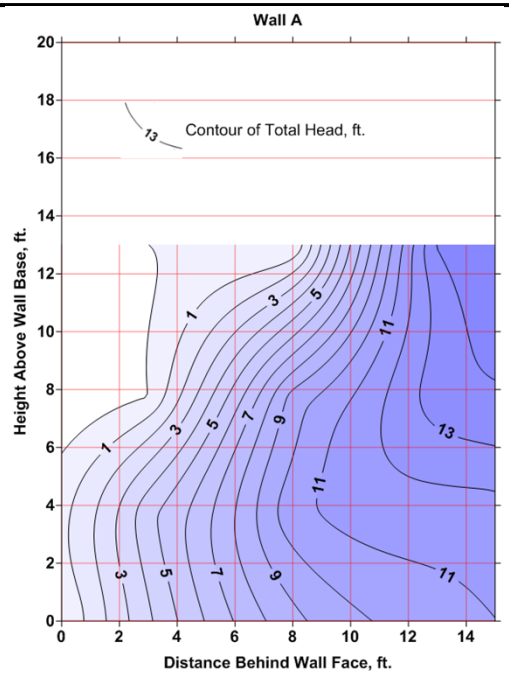
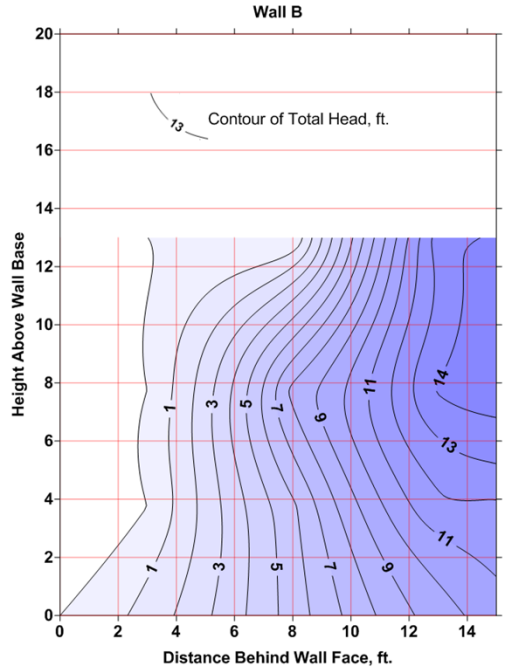
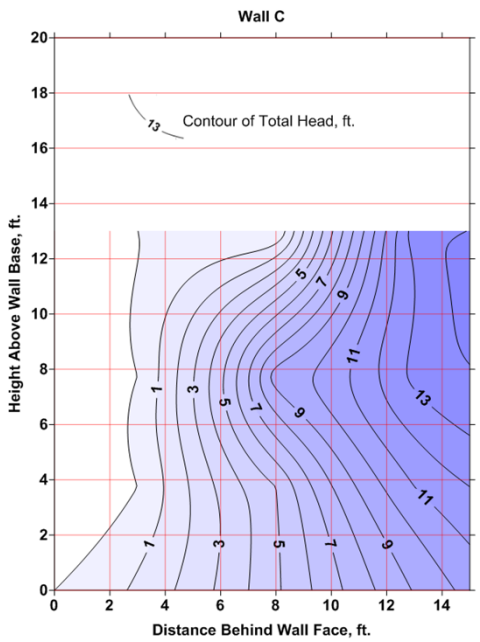
Full Scale Field Test
NCHRP Project 24-22

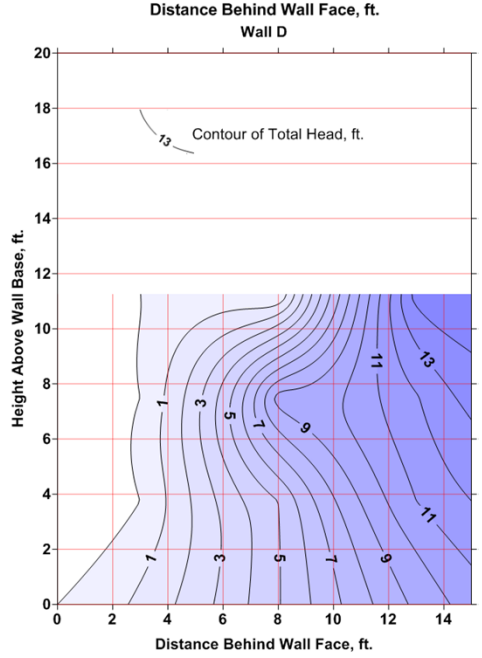
Hydrotest # 2
Piezometer Readings
All Walls

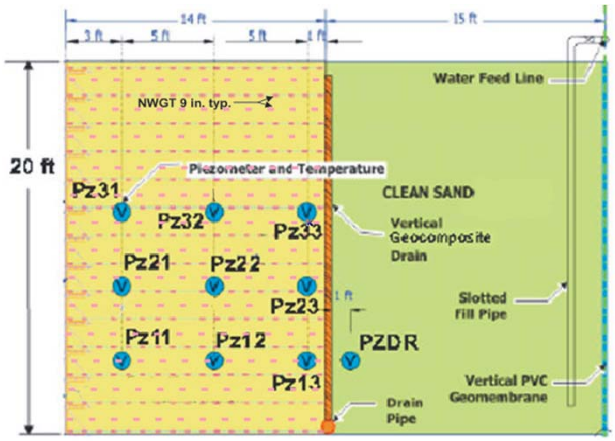
Figure B-43











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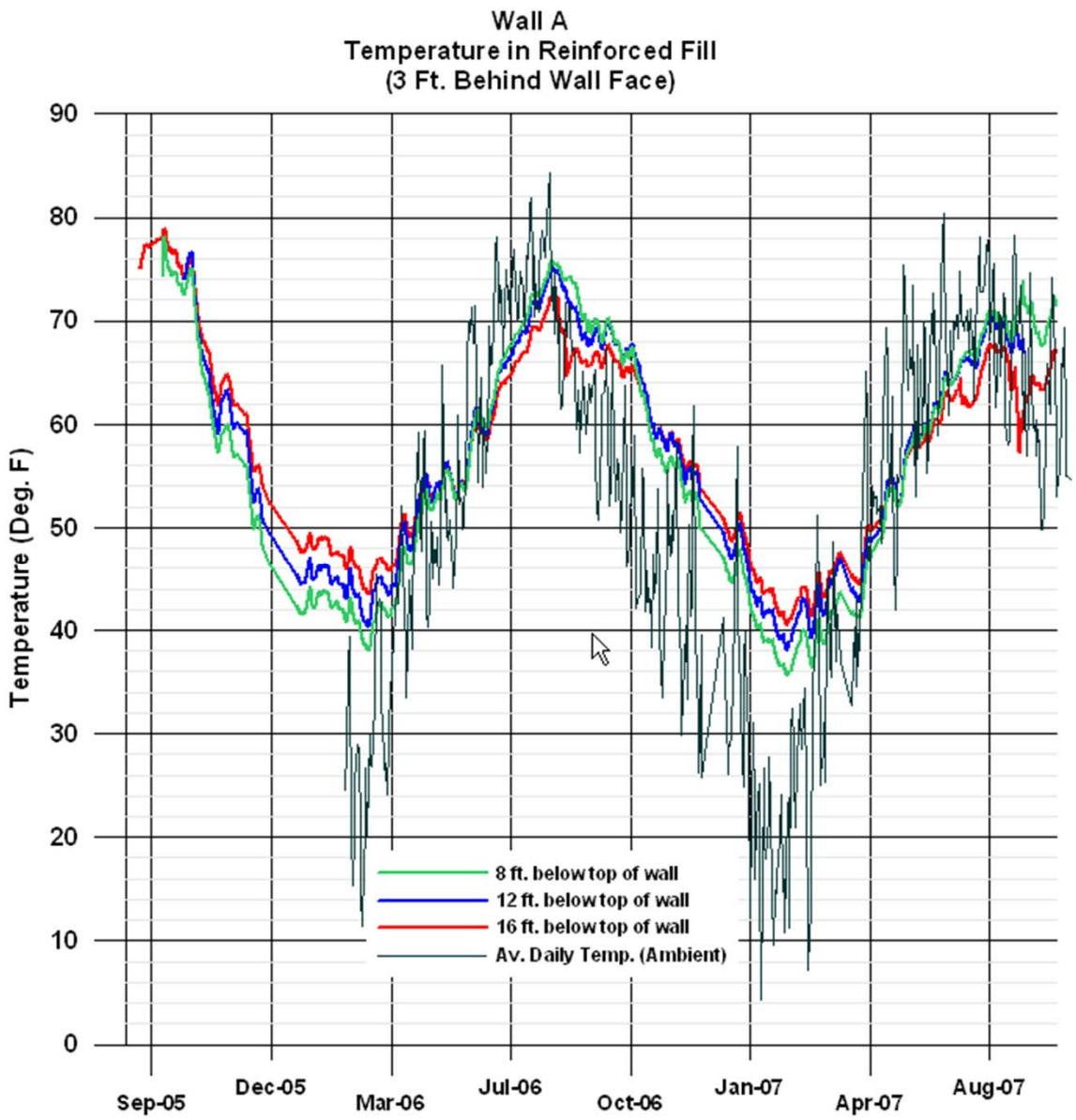
Full Scale Field Test
NCHRP Project 24-22

