

Report No. UT-07.04

**ASSESSMENT OF SPECIFIC
GRAVITY TESTING WITH A
MUD BALANCE FOR
QUALITY ASSURANCE OF
CEMENT GROUT IN
GROUND NAIL
INSTALLATION**

Prepared For:

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RESEARCH



UDOT RESEARCH & DEVELOPMENT REPORT ABSTRACT

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16. Abstract In early 2005, construction began on the most current segment of Provo Canyon reconstruction, US-189 Wildwood to Deer Creek. This construction project was undertaken to widen an approximate 5-mile stretch of the existing 2-lane highway to 4-lanes. The project was located in a very challenging geologic setting where many of the cut slopes required either ground nail (soil nail and rock dowel) or tieback reinforcement. A neat cement grout, consisting of a 0.5 water-cement ratio (by weight), was used for both tieback and ground nail installation. The primary means of quality assurance of the cement grout by specification included creating daily grout cubes with random sampling of the wet grout and later laboratory testing the compressive strength of the grout cubes at specified intervals of time. The Contractor suggested an alternative method for quality assurance of the neat cement grout, where the specific gravity of the wet grout would be measured with a mud balance and the potential compressive strength of the grout drawn from specific gravity and compressive strength correlations. Periodic cube sampling of the grout would continue to be maintained to verify the strength correlations. Rather than completely abandon the daily cube sampling required by specification, both test methods were evaluated simultaneously. This report summarizes the findings of that evaluation, as regular cube sampling is compared to specific gravity testing with a mud balance device, as means of quality assurance of a neat cement grout.			
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1 Introduction

1.1 Project Background

In early 2005, construction began on the most current segment of Provo Canyon reconstruction, US-189 Wildwood to Deer Creek. US-189 is a principal arterial highway that runs from Provo, Utah, paralleling the Provo River up into the Wasatch Mountains, to Heber City, Utah. This \$85 million construction project, performed by the Utah Department of Transportation (UDOT) in conjunction with Ames Construction, Inc., was undertaken to widen an approximate 5-mile stretch of the existing 2-lane highway to 4-lanes (See UDOT, 2004a & 2004b).

The Provo Canyon Reconstruction Project was located in a very challenging geologic setting, with the highway often constrained by the Provo River on one side and steep rocky limestone and quartzite slopes on the other. This rock in many instances is highly fractured and very susceptible to weathering, thus causing a very high rockfall hazard to the travelling public. Additionally, the highway traverses a large historic landslide area known as the Hoover Slide. This deep-seated geologic feature has shown localized creeping for decades and is associated with the Manning Canyon Shale Formation, a soft, black, low strength material that is also highly susceptible to weathering. The highway was completely realigned through the Hoover Slide area by shifting it further away from the river to a more stable location up on the slide mass. The preliminary project geotechnical investigation and recommendations are summarized in Parsons Brinkerhoff Quade and Douglas, Inc. (2003a, 2003b, and 2004). Additional design and construction issues associated with this project can be further referenced in Farnsworth (2006), Farnsworth et. al. (2007), and Lee et. al. (2007).

The reconstruction project included many challenging construction features associated with mitigating landslide and rockfall potential along the highway. To create sufficient room for the widened highway, large cuts towering as high as 180 feet were required in the adjacent slopes. To ensure stability of these large cut slopes, ground nails (i.e., soil nails and rock dowels) with reinforced shotcrete were utilized. More than 12,700 ground nails varying from 10 feet to 50 feet in length totaling nearly 230,000 linear feet, along with more than 6,500 cubic yards of structural shotcrete, were used to support the steep soil/rock cut slopes. Tiebacks were utilized in the soil cut slopes through the Hoover Slide area as a means of slope stabilization. Approximately 300 tiebacks up to 160

feet in length, totaling around 33,500 linear feet, were used to support these cut slopes within the Manning Canyon Shale Formation. A neat cement grout (consisting of only cement and water) was used for both ground nail and tieback applications.

The primary means of quality assurance for the cement grout as specified in the contract documents was obtaining cube samples daily from the project site to be later laboratory strength tested at specified time intervals (UDOT 2004a). During the submittal review shortly after the contract was awarded, the sub-contractor over the ground nail and tieback installation (Schnabel Foundation Company) requested that the specifications be modified to allow for specific gravity testing with a mud balance device along with periodic cube samples gathered for strength testing. Rather than completely abandon the daily cube sampling in lieu of mud balance testing at that time, it was determined a better alternative for UDOT would be to seize the opportunity to study the feasibility of using the mud balance for future applications. Furthermore, the topic was presented at UDOT's annual UTRAC meeting in March, 2006 to both the geotechnical and materials groups (UDOT 2006 & 2007) where it was subsequently prioritized as being a topic that would benefit UDOT. This report summarizes the Provo Canyon Team's experiences with using a mud balance for grout strength correlation and comparison with daily cube sampling as used on the Provo Canyon Reconstruction Project.

1.2 Objectives of Mud Balance Study

The overall purpose of this study was to evaluate the feasibility of using a mud balance device for specific gravity testing and subsequently correlating the strength of a neat cement grout. The primary objectives/tasks included the following:

1. Literature search for previous application of mud balance testing
2. Use the actual construction project as a means of gathering mud balance and grout cube data for further identification of any correlation that may exist between the two
3. Report the findings and make recommendations

2 Cement Grout

2.1 Grout Production

Ground nails, or more specifically soil nails and rock dowels, were used extensively on the Provo Canyon Reconstruction Project. Installation of the ground nails takes place as the excavation of the cut slope proceeds from the top down. The basic steps of installation include the following (as described in FHWA 1994):

1. Excavate a small height cut
2. Drill hole for nail
3. Install and grout soil nail tendon
4. Place geocomposite drain strips, initial shotcrete layer, and install bearing plates and nuts
5. Repeat process, incrementally downward to final grade, and
6. Place final facing (if required for permanent walls)

To facilitate the enormous amount of ground nailing being performed throughout the Provo Canyon Reconstruction Project, as well as ensure that these steps were being performed efficiently, there were as many as 5 drills located across the project. Each drill (or set of drills where they may have been working in tandem) essentially had a grout mixer located with it, and the nails were grouted toward the end of each workday. Each batch of grout was prepared by first filling the grout mixer with water, and the construction workers then mixing in several bags of cement with the water. One batch of grout was generally sufficient to fill 2 to 3 short (~15 ft) holes. However, for longer drill holes or holes that were prone to grout loss, more than one batch of grout had to be mixed to fill just the one hole. At any rate, it is important to point out that there were numerous batches of grout mixed at several different locations throughout the project each day.

Tiebacks were also used at two different locations on the Provo Canyon Reconstruction Project. Installation of tieback reinforcement essentially follows the general installation method previously described for ground nails, but tieback reinforcement does include several functional differences. Ground nails usually consist of a steel bar while tiebacks are fabricated with a series of steel tendons. Ground nails are completely grouted in an at-rest state and the loading conditions are later mobilized as further excavation takes place below. Tieback reinforcement on the other hand, consists of bonded and unbonded portions within the drilled

hole. The bottom portion of the tieback is generally anchored in a more stable material where it is bonded to the hole. Each tieback is grouted after placement of the tieback into the hole, and thus a grout mixing station is required on site. The grout is then allowed to cure to a specified strength. An anchor block is used as bearing at the surface and the tieback is then tensioned and locked off at the required load upon adequate curing of the grout. The unbonded portion of the tieback facilitates the tensioning between the bonded portion of the tieback and the surface bearing block.

The grout utilized on the Provo Canyon Reconstruction Project for both ground nail and tieback reinforcement application consisted of a neat cement grout, or simply water and cement. The target water-cement ratio was 0.5, by weight. Grout production therefore consisted of simply mixing a certain number of bags of cement with a certain volume of water, with an equivalent weight ratio of 0.5. As an example, 6 bags of cement (approximately 100 lbs each – so ~600 lbs total) were often mixed with 35 gallons of water (~300 lbs total weight), to achieve a grout batch with a water-cement ratio of 0.5.

“Special Provision 02362S, Open Cut Ground Reinforcement” (Included in Appendix A), contained the following cement grout requirements (Part 2.1.B.1):

1. Provide cement grout with a minimum compressive strength of 1,500 psi within 24 hours after placement, and a minimum compressive strength of 4,000 psi within 28 days after placement, nonshrink, and using materials conforming to the following:
 - a. Portland cement Type II for all soil and rock formations except in weathered or unweathered Manning Canyon Shale use Type V
 - b. Potable water
 - c. Admixtures, non-corrosive to bars and subject to Engineer’s approval

Testing requirements from “Special Provision 02362S” (Part 3.9.A) for gathering and testing cube samples included the following:

1. Provide unconfined compressive strength test results from two grout cube samples taken from the grout mix during each working day. Test the unconfined compressive strength of one of these samples at 14 days; and test the other sample at 28 days

“Special Provision 02496S, Permanent Tiebacks” (Included in Appendix A), contained similar strength and testing requirements, including specifically the minimum 4,000 psi strength allowance and testing of two cube samples obtained from each batch of grout.

It should be noted that both specifications allowed for the use of admixtures, with the approval of the Engineer. With an admixture, the specific gravity of the cement grout becomes a function of the weights of at least three different types of materials as opposed to just two. Where admixtures are used in very small concentrations, as is often typical, their effect on the specific gravity of the grout is minor, if not negligible. The larger the admixture concentration, the larger the effect. Despite the allowance for the use of admixtures, a neat cement grout containing only cement and water was used primarily throughout the project. There was one limited exception where an admixture was included in the grout, but data from that specific instance has not been included in the data set for this report. Studying the effects of the use of admixtures in cement-grout is outside of the scope of this report, but their potential use and impact on the specific gravity procedure is duly noted.

It should further be noted that although the specification contained a provision requiring cement grout with a strength of 1,500 psi within 24 hours after placement be used, there was not a specific item contained within the specification to regularly monitor this requirement. This report will later address this particular provision.

2.2 Grout Cube Testing

The project specifications required that two grout cube samples be taken from the project each day (See Appendix A for Special Provision 02362S Open Cut Ground Reinforcement and Special Provision 02496S Permanent Tiebacks). These cube samples were to be later broken in a laboratory upon curing for a specified amount of time to ensure that the grout had reached adequate strength. By specification, the compressive strength tests were to be performed at 14 and 28 days. The grout cube sampling and testing procedure follows ASTM C109, Compressive Strength of Hydraulic Cement Mortars (ASTM International, 2006).

The grout cubes were created on site using brass grout cube molds, as shown in Figure 2-1. The field instruction sheet for collecting grout cube samples is shown in Appendix B. To create the grout cubes, the cube molds are first assembled and gently lubricated with a light oil to keep the cement grout from bonding to the mold. A sample of the grout is



Figure 2-1. Brass grout cube molds



Figure 2-2. Filling the grout cube mold with grout sample



Figure 2-3. Grout cubes ready for curing

then collected from the grout mixer and poured into the molds, as shown in Figure 2-2. Since the cubes are prone to some slight shrinkage, the cubes are slightly overfilled (Figure 2-3). At this point the cubes are ready to be cured. However, rather than remove the molds from the project site while the grout is still wet, the molds are placed in a cooler (as in Figures) for a period of approximately 24 hours. The cooler not only protects the molds from being disturbed, but also insulates the cubes to some degree from any extreme outside air temperatures. The next day, the molds are then stripped and the cubes removed and transferred to a moisture room at the laboratory where they then continue to cure until they are later broken.

The primary purpose for taking grout cube samples is to verify that the strength of the grout is meeting the minimum established criteria. However, there are a number of negative issues associated with the preparation and testing of grout cubes including the following:

1. Preparation and testing of the cubes is quite labor intensive and thus quite costly (preparation of the samples in the field, removal from the site the following day, storage, compression testing, and all associated tracking and reporting)

2. Sample disturbance from movement, especially during the first day of curing
3. Adverse effects of extreme temperatures and/or moisture
4. Inconsistent grout cube dimensions due to shrinkage of grout
5. Inconsistent break results depending on equipment and testing methods used

To help alleviate some of the inconsistencies, the project team made every effort to ensure that the same techniques and procedures were utilized by the various people participating in the sample and data gathering. Furthermore, those participating were encouraged to use caution and care at every level of sampling and testing to ensure that error could be minimized. While the testing is conducted in a laboratory under fairly controlled conditions, the grout cubes are created in the field where there are a number of variables (weather, vibration, unclean environment, etc.) that simply cannot be controlled.

There are a couple of other concerns associated with using grout cube testing that should be noted. First, the compressive strength results are not known for at least two weeks after the ground nail has been installed. In many instances, the bench may have been excavated down in the meantime and if the break showed that the grout was not achieving an adequate strength, it is not easy to get back up to the level of nails with the inadequate grout.

A second drawback is only gathering two grout cube samples per working day from the project. With several grout mixing stations located throughout the project and a number of batches being mixed at each mixing station, gathering only two cube samples becomes a random attempt at finding a representative batch and assuming that all batches are the same. Additionally, when a cube sample is to be taken, it takes some time to get everything set up in preparation. The person working the grout mixer is given sufficient time to recognize that a test is going to be taken and the person may increase the cement in the grout to achieve better results. It would be very impractical, however, to consider doing more extensive grout cube testing based on the associated cost and time.

2.3 Mud Balance Testing

The mud balance test is essentially a means of measuring the specific gravity of a liquid, in this case cement grout. According to the Post-Tensioning Institute (PTI, 2004) the behavior of neat cement grouts (grouts consisting of only cement and water) without admixtures is well

understood. Wet density testing (mud balance testing) will ensure that grouts with the desired water to cement ratio are mixed. Furthermore, the Post-Tensioning Institute recommends that for performance based specification of a neat cement grout on a permanent structure, the primary means of quality control testing should be conducted via regular specific gravity testing. The measurement of the wet density (specific gravity) of the mixed grout permits the water-cement ratio to be determined, so confirming that the grout has been correctly batched. The Post-Tensioning Institute further concludes that the strength can then be predicted with accuracy, because the strength correlates very well with the water-cement ratio (See Appendix C for PTI correlations).

An actual mud balance device can be seen in Figure 2-4. This is the type of mud balance that was purchased by the project team for this project. The mud balance itself consists of a mud cup at one end of the beam with a fixed counterweight on the other. A sliding-weight rider can then be moved along the graduated scale to provide the specific gravity of the mud within the cup. A bubble level is also included in this particular model to ensure accurate balancing and thus accurate reading. The spec sheet for this mud balance is included in appendix D.



Figure 2-4. Mud balance (from OFI Testing Equipment, Inc.)



Figure 2-5. Measuring the specific gravity of cement grout



Figure 2-6. Leveling the mud balance

Using a mud balance device is fairly straightforward. The field instruction sheet for gathering the specific gravity of the cement grout with a mud balance is shown in Appendix B. A grout sample is removed from the grout mixer and poured into the grout cup of the device. It is important that the reader ensure that the mud cup is filled completely (See Figure 2-5) and wiped clean so that an accurate reading can be taken. Furthermore, the mud balance case should be set on a fairly level surface. The slider is then moved along the beam, until the beam is level. The specific gravity of the cement grout is then read off the graduated scale and recorded (See Figure 2-6). As previously identified, the specific gravity identifies the water-cement ratio of the cement grout, which in turn correlates with the compressive strength.

The compressive strength of the cement grout and the rate of strength gain are dependent upon the ratio (by weight) of mixing water to Portland cement and the method of mixing. Since water and cement are the only items used in the neat cement grout, the relationships between the specific gravity and the water-cement ratio and water-cement ratio and the compressive strength are well defined (see PTI correlations in Appendix C).

The use of the mud balance provides a significant advantage over grout cube testing. The adequacy of the cement grout is known immediately, prior to even grouting the nails, as opposed to having to wait for two weeks to find out the strength results. The mud balance test is conducted very quickly and tests can essentially be run anytime and anywhere that grout mixing is taking place.

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3 Project Data

The data gathered during construction primarily consists of 14 and 28 day compressive strengths from laboratory grout cube testing and the corresponding specific gravities obtained from mud balance testing of the neat cement grout while making the cube samples. There were additional grout cubes tested at 1, 3, and 7 days for some of the samples taken. It should be noted that the full data set does contain several occasions where tests were performed on days other than the target days just listed. The target specific gravity of the cement grout for this project was 1.85, which correlates to an approximate water-cement ratio (by weight) of 0.5 (from PTI correlations, Appendix C). However, specific gravity values measured on the project varied anywhere from 1.70 to 1.91. The project data is contained in its entirety in Appendix E.

The primary purpose of gathering the data was to identify the correlations that may exist between the compressive strength and the specific gravity of the cement grout, using field data. Figure 3-1 shows a plot of the compressive strengths obtained in the laboratory vs. the specific gravities measured in the field for all cube samples broken at day 7. The linear regression trend line shown in the figure is a function of all the break data and includes a corresponding R^2 value of 0.1913. The average values for each specific gravity are also shown on the plot. Similar plots for all cube samples broken at days 14 and 28 are shown in Figure 3-2 and Figure 3-3, respectively. For the day 14 break data the R^2 value is 0.2449 and likewise for the day 28 break data the R^2 value is 0.2100. Each of these R^2 values is low, meaning that there is a lot of vertical spread in the data along the regression lines. However, all three instances show a distinct upward trend with compressive strength increasing in relation to increasing specific gravity. It should be noted that there has not been any data removed from the data set, including some data thought to perhaps be suspect where low compressive strength values were obtained during extremely cold wintertime sampling. Furthermore, the nature of testing cement cube samples provides a pretty decent range of results anyhow. Finally, this data is representative of actual construction data and therefore the cube samples are subject to field error. To obtain a tighter data set, this project would have to be completed in a laboratory setting under tighter controlled conditions.

