24-28(a)

COPY NO.

EXTEND AND VALIDATE RESULTS FROM NCHRP PROJECT 24-28 VIA IMPROVEMENTS IN NDT PROTOCOLS, IN SITU MEASUREMENT OF FILL RESISTIVITY, AND ADDITIONS TO THE PERFORMANCE DATABASE

FINAL REPORT

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
National Research Council

Kenneth L. Fishman, Ph.D., P.E.
McMahon & Mann Consulting Engineers, P.C.
BUFFALO, NY

June 2011

ACKNOWLEDGEMENT OF SPONSORSHIP AND DISCLAIMER

Acknowledgement

This work was sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program, which is administered by the Transportation Research Board of the National Research Council.

24-28(a)

COPY NO.

EXTEND AND VALIDATE RESULTS FROM NCHRP PROJECT 24-28 VIA IMPROVEMENTS IN NDT PROTOCOLS, IN SITU MEASUREMENT OF FILL RESISTIVITY, AND ADDITIONS TO THE PERFORMANCE DATABASE

FINAL REPORT

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
National Research Council

Kenneth L. Fishman, Ph.D., P.E.
McMahon & Mann Consulting Engineers, P.C.
BUFFALO, NY

June 2011

TABLE OF CONTENTS

	Page
ACKNOWLEDGEMENT OF SPONSORSHIP AND DISCLAIMER……..	i
TABLE OF CONTENTS…….	ii
LIST OF FIGURES…….	iv
LIST OF TABLES……...	v
ACKNOWLEDGEMENTS…….	vi
ABSTRACT……	vii
EXECUTIVE SUMMARY…….	1
INTRODUCTION…….....	1
RESEARCH APPROACH……..	2
FINDINGS……...	3
Resistivity Measurements……	3
Monitoring of MSES in the Field……	3
Implementation of IR Test for NDT of Rock Bolts……	4
CONCLUSIONS……	6
RECOMMENDATIONS……	7
CHAPTER 1 – BACKGROUND……	8
OBJECTIVES……..	9
Type I Reinforcements…….	9
Type II Reinforcements……..	9
CHAPTER 2 – RESEARCH APPROACH…….	10
CHAPTER 3 – FIELD EVALUATION OF NDT FOR ROCK BOLTS……	11
INSTALLATION OF DUMMY ROCK BOLTS……	11
NONDESTRUCTIVE TESTING……	12
Sonic Echo……..	12
Impulse Response……...	15
Ultrasonic……	19
Electrochemical Tests…….	19
CHAPTER 4 – IN SITU RESISTIVITY MEASUREMENTS……	29
CONSTRUCTION OF EDI TEST EMBANKMENT…….	29
Resistance Measurements……	31
Calculation and Verification of Resistivity……..	32
APPLICATION OF INSITU RESISTIVITY MEASUREMENTS……	33
Caltrans Sites……..	34
Data Collection……	35

TABLE OF CONTENTS (Continued)

	Page
CHAPTER 5 – CONCLUSIONS AND RECOMMENDATIONS…….	53
CONCLUSIONS…….	53
RECOMMENDATIONS……	54
REFERENCES…….	55

APPENDIX I- INSTALLATION AND NDT OF DUMMY ROCK BOLTS AT THE BARRON MOUNTAIN TEST SITE, WOODSTOCK, NH

APPENDIX II – CONSTRUCTION, INSTALLATION OF DUMMY REINFORCEMENTS AND COUPONS, AND TESTING AT THE EDI TEST EMBANKMENT, ELMA, NY