Contours Of
Max. Achieved Head
Hydrotest #2
All Walls

Figure B-44



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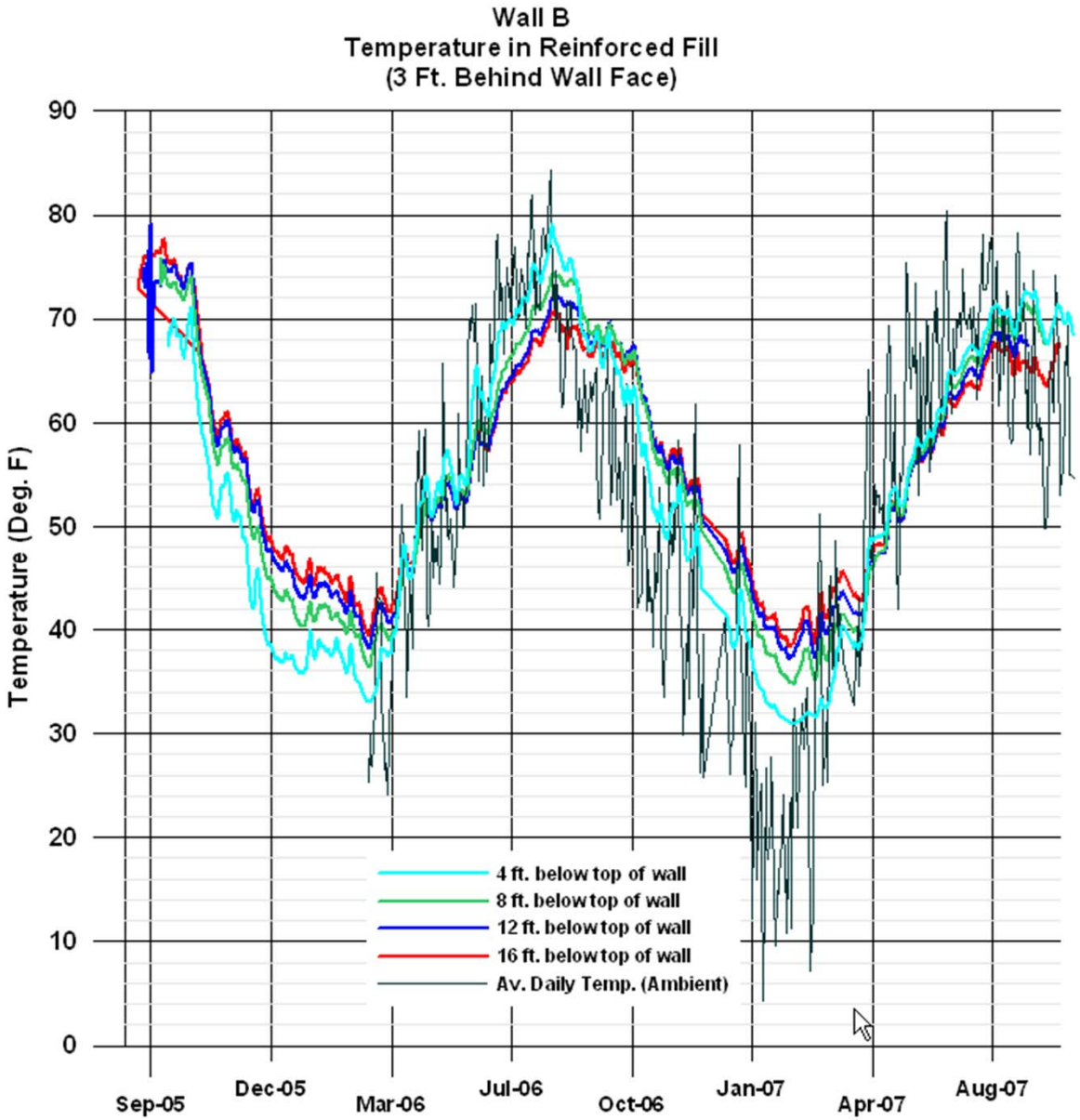
Full Scale Field Test
NCHRP Project 24-22