The trend lines shown for each of the previous three figures are based on the simple linear regression model. An example of an ideal normal, simple linear regression model is shown in Figure 3-4. Ramsey and Schafer (1997) list four different assumptions that the ideal normal,

7 Day Break Data

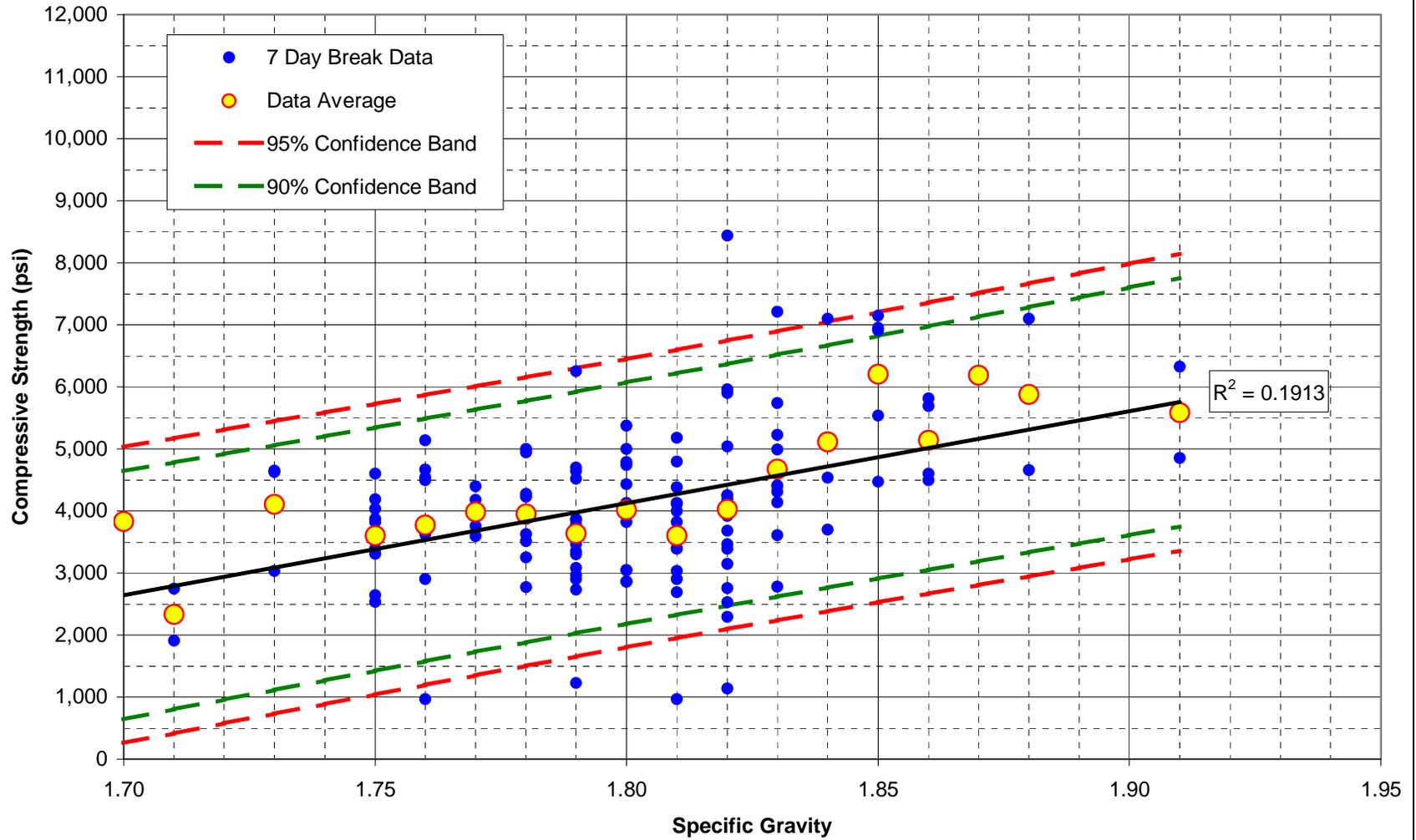


Figure 3-1. Compressive strength vs. specific gravity; break data for day 7

14 Day Break Data

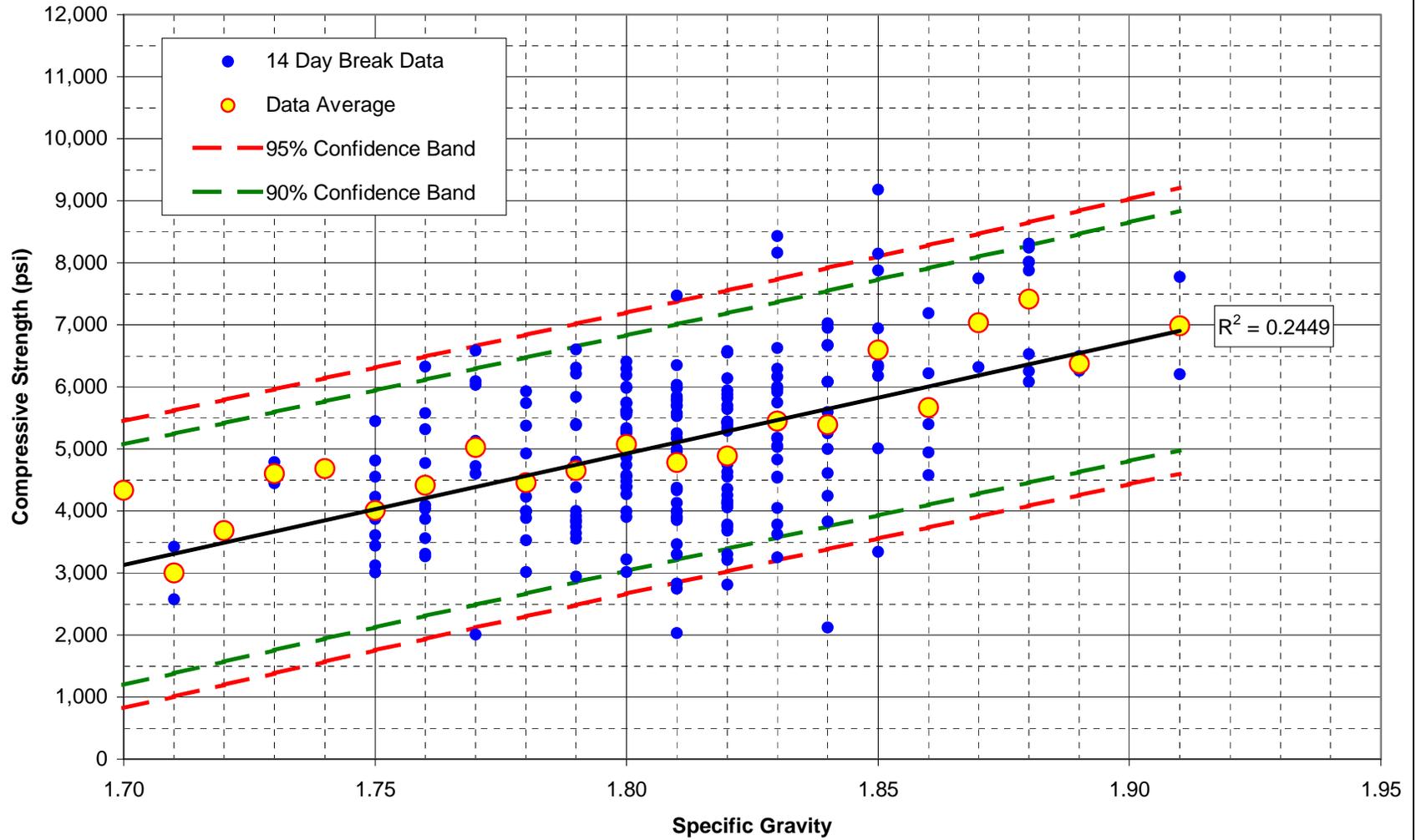


Figure 3-2. Compressive strength vs. specific gravity; break data for day 14

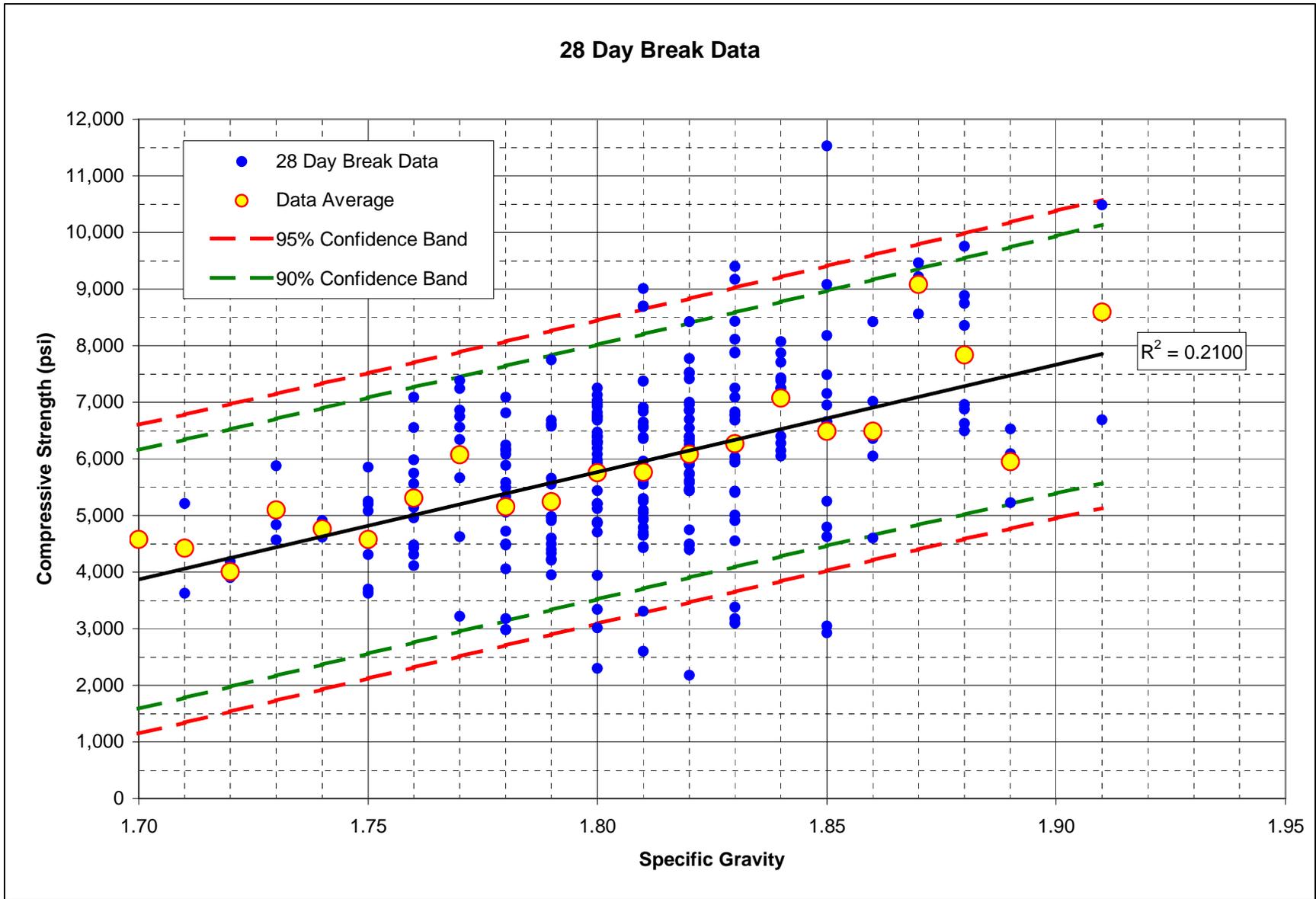


Figure 3-3. Compressive strength vs. specific gravity; break data for day 28

simple linear regression model utilizes. These model assumptions include the following:

1. There is a normally distributed subpopulation of responses for each value of the explanatory variable
2. The means of the subpopulations fall on a straight-line function of the explanatory variable
3. The subpopulation standard deviations are all equal
4. The selection of an observation from any of the subpopulations is independent of the selection of any other observation

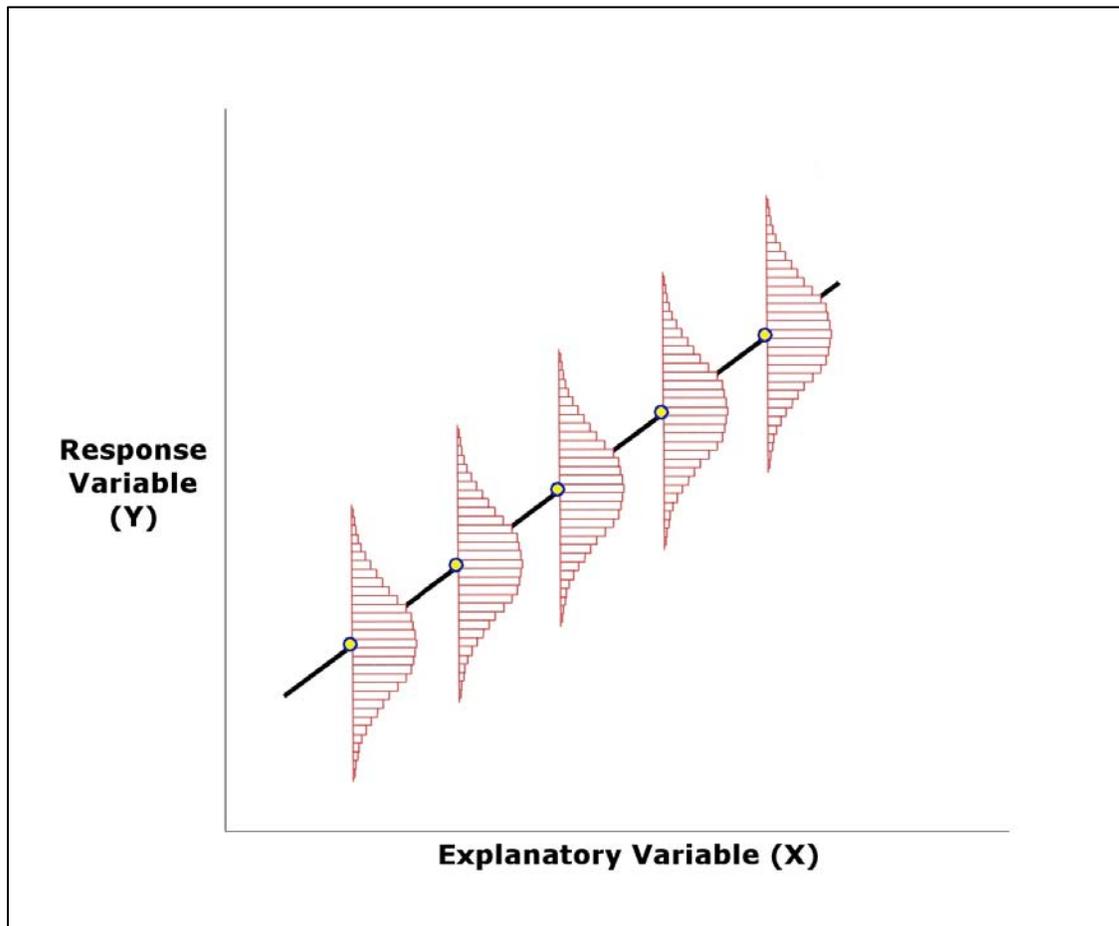


Figure 3-4. The ideal normal, simple linear regression model

Figure 3-4 illustrates that even if there is a large standard deviation of error for each of the subpopulations, the average value for each of the subpopulations falls upon the trend line. Furthermore, the larger the standard deviation of error for each of the subpopulations, the smaller the R^2 value will be for all the data along the trend line. This means that despite the low R^2 values obtained from each of the three data sets, the

linear regression trend lines shown in Figures 3-1 through 3-3 may still be a very good representation of the average value for each specific gravity within the dataset.

It should be noted that for each specific gravity value where many tests were run, the average values fall very near the trend line, as would be expected in a normal, simple linear regression model. An example of this can be seen in Figure 3-3 (the 28-day break data) for specific gravity values of 1.80 through 1.82. For each of these specific gravity values, at least 33 tests were performed. In each instance the average value falls very near the linear regression trend line. The standard deviation for each of these subpopulations is 1218, 1441, and 1166, for the specific gravity values of 1.80 through 1.82, respectively. These standard deviations are all very near each other and would continue to converge upon a single value if more data were obtained, as described in the model assumptions. These three subpopulations are good examples for showing that the dataset seems to follow the normal, simple linear regression model assumptions.

It should also be noted that there are average values for various specific gravity subpopulations that still fall near the linear regression trend line with lesser quantities of data. Likewise, there are several specific gravity subpopulations with only a few data points where the average value does not fall very near the linear regression trend line at all. In both instances, this is simply a function of random sampling with small sample sizes. As illustrated in Figure 3-4, a true linear regression model would have equivalent normal distributions with the average value located on the linear regression trend line for each of the explanatory variables (specific gravity values), if enough data points were gathered for each variable along the X-axis.

With such a large standard deviation for each of the subpopulations, it becomes very useful to quantify the magnitude of that spread for the entire dataset. This can be done by showing confidence bands about the entire dataset. Confidence bands can be generated by calculating the standard error for the dataset, adjusting the standard error with a Scheffé multiplier, and then adding and subtracting this value from the estimated mean (each value along the linear regression model). The Scheffé multiplier utilizes an F-distribution as a means of determining the desired confidence level. Using this method, confidence bands have also been shown in Figures 3-1 through 3-3, at the 90% and 95% level. Although labeled as confidence bands, the bands that have been calculated are technically prediction intervals. This means that the confidence bands that are shown account for the compound uncertainty that arises when

seeking multiple responses at many different values of the explanatory variable (in the range covered by the data, of course). These bands are extremely useful for showing the prediction limits for inclusion of 90% and 95% of future data.

