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Research and Development Section
Research Report

VERIFICATION OF ABUTMENT AND RETAINING WALL DESIGN ASSUMPTIONS

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March 2015

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STATE OF VERMONT
AGENCY OF TRANSPORTATION

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16. Abstract

The American Association of State Highway and Transportation Officials (AASHTO), along with some other federal and state guidelines, suggest a maximum soil fines (particles finer than 0.075 mm) content in granular structural backfill be used behind bridge abutments and retaining walls. This fines content limit is currently set at 6 percent (by weight) by the Vermont Agency of Transportation (VTrans) and is usually between 5 and 12 percent in most states, according to a canvassing of state Department of Transportation (DOT) practices. The fines content limit is an attempt to assure a free-draining backfill condition so water is not retained behind the structure, thereby eliminating the need to design the abutments and retaining walls for hydrostatic pressures. It appears that this maximum fines content is adopted largely as a rule-of-thumb considering that hydraulic conductivity of a soil is expected to decrease with increasing fines content. In Vermont and many other regions the availability of high-quality structural backfill with naturally low fines content is declining, which warrants an evaluation of whether granular backfill materials with greater than 5% fines content could be successfully used in practice.

This research project was set up with two broad over-arching goals. The first goal was to verify that the backfill and drainage details currently used in cast-in-place concrete cantilevered retaining walls and bridge abutments on VTrans projects perform as expected and that the backfill has the engineering properties assumed in the design. The second goal was to find the most cost effective backfill details. To evaluate the above two overarching goals, the specific objectives of this research were to: (1) survey other state Departments of Transportation on their practices for abutment and retaining walls; (2) study the effects of fines on a typical granular structural backfill by performing hydraulic conductivity and shear strength tests at varied non-plastic fines contents; (3) monitor differential water levels between the stream and the backfill at two field sites; (4) analyze the collected data and develop specific recommendations for VTrans; and (5) prepare this final report.

To assess if any differential water pressures exist in existing cast-in-place reinforced concrete retaining walls installed by VTrans, a field monitoring program was implemented at two sites in Vermont. The laboratory investigation included flexible wall, hydraulic conductivity tests on a granular structural backfill with 0, 5, 10, 15, 20, and 25% non-plastic fines content at 41, 83, and 124 kPa (6, 12, and 18 psi) confining pressures followed by consolidated drained triaxial compression tests for obtaining associated drained shear strength parameters of these gradations. The 15.2 cm (6 in.) diameter specimens were prepared at optimum moisture content and 95% of maximum standard Proctor density. Some tests were conducted at 90% of maximum standard Proctor density. To enable a comparison with respect to modified Proctor maximum densities, modified Proctor tests were also performed for all base soil-fines content mixtures. The experimental results were compared with relevant studies found in the literature.

The results of the field monitoring program were inconclusive. The results of the laboratory investigation indicated that a non-plastic fines content up to 10% may be justified in structural backfill specifications for retaining walls and abutments.

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EXECUTIVE SUMMARY

The Vermont Agency of Transportation sponsored this research in an effort to evaluate if our current design practices for the backfill of cast-in-place concrete cantilevered retaining walls and bridge abutments are appropriate and cost effective. The VTrans Geotechnical Section was principal liaison and expert support for this research. Specifically, the research objectives were to: (1) verify that the backfill and drainage details currently used in cast-in-place concrete cantilevered retaining walls and bridge abutments on VTrans projects perform as expected, i.e. differential hydraulic pressures do not develop in the backfill; and (2) assess if the current backfill specifications are adequate and cost-effective.

Backfill strength, hydraulic conductivity and compactability are critical parameters to the substructure design. To assess if any differential hydraulic pressures develop in existing cast-in-place reinforced concrete retaining walls installed by VTrans, a field-monitoring program was implemented at two sites in Vermont. To evaluate the current backfill specification, a laboratory investigation was conducted that included flexible wall, hydraulic conductivity tests on a granular structural backfill. The field investigation results were not sufficiently conclusive. The laboratory assessment suggested that a fines content of up to 10% is potentially allowable as compared to the currently allowed fines content of 6% in VTrans specifications. Earlier research, reported in "*Evaluation of a Proposed Sand Borrow to Include the Percentage Passing the 0.02mm Size*" (Report 1990-04, March 1990, Adams and A-Baki) conducted by VTrans established that a reliable correlation between the fines that contribute to frost susceptibility and fines measured by sieve analysis is adequately protective. If greater fines content is allowed, it may increase availability and decrease cost of suitable borrow materials. The survey results of other state transportation agencies and experimental results led to a recommendation that the structural backfill specification should require 95% of maximum dry density as determined by AASHTO T99. Based on these recommendations, the Agency will implement two changes.

The specifications will be revised for compaction to 95% of maximum dry density as determined by AASHTO T99. The revision will assure better compaction at lower costs and enhanced embankment to bridge transition performance. As a follow up, an experimental installation on three new projects will deploy a granular structural backfill with up to 8% fines. In such cases, a field-monitoring program must be implemented to evaluate if the backfills function as largely free draining materials without development of differential hydraulic pressures.

- Christopher Benda, P.E., Geotechnical Engineering Manager

ABSTRACT

The American Association of State Highway and Transportation Officials (AASHTO), along with some other federal and state guidelines, suggest a maximum soil fines (particles finer than 0.075 mm) content in granular structural backfill be used behind bridge abutments and retaining walls. This fines content limit is currently set at 6 percent (by weight) by the Vermont Agency of Transportation (VTrans) and is usually between 5 and 12 percent in most states, according to a canvassing of state Department of Transportation (DOT) practices. The fines content limit is an attempt to assure a free-draining backfill condition so water is not retained behind the structure, thereby eliminating the need to design the abutments and retaining walls for hydrostatic pressures. It appears that this maximum fines content is adopted largely as a rule-of-thumb considering that hydraulic conductivity of a soil is expected to decrease with increasing fines content. In Vermont and many other regions the availability of high-quality structural backfill with naturally low fines content is declining, which warrants an evaluation of whether granular backfill materials with greater than 5% fines content could be successfully used in practice.

This research project was set up with two broad over-arching goals. The first goal was to verify that the backfill and drainage details currently used in cast-in-place concrete cantilevered retaining walls and bridge abutments on VTrans projects perform as expected and that the backfill has the engineering properties assumed in the design. The second goal was to find the most cost effective backfill details. To evaluate the above two overarching goals, the specific objectives of this research were to:

(1) Survey other state Departments of Transportation on their practices for abutment and retaining walls;
(2) Study the effects of fines on a typical granular structural backfill by performing hydraulic conductivity and shear strength tests at varied non-plastic fines contents;
(3) Monitor differential water levels between the stream and the backfill at two field sites;
(4) Analyze the collected data and develop specific recommendations for VTrans; and
(5) Prepare the final report

To assess if any differential water pressures exist in existing cast-in-place reinforced concrete retaining walls installed by VTrans, a field-monitoring program was implemented at two sites in Vermont. The laboratory investigation included flexible wall, hydraulic conductivity tests on a granular structural backfill with 0, 5, 10, 15, 20, and 25% non-plastic fines content at 41, 83, and 124 kPa (6, 12, and 18 psi) confining pressures followed by consolidated drained triaxial compression tests for obtaining associated drained shear strength parameters of these gradations. The 15.2 cm (6 in.) diameter specimens were prepared at optimum moisture content and 95% of maximum standard Proctor density. Some tests were conducted at 90% of maximum

standard Proctor density. To enable a comparison with respect to modified Proctor maximum densities, modified Proctor tests were also performed for all base soil-fines content mixtures. The experimental results were compared with relevant studies found in the literature.

The results of the field-monitoring program were inconclusive. The results of the laboratory investigation indicated that a non-plastic fines content up to 10% may be justified in structural backfill specifications for retaining walls and abutments.

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1. INTRODUCTION

1.1 MOTIVATION

The analysis and design of retaining walls and bridge abutments have structural (e.g., selection of stem and foundation dimensions, reinforcement details) and geotechnical (e.g., backfill soil permeability and shear strength, the extent of structural backfill behind the structure, drainage provision, depth of foundation) engineering components among others. The choices made in the geotechnical aspects of analysis and design affect the design of structural components and costs. This research evaluated two specific geotechnical engineering aspects of structural backfills – (1) their hydraulic conductivity and shear strength properties; and (2) field observations of differential water levels between the stream and the backfill.

Typically, the design of retaining walls and bridge abutments relies on the assumption that the soil material used to backfill the structure is 'free-draining' and will not produce hydrostatic pressure. If the backfill is not expected to be drained, the abutment or retaining wall must be designed for earth pressure loads plus hydrostatic pressure due to the presence of water. However, there is insufficient readily accessible information regarding the limits of what constitutes free-draining backfill and which current design practices satisfactorily avoid the potential for unexpected hydrostatic pressure. Differential water pressures, as depicted in Figure 1.1, could be of concern particularly for retaining structures near water bodies such as bridge abutments over rivers and streams.

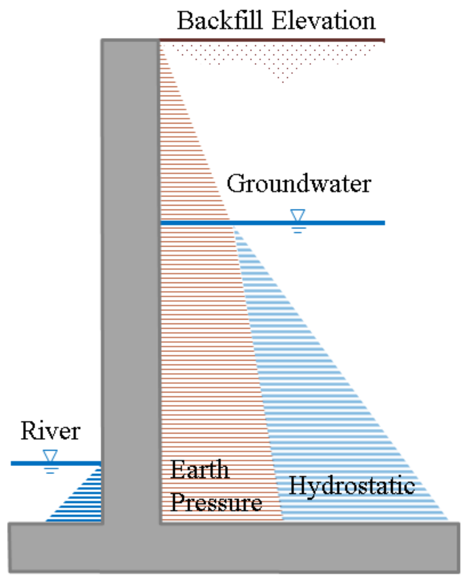


Figure 1.1: Typical Lateral Pressures Acting on a Retaining Wall

A survey conducted in 2011 by the Vermont Agency of Transportation (VTrans) to determine common abutment wall design practices at transportation agencies throughout the United States, received a total of 53 responses; representing 35 states (see Chapter 2 for details). Questions on the fines content allowed in structural backfills, hydrostatic pressure assumptions, typical details and specifications were asked in relation to abutment designs. The majority of responses indicated that drainage systems were used to allow designers to neglect hydrostatic pressure or else the thickness and size of the abutment wall were increased. Additionally, 70% of respondents indicated that backfill with greater than 5% fines content was currently utilized – most states were between 10 – 15% fines content.

The American Association of State Highway and Transportation Officials (AASHTO) recommends that hydrostatic water pressure should be avoided, if possible, in all abutment and retaining wall designs by means of an appropriate drainage system. AASHTO advises that the use of weep holes or drains at the wall face do not assure fully drained conditions. It recommends that an effective design will use pipe drains, gravel drains, perforated drains, geosynthetic drains, or backfilling with crushed rock. However, in a follow-up survey, many states indicated the use of weep-holes and wall face drains as their primary drainage provision. Typical details and specifications standards were then compiled and analyzed.

It was apparent from the survey results that it has generally been recognized that the soil material used as backfill must be drained either through the use of permeable material or by use of an effective drainage system, or both. If not, the wall must be designed for earth pressure loads plus hydrostatic pressure. AASHTO, along with some other federal and state guidelines, recommend a maximum soil fines (particles finer than 0.075mm) content for granular structural backfill behind bridge abutments and retaining walls. The fines content limit is set at 6 percent by VTrans and is usually limited to between 5 and 12 percent in most states, according to the previously mentioned survey results. The fines content limit is an attempt to assure a free-draining backfill condition so water is not retained behind the structure, thereby eliminating the need to design the abutments and retaining walls for hydrostatic pressures. It appears this maximum fines content is adopted largely as a rule-of-thumb considering that hydraulic conductivity of a soil is expected to decrease with increasing fines content. In Vermont and many other regions the availability of high-quality structural backfill with naturally low fines content is declining, which warrants an evaluation of whether granular backfill materials with greater than 5% fines contents could be successfully used in practice. Limited resources and a continually aging transportation system require verification of these design assumptions made in current practice. In some cases, it is possible design assumptions are too conservative; and as a result, construction costs are unnecessarily increased.

1.2 RESEARCH OBJECTIVES

This research project was set up with the following two broad over-arching goals:

(1) The first goal of this research was to verify that the backfill and drainage details currently used on cast-in-place concrete cantilevered retaining walls and bridge abutments on VTrans projects perform as expected and that the backfill has the engineering properties assumed in the design.

(2) The second goal was to find the most cost effective backfill details.

To evaluate the above two overarching goals, the specific objectives of this research were to:

(1) Survey other state Departments of Transportation on their practices for abutment and retaining walls;

(2) Study the effects of fines on a typical granular structural backfill by performing hydraulic conductivity and shear strength tests at varied non-plastic fines contents;

(3) Monitor differential water levels between the stream and the backfill at two field sites;

(4) Analyze the collected data and develop specific recommendations for VTrans; and

(5) Prepare this final report.

1.3 ORGANIZATION OF THIS REPORT

Chapter 2 presents the results of the survey conducted of state DOTs. Chapter 0 presents the literature review of previous relevant studies. Chapter 0 presents the laboratory methods used to investigate the hydraulic conductivity and shear strength of a representative granular backfill with varying non-plastic fines content, test results, and their analysis. Chapter 5 presents the results of the field monitoring. A summary of the overall conclusions and recommendations for future work is presented in Chapter 6.

2. STATE SURVEY OF DESIGN PRACTICES

The Vermont Agency of Transportation (VTrans) created a survey (Appendix A) regarding backfill practices of state transportation agencies, and an invitation to take this survey was emailed by VTrans to state geotechnical and structural experts in July 2011. The survey had a good response rate with 53 complete responses. Four of these did not give contact information, so it could not be determined which states these responses represent. The 49 remaining responses represented 35 states. The survey results were aggregated by state to determine general trends. In some cases, respondents for the same state gave contradictory answers, and the responses were analyzed considering both answers. For example, if the answers to a survey question were as follows:

State	Response
State 1	Yes
State 2	Yes
State 2	No
State 3	Yes
State 3	Yes

The five responses would be aggregated by state into three responses. Since State 2 answered both "Yes" and "No," "Yes" responses would have a range between 2 out of 3 states and 3 out of 3 states. The survey results would be displayed in this way:

Yes	67-100%
No	0-33%

Overall, these contradictions had little effect on the trends of the data.

2.1 SURVEY RESULTS

Each survey question is followed by the results and then any relevant commentary.

1. How do you account for hydrostatic pressure in your design assumptions?

Ignore it.	9%
Design for it.	23%
Install a drainage system in order to not design for it.	89%
None of the above.	0%

The trend among the states is clearly to install a drainage system. Note that the percentages above total to greater than 100%. Some respondents chose more than one answer.

2. Do you utilize backfill material with greater than 5% fines?

Yes	63-77%
No	23-37%

A majority of states allow greater than 5% fines in their backfill, but this finding does not take into account states that allow a little more than 5%. Respondents were asked about their allowable percent fines in follow-up interviews, and answers ranged from 8% (a New England state) to clay (a Midwestern state) with most states falling in between 10% and 15%. On the other hand, some states only allow gravel or use low-strength concrete, so there is no clear trend across states.

3. Has your organization done formal studies to investigate if greater fines contents could be used or if alternative materials could be used/added?

Yes	6-14%
No	86-94%

The analysis of this question was a bit complicated because it asked about two items. Six respondents answered “Yes,” but one of them did not give contact information. UVM followed up on each "Yes" respondent by examining that state's backfill guidelines and relevant reports that could be found online. Several reports on alternative backfill materials were found but no reports on allowing greater fines content. Next, each respondent was contacted requesting information about their studies and, in some cases, asking for clarification on their backfill guidelines. None of the five respondents could remember his/her state studying additional fines content in backfill for cantilevered cast-in-place (CIP) abutments.

4. Please check all applicable backfill materials your DOT uses or would consider using in the future:

Shredded tires	29-31%
Geofoam Blocks	57-71%
Recycled concrete	17-31%
Recycled pavement	20-26%
Granular backfill	91-97%
In-situ soils	37-49%
Other	23-26%

Granular backfill is the most commonly accepted material. Geofoam blocks are also popular, and many states use or would consider using in-situ soils. Given that 80% of the states have standard specifications for backfill material (see Question 6 below), it seems likely that in situ soils would still need to meet a state's specifications to be allowed. The other materials listed are allowed or would be considered by about 25% of the states. If a respondent selected one of these alternative materials, more often than not s/he selected all of them. These results suggest that such states are open to many different types of alternative backfill.

5. Do you have standard details for abutment and wingwall backfill?

Yes	69-74%
No	26-31%

Most states have standard details for abutment and wingwall backfill.

6. Do you have standard specifications for abutment and wingwall backfill methods and materials?

Yes	77-80%
No	20-23%

Most states have standard specifications for abutment and wingwall backfill methods and materials, and standard specifications were slightly more common than standard details. Most states that have one, have the other; although 7 out of the 35 states have only one or the other.

7. Have you changed your details in the past to provide a more cost-effective backfill detail, or do you currently vary your details on a project by project basis based on cost?

Yes	23-31%
No	69-77%

About a quarter to a third of states have factored cost into their details.

8. Do you vary your design and details for backfill based on other non-geotechnical parameters, such as the average daily traffic (ADT)?

Yes	6-9%
No	91-94%

Most states do not use non-geotechnical parameters when considering backfill details.

2.2 RESPONDENTS' COMMENTS

In corresponding with some respondents, some relevant information not associated with a specific question came to light. These findings are detailed below.

In 2005, Virginia Tech conducted a study on using less-than-ideal soils in non-critical mechanically stabilized earth (MSE) walls, but the report focused on MSE walls and did not characterize hydraulic conductivity of the soils. Some states participated in the National Cooperative Highway Research Program (NCHRP) study 24-22 "Selecting Backfill Material for

MSE Walls." The report from this project is not yet available, but a representative of NCHRP said that their study would be of marginal relevance to the study presented here.

A number of respondents volunteered that they rarely use CIP walls due to their cost and use MSE walls instead. An engineer from a Miswestern state explained that CIP walls usually cost $110-150/sf while MSE walls cost $35-45/sf. He also explained that these numbers are based on final costs of their previous DOT projects and may differ due to costs associated with individual projects (piles, ground improvement, etc.). His state requires that CIP walls be designed assuming full hydrostatic pressure in the backfill, so they would require more concrete than Vermont. However, their backfill specifications are broader than Vermont's (clay is allowed), so their backfill may be cheaper as well. This engineer believed that forming is responsible for the extra costs of CIP walls, and MSE walls avoid the additional labor and time to cure. He also commented that MSE walls are not part of the structural integrity of the bridge. Rather, the abutment is founded on piles and the MSE is simply to fill in space. The bridge will stand even if the MSE is washed away. Slopes may be armored to resist scour, but it is not crucial to the structure. However, scour has not been a particular problem for MSE walls in wet environments in his opinion. This engineer also provided a report on a project that used an MSE wall to stabilize an eroding bank in a river near a highway, so migration of soil behind an MSE wall is not a large concern of them. The engineer could not recall if this project has faced any difficulties with soil migration.

An engineer from a Southern state stated that his department tries to avoid abutments near water. He has found that it is often cheaper to build longer bridges that avoid scour issues as well as channel constriction and FEMA water surfaces. When it cannot be avoided, they choose cantilevered walls about as often as they choose MSE walls. He could not pin down why one would be chosen over the other because every project was site specific, noting that MSE walls are more flexible while cantilevered walls are more resistant to flow. In his experience, MSE walls have performed fine. To protect against scour, they may use sheet piling, tangent drilled shafts, embed the wall, or other methods. Typically, 10 feet of scour is accounted for in their designs.

Similarly, an engineer from another Southern state explained that they also try to avoid abutments in stream crossings. Instead, they use slopes plated with riprap. If an abutment near a stream is unavoidable, they use MSE walls and bury the bottom of panels 2 feet below scour depth. They also found MSE walls consistently less expensive than CIP walls so they rarely use the latter. Both of the above-mentioned engineers said they only use CIP walls when horizontal space is at a premium, such as in mountainous terrain.

The respondents from the Northeast states were also contacted about their experience with MSE walls in stream crossings. Not all states responded to the survey, but two other Northeast states DOTs appear to avoid MSE walls in stream crossings due to a strong possibility of washout. An engineer from another Northeast state said that MSE walls are significantly cheaper than CIP walls, but, similar to the two Southern states above, they prefer not to make the reinforced soil a structural component of the bridge in case the soil washes out.

None of the states contacted have standard details for MSE walls in wet crossings. It appears that one Northeastern state uses the FHWA manual (FHWA-NHI-10-024/025) for design of MSE walls.

In summary, state DOTs follow a wide variety of abutment wall practices. A variety of backfill types and qualities are used including gravel, low-fines granular material, and materials with high fines content. Other materials include low-strength concrete and geofoam blocks. While some states appear to be moving away from cantilevered cast-in-place walls, other states still find them useful, and some states, appear to be reluctant to use MSE walls in wet environments. There appears to be a common thought that MSE walls should not be part of the structural integrity of a bridge, but some states have found it more cost effective to use piles and MSE walls rather than CIP walls. Many states are moving abutments and approaches away from wet environments both for the benefits of not infringing on the channel and avoiding scour concerns.

2.3 STANDARD DETAILS AND SPECIFICATIONS

A follow-up survey was conducted because several respondents to the original survey indicated that standard details and specifications are currently in use within their DOTs. Construction drawings of typical details of cast-in-place cantilever bridge abutments were obtained when possible.

Table 2.1 summarizes the follow-up survey responses from the states, as well as information obtained independently. The information obtained independently was typically based on some project information found online, and has not been verified to be a standard practice. The data summarized in Table 2.1 indicate that the range of allowable percent fines varies between 0 and 15% among the states that responded, but the survey respondents did not know the basis that led to the specific fines content specification. Also, the specified minimum relative compaction varies between 90 and 100% based on standard Proctor maximum dry density or between 95 and 97% based on modified Proctor maximum dry density. A related question is how far the structural backfill should extend behind an abutment or a retaining wall. State DOTs have differing specifications ranging from a vertical limit to a 1.5H:1V slope from the heel of the wall footing as summarized in Table 2.1.

Table 2.1: Summary of State Survey Responses

State	% Fines	Backfill Slope (H:V)	Backfill Offset from Footing Heel (in.)	In-Place Relative Compaction Required (%)	In-Place Moisture Content Required (%)	Proctor Type*
1 (Vermont)	0 – 6	Vertical	24"	90, 95, 100	Optimum ± 2	Standard
2[+]	0 – 5	1.5 : 1	0"	95	Optimum	Modified
3	0 –	Vertical	12"	95	Optimum	Standard
4	0 –	Vertical	12"	95, 98	Optimum ± 2	Standard
5	0 –	1.5 : 1	0 - 39"	95	Optimum	Standard
6	0	1: 2	0"	92, 97	Optimum ± 2	Modified
7	0 – 2	Vertical	0"	90, 93, 95	Optimum	Standard
8	0 – 7	1 : 1	18"	95	Optimum ± 3	Standard
9	0	Vertical	24"	98	Optimum	Standard
10	0	1.5 : 1	12"	90, 95	unavailable	Modified
11	5 –	1.5 : 1	48"	95	Optimum ± 2	Modified

<u>Note:</u>
**Modified refers to ASTM D1557 and AASHTO T180 Proctor test or equivalent.*
Standard refers to ASTM D698 and AASHTO T99 Proctor test or equivalent.

Table 2.2 summarzies the drainage provisions specified by various states. Table 2.2 indicates that the majority of the states utilize multiple drainage provisions in an effort to minimize hydrostatic pressure behind the abutment wall.

Table 2.2: Drainage provisions for bridge abutments used in various states

State	Weep holes	Underdrain[1]	Composite[2]
1 (Vermont)	X	--	--
2[3]	X	X	X
3	X	X	--
4	X	X	X
5	X	X	--
6	X	X	--
7	--	--	--
8	X	--	--
9	X	X	X
10	--	X	--
11	--	X	--

Note:
1. Underdrain refers to a pipe placed behind abutment wall, daylighting at weepholes or sides of abutment.

2. Composite drainage consists of geotextile /geocomposite materials placed against the face of the backfilled wall - to direct water to weepholes or underdrain locations.

3. This state specifies a 3 foot minimum differential hydrostatic pressure to be considered for design of structures along rivers.

Figure 2.1 and Figure 2.2 compare gradation limits for backfill material prescribed by VTrans and some other states. Specifically, Figure 2.1 shows the gradation specifications for VTrans compared to two other New England DOTs. The three sets of gradation requirements are fairly comparable. Figure 2.2 shows the gradation specifications of VTrans compared to a Northeastern DOT and Midwestern DOT – illustrating the wide acceptance range of the Northeastern state and the lower restrictive acceptance range of the Midwestern state.

2.4 ANALYSIS OF DATA FROM PAST VTRANS BRIDGE ABUTMENT PROJECTS

VTrans provided grain size analysis and field compaction test data from a number of their bridge abutment projects. These data are analyzed in the following.

Soil gradation data from a total of 12 past VTrans bridge abutment projects were made available. The data contained the results of sieve analyses performed on backfill soil samples of in-place material, located at either abutments or stockpiled material on-site. In Figure 2.3, all of the project data have been plotted with the specification requirements. The soil gradation from the Bridgewater project has been indentified since, because this borrow material was used as the base soil in the experimental investigation presented later in this report. Data on gradations from 55 granular backfill sources available for abutment projects in Vermont have been plotted in Figure 2.4. This plot also includes the VTrans gradation specification for comparison.

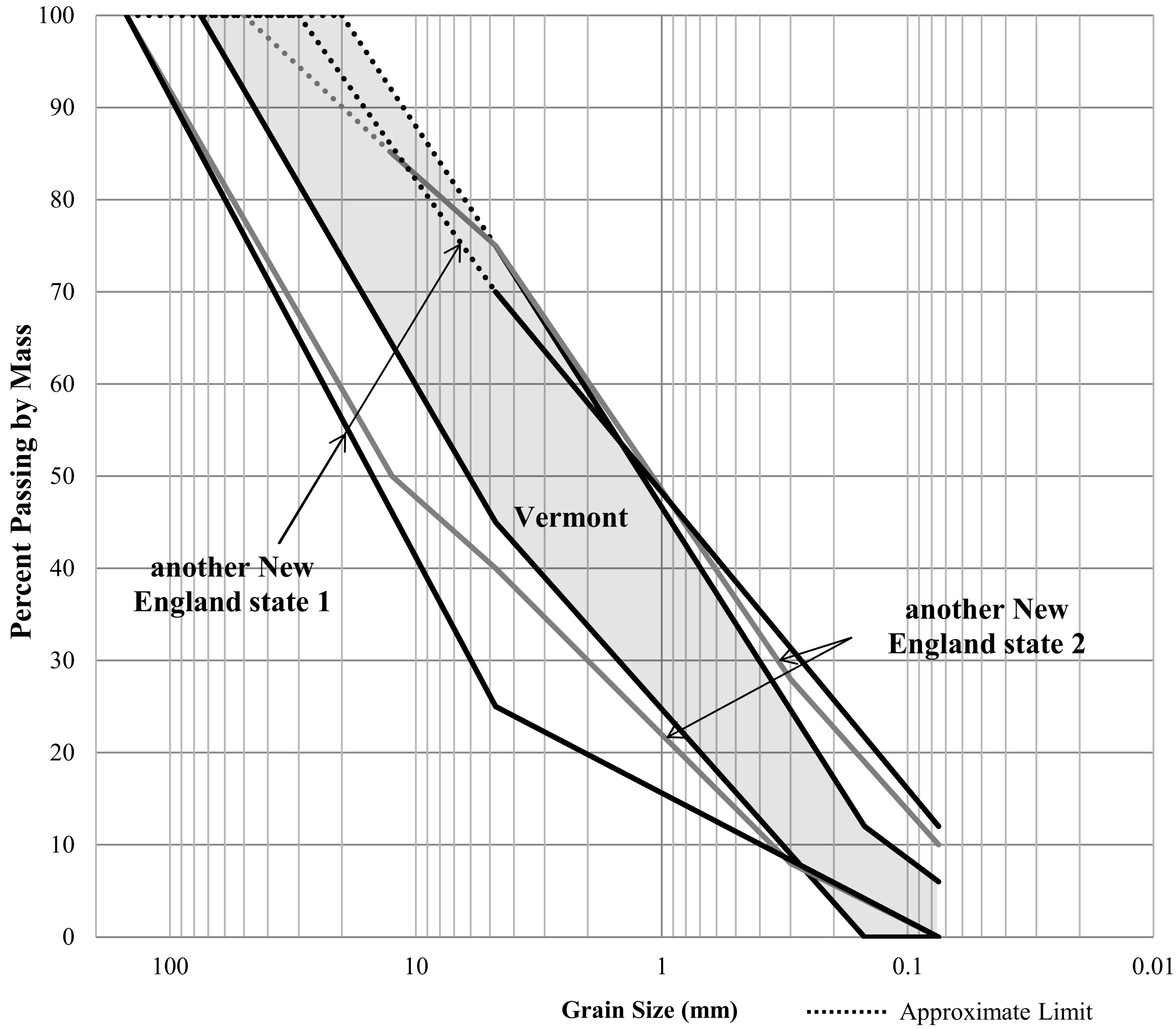


Figure 2.1: Backfill gradation specification of VTrans compared to that of two other New England states