Time vs. Temperature
Wall A

Figure B-45



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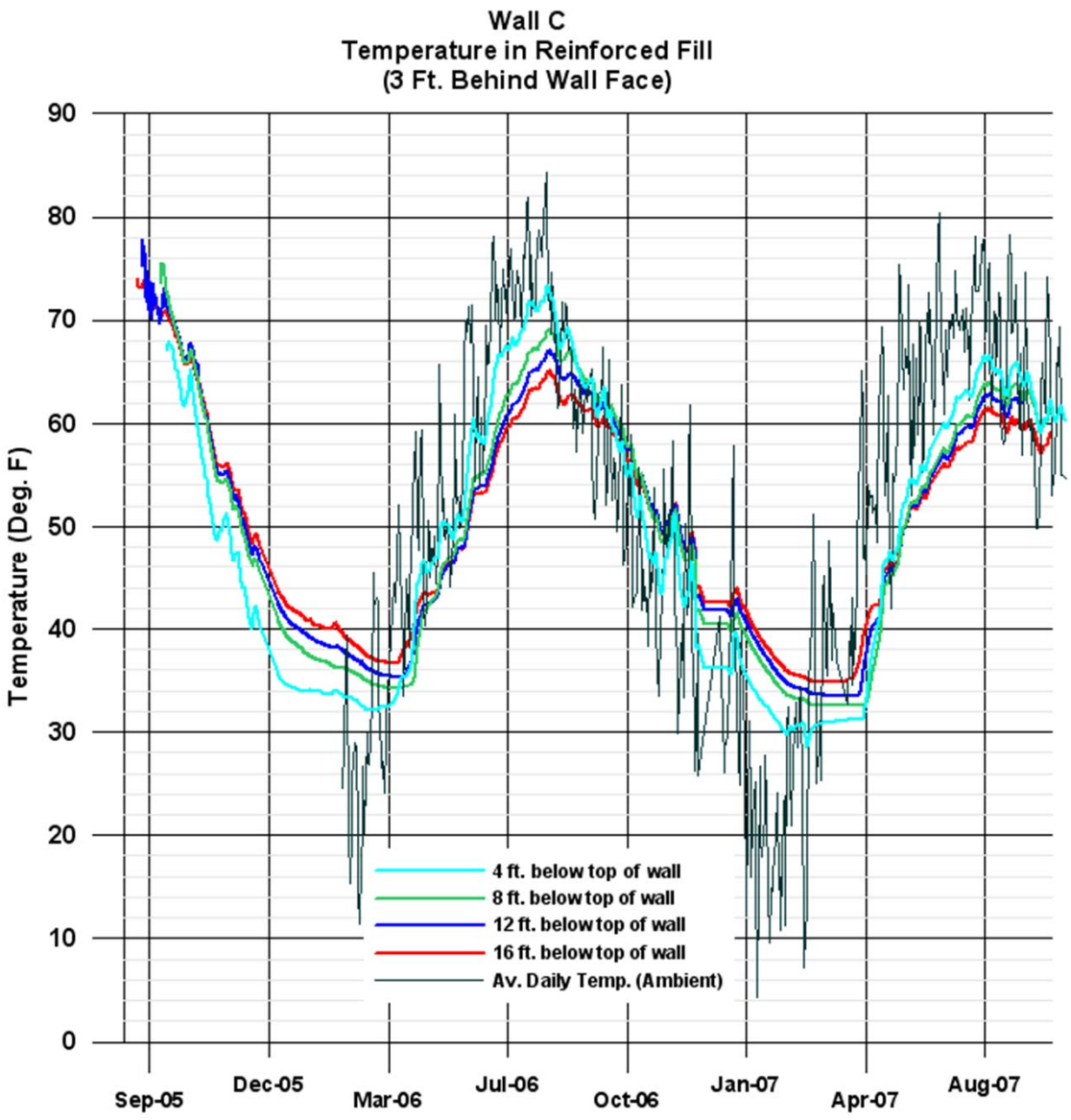
Full Scale Field Test
NCHRP Project 24-22

Time vs. Temperature
Wall B

Figure B-46



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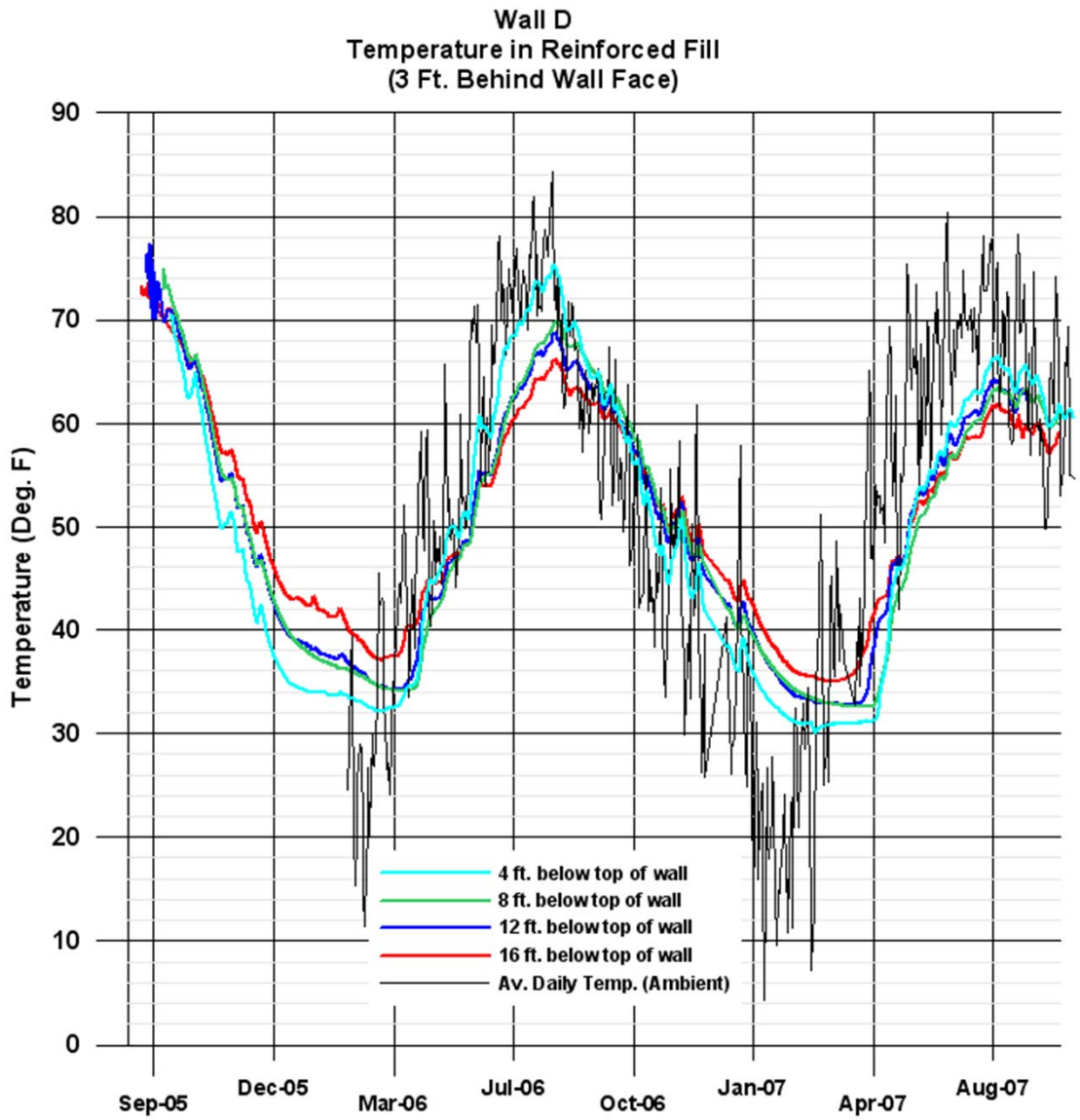
Full Scale Field Test
NCHRP Project 24-22