A compilation of the complete dataset is shown in Figure 3-5 (essentially a combination of the data from Figures 3-1 through 3-3). This figure shows that there is a tremendous amount of overlap in the data for the three different days included. It almost seems that with so much overlap, it would be difficult to identify any distinct differences between the datasets for each of the three different days. However, this is not the case. The linear trend lines are also shown in Figure 3-5, for the data from each of the three days. To further clarify the differences between each of the three datasets, Figure 3-6 shows the same linear trend lines, this time however, with only the average data for each of the three datasets shown. The purpose of these figures is to show that the linear trend lines are essentially parallel to each other, thus representing a consistent trend in strength gain between the three different days tested. This illustrates that despite the tremendous overlap in the datasets, there is a distinct difference between the datasets and that the difference is statistically significant. In general, the linear trend lines exhibit two distinct characteristics. The change in the slope intercept, in other words the trend line shifting higher with time, demonstrates that there is an increase in strength gained over time. Furthermore, the increasing slope of the linear trend lines shows that there is also an increase in strength with an increase in cement content within the grout.

The average compressive strength vs. time for several different specific gravity values is shown in Figure 3-7, including the approximated values for a specific gravity of 1.85 from PTI (2004). This figure essentially shows the rate at which strength is obtained in the cement grout as it cures. Furthermore, this figure also shows that there is an increase in compressive strength as the specific gravity increases. There are three additional items that should be noted from this figure. The Contractor on the project was continuously asking how much time had to pass between installation and grouting of the ground nails, in other words when adequate strength would be gained, so that excavation could take place for the subsequent row of nails. This figure indicates that with a cement grout specific gravity of around 1.88, the average compressive strength meets the 4,000 psi minimum value after about 3 days. With the lower specific gravity of around 1.75, the average compressive strength is only about 3,500 psi at day 7, reaches 4,000 psi about day 14, and ends up around 4,500 psi at day 28. This meets the minimum required strength, but the

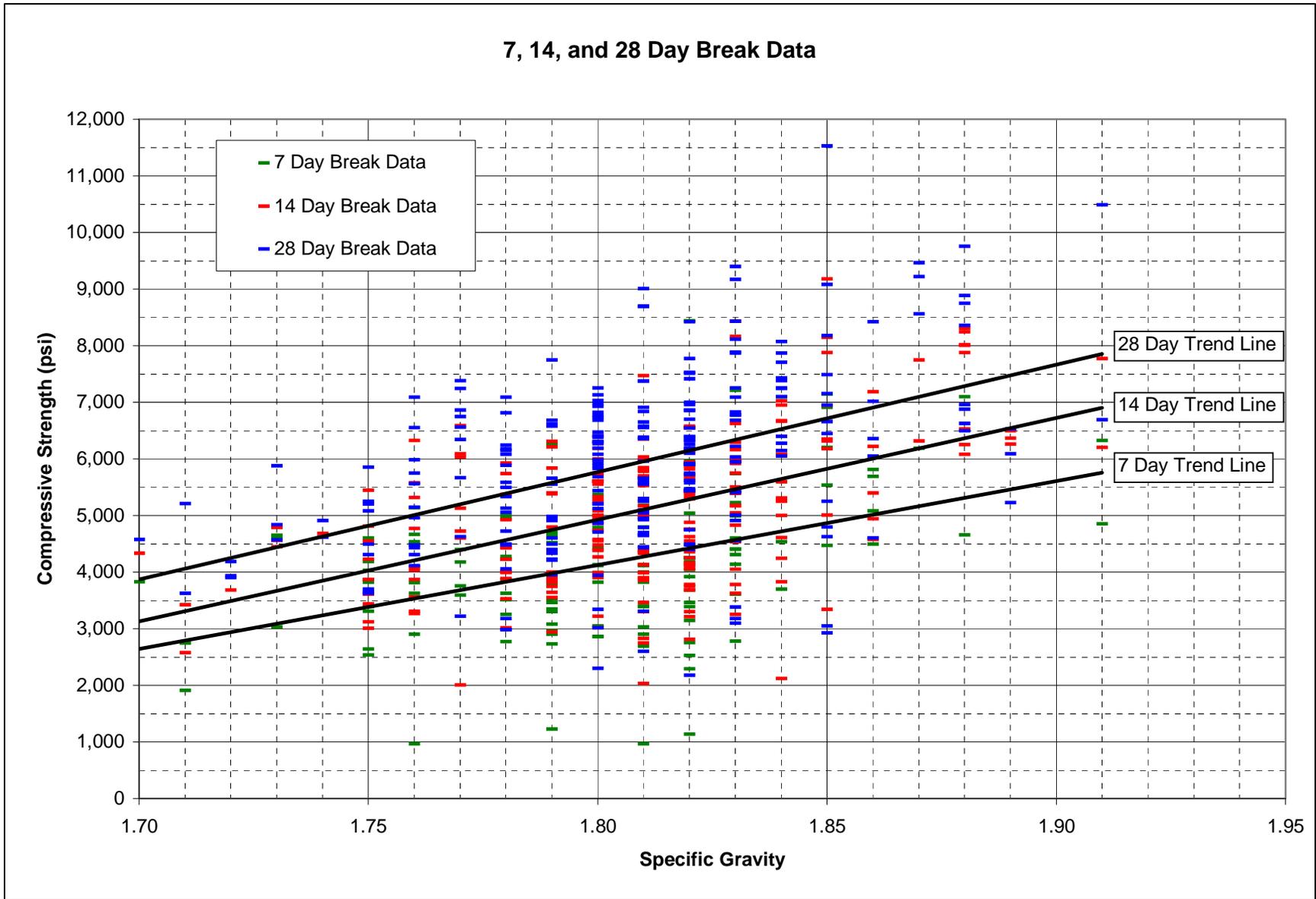


Figure 3-5. Compressive strength vs. specific gravity; days 7, 14, & 28

Average Break Data

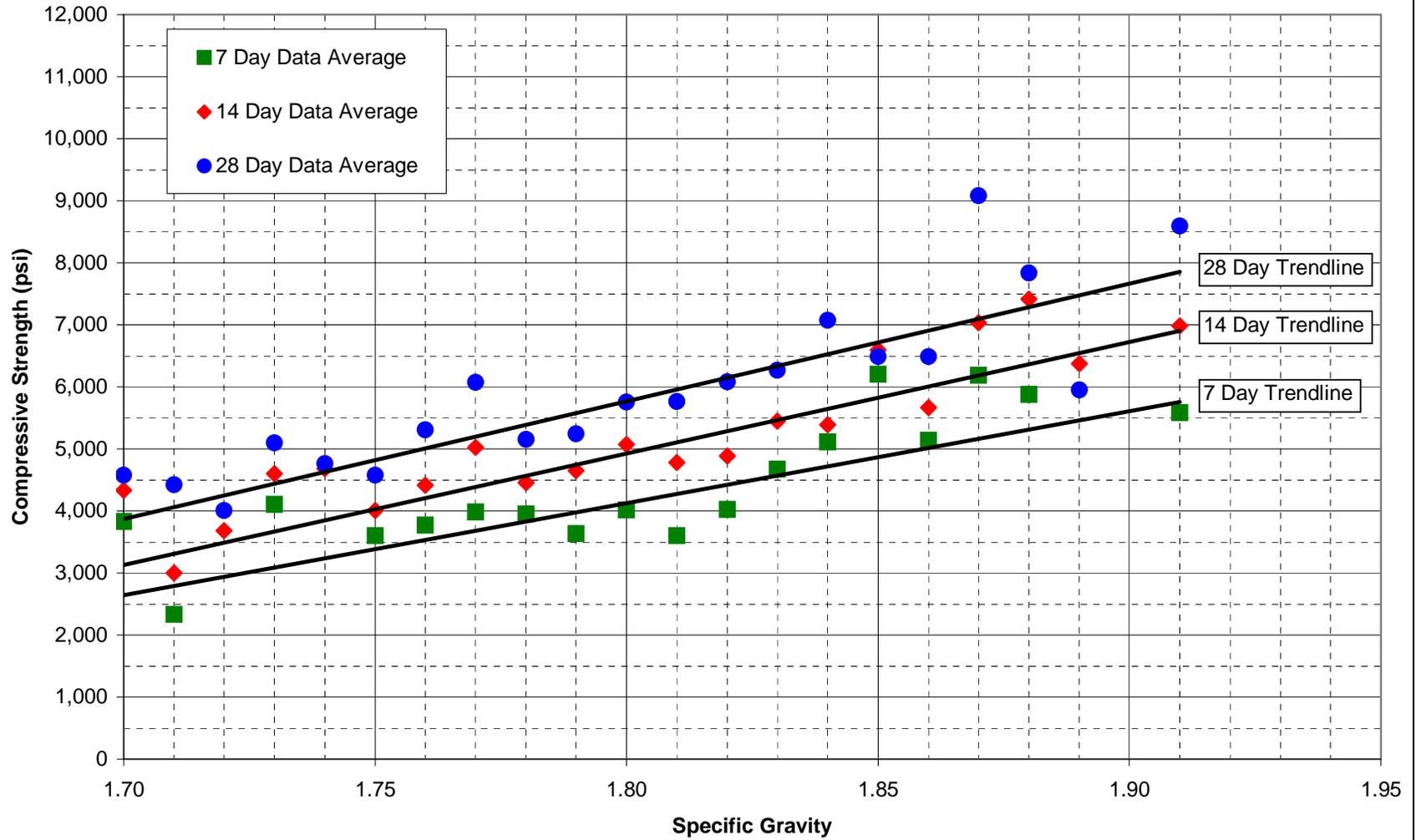


Figure 3-6. Average compressive strength vs. specific gravity; days 7, 14, & 28

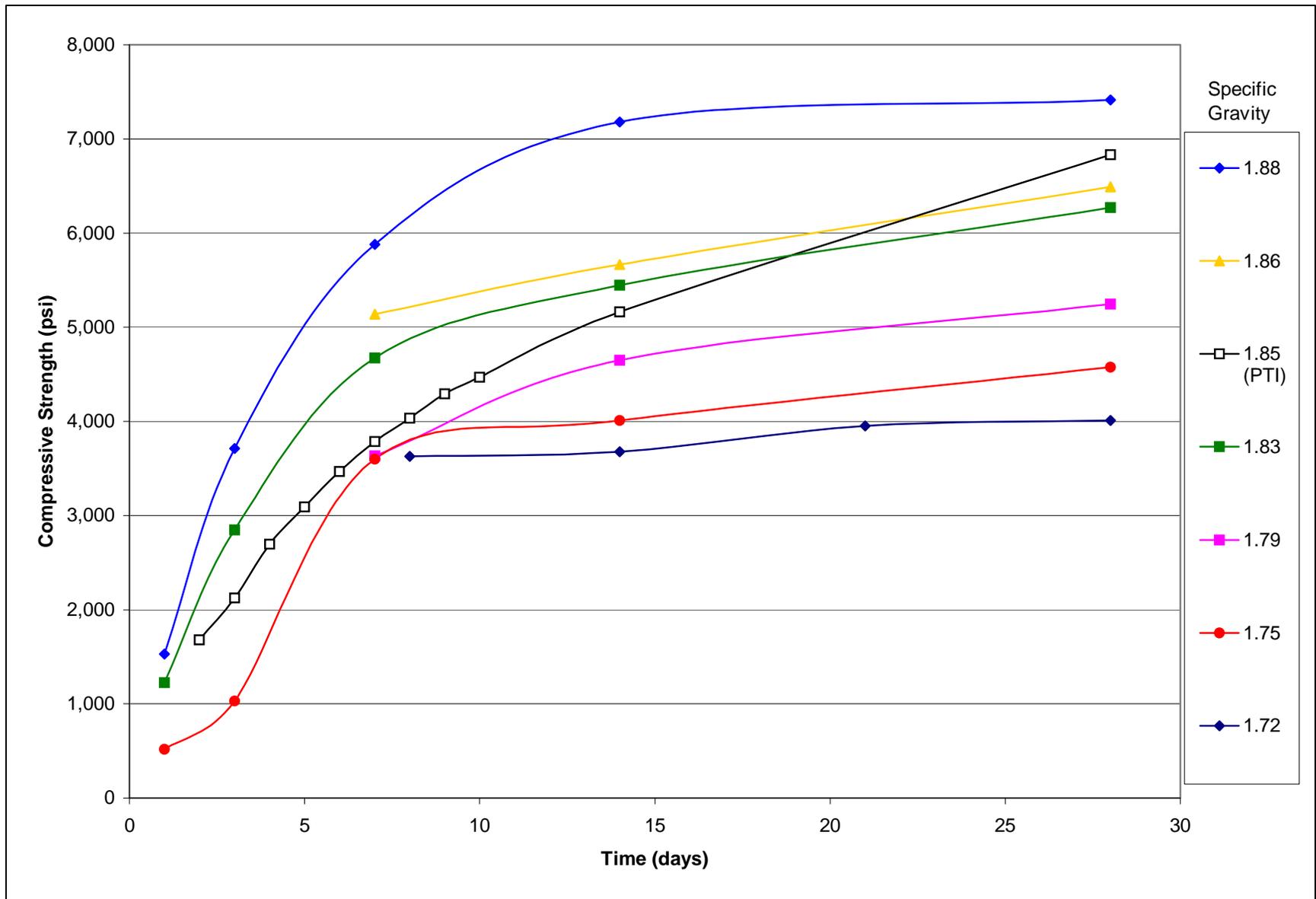


Figure 3-7. Average compressive strength vs. time for varying specific gravities

value shown is an average and does not leave a lot of room for those values that fall below this average to still meet the minimum. This brings up the second point. The lower specific gravities shown (1.72 and 1.75) barely reach or remain slightly beneath the minimum compressive strength value of 4,000 psi within the 28 days, for the average value. Therefore, it may be better to establish a higher minimum specific gravity (perhaps 1.80) to ensure that there is some factor of safety in meeting the minimum compressive strength. Figure 3.7 is alternatively useful for assisting in establishing the time that must pass for average sufficient strength gain prior to excavating below for subsequent rows of nails. Finally, this figure includes approximated values of compressive strength vs. age for a specific gravity of 1.85 from the Post Tensioning Institute (2004). The shape of the PTI curve in general resembles the shapes of the curves obtained in this study. However, the rates of strength gain from the data within this report exhibit more rapid strength gain initially, with a milder strength gain later. An actual statistical comparison of the differences between the PTI curve with the curves from this report is outside the scope of this project. However, the PTI curve is included as a point of reference. The differences do seem to suggest that establishing project specific correlations at the beginning of the project, utilizing the specific type of cement, mixing procedure, etc., may be wise.

It should be noted that specification 02496S 2.1.B.1. required that the grout reach a minimum compressive strength of 1,500 psi within 24 hours of grout placement (UDOT, 2004a). According to the specification and based on Figure 3.7, this requirement is only met for the average compressive strength from a specific gravity of 1.88, or of course higher. In other words, this one requirement seems to govern the minimum specific gravity rather than the 28-day compressive strength. Oddly enough, the specification did not make any provision for ensuring that this part of the specification was met since only 14 and 28 cube samples were required. However, if this requirement is to be utilized for future jobs, it appears that the minimum specific gravity would need to be set sufficiently high to ensure that this requirement is met.

For this project, the minimum acceptable 28-day grout strength was established at 4,000 psi. This value seems to be a fairly standard value accepted in the industry as the minimum ultimate strength for a neat cement grout. The specifications (UDOT, 2004a) required that the nails be replaced if the grout cubes did not meet this tolerance. As can be seen in Figure 3.3, there were a number of tests that did not meet the minimum acceptable value. In the majority of the instances it was determined that the lower values were simply a function of external influences such as adverse temperature conditions or improper

preparation and handling of the grout cubes rather than inadequate grout strength. Re-installation of the nails was therefore not required. However, this brings several key design issues to mind for situations where the grout truly may be inadequate. First, it would be useful to have a better understanding of how the factor of safety of a soil nail wall is affected by the grout strength. In other words, how critical is the grout strength of 4,000 psi in the soil nail design? Second, can an acceptable penalty be established for grout cubes that may exhibit results below the minimum acceptable value rather than simply requiring re-installation of the soil nails? Finally, it may be prudent to ensure that minimum specific gravity thresholds be utilized that adequately account for the large standard error associated with testing grout cubes. These issues should be addressed up front during the design phase of the project. The current project did not specifically address these issues, but it would have been valuable to have had a better understanding of each.

The grout cube dataset contained within this report is an actual sample of the type of results that can be generated while performing quality assurance of a cement grout by means of grout cube testing. It was outside the scope of this research to measure the actual strength of the grout in-situ. It is unknown then, how much error is introduced into the break data by the grout cube testing process and therefore how representative these results actually are of the actual strength of the in-situ grout. It seems that the actual in-situ grout strength would still have some variance with it, but probably not as large of error as that introduced by the grout cube testing process. The reality is however, that the quality assurance of the cement grout is still dependent upon the values obtained from the grout cube testing and tolerances should probably be set for the expected minimum values as opposed to the expected average values. Similar minimum thresholds for using the mud balance as a means of quality assurance should also probably be utilized.

4 Conclusions

4.1 Project Summary

Cement grout is used in the installation of ground nails and tiebacks. In many instances this grout consists of only water and cement. When this is the case, it is identified as a neat cement grout. Where cement and water are the only ingredients used in the grout, the specific gravity of the grout essentially identifies the cement content of the grout. The more cement there is in the grout, the higher the specific gravity.

Specific gravity testing is recommended by the Post Tensioning Institute's (PTI) "Recommendations for Prestressed Rock and Soil Anchors" (2004) for the verification of the grout compressive strength. Specific gravity testing is performed by use of a mud balance. These tests can be performed quickly without interfering with the work, at any time and thus more frequently throughout the day, and provide an immediate indication as to the quality of the mixed grout. This provides the ability to perform regular testing throughout the day where numerous batches of grout are being produced at several locations throughout the project. Furthermore, it provides the ability to immediately test the quality of the grout when there may be a question about the consistency of the grout. In these types of situations, it is not always feasible to simply take grout cubes and wait for them to cure for 28 days to ensure that the grout truly was adequate. When the grout cube results come back later and show that the grout was inadequate, the project has often already progressed further, with excavation work and further nailing beneath the questionable nails having already commenced. It is not practical to bring the fill back in to bench back up and reinstall additional nails. The use of specific gravity testing of the grout provides the ability to eliminate the waiting time to see if the grout was good or not.