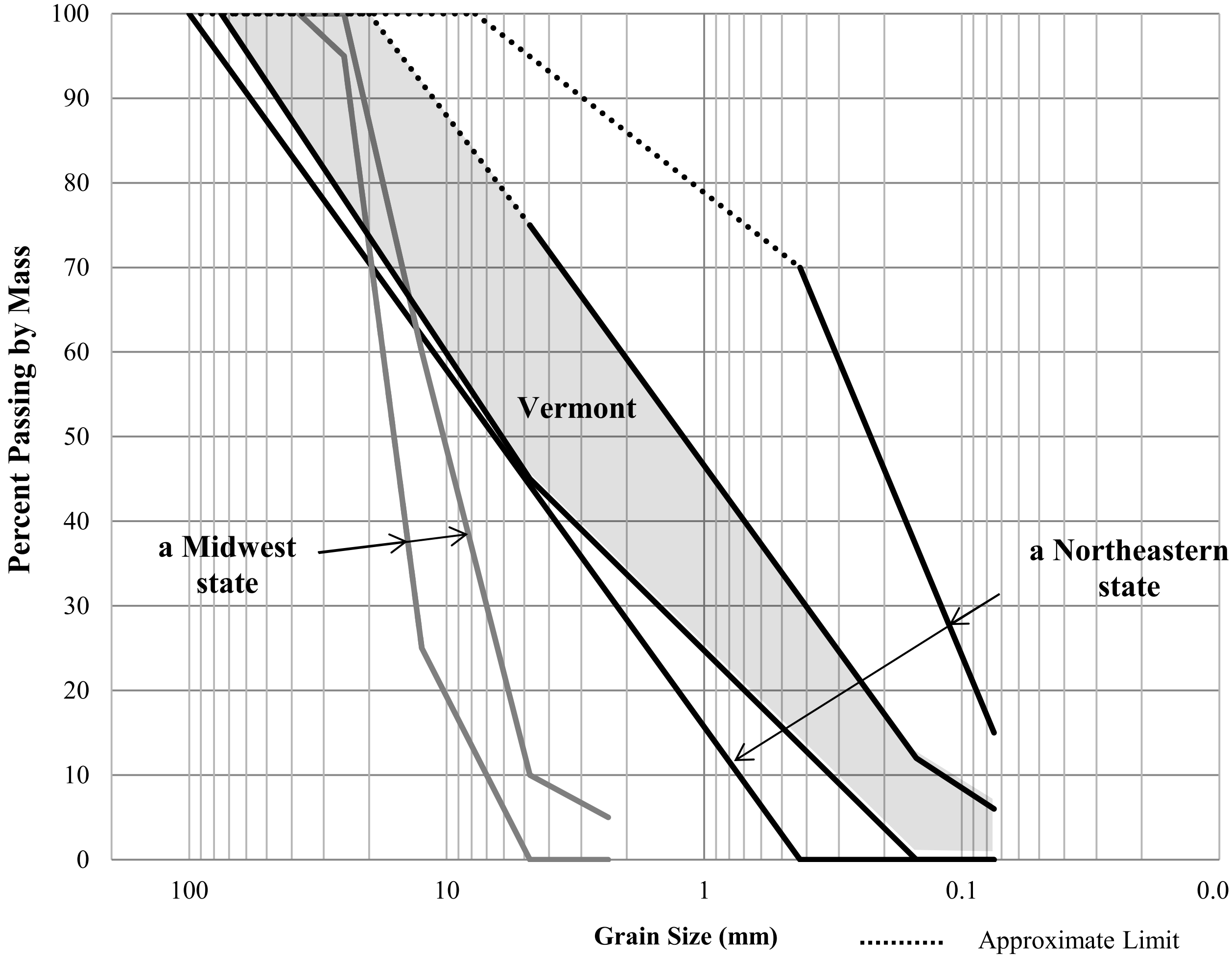


Figure 2.2: Backfill gradation specification of VTrans compared to a Northeast state and a Midwest state

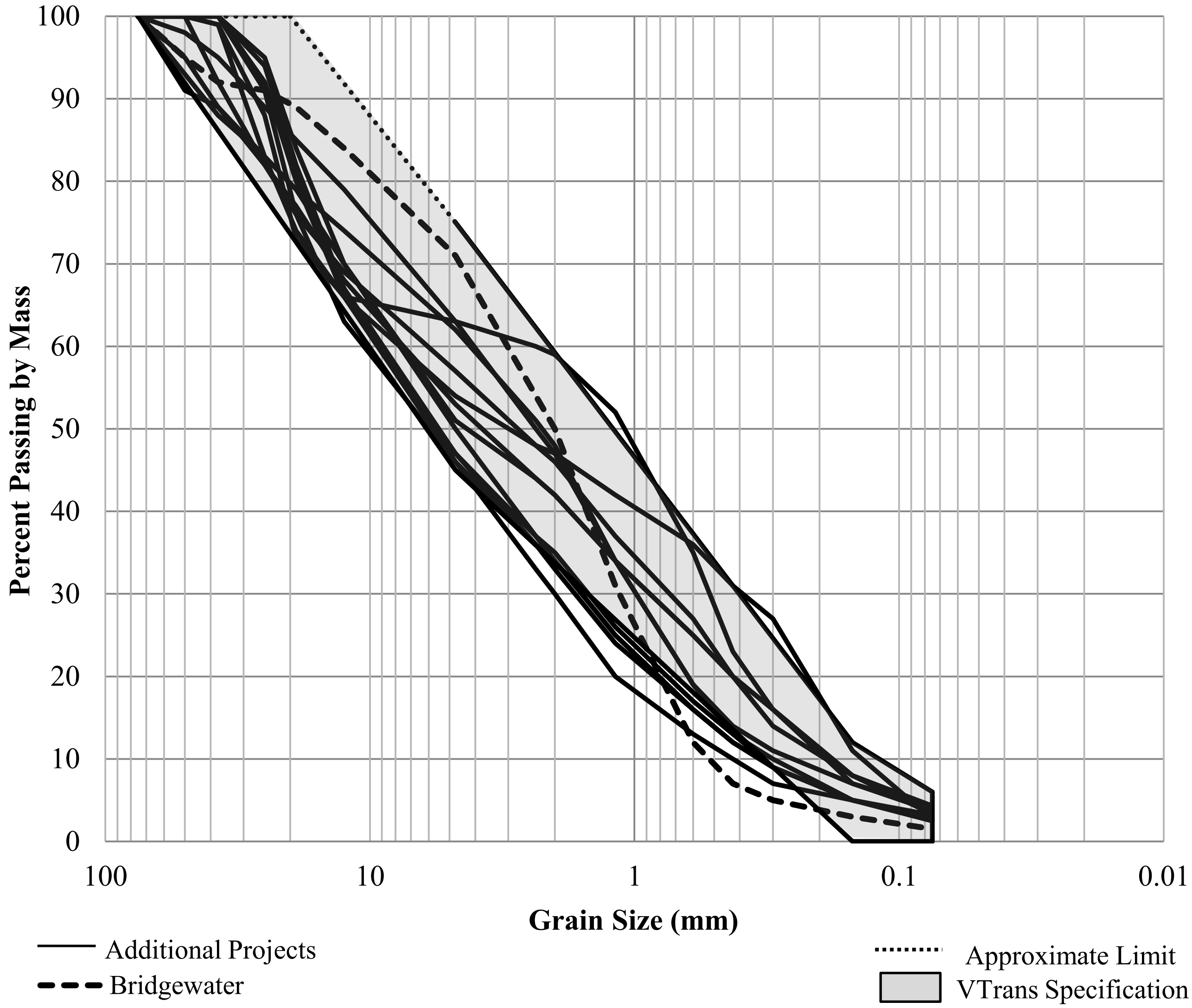


Figure 2.3: Gradations of backfills of past abutment projects of VTrans

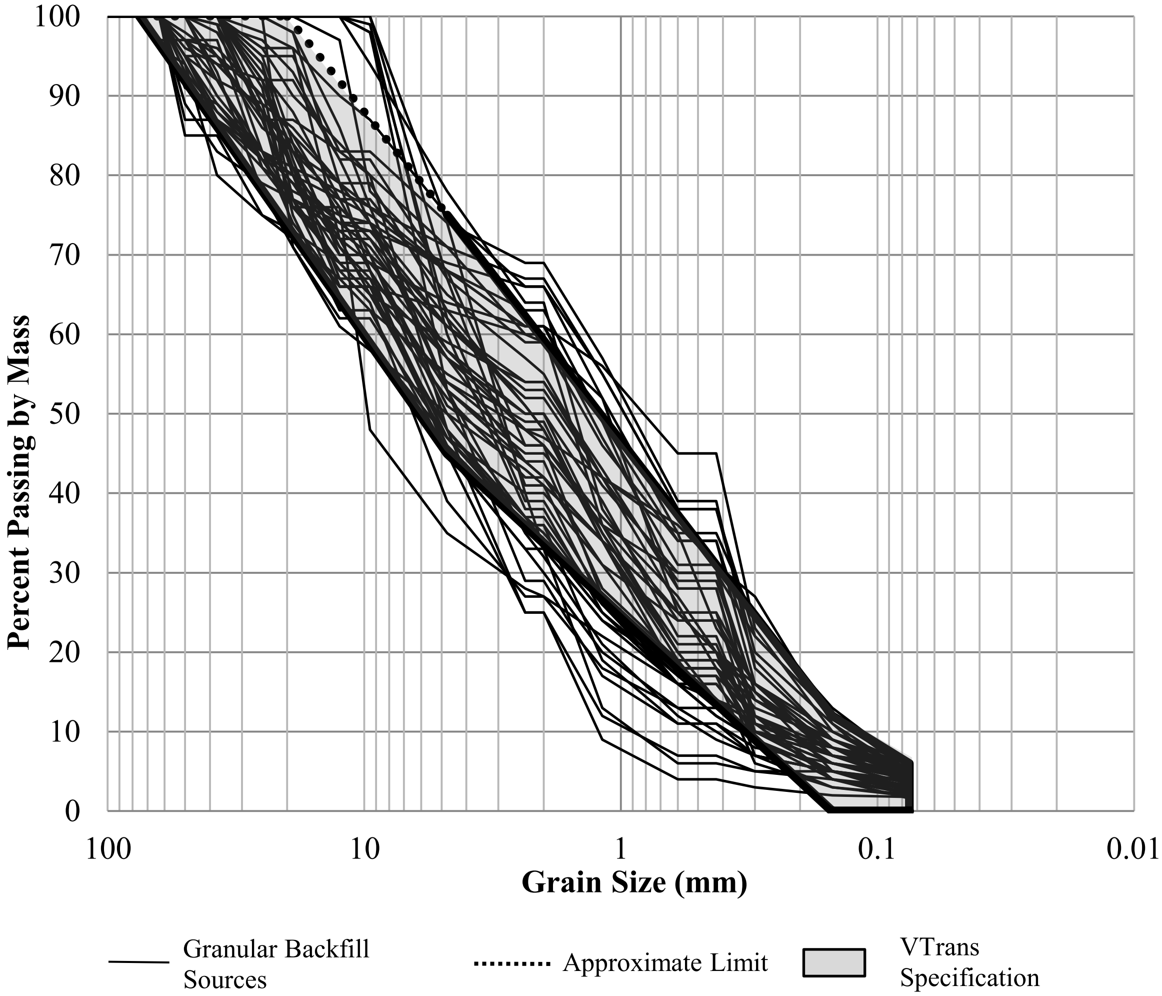


Figure 2.4: Gradations of some source soils available in Vermont

Moisture-density reports were also analyzed to investigate the compaction of granular backfill material used in past VTrans projects. Data were available from a total of 52 projects. For each site, relative compaction and moisture content measurements were available for between 1 and as many as 18 locations. A total of 211 data points were available from the 52 projects. The data contained field density and mositure content measurements made using nuclear gauges within abutment backfills during construction. The data sheets often included the minimum compaction required by VTrans at the measurement locations. These were specifically reported for 165 measurements. Figure 2.5 presents results of the relative compaction tests achieved in a histogram format. Figure 2.6 summarizes the minimum relative compaction required by VTrans at the measurement location again in a histogram format.

The results of Figure 2.5 indicate that a range of minimum relative compaction values (based on standard Proctor test) were obtained on past VTrans bridge abutment projects. Approximately 75% of the measurements recorded relative compaction between 95 and 100%. The mean for the relative compaction test data is approximately 97.25%. Figure 2.6 shows that the majority of tests have a specified minimum relative compaction of 95%. Approximately 40 tests were excluded because they did not report minimum required relative compaction. As seen in Figure 2.7, about 70% of the data points recorded a moisture content more than 2% below the optimum.

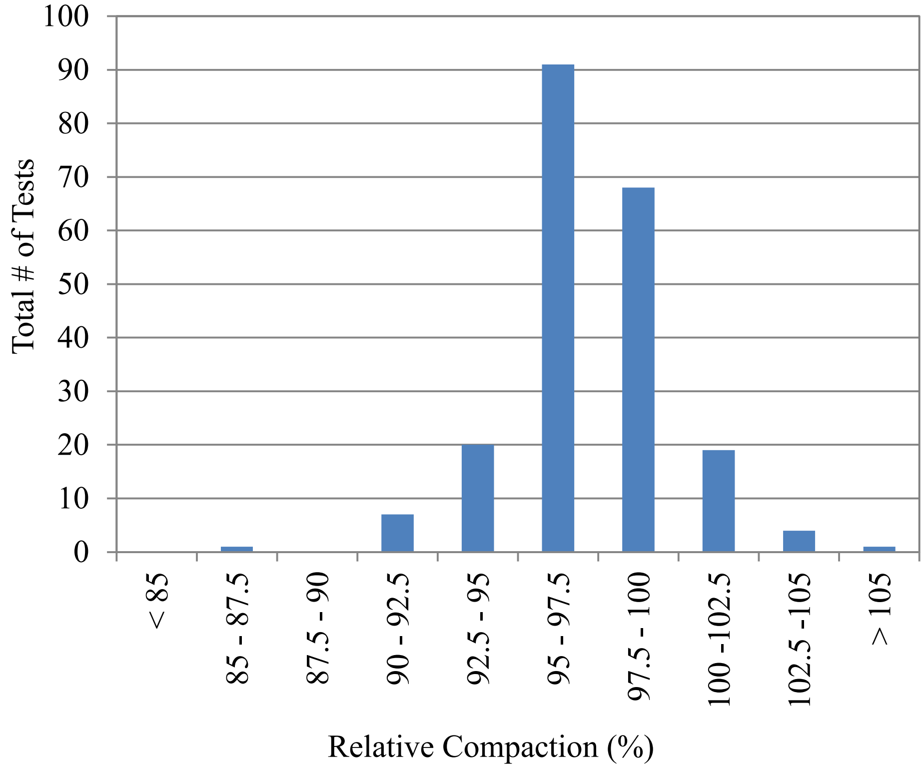


Figure 2.5: Histogram of relative compaction achieved in the field for VTrans abutment projects

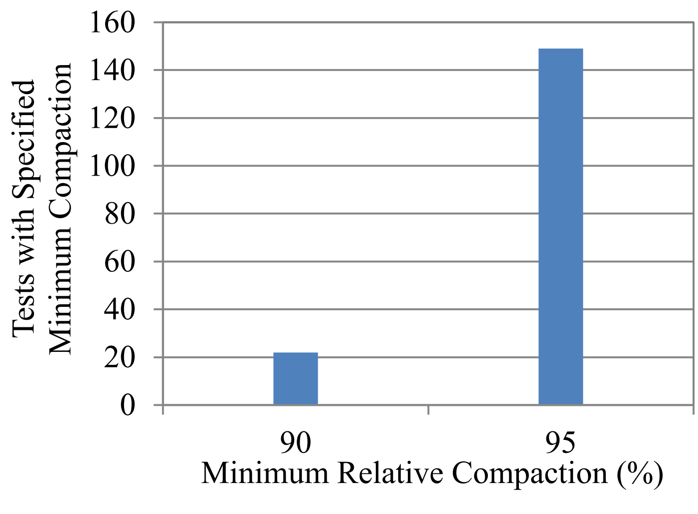
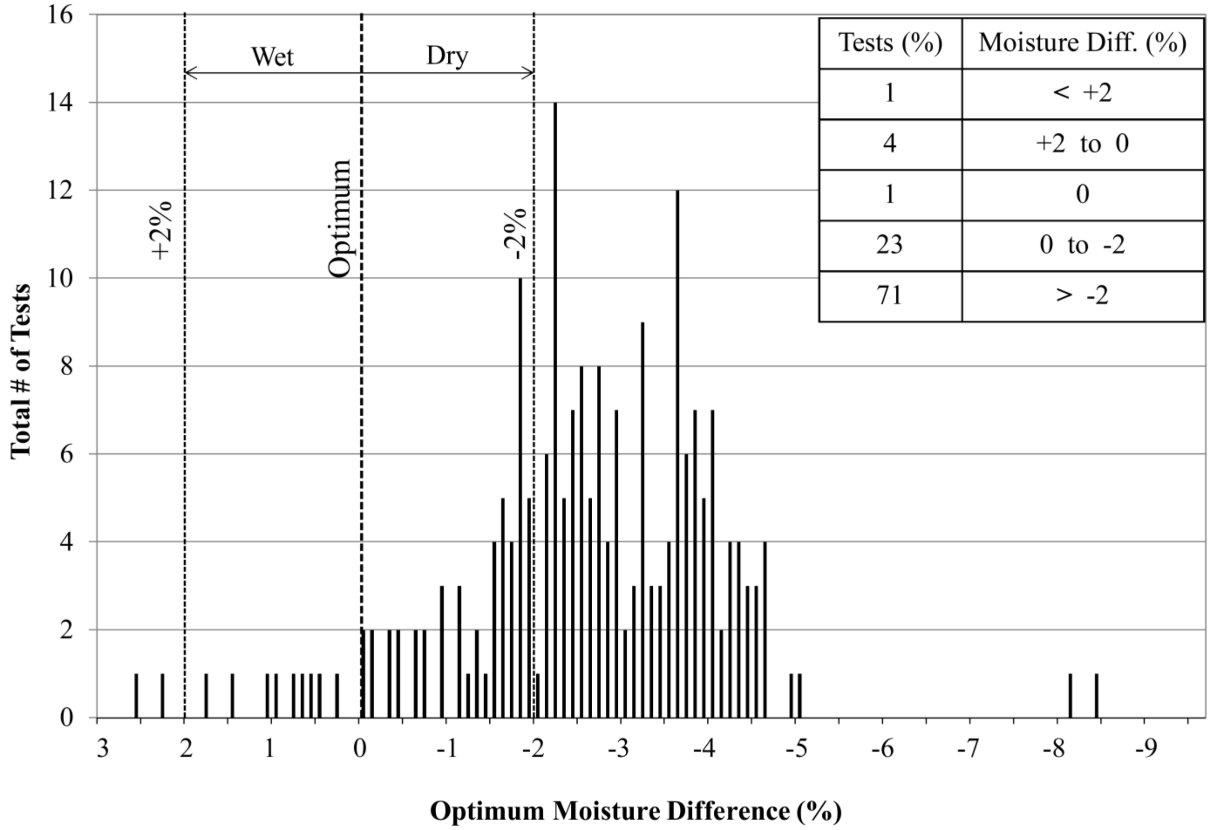


Figure 2.6: Histogram of the specified minimum relative compaction (based on standard Proctor test) for dry density measurements made in VTrans abutment projects



Tests (%)	Moisture Diff. (%)
1	< +2
4	+2 to 0
1	0
23	0 to -2
71	> -2

Figure 2.7: In-place optimum moisture (based on standard Proctor test) difference for past VTrans abutment projects

3. LITERATURE REVIEW

3.1 BACKGROUND

The Vermont Agency of Transportation (VTrans) currently limits the maximum allowable fines content, within granular structural backfill for cantilever retaining walls, to 6%. The American Association of State Highway and Transportation Officials (AASHTO) and other federal guidelines generally recommend a limit of 5%. The assumption appears to be that reduced fines content within the structural backfill will promote a free-draining condition, thereby eliminating the need to design an abutment or retaining wall to withstand additional hydrostatic pressure. There appears to be limited information or data to justify this maximum fines content recommendation made by various agencies, as well as the current maximum specified by VTrans. Therefore, this research included:

(1) An experimental study on a natural granular backfill material currently used by VTrans. The effects of increased fines content and confining pressure on hydraulic conductivity and shear strength parameters of the backfill material were determined using a flexible wall triaxial apparatus fitted with flow pumps; and

(2) Field observations of differential water levels between the stream and the backfill at two sites.

In this chapter, a summary of relevant literature on various aspects of the effects of non-plastic fines content on saturated hydraulic conductivity and shear strength of granular soils and associated experimental techniques is presented.

3.2 MAXIMUM ALLOWABLE FINES CONTENT

The maximum allowable fines content refers to the percentage of soil mass particles passing a No. 200 sieve (<0.075mm) compared to the total mass of the soil. Maximum allowable fines content is specified to limit the uncertainty introduced by certain fines types. The evaluation of the stress induced by cohesive soils is highly uncertain due to their sensitivity to shrinkage and swelling as well as a varied degree of saturation (AASHTO 2010). Furthermore, higher fines content can reduce the ability of the backfill to drain properly. The term 'free-draining' is commonly used in test method standards and design specifications to describe the ability of water to drain from soil.

Various Federal guidelines from the literature were reviewed for their recommendations regarding backfill materials. Most of these sources mentioned free-draining but no specific hydraulic conductivity was found to be associated with this term. Table 3.1 summarizes backfill recommendations from five different sources:

Table 3.1: Summary of fines limitations from various publications

Source	Maximum Fines Content	Notes (regarding fines)
AASHTO (2010)	5%	Free-draining defined as < 5%
FHWA (2006)	5%	15% if rapid drainage is not required
NAVFAC (1986)	15%	Ensure proper drainage
UFC (2005)	5%	Free-draining defined by soil type
USACE (1989)	Not Prescribed	Clean Sands and Gravels

The U.S. Army Corps of Engineers (USACE) and Naval Facilities Engineering Command (NAVFAC), along with the Air Force Civil Engineer Support Agency (AFCESA), are responsible for administration of the Unified Facilities Criteria (UFC) documents, which is primarily responsible for providing technical criteria for military construction. Although USACE (1989) did not prescribe a maximum fines limit of 5% fines, clean sands and gravels are limited to 5% fines by definition, as per the Unified Soils Classification System (USCS). NAVFAC DM 7.02 suggests up to 15% fines; however, clays and other fine-grained soils, as well as granular soils with considerable amount of clay and silt (greater than or equal to 15%) are not commonly used as backfill material for retaining walls or abutments. Where they must be used, it is typically recommended that the earth pressure should be calculated using at-rest conditions or higher pressure – to account for the potentially poor drainage conditions, swelling, and frost action that may occur (NAVFAC 7.02, 1986).

AASHTO (2010) advises caution in the determination of lateral earth pressures when using cohesive soils and recommends that if possible, cohesive or other fine-grained soils should be avoided as backfills. AASHTO defines a free-draining backfill as a material containing less than 5% passing a No. 200 sieve. If the material is free-draining (a fines content limited to 5%), the need to design the structure to withstand hydrostatic pressure is eliminated (AASTO 2010).

To account for hydrostatic pressure, the majority of respondents from the previously mentioned state survey indicated that drainage was incorporated in the design rather than assuming the presence of water pressures. The California Department of Transportation (Caltrans) Bridge Design Specification (2004) requires drainage provision and restrictions to high plasticity materials as backfill in locations where retaining walls and abutments are used so that hydrostatic pressure does not need to be considered. However, the design specifications also state that structures along rivers and canals shall require a minimum differential hydrostatic pressure equal to 3 ft of water be considered for design. It also states that in situations of rapid drawdown or significant river fluctuations, a greater differential may be required or a more rapidly free-draining backfill material may be used, such as open graded coarse gravel or shot rock (rip-rap).

Merriman (1955) showed that soil types can be grouped by effectiveness of vibratory compaction (

Table 3.2). The compaction of a soil relies on the ability of water to move through soil pores as the void space decreases. Hydraulic conductivity tests performed by Merriman (1955) on these soil types revealed that due to the strong variation in hydraulic conductivity with little variation in gradation, it cannot be used as a measure for how well a soil may compact. In

Table 3.2, the soil types GW-GM, GW-GC, GP-GM, GP-GC are considered generally suitable with less than 8% fines; and soils SM and SC can contain up to 16% fines, requiring special consideration. The two primary factors that affected the compaction of these materials were the gradation and the plasticity of the fines present. The larger the particle size and the more uniform the gradation, greater is the allowable fines content to obtain satisfactory densities. Also, with less plasticity of the fines, a greater fines content was allowed (Merriman, 1955).

The soil types presented in Table 3.2 are also discussed in UFC (2005). Soil types GW, GP, SW and SP are defined as free-draining, pervious soil types with a maximum of 5% fines. This maximum fines content is defined by the Unified Soil Classification System (USCS), and therefore, contains less than 5% fines by definition.

The current ASTM test method (D4254 and D4253) for determining the minimum and maximum dry densities, respectively, defines free-draining as a cohesionless material with less than 15% fines content. The ability of the soil to drain without developing pore pressure allows the soil to be effectively compacted.

Table 3.2: Suitability of soils for compacted backfill (Merriman 1955)

Soil type	Limitation[1]
GW, GP, SW, SP	All soil types are suitable. Fines in these soils are limited to less that 5 percent by definition.
GW-GM, GW-GC GP-GM, GP-GC	May or may not be suitable, depending on gradation and plasticity.
SW-SM, SP-SM, SP-SC	Test section may be required. Fines are limited to 5 to 12 percent by definition.
SM, SC	Normally unsuitable.

[1] Fines are particles smaller than U.S.A. Standard 75 μm sieve (No. 200).

3.3 TESTING METHODS

Testing of granular, cohesionless material can produce high variation and difficulty in reproducibility. A cooperative study was performed under the sponsorship of the American Society for Testing and Materials (ASTM), where 41 soil laboratories carried out typical soil mechanics tests to determine the variation associated with gradation, minimum and maximum density, and Proctor compaction tests on identical specimens of fine sand and gravelly sand (Tavenas, 1973). All tests indicated a large variability and low reproducibility within the different types of tests performed. Also, variation among different laboratories was as high as two to three times greater than variation between duplicate tests. Some of the factors that have the largest effect on the variation and reproducibility on identical tests include the size of the sample and the method of physically sampling or handling the soil material (Tavenas, 1973).

Current ASTM standard D6913 addresses the sample size to account for the maximum particle within the soil. The ASTM standard D6913 addresses the issue related to sampling by suggesting that the splitting or partitioning of samples for various tests be excluded or limited to no more than a few times, otherwise the sample will no longer be representative. This will limit the segregation of particles where samples would have decreased fines and increased larger particle sizes. The standard suggests using moist sampling to provide temporary cohesion and is especially recommended if the maximum particle size is less than 19 mm (¾" sieve).

3.4 EFFECT OF FINES ON SOIL PROPERTIES

3.4.1 COMPACTION

Table 3.3 presents recommendations for compaction of various USCS soil types made by UFC (2005) along with a comparison of compaction test types described in Table 3.4. As seen in Table 3.4, the CE 55 test is comparable to the AASHTO T-180 (ASTM D1557). For the GW, GP, SW and SP soil types, the in-place water content for compaction is 100% saturation. The NAVFAC 7.02 (1986) manual describes soils that are sensitive and not sensitive to compaction moisture. Silts and silty-sands typically have steep moisture-density curves and can be more difficult to compact to the specified relative compaction in the field due to their sensitivity to moisture. Soils that are coarse-grained and well-graded with less than 4% fines (or less than 8% fines for soil of uniform gradation) are typically not sensitive to compaction moisture. These materials are capable of being compacted at near fully saturated moisture contents in the field. If a soil has a hydraulic conductivity greater than 1 x 10^{-3} cm/sec, it is considered to be insensitive to compaction moisture (NAVFAC 7.02 1986).

Table 3.3: Summary of compaction criteria (UFC, 2005)

Soil Group	Soil Types		Degree of Compaction	Fill and Backfill — Typical Equipment and Procedures for Compaction: Equipment	No. of Passes or Coverages	Comp. Lift Thick., in.	Placement Water Content	Field Control
Pervious (Free-Draining)	GW, GP, SW, SP	Compacted	90 to 95% of CE 55 maximum density	Vibratory rollers and compactors	Indefinite	Indefinite	Saturate by flooding	Control tests at intervals to determine degree of compaction or relative density
			75 to 85% of relative density	Rubber-tired roller[b]	2-5 coverages	12		
				Crawler-type tractor[c]	2-5 coverages	8		
				Power hand tamper[d]	Indefinite	6		
		Semicompacted	85 to 90% of CE 55 maximum density	Rubber-tired roller[b]	2-5 coverages	14	Saturate by flooding	Control tests as noted above, if needed
			65 to 75% of relative density	Crawler-type tractor[c]	1-2 coverages	10		
				Power hand tamper[d]	Indefinite	8		
				Controlled routing of construction equipment	Indefinite	8-10		
Semipervious and Impervious	GM, GC, SM, SC, ML, CL, OL, OH, MH, CH, OH	Compacted	90 to 95% of CE 55 maximum density	Rubber-tired roller[b]	2-5 coverages	8	Optimum water content	Control tests at intervals to determine degree of compaction
				Sheepsfoot roller[e]	4-8 passes	6		
				Power hand tamper[d]	Indefinite	4		
		Semicompacted	85 to 90% of CE 55 maximum density	Rubber-tired roller[b]	2-4 coverages	10	(A) Optimum water content (B) By observation; wet side-maximum water content at which material can satisfactorily operate, dry side-minimum water content required to bond particles and which will not result in voids or honeycombed material	(A) Control tests as noted above, if needed (B) Field control exercised by visual inspection of action of compacting equipment
				Sheepsfoot roller[e]	4-8 passes	8		
				Crawler-type tractor[c]	3 coverages	6		
				Power hand tamper[d]	Indefinite	6		
				Controlled routing of construction equipment	Indefinite	6-8		

Note: The above requirements will be adequate in relation to most construction. In special cases where tolerable settlements are unusually small, it may be necessary to employ additional compaction equivalent to 95 to 100% of compaction effort. A coverage consists of one application of the wheel of a rubber-tired roller or the threads of a crawler-type tractor over each point in the area being compacted. For a sheepsfoot roller, one pass consists of one movement of a sheepsfoot roller drum over the area being compacted.

a From TM 5-818-1.
b Rubber-tired rollers having a wheel load between 18,000 and 25,000 lb and a tire pressure between 80 and 100 psi.
c Crawler-type tractors weighing not less than 20,000 lb and exerting a foot pressure not less than 6-1/2 psi.
d Power hand tampers weighing more than 100 lb; pneumatic or operated by gasoline engine.
e Sheepsfoot rollers having a foot pressure between 250 and 500 psi and tamping feet 7 to 10 in. in length with a face area between 7 and 16 sq in.

Table 3.4: Compaction test comparison (UFC, 2005)

Test Designation	Blows Per Layer	No of Layers	Hammer Weight lb	Hammer Drop in	Mold	
					Volume cu ft	Diameter in
US Army Corps of Engineers (MIL-STD-631A)						
CE 55	55	5	10	18	0.0736	6
CE 12	12	5	10	18	0.0736	6
ASTM						
D-1557 Modified Proctor	25	5	10	18	0.0333	4
	56	5	10	18	0.0750	6
Standard Proctor	25	3	5.5	12	0.0333	4
American Association of State Highway and Transportation Officials (AASHTO)						
T-180 Modified AASHTO	25	5	10	18	0.0333	4
	56	5	10	18	0.0750	6
T-99 Standard AASHTO	25	3	5.5	12	0.0333	4

The ASTM test methods for standard and modified Proctor densities (ASTM D698 and D1557) state that the test methods will generally produce a well-defined maximum dry density compaction curve for non-free draining soils (fines content greater than 15%). However, if either method is used for free-draining soils, the maximum dry density may not be well defined and may be more easily obtained using ASTM D4253 Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table (ASTM D698 and D1557). The D4253 test method is suggested for use on free-draining materials, which is defined as having a maximum fines content of 15%.

In the study performed by Tavenas (1973), the results of their standard and modified Proctor compaction tests showed better reproducibility than maximum densities obtained using a vibratory table. The use of the vibratory table method can reduce crushing of particles and was expected to show higher reproducibility. Due to the magnification of error in the maximum and minimum densities, Tavenas (1973) recommends using relative compaction based on the modified Proctor density as a good evaluator for compactness in cohesionless soils.

For many types of free-draining, cohesionless soils, the Proctor test methods can cause a moderate amount of degradation due to the amount of compaction energy. When degradation occurs, typically there is an increase in the maximum dry density that is recorded in the laboratory. Therefore, the laboratory dry density obtained will not be representative of field conditions and in some cases may not be achievable in the field due to the misleading maximum dry density measurement (ASTM D1557).