Time vs. Temperature
Wall C

Figure B-47



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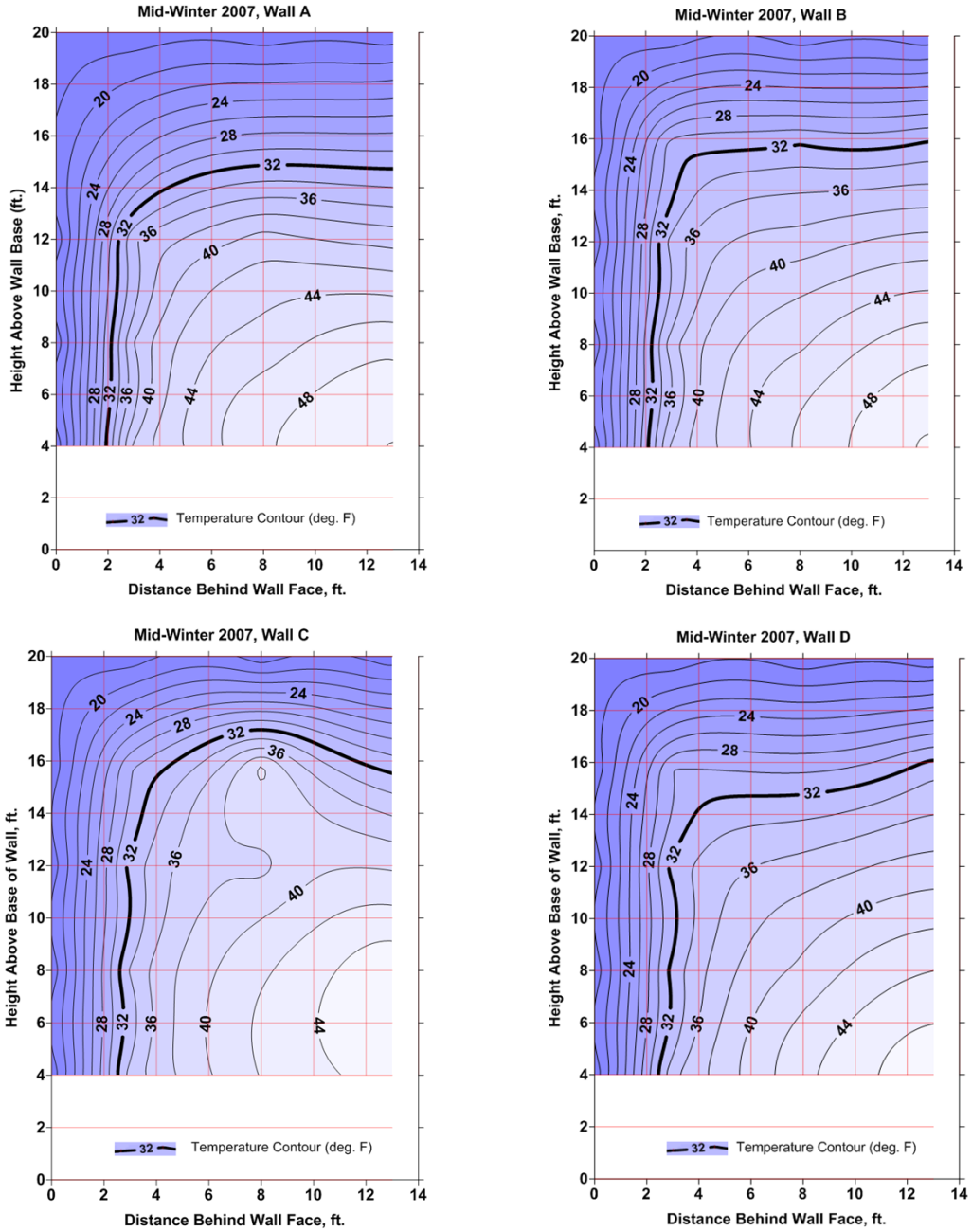
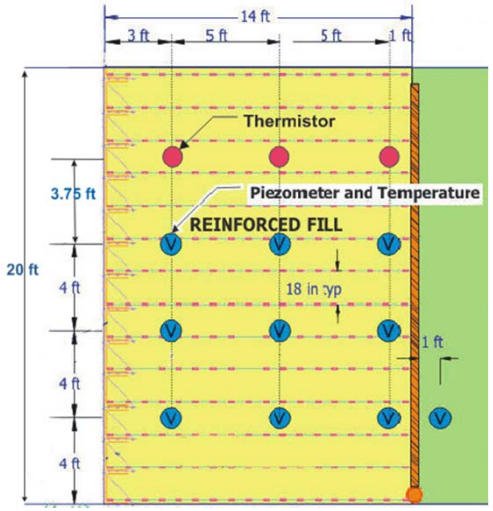
Full Scale Field Test
NCHRP Project 24-22

Time vs. Temperature
Wall D

Figure B-48





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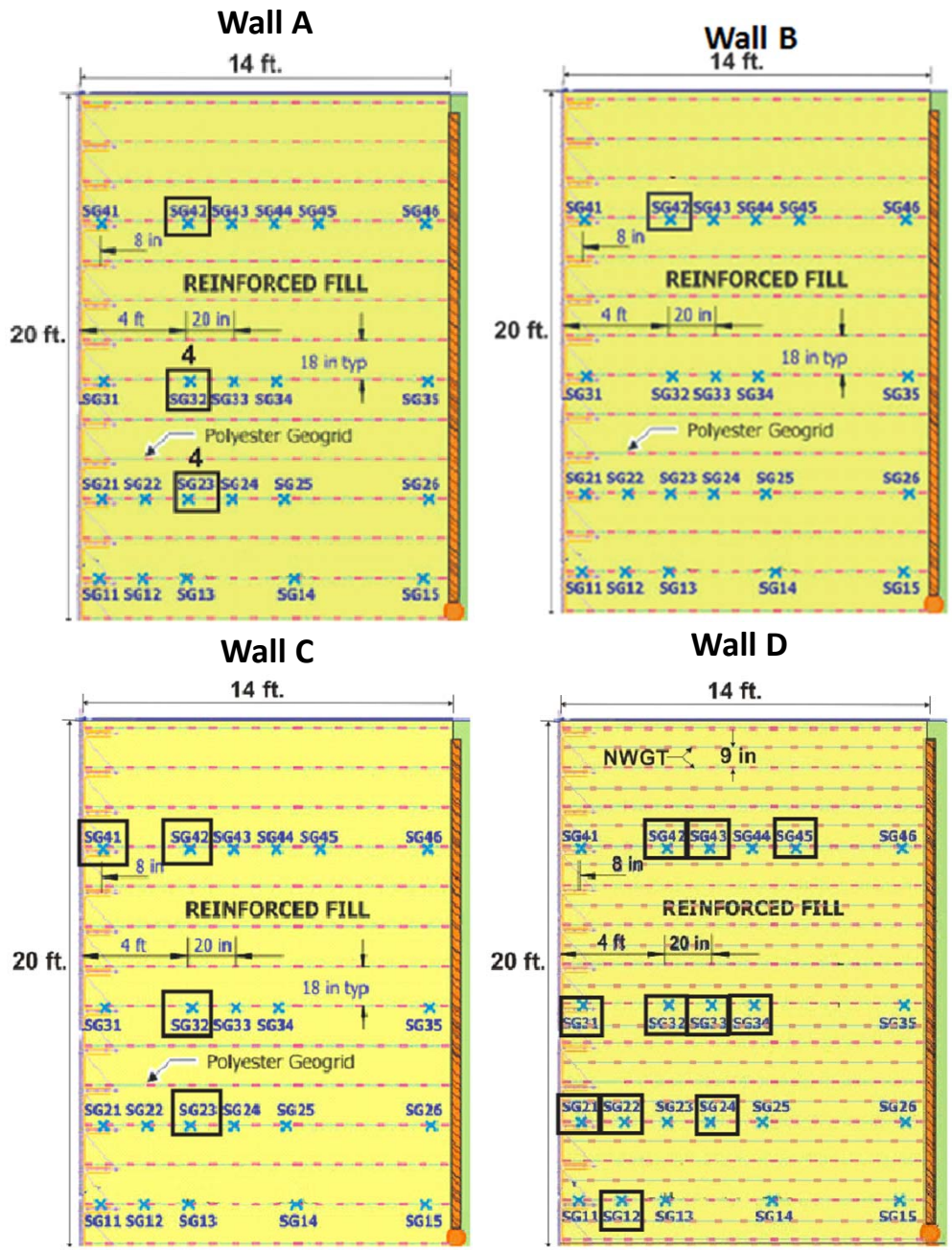
Full Scale Field Test
NCHRP Project 24-22