On the Provo Canyon Project the use of the mud balance device proved to be very beneficial when the UDOT inspectors and the Contractor had disputes about whether or not the grout was adequate to be installed. Although not enforceable by specification on this project, the Contractor had established a minimum specific gravity for the field employees to follow. This allowed the UDOT inspectors to see that the Contractor was meeting their own standard.

The mix design for this project was water mixed with Portland Cement at a ratio of 0.5 by weight. The corresponding specific gravity of the grout

with a water-cement ratio of 0.5 is approximately 1.85. As the amount of water in the mix increases, the specific gravity of the mix decreases, which in turn would be a decrease in compressive strength. Thus a minimum acceptable specific gravity for a grout batch, where the required strength will still be met, needs to be established. From the project results it appears that a reasonable estimate for meeting the minimum compressive strength would be to establish a minimum allowable specific gravity of 1.80. Figure 3-7 seems to indicate that a specific gravity of 1.72 would barely meet the minimum compressive strength at day 28. If this figure were solely used to determine the minimum allowable specific gravity meeting the minimum compressive strength at day 28, it appears that anything with a specific gravity of 1.72 or larger would be sufficient. Figure 3-3 shows that the average compressive strength at day 28 for a specific gravity of 1.72 is around 4,300 psi, but that the compressive strength is only approximately 1,600 psi and 1,100 for the 90% and 95% confidence intervals, respectively. The same figure shows that by raising the specific gravity to 1.80, the average compressive strength increases to approximately 5,900 psi, but is still only 3,500 psi and 3,000 psi for the 90% and 95% confidence intervals, respectively. To meet the minimum compressive strength of 4,000 psi with the 95% confidence interval, the minimum specific gravity must be raised to 1.85. Using the larger value of 1.85 for the minimum specific gravity would provide some additional factor of safety. However, it should be remembered that the values included in this report represent actual field results, and that a tighter set of data (smaller standard error) could probably be achieved with laboratory test data. For this reason, using a minimum specific gravity of 1.80 would still probably provide adequate results.

It should be noted that in some instances the time to strength gain may become an issue, especially if the work is progressing rapidly and excavation needs to take place for additional rows of nails. When time becomes an issue and it is necessary to have sufficient strength much sooner than 28 days later, the minimum specific gravity can be increased so that the strength is gained quicker. Figure 3.7 indicates that by using a minimum specific gravity of 1.83, the minimum compressive strength is met within about a 5-day period, for the average compressive strength value. Likewise, with a minimum specific gravity of 1.88, the minimum compressive strength is reached even faster for the average value, in about a 3-day period. For very-rapid construction sequencing situations, the minimum specific gravity should be increased to reflect adequate strength gain in the desired duration of time.

As mentioned previously, the project data set included within this report comes from actual field conditions and is therefore inherently subject to

field error. The average values ought to be pretty close where sufficient data was gathered. However, the standard deviation at each point would certainly be tightened up with laboratory conditions. To verify that the cement-grout follows previously established curves or to simply reestablish the correlations, grout cube and mud balance testing could be performed within a laboratory setting at the beginning of the project. This process can be used to assist in establishing the minimum specific gravities to be allowed. Furthermore, rather than completely eliminate the use of grout cubes on the project in lieu of the mud balance test, periodic cube sampling could still take place. This could be done on a cement volume and/or time basis, for example, every 50 tons of cement used and at least once a week, respectively. These periodic cube samples can be used to verify that the cement powder being used is still providing consistent results and that the correlated compressive strengths are still being met. Again, it should be noted that although the average values should remain fairly consistent, if confidence bands are established within the laboratory setting, the field cubes may provide a larger standard error.

One further item that should be noted is the variability in the grout mixing process due to random variables that cannot be controlled such as temperature and humidity. Additional variables that allow for a little better control include the level of mixing, quality of cement, etc... In either case, the curing of the cement-grout is surely affected by these variables. The variability of the strength of the installed grout due to these external variables is not taken into account with specific gravity testing, nor is the difference between cube samples and the installed grout actually verified by cubes samples that are cured in a laboratory. However, the specific gravity testing still provides an immediate indication as to what compressive strength will ultimately be achieved, and the periodic cube samples then provide an indication as to what compressive strength was ultimately achieved.

4.2 In-Situ Testing

The use of maturity testing is an emerging technology that may be of use for in-situ testing of the cement-grout. The use of this technology is currently being used primarily for structural applications using concrete, such as bridge decks and pavements, but there does not appear to be any current information in the literature available for this type of geotechnical application. A maturity number can be established for the concrete mix identifying when the concrete has reached the minimum compressive strength, as a function of time and temperature. The

maturity number is essentially the area beneath the time and temperature curve. These time-temperature records can be used to “verify proper placement and curing processes, prevent concrete defects, ensure structural safety, and to speed construction” (UDOT, 2003). In addition to these benefits, this type of technology can generate “enormous cost and time savings for a project, particularly since concrete work is often such a critical part of a project’s schedule” (UDOT, 2003). Using time-temperature curves and thus maturity numbers for a cement-grout mixture in this type of geotechnical application could very well be conceivable.

The maturity meter is a device that can then be installed within the grout placed in the field, which measures and records the time and in-situ temperature. Potential constructability issues include ensuring proper installation of the meter, protection of any wires during placement of the grout and/or shotcrete, and maintaining accessibility to the wires to retrieve the data. The maturity number is reached, and hence the minimum compressive strength, when an equivalent function of time and temperature has elapsed. While specific gravity and grout cube testing provide an indication of the ultimate compressive strength of the cement-grout, the use of an in-situ method would provide the ability to know what is actually taking place with the installed grout and helps to account for some of those other variables. This type of work was not however performed on the Provo Canyon Construction Project, and further research is therefore needed prior to implementing this method.

4.3 Implementation

The primary means of quality assurance for the neat cement-grout used on the Provo Canyon Construction Project required that two cube samples be taken from the cement-grout mixed throughout the project each working day, and be tested to verify the compressive strength. It is recommended that the specification be modified to allow specific gravity testing as the primary means of quality assurance of a neat cement grout, with periodic cube sampling as a means of verifying the established correlations. Correlations can be established at the beginning of the project by conducting mud balance and grout cube testing within the laboratory. This can be further enhanced by field testing to identify potential differences. Once the correlations have been established, the rate of cube sampling may be reduced to periodic testing at whatever rate is determined to be appropriate. It is believed that this testing procedure will still maintain the compressive strengths required in the current special provision, but ultimately will establish a better overall

product by providing the ability for more frequent testing. Furthermore, the use of the mud balance procedure provides an early estimate of grout strength.

The recommendations for the mud balance testing procedure itself as used with a neat cement-grout specifically consist of the following requirements:

1. Mud balance testing performed at least one time per mixing station per shift daily, with the option that UDOT can require a mud balance test at any time. The Contractor will submit to the Engineer the daily mud balance results. The UDOT inspectors may also carry their own mud balance and therefore test a grout batch at any time themselves.
2. Cube samples will be obtained and tested at a minimum rate of at least 1 test set per 50 tons of cement to verify and maintain the strength correlation. This number may be adjusted to a more frequent testing rate depending upon the size of the project and the usage of each localized grout batch plant. Each test set will consist of 5 cube samples to be tested at 1, 3, 7, 14, and 28 days. For rapid construction situations, additional testing for the 1, 3, and 7 day intervals may be warranted. These grout cube test results will also be submitted to UDOT.
3. The target specific gravity is 1.85 for each grout batch, with a minimum acceptable value of 1.80 for 28 day strength gain. This value should be increased for rapid-construction sequencing requiring adequate strength gain in a shorter period of time.
4. Admixtures added to a cement grout have an affect on the specific gravity of the mixture. Specific gravity testing will continue to be valid ONLY if new strength correlations specific to the new grout mixture are established, AND the ratios of the volume of water, cement, and admixtures remain constant for every batch of grout. Unfortunately, this would require additional supervision ensuring that each and every batch of grout was mixed with the same proportions to continue to utilize specific gravity testing with periodic cube sampling. Without this type of control, there is the potential for variability in the results and the testing must revert back to more frequent cube sampling. The effects of admixtures on the specific gravity when used in very small concentrations may be negligible, yet may still require specific grout mix strength correlations.

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APPENDIX A
Project Special Provisions

02362S Open Cut Ground Reinforcement

02496S Permanent Tiebacks

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SPECIAL PROVISION

***NH-0189(12)14**

SECTION 02362S

OPEN CUT GROUND REINFORCEMENT

PART 1 GENERAL

1.1 SECTION INCLUDES

- A. This work consists of furnishing, and installing soil nails, rock dowels, weeps, filter fabric, drainage mats, reinforced shotcrete, optical survey targets, temporary benchmarks, and accessories at locations shown on the plans or as required to support the open cuts, and performing pullout tests in accordance with the requirements of the contract. Supply all labor, equipment and material necessary to properly complete the installation of soil nails and rock dowels, so as to attain the lengths and spacings identified in the plans.

1.2 RELATED SECTIONS

- A. Section 02075 - Geotextiles
- B. Section 02316 - Roadway Excavation
- C. Section 02342S - Shotcrete for Ground Support
- D. Section 03055 - Portland Cement Concrete
- E. Section 03211 - Reinforcing Steel and Welded Wire

1.3 REFERENCES

- A. ASTM A615 GRADE 60: Structural Steel.
- B. ASTM A722: Uncoated High Strength Steel Bars for Prestressed Concrete.
- C. ASTM C109: Compressive Strength of Hydraulic Cement Mortars.
- D. ASTM C150: Portland Cement.

- E. ASTM A36
- F. ASTM A325
- G. AASHTO M284
- H. ASTM D1785 Schedule 40.

1.4 DEFINITIONS

- A. Ground Reinforcement - is defined as support components that reinforce the in situ ground and increase the ability of the ground to support itself. Related support components include, but are not limited to soil nails, rock dowels, steel washers, nuts, bearing plates, weeps, drainage mats, and structural shotcrete.
- B. Soil Nails – “Soil nails” are defined as a fully encapsulated ground support element installed in soil without tensioning. A soil nail consists of a deformed steel bar or threaded bar installed in a drilled hole or cased drill hole if necessary and encapsulated with cement grout, with centralizers, and end hardware and other accessories as required. “Nail” in this document or the Plans means Soil Nail. Soil Nails are used in estimating quantities for mixed condition of soil and rock.
- C. Rock Dowels – “Rock dowels” are defined as a fully encapsulated ground support element installed in rock without tensioning. A dowel consists of a deformed steel bar or threaded bar installed in a percussion or core drilled hole encapsulated with cement grout, centralizers, end hardware and other accessories as required. “Dowel” in this document or the Plans means Rock Dowel
- D. End Hardware – “End Hardware” is defined as one bearing plate, one flat washer, one beveled washer, and one hexagonal nut assembled on the end of a soil nail or rock dowel.
- E. Pattern Rock Dowels – “Pattern Rock Dowels” are defined as dowels installed in the pattern shown on the plans.
- F. Spot Rock Dowels are dowels installed selectively as required by local ground conditions or as otherwise at the discretion of the Engineer.
- G. Bond Length - Portion of the soil nails or rock dowels, which transfers the tensile force from the nails or dowels to the ground. Also referred to as the fixed length.

- H. Weep is defined as a pipe installed where shown on the plans to collect seeping water. A weep consists of perforated, geotextile fabric wrapped PVC pipe installed in a percussion or core drilled or cased drilled hole in soil if necessary, dry packed into place with mortar and capped prior to shotcreting.
- I. Design Load - Anticipated final maximum effective load in the soil nail or rock dowel after allowance for time dependent losses.
- J. Proof Load - Temporary prestressing load in a soil nail or rock dowel at a force level greater than its design load for testing purposes.
- K. Verification Test – A test that is to verify that installation methods will provide a soil nail or rock dowel capable of achieving the specified design adhesion capacity with a specified factor of safety. It is an incremental test loading and unloading of a soil nail or rock dowel, with recording of the movement of the tendon at each increment. Determine by the verification test whether the soil nail or rock dowel has sufficient load carrying capacity and determine the residual movement or permanent set of the soil nail or rock dowel.
- L. Proof Test - Incremental loading of a soil nail or rock dowel, with recording of the movement of the tendon at each increment up to a maximum test load, typically 150 % of the design adhesion capacity. The test will provide information necessary to evaluate the ability of production soil nails or rock dowels to safely withstand design loads without excessive structural movement or long-term creep over the structure's service life.
- M. Creep test – Performed as part of the verification and proof tests. It is conducted at a specified, constant test load, with movement recorded at specified time intervals.
- N. Alignment Load - A nominal load maintained on a performance-tested soil nail or rock dowel when the nail or dowel is unloaded to keep the testing equipment properly positioned.

1.5 SUBMITTALS

- A. At least 14 days prior to beginning work, submit shop drawings, working drawings, and data prior to proceeding with the work. Resubmittal of the Contract Drawings or any portion thereof is unacceptable.
 - 1. Shop drawings and working drawings to include, but not limited to:
 - Installation procedures for soil nails or rock dowels, weeps, drainage mats, and structural shotcrete, and including details, arrangements, and methods of excavation.

2. Construction sequence, including time after excavation or blasting and distance from ground surface or bench at installation.
 3. Cement grout mix design, including strength versus time curves and data on grout admixtures.
 4. Data on PVC weep pipe and geotextile fabric.
 5. Proposed testing equipment and procedures.
 6. Proposed drilling equipment.
- B. Samples – At least 14 calendar days prior to the purchase of ground reinforcement and weeps, submit the following:
1. Two samples each including end hardware of the size and type to be used shall be obtained from the normal stock of the manufacturer.
 2. Two samples of the PVC weep pipes, perforated, wrapped with geotextile fabric, and capped as proposed for installation.
 3. Applicable manufacturer’s data for the reinforcement, hardware, grout, and weeps including the manufacturer’s recommended installation procedures.
 4. Samples will be returned to the Contractor after approval.
- C. Certificates – Submit the following:
1. Certificate stating that samples for testing are from normal stock.
 2. Certified mill reports of the bars including tensile and yield points, and elongation results.
- D. Installation Record – Submit daily records indicating the quality of grout placed in each hole.
- E. Test Reports – Submit reports of each pull out test within 24 hours after each soil nail or rock dowel test is completed.
- F. Installation Experience – Submit the resume of the person proposed to supervise the installation of ground reinforcement. At least 3 years of experience in the installation of soil nails and rock dowels in similar site and subsurface conditions is required. Provide the names of individuals (with address, affiliation, title, and telephone number) who can attest to the adequacy of the work done on those projects.
- G. Resubmit, as specified above for original submittal, if any procedures, equipment or materials are changed.

1.6 PROJECT CONDITIONS

- A. Safety of all workers and representatives observing the installation and testing of the soil nails and rock dowels is the responsibility of the Contractor.
- B. Limited variations from the numbers, patterns or locations shown on the plans will be permitted to accommodate local conditions, subject to approval of the Engineer.

PART 2 PRODUCTS

2.1 MATERIALS

- A. Provide soil nails and rock dowels that are deformed steel bars, threaded on one end or the equivalent threadbar type. No hollow, self grouting type of drilled bars allowed. Provide each bar with end hardware. Provide length of bars as shown on the plans. All bars shall be epoxy-coated in accordance with Section 03211 - Reinforcing Steel and Welded Wire. Epoxy coating shall be minimum 12 mils meeting AASHTO requirements. Bend test requirements shall be waived.
 - 1. Furnish soil nails or rock dowels full length. Splicing shall not be permitted.
 - 2. Provide steel bearing plates that are square and not less than the dimensions shown on the plans.
 - 3. Provide steel or malleable iron beveled washers. Provide quenched and tempered steel flat washers.
 - 4. Provide hexagonal, heavy duty type nuts. For threaded bars, provide nuts conforming to the bar manufacturer's specifications.
 - 5. Fabricate centralizers for pumped-cement grouted bars of plastic, which support the bar near the center of the hole, and that are ½ inch smaller in outside dimension than the diameter of the hole allowing free flow of grout.
- B. Cement Grout
 - 1. Provide cement grout with a minimum compressive strength of 1,500 psi within 24 hours after placement, and a minimum compressive strength of

4,000 psi within 28 days after placement, nonshrink, and using materials conforming to the following:

- a. Portland cement Type II for all soil and rock formations except in weathered or unweathered Manning Canyon Shale use Type V.
 - b. Potable water
 - c. Admixtures, non-corrosive to bars and subject to Engineer's approval.
2. Payment for cement grout separate from nails/dowels.
- C. Mortar Packing – Use a mortar packing consisting of cement, sand, with sufficient water to provide a workable mixture. Use mortar with the same strength as specified for grout.
- D. Weeps – Use weeps of PVC pipe of size shown on the plans. Use a geotextile fabric wrap that conforms to the requirements of Section 02075.
- E. Centralizers and Spacers - Centralizers and spacers may be made of any material, except wood, that is not deleterious to the steel bars.
- F. Miscellaneous Steel Hardware
1. Steel plates to conform to ASTM A 36.
 2. All bolts, nuts and washers to conform to the tendon manufacturer's specifications.
 3. Develop at least 95 percent of the minimum guaranteed ultimate strength of the soil nail or rock dowel for all anchorage components and couplers.
- G. Shotcrete – As specified in Section 02342S.
- H. Geosynthetics Sock – Use a sock made of polyester fibers capable of confining and holding cement grout in place under pressure. The sock is used to avoid the excessive loss of grout into the voids encountered during drilling for dowels in rock formations.

PART 3 EXECUTION

3.1 QUALITY ASSURANCE

- A. Obtain ground reinforcement materials and testing equipment from established manufacturers who have regularly produced such products for at least 5 years.
- B. Alignment and Tolerances
 - 1. Install soil nails and rock dowels within 3 degrees of the required inclination and horizontal alignment shown on the plans.
 - 2. Install soil nails and rock dowels within 4 inches of the required location in the grid pattern unless otherwise approved or directed by the Engineer.
- C. Manufacturer's Recommendations – Perform installation of ground reinforcement in accordance with the recommendations of the manufacturers. When such recommendations differ markedly from the requirements of this section, such recommendations are subject to the approval of the Engineer.
- D. Replace deficient soil nails and rock dowels or install additional soil nails or rock dowels as determined by the Engineer, at no additional cost to the Department.
- E. Unconfined compressive strength tests results of grout cubes are to be submitted to the Engineer within one week upon completion of the tests.