APPENDIX III – CALTRANS TEST SITES AND MSE WALLS FIELD MONITORING AND LABORATORY DATA

LIST OF FIGURES

No.		Page
1(a)	Sketch of 10’ long dummy rock bolt installations	23
1(b)	Sketch of 15’ long dummy rock bolt installations	24
1(c)	Sketch of 20’ long dummy rock bolt installations	25
2	Locations (Elevation View) of dummy rock bolts 1-6.	26
3	Typical IR Test Results a) Time-history of impact b) Force spectrum c) Time-history of element response d) Acceleration response spectrum e) Mobility curve	27
4	Ultrasonic Test Results a) UT results for Bolt #4 b) UT results for Bolt #5 c) UT results for Bolt #6	28
5	Cross section of EDI test embankment and placement of dummy reinforcements and coupons.	48
6	Grain size of Materials #1 and #2 for EDI test embankment.	49
7	Locations of sites in California listed in Table 9.	50
8	Illustration of Cylinder Probes Developed for NCHRP 21-06 (Corrpro, 2009)	51
9	Illustration of DMP Developed for NCHRP 21-06 (Corrpro, 2009)	52

LIST OF TABLES

No.		Page
1	Details of "Dummy" Rockbolts Installed at the Barron Mountain Rock Cut	20
2	Predominant and Resonant Frequencies Observed from Mobility Curves	21
3	Summary of Electrochemical Test Results for Dummy Rock Bolts	22
4	Parameters For Fine Sand and Fine Gravel Used for the Test Embankment	38
5	Summary of In-Place Density Measurements	39
6	Resistivity Measurements from Samples Retrieved During Construction of the EDI Test Embankment	40
7	Calculations of ρ from Measured Resistance of a Single Element – Three Electrode Configuration	41
8	Calculations of ρ from Measured Resistance Between Wide Grids– Two Electrode Configuration	42
9	Summary of Sites in California with Observations from 2010	43
10	Comparison of Lab, Field and In Situ Resistivity Measurements	44
11	Reinforcement Age and E_{corr} Measurements from Sites in California	45
12	Soil Properties Measured at the Cypress Bunker, Emeryville, CA	46
13	Comparison of Laboratory and In Situ Measurements of Resistivity from Coupons at the Cypress Corrosion Test Site	47

ACKNOWLEDGEMENTS

This project included installations and construction of test facilities and fieldwork at selected sites providing a basis for verification of enhancements and improvements to test methods and protocols for NDT of earth reinforcements This would not have been possible without cooperation from the New Hampshire Department of Transportation and Caltrans; and considerable contributions of time, equipment, manpower, and expertise that we received from private companies including Earth Dimensions, Inc., 3rd Rock, Inc., and Janod, Inc.

The New Hampshire Department of Transportation (NHDOT) assisted with fieldwork and installation of dummy rock bolts necessary for demonstration, evaluation and verification of NDT at the site of the Barron Mountain Rock Cut in Woodstock, New Hampshire. Mr. Richard Lane, Mr. Dave Merrill, and Ms. Krystle Pelham from NHDOT provided site access, and helped to coordinate on site activities needed to collect data from this active construction site. Janod, Inc. installed “dummy” rocks bolt at the site, and the assistance provided by Messrs. Daniel Journeaux and Andrew Salmaso is greatly appreciated.

Earth Dimensions, Inc. (EDI) donated space at their facility in Elma, New York, and provided personnel and equipment for construction of the test embankment for evaluating in situ measurements of fill resistivity. The efforts and generosity of Mr. Brian Bartron and his staff from EDI greatly contributed to the success of this project. 3rd Rock, Inc. donated field-testing services to measure in situ moisture content and density during construction. The Reinforced Earth Company (RECO) donated reinforcement samples that were placed within the embankment.

Messrs. Rob Reis and Joe Shanabrook from Caltrans Corrosion Technology Branch, Materials Engineering and Testing Services Unit helped to select and provide access to sites in California, assisted with the field work, and provided results from laboratory testing from samples of fill and reinforcements that were retrieved from these sites.

ABSTRACT

The results described herein are from a follow-up study related to NCHRP Project 24-28 "LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal Reinforcements in Geotechnical Applications." Recommendations following NCHRP 24-28 are to address the high uncertainty regarding the performances of MSE reinforcements within marginal quality fills, obtain additional performance data from older installations of earth reinforcements, and implement more robust test techniques to obtain better information about the existing conditions of rock bolts, soil nails and ground anchors. To address these needs the follow-up study focused on protocols to achieve better correspondence between the rate of corrosion and resistivity of surrounding earthen materials, collecting additional field performance data, and implementing more sophisticated dynamic testing for rock bolts, soil nails and ground anchor systems.