In previous studies, the increase in fines content has produced higher maximum dry densities for natural granular soils. Merriman (1955) tested sands and granular soils with fines ranging from 0-18% and showed that maximum dry density increased with an increase in fines content for relative density measurements. For the 95% standard Proctor density tests, the density of the sand-gravel mixture reached a maximum at 13% fines content and remained unaffected by an increase in fines between 13 – 18%. For relative density tests, compacted with a vibratory table, the densities began decreasing at 9% fines content.

Siswosoebrotho et al. (2005) showed that a critical fines content existed for a sand with added non-plastic fines ranging from 0 – 16%. The maximum dry density increased with additional fines and began to decrease at a fines content of 9%.

3.4.2 HYDRAULIC CONDUCTIVITY

Published data in the open literature on the effects of fines on the hydraulic conductivity and shear strength of compacted granular backfill soils by systematically varying the fines content appears to be sparse. Relevant laboratory data found in the literature on hydraulic conductivity measurements of granular soils with varying fines content (non-plastic or nearly non-plastic) are summarized in Table 3.5. For brevity the ranges of hydraulic conductivity are provided in the last column of Table 3.5. The smaller hydraulic conductivity is typically associated with higher end of fines content.

Table 3.5: Comparison of hydraulic conductivity testing with other investigations

Investigation	**Base Soil Type** AASHTO (8) (USCS) (9)	**Fines Content (%)**	**Moisture Content**	**Density**	**Permeameter Type and Sample Diameter**	**Confining Pressure kPa (psi)**	**Hydraulic Conductivity (cm/s)**
Merriman (1955)	Fine sand and coarse sand A-3 (SP); sand-gravel mixture A-1-b (SP*)	0-18	Optimum	95% of std. Proctor or 70% RD	Consolidometer, falling head 10.8 cm (4.23 in.)	138 (20) Normal Stress	Fine sand: 1×10^{-3} to 4×10^{-4} Coarse sand: 2×10^{-3} to 8×10^{-4} Sand-gravel mixture: 9×10^{-3} to 2×10^{-5}
Siswosoebrotho et al. (2005)	A-1-a (SP*)	0-16	Optimum	95% of mod. Proctor	Rigid compaction, falling head 15.2 cm (6.0 in.)	NA	9.6×10^{-3} to 1.3×10^{-3}
Bandini and Sathiskumar (2009)	50-50 sand A-1-b (SP); ASTM sand A-3 (SP)	0-25	NA	NA	Flexible wall, constant and falling head 7.0 cm (2.75 in.)	50 (7), 100 (15), 200 (29), 300 (44)	2×10^{-3} to 2.2×10^{-5}

* Although the authors do not specifically report, particles greater than 19 mm (3/4 in.) were probably removed from the base soil prior to testing.
NA: not applicable

Merriman (1955) conducted falling head permeability tests in a rigid permeameter on three types of compacted natural soils (a fine sand, a coarse sand, and a sand-gravel mixture). Figure 3.1 shows the gradations of soils tested by Merriman (1955). The soils were first washed to remove the natural fines and then incremented with natural non-plastic 0 – 18% silt. Results showed that the addition of non-plastic fines between 0 - 18% reduced the hydraulic conductivity significantly as summarized in Table 3.5. Merriman (1955) concluded hydraulic conductivity to be an unreliable indicator of how well a soil could be compacted. Additionally, the higher plasticity fines tended to reduce the hydraulic conductivity more than non-plastic fines.

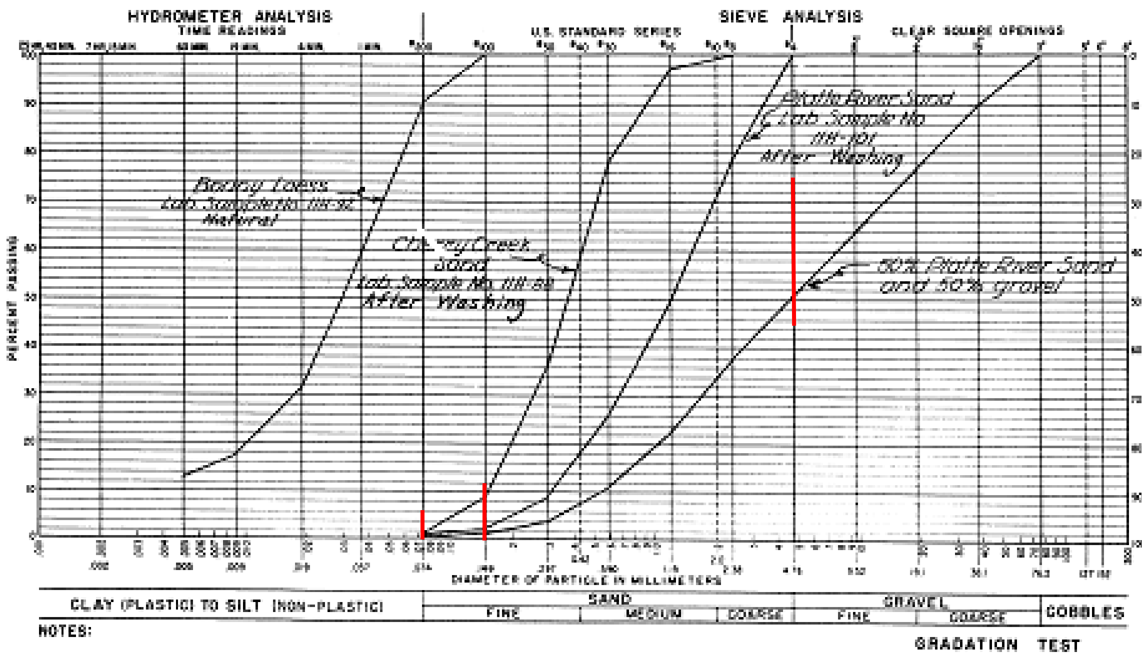


Figure 3.1: Soil types tested by Merriman (1955) with VTrans gradation specification limits shown

Tests performed by Siswosoebrotho et al. (2005) on granular aggregate with fines contents of 0 – 16% showed hydraulic conductivity decreases with the addition of fines. Values of hydraulic conductivity varied within one order of magnitude for the non-plastic fines between 9.6 x 10^{-3} to 1.3 x 10^{-3} cm/s for 0 – 16% fines. Mixtures of plastic fines, with a plasticity index range of 5 to 13, had a range of hydraulic conductivity between 1.2 x 10^{-3} to 2.6 x 10^{-5} cm/s.

Tests were performed in a 150 mm compaction permeameter using constant head and falling head tests with samples prepared with the modified Proctor method.

Laboratory hydraulic conductivity tests performed by Bandini and Sathiskumar (2009) on fine sand with fines contents of 0-25% showed that hydraulic conductivity decreased with an increase in fines content and confining pressure (Figure 3.2 and Figure 3.3, respectively). Comparisons were made between two types of fine sand, an Ottawa sand and ASTM 20-30. Both materials are poorly graded. The silt used was a ground silica Sil-Co-Sil manufactured material. Their tests were performed using a flexible wall permeameter on specimens prepared at 160 mm (6.3 in.) height by 70 mm (2.8 in.) diameter. Tests were performed using a pressure control panel with burettes. As seen in Figure 3.2 and Figure 3.3 hydraulic conductivity decreased with increasing fines content and confining stress, respectively. It appears that at fines contents greater than about 15%, the decrease in hydraulic conductivity was more pronounced. However, comparison of the effects of fines on hydraulic conductivity at specified relative compaction (which could be related to field compaction) was not done in this study.

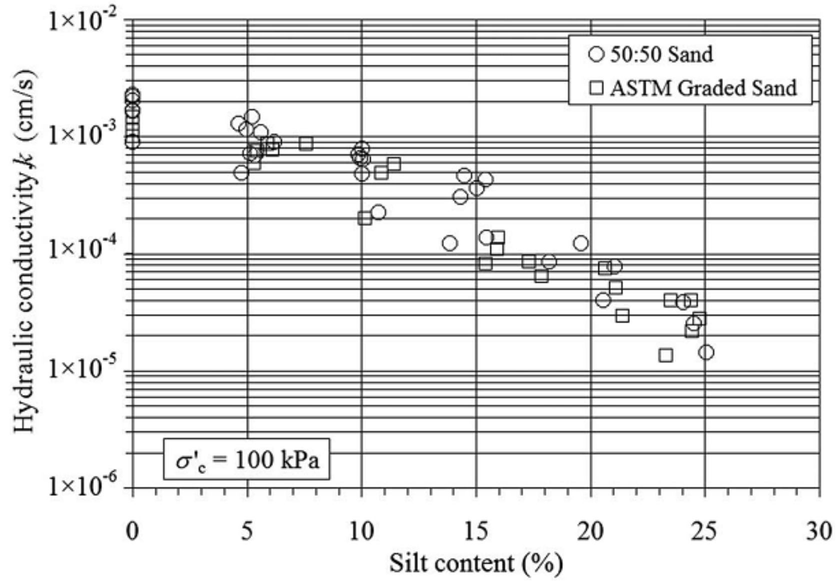


Figure 3.2: Hydraulic conductivity versus silt content (Bandini and Sathiskumar, 2009)

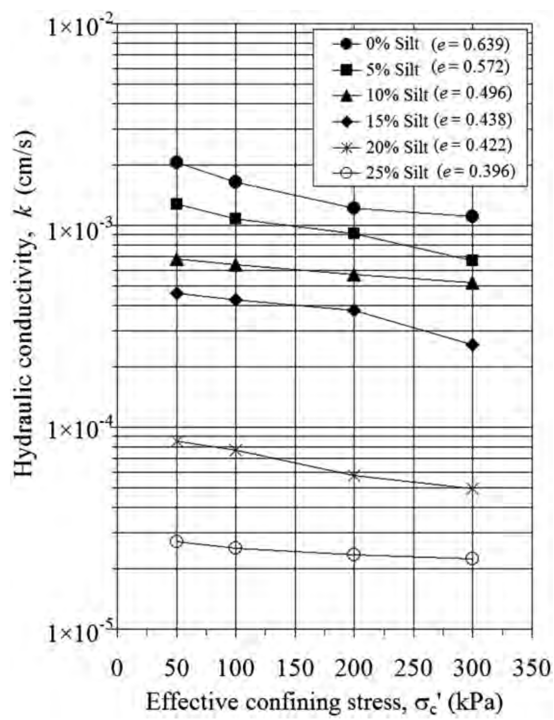


Figure 3.3: Hydraulic conductivity versus confinement (Bandini and Sathiskumar, 2009)

NAVFAC DM 7-1 (1986) has guidelines for coarse grained materials mixed with different types of fines, shown in Figure 3.4. For silt contents between 0 – 15% the hydraulic expected conductivity ranges from 5 x 10^{-3} cm/s to 5 x 10^{-8} cm/s, respectively. This guideline does not mention how the hydraulic conductivities were measured; using a flexible wall permeameter or a rigid wall compaction permeameter. As mentioned in the previous section, NAVFAC 7.02 (1986) defines soils that are not affected by compaction moisture as having hydraulic conductivities greater than approximately 1 x 10^{-3} cm/sec.

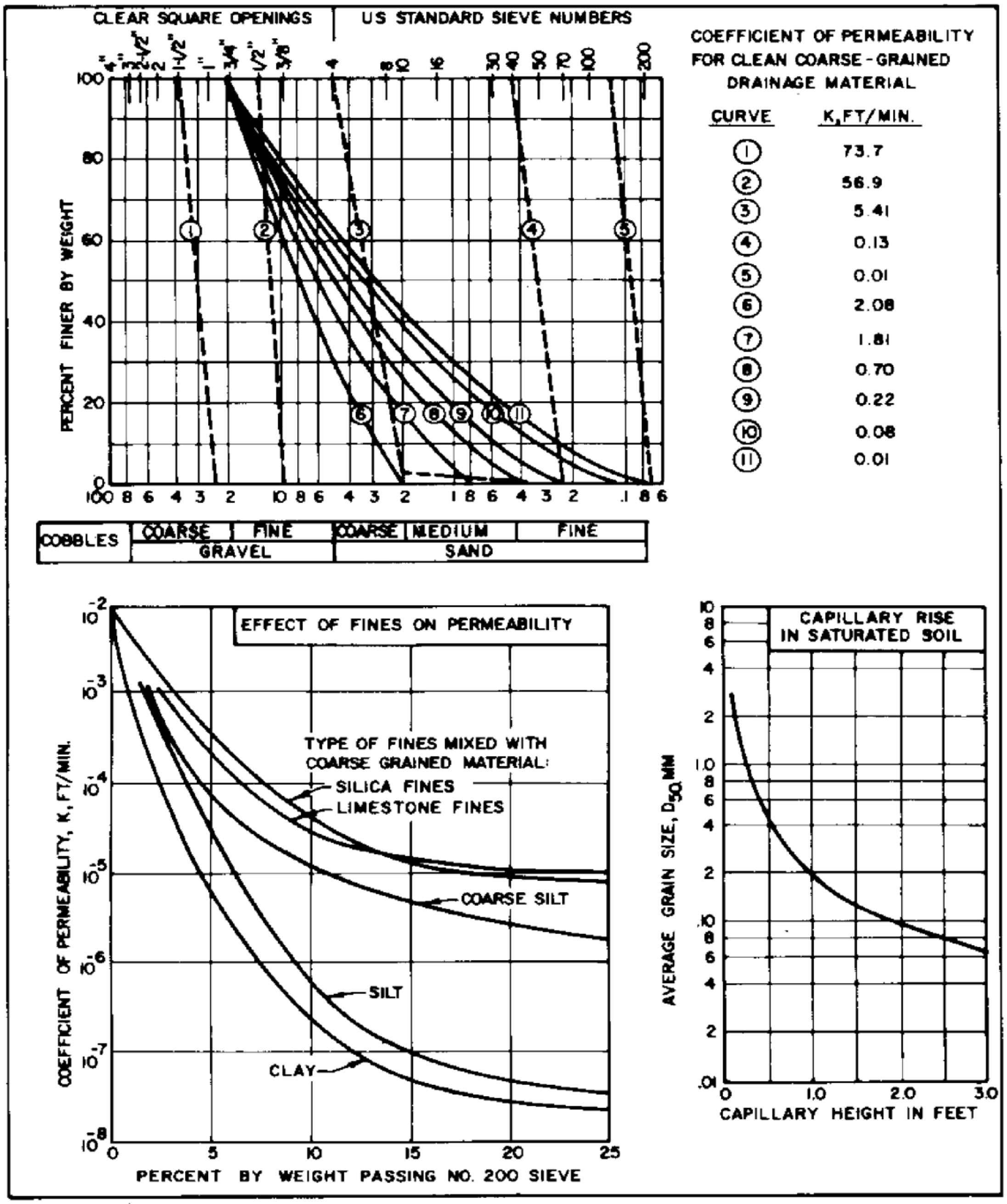


Figure 3.4: Grain size distribution and hydraulic conductivity (NAVFAC DM7-1, 1986)

NAVFAC DM 7-2 (1986) has typical values of compacted soils based on soil type classification in

Table 3.6. The values shown for hydraulic conductivity were tested at maximum dry density using the modified Proctor method. The soil types described as free-draining previously in UFC (2005) are shown. The hydraulic conductivities for GW and GP are 2.5 x 10^{-2} and 5 x 10^{-2} cm/sec, respectively. Soils SW and SP are both characterized by hydraulic conductivities greater than 5 x 10^{-4} cm/sec. The hydraulic conductivity decreases with increasing fines based on soil types. Within the group of free-draining soils, the fines content does not change. However, with decreased gravel content (SW and SP types) the hydraulic conductivity increases. The hydraulic conductivity appears to be affected by the overall soil gradation as opposed to solely the fines content. For silty gravel (GM, containing greater than 12% fines), the hydraulic conductivity is greater than 5 x 10^{-7} cm/sec. Figure 3.5 shows the hydraulic conductivity could range from slightly less than 10^{-2} cm/s to almost about 10^{-8} cm/sec. Well graded gravel, (GW) containing less than 5% fines, is expected to have a hydraulic conductivity greater than 2.5 x 10^{-2} cm/sec.

Terzaghi and Peck (1967) discussed drainage properties of different soil types as seen in Figure 3.6. For soil types with a hydraulic conductivity range between 10^{2} to 10^{-4} cm/s, the drainage performance was considered good. Soils with hydraulic conductivities as low as 10^{-3} cm/s include clean gravel, clean sands and gravel mixtures. Hydraulic conductivity in a range of 10^{-3} to 10^{-4} cm/s would indicate good drainage in soils including very fine sands, organic and inorganic silts, mixtures of sand silt and clay, etc. Terzaghi and Peck (1967) recommend that a constant head permeameter be used for testing hydraulic conductivity greater than 10^{-3} cm/s. A falling head permeameter is recommended for soils with hydraulic conductivities between 10^{-3} to 10^{-6} cm/s, but Terzaghi and Peck (1967) caution that rigid wall permeameter tests are unreliable and user experience is needed for testing. They also recommend that for hydraulic conductivity values less than 10^{-6} cm/s, falling head permeameter tests are fairly reliable, but even greater user experience is required for testing.

Terzaghi and Peck (1967) recommend using soil filter materials with a maximum particle size of 3 in. and not containing greater than 5% passing the No. 200 sieve. The filter material refers to the material placed between the native soil and a drain pipe. This type of drainage is typically used behind retaining walls. The filter material should also be uniformly graded as opposed to gap graded. They found that filter material needs to have voids that are small enough to prevent the migration of fines but large enough to allow water to flow out.

ASTM D2434 Standard Test Method for Permeability of Granular Soils (constant head) limits the test standard to granular material with a maximum fines content of 10% and for specimens with a hydraulic conductivity greater than 1 x 10^{-3} cm/s. This standard also indicates that the required porous stones should not have openings greater than the fines' particle size (0.075mm). ASTM D5084 Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter, states that the porous stones must have a significantly greater hydraulic conductivity so they do not inhibit or control the measured hydraulic conductivity value of the soil. The standard, however, does not specify a maximum pore size for porous stones.

Table 3.6: Typical properties of compacted materials (NAVFAC DM7-2, 1986)

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, percent	Typical value of compression: At 1.4 tsf (20 psi), percent of original height	Typical value of compression: At 3.6 tsf (50 psi), percent of original height	Typical strength characteristics: Cohesion (as compacted) psf	Typical strength characteristics: Cohesion (saturated) psf	Typical strength characteristics: ϕ (Effective stress envelope) degrees	Typical strength characteristics: Tan ϕ	Typical coefficient of permeability ft/min.	Range of CBR values	Range of subgrade modulus k lb/cu in.
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix.	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	....	....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	....	....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	5×10^{-5}	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	2×10^{-6}	....	
SC	Clayey sands, poorly graded sand-clay mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5×10^{-7}	5 - 20	100 - 300
ML	Inorganic silts and clayey silts .	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	10^{-5}	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	5×10^{-7}	....	
CL	Inorganic clays of low to med. plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	10^{-7}	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	....	....	....	....	....	.....	....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	5×10^{-7}	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	10^{-7}	15 or less	50 - 150
OH	Organic clays and silty clays . . .	65 - 100	45 - 21	....	....	....	....	....	.....	....	5 or less	25 - 100

Notes:

1. All properties are for condition of "standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown. (....) indicates insufficient data available for an estimate.

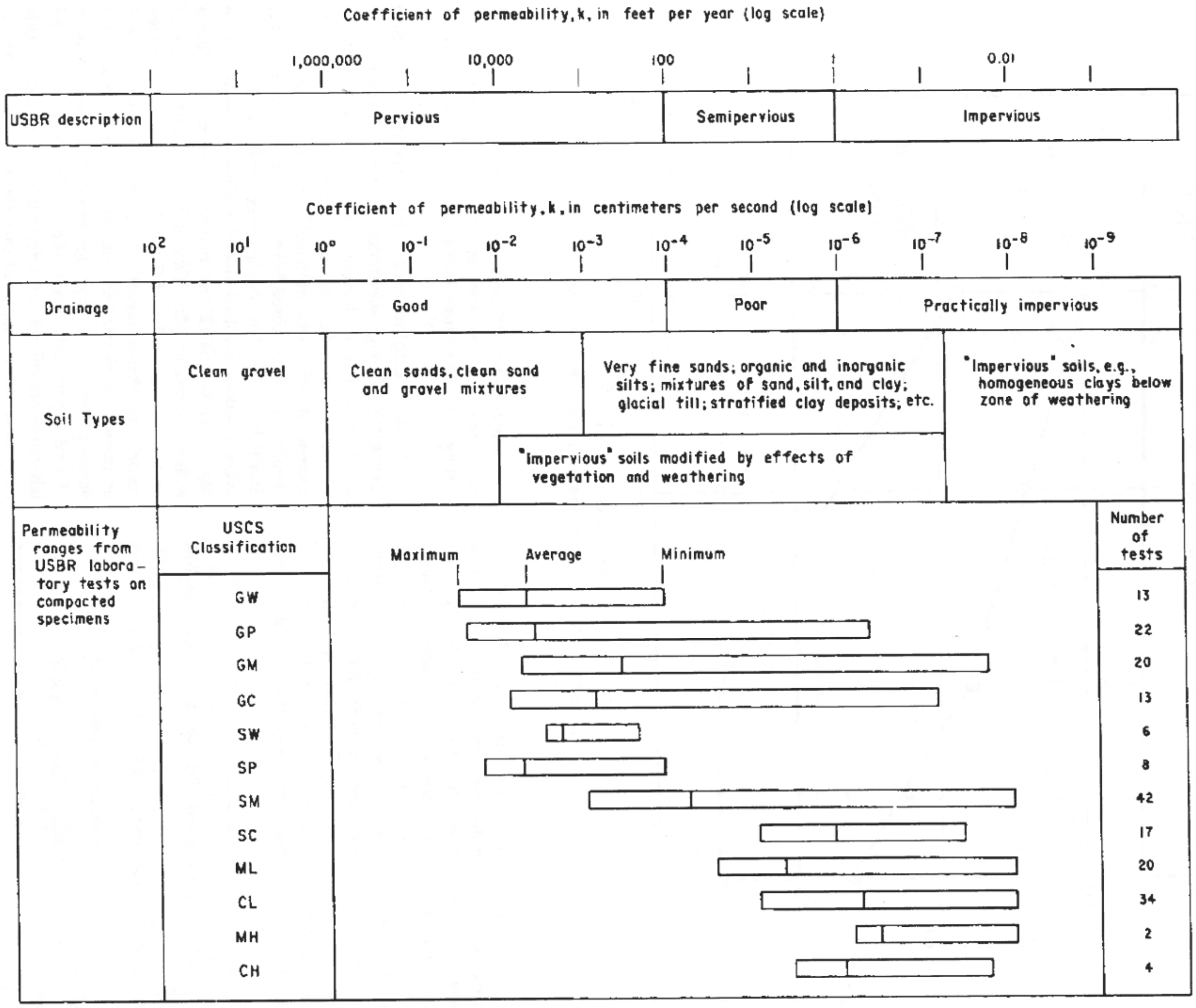


Figure 3.5: Typical hydraulic conductivity values for USCS soil types (USBR Earth manual part 1, 1998)

*Permeability and Drainage Characteristics of Soils**

Coefficient of Permeability k in cm per sec (log scale)

10^{2} | 10^{1} | 1.0 | 10^{-1} | 10^{-2} | 10^{-3} | 10^{-4} | 10^{-5} | 10^{-6} | 10^{-7} | 10^{-8} | 10^{-9}

Drainage	Good	Poor	Practically Impervious	
Soil types	Clean gravel	Clean sands, clean sand and gravel mixtures	Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.	"Impervious" soils, e.g., homogeneous clays below zone of weathering
			"Impervious" soils modified by effects of vegetation and weathering	
Direct determination of k	Direct testing of soil in its original position—pumping tests. Reliable if properly conducted. Considerable experience required			
	Constant-head permeameter. Little experience required			
Indirect determination of k		Falling-head permeameter. Reliable. Little experience required	Falling-head permeameter. Unreliable. Much experience required	Falling-head permeameter. Fairly reliable. Considerable experience necessary
	Computation from grain-size distribution. Applicable only to clean cohesionless sands and gravels			Computation based on results of consolidation tests. Reliable. Considerable experience required

* After Casagrande and Fadum (1940).

Figure 3.6: Hydraulic conductivity and drainage of given soil types (Terzaghi and Peck 1967)

The testing standards (D2434 and D5084) suggest that soils with up to 10% fines can have a hydraulic conductivity of 1 x 10^{-3} cm/s or greater and that soils with greater than 10% fines can have hydraulic conductivities smaller than 1 x 10^{-3} cm/s. The pores must be small enough for the porous stones to prevent the passage of fines, but large enough to allow water to flow. Typically, filter paper is included with the use of porous stones to prevent fines migration, but its effects on the overall hydraulic conductivity may lead to some error. A test should be performed using the intended apparatus without any specimen in the permeameter to evaluate the head losses (ASTM D5084).

ASTM D2434 suggests using a rigid wall permeameter for soils with hydraulic conductivity greater than 1 x 10^{-3} cm/s. ASTM D5084 suggests using a flexible wall permeameter for soils with hydraulic conductivity smaller than 1 x 10^{-3} cm/s. It appears that at the cut-off of 1 x 10^{-3} cm/s, there is the potential to obtain unreliable hydraulic conductivity measurements, which had been observed by Terzaghi and Peck (1967). ASTM D5084 stipulates that for testing such soils (with hydraulic conductivity around 10^{-3} cm/s) the tubing sizes of the apparatus must be increased along with the porosity of the porous stones. Increasing the porous stone porosity may not be possible due to the potential for fines to migrate through pores of the stones. If an increase in porous stone is not made, a test of the overall system head loss is recommended to ensure that the porous stones are not significantly limiting the measured hydraulic conductivity of the specimen (ASTM D5084). For a constant rate of flow, the system must be able to maintain a flow to +/- 5% as well as be capable of measuring head loss to this accuracy. This is not easily performed without the use of transducers and flow-pumps (ASTM D5084). Note that in the study presented in this report the head loss test was performed and the conditions were determined to be satisfactory.

3.4.3 SHEAR STRENGTH PARAMETERS

One of the most commonly used methods of measuring the friction angle of soils is to perform a series of triaxial compression tests. Different types of tests exist for this method based on the soil type and loading/drainage conditions expected in the field. For granular material, the consolidated drained (CD) test is typically used to measure the shear strength parameters. The test condition represents soil that has been allowed to consolidate for sufficient time. During compression, the soil is then allowed to drain without developing any excess pore pressure.

Bowles (1982) stated that the results from the CD, CU and UU tests performed on granular free-draining material will produce very similar results; no data were found to verify this suggestion.

Table 3.7 shows typical values of drained internal friction angles for granular soils. The internal friction angle for sands and gravels ranges from 26 to 48 degrees.

Table 3.7: Friction Angles of Granular Soils (Lambe and Whitman, 1979)

Soil Type	Friction angle, ϕ (degrees)	
	Ultimate	Peak
Medium-dense silt	26–30	28–32
Dense silt	26–30	30–34
Medium-dense uniform fine to medium sand	26–30	30–34
Dense uniform fine to medium sand	26–30	32–36
Medium-dense well-graded sand	30–34	34–40
Dense well-graded sand	30–34	38–46
Medium-dense sand and gravel	32–36	36–42
Dense sand and gravel	32–36	40–48

After Lambe and Whitman (1979).

In

Table 3.6, the soil types defined by UFC (2005) have effective friction angle values from tests performed on samples compacted using the standard Proctor method at maximum dry density. For the GW and GP soils, the friction angle is greater than 38 and 37 degrees, respectively. Similarly, for SW and SP, the friction angle is 38 and 37 degrees, respectively.

It appears that the term cohesionless is used for describing a free-draining soil in the more current publications. The Unified Facilities Guide Specifications (UFGS 2008) describes cohesionless soils as those classified in the USCS as GW, GP, SW, and SP. Cohesive materials include materials classified as GC, SC, ML, CL, MH, and CH. Materials that are classified as GM, GP-GM, GW-GM, SW-SM, SP-SM, and SM shall be identified as cohesionless only when the fines are non-plastic.

NAVFAC DM 7-2 (1986) shows a relationship between friction angle and soil's index properties (unit weight and void ratio) for different soil types (Figure 3.7). Friction angle values are effective values obtained from tests that involved cohesionless soils with non-plastic fines. In general, the plot indicates that internal friction angle decreases as soil is more uniformly graded, less dense, and has higher fines content.

Thevanayagam (1998) performed consolidated undrained (CU) triaxial compression tests on sand samples of 10 cm diameter and 20 cm height with 0%, 10% and 25% fines to determine their strength. Results showed that the friction angle decreased as fines increased but not significantly. The difference between the average friction angle for soils with 0-2% Kaolin silts and 27% Kaolin silts was only about 3°.

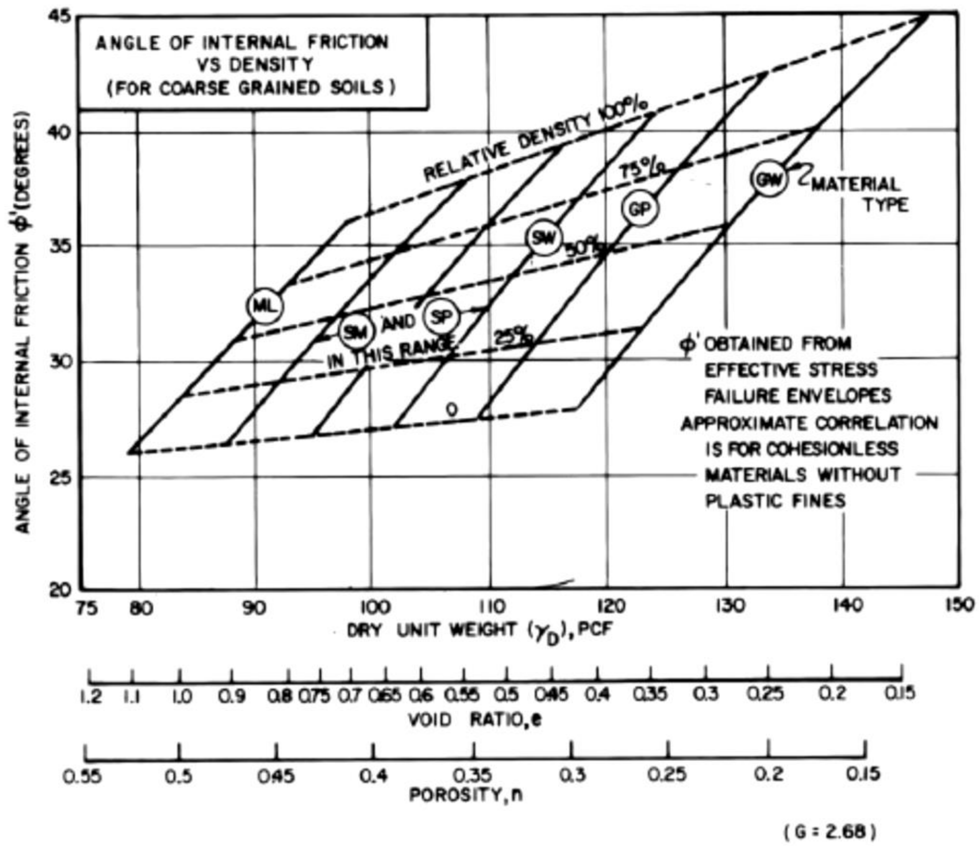


Figure 3.7: Friction Angle for Given Soil Type (NAVFAC DM7-2, 1986)

4. EFFECTS OF FINES ON HYDRAULIC CONDUCTIVITY AND SHEAR STRENGTH

4.1 SOILS, SAMPLE PREPARATION AND TESTING METHODS

The grain size distribution of the base granular soil used in this study (from the borrow source of the Bridggewater project) is shown in Figure 4.1. For reference, the VTrans current standard specification of structural backfill for abutments and retaining walls is also included. In addition, grain size distributions of the base soils used in other investigations found in the literature (Table 3.5) are also included.