3.2 PREPARATION

- A. Deliver and prepare steel bars in accordance with approved shop drawings and free of dirt, detrimental rust, or other deleterious substances.
- B. No solvent residue to remain on the steel bars.

3.3 DELIVERY, STORAGE AND HANDLING

- A. Handle and store all ground reinforcement components in such a manner as to avoid corrosion and physical damage.
- B. Damage, such as abrasions, cuts, nicks, welds, weld spatters or heavy corrosion and pitting, will be a cause for rejection of the element. Rejected elements are to be replaced at no cost to the Department in terms of either material replacement or resulting time delays.

3.4 CONSTRUCTION CONTROL

- A. Inspect the steel bars before placement into the borehole. While inserting the steel bar into the hole, protect it from any damage, especially damage to the corrosion protection media.
- B. Insert steel bars freely to the prescribed length in the hole.
- C. Shortening the minimum length of the soil nails and rock dowels specified on the plans will not be allowed.
- D. Place steel bar concentric in the hole.
- E. Use a high speed colloidal mixer with shearing action for mixing grout.
- E. Measure grout pressure at the point of injection. Clean grout gauge mechanism prior to delivery to the site and periodically during the project to prevent clogging.
- F. Mechanically mixed grout components for 5 to 10 minutes to ensure proper dispersion of cement.
- G. Accurately control the water cement ratio.
- H. Commence pumping and injection of the grout immediately after mixing.
- I. Continue grouting until the returning grout escaping from the hole is of the same composition as grout being injected.
- J. Prior to testing soil nails and rock dowels, verify that the grout has sufficient strength to transfer the nail or dowel load to the ground.

3.5 INSTALLATION

- A. Drill holes for soil nails and rock dowels at the station and grid pattern spacing indicated on the plans.
- B. Drill each hole within 4 inches of the stationing and grid pattern as shown on the plans, measured at the collar, and the angle of entry within 3 degrees of horizontal or the angle shown on the plans. Use core, rotary, or percussion drilling to advance the hole. Case drill holes in soils and rock during drilling as needed. Do not use water or other liquids as a cutting or flushing medium, but air may be use for this purpose. Perform the drilling operations in a manner which will not damage the surrounding shotcrete.

- C. Place centralizers at ten foot intervals in soil nail and rock dowel length starting at the end so that no less than 1 inch of grout cover is achieved along the soil nail or rock dowel.
- D. Place centralizers and spacers to allow for the free flow of grout around the steel bars.
- E. Maintain an obstruction-free and open hole for grouting the soil nail and rock dowel. Determine the drilling method and grouting pressures subject to the provisions stated herein and review by the Department. Use existing ground conditions to determine the grouting pressure and grouting method.
- F. Drilling method criteria:
 - 1. Cause minimum disturbance to the surrounding ground and not result in any ground loss.
 - 2. Not result in collapse of the hole during drilling.
 - 3. Maintain the position and inclination of the drilled hole, allow the hole to reach the design depth, and produce the design diameter of the drilled hole.
- G. If the hole tends to collapse during drilling or placement of the soil nail or rock dowel, use temporary casing. Remove temporary casing as grout is placed.
- H. Modify soil nail or rock dowel installation procedures as required to prevent reoccurrence of obstructed or otherwise unsatisfactory holes.
- I. Any damage to existing site conditions by such operations is cause for immediate halting of operations and repair to the satisfaction of the Department.
- J. Inject grout at the lowest point of the nail or dowel hole. Grouting hole without formation of air voids and grout progressively from the bottom to top.
- K. Use gravity-flow or low pressure pumping for the nail and dowel length.
- L. Provide grouting equipment capable of continuous mixing and produce a grout free of lumps. Provide a grout pump with a grout pressure gauge at the nozzle capable of measuring at least 150 psi or twice the actual pressure used.
- M. If grout loss from the drilled hole exceeds three times the volume of the annular space between the drilled hole and steel bar, then discontinue soil nail or rock dowel procedure and remove the rebar and clean hole. Provide 5 psi cement grout above hydrostatic pressure to the drilled hole, redrill the hole 24 hours after the

grout sets, and reinstall the rebar as described herein above. Grout mixes and injection pressures for the pressure grouting require the Department's approval.

- N. The Contractor may, at his option, conduct a water pressure test in the drilled hole prior to grouting. When water loss is greater than 0.25 gallons per minute at a pressure of at least 5 psi above hydrostatic pressure (within the bonded length of the drill hole) measured for at least 10 minutes, then use pressure grout as described above.
- O. The Contractor shall expect to encounter voids within rock formations during drilling. To minimize excessive loss of cement grout, use geosynthetics sock or other grouting alternatives as proposed by the Contractor to plug the voids with cement grout. Undertake due care not to damage the geosynthetics sock by not forcing or twisting the dowel into the hole. Replace any damaged sock at no cost to the Department.
- P. Mix grout continuously in a manner that will produce a pumpable mixture free of lumps and of the proper water cement ratio to attain the specified strength. Inject grout at the lowest point of each drill hole and fill the hole with grout progressively from the bottom to the top. Pump grout through tubes, casing or drill rods. Keep accurate records of the quantity of grout used for each hole.
- Q. Place each steel bar, with centralizers along the bar, in the hole prior to filling the hole with grout. Perform secondary grouting if necessary, to completely fill the hole. Place mortar packing at the shotcrete surface to seal in the grout and to provide an even bearing surface for the bearing plate. Place end hardware as required and hand tighten the nut. Not less than 24 hours after the grout has been placed, tighten the nut with a wrench to bring the bearing plate in tight contact with the shotcrete or rock surface.
- R. Weeps – Perforate wrap, and cap weeps as shown on the plans prior to installation. Minimize damage to geotextile fabric during insertion of the weep pipe into the drill hole. Dry pack the weep in place as shown on the plans. Place temporary caps over the outer pipe end during installation to prevent entry of materials into the weep. Remove the temporary cap after the installation is completed.
- S. Optical Survey Targets and Temporary Bench Marks – Install optical survey targets and temporary bench marks as shown on the plans or as directed by the Engineer.

3.6 PULL OUT TESTING

- A. Pullout tests include both verification tests and proof test. Perform pullout tests in the presence of the Engineer. Provide not less than 24 hour notice to the Engineer prior to the start of a test.
- B. Submit copies of all test results and graphs to the department as each test is completed.
- C. Provide the equipment to be used for performing pullout tests and maintain it in good working conditions. Tension steel bars by direct pull with a hollow ram hydraulic jack or a model recommended by the manufacturer, so mounted as to prevent bending of the soil nail or rock dowel. Wait until cement grout has set prior to commencing tensioning of the steel bar.
- D. Failure of a soil nail or a rock dowel consists of not meeting the minimum forces shown in Section 3.8 Verification and Proof Testing.
- E. Provide pullout test equipment consisting of a suitably sized hollow ram jack; an adjustable bearing truss for aligning the direction of pull with the centerline of the bar, an extension bar for attaching the jack to the soil nail or rock dowel; a hydraulic pump with a gauge calibrated to read readily in pounds per square inch for the ram being used; a dial gauge which reads in increments of 0.001 inch over a range of 2 inches; a magnetic or independent dial gauge mounting; a load cell and a readout; and all other necessary accessories.
- F. Use load cell during all pullout tests. Select type and brand of load cells compatible with testing per specifications.
- G. Prior to performing pullout tests, calibrate the hydraulic pump gauge with the load cell while connected to the jack, by a uniaxial testing machine. Recalibrate the hydraulic pump gauge at subsequent times as directed by the Engineer during the period of construction.
- H. To prevent delays in the work, keep spare parts in stock for the testing equipment.
- I. Make pullout tests not less than one or more than 7 days after installation of the ground reinforcement. If pullout tests are not made within the time frame specified above, the results thereof will be rejected and an additional pullout test shall be performed in accordance with specification requirements at no cost to the Department.
- J. Spread the reaction load for pullout testing over the surface of the shotcrete or rock. Use techniques where pressure from the reaction system against the shotcrete or rock does not crack or other damage the shotcrete or rock.

- K. Verify grout strength by the pullout tests.

3.7 EXCAVATION

- A. Take care during excavation to prevent disturbing the natural foundation material behind the face of excavation.
- B. During initial mass grading, do not excavate the full wall height to the wall alignment as shown on the plans, but maintain a working berm of native material in front of the wall to serve as work bench for the drill equipment. Extend the working berm out from the wall a minimum distance of 15 feet and cut down from that point at a slope as approved by the Engineer.
- C. Incorporate a working berm constructed from the top down in a staged lift sequence when excavating to the final wall alignment for the full wall height as approved by the Engineer. Do not excavate the ground level in front of the wall face more than 4 feet below the level of the row of soil nails or rock dowels and weeps to be installed in that same lift. Cover the face of the open cut, maximum height 8 foot, if it remains in place for over 24 hours, with a layer of not less than 4-inch thick shotcrete to mitigate potential sloughing off of the temporary cut slope.
- D. Advance subsequent excavation lifts until ground reinforcement installation (including bearing plate and nut), structural shotcrete placement, and testing for the preceding lifts are complete and acceptable to the Engineer. Prior to advancing the excavation, reach 100 and 50 percent of the required minimum 28-day compressive strength for nail grout and shotcrete on the preceding lift, respectively.
- E. Responsibility of cobbles and boulders which are encountered at the soil face during excavation and which protrude from the soil face are that of the Contractor. Construct the structural shotcrete and the finish sculpted shotcrete facing to the specified minimum thickness and to the line and grade indicated on the plans. Determine the removal of face protrusions necessary to accomplish this construction. Notify the Engineer of the proposed method for mitigation of face protrusions at least 24 hours prior to initiation of the work. If the removal of face protrusions results in voids beyond the finish face line, the Contractor determines the appropriate method of backfilling and submits to the Engineer such method(s) at least 24 hours prior to initiating the work. UDOT pays for backfilling under other items of work or as extra work as determined by the Engineer.

3.8 VERIFICATION AND PROOF TESTING

- A. Verification Testing

1. Perform verification testing in sacrificial nails and dowels prior to installation of production soil nails and rock dowels to verify the Contractor's installation methods, soil/rock conditions, nail pullout capacity, and design assumptions. Perform two verification tests at every 200 foot long section of the soil nail/rock dowel walls, unless otherwise directed by the Engineer. The soil nails and rock dowels used for the verification tests are sacrificial and not to be incorporated as production soil nails and rock dowels. Payment for additional verification tests required due to differing site conditions, as determined by the Engineer, will be paid as per the contract unit price.
2. Develop the details of the verification testing arrangement, including the method of distributing test load pressures to the excavation surface (reaction frame), test nail/dowel bar size, grouted hole diameter and reaction plate dimensioning and submit to the Engineer for approval. Construct test nails/dowels using the same equipment, methods, and hole diameter as planned for the production nails/dowels. Changes in the drilling or installation method will require resubmittal, as per subsection 1.5.G above, and additional verification testing, as determined by the Engineer, at no additional cost to the Department.
3. Test nails/dowels have bonded lengths shown on the plans. Prior to testing, only grout the bonded length of the test nail/dowels. Make the unbonded length of the test nail/dowels at least 5 feet. The bonded length of the test nail/dowel to be determined by the Engineer based on the bar size grade and size such that the allowable bar structural load is not exceeded, but not be less than the length shown on the plans. The allowable bar structural load during testing not to be greater than 80 percent of the ultimate strength of the steel for Grade 150 bars.
4. The verification test bonded length L_{BV} not to exceed the test allowable bar structural load divided by 2 times the design adhesive value. Use the following equation for determining the test nail bond length to avoid structurally overstressing the verification nail bar size:

$$L_{BV} \leq C f_y A_s / 2 A_D$$

Where L_{BV} = Maximum Verification Test Nail Bond Length (ft)

f_y = Bar Yield Stress (ksi)

A_s = Bar Area (square inches)

A_D = Design Adhesion (kips/foot)

C = 0.8 for Grade 150 bars.

Determine the design test load during testing by the following equation:

$$DTL = L_B \times A_D$$

Where L_B = As-built bonded test length (ft)

A_D = Design Adhesion (specified as kips/ft)

5. Incrementally load verification test nails to twice the design test load (DTL) followed by unloading in accordance with the following schedule. Record the soil nail movements at each load and unload increment.

Loading

<u>Load</u>	<u>Hold Time</u>
AL (.05 DTL max)	1 minute
0.25 DTL	10 minutes
0.50 DTL	10 minutes
0.75 DTL	10 minutes
1.00 DTL	10 minutes
1.25 DTL	10 minutes
1.50 DTL	60 minutes
1.75 DTL	10 minutes
2.00 DTL	10 minutes

Unloading

<u>Load</u>	<u>Hold Time</u>
1.75 DTL	Until Stable
1.50 DTL	Until Stable
1.25 DTL	Until Stable
1.00 DTL	Until Stable
0.75 DTL	Until Stable
0.50 DTL	Until Stable
0.25 DTL	Until Stable
AL	Until Stable

6. The alignment load (AL) to be the minimum load required to align the testing apparatus and not to exceed 5 percent of the design test load (DTL). Set dial gauges at “zero” after applying the alignment load.

7. Hold each load increment for at least 10 minutes. Monitor the verification test nail for creep at the 1.5 DTL load increment. Measure nail movements during the creep portion of the test and record at 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes. Maintain the load during the creep test within 2 percent of the intended load by use of the load cell. Unload the nail in increments of 25% of the DTL with movements recorded at each unload increment. Hold each unload increment only for a sufficient time to allow stabilization of the movement reading.
8. The Engineer evaluates the results of each verification test. Reject installation methods which do not satisfy the nail testing requirements. The Contractor proposes alternative methods and installs replacement verification test nails. Install replacement test nails/dowels and test at no additional cost to the Department.

B. Proof Testing

1. Perform proof tests on 10 percent of the total production nails/dowels installed. The Engineer will initially designate at random one soil nail and one rock dowel out of each ten installed for proof testing. If the test fails due to Contractor's faulty installation methods, two (2) additional installations from the same group will be designated by the Engineer for proof testing. If either of these fail, all remaining ten installations presented by the test will be rejected by the Engineer and shall be tested. Replace installations that fail the proof test due to the Contractor's faulty installation methods at no cost to the Department.
2. The Engineer may reduce the initial frequency of the proof tests (i.e., 10% of the total production nails/dowels) upon successful demonstration of consistent compliance with specified requirements. The Engineer may then randomly designate one soil nail or one rock dowel out of each twenty installed for proof testing (i.e., 5% of the total production nails/dowels). Should one failure occur due to Contractor's faulty installation methods, the Engineer may require the initial frequency of testing to be resumed. Replace installations that fail the proof test due to the Contractor's faulty installation methods at no cost to the Department.
3. The Engineer may require that the Contractor replaces some or all of the installed production nails between the failed proof test nail/dowel and the adjacent passing proof test nail. Alternatively, the Engineer may require proof testing of additionally installed proof test nails/dowels to verify that the adjacent previously installed production nails have sufficient load

carrying capacity. If such a test fails, the Engineer's modifications may include installation of additional test and/or production nails/dowels (i.e., decreased nail/dowel spacing from that shown on the plans), installing longer production nails, increasing the drillhole diameter, or modifying the installation methods. Costs due to additional proof tests or installation of additional or modified nails/dowels as a result of proof test nail failure(s) at no additional cost to the Department, unless determined by the Engineer to be due to causes beyond the Contractor's control.

4. Proof test nails/dowels have both bonded and unbonded lengths. Prior to testing, grout only the bonded length of the test nail/dowel. The Engineer verifies the bonded and unbonded lengths of each test nail/dowel. The unbonded length of the test nail/dowel is at least 5 feet unless otherwise approved by the Engineer. The Engineer verifies the bonded length of the test nail/dowel based on the bar grade and size such that the allowable bar structural load is not exceeded, but be not less than the length shown on the plans. The allowable bar structural load during testing is not be greater than 80 percent of the ultimate strength of the steel for Grade 150 bars.
5. The proof test bonded length L_{BP} not to exceed the test allowable bar structural load divided by 1.5 times the design adhesive value. Use the following equation for determining the test nail/dowel bond length to avoid structurally overstressing the production nail/dowel bar size:

$$L_{BP} \leq C f_y A_s / 1.5 A_D$$

Where L_{BP} = Maximum Proof Test Nail Bond Length (ft)

f_y = Bar Yield Stress (ksi)

A_s = Bar Area (square inches)

A_D = Design Adhesion (kips/foot)

C = 0.8 for Grade 150 bars.

6. Incrementally load proof test nails to 150% of the design test load (DTL). Determine the design test load as for verification test nails. Measure the nail movement at each load and record in the presence of the Engineer in the same manner as for verification tests. Monitor the load by a pressure gauge with a sensitivity and range meeting the requirements of pressure gauges used for verification test nails/dowels. At load increments other than maximum test load, hold the load long enough

to obtain a stable reading. Incremental load for proof tests in accordance with the following schedule:

AL (.05 DTL max)
0.25 DTL
0.50 DTL
0.75 DTL
1.00 DTL
1.25 DTL
1.50 DTL (Maximum test load)

AL = Nail Alignment Load
DTL = Nail Design Test Load

The alignment load (AL) is the minimum load required to align the testing apparatus and not to exceed 5 percent of the design test load (DTL). Set the dial gauges at “zero” after the alignment load has been applied.

7. Maintain all load increments within 5 percent of the intended load. Depending on performance, perform either 10 minute or 60 minute creep tests at the maximum test load (1.50 DTL). Start the creep period as soon as the maximum test load is applied and measure the nail/dowel movement and record at 1 minute, 2, 3, 5, 6, and 10 minutes. Where the nail movement between 1 minute and 10 minutes exceeds 0.04 inch, maintain the maximum test load an additional 50 minutes and record movements at 20 minutes, 30, 50, and 60 minutes.