The variability inherent to the observed performance of marginal quality fills is likely due to uncertainty with respect to fill properties, which may also be inherently variable. Measurements of fill resistivities at the times and locations of corrosion rate measurements will address a major source of this uncertainty. The current protocol for corrosion monitoring of MSE reinforcements with the linear polarization resistance test includes measurement of total resistance of earth materials that surround the reinforcements. Results from research described in this report demonstrate how these measurements can be related to resistivity. The relationship between measured resistance and resistivity was studied by constructing a test embankment incorporating a variety of reinforcement geometries and fill conditions. An equation from the existing literature relating the resistance of a single ground rod to the resistivity of the surrounding medium was found to work well with earth reinforcements idealized as ground rods. The veracity of the ground rod resistance formula was further demonstrated via field measurements from selected sites in California. Selected sites incorporated marginal quality fill materials for construction of MSE walls, and included older MSE walls from the Caltrans inventory. Data collected in California demonstrated a good correlation between measurements of corrosion rate and fill resistivity, and helped to explain the high variance associated with the observed performance of marginal quality fills.

We also obtained data from the oldest MSE wall in California, which was constructed in 1971 and was 39-years old at the time of observations in 2010. These data were useful to observe the response of older galvanized reinforcements wherein the base steel may be exposed subsequent to depletion of the zinc coating. The observed corrosion rates of the base steel were less than what would be anticipated compared to past practice that considers the base steel to perform similar to plain steel elements that have not been galvanized. These data suggest that the benefits of galvanization are realized even after zinc has oxidized, and the presence of zinc oxide appears to have a positive effect on the durability of the base steel.

Use of dynamic tests, including the sonic echo (SE) and impulse response (IR) tests, were evaluated for condition assessment of rock bolts, soil nails and ground anchors. Although both of these tests involve impacting the end of an element and monitoring the

corresponding response, they differ with respect to the level of instrumentation and data interpretation. The sonic echo test has been applied in the past and only includes measurements of signals received at the ends of the elements subsequent to impact, which are processed in the time domain. The IR test is an enhancement to the SE test, and involves both measurements of the impact and the element response, which allows data to be processed in the frequency domain in terms of the cyclic mobility.

Six dummy rock bolts were installed at the Barron Mountain Rock Cut, along I-93 near Woodstock, NH to study the potential benefits associated with IR vs. SE testing. Half of the installations incorporated known anomalies or defects, and half were "normal" installations. It was found that the locations of defects could be identified by the SE test, but the IR test provided much higher resolution with respect to location, and also information about the size and nature of the defect. Thus, the IR test is recommended for probing and condition assessments of rock bolts, soil nails and ground anchors.

This research achieved improvements in test protocols and methods to better describe existing conditions and parameters needed to model the in situ performances of earth reinforcements. The test protocols and techniques described in this report will serve to enhance performance data collected and archived as part of NCHRP Project 24-28, and will be useful to further validate and improve predictive models for corrosion potential, metal loss, and service life of metal reinforced systems.

EXECUTIVE SUMMARY

INTRODUCTION

The results described herein are a follow-up to NCHRP Project 24-28 "LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal Reinforcements in Geotechnical Applications." NCHRP Project 24-28 includes assessments and improvements to models for corrosion potential, metal loss, and service life of metal-reinforced systems used in retaining walls, and highway cuts and fills. A performance database for earth reinforcements was compiled to describe the existing condition and measured corrosion rates of in service reinforcements using data obtained from approximately 170 sites distributed throughout the United States and Europe. Data are obtained from reinforcement types typical of mechanically stabilized earth systems (MSES), and installations of rock bolts and ground anchors. These data were useful to identify trends that can be grouped in terms of the character of fill or insitu earth materials, reinforcement type and various site conditions. Based on these trends the reliability of metal loss modeling and service life predictions were computed and the corresponding biases with respect to nominal tensile strengths used in design were assessed and used to calibrate appropriate resistance factors for use in LRFD.