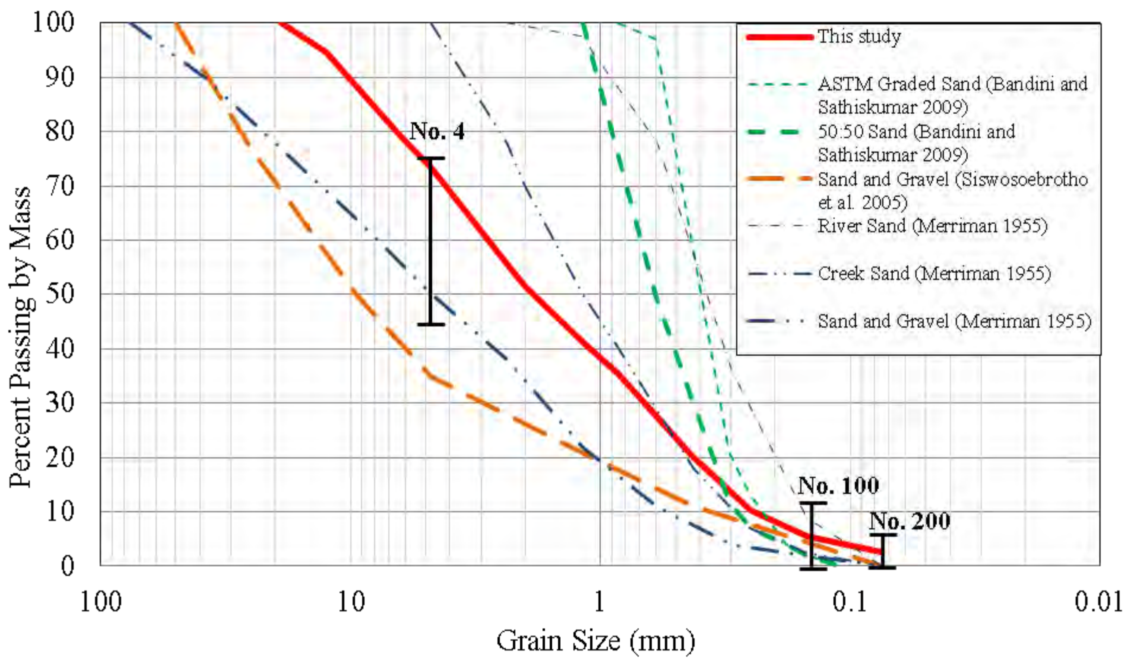


Figure 4.1: Grain size distribution of the base material of this study, compared to VTrans' specifications (vertical bars); base soils used in relevant published studies are also included.

An abundant source of fines was identified within the same quarry pit where the base material was obtained to allow systematic variation of the fines content from 0 to 25%. Hydrometer analysis (ASTM D422) and Atterberg limit (ASTM D4318) results indicated that the fines had similar properties to the natural fines in the base material and were non-plastic or had very low plasticity. Prior to testing, the base material was first sieved and washed to remove the natural fines and then incremented with the selected fines.

For the majority of the tests, the target specimen densities were 95% of maximum dry densities per standard Proctor test at optimum moisture content. The Standard Proctor density is most commonly specified, as per the survey (Table 2.1). Standard Proctor tests were performed on six combinations of the base soil and fines contents (0, 5, 10, 15, 20, and 25% by mass). For comparison purposes, modified Proctor tests were also performed. Specimen densities of 95% of maximum dry density per standard Proctor tests related to about 91% of maximum dry density per modified Proctor tests. All Proctor tests were performed in a 15.2 cm (6 in.) mold per standards ASTM D698 and D1557 because the soil contained particle sizes up to 19 mm (3/4 in.). An automated hammer mechanism was used. Table 4.1 summarizes maximum dry densities and optimum moisture contents obtained from the Proctor tests. In general, the maximum dry density increased and optimum moisture content decreased as the fines content increased. As expected, maximum dry densities and optimum moisture contents from modified Proctor tests were greater and smaller, respectively, than those from standard Proctor tests for a given fines content. A small number of tests were conducted at 90% relative compaction and optimum moisture content per standard Proctor test.

Table 4.1: Summary of standard and modified Proctor test results

Fines Content (% by mass)	AASHTO Soil Classification	USCS Soil Classification	Standard Proctor		Modified Proctor	
			Maximum Dry Density, g/cm^3 (pcf)	Optimum Moisture Content (%)	Maximum Dry Density, g/cm^3 (pcf)	Optimum Moisture Content (%)
0	A-1-b	SP	2.01 (126)	10.5	2.11 (132)	7.7
5	A-1-b	SP-SM	2.09 (130)	8.5	2.19 (137)	6.0
10	A-1-b	SP-SM	2.12 (132)	8.0	2.23 (139)	5.3
15	A-1-b	SM	2.18 (136)	7.3	2.26 (141)	5.3
20	A-1-b	SM	2.22 (139)	7.4	2.24 (140)	7.0
25	A-1-b	SM	2.17 (135)	7.9	2.21 (138)	6.0

Hydraulic conductivity and triaxial tests were performed using automated Geocomp Flowtrac II flowpumps, 4.4 kN (10 kip) load frame, and a triaxial cell that accommodated 15.2 cm (6 in.) diameter and about 30.5 cm (12 in.) high specimens. Flexible wall permeability tests were chosen to allow for back pressure saturation and to reduce potential side leakage which is common in rigid permeameters. The specimens were prepared using a split mold to allow the soil mixture to be compacted to the desired initial density. The dry mass of base material and silt was added together, mixed, and then water was added to optimum moisture. This mixture was then compacted in equal layers in the split mold by hand to a fixed height, to the target density. A membrane thickness of 0.635 mm (0.025 in.) was used to reduce the chance of puncture during compaction, and membrane correction was applied in data reduction. After sample preparation, the tubing and pumps were de-aired while the sample was allowed to saturate with deaired water under a low gradient prior to being connected to flow pumps. Back pressure saturation was performed to verify the B parameter of 0.95 or greater using the automated flow pumps.

Hydraulic conductivity tests were performed using constant flow. No loss of fines was observed during or after testing and the porous stones were inspected and cleaned prior to each test.

A head loss check was performed on the system prior to testing as suggested by ASTM D5084. This test was performed using a hollow Plexiglas cylinder, in place of the soil sample, with a membrane to observe the effect of the tubing and porous stones without any soil. This check showed that the porous stones and losses in other components of the apparatus had little effect on the measured hydraulic conductivities of the soil specimens.

For each combination of the base soil and fines content, three triaxial specimens were prepared. Tests were performed using ASTM D7181 for consolidated drained triaxial compression (CD) tests. Hydraulic conductivity was measured at 41 kPa (6 psi) confining pressure followed by a CD test for the first specimen. For the second specimen, hydraulic conductivity was measured at a confining pressure of 83 kPa (12 psi) after the 41 kPa (6 psi) measurement and then a CD test was performed. Hydraulic conductivities were measured at 41, 83, and 124 kPa (6, 12, and 18 psi) confining pressures followed by a CD test for the third specimen. Occasionally, an additional specimen was prepared for repeating a test. All CD tests were conducted at a shearing rate of 0.01%/min.

4.2 EXPERIMENTAL RESULTS

The hydraulic conductivity measurements are plotted in Figure 4.2 through Figure 4.5. Hydraulic conductivities were measured on three specimens at 41 kPa (6 psi), on two specimens at 83 kPa (12 psi), and once at 124 kPa (18 psi), and were fairly repeatable. As expected, hydraulic conductivity decreased with increasing confining pressure as seen in Figure 4.2. As seen in Figure 4.3, hydraulic conductivities of 0 - 10% fines contents were close to each other, and greater than 10^{-4} cm/s (0.14 in/hr) for up to 15% fines content. There is a distinct drop in hydraulic conductivity between 10 and 15% fines content and continued to decrease with higher fines content. Hydraulic conductivity of specimens compacted at 90% relative compaction was generally higher than that of specimens compacted at 95% relative compaction, as expected.

This investigation included only one base soil. To evaluate if the above conclusions could be generalized to other granular soils, the results obtained here are combined in Measured hydraulic conductivity versus fines content (the dashed lines are based on judgment and not statistically determined)

Figure 4.6 with other relevant results found in the literature (Merriman, 1955; Siswosoebrotho et al., 2005; Bandini and Sathiskumar, 2009). The gradations of the soils used by these investigators are included in Figure 4.1. Other specifics of their test conditions are summarized in Table 3.5. The base soil types and test conditions in the investigations varied significantly. For example, the base soils included fine sand to mostly gravelly soils. Both rigid and flexible wall permeameters were used and the techniques of measuring hydraulic conductivity also varied (constant head, falling head and constant flow). The confining pressures also differed. Despite these differences, the estimated best fit of the hydraulic conductivities summarized in Measured hydraulic conductivity versus fines content (the dashed lines are based on judgment and not statistically determined)

Figure 4.6 are consistent with this study in that the hydraulic conductivity of the granular base soils did not change appreciably for non-plastic fines content of up to 10%.

As described earlier, a consolidated drained strength test was conducted on each specimen after the hydraulic conductivities were measured. The effective internal friction angle and effective cohesion values for peak and ultimate failure conditions are summarized in Table 4.2. The internal friction angle decreased with an increase in fines content. The small effective cohesions are due to a slight decrease that occurred in peak friction angle with increased confinement and applying a straight-line fit to each of the corresponding failure circles. The peak internal friction angle decreased from about 39^{o} for zero percent fines to about 33^{o} for 25% fines. For the same fines content range, the ultimate friction angle decreased from about 35^{o} to 32^{o}. The data from triaxial compression tests are included in Appendix B.

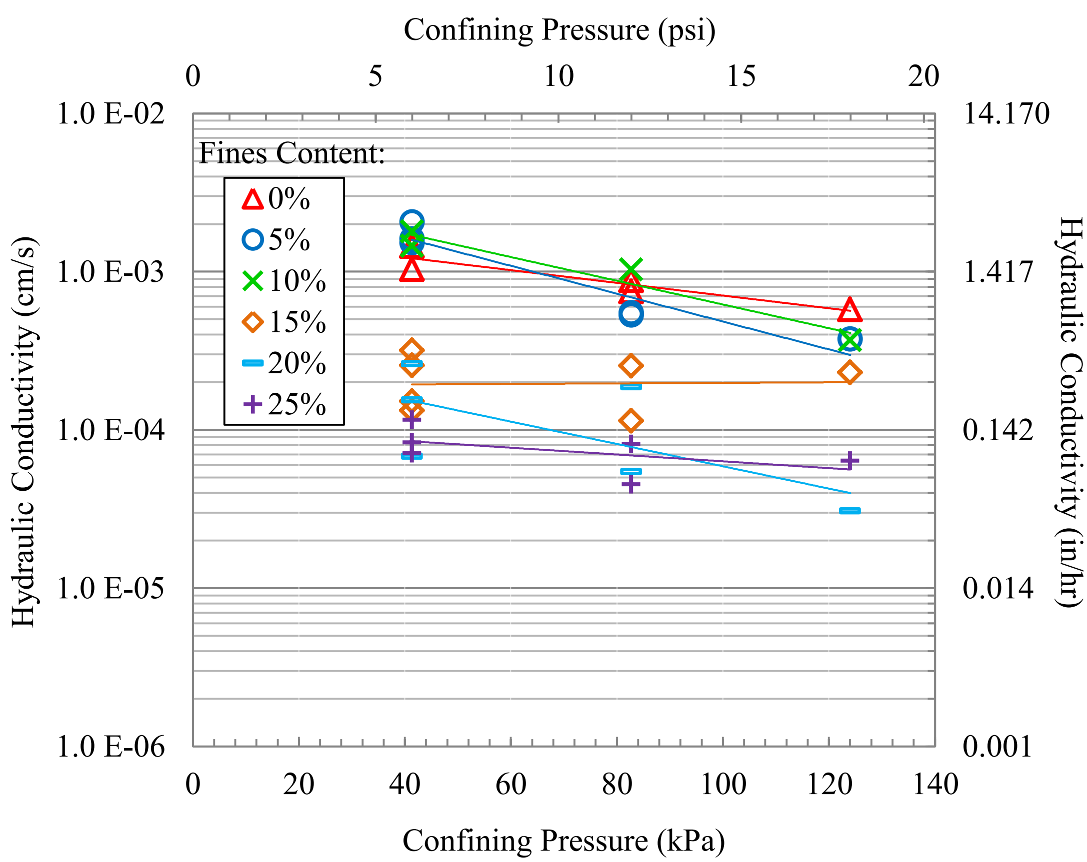


Figure 4.2: Measured hydraulic conductivity versus confining pressure for 95% RC

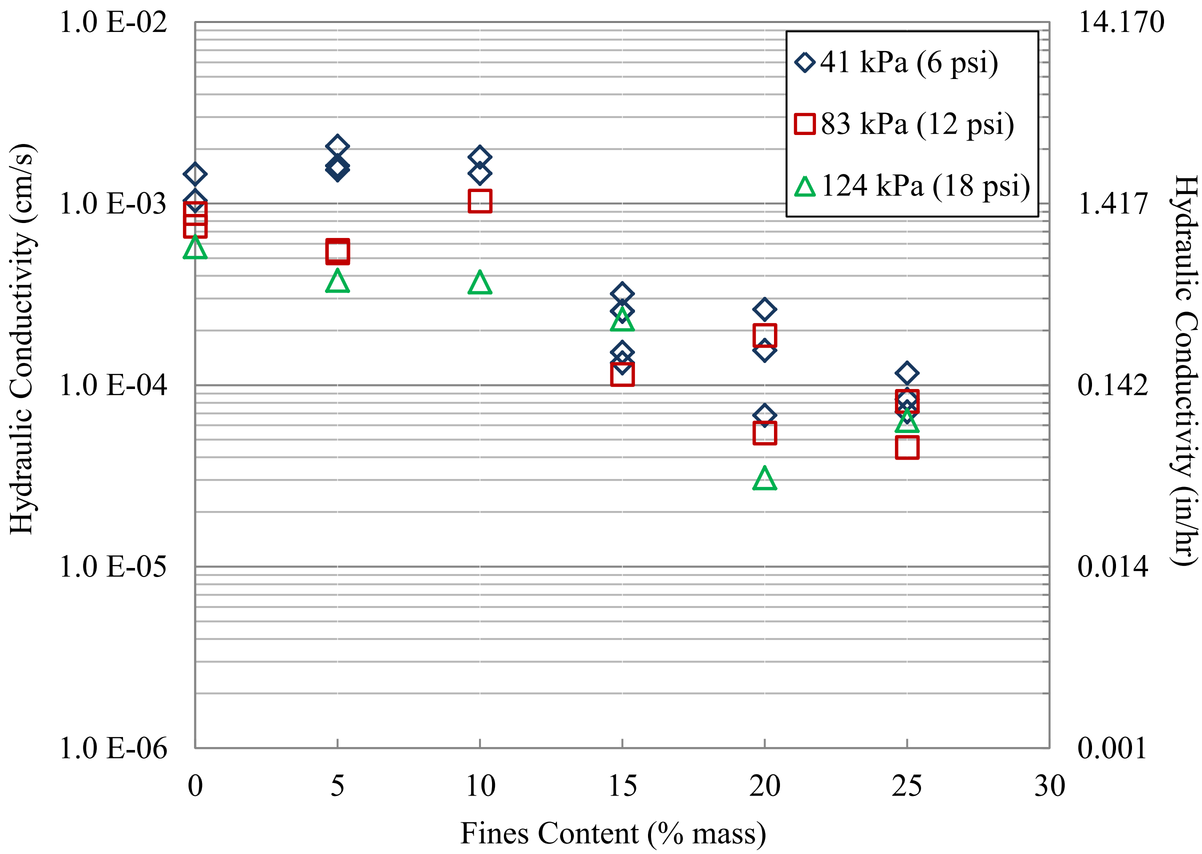


Figure 4.3: Effect of Fines on hydraulic conductivity at varied confinement for 95% RC

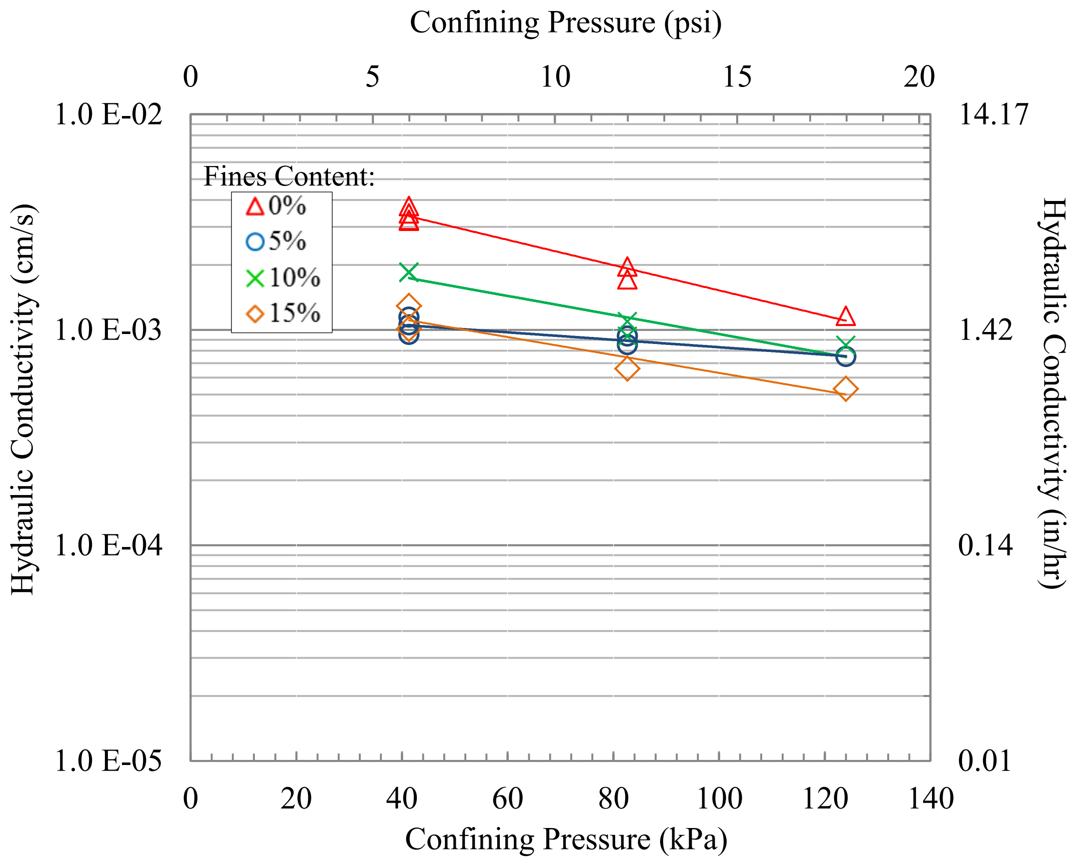


Figure 4.4: Measured hydraulic conductivity versus confining pressure for 90% RC

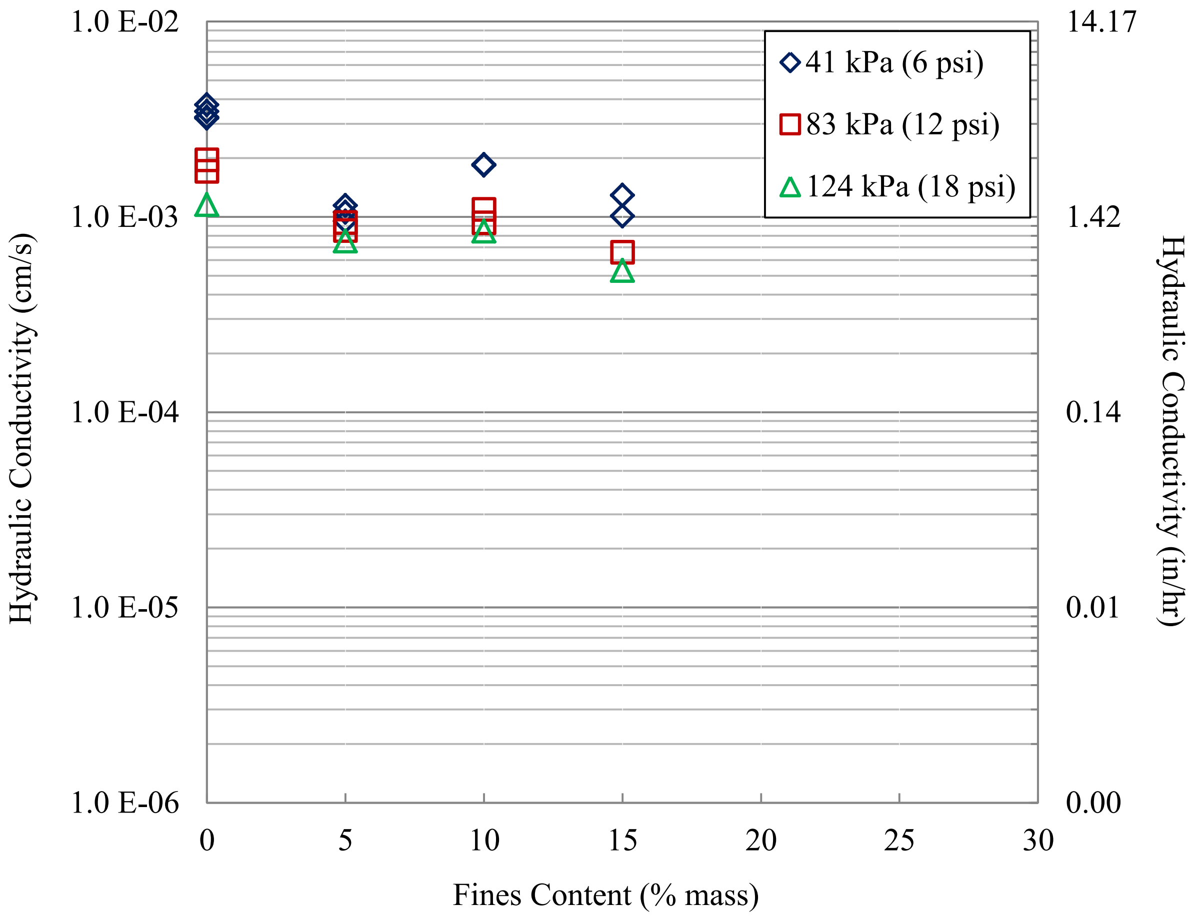
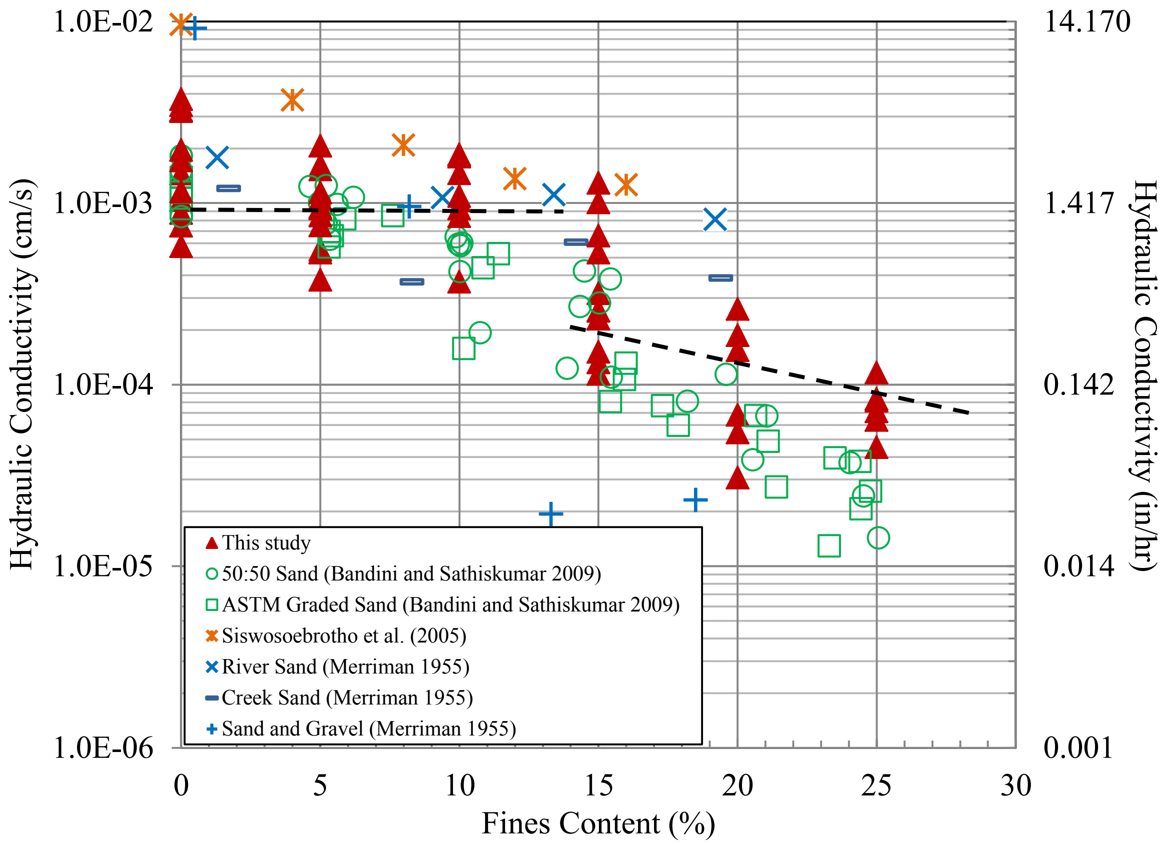


Figure 4.5: Effect of Fines on hydraulic conductivity at varied confinement for 90% RC



Measured hydraulic conductivity versus fines content (the dashed lines are based on judgment and not statistically determined)

Figure 4.6: Hydraulic conductivities from this study compared to other relevant studies

Table 4.2: Summary of drained shear strength parameters

Fines Content (%)	Peak Strength		Ultimate Strength	
	Cohesion, kPa (psi)	Friction Angle, degrees	Cohesion, kPa (psi)	Friction Angle, degrees
0	12.4 (1.78)	38.9	2.3 (0.34)	34.5
5	11.9 (1.72)	39.7	0.0 (0.00)	35.3
10	8.2 (1.19)	36.4	0.8 (0.12)	34.2
15	19.1 (2.77)	34.1	0.4 (0.06)	35.5
20	6.2 (0.90)	33.4	4.1 (0.60)	32.2
25	5.2 (0.75)	33.1	4.3 (0.62)	31.7

4.3 SUMMARY OF THE EXPERIMENTAL RESULTS

For the soils investigated, the measured hydraulic conductivities for 0%, 5%, and 10%, fines contents were quite close to each other. Hydraulic conductivities were significantly lower for fines content in excess of 15%. These results compared well with other relevant studies found in the literature that included varying granular soil types (fine sand to mostly gravel) and test conditions. A non-plastic fines content of up to about 10% for free-draining structural backfill is well supported by this study and data reported in published work by others.

5. FIELD MONITORING

5.1 MOTIVATION

Typically, the design of retaining walls and bridge abutments relies on the assumption that the soil material used to backfill the structure is ‘free-draining’ and will not produce differential hydrostatic pressure. If the backfill is not expected to be drained, the abutment or retaining wall must be designed for earth pressure loads plus hydrostatic pressure due to the presence of water. However, there is insufficient readily accessible information regarding what constitutes free-draining for purposes of preventing hydrostatic pressure against walls and abutments. Differential water pressures could be of concern particularly for retaining structures near water bodies such as bridge abutments over rivers and streams. To assess if any differential water pressures exist in existing cast-in-place reinforced concrete retaining walls installed by VTrans, a field monitoring program was implemented, which is described below along with the measurements and analysis.

5.2 MONITORING SITES AND INSTRUMENTATION

Two sites – Bridgewater and Williamstown, both in Vermont, were selected for installing field instrumentation by VTrans. The locations of these bridges are depicted in the map shown in

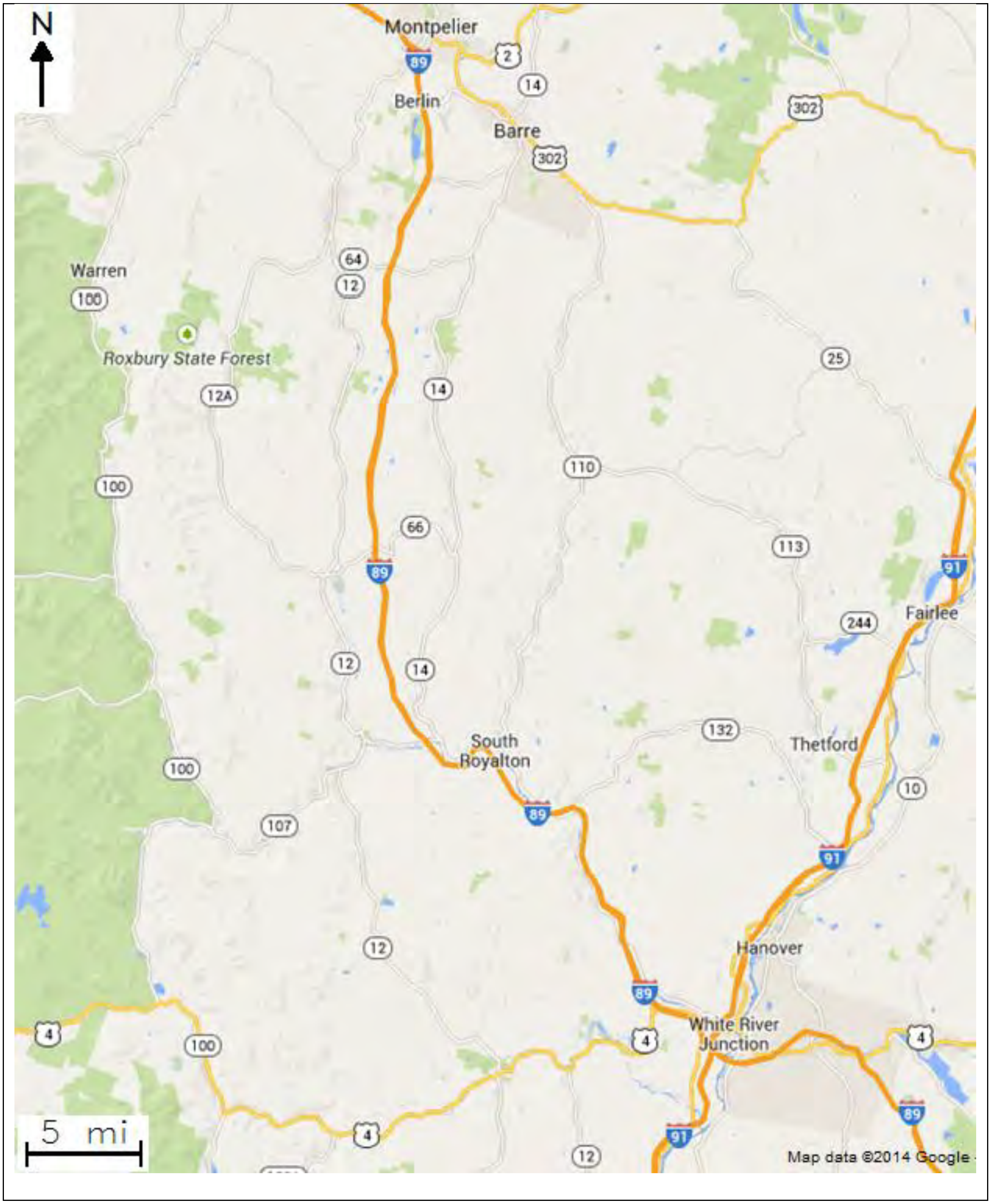

N
Montpelier
Berlin
Barre
Warren
Roxbury State Forest
South Royalton
Fairlee
Thetford
Hanover
White River Junction
5 mi
Map data ©2014 Google

Figure 5.1. The Bridgewater abutment was new construction in 2012 whereas Williamstown site is an existing bridge (built in 2010).