C. Test Nail/Dowel Acceptance Criteria – Consider a test nail acceptable when:

1. For verification tests, a creep rate less than 0.08 inch per log cycle of time between the 6 and 60 minute readings is observed during creep testing and the rate is linear or decreasing throughout the creep test load hold period.
2. For proof tests: (a) a total creep movement of less than 0.04 inch is observed between the 1 and 10 minute readings or a total creep movement of less than 0.08 inch is observed between the 6 and 60 minute readings and (b) the creep rate is linear or decreasing throughout the creep test load hold period.
3. The total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the test nail/dowel unbonded length.
4. A pullout failure does not occur at the maximum test load. Pullout failure is defined as the load at which attempts to increase the test load simply result in continued pullout movement of the test nail/dowel. Record the pullout failure load as part of the test data.

D. Acceptance as Production Nails/Dowels

1. At the Contractor's option, successful proof test nails/dowels meeting the above test acceptance criteria may be incorporated as production nails/dowels, provided that (1) the unbonded test length of the nail hole has not collapsed during testing, (2) the minimum required hole diameter has been maintained, (3) the specified corrosion protection is provided, and (4) the test nail/dowel length is equal to or greater than the scheduled production nail/dowel length. Complete test nails/dowels meeting these requirements by satisfactorily grouting the unbonded test length. Maintaining the temporary unbonded test length for subsequent grouting is the Contractor's responsibility. If the unbonded test length of production proof test nails/dowels cannot satisfactorily be grouted subsequent to testing, the proof test soil nail/rock dowel becomes sacrificial and the Contractor replaces the proof test soil nail/rock dowel with a production nail/dowel installed to the satisfaction of the Engineer and at no additional cost.

3.9 TESTING OF GROUT CUBES

- A. Provide unconfined compressive strength test results from two grout cube samples taken from the grout mix during each working day. Test the unconfined compressive strength of one of these samples at 14 days; and test the other sample at 28 days.

3.10 RECORD OF WORK

- A. Record accurately and completely documentation of all work done. Include drilling of the nail/dowel hole, water testing, quality, strength, and volume of cement grout, testing and stressing of soil nails/rock dowels, equipment used for testing and their calibration data, type of steel bars, materials and procedures used for corrosion protection of the bars. Submit the records to the Engineer daily.

END OF SECTION

June 1, 2004

SPECIAL PROVISION

***NH-0189(12)14**

SECTION 02496S

PERMANENT TIEBACKS

PART 1 GENERAL

1.1 SECTION INCLUDES

This section applies to the construction, installation and testing of permanent tiebacks. Permanent tiebacks consist of drilling holes in foundation material, inserting grouted steel bars or strands and anchorage assemblies conforming to the details on the plans and these specifications.

1.2 SCOPE

The Contractor supplies all labor, equipment, material, and incidentals necessary to construct the tiebacks as shown on the plans and as specified in these specifications. The work includes:

- A. Providing adequate bond length and stressing length for tiebacks in order to satisfy requirements specified herein and shown on the plans.
- B. Providing a double corrosion protection system for tiebacks as shown on the plans and as specified herein.
- C. Providing labor, materials and equipment required for fabricating and installing tiebacks as shown on the plans and specified herein to achieve the load capacities specified on the plans.
- D. Prestressing and testing all tiebacks as specified herein.

1.3 REFERENCES

<u>Sponsor</u>	<u>Number</u>	<u>Subject</u>
ASTM	A36	Structural Steel

ASTM	A53	Pipe, Steel, Black and Hot-Dipped, Zinc Coated Welded and Seamless
ASTM	A416	Uncoated Seven-Wire Stress Relieved Steel Strand for Prestressed Concrete
ASTM	A722	Uncoated High Strength Steel Bars for Prestressed Concrete
ASTM	C109	Compressive Strength of Hydraulic Cement Mortars
ASTM	C150	Portland Cement
ASTM	D1248	Polyethylene Plastics Extrusion Materials for Wire and Cable
ASTM	D1784	Rigid Poly Vinyl Chloride (PVC) Compounds and Chlorinated Poly Vinyl Chloride (CPVC) Compounds
PTI		Post Tensioning Manual, 5th Edition
PTI		Guide Specification Recommendations for Prestressed Rock and Soil Anchors, 3 rd Edition
PTI		Specification for Unbonded Single Strand Tendons, 2nd Edition
PTI		Guide Specification for Grouting of Post-tensioned Structures

1.4 DEFINITIONS

- A. Tieback - A high strength steel tendon that is fitted with an anchorage at the exposed end and a grouted anchor that permits force transfer to the ground on the other end.
- B. Permanent Tieback - A tieback used for permanent support as shown on the plans, and which is provided with double corrosion protection.
- C. Anchorage - Portion of the tieback, including anchor head, anchor plate and protective cap, which is used to transfer load from the supporting concrete to the permanent tieback.
- D. Bonded Length - Portion of tieback which transfers the tensile force from the tieback to the ground. Also referred to as the fixed length.

- E. Stressing Length - Portion of the tieback that is free to elongate elastically during stressing. Also referred to as the free length or unbonded length.
- F. Smooth Sheathing - Enclosure around prestressing steel in the tieback stressing length that prevents permanent bond between the prestressing steel and the surrounding grout, and that provides corrosion protection.
- G. Corrugated Sheathing - Enclosure around prestressing steel in the bond length, and where required, in the stressing length of the tieback that provides corrosion protection and that transfers applied tension load to the surrounding ground.
- H. Design Load - Anticipated final maximum effective tension load in the tieback after allowance for time dependent tension losses.
- I. Proof Load - Temporary prestressing test load in a tieback at a force level greater than its design load.
- J. Performance Test - Incremental test loading and unloading of a tieback, with recording of the movement of the tendon at each increment. Use the performance test to determine if the tieback has sufficient load carrying capacity, if the stressing length has actually been established, and the amount of residual movement or permanent set of the tieback.
- K. Proof Test - Incremental loading of a tieback, with recording of the movement of the tendon at each load increment. The proof test is used to verify the load carrying capacity of each tieback that is not performance tested, as well as a means of preloading the tieback.
- L. Transfer Load - Prestressing force in a tieback after proof loading, immediately after the force has been transferred from the jack to the anchorage. Also referred to as the lock-off load.
- M. Lift-Off Test - A check performed immediately after transferring the load to the anchorage. This test determines the deviation of the actual transfer load from the desired transfer load.
- N. Alignment Load - A nominal load maintained on a performance-tested tieback when the tieback is unloaded. The alignment load helps to keep the testing equipment properly positioned.

1.5 SUBMITTALS

- A. Submit shop drawings and working drawings 30 days prior to proceeding with the work.
1. Shop drawings and working drawings include, but not limited to:
 - a. A tieback schedule that provides the following items for each tieback:
 - (1) Tieback number and location;
 - (2) Design load;
 - (3) Tieback type and size;
 - (4) Minimum bonded length;
 - (5) Minimum stressing length.
 - b. A drawing of the tieback system and corrosion protection that includes:
 - (1) Centralizers and their location;
 - (2) Anchorage;
 - (3) Anchorage corrosion protection system.
 - c. Submit design calculations that include a determination of the bonded length for each tieback.
 - d. Proposed sequence for tieback installation.
 2. Submit the following information to the Engineer for review at least 30 days prior to commencement of tieback installation:
 - a. Stressing bar or strand manufacturer's mill test reports for the tiebacks.
 - b. Applicable literature from cement grout suppliers that provides details about setting times as a function of temperature, strength gain with time, and recommended storage, mixing and placement procedures.
 - c. Applicable manufacturer certification and/or literature for anchorage fittings and accessories.
 - d. Certification that tieback materials and testing equipment will be obtained from established manufacturers who have regularly produced such products for at least five years.
 - e. Detailed description of proposed procedures, including specific brands and models of equipment that will be used to drill, place, grout, tension and lock off tiebacks.
 - f. Detailed description of proposed procedures and applicable manufacturer's literature for the equipment that will be used to test tiebacks, including but not limited to, the following:
 - (1) Diagrams that show arrangement of testing equipment relative to the tieback and anchorage hardware.
 - (2) The method for locking-off the required transfer load.

- (3) Calibration data for the system of loading jacks and pressure gauges.
 - (4) The proposed apparatus for monitoring elongation during testing of tiebacks.
 3. During grouting operations, the contractor records the following data for each batch of grout and submits to the Engineer:
 - a. Type of mixer and grout pump.
 - b. Type of cement.
 - c. Water-cement ratio.
 - d. Types of additives and their concentrations in mix.
 - e. Grout injection pressure.
 - f. Test sample strength.
 - g. Volume of grout placed.
 4. The Contractor also submits a report to the Engineer within 10 days after completion of installation of each row of tiebacks. The report contains as-built drawings showing the locations, inclinations and horizontal alignments of the tiebacks, total tieback lengths, stressing lengths, and bonded lengths. This report is in addition to the individual tieback reports that are required by Paragraph 3.4.B of this specification.
 5. A Manufacturers Material Safety Data Sheet (MSDS) must be submitted for each product, when applicable.
- B. Submit records that document at least five years experience in tieback fabrication, installation, and testing and a minimum of five previous tieback jobs of similar or greater scope as qualification for this project. At least three of these five projects used the approved or similar tieback systems to stabilize landslides. Submit the names of individuals (and their address, affiliation, title, and telephone number) who can attest to the adequacy of the work performed on those projects.
- C. Submit the name of the laboratory testing firm with an established record of 5 years of experience in performing the specific tests that are required for the tieback installation as specified herein.
- D. Submit the name of a land surveyor licensed in the State of Utah to lay out the work and verify the locations and inclinations of the tiebacks.

1.6 QUALITY ASSURANCE

- A. If not otherwise stated in this specification, work performed for this project is performed in accordance with the most recent edition of "Recommendations for Prestressed Rock and Soil Anchors", Post Tensioning Institute (PTI), Phoenix, AZ.
- B. Do not install tiebacks until the Engineer approves the shop drawings, working drawings, submittals and the sequence of installation for all tiebacks.
- C. Alignment and Tolerance
 - 1. Install tiebacks to within 2 degrees of the required inclination and horizontal alignment shown on the plans.
 - 2. Install tiebacks within 12 inches of the required locations, unless otherwise approved or directed by the Engineer
- D. Collection of grout cube samples for testing
 - 1. At least two cube samples are to be collected from each batch of grout for testing.
 - 2. The Contractor is responsible for sample preservation, storage, and transporting to an approved testing laboratory. All grout cube samples for testing are selected by the Engineer. Laboratory tests include unit weight and unconfined compressive strength measurements on the grout cube samples tested in accordance with ASTM Designation C109.
 - 3. Grout to have a minimum strength of 4,000 psi or as otherwise stated on the plans.

1.7 DELIVERY, STORAGE, AND HANDLING

- A. Handle and store tiebacks and their components in a manner that prevents corrosion and physical damage to these items.
- B. Damage, such as abrasions, cuts, nicks, welds, weld spatters or heavy corrosion and pitting, are a cause for rejection of tieback elements. Contractor replaces rejected elements at no cost to the Department for either material replacement or resulting time delays.

1.8 PROJECT CONDITIONS

- A. The contractor is fully responsible for the safety of workers and for representatives of the Engineer who observe installation and testing of tiebacks.

- B. Limited variations from numbers, patterns or locations of tiebacks as shown on the plans is permitted to accommodate local conditions. The Engineer approves such variations. Additional tiebacks other than those shown on the plans may be required in localized areas at the discretion of the Engineer.
- C. The Contractor is responsible to provide tiebacks that satisfy all tieback test acceptance criteria, including required load capacity. Replace deficient tiebacks or install additional tiebacks as determined by the Engineer, at no additional cost to the Department.

PART 2 - PRODUCTS

2.1 MATERIALS

- A. Provide steel tendons for tiebacks of low relaxation, high tensile, seven-wire strand that satisfies requirements of ASTM A416 Grade 270 or continuously threaded Grade 150 steel bars that satisfy the requirements of ASTM A722, Type V or Type II where approved.
 - 1. The sheath that encapsulates the tendons is capable of safely withstanding deformations that occur during transportation, installation, stressing, testing, and load transfer to the tendons.
 - 2. The Contractor determines the bonded length necessary to meet acceptance criteria specified herein.
 - 3. Stressing length of tieback tendons not less than the lengths shown on the plans.
 - 4. For tiebacks with steel strand tendons, each tieback is comprised of an adequate number of strands, such that, the maximum test load does not exceed 80% of the ultimate tensile strength of the prestressing steel.
 - 5. For tiebacks with steel strand tendons, the stressing length consists of strands that are individually coated with corrosion inhibiting grease and encapsulated by a smooth high density polyethylene sheath. The sheath is hot melt extruded onto the strand ensuring that all spaces between the sheath and the strand are filled with corrosion inhibiting grease. The minimum wall thickness of the sheath 50 mils. The polyethylene sheath conforms to ASTM D1248. The stressing length also includes a sleeve around the entire tendon for added corrosion protection.
 - 6. For tiebacks with steel strand tendons, a corrugated plastic PVC sleeve encapsulates the entire bonded length of the tendon. The smooth sheath of

the stressing length is attached and sealed to the end of the corrugated sleeve. Portland cement grout completely covers the tendon within the corrugated and smooth plastic sleeves. A suitable end cap is securely fastened at the end of the corrugated sleeve.

7. For tiebacks with threaded steel bar tendons, the corrugated sheathing extends the full length of the anchor, and fully grout the annular space between the steel bar and the corrugated sheathing before the anchor is installed. For such tiebacks a smooth sheathing encapsulates the corrugated sheathing within the stressing length. Securely fasten a suitable end cap at the bottom end of the corrugated sleeve.
8. Fabricate the bearing plate from mild steel that effectively distributes the design force to the supporting concrete. The structural steel bearing stress not to exceed F_y for the steel, and the concrete bearing stress not to exceed the allowable limits shown in Section 3.1.7 of the PTI "Guide Specification for Tensioning Materials." The bending stress in the bearing plate not to exceed $0.75 F_y$.
9. Seal weld a steel trumpet, having an inside diameter equal to the larger of the hole in the bearing plate or 0.5" greater than the outside diameter of the smooth sheathing to the bearing plate.
10. Completely encase the anchorage within a hot dipped galvanized steel cap. Fill all voids within the anchorage and steel cap with grout.

B. Plastic Sheathing/Sleeves

1. Smooth and corrugated sheathing/sleeves of Polyvinyl Chloride (PVC) conforming to ASTM D1784 with a minimum compressive strength of 15,000 psi and a minimum tensile strength of 7000 psi. The corrugated PVC sheathing with a minimum wall thickness of 50 mils. The PVC material free of water soluble chlorides and other ingredients that might cause corrosion, hydrogen embrittlement, or stress corrosion of the prestressing steel. The PVC plastic to be non-reactive with the grout and its ingredients, steel or corrosion inhibiting grease. The sheathing, including joints, is grout-tight and watertight.

- C. Cement – Unless otherwise stated on the Plans, use Type V Cement that conforms to the requirements of ASTM C150 for grouting tiebacks in all tieback locations except between Station 190+00 and 217+50 where Type II cement may be used. The Contractor samples and tests ground water during drilling operations for dissolved and suspended materials. Based on the results of this testing, the Engineer may decide to authorize use of Type II Cement.

1. Do not use cement that has been stored more than 30 days for grouting tiebacks.
2. Prior to mixing, maintain cement in a dry condition and store under cover.

D. Grout

1. Use potable water, clean and free of substances that are known to be harmful to Portland cement or to prestressed steel.
2. Use the lowest practical water-cement ratio with acceptable workability for the grout mix, except use no more than 5 gallons of water per 94 lb sack of cement.
3. Do not use accelerators.
4. For drilled holes 8” or greater in diameter, fine aggregate may be used for grout mix design but only to the extent that the cement content of the grout is not less than 846 lbs/cy of grout.
5. Admixtures may be used subject to approval of the Engineer.

E. Centralizers and Spacers

1. Place centralizers at ten-foot intervals in the bonded length starting at the end so that at least 1.0 inch of grout covers the bonded portion of the tendon.
2. Use strand spacers in the bonded length of tiebacks, and placed at five foot intervals. The strand spacers separate the strands from the PVC sheathing, and the individual strands, so that the entire surface of each strand is bonded in the grout.
3. Centralizers and spacers may be made of any material, except wood, that is not deleterious to the prestressing steel or to the plastic sheath.
4. Centralizers and spacers permit the free flow of grout around the tendon or strands.

F. Miscellaneous Steel Hardware

1. Steel plates conform to the requirements of ASTM A36 or ASTM A572, Grade 50.

2. All bolts, nuts and washers conform to the tendon manufacturer's specifications.
 3. Trumpets conform to the requirements of ASTM A53, and have a wall thickness of at least 0.2 inches.
 4. All bearing plates, anchorage components and couplers develop at least 95 percent of the minimum guaranteed ultimate strength (MGUS) of the tieback tendon.
- G. Corrosion Inhibiting Grease
1. Corrosion inhibiting grease has the physical properties listed in Table 3.2.1 of the Post-Tensioning Manual, 5th Edition by the Post-Tensioning Institute.

PART 3 - EXECUTION

3.1 PREPARATION

- A. Fabricate tendons in accordance with approved shop drawings, and be free of dirt, rust, and other deleterious substances.
- B. Degrease the bonded tendon prior to insertion in the tieback drill hole. No solvent residue to remain on the tendon.

3.2 CONSTRUCTION CONTROL

- A. Inspect the tieback before insertion into the drill hole. During insertion of the tieback into the hole, protect the tieback from any damage, especially damage to the corrosion protection media.
- B. Insert tiebacks freely to the prescribed length in the hole. Do not drive into the hole nor cut off in order to facilitate insertion.
- C. In no case shorten the stressing length of a tieback to less than the minimum length required by the plans or by this specification.
- D. Center the tendon in the drill hole.
- E. Measure the grout pressure at the point of injection. Clean the grout gauge mechanism prior to delivery to the site and periodically during the project to prevent clogging.