MSES reinforcements consist of galvanized steel strips, mats or grids. The possibility of metal loss in design is considered with respect to selection of fill materials that are relatively nonaggressive, and by including sacrificial steel in the reinforcement cross section. It was found that estimates of sacrificial steel requirements and remaining strength were reliable considering reinforcements surrounded by high or good quality fill materials. However, the performance estimates for reinforcements in marginal quality fills are highly uncertain. Also, the estimated performance of galvanized reinforcements after base steel that has been exposed subsequent to the depletion of zinc is subjective due to a lack of data from older installations. This may mean that service life estimates are overly conservative, rendering recommendations of resistance factors for use in LRFD that are too low.

Rock bolts, soil nails and ground anchors include features that differ from MSES reinforcements including components of single or double corrosion protection systems, and use of high strength steel elements that are often prestressed during installation (soil nails are not prestressed). Thus, performance and service lives of these types of earth reinforcements are related to the details of the installation, the existing conditions of elements incorporated into the corrosion protection systems; and, forms of corrosion that may include localized rather than uniform or general corrosion including stress crack corrosion or hydrogen embrittlement. Better information on the in service conditions of rock bolts, soil nails and ground anchors is needed to assess factors that may have significant effects on performance. Construction records are often not available from rock bolt installations, so information from in service reinforcements is needed relative to the geometry and quality of these installations.

Recommendations following NCHRP 24-28 are to address the high uncertainty regarding the performances of MSES reinforcements within marginal quality fills, obtain additional performance data from older installations of MSES reinforcements, and implement more robust test techniques to obtain better information about the existing conditions of rock bolts, soil nails

and ground anchors. This report describes results from Project 24-28(a), which was intended to address these recommendations and further validate results from NCHRP 24-28 and predictive models for corrosion potential, metal loss, and service life of metal reinforced systems.

A complete description of NCHRP Project 24-28 has been published as NCHRP Report No. 675, and this report serves as a supplement to the former.

RESEARCH APPROACH

Marginal quality fills are classified according to minimum resistivity (AASHTO T 288), which can be very different from the in situ values. Samples of fill for resistivity testing are often collected prior to construction from stockpiles representing potential sources of fill material. However, sources actually used for construction are uncertain. Therefore, the variability inherent to the observed performance of marginal fills is likely due to uncertainty with respect to the fill properties, which may also be inherently variable. These effects are not as prevalent for good or high quality fills wherein sources of materials are more certain, variability is less, and the rate of metal loss is not as sensitive to changes in fill resistance. Therefore, the approach to reducing the uncertainty with respect to performance of marginal quality fills is to obtain better measurements for fill resistivity, and corresponding correlations with measurements of corrosion rate. Thus, there is a need to identify and implement methods to measure fill properties at the time and location of the corrosion rate measurements.

For rock bolts, soil nails and ground anchors there is a need to extract more information on existing condition. Testing and data analysis techniques must be refined to meet this objective. Others have applied the impulse response (IR) technique to study the condition of deep foundation elements, and these results have been useful to locate and identify the size and shape of anomalies along concrete drilled shafts. This is considered an improvement over the sonic echo (SE) technique, which is the existing practice for condition assessment of rock bolts. The SE technique is only useful to identify the locations of sources of reflections, and to provide qualitative information about prestress levels. Thus, the suitability of the IR test for rock bolt installations is explored as part of NCHRP Project 24-28(a).

The following tasks were performed consistent with the stated objectives:

1. Evaluate the effectiveness of the impulse response technique to identify and render more information on the in situ condition of rock bolt installations. An important component of this task is installation and testing of "dummy" rock bolts incorporating planned anomalies to provide a basis for comparison.

2. Identify methods to determine resistivity for in situ materials surrounding earth reinforcements at the time and location of corrosion rate measurements. An important part of this task is to demonstrate the veracity of these methods by constructing an earth embankment that incorporates earth reinforcements, testing and comparison with known conditions.