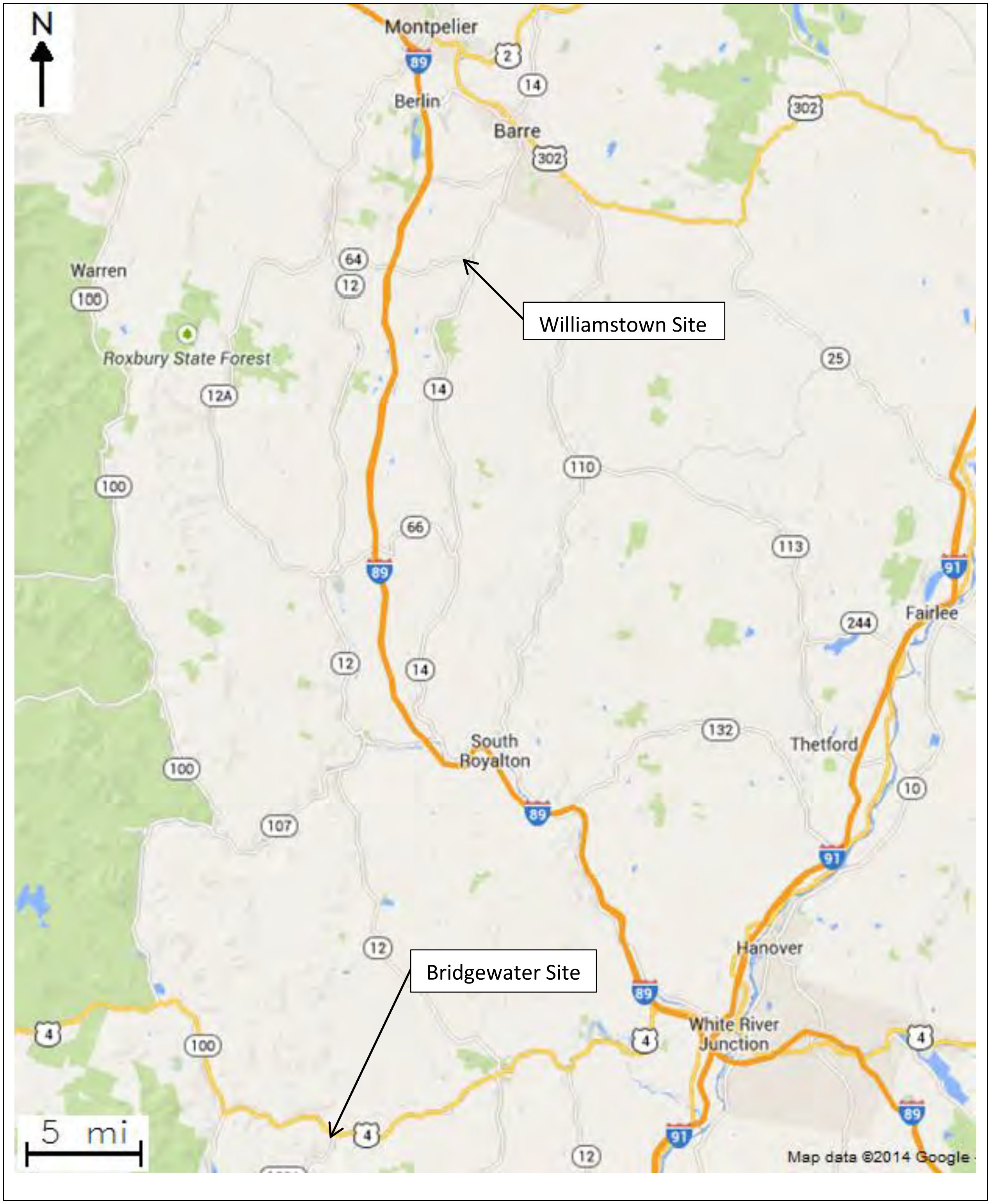


Figure 5.1: Location map of Bridgewater and Williamstown

At each site, three piezometers were installed in the backfill and an additional pressure transducer was installed to measure stream water level elevation. Figure 5.2 shows a photograph during the installations of piezometers at Bridgewater and Figure 5.3 shows a plan view of the instrumentation.

Figure 5.2: Bridgewater piezometer installation

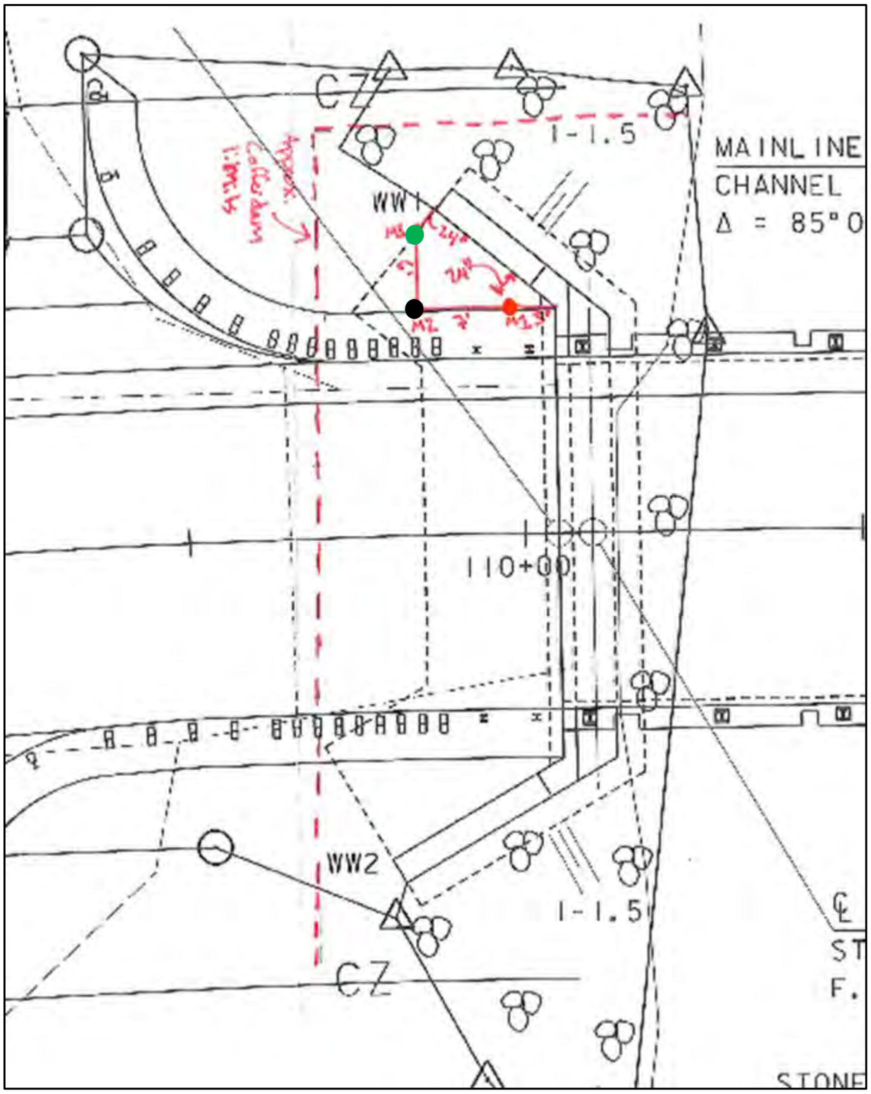


Figure 5.3: Plan view of instrumentation at Bridgewater site

Figure 5.4 shows a photograph during the installations of piezoemeters at Williamstown and Figure 5.5 shows a plan view of the instrumentation.

Figure 5.4: Williamstown piezometer installations

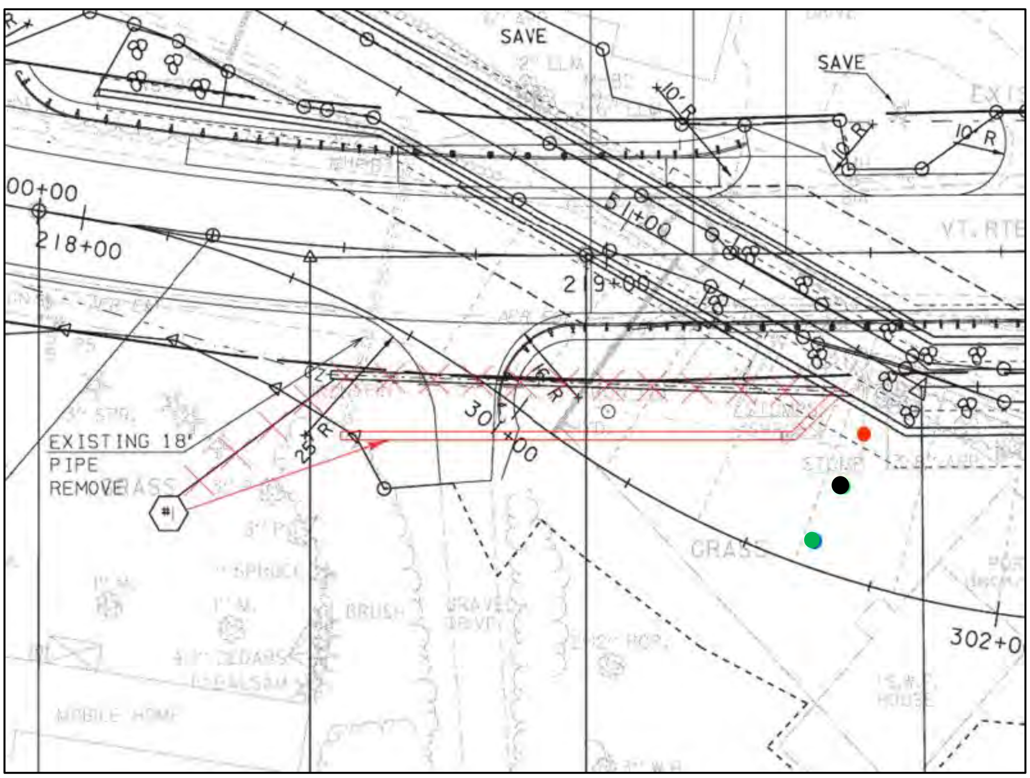


Figure 5.5: Plan view of the instrumentation at the Williamstown site

5.3 MEASUREMENTS AND ANALYSIS

Monitoring periods were from January 2012 through July 2014. Figure 5.6 and Figure 5.7 show groundwater and river water level data from Bridgewater site. Figure 5.6 summarizes water levels in terms of elevations. Figure 5.7 shows differential water levels; the water level of the river is subtracted from the water level in the backfill. A positive number would indicate that the water level in the backfill was higher than that in the stream. Figure 5.8 shows a close-up of the data from Figure 5.6 for a shorter time period to assess if there is any time lag between the water level changes in the stream versus those in the backfill.

Figure 5.9, Figure 5.10 and Figure 5.11 present data in formats similar to Figure 5.6, Figure 5.7 and Figure 5.8, but for Williamstown site.

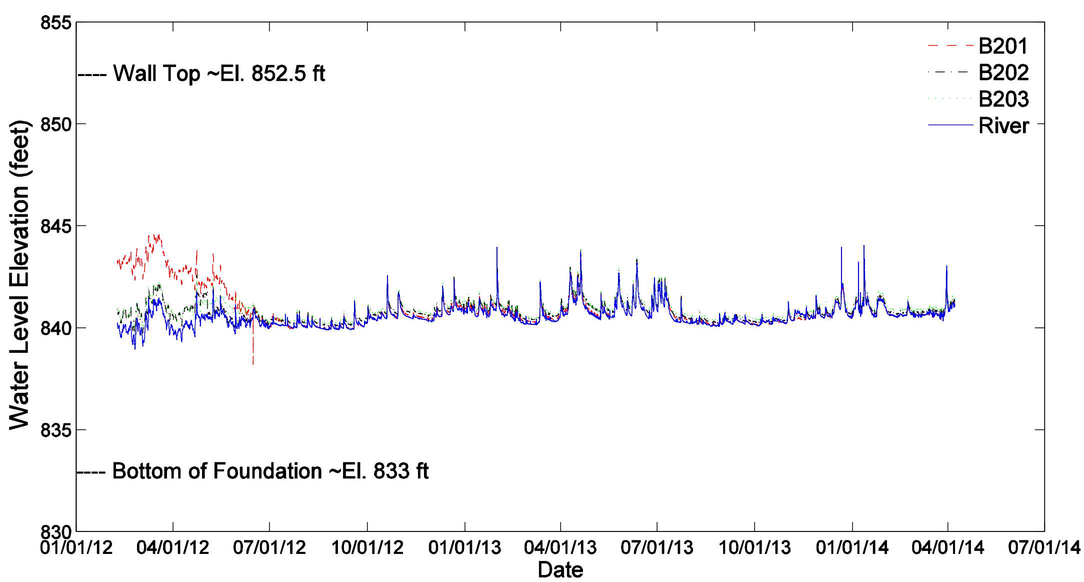


Figure 5.6: Measured water levels at Bridgewater site

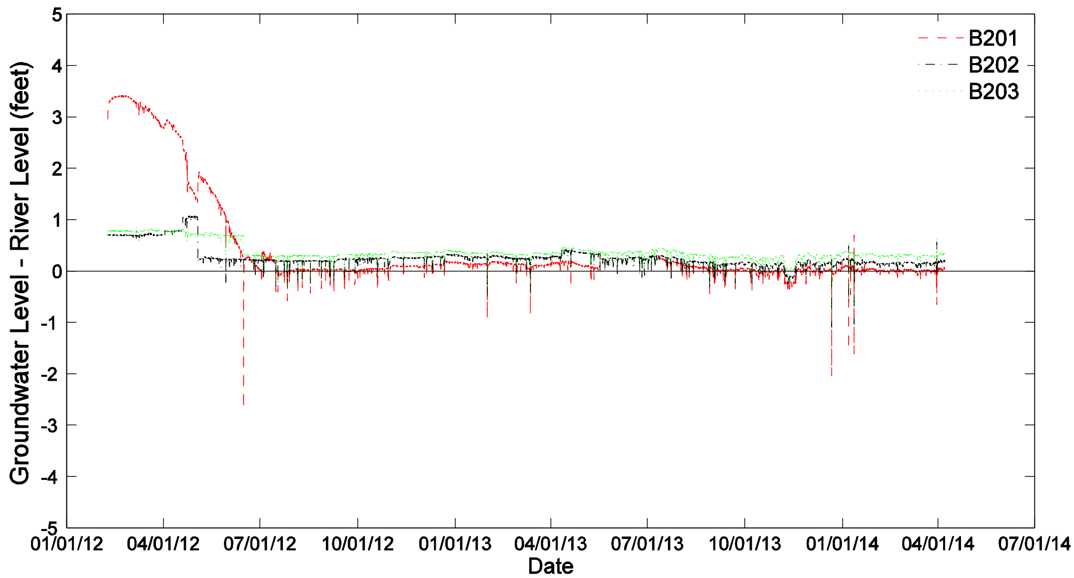


Figure 5.7: Measured differential water levels at Bridgewater site

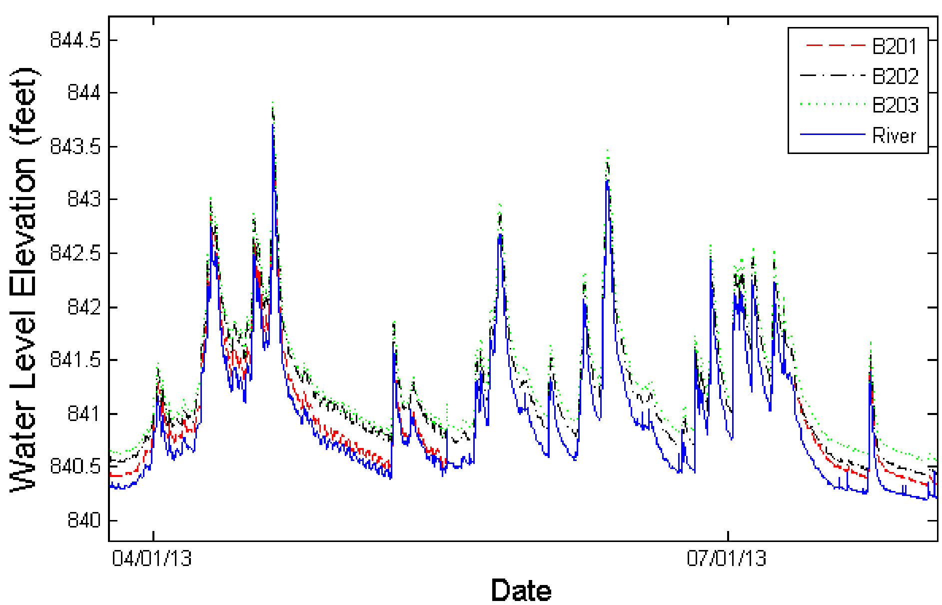


Figure 5.8: Measured water levels at Bridgewater site (close-up of a portion from Figure 5.6)

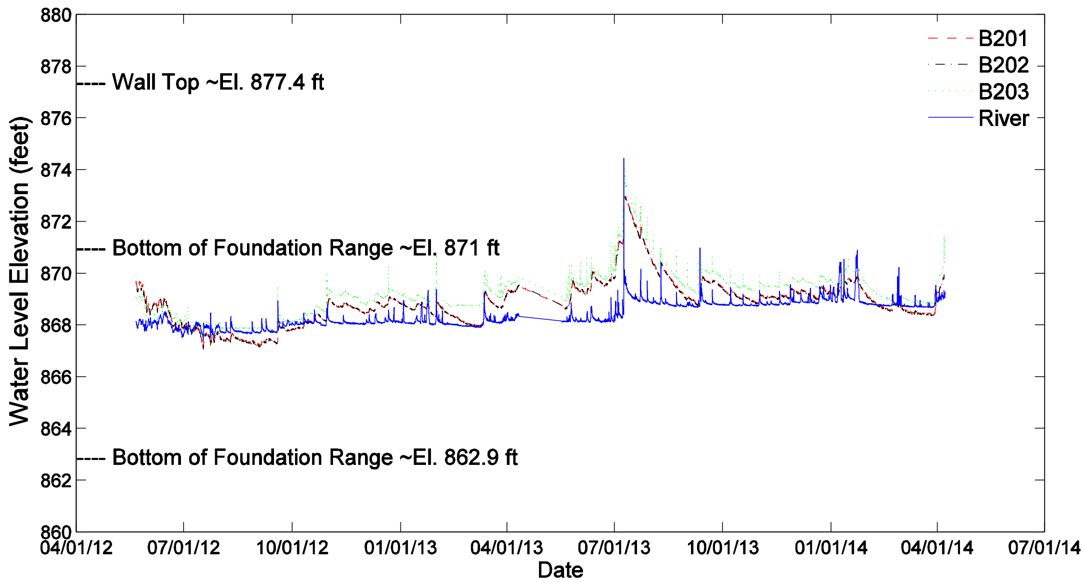


Figure 5.9: Measured water levels at Williamstown site

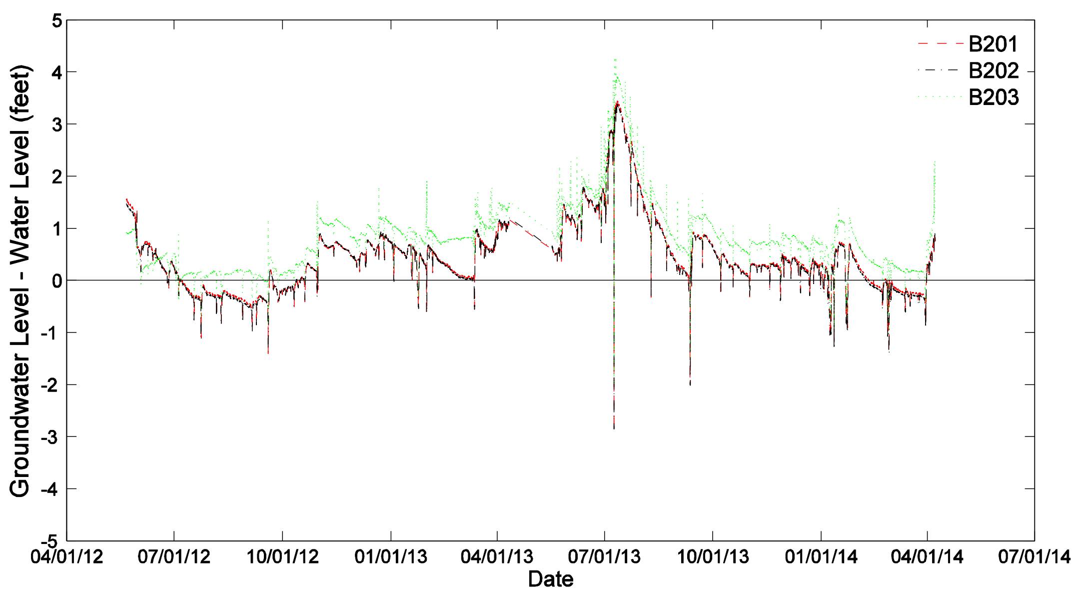


Figure 5.10: Measured differential water levels at Williamstown site

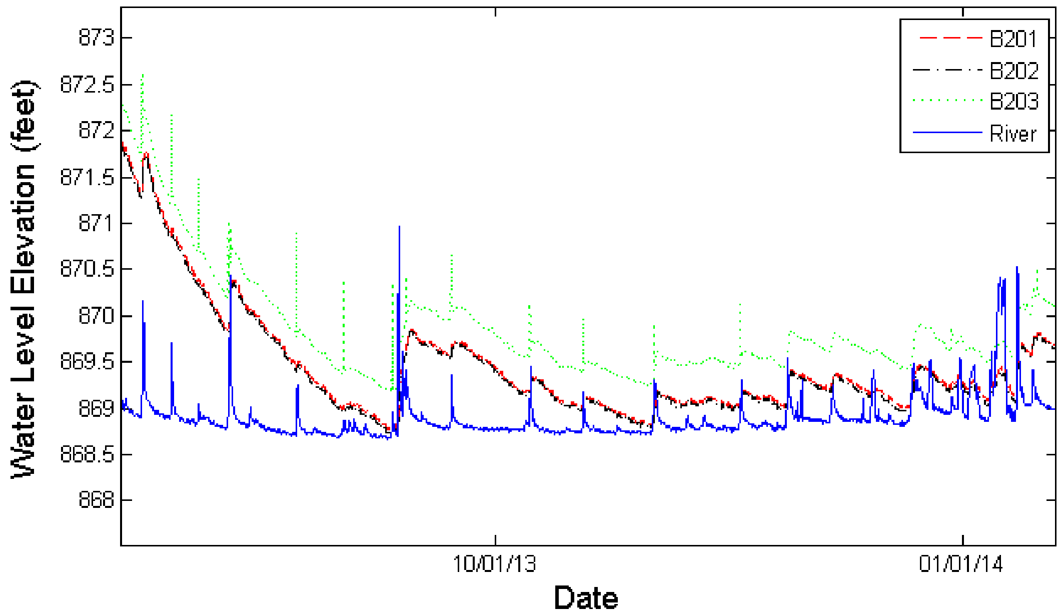


Figure 5.11: Measured water levels at Williamstown site (close-up of a portion from Figure 5.9)

5.3.1 PRECONSTRUCION WATER LEVELS

Bridgewater

Preconstruction ground water level data from three borings (B-101, B-102, and B-201) drilled for the project between 2005 and 2008 at the south abutment (abutment no. 1) were limited to observations during drilling. The water level in boring B-201 taken during drilling (casing extent was not reported) was approximately at the river bed level shown on the plans. The river is normally shallow in this stretch correlating with groundwater matching river level. Caving was reported at boreholes B-101 and B-102 at about 8 and 3 feet higher, respectively, without specific groundwater level data. Overall there is no direct data on prior water levels aside from these reports. Soils reported on the logs between ground level and below proposed Bottom of Footing elevation are granular with between 10 and 22 percent fines, indicating a relatively high conductivity in the stream bank and foundation soils.

Williamstown

Preconstruction ground water level data near the new observation wells is limited to observations during drilling at two borings (B-2 and B-4). No ground water table was encountered at boring B-2. At boring B-4, the ground water table was reported at 14.1 feet deep. Based on this data it appears that ground water was near or below top of rock in the vicinity of the new monitoring wells, MW-1 through MW-3. Backfill permeability is unclear based on the limited amount of sampling obtained and several cored boulders at boring B-4 could be interpreted as indicative either of low permeability glacial till or more pervious conditions. The foundations are shown to be placed directly on top of the bedrock.

5.3.2 POST-CONSTRUCTION WATER LEVELS

Bridgewater

Aside from what appear to be anomalous levels measured at observation well B-201 between January and July 2012, the ground water levels measured in the three observation wells nearly match the river levels. As illustrated in Figure 5.7, the backfill water levels essentially match the river levels during river rise. There is at most a six inch lag in the backfill ground water levels during river fall.

Williamstown

Figure 5.10 illustrates that groundwater levels in the observation wells are between 12 and 18 inches above the river level during relatively consistent river level conditions. During periods of rapid river level fluctuations the groundwater levels have risen as much as 3 to 4 feet above river level. The groundwater levels in observation wells B-201 and B-202 approximately match the amount of the river level rise and lag behind in their rate of rise. The groundwater levels in observation well B-203 usually match the rate and often exceed the amount of the river level rise. It is puzzling that B-203 levels are higher and more erratic than at the other observation wells.

The river and groundwater levels during the July 9, 2013 flooding in Williamstown (Refer to Figure 5.9 for water level data) provide helpful data in comparing river and groundwater levels. Groundwater levels rose essentially as quickly as the river level although they lagged lower than the total river level rise. Following that short burst of flooding the ground water levels gradually dropped over the course of about 2 months while river levels dropped in a matter of about a week. Aside from the unusual July 2013 flooding period, the groundwater levels in all three observation wells have usually been about 12 inches and rarely more than 18 inches above the adjacent river level during the approximately 2 year monitoring record.

5.4 SUMMARY

Bridgewater

Groundwater levels in Bridgewater essentially match the river levels with a maximum difference of about 6 inches in the backfill groundwater above the river during falling river levels. This appears to be the result of the abutment bearing on and being surrounded by relatively free draining natural soils and adequate drainage characteristics in the abutment backfill and weep holes within the abutment itself.

Williamstown

Groundwater levels in Williamstown have mostly been within 1 foot above the river level with the exception of the July 9, 2013 flooding. Observation well B-203 water levels are higher and more erratic than expected for this well as compared to the other two wells which have more stable and typical groundwater level fluctuations. Its response seems more likely of a well closer to the river and placed beside a weep hole.

The groundwater flow regime in the backfill of this project's unusually long abutment and wing wall configuration is unclear. The extent to which surface water can infiltrate the wall backfill is important. The groundwater monitoring data indicates that the water level in the backfill is usually between 1 and 3 feet above the river. The extent to which this differential reflects typical retaining wall conditions versus a groundwater and surface water regime unique to this project configuration is unknown. More data on the construction details described above, perhaps by means of project photographs, will be helpful. In the end, it will most likely take comparison of Williamstown's data with that from other projects to decide if and how to generalize this project's findings to other backfill situations.

6. OVERALL CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

6.1 MAIN CONCLUSIONS

This report presented:

(1) A laboratory study investigating effects of fines on the hydraulic conductivity and shear strength of a typical granular backfill, and

(2) Results of field monitoring of river level and groundwater level in the backfill at two cast in place abutment wall sites in Vermont. The following conclusions are drawn:

For the soils investigated, the measured hydraulic conductivities for 0%, 5%, and 10%, fines contents were quite close to each other. Hydraulic conductivities were significantly lower for fines content in excess of 15%. These results compared well with other relevant studies found in the literature that included varying granular soil types (fine sand to mostly gravel) and test conditions. A non-plastic fines content of up to about 10% for free-draining structural backfill is well supported by this study and data reported in published work by others. On an empirical basis, the survey of state transportation agencies indicates that some states are using fines contents from 5 to 15% (with one state at 20%) without reports of adverse effects.

How much, if any, of a fines content above 10% can be justified is less clear. Beyond about 10% there is both greater variability and decreasing permeability such that the free draining designation is perhaps not justifiable for a broadly applied specification for structural wall backfill. In specific situations which warrant investing in the combination of additional testing, design effort, and attention to controlling material variability in construction, the permeability data reported here indicates that the fines content could potentially be increased a small amount if the specific soils have favorable hydraulic conductivity. However, attention would need to be paid to the potential for greater frost susceptibility and material variability which would result in added quality control costs for the contractor and quality assurance expenses for the owner.

The effect of fines on drained shear strength showed a decrease in the effective internal friction angle with increased fines. Values of peak and ultimate friction angles varied between 39 and 33 degrees and between about 34.5 and 31.5 degrees, respectively, for fines content between 0 - 25%. If the fines content of up to 10% was allowed, the peak internal friction angle may decrease from about 39° for zero percent fines to about 34.5° for 10% fines. For the same fines content range (0 to 10%), the ultimate friction angle may decrease from about 35 to 34 degrees. To generalize this conclusion however, shear strength tests should be conducted on additional soils.

It is important to note that the objective of considering a backfill soil specification with higher fines content than presently used was for cost savings. The abutment and retaining walls under consideration are considered to be primarily in river crossing settings and wall performance with the current backfill and bottom of wall weep hole configuration in Vermont has been satisfactory.

The Vermont Agency of Transportation (VTrans) evaluated borrow source availability in 1993 (Conrad and Dudley, 1993) finding that available sources were being depleted and that 94.5% of Vermont's remaining deposits are not available for extraction due to inaccessibility, conflicting land use, environmental sensitivity and poor quality. Personal communications with contractors in Vermont indicate that up to 20 percent savings in the unit cost price of the retaining wall backfill item could be achieved by allowing a fines content by weight increase from the current Vermont standard specified maximum of 6% to 10%. If the allowable fines content is increased from 6% to 10%, it is recommended that it is adopted on a trial basis at three or so sites and these walls be monitored for differential water levels.

The effects of initially unsaturated conditions or aging and other characteristics of compacted soils on their permeability and strength and how these characteristics change with fines content were not specifically evaluated in the testing for this and the referenced studies, which could be a topic for future investigations.