- F. Mix grout components mechanically for 5 to 10 minutes to ensure proper dispersion of cement in the grout mixture.
- G. Accurately control the established water-cement ratio.
- H. Immediately commence the pumping and injection of grout after mixing of the grout components.
- I. Continue injection of grout until grout that exits from the drill hole has the same composition as injected grout. Sample and test compressive strength of exiting grout on a routine basis (i.e., one out of every five holes to be injected or as otherwise requested by the Engineer) in order to verify compliance with this requirement.
- J. Extend the trumpet well over the tendon sheathing and do not damage during placement and stressing. Do not touch the tendon to the trumpet at the bearing plate.
- K. Prior to testing an anchor, verify by laboratory testing of cylinder samples that the compressive strength of the batch of grout used in the anchor satisfies the design requirements provided on the drawings and herein.

3.3 INSTALLATION

- A. Drill tieback holes at locations indicated on the plans.
- B. Select the diameter of tieback holes in order to achieve the design load capacities and the specified minimum grout cover. Extend the tieback hole one foot beyond the tendon length to be installed.
- C. The Contractor maintains an obstruction-free and open hole for grouting the tieback. The Contractor is solely responsible for determining the drilling method, grouting pressures, and tieback bonded length, subject to the provisions stated herein and on the plans, and subject to review by the Engineer. The Contractor designs the bonded length to satisfy the tieback testing acceptance criteria in accordance with the design loads. Select the grouting pressure and grouting method based on existing ground conditions.
- D. Design the drilling method selected by the Contractor to:
 - 1. Minimize disturbance to the surrounding ground and to prevent ground loss.
 - 2. Prevent collapse of the hole during drilling.

3. Maintain the position and inclination of the drilled hole, allow the hole to reach the design depth, and produce the design diameter of the drilled hole.
- E. Use temporary casing in tieback drill holes that have a tendency to collapse during drilling or during installation of the tieback. Withdraw the temporary casing as grout is placed.
- F. The Contractor immediately revises his operations to prevent reoccurrence of obstructed or otherwise unsatisfactory holes, and modifies tieback installation procedures as required.
- G. Immediately halt operations and repair to the satisfaction of the Engineer and owners directly impacted if any damage to existing site conditions occurs. The Contractor immediately revises his operations to prevent reoccurrence of such damage.
- H. Inject grout for all stages of tieback construction at the low end of the void being filled and expel at the high end until there is no evidence of entrapped air, water, or diluted grout. Place the grout using grout tubes, unless another method is approved by the Engineer.
- I. Inject the same grout in the stressing length zone as the grout that is injected in the bonded length zone.
- J. Initially grout tiebacks in holes of 6-inch diameter and smaller to a location not closer than 6 inches from the end of the steel tube. Do not place grout in the stressing length under pressure. After placing initial grout, do not disturb the anchor until the grout has reached strength sufficient to provide anchorage during testing operations. Provide all fittings and components required to perform these grouting operations.
- K. Initially grout tiebacks in holes greater than 6-inch diameter within the bond length. After placing initial grout, do not disturb the tieback until the grout has reached a strength sufficient to provide anchorage during testing operations. Grout in the stressing length and 6 inches into the steel tube after successful testing and lock-off of tieback. Provide all fittings and components required to perform these grouting operations.
- L. Use grouting equipment capable of continuous mixing and that produces a lump free grout. At the location of the nozzle, equip the grout pump with a grout pressure gauge capable of measuring at least 150 psi or at least twice the actual pressure used, which ever is higher.
- M. Expect voids within the rock formations through which the hole is drilled. If grout loss from a drilled tieback hole exceeds three times the volume of the annular space

between the drilled hole and tieback, then discontinue tieback installation and remove the tendon from the hole and clean. Fully pressure grout the drilled hole with a cement grout at a pressure of a least 5 psi above hydrostatic pressure, re-drill the hole 24 hours after the grout sets, and reinstall the tieback as described above in this specification. The Contractor determines the grout mixes and injection pressures for the pressure grouting and the Engineer approves.

- N. If the Engineer is concerned that grout losses from a tieback drill hole are excessive, the Engineer is entitled to direct the Contractor to conduct a water pressure test in a drilled tieback hole prior to grouting. The Contractor may also elect to perform a water pressure test. In either case, perform the testing at the Contractor's expense. If water loss (within the bonded length of the drill hole) exceeds 0.25 gallons per minute at a pressure of at least 5 psi above hydrostatic pressure for at least 10 minutes, then pressure grout the drill hole as described above.
- O. If use of pressure grouting does not satisfactorily plug the voids, consideration of using geosynthetic socks will be made at the discretion of the Engineer. Socks and excessive grout to be considered as extra work.
- P. Dispose of water used for drilling, ground water released by drilling operations, and water used for water pressure testing in a manner that causes no damage to the project or to the environment. The disposal method selected by the Contractor complies with all applicable environmental regulations.
- Q. Install load cells on 10 tiebacks to be determined by the Engineer. Use type and brand of load cells compatible to the testing requirements as specified herein this specification. The load cells (1) can be wired to a solar-powered readout box with at least 32 MB memory to store data, and (2) must carry compatibility software, accessories, and materials to permit future readings. Portable readout units and software accessories for data collection will become the property of the Department upon completion of the work.
- R. When freezing weather conditions will prevail during and following the placement of grout, the Contractor provides adequate means to protect the grout in the ducts from damage by freezing or other causes.
- S. When hot weather conditions would contribute to quick stiffening of the grout, cool the grout by approved methods as necessary to prevent blockages during pumping operations.

3.4 TIEBACK TESTING

- A. Test each tieback.

- B. Immediately after completion of testing of a tieback, transmit test results including graphs of the test data, to the Engineer for review and approval.
- C. Tension tiebacks with a hollow ram hydraulic jack or with other equipment recommended by the anchor manufacturer that is mounted in a fashion that prevents bending of the tieback.
- D. Provide ram travel of jacks that apply tension loads to tiebacks at least equal to the theoretical elastic elongation of the sum of the stressing and bonded lengths at the maximum test load. Use a pressure gauge with each jack. Calibrate gauges with a single jack. Submit proof of calibration before testing. All gauges sufficiently accurate to read 100 psi changes in hydraulic pressure. For performance tests, use a jack having two calibrated gauges: a master gauge and a backup gauge. The pump capable of applying each tieback load increment in less than 60 seconds.
- E. Measure changes in load during the load-hold portion of the performance tests (see Paragraph 3.4.K.2 of this specification) using a load cell, which has been calibrated by a certified independent testing laboratory no more than 20 days prior to the start of testing. A load cell must be used to measure forces applied to tiebacks during performance tests. However, a load cell is not required for proof tests. Provide the Engineer with the calibration curve for the load cell prior to testing.
- F. During performance tests, connect both the master gauge and the backup gauge to the pressure hose that connects between the pump and jack, and use both gauges to measure the applied loads. Recalibrate as a unit, the jack, master gauge, and backup gauge at no expense to the Department if the load measured by the master gauge and backup gauge differ by more than 10 percent.
- G. Support externally the weight of the jack and load cell, and not by the tendon that projects out of the tieback drill hole.
- H. Measure the elongation of the tendon with a micrometer dial gauge that can be read to a precision of 0.001 inches and that has at least 2 inches of travel perpendicular to the loading head. Support the dial gauge on an independent reference point and be in contact with the tendon head or an extension of the tendon head.
- I. Perform all testing in the presence of the Engineer, or his representative. Give notice to the Engineer at least 24 hours prior to the start of a load test.
- J. Except for the maximum test load, apply each increment of load in less than one minute and hold for at least one minute and until movement of the tieback stops. Start the observation period for the load hold when the pump begins to apply the last increment of load.

K. Performance Testing

1. Unless otherwise stated on the Plans, performance test at least 10 percent of the tiebacks, and performance retest five of these tiebacks within one year after their first performance tests. The Engineer will designate which tiebacks to performance test and which 5 tiebacks to performance retest.
2. The performance test includes stressing and monitoring a tieback. During testing, monitor tieback movement, measured at the anchor head, for each load increment to the nearest 0.001 inch from an independent, fixed reference point. The loading sequence follows:

Cycle	Load	Cycle	Load
1	AL	4	0.25 P
	0.25 P		0.50 P
	AL		0.75 P
			1.00 P
2	0.25 P		0.75 P
	0.50 P		0.50 P
	0.25 P		0.25 P
	AL		AL
3	0.25 P	5	0.25 P
	0.50 P		0.50 P
	0.75 P		0.75 P
	0.50 P		1.00 P
	0.25 P		1.25 P
	AL		1.50 P (Hold for creep test)
			Adjust to Transfer Load (Lock-off Load)

Where:

P = Design load (shown on plans)

AL = Alignment Load (0.05P)

3. The lock-off load as shown on the plans.
4. Maintain the maximum test load for 60 minutes. Record total movements with respect to a fixed reference point at 1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 40, 50 and 60 minutes.

5. Plot the tendon head movement versus load for each load increment. Also plot the creep movement for the load-hold stage as a function of the logarithm of time. The Engineer will review these data from each performance test to determine whether the tieback is acceptable.
6. If in the opinion of the Engineer, results of an individual performance test are significantly different than results of previous performance tests, the Engineer is entitled to direct the Contractor to perform an additional performance test on the next adjacent tieback that is installed. Perform this additional testing at the expense of the Contractor.

L. Proof Testing

1. Proof test all tiebacks that are not performance tested. Requirements for loading and monitoring of proof tests and performance tests are the same, except that the load sequence for proof tests are as follows:

Cycle	Load
1	AL
	0.25 P
	0.50 P
	0.75 P
	1.00 P
	1.25 P
	1.50 P (Hold)
	Adjust test load to lock-off load

Where:

P = Design Load

AL = Alignment Load (0.05P)

2. In proof tests, maintain the maximum proof load for 10 minutes. Re-pump the jack as necessary in order to maintain a constant load. Start the load-hold period as soon as the maximum test load is applied and the tieback movement is measured and recorded at 2, 3, 4, 5, 6 and 10 minutes. If the tieback movement between 1 minute and 10 minute exceeds 0.04 inches, hold the maximum test load for an additional 50 minutes and record the tieback movement at 15, 20, 30, 45 and 60 minutes. Prepare a plot of tieback movement versus load for each load increment in the proof test. Also, prepare a graph of tieback movement versus time for the final (hold) load increment. Formats of these graphs as approved by the Engineer prior to testing.

3. Plot the tendon head movement versus load for each load increment. Also plot the creep movement for the load-hold stage as a function of the logarithm of time. The Engineer will review these data from each performance test to determine whether the tieback is acceptable.
4. Any significant deviations in proof test results from previous performance test results will necessitate an additional performance test on the next adjacent anchor to be installed. Perform this additional testing at the expense of the Contractor.

3.5 ACCEPTANCE CRITERIA

A. Satisfy the following three criteria for each tieback:

1. Displacement of the anchor head greater than $0.8 PL_S/AE$, where
 - $P =$ applied load
 - $L_S =$ length from jack pulling head to bottom of stressing length
 - $A =$ total cross sectional area of steel tendon
 - $E =$ modulus of elasticity of steel tendon
2. Displacement of the tendon less than $P(L_S+L_b/2)/AE$ where,
 - $L_b =$ bonded length of tendon
3. Creep per log cycle, $(d_2-d_1)/\log_{10}(t_2/t_1)$, less than 0.040 inch between the 1 and 10 minute readings and less than 0.080 inch between the 6 and 60 minute readings, where,
 - $d_1 =$ measured displacement at time t_1
 - $d_2 =$ measured displacement at time t_2
 - $t_1 =$ time of first displacement measurement
 - $t_2 =$ time of second displacement measurement

B. Each tieback that does not satisfy Criterion 1 is replaced by the Contractor with a tieback that is installed in a manner and location that are acceptable to the Engineer

and that satisfy the requirements of Paragraph 1.9C of this specification. If a tieback does not satisfy either Criterion 2 or Criterion 3 of the acceptance criteria, the Engineer will be entitled to determine a portion of the design load that will be supported by the tieback and to require installation of additional tiebacks to compensate for the load capacity deficit of the original tieback. Install these additional tiebacks in a manner and at locations that are acceptable to the Engineer and that conform to the requirements listed in Paragraph 1.9C of this specification.

C. Lock-Off

1. After successful testing of the tiebacks, tension the tiebacks against the supporting concrete and lock off at the load shown on the contract plans. The lock-off force is the load on the jacks which is maintained while the anchor head or anchor nuts on the tieback are permanently set. Immediately after applying the lock-off load to the tieback and prior to removing the jack, perform a lift-off load reading to demonstrate that the specified lock-off force was obtained. Make adjustments in the shim thickness if required to maintain the specified lock-off force.
2. For strand tendons, fully set the permanent wedges in the anchor head while the tendon is stressed to the maximum test load, and then locked off at the lock-off force by removal of the shims or other appropriate methods.

3.6 RECORD OF WORK

- A. Record all work done and document accurately and completely. This documentation includes drilling of the tieback hole, water pressure testing, grouting, testing and stressing of tiebacks, equipment used for testing and associated calibration data, type of steel tendons, and materials and procedures used for corrosion protection of strands.

END OF SECTION

APPENDIX B
Sampling Field Instruction Sheet

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Grout Cube Field Notes

Mud Balance	Date Cast	Time Cast	Date Removed From Cooler	Time Removed From Cooler	Date Removed From Mold	Time Removed From Mold	Notes

Instructions:

- Step 1 Notify contractor that you will be taking a sample. The sample should be taken from grout that is mixed and ready for casting. Find out from which mixing bin you can take your sample. DO NOT collect a sample until the mixer blades have stopped moving.
- Step 2 Perform a mud balance test
 - 2a Fill the mud balance ladel with grout and place the lid on it. The ladel should be full so that grout is pushed out the sides when the lid is placed.
 - 2b Wipe off all excess grout and moisture from the outside surface of the ladel and lid with a paper towel.
 - 2c Place the ladle on the balance
 - 2d Adjust the gauge until the ladel is level and steady. Position the case so that ladel is shielded from the wind.
 - 2e Read the gauge value and input it above as the Mud Balance
- Step 3 Collect a grout sample from the mixer with the included bucket. The sample should be taken from the same grout mix that the mud balance test was taken. Ensure that the bucket is clean and dry prior to collecting the sample. Collect enough grout to fill all three cubes in the mold.
- Step 4 Carry the sample to the UDOT cooler where the molds are stored. Set the sample aside and check that the cooler is level. Adjust the cooler as needed.
- Step 5 Select a mold for casting. Ensure that it is clean and dry. Ensure that the nuts are tight. Spray the mold cubes with lubricating oil. Place the mold at the bottom of the cooler for casting. Ensure that it is level.
- Step 6 Using the included measuring cup, scoop grout out of the bucket and fill each of the three cubes in the mold. Fill each cube so that the grout comes up over the top face of the mold about 1/8". Do not move or touch the mold.
- Step 7 Close the lid and input the date and time cast above.
- Step 8 The next day gently remove the mold from the cooler, wrap it in a blanket. Input the date and time it was removed from the cooler above.
- Step 9 Return the mold to the office with this form.

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APPENDIX C

Water-Cement Ratio Correlations

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Figure 7.2 Wet Density vs. w/c Ratio

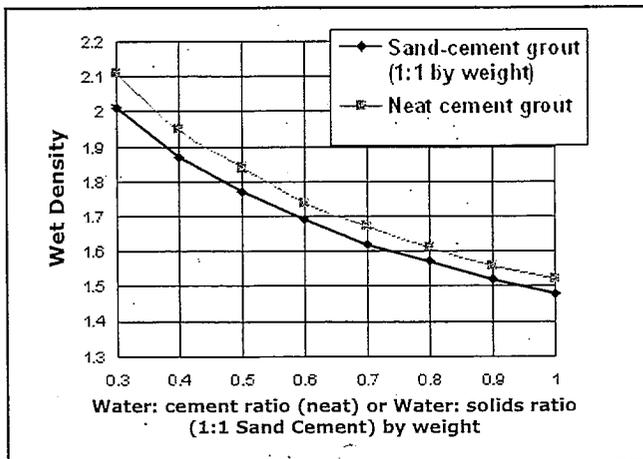
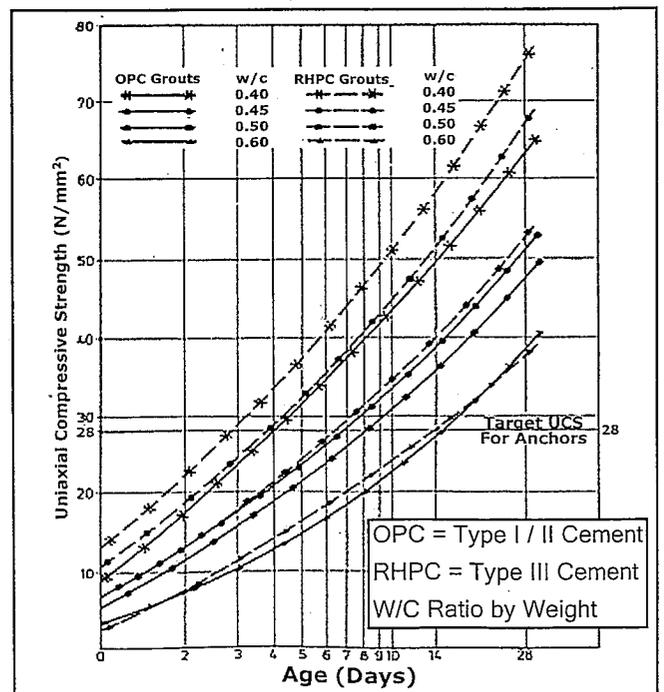


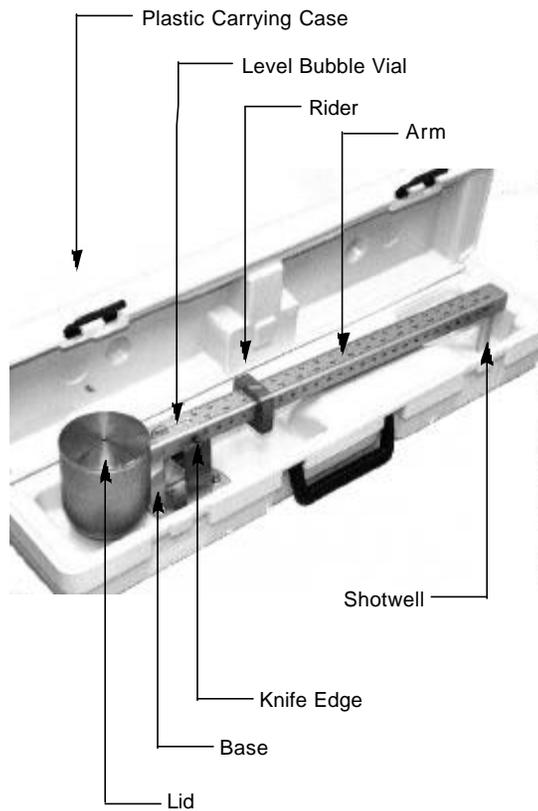
Figure 7.3 Strength for Various Water-Cement Ratios.