3. Further verify the test methods and relationships developed in Task 2 via field-testing. This is accomplished via measurements of corrosion rates and resistivity from sites with marginal fills where fill samples can be retrieved and tested for resistivity using a standard test box as a basis of comparison.

4. Field data are obtained from older sites to evaluate the long-term performance of base steel subsequent to depletion of zinc from older reinforcements that are in service.

FINDINGS

Resistivity Measurements

The linear polarization resistance (LPR) technique has been used to determine the corrosion rate of in service earth reinforcements. A three-electrode configuration is commonly employed whereby a current is impressed between two in service reinforcements that serve as the working and counter electrodes, and the surface potential of the working electrode is measured with respect to a half-cell that serves as reference electrode. The earth material surrounding the working electrode (earth reinforcement under test) serves as an electrolyte and measurements of polarization resistance must be corrected to consider the effects of this additional impedance. Current practice is to measure the resistance of the earth material via an AC impedance technique immediately following the DC polarization measurements. Results from this study demonstrate that earth resistivity (ρ) can be computed from measurements of earth resistance as:

$$\rho = \frac{2 \times \pi \times L \times R}{\left[\ln\left(\frac{8 \times L}{D} \right) - 1 \right]} \qquad \text{(i)}$$

where L and D are the electrode length and diameter, respectively, and R is the measured earth resistance. This is a very simple expression that appears to render good results in most cases. Thus, measurements of earth resistivity can be made at the same time and location as corrosion rate measurements for earth reinforcements.

A test embankment was constructed to verify the use of Equation (i) for relating the measured fill (earth) resistance to resistivity. The embankment was split into two sections each with a different fill material of known resistivity. Different reinforcement types, sizes and shapes were installed and the spacing between the reinforcements was varied. Measurements from the test embankment and use of Equation (i) were in reasonable agreement with the known values of resistivity as long as the reinforcements were located at least two feet away from the surface of the embankment. On average resistivities computed with Equation (i) were within ten to twenty percent of the known values.

Monitoring of MSES in the Field

Use of Equation (i) was further verified via data from testing in service MSES reinforcements at sites located in California, and under the jurisdiction of Caltrans. Five sites were included in the

field investigation with ages of reinforcements that ranged between five and 39-years old. In situ measurements of resistivity were obtained from the three-electrode, AC impedance technique, and use of Equation (i). Additionally, samples of fill material were retrieved through access holes penetrating the precast concrete wall face units and tested for resistivity via a soil box similar to AASHTO T 288. The latter test results served as a basis for comparison with in situ measurements of resistivity. Reasonable results were obtained wherein in situ testing rendered a range of results that was consistent with baseline values obtained from samples tested in the soil box. Furthermore, corrosion rate measurements were negatively correlated with resistivity (i.e., higher corrosion rates were associated with lower resistivity). Thus, results from corrosion rate measurements correlated very well with in situ measurements of resistivity.

The fieldwork included the oldest MSE wall in the United States, which was constructed in 1971, along Route 39 through the San Gabriel Mountains near Los Angeles, and 39 years old at the time when observations were made. Reinforcements unearthed from this site appeared to be in excellent condition with no significant metal loss, although base steel was exposed in some locations. Observations of corrosion rates were less than those anticipated from the current metal loss rates suggested by AASHTO for design of MSES. These data suggest that the benefits of galvanization are realized for much longer then the 16 years implied by the current edition of the AASHTO LRFD Bridge Design Specifications. Thus, measurements of corrosion rate obtained from San Gabriel are extremely interesting and are some of the first confirmed observations of the rate of steel consumption subsequent to depletion of zinc for galvanized reinforcements used in MSE construction in the United States. These rates are much less than those used in design and appear to be closer to the corrosion rates used for zinc, implying that a discrete change in corrosion rates as zinc is depleted, as implied by the current AASHTO model for metal loss of MSE reinforcements, does not necessarily occur.