The field monitoring at Bridgewater site showed that groundwater levels in the backfill essentially matched the river levels with a maximum difference of about 6 inches in the backfill groundwater above the river during falling river levels. This appears to be the result of the abutment bearing on and being surrounded by relatively free draining natural soils and adequate drainage characteristics in the abutment backfill and weep holes within the abutment itself.

Groundwater levels in Williamstown were within 1 foot above the river level with the exception of the July 9, 2013 flooding. Observation well B-203 water levels were higher and more erratic than expected for this well as compared to the other two wells which have more stable and typical groundwater level fluctuations. Its response seems more likely of a well closer to the river and placed beside a weep hole. The groundwater flow regime in the backfill of this project's unusually long abutment and wing wall configuration is unclear. The extent to which surface water can infiltrate the wall backfill is important. The groundwater monitoring data indicates that the water level in the backfill was usually between 1 and 3 feet above the river. The extent to which this differential reflects typical retaining wall conditions versus a groundwater and surface water regime unique to this project configuration is unknown. More data on the construction details described above, perhaps by means of project photographs, will be helpful. In the end, it will most likely take comparison of Williamstown's data with that from other projects to decide if and how to generalize this project's findings to other backfill situations.

6.2 RECOMMENDATIONS FOR FUTURE RESEARCH

Additional research into the following areas is recommended. For the experimental work, we recommend the following.

(1) Additional granular backfill soils should be tested to study the effects of gradation properties in addition to the effects of fines content on compaction, hydraulic conductivity and shear strength.

(2) In this investigation, grain size distributions of only the base soil were determined before and after compaction to examine if additional fine grained partciles generate because of compaction. In general, less than 1% fines were generated. In future, if additional backfill soils are investigated, the effects of compaction on soil gradation could be further investigated.

(3) Constant and falling head hydraulic conductivity tests performed in a rigid compaction permeameter could be performed for comparison to the flexible wall permeameter. If the results compare well, the tests could be conducted much faster.

(4) The effects of strain rate on shear strength could be investigated. In this research it was assumed that the effect of strain rate on a free-draining soil is negligible.

For field monitoring, we recommend supplementing the Bridgewater and Williamstown monitoring with additional project monitoring in order to more reliably conclude how well the currently specified wall backfill and weep-hole details limit the groundwater differential at abutments. Bridgewater appears to represent the case of an abutment in relatively free draining granular soils behind and below the wall, and with relatively limited seepage into the backfill from natural ground behind it. Williamstown represents the case of a retaining wall bearing on bedrock with a currently undetermined amount of seepage from behind, and from surface infiltration.

Specifically, we recommend monitoring on projects where preconstruction groundwater data is available for comparison with post construction monitoring data. We recommend the following monitoring cases:

(1) Abutment bearing on low permeability soil (e.g., glacial till or clay) with natural granular soil above, with groundwater levels well documented in natural soils behind the new backfill/abutment.

(2) Abutment bearing on bedrock with groundwater levels well documented in natural soils behind the new backfill/abutment (Williamstown case but with shorter, more typical, wing walls.

ACKNOWLEDGEMENTS

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APPENDIX A – STATE SURVEY

The Vermont Agency of Transportation is evaluating the current AASHTO-recommended design criteria (granular backfill with less than 5% fines) and specifications for backfill in association with abutments and retaining walls. These design assumptions presume a free-draining structure that results in no differential groundwater levels on opposite sides of the retaining wall. Consequently, unbalanced loading due to hydrostatic pressure is ignored. However, some engineers are concerned that unbalance loading may be occurring. Furthermore, in some areas the availability of high-quality structural backfill has been declining. The purpose of this research initiative is to examine these relationships along with other possible cost effective backfill alternatives and establish design guidelines for our organization.

As part of this research projects, we are surveying other state DOTs to determine what types of backfill materials and drainage details are currently being used on cast-in-place concrete cantilevered retaining wall and bridge abutments.

The survey is expected to 2 to 5 minutes to complete. Please make sure to include your contact information. Surveys results will be shared with everyone that completes the survey. We thank you for your time.

Page 1 - Question 1 - Choice - Multiple Answers (Bullets)

How do you account for hydrostatic pressure in your design assumptions?

- ❑ Ignore it.
- ❑ Design for it.
- ❑ Install a drainage system in order to not design for it.
- ❑ None of the above.

Page 1 - Question 2 - Yes or No

Do you utilize backfill material with greater than 5% fines?

- ❍ Yes
- ❍ No

Page 1 - Question 3 - Yes or No

Has your organization done formal studies to investigate if greater fines contents could be used or if alternative materials could be used/added?

- ❍ Yes
- ❍ No

Page 1 - Question 4 - Choice - Multiple Answers (Bullets)

Please check all applicable backfill materials your DOT uses or would consider using in the future:

- ❑ Shredded tires
- ❑ Geofoam Blocks
- ❑ Recycled concrete

- ❑ Recycled pavement
- ❑ Granular backfill
- ❑ In-situ soils
- ❑ Other

Page 1 - Question 5 - Yes or No

Do you have standard details for abutment and wingwall backfill?

- ❍ Yes
- ❍ No

Page 1 - Question 6 - Yes or No

Do you have standard specifications for abutment and wingwall backfill methods and materials?

- ❍ Yes
- ❍ No

Page 1 - Question 7 - Yes or No

Have you changed your details in the past to provide a more cost-effective backfill detail, or do you currently vary your details on a project by project basis based on cost?

- ❍ Yes
- ❍ No

Page 1 - Question 8 - Yes or No

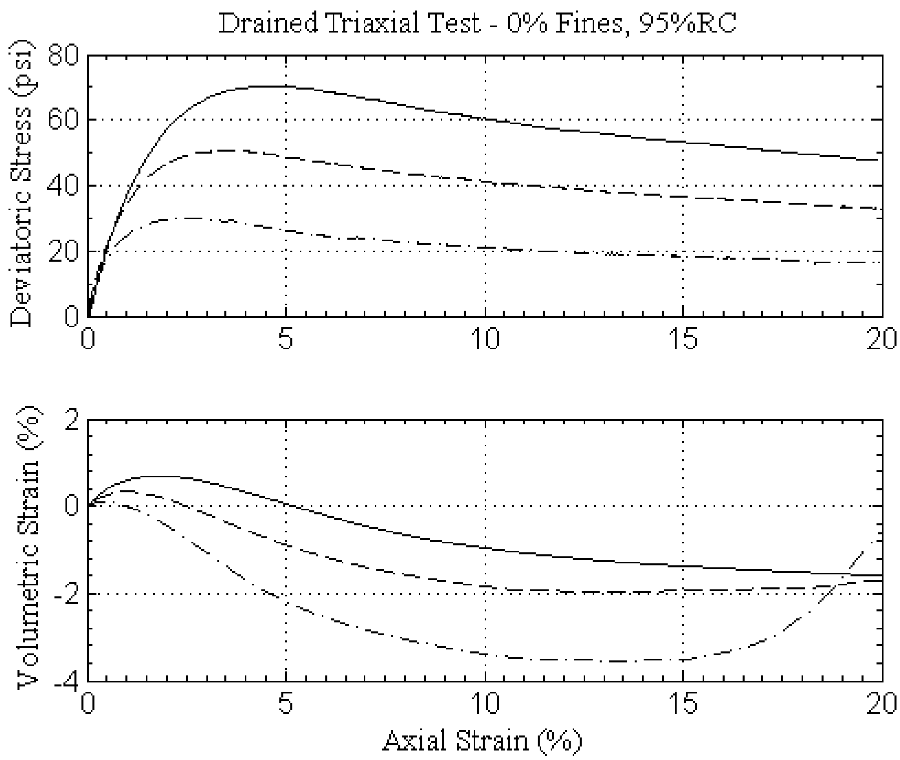
Do you vary your design and details for backfill based on other non-geotechnical parameters, such as the average daily traffic (ADT)?

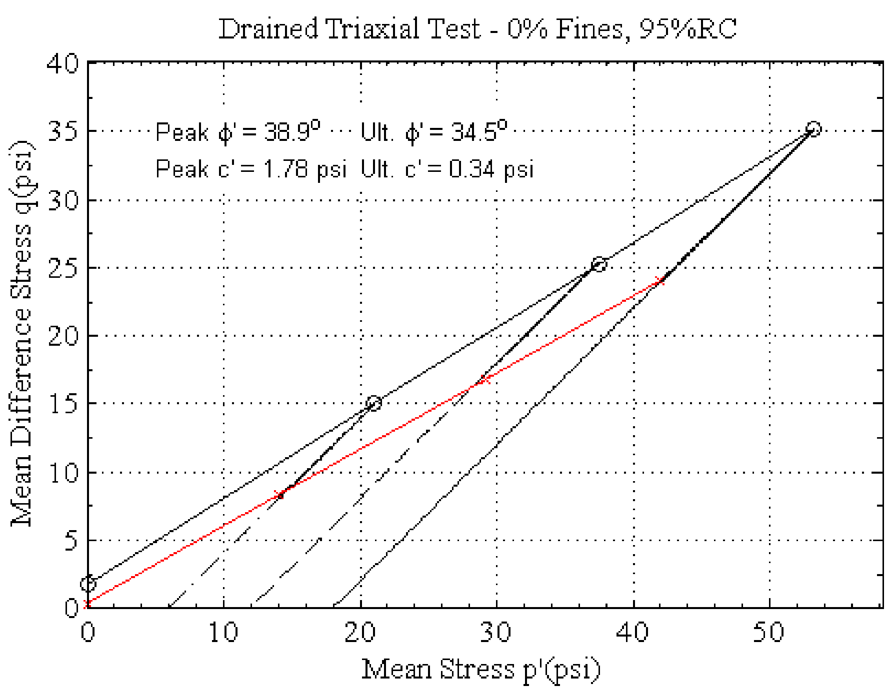
- ❍ Yes
- ❍ No

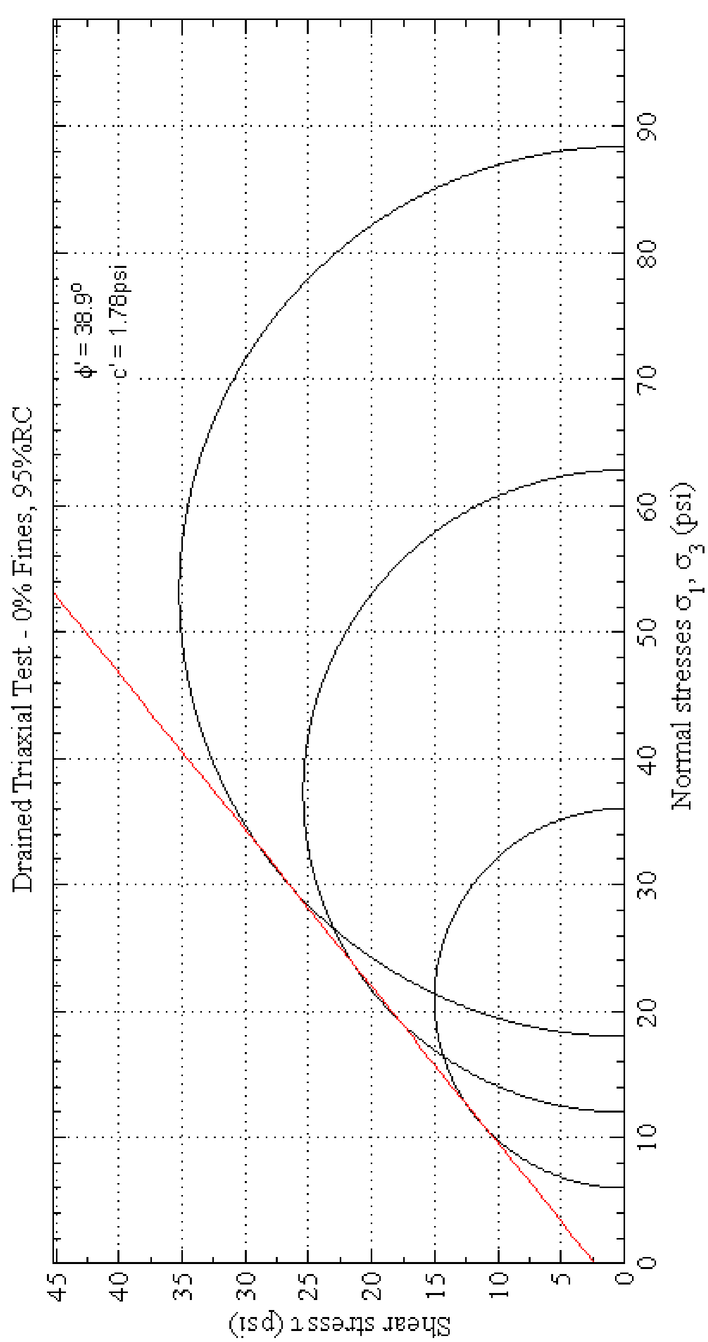
Page 1 - Question 9 - Open Ended - Comments Box

Please provide contact information for follow-up clarification and distribution of survey results:

APPENDIX B – TEST RESULTS






Drained Triaxial Test - 0% Fines, 95%RC
ϕ' = 38.9°
c' = 1.78psi
Shear stress τ (psi)
Normal stresses σ1, σ3 (psi)
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5
10
15
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30
35
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45
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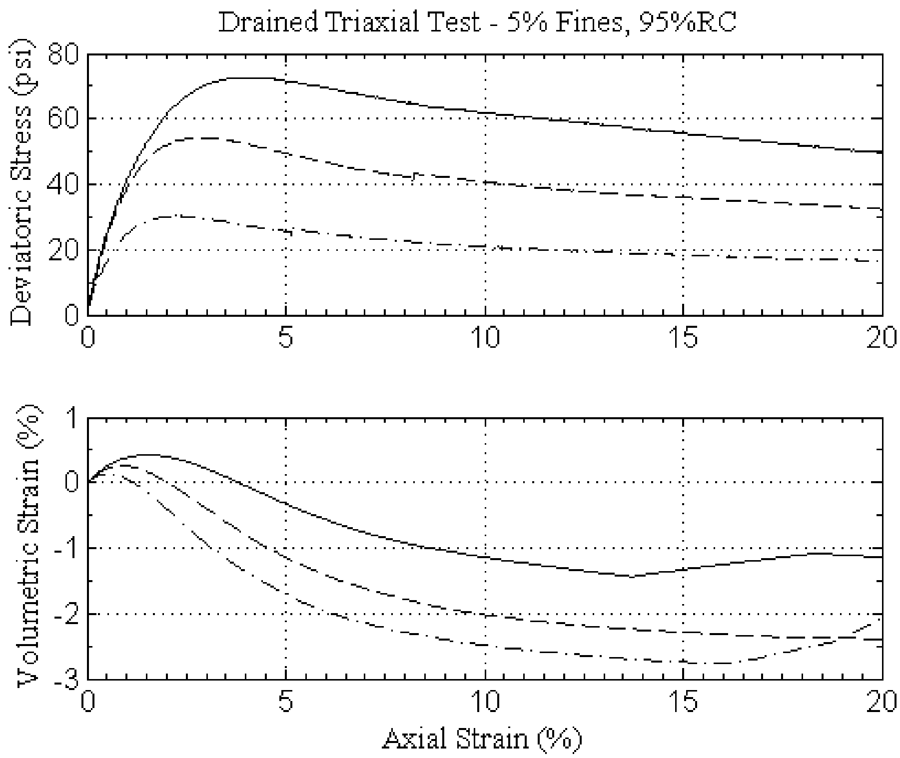
60
70
80
90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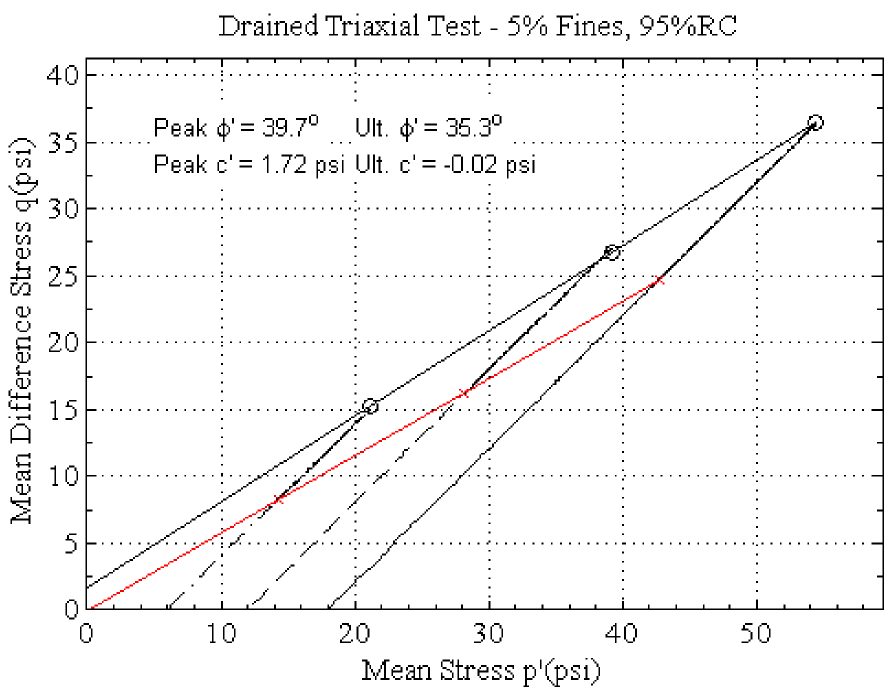
Deviatoric Stress (psi)
Volumetric Strain (%)
Axial Strain (%)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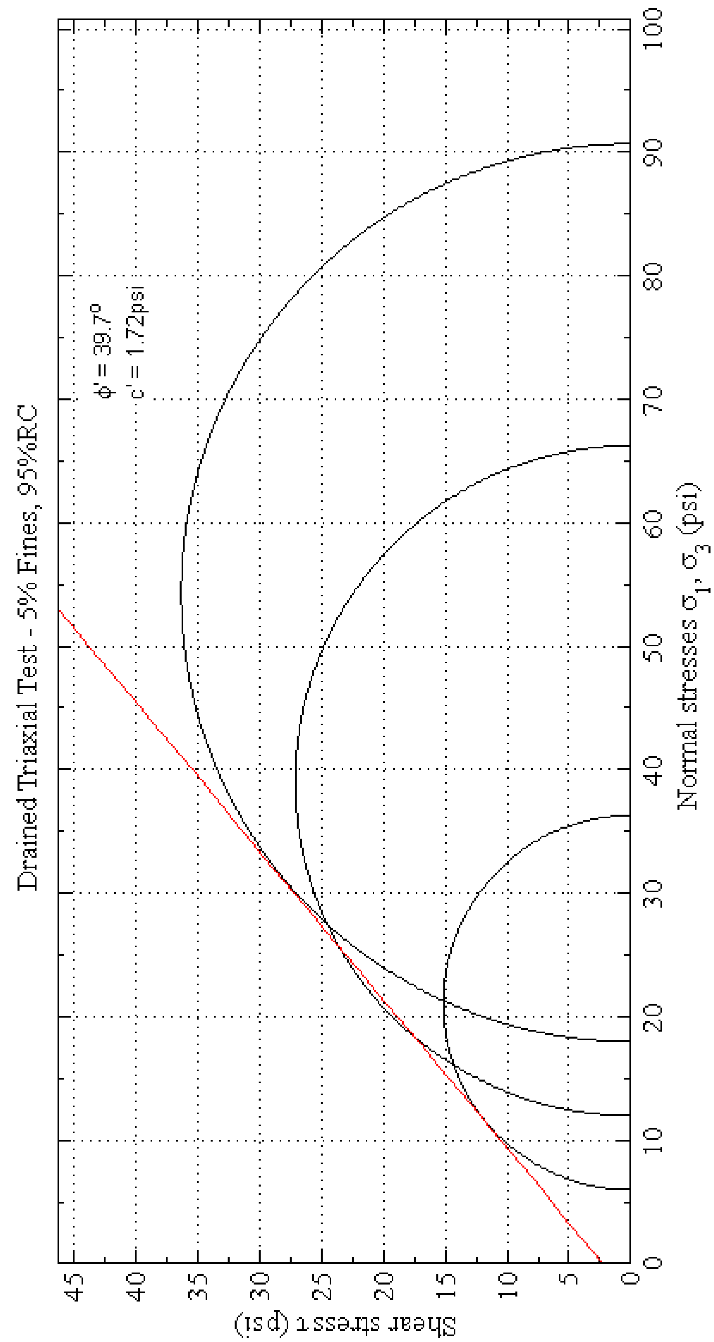
Peak ϕ' = 39.7°
Ult. ϕ' = 35.3°
Peak c' = 1.72 psi
Ult. c' = -0.02 psi
Mean Difference Stress q(psi)
Mean Stress p'(psi)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φ' = 39.7º
c' = 1.72psi
Shear stress τ (psi)
Normal stresses σ1, σ3 (psi)
0
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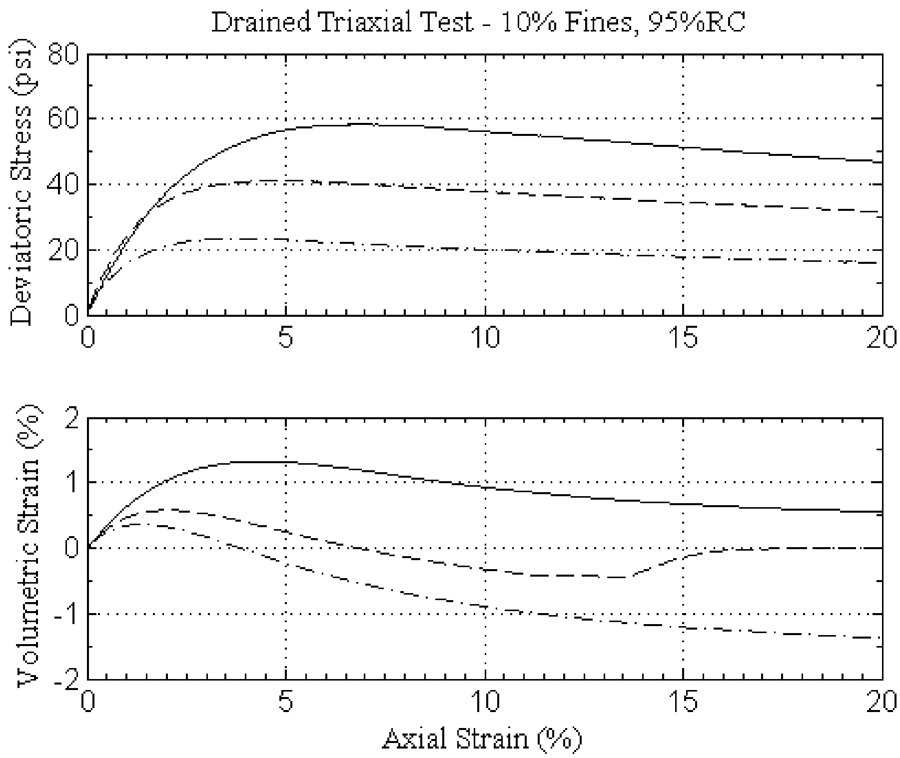
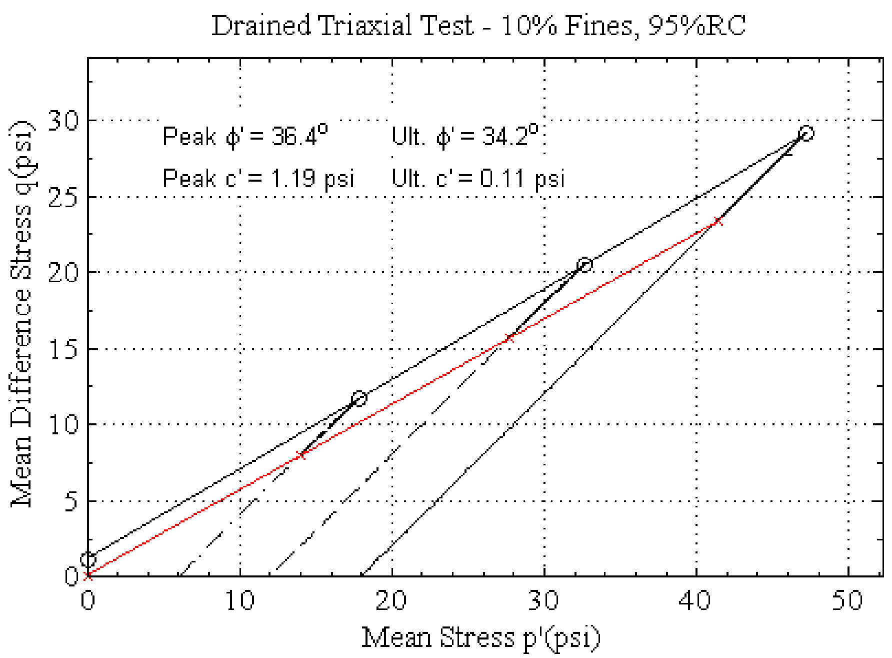
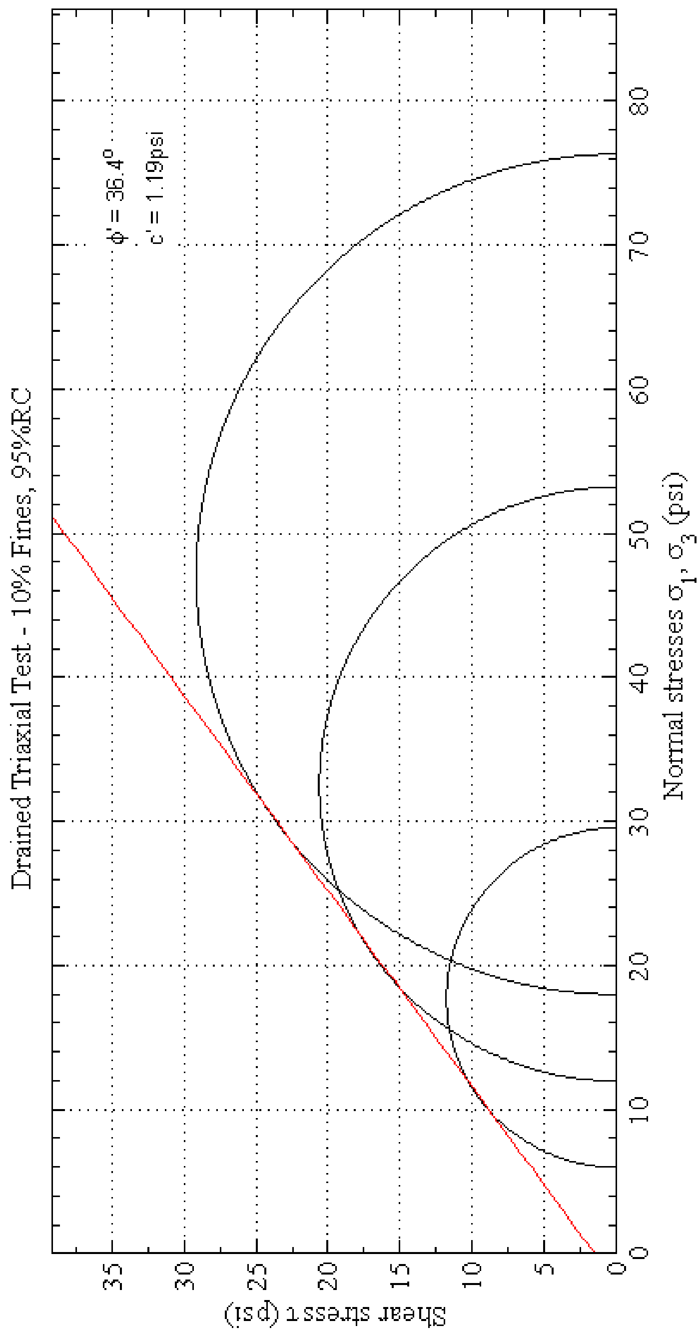


Drained Triaxial Test - 10% Fines, 95%RC
Deviatoric Stress (psi)
Volumetric Strain (%)
Axial Strain (%)