Temperature Contours
Mid-Winter 2007
All Walls

Figure B-49

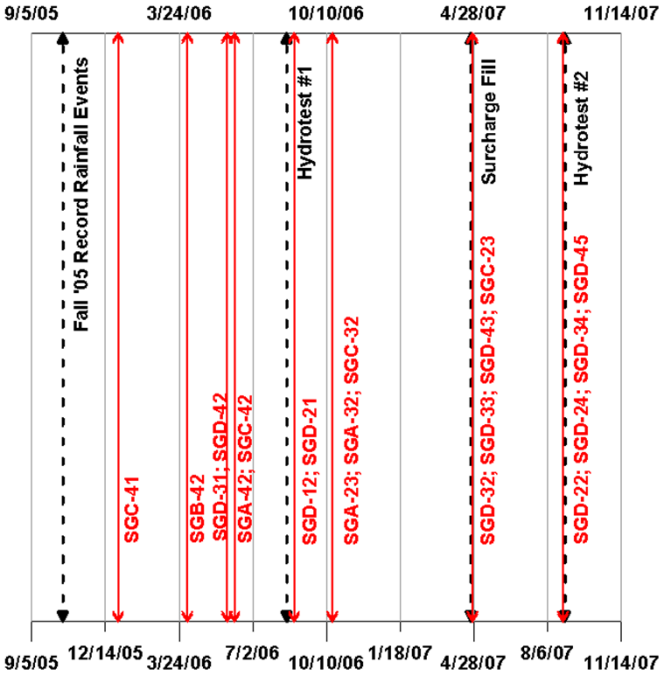


Legend

SG23

Rectangle indicates location of strain gages that experienced mechanical failure.

Time History of Strain Gage Mechanical Failure



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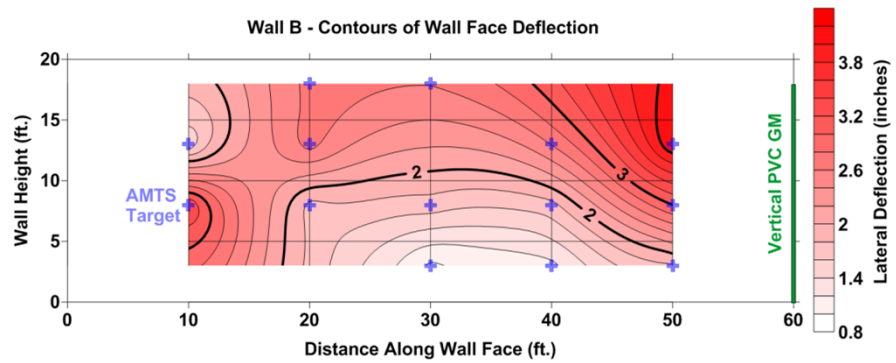
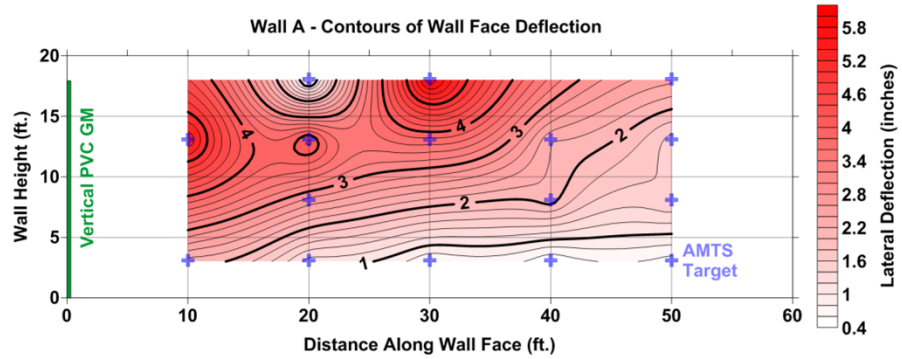
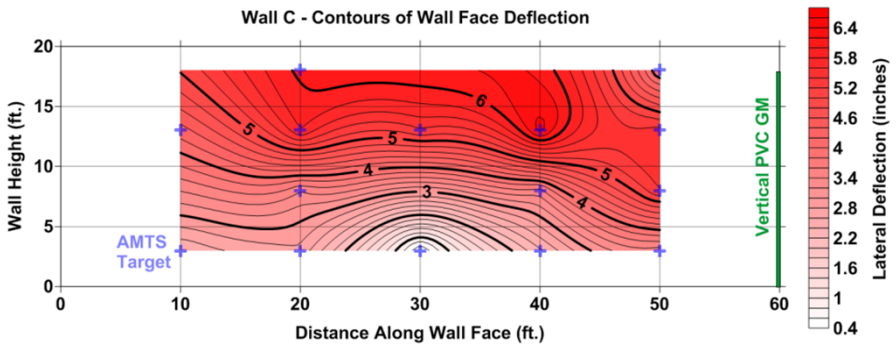
Full Scale Field Test
NCHRP Project 24-22

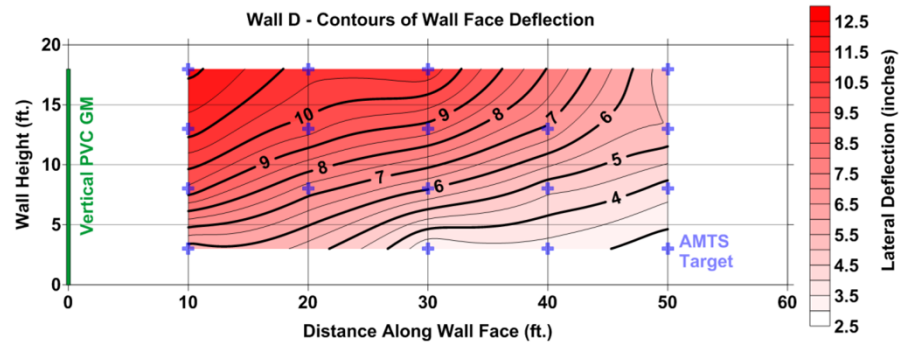
Summary of Strain Gage Malfunction
All Walls