Implementation of IR Test for NDT of Rock Bolts

The impulse response test (IR) is implemented for probing the lengths of rock bolts to access details of the installation and conditions surrounding the structural elements (i.e. steel rods). The IR test involves application of an impact to the free end of the rock bolts with an instrumented hammer and measuring the response of the element with a transducer attached near the point of impact. Measurements of the impact force and the resulting response are processed in the frequency domain. A mobility curve is obtained by dividing the velocity response spectrum by the force spectrum, and incorporates a range of frequencies corresponding to the energy content of the impact (0-4000 Hz). The initial slope of the mobility plot renders the dynamic stiffness of the system, and the frequency of the peaks at resonance describes the geometry. By comparison the SE test does not require an instrumented hammer, and only involves measuring the response of the rock bolt after impact, which is presented in the time domain.

The mobility curve renders the impedance of the rock bolt corresponding to the material properties and geometry of the installation. The impedance of the rock bolt is related to the geometric mean of the heights of the resonant peaks in the portion of the mobility curve where the shaft response is in resonance. Mobility is related to impedance as (Davis and Robertson, 1975)

$$N = \frac{1}{\rho \times V_p \times A} \qquad \text{(ii)}$$

where, ρ is mass density = 4.66 ($lb*s^2/ft^4$) for grouted rock bolts, V_p is compression wave velocity = 12,000 ft/s for grouted rock bolts, and A is the cross sectional area corresponding to the resonant portion of the mobility plot. Thus, the cross sectional area corresponding to the resonant portion of the mobility plot can be determined from measurements of cyclic mobility and knowledge of the density and compression wave velocity of the grout surrounding the earth reinforcement.

The utility of IR test was evaluated from testing six dummy rock bolts installed in cooperation with the New Hampshire Department of Transportation at the site of the Barron Mountain Rock Cut along I-93 near Woodstock, NH. Dummy rock bolts were installed using materials and techniques similar to actual rock bolt installations. Installations varied with respect to geometry including the free lengths, bonded lengths, and total lengths of the rock bolts. Half of the installations were "normal", and the other half had known defects including reduced cross-sections, breached sheathing along the free length, or voids in the grout along the bonded lengths.

The SE and IR tests were applied to the dummy rock bolts, and the veracity of the results was confirmed based on comparisons with known conditions. Both results from the IR test and the SE tests rendered useful information for condition assessment. Levels of prestress were interpreted from SE test results in qualitative terms as high, moderate or relatively low based on a comparison of results between samples. This capability has been recognized previously, and the results from the IR do not render any improvements in this regard. Clear reflections were apparent from the interfaces between the free lengths and the bonded zones, such that results from the SE test were useful to identify the free lengths of the test elements. However, details of anomalies and conditions within the bonded zones were difficult to discern, or not discernable, from results of SE testing. Important features of the installations were more apparent from results from IR tests, compared to results from SE testing, including:

- Interfaces between bonded and unbonded zones are more distinct.
- Details and conditions within the bonded zone are apparent in the results from IR test, which cannot be discerned from the results of SE testing.
- Mobility plots are affected by anomalies in terms of distinct reductions in the energy (peaks) and frequencies at resonance. This is due to the energy loss from additional wave reflections caused by the anomalies, and corresponding reductions in the propagation velocities for compression waves traveling within the grout.
- The cross sectional areas of the elements at the sources of reflections were observed from results of IR testing, which are reconciled from the mobility at resonance.

CONCLUSIONS

This study addresses recommendations resulting from NCHRP Project 24-28 to (1) obtain more reliable data and reduce uncertainty with respect to the performance of MSES constructed with marginal quality fills, (2) obtain additional performance data from older installations of MSES reinforcements, and (3) implement more robust test techniques to evaluate the existing conditions of rock bolts, soil nails and ground anchors. Results presented in this report serve to further validate results from NCHRP 24-28 and predictive models for corrosion potential, metal loss, and service life of metal reinforced systems.

1. The variability of the observed performance of marginal fills resulting from NCHRP Project 24-28 is due to uncertainty with respect to the fill properties, which may also be inherently variable. Measurements of fill resistivity