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APPENDIX D
Mud Balance Spec Sheet

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**4-Scale Metal Mud Balance
Complete with Carrying Case
115-00**

Components:

#115-01 4-Scale Metal Mud Balance w/o Case

- #100-25 Rider
- #100-29 Level Bubble Vial
- #100-56 Lead Shot
- #115-02 Machined Arm
- #115-06 Lid, Stainless Steel
- #115-22 Base, Stainless Steel
- #115-32 Knife Edge
- #115-34 Shotwell

Case:

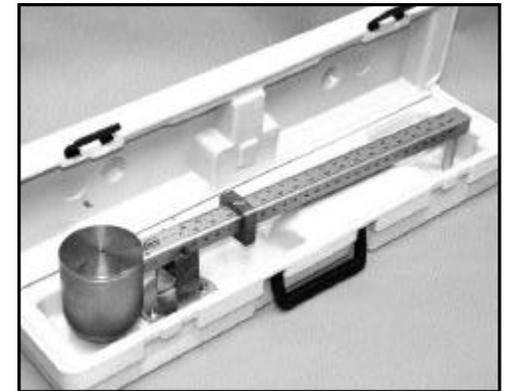
- #100-40 Plastic Carrying Case

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Dependable Products From People You Trust



**4-Scale Metal Mud Balance
(Machined Balance)**

Item# 115-00

Instruction Manual

**Updated 2/24/05
Ver. 1.2**

OFI Testing Equipment, Inc.

Introduction:

The density or weight of a given volume of liquid is determined by using a mud balance. The arm is graduated and permits accurate measurements to within ± 0.1 pounds per gallon or ± 0.01 specific gravity. The balance is constructed so that the fixed volume cup at one end of the beam is balanced by a fixed counterweight at the opposite end, with a sliding weight rider free to move along the graduated scale. A level bubble mounted on the beam indicates when the system is in balance.

Specifications:

6.5 - 23.0 lbs/gal
 0.79 - 2.72 specific gravity
 49 - 172 lbs/ft³
 340 - 1190 psi/1000 ft

Calibration:

OFITE mud balances are calibrated at the factory with the lid included in the mud balance kit. However, the balance should be re-calibrated, if necessary, on site. Any time a mud balance lid, or any other part, is replaced, the instrument should be re-calibrated.

1. The calibration of the instrument may be easily checked by measuring the density of fresh water.
2. Fill the cup with fresh water at around 70°F (21°C), and set the rider on 8.3 pounds per gallon or 1.0 specific gravity. Add or remove lead shot from the shotwell until the instrument is in balance.

Procedure:

1. Place the mud balance base (preferably in carrying case) on a flat level surface.
2. Measure the temperature of the fluid and record on the appropriate mud report form.
3. Fill the clean, dry cup to the top with the freshly obtained mud sample to be weighed.
4. Place the lid on the cup and set it with a gentle twisting motion. Be sure that some mud is expelled through the hole in the cap as this will ensure the cup is full and also will free any trapped air or gas.
5. Cover the hole in the lid with a finger and wash all mud from the outside of the cup and arm. Then thoroughly dry the entire balance.
6. Place the balance on the knife edge and move the rider along the outside of the arm until the cup and arm are balanced as indicated by the bubble.
7. Read mud weight at the edge of the rider toward the mud cup.
8. Clean and dry the mud balance after each use.

Results:

Report the mud weight to the nearest 0.1 pound per gallon, 1.0 pound per cubic foot, 0.01 gram per cubic centimeter (specific gravity) or 10 psi/1000 ft.

Density Conversions:

Pounds Per Gallon (lb/gal.)	Pounds per Cubic Foot (lb/ft ³)	Specific Gravity ^a (sg)	Kg per Meter ³ (kg/m ³)
6.5	48.6	0.78	780
7.0	52.4	0.84	840
7.5	56.1	0.90	900
8.0	59.8	0.96	960
8.3	62.3	1.00	1000
8.5	63.6	1.02	1020
9.0	67.3	1.08	1080
9.5	71.1	1.14	1140
10.0	74.8	1.20	1200
10.5	78.5	1.26	1260
11.0	82.3	1.32	1320
11.5	86.0	1.38	1380
12.0	89.8	1.44	1440
12.5	93.5	1.50	1500
13.0	97.2	1.56	1560
13.5	101.0	1.62	1620
14.0	104.7	1.68	1680
14.5	108.5	1.74	1740
15.0	112.5	1.80	1800
15.5	115.9	1.86	1860
16.0	119.7	1.92	1920
16.5	123.4	1.98	1980
17.0	127.2	2.04	2040
17.5	130.9	2.10	2100
18.0	134.6	2.16	2160
18.5	138.4	2.22	2220
19.0	142.1	2.28	2280
19.5	145.9	2.34	2340
20.0	149.6	2.40	2400
20.5	153.3	2.46	2460
21.0	157.1	2.52	2520
21.5	160.8	2.58	2580
22.0	164.6	2.64	2640
22.5	168.3	2.70	2700
23.0	172.1	2.76	2760
23.5	175.8	2.82	2820
24.0	179.5	2.88	2880

^a Specific gravity same as Grams per Cubic Centimeter (g/cm³)

Appendix E

Field Data

Set 1 – Schnabel Cube Sampling: Pages 53-56
Set 2 – UDOT Cube Sampling: Pages 57-58

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Set #	Date Cast	SG	Day 1	Day 3	Day 7	Day 8	Day 9	Day 10	Day 14	Day 15	Day 16	Day 28	Day 28
1	4/13/2005	1.83	1,552	3,030	4,990				6,000			6,780	
2	4/14/2005	1.75	520	1,030	4,030				5,450			5,850	
3	4/15/2005	1.85	2,010	3,100	6,950				8,150			6,660	
4	4/18/2005	1.83	1,010	2,670	3,610				5,170			8,110	
5	4/19/2005	1.85	1,060	3,990	5,540				6,940				
6	5/4/2005	1.83			4,600				8,430			8,430	
7	5/6/2005	1.82			4,250				5,440			8,420	
8	5/12/2005	1.85			7,150				9,180			11,530	
9	5/17/2005	1.85			4,470				3,340			6,660	
10	5/20/2005	1.83			7,210				8,160			7,890	
11	5/23/2005	1.83			5,230				5,970			9,400	
12	5/25/2005	1.84			7,100				6,670			7,430	
13	5/27/2005	1.73			4,650				4,560			5,880	
14	5/31/2005	1.83			5,740				5,180			9,170	
15	6/2/2005	1.83			4,140				5,920			7,870	
16	6/7/2005	1.80			3,040				4,740			6,190	
17	6/10/2005	1.81			4,112				5,700			8,690	
18	6/13/2005	1.81			4,000				5,530			5,960	
19	6/14/2005	1.77			4,180				4,720			5,670	
20	6/16/2005	1.82			5,960				6,550				
21	6/21/2005	1.82			8,440				4,760			6,700	
22	6/23/2005	1.73			3,030				4,450			4,570	
23	6/27/2005	1.84			3,700				5,590			7,240	
24	6/28/2005	1.86			5,690				7,190			8,420	
25	7/7/2005	1.82			5,900				6,140			7,530	
26	7/15/2005	1.88			7,100				7,880			8,360	
27	7/19/2005	1.78			4,940				5,370			6,150	
28	7/20/2005	1.82			5,040				5,640			5,720	
29	7/21/2005	1.79			4,700				6,210			6,620	
30	7/22/2005	1.82			4,040				5,880			6,250	

Set #	Date Cast	SG	Day 1	Day 3	Day 7	Day 8	Day 9	Day 10	Day 14	Day 15	Day 16	Day 28	Day 28
31	7/26/2005	1.80			3,990				5,550			6,730	
32	7/28/2005	1.83			4,410				4,540			5,420	
33	7/29/2005	1.78			5,000				5,930			6,180	
34	8/1/2005	1.81			5,180				5,170			6,380	
35	8/2/2005	1.84			4,540				4,610			6,150	
36	8/3/2005	1.82			4,250				5,420			6,390	
37	8/18/2005	1.77	1,600	3,580	3,760				4,600			4,630	
38	8/23/2005	1.83	1,110	2,840	4,410				5,030			6,220	
39	8/30/2005	1.78	1,270	2,870	4,280				5,740			5,890	
40	9/13/2005	1.87	3,240	4,510	6,190				6,320			8,560	
41	9/20/2005	1.85	2,750	3,270	6,910				5,010			7,490	
42	9/27/2005	1.91	4,460	4,610	6,326				7,770			10,490	
43	10/3/2005	1.88	1,530	3,710	4,660				8,020			6,960	
44	10/11/2005	1.82	1,670	1,970	2,290				3,210			5,620	
45	10/18/2005	1.91	2,600	3,890	4,850				6,200			6,690	
46	10/25/2005	1.78	1,990	2,040	2,770				3,530			4,500	
47	11/1/2005	1.85	3,460	3,540	6,200				6,350			6,450	
48	11/16/2005	1.81	2,340	2,580	3,820				6,350			6,550	
49	12/15/2005	1.81			3,400				3,460			5,660	
50	12/16/2005	1.75			2,540				3,010			4,310	
51	12/19/2005	1.83							3,630			4,910	4,550
52	12/20/2005	1.78								2,190		3,180	2,980
53	12/21/2005	1.85									2,850	3,050	2,930
54	12/29/2005	1.78				2,770			3,890			4,060	
55	1/10/2006	1.76				2,820			3,270			4,110	
56	1/11/2006	1.80					2,610		3,020			3,020	
57	1/13/2006	1.78						2,950	3,020			2,980	
58	1/16/2006	1.79			2,730				2,940			4,210	
59	1/18/2006	1.77					2,200		2,010			3,220	
60	1/20/2006	1.81						3,510	5,250			3,310	

Set #	Date Cast	SG	Day 1	Day 3	Day 7	Day 8	Day 9	Day 10	Day 14	Day 15	Day 16	Day 28	Day 28
61													
62													
63													
64	2/6/2006	1.82			3,150				4,360			4,500	
65	2/8/2006	1.76			970				3,310			4,480	
66	2/9/2006	1.80			2,860				4,400			5,980	
67	2/10/2006	1.81			970				4,360			6,360	
68	2/14/2006	1.80			4,110				4,480			5,930	
69	2/15/2006	1.82			2,760				4,160			5,440	
70	2/20/2006	1.72				3,630			3,680			3,900	
71	2/27/2006	1.80				3,190			3,220			3,940	
72	2/28/2006	1.82					3,820		3,780			5,440	
73	3/5/2006	1.82			1,140				2,810			2,180	
74	3/7/2006	1.75			3,390				3,610			3,630	
75	3/10/2006	1.78			4,230				4,430			6,240	
76	3/18/2006	1.86			5,080				4,940			7,020	
77	3/22/2006	1.81			3,390				3,990			4,670	
78	3/23/2006	1.80			3,050				3,990			5,200	
79													
80	3/28/2006	1.75			3,820				4,810			5,200	
81	4/3/2006	1.78			4,020				4,430			5,130	
82													
83													
84	4/12/2006	1.80			3,820				4,860			4,890	
85	4/13/2006	1.79			3,460				3,820			4,500	
86	4/21/2006	1.76			2,900				3,560			4,310	
87	4/24/2006	1.76			4,540				5,320			5,560	
88	4/24/2006	1.79			1,230				3,920			4,960	
89	5/1/2006	1.86			4,600				5,400			6,050	
90	5/2/2006	1.76			4,670				4,770			5,980	

Set #	Date Cast	SG	Day 1	Day 3	Day 7	Day 8	Day 9	Day 10	Day 14	Day 15	Day 16	Day 28	Day 28
91	5/3/2006	1.76			3,630				4,090			4,960	
92	5/5/2006	1.80			2,860				3,900			2,300	
93	5/8/2006	1.82			3,390				4,760			4,480	
94	5/9/2006	1.80			5,370				5,980			5,440	
95	5/11/2006	1.78			3,870				4,230			4,720	
96	5/15/2006	1.80			4,430				5,610			6,410	
97	5/16/2006	1.79			4,520				5,840			4,910	
98	5/17/2006	1.86			5,810				6,220			6,360	
99	5/23/2006	1.83			2,780				6,630			6,770	
100	5/25/2006	1.80			4,790				4,380			3,340	

Sample #	Date Cast	Mud Balance	Elapsed Time (Days)	Break Stress 1 (psi)	Break Stress 2 (psi)	Break Stress 3 (psi)	Avg Break Stress (psi)	Elapsed Time (Days)	Break Stress 1 (psi)	Break Stress 2 (psi)	Break Stress 3 (psi)	Avg Break Stress (psi)
U1	5/22/2006	1.78	23	4800	5425	5193	5139	28	5498	5585	6078	5720
U2	5/23/2006	1.81	22	7285	-	-	7285	28	7375	7373	-	7374
U3	5/24/2006	1.72	21	3953	-	-	3953	28	3938	4188	-	4063
U4	5/25/2006	1.78	20	6375	-	-	6375	28	6810	7090	-	6950
U5	5/30/2006	1.85	15	6703	-	-	6703	28	6950	7153	-	7052
U6	5/31/2006	1.81	14	7470	-	-	7470	28	8700	9008	-	8854
U7	6/1/2006	1.87	14	7750	-	-	7750	28	9220	9463	-	9342
U8	6/5/2006	1.85	14	7880	-	-	7880	28	9078	8180	-	8629
U9	6/6/2006	1.82	14	6553	-	-	6553	29	8145	6830	-	7488
U10	6/7/2006	1.76	14	6328	-	-	6328	28	6553	7089	-	6821
U11	6/8/2006	1.74	14	4685	-	-	4685	28	4910	4618	-	4764
U12	6/12/2006	1.77	14	6588	-	-	6588	28	6745	6863	-	6804
U13	6/13/2006	1.77	14	6090	-	-	6090	28	7383	7240	-	7312
U14	6/14/2006	1.82	14	6580	-	-	6580	28	7770	7418	-	7594
U15	6/15/2006	1.88	14	8005	8308	8243	8185	28	9758	8745	8883	9129
U16	6/19/2006	1.80	14	6000	6410	6183	6198	28	6975	6683	6825	6828
U17	6/20/2006	1.82	15	6523	6093	5860	6159	28	6333	6998	6960	6764
U18	6/26/2006	1.84	14	6080	6078	-	6079	29	6840	6820	-	6830
U19	6/27/2006	1.82	14	5818	5440	5700	5653	28	6858	6548	6850	6752
U20	6/28/2006	1.81	14	5985	5695	6030	5903	28	6580	6913	6835	6776
U21	6/29/2006	1.80	14	5610	5295	-	5453	28	6763	5120	6268	6050
U22	7/5/2006	1.84	14	6948	6678	7023	6883	28	7708	8070	7868	7882
U23	7/6/2006	1.82	14	5292	5395	5343	5343	28	5905	6318	6188	6137
U24	7/11/2006	1.80	14	5728	5600	5610	5646	28	4860	4710	5210	4927
U25	7/12/2006	1.89	14	6363	6260	6493	6372	28	5230	6530	6090	5950
U26	7/13/2006	1.83	14	5745	6163	-	5954	28	5940	5010	5410	5453
U27	7/17/2006	1.85	14	6320	6178	-	6249	28	5250	4630	4800	4893
U28	7/19/2006	1.81	14	5805	5848	5578	5744	28	2600	4650	4650	3967

Sample #	Date Cast	Mud Balance	Elapsed Time (Days)	Break Stress 1 (psi)	Break Stress 2 (psi)	Break Stress 3 (psi)	Avg Break Stress (psi)	Elapsed Time (Days)	Break Stress 1 (psi)	Break Stress 2 (psi)	Break Stress 3 (psi)	Avg Break Stress (psi)
U29	7/20/2006	1.83	14	5388	5970	5503	5620	28	3180	3380	3100	3220
U30	7/26/2006	1.81	14	3890	4370	4330	4197	28	4700	5100	4790	4863
U31	7/27/2006	1.81	14	3991	5780	5980	5250	28	5300	5580	5550	5477
U32	7/31/2006	1.82	14	4130	4630	3300	4020	28	5920	5750	5470	5713
U33	8/1/2006	1.84	14	4240	2125	3830	3398	28	6050	6280	6400	6243
U34	8/3/2006	1.81	14	2750	2030	2830	2537	28	5630	4930	5250	5270
U35	8/8/2006	1.83	14	4050	3780	3250	3693	28	5430	5950	6030	5803
U36	8/9/2006	1.84	14	5310	5000	5250	5187	28	7100	7250	7380	7243
U37	8/10/2006	1.82	14	4550	4250	4880	4560	28	6100	5580	6180	5953
U38	8/14/2006	1.80	14	5000	5080	5330	5137	28	7030	7130	7250	7137
U39	8/15/2006	1.80	14	4580	4550	4950	4693	28	5680	5830	5880	5797
U40	8/16/2006	1.80	14	5600	5250	5750	5533	28	6080	6300	6380	6253
U41	8/17/2006	1.83	14	5050	4830	4550	4810	28	6680	6830	7250	6920
U42	8/23/2006	1.88	14	6250	6080	6530	6287	28	6880	6500	6630	6670

UDOT
tested
RBG
tested