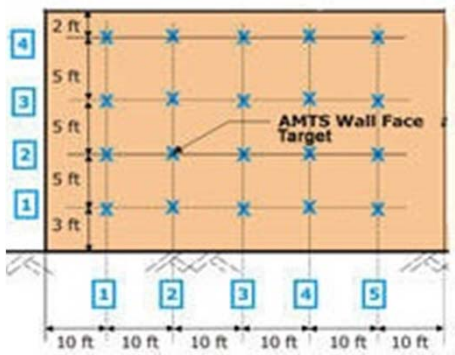
Figure B-505











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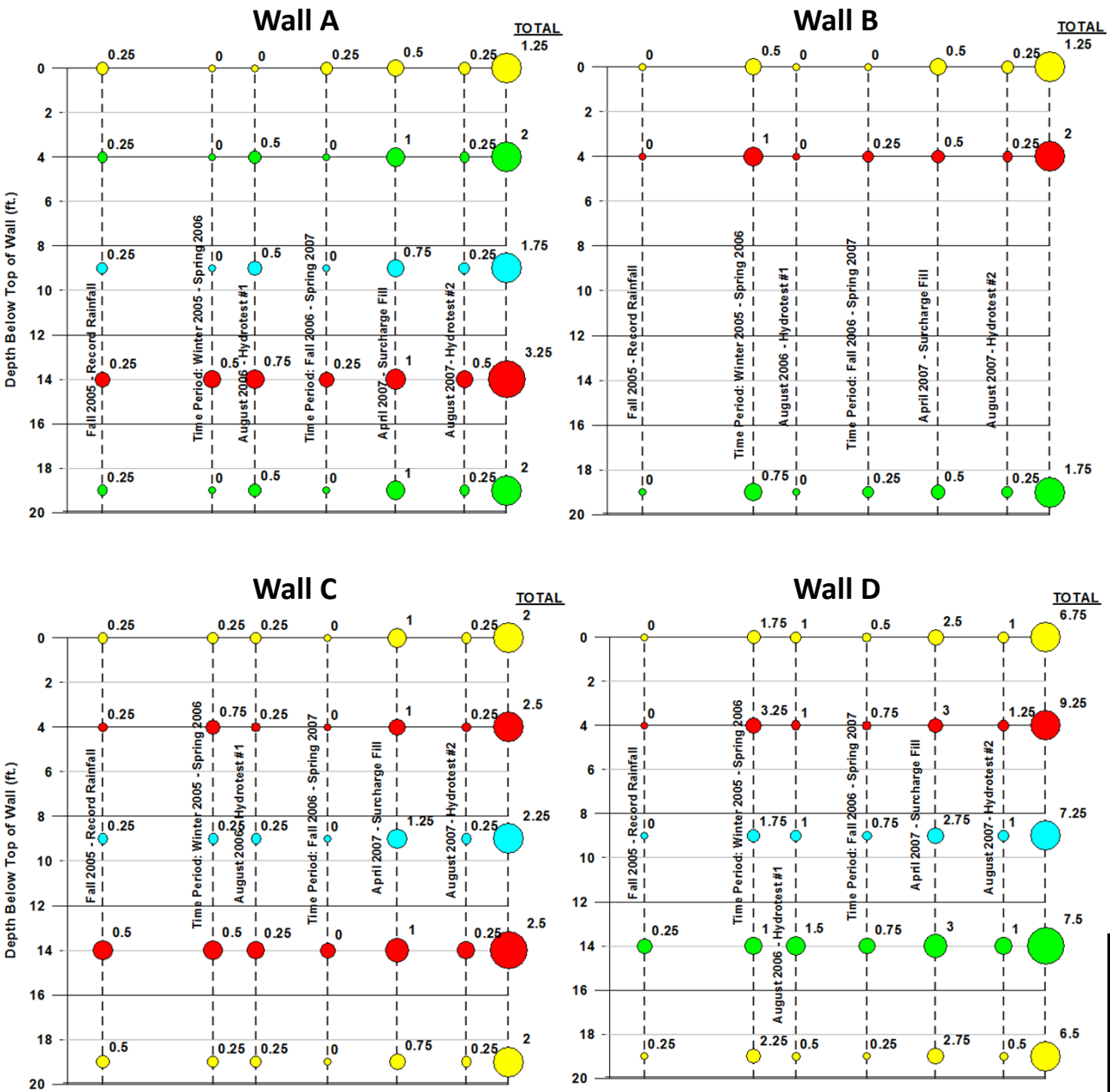
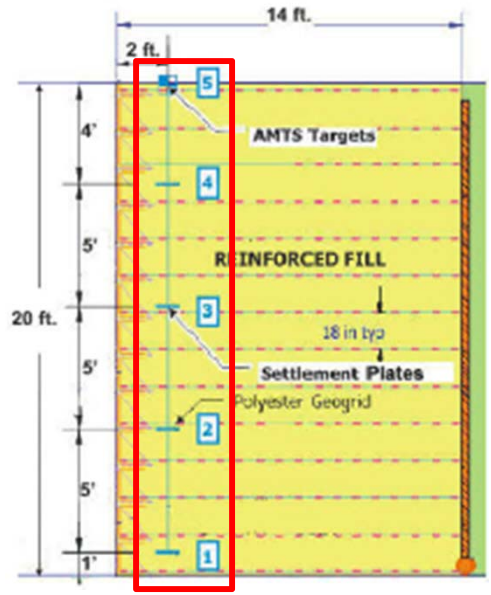
Full Scale Field Test
NCHRP Project 24-22

Contours of Horizontal Movement of Wall Face at End of Field Test All Walls

Figure B-51





Legend

1 Settlement (inches)

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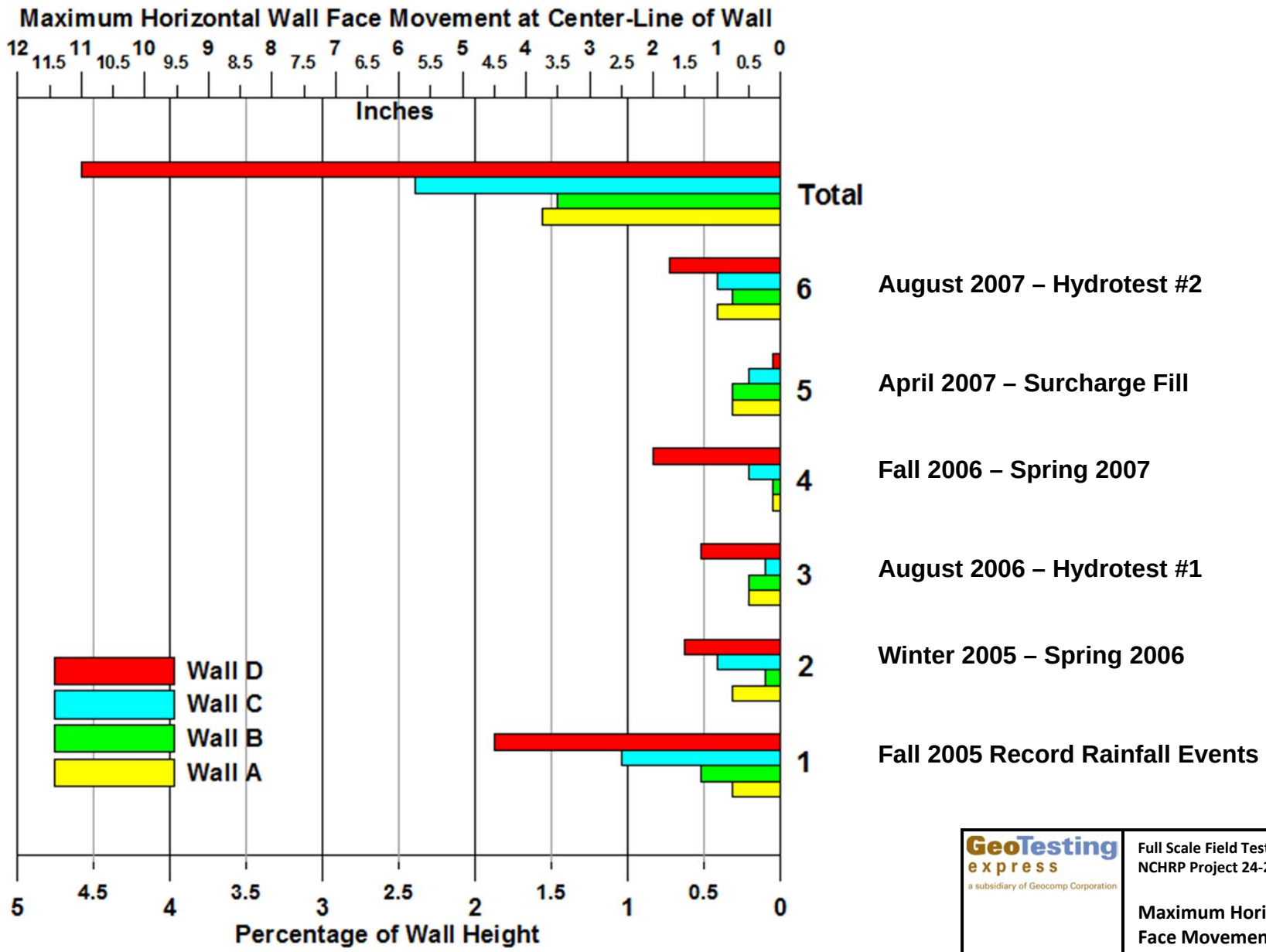
Full Scale Field Test
NCHRP Project 24-22