Drained Triaxial Test - 10% Fines, 95%RC
Peak φ' = 36.4°
Ult. φ' = 34.2°
Peak c' = 1.19 psi
Ult. c' = 0.11 psi
Mean Difference Stress q(psi)
Mean Stress p'(psi)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φ' = 36.4°
c' = 1.19psi
Normal stresses σ1, σ3 (psi)
Shear stress τ (psi)
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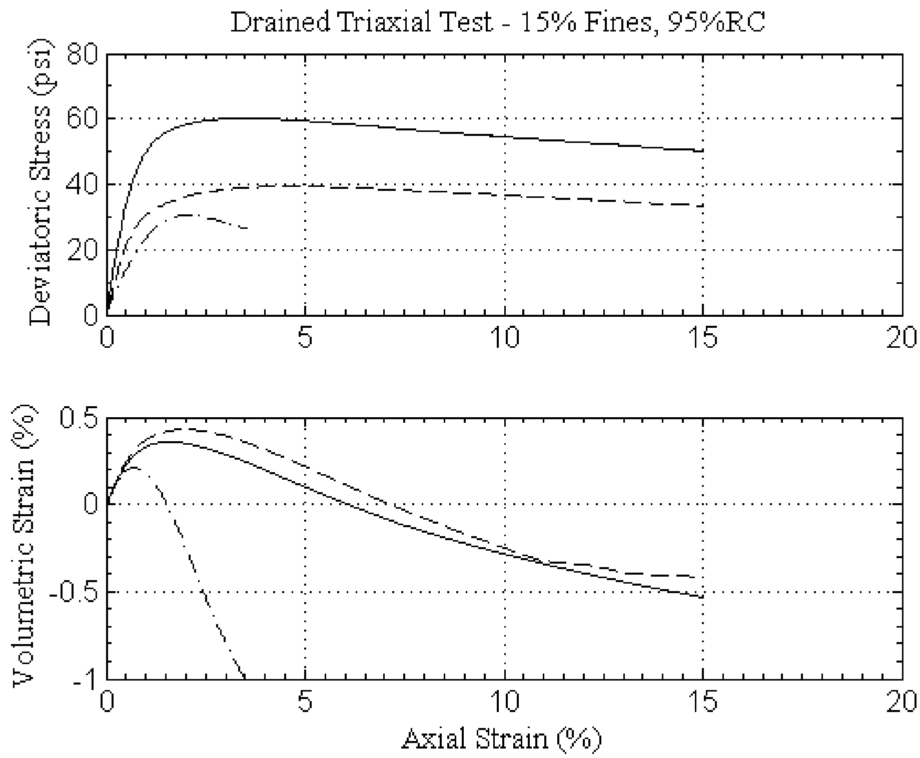
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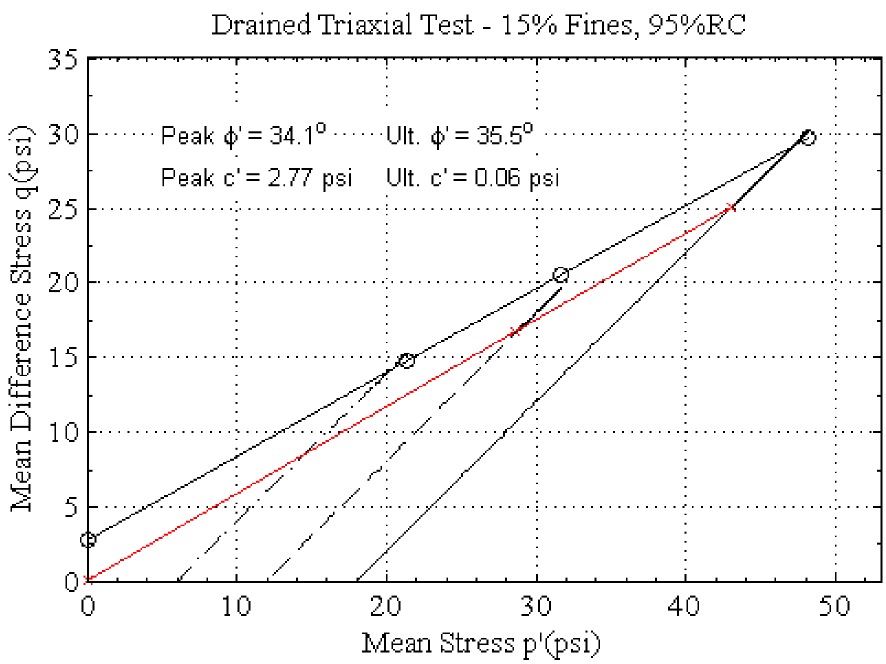
Deviatoric Stress (psi)
Volumetric Strain (%)
Axial Strain (%)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Ult. c' = 0.06 psi
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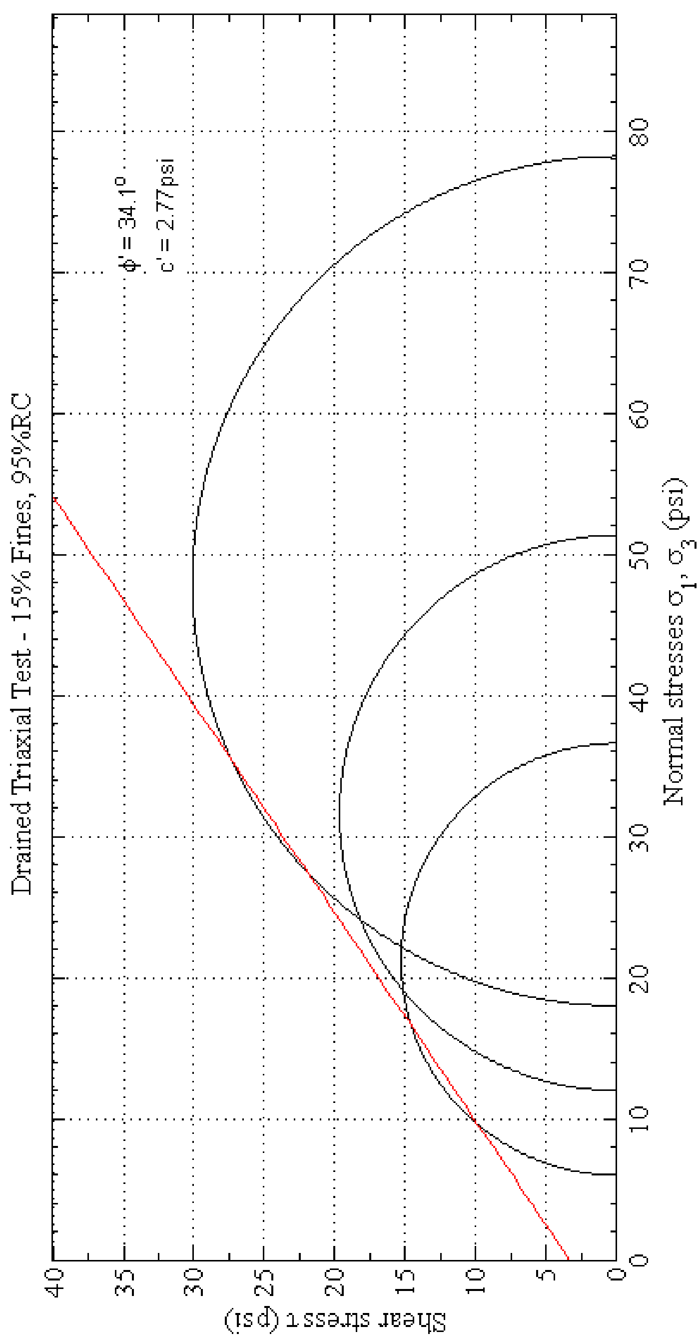
Mean Stress p'(psi)



Drained Triaxial Test - 15% Fines, 95%RC
φ' = 34.1°
c' = 2.77psi
Shear stress τ (psi)
Normal stresses σ1, σ3 (psi)
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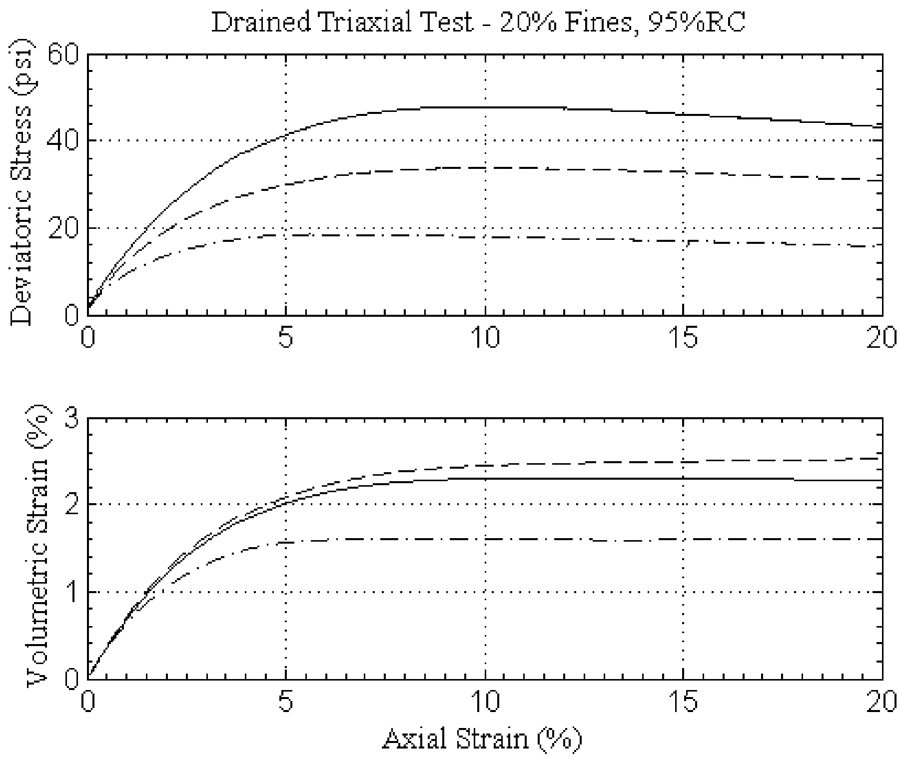
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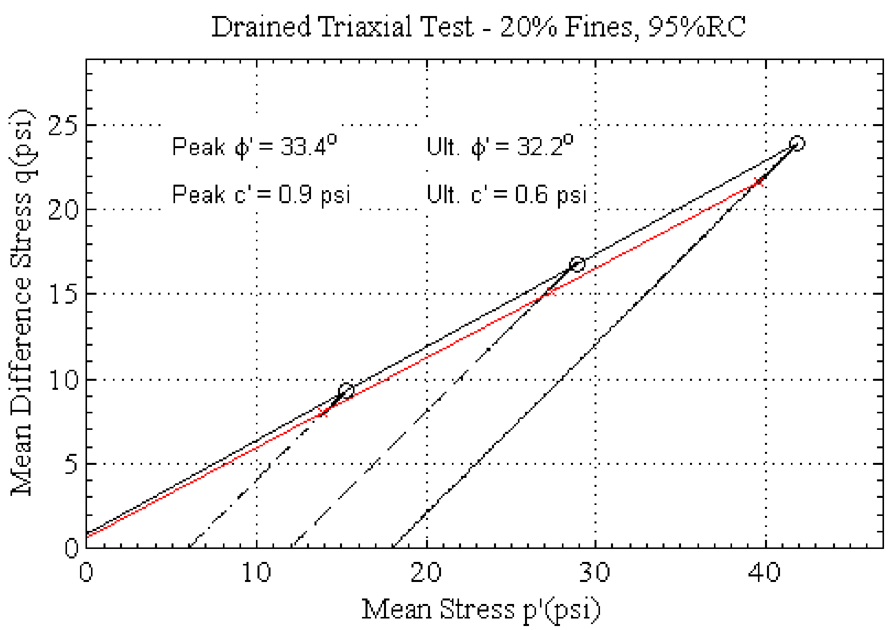
Deviatoric Stress (psi)
Volumetric Strain (%)
Axial Strain (%)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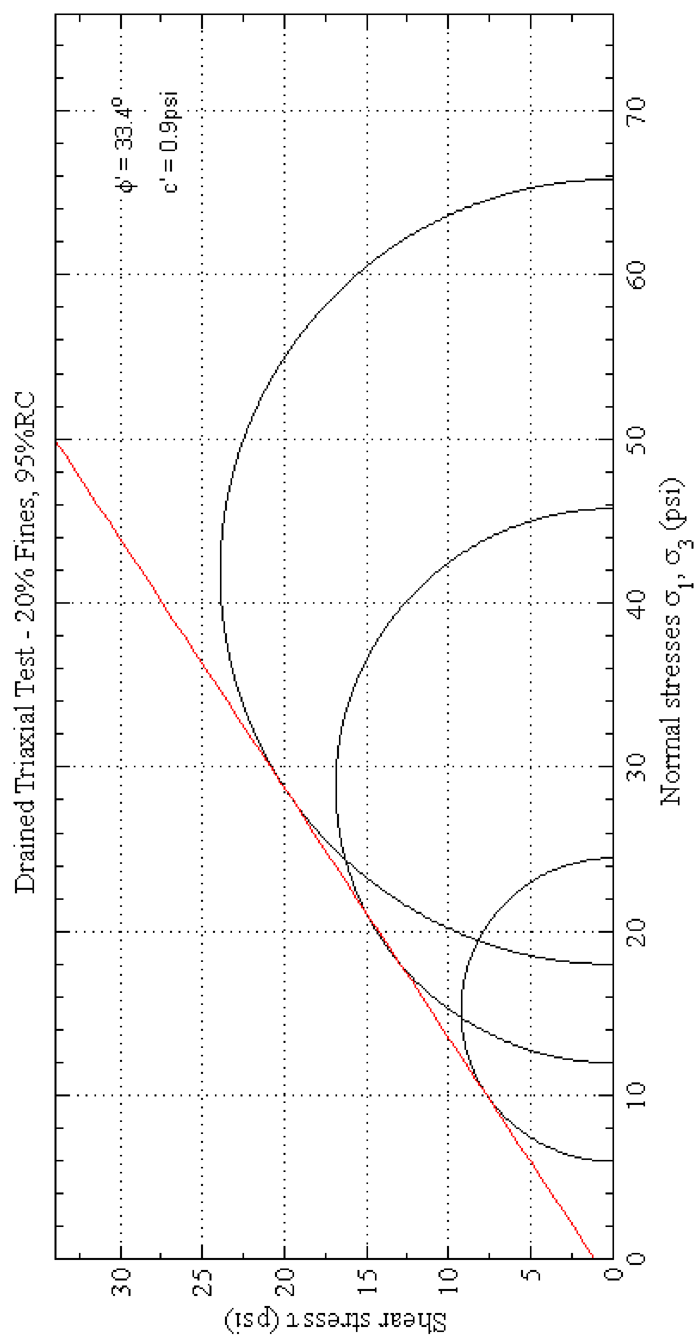
Ult. c' = 0.6 psi
Mean Difference Stress q(psi)
Mean Stress p'(psi)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Shear stress τ (psi)
Normal stresses σ_1, σ_3 (psi)
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c' = 0.9psi
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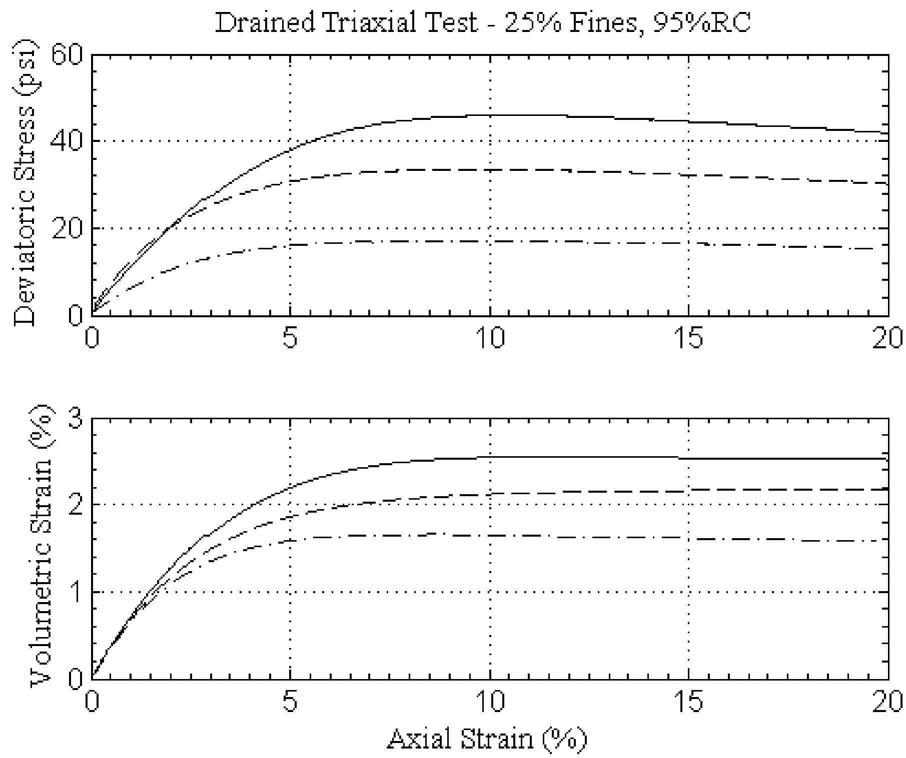
70



Drained Triaxial Test - 25% Fines, 95%RC
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Volumetric Strain (%)
Axial Strain (%)
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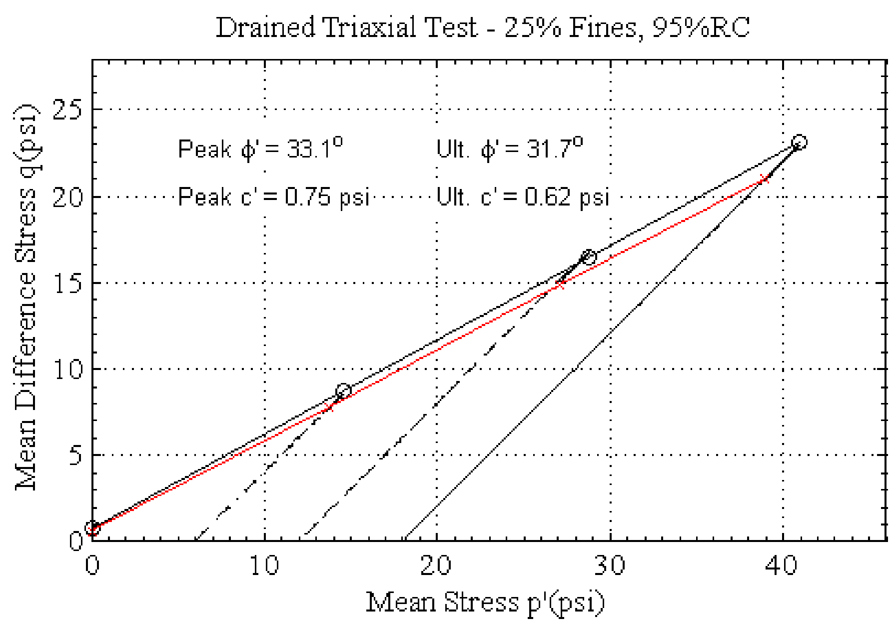
0
3
2
1



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Peak ϕ' = 33.1°
Ult. ϕ' = 31.7°
Peak c' = 0.75 psi
Ult. c' = 0.62 psi
Mean Difference Stress q(psi)
Mean Stress p'(psi)
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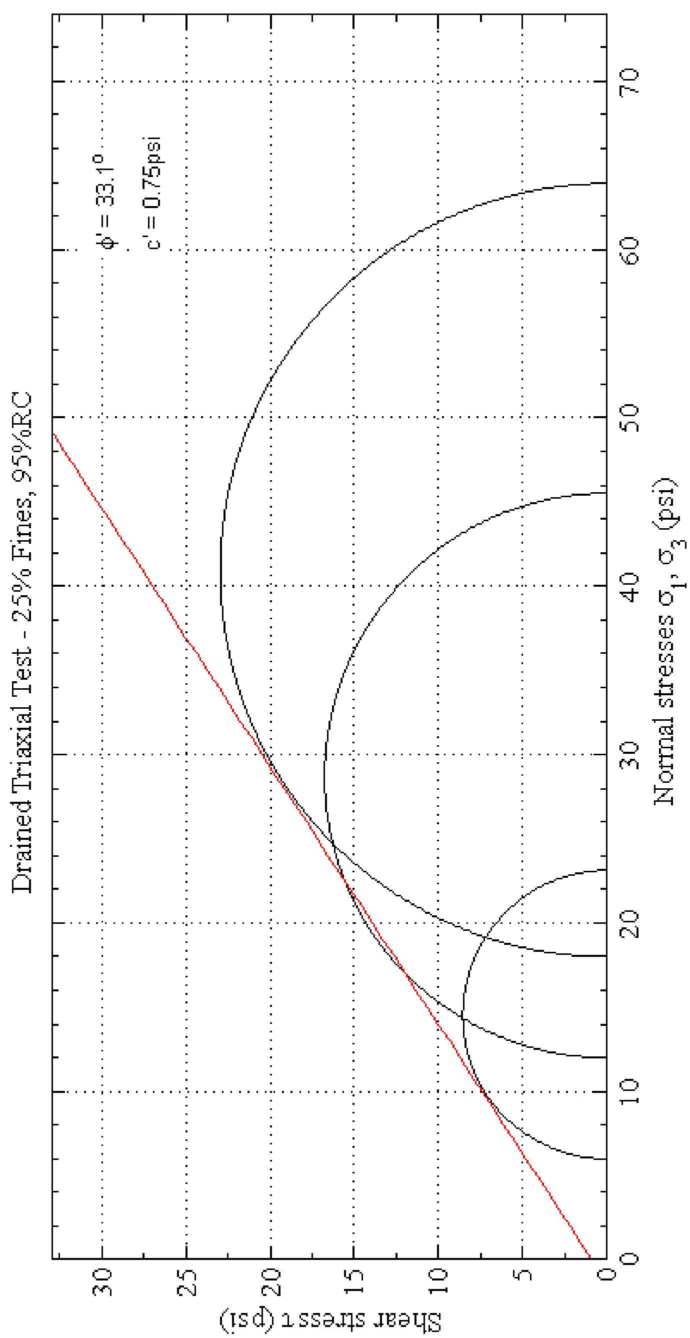
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c' = 0.75psi
Normal stresses σ1, σ3 (psi)
Shear stress τ (psi)
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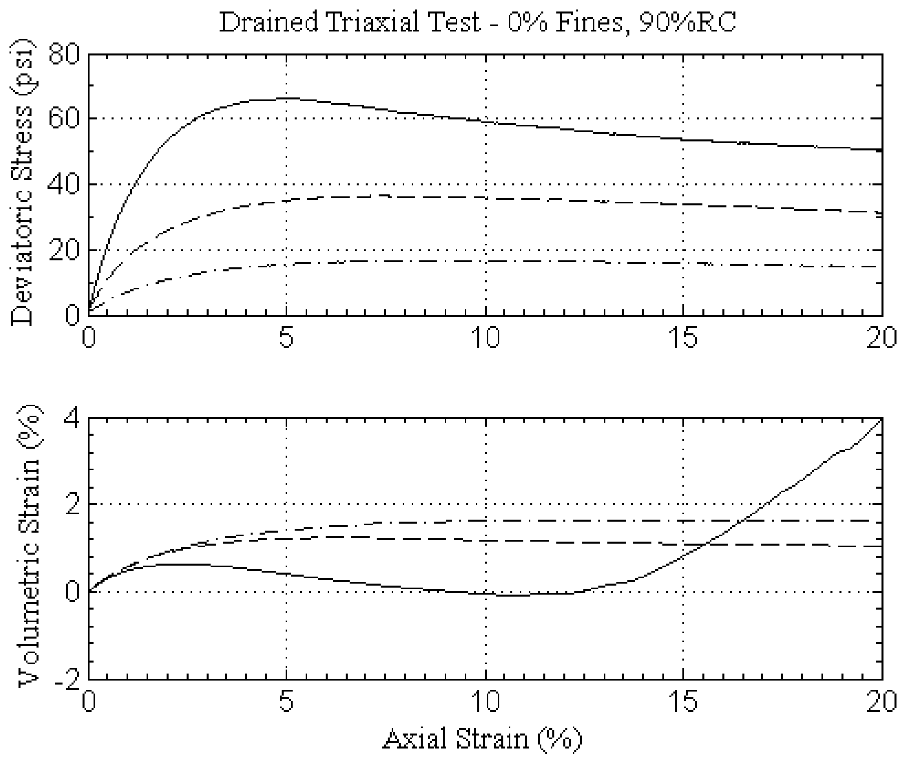
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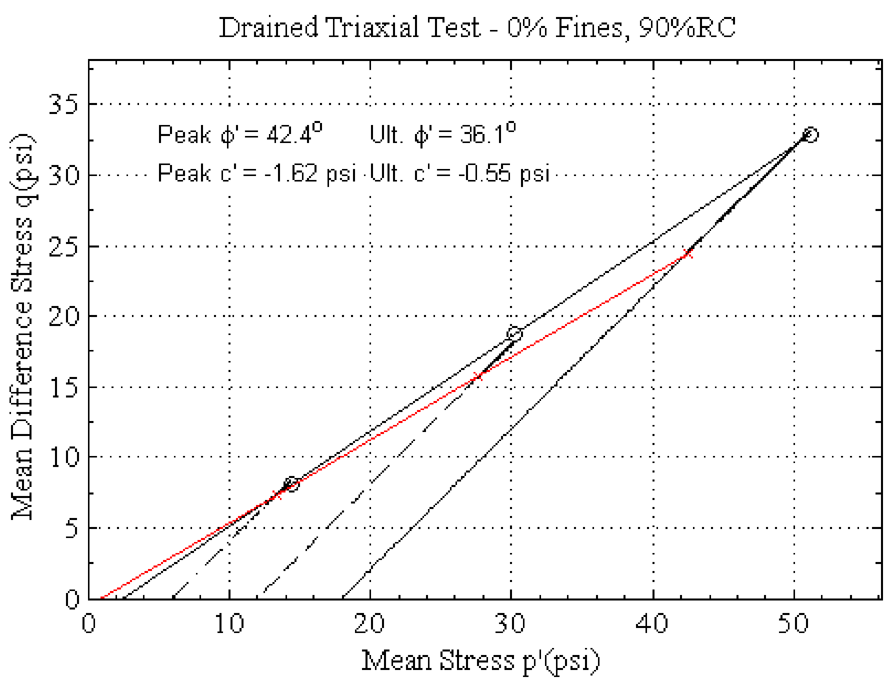
Deviatoric Stress (psi)
Volumetric Strain (%)
Axial Strain (%)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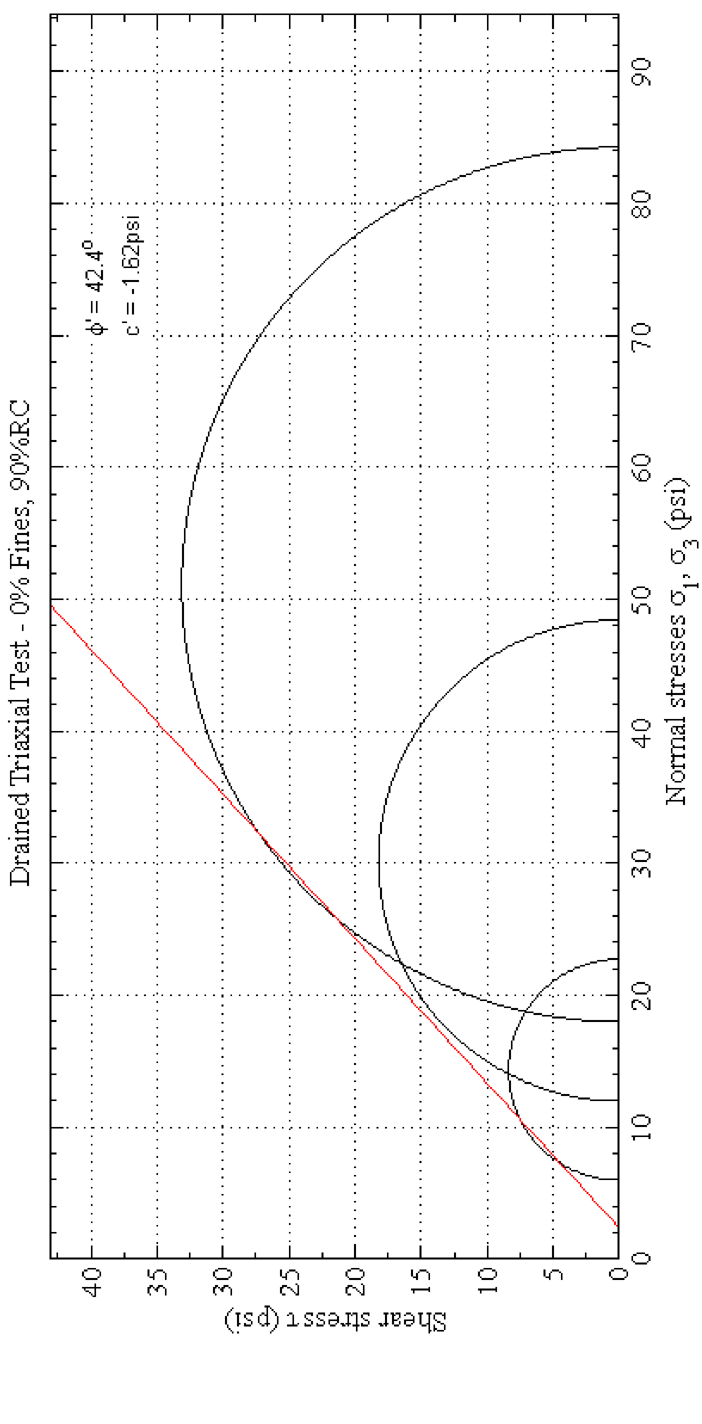
Mean Difference Stress q(psi)
Mean Stress p'(psi)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φ' = 42.4°
c' = -1.62psi
Shear stress τ (psi)
Normal stresses σ1, σ3 (psi)
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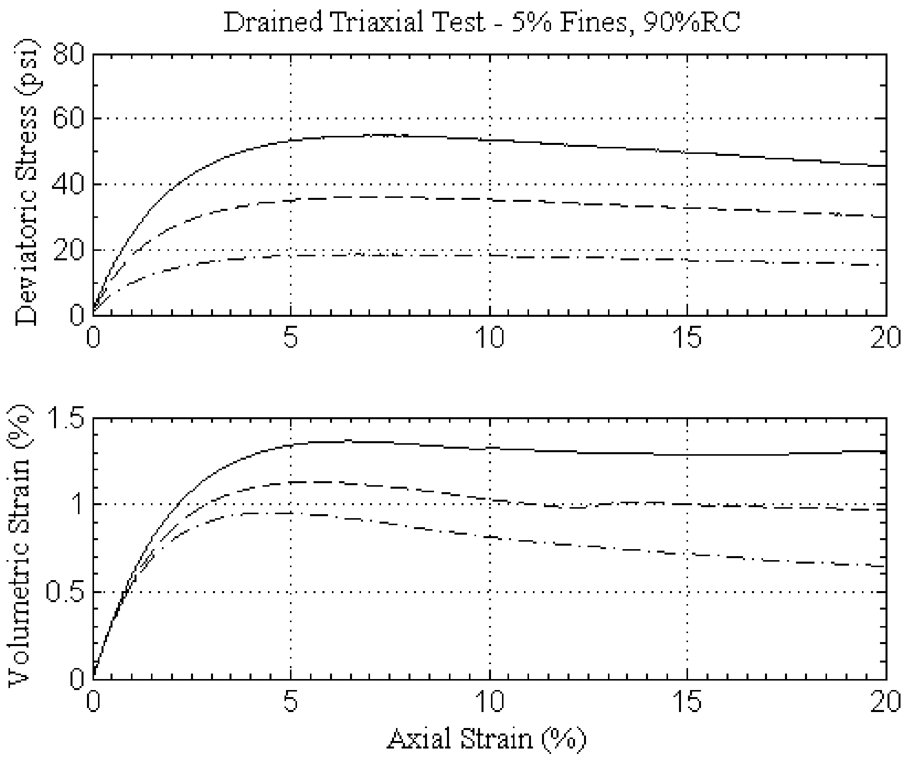
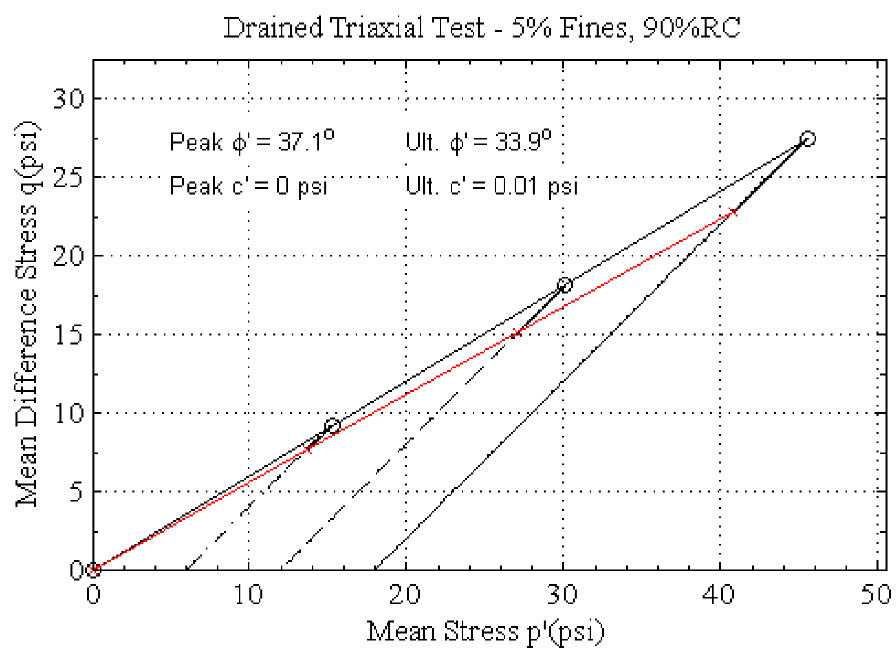
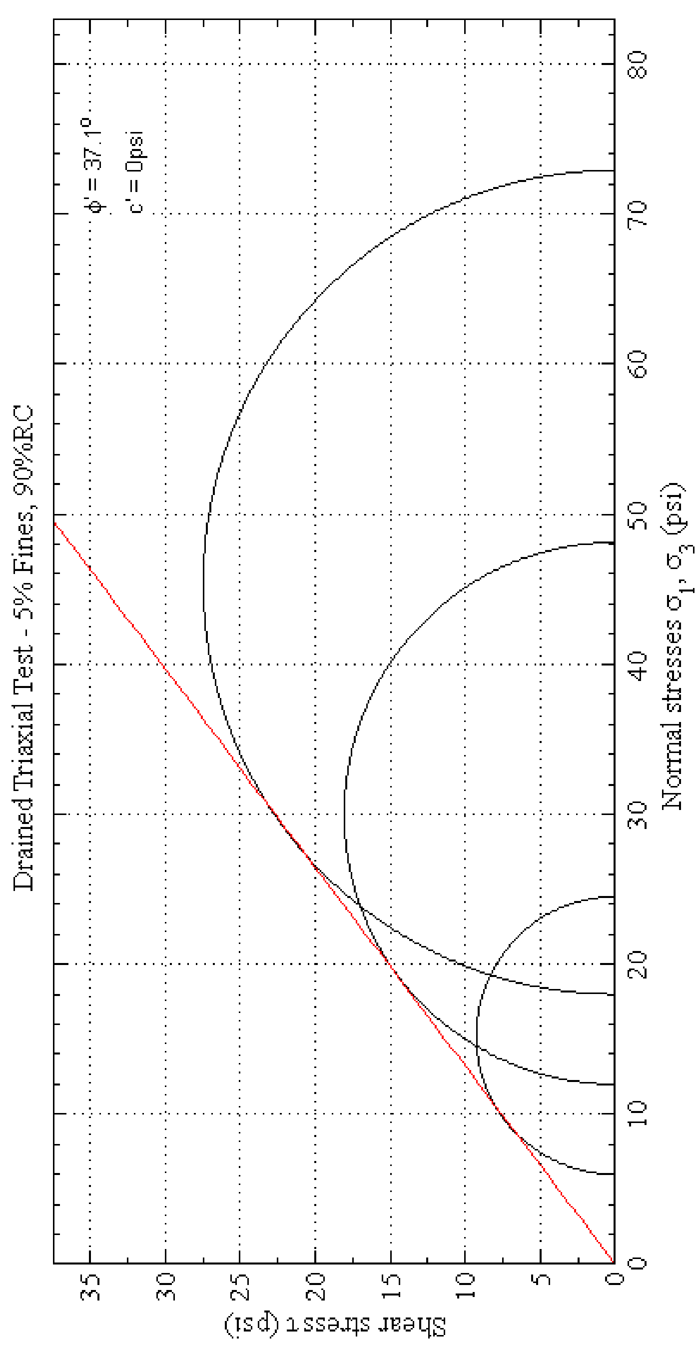


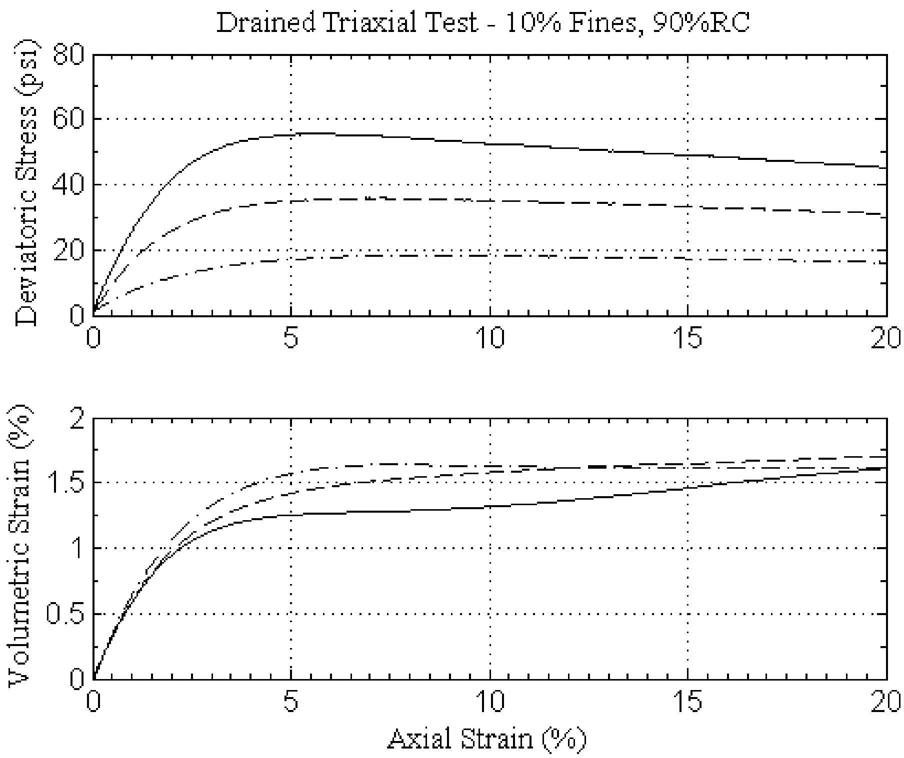
Drained Triaxial Test - 5% Fines, 90%RC
Deviatoric Stress (psi)
Volumetric Strain (%)
Axial Strain (%)

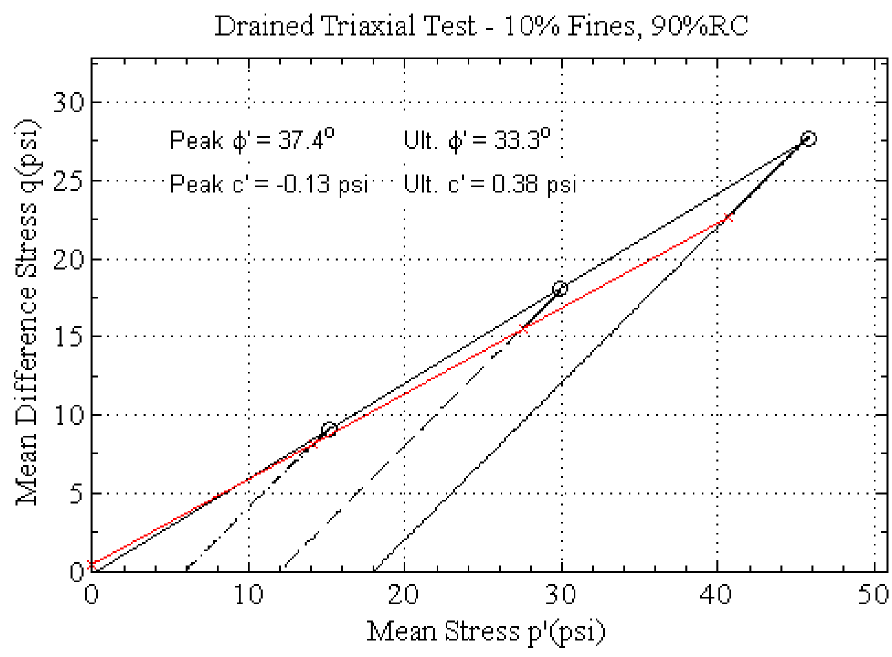


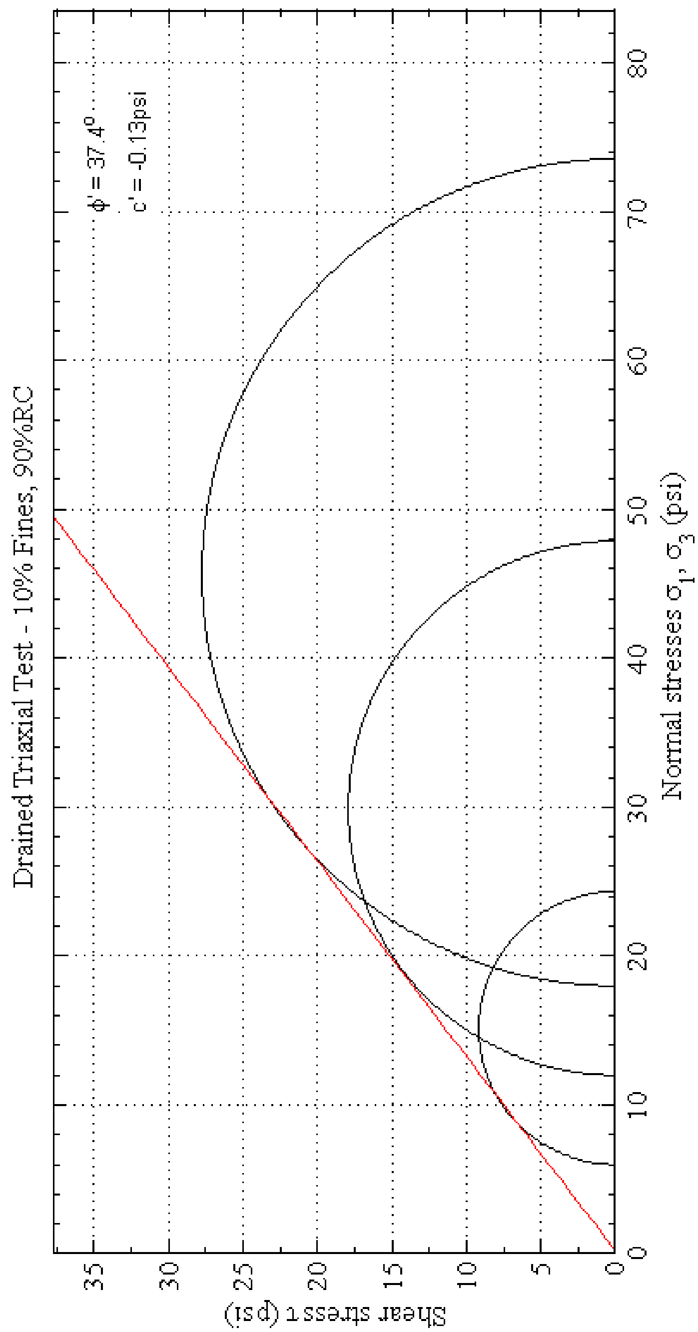
Drained Triaxial Test - 5% Fines, 90%RC
Peak ϕ' = 37.1°
Peak c' = 0 psi
Ult. ϕ' = 33.9°
Ult. c' = 0.01 psi
Mean Difference Stress q(psi)
Mean Stress p'(psi)

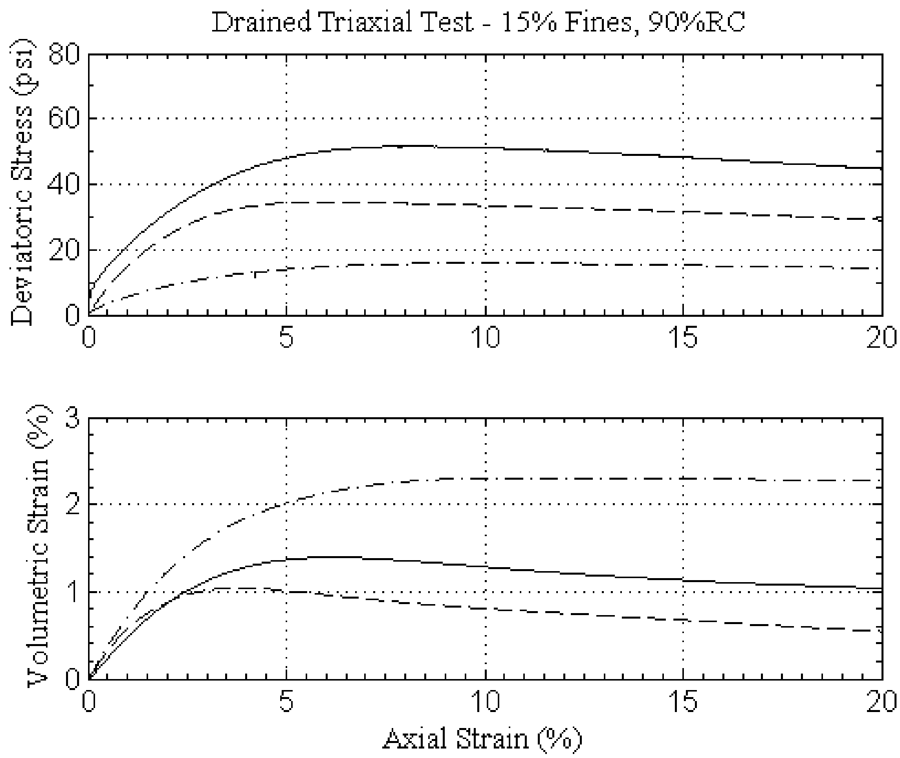


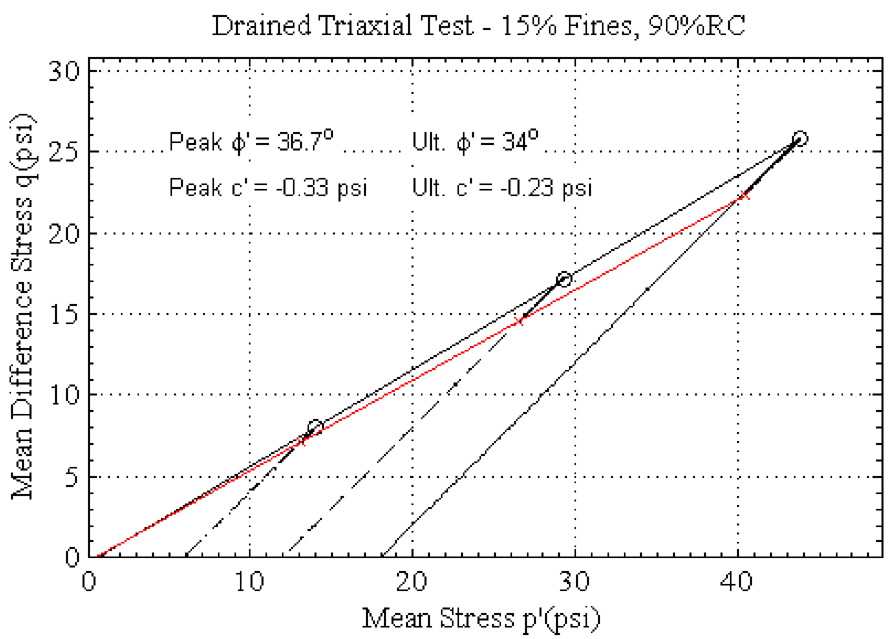
Drained Triaxial Test - 5% Fines, 90%RC
φ' = 37.1°
c' = 0psi
Shear stress τ (psi)
Normal stresses σ1, σ3 (psi)
0
5
10
15
20
25
30
35
0
10
20
30
40
50
60
70
80

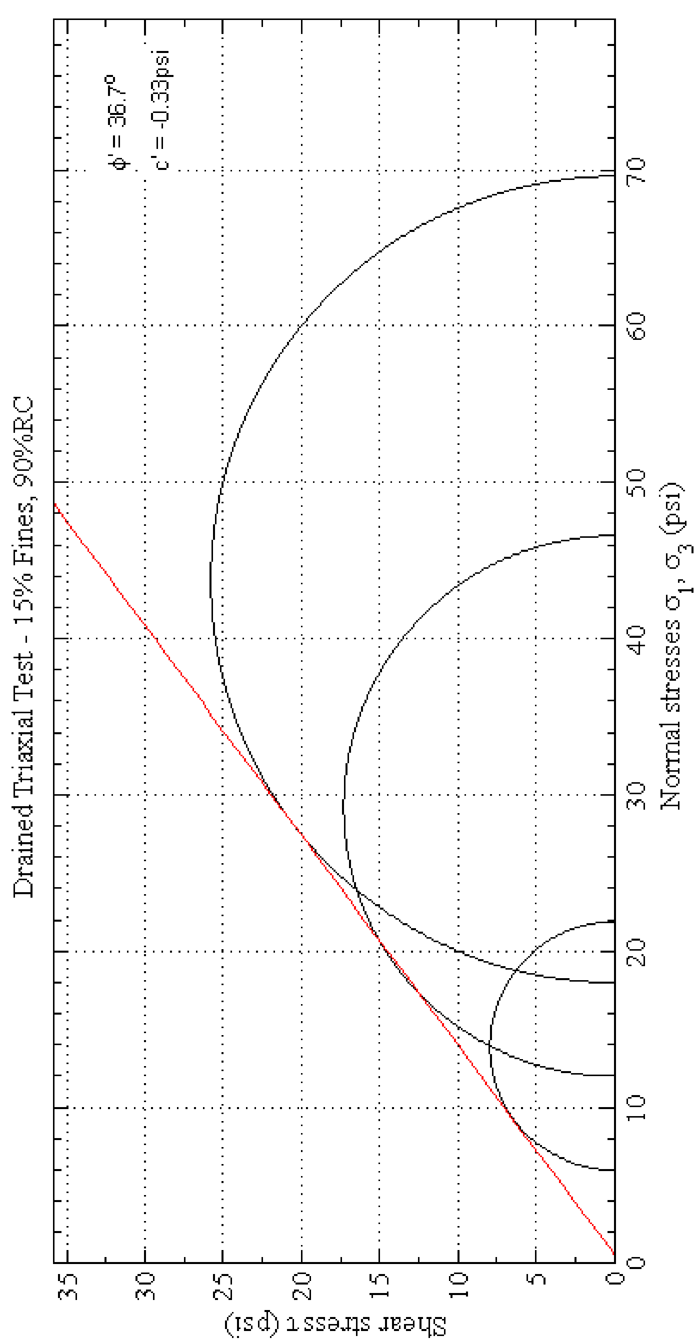

Drained Triaxial Test - 10% Fines, 90%RC
Deviatoric Stress (psi)
80
60
40
20
0
0
5
10
15
20
Volumetric Strain (%)
2
1.5
1
0.5
0
0
5
10
15
20
Axial Strain (%)


Drained Triaxial Test - 10% Fines, 90%RC
Peak ϕ' = 37.4°
Ult. ϕ' = 33.3°
Peak c' = -0.13 psi
Ult. c' = 0.38 psi
Mean Difference Stress q(psi)
30
25
20
15
10
5
0
0
10
20
30
40
50
Mean Stress p'(psi)

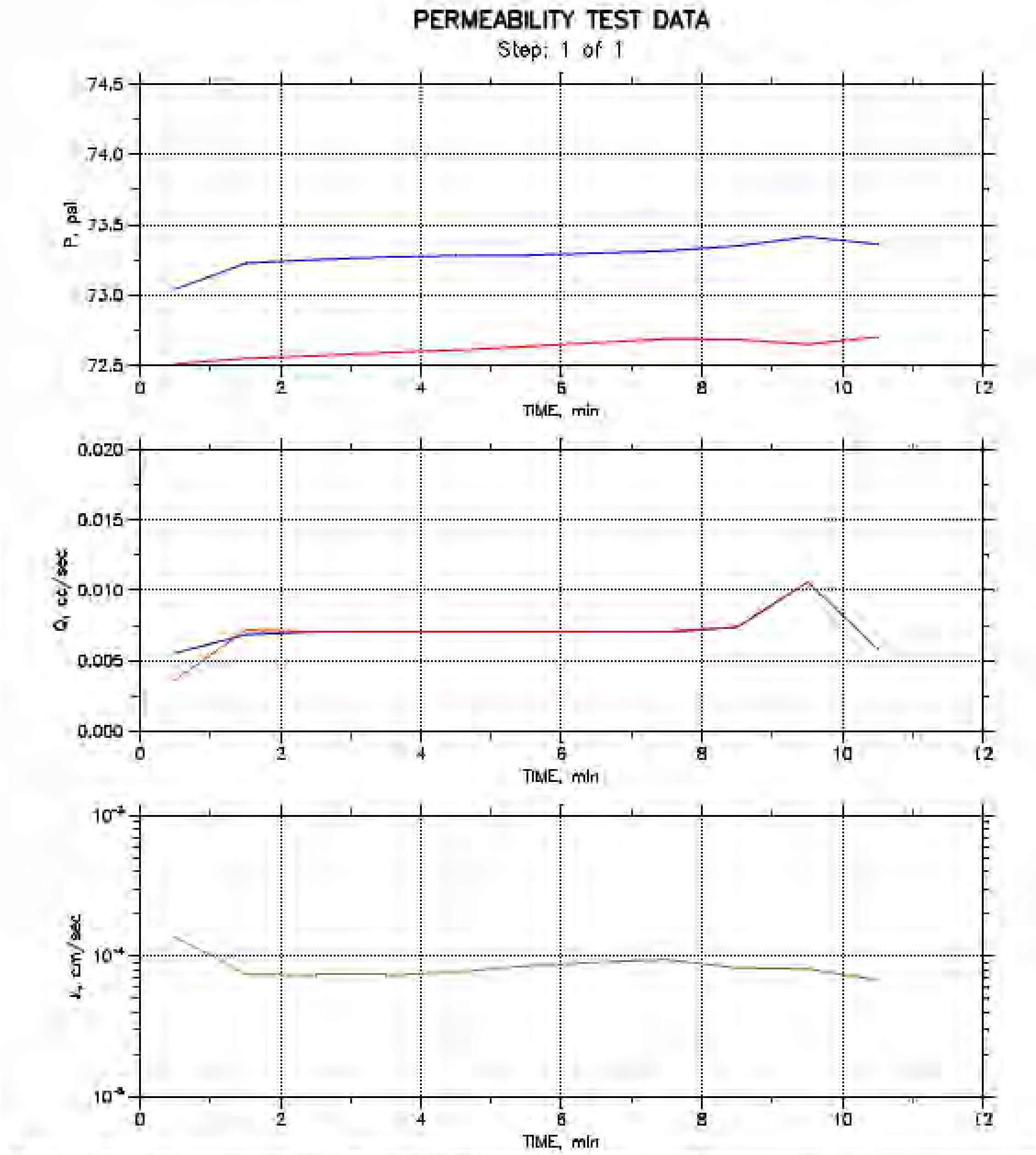

Drained Triaxial Test - 10% Fines, 90%RC
φ' = 37.4°
c' = -0.13psi
Normal stresses σ1, σ3 (psi)
Shear stress τ (psi)


Drained Triaxial Test - 15% Fines, 90%RC
Deviatoric Stress (psi)
Volumetric Strain (%)
Axial Strain (%)

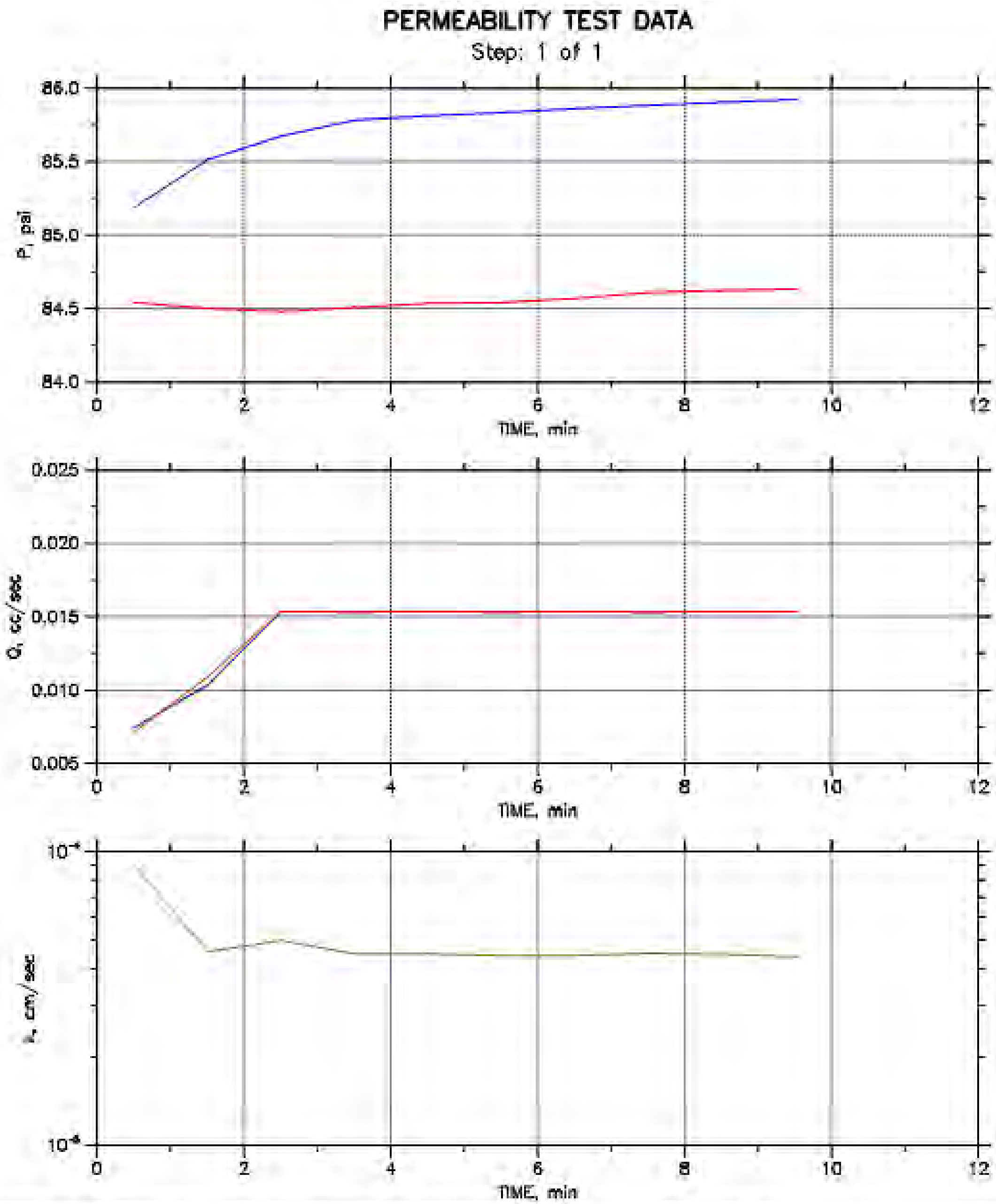

Drained Triaxial Test - 15% Fines, 90%RC
Peak φ' = 36.7°
Ult. φ' = 34°
Peak c' = -0.33 psi
Ult. c' = -0.23 psi
Mean Difference Stress q(psi)
Mean Stress p'(psi)


Drained Triaxial Test - 15% Fines, 90%RC
φ' = 36.7°
c' = -0.33psi
Shear stress τ (psi)
Normal stresses σ1, σ3 (psi)

(I) TYPICAL HYDRAULIC CONDUCTIVITY TEST DATA



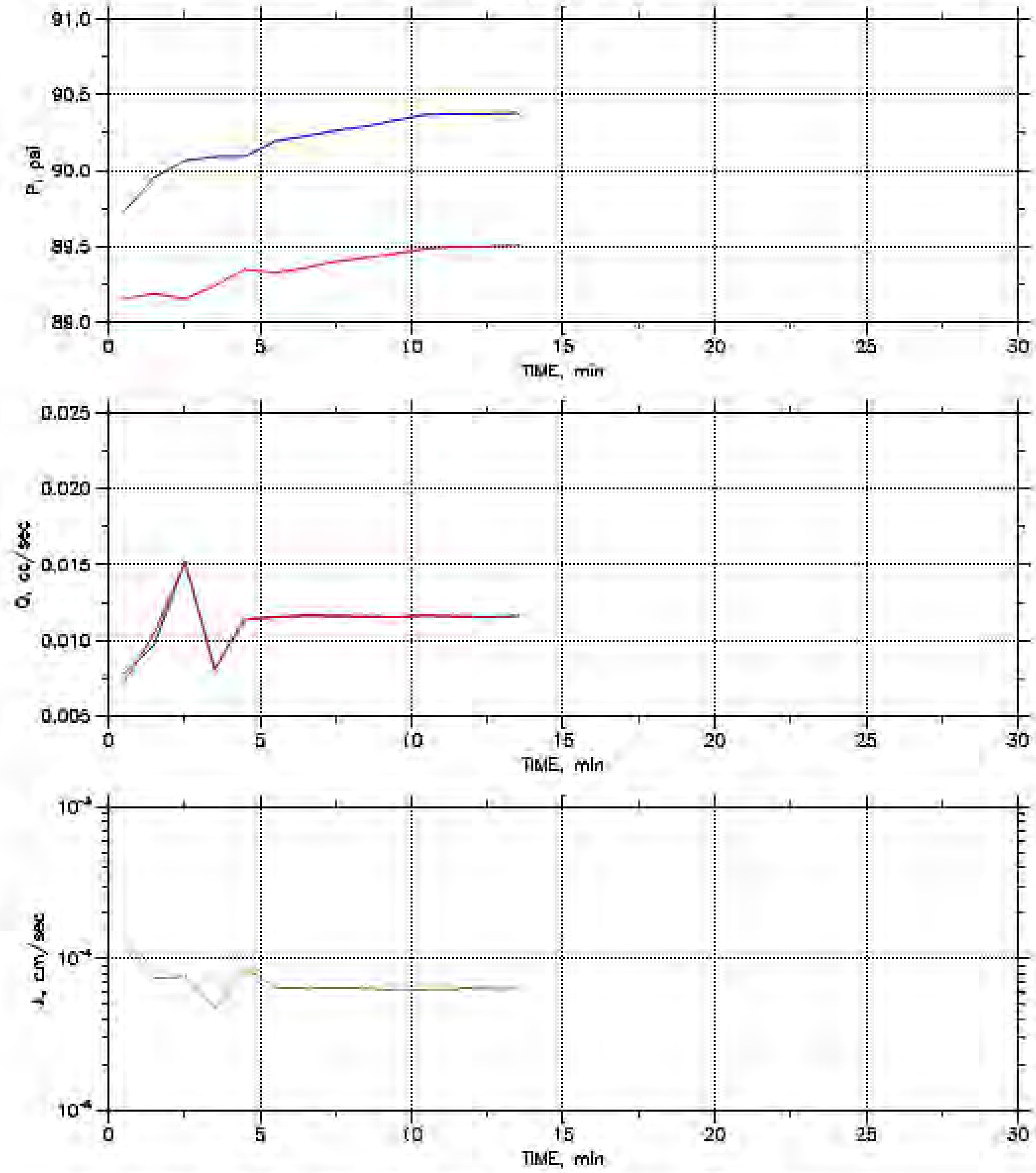
Project: VTrans Abutment Backfill	Location: UVM Engineering	Project No.:
Boring No.:	Tested By: BG	Checked By:
Sample No.: 1	Test Date: 5-24-13	Depth:
Test No.: 1	Sample Type: 25% Fines	Elevation:
Description: 95% RC, 6 psi confinement, CD test 6psi		
Remarks:		



Project: VTrans Abutment Backfill	Location: UVM Engineering	Project No.:
Boring No.:	Tested By: BG	Checked By:
Sample No.: 1	Test Date: 6-26-13	Depth:
Test No.: 1	Sample Type: 25% Fines	Elevation:
Description: 95% RC, 12 psi confinement, CD test 12psi		
Remarks:		

PERMEABILITY TEST DATA

Step: 1 of 1



Project: VTrans Abutment Backfill	Location: UVM Engineering	Project No.:
Boring No.:	Tested By: BC	Checked By:
Sample No.: 1	Test Date: 5-24-13	Depth:
Test No.: 0	Sample Type: 25% Fines	Elevation:
Description: 95% RC, 15 psi confinement, CD test 15psi		
Remarks:		