Settlement Plate Movement vs. Field Test Events, All Walls

Figure B-52



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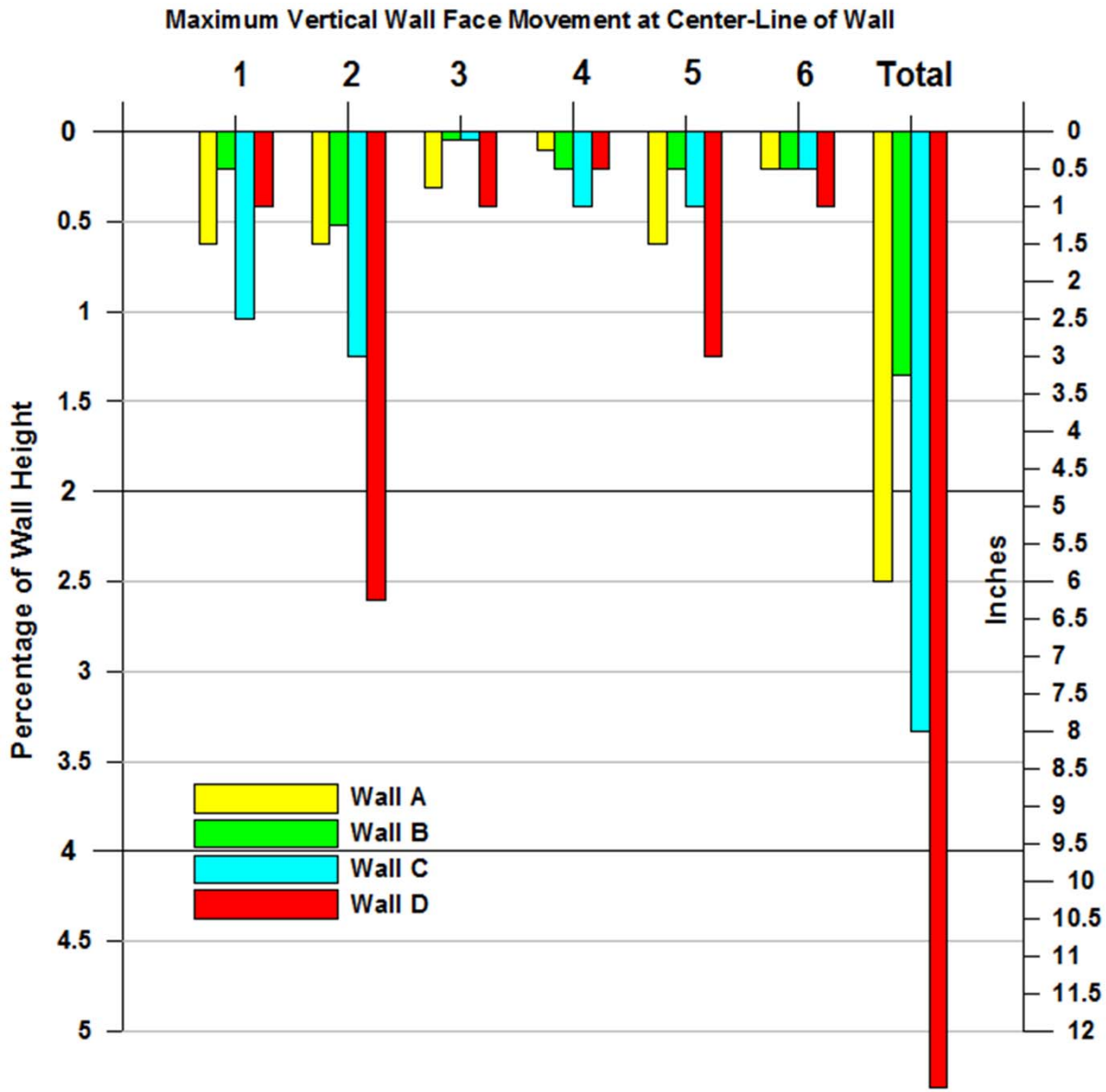
Full Scale Field Test
NCHRP Project 24-22

Maximum Horizontal Wall Face Movement vs. Field Test Events – All Walls

Figure B-53



Legend

1 Fall 2005 – Record Rainfall Events
2 Winter 2005/Spring 2006
3 August 2006 – Hydrotest #1
4 Fall 2006/Spring 2007
5 April 2007 – Surcharge Fill
6 August 2007 – Hydrotest #2

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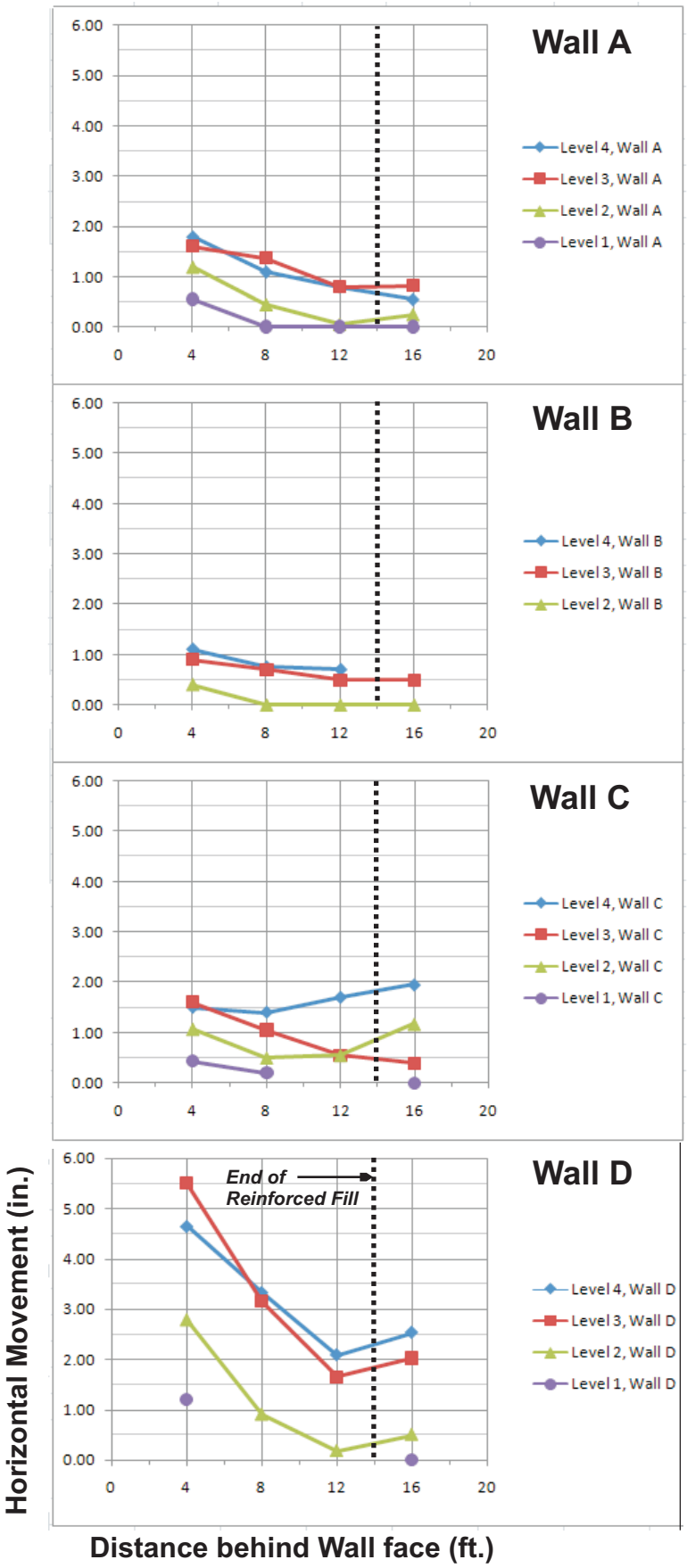
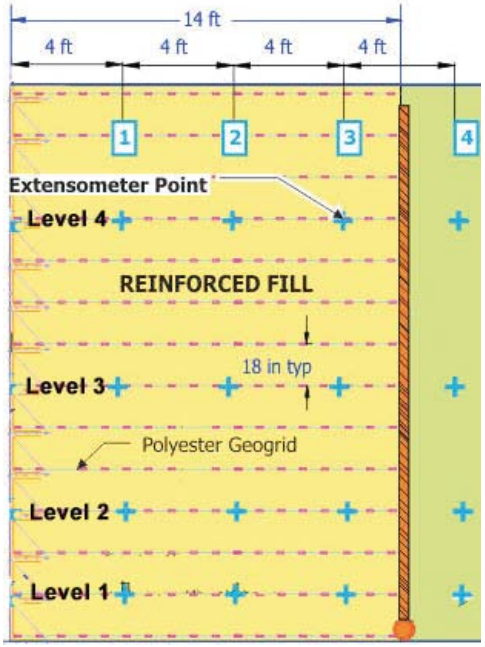
Full Scale Field Test
NCHRP Project 24-22

Maximum Vertical Wall Face Movement vs. Field Test Event s – All Walls

Figure B-54



Note:
Total horizontal movement measured by rod extensometers at the end of the field test.



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Full Scale Field Test
NCHRP Project 24-22

Horizontal Movement @ End of Field Test (Rod Extensometers)

Figure B-55

APPENDIX C

Guide Technical Specification – MSE Reinforced Fill with “High Fines

7.1 - Reinforced Fill Material *("AASHTO LRFD Bridge Construction Specifications, 3rd Edition, 2010" –* ***proposed modification noted in bold italic text***):

Structure backfill material for mechanically stabilized earth walls shall conform to the following grading, internal friction angle and soundness requirements:

Sieve Size	Percent Passing
4.0 in. (100 mm)	100
No. 40 (425 mm)	0-60
No. 200 (75 mm)	***0-25***

Plasticity Index (PI), as determined by AASHTO T90, shall not exceed 6.

The material shall exhibit an angle of internal friction of not less than 34 degrees, as determined by the standard Direct Shear Test, AASHTO T 236 (ASTM D3080), on the portion finer than the No. 10 (2.00-mm) sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T 99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. ***Each sample should be tested at three different normal stresses, approximately equal to ½, 1 and 1½ times the vertical stress at the elevation of the lowest reinforcement layer. Each source should be tested with a minimum of one direct shear test series.*** No testing is required for backfills where 80 percent of sizes are greater than 0.75 in.

The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles.

Additionally, the backfill material shall meet the following electrochemical requirements:

When steel soil reinforcement is to be used:

- pH of 5 to 10 ***(AASHTO T-289)***
- Resistivity not less than 3000 Ohm-cm (***AASHTO T-288***)
- Chlorides not greater than 100 ppm (**ASTM D4327**)
- Sulfates not greater than 200 ppm (***ASTM D4327***) AND
- ***Oganic Content less than 1% (AASHTO T-267).***

When geosynthetic reinforcement is to be used:

- ***pH of 3 to 9(AASHTO T-289) – Polymer Polyester (PET)***
- ***pH greater than 3(AASHTO T-289) – Polymer Polyolefin (PP & HDPE).***

7.1.1 - Permeability Requirements

Laboratory permeability testing, AASHTO (ASTM D2434 or D5084) shall be performed on both the reinforced and retained fill materials. Backfill behind the limits of the reinforced fill shall be

considered as retained fill for a distance equal to 50 percent of the design height of the MSE wall or as shown on the plans.

7.2 - Reinforced Fill Placement

7.2.1 - General

Reinforced fill placement shall closely follow erection of each course of facing panels. Backfill shall be placed in such a manner as to avoid damage or disturbance of the wall materials, misalignment of facing panels, or damage to soil reinforcement or facing members. The contractor shall place backfill to the level of the connection and in such a manner as to ensure that no voids exist directly beneath reinforcing elements.

For walls with modular block facing units, the reinforced fill shall not be advanced more than the height of a modular block unit until the drainage fill, core fill and all fill in all openings within the blocks at that level have been placed. The filled units shall be swept clean of all debris before installing the next level of units and/or placing the geogrid materials.

For walls with flexible facing with gabion style facing, the rock near the wall face shall be hand-placed in accordance with the recommendations of the wall manufacturer.

The maximum lift thickness before compaction shall not exceed eight (8) inches. The contractor shall decrease this lift thickness, if necessary, to obtain the specified density.

For geosynthetic reinforcements, the reinforced fill shall be spread by moving the machinery parallel to or away from the wall facing and in such a manner that the geogrid remains taut. Construction equipment shall not operate directly on the geogrid. A minimum fill thickness of six (6) inches over the geogrid shall be required prior to operation of vehicles. Sudden braking and sharp turning shall be avoided.

For metallic reinforcements, the reinforced fill shall be spread by moving the machinery parallel to or away from the wall facing and in such a manner that the steel reinforcement remains normal to the face of the wall. Construction equipment shall not operate directly on the steel reinforcement. A minimum fill thickness of three (3) inches over the steel reinforcement shall be required prior to operation of vehicles. Sudden braking and sharp turning shall be avoided.

Wall materials which are damaged during backfill placement shall be removed and replaced by the contractor, at no additional cost to the Department. The contractor may submit alternative corrective procedures to the Engineer for consideration. Proposed alternative corrective procedures shall have the concurrence of the MSE wall supplier and designer, in writing, prior to submission to the Engineer for consideration. All corrective actions shall be at no additional cost to the Department

7.2.2 – Compaction

Reinforced fill shall be compacted to 95 percent of the maximum dry density as determined in accordance with the requirements of AASHTO T 99 (Standard Proctor).

Reinforced fill shall be compacted using a static-weighted or vibratory roller. Sheeps-foot or grid-type rollers shall not be used for compacting material within the limits of the soil reinforcement. The contractor shall perform soil density tests to ensure compliance with the specified compaction requirements. Soil density tests shall be taken at intervals of not less than one for every 2000 cubic yards, with a minimum of one test per lift. Compaction tests shall be performed at locations determined by the Engineer.

The backfill density requirement within three (3) feet of the wall facing shall be 90 percent of maximum dry density as determined by AASHTO T 99 (Standard Proctor). Compaction within three (3) feet of the wall shall be achieved by a minimum number of four (4) passes of a lightweight mechanical tamper or roller system.

7.2.3 – Moisture Control

The moisture content of the reinforced fill material prior to and during compaction shall be uniformly dispersed throughout each lift. Reinforced fill materials shall have a placement moisture content of ±2% of optimum moisture, w_{opt}. , as determined in accordance with the requirements of AASHTO T99 (Standard Proctor). Reinforced fill material with a placement moisture content in excess of +2% above optimum shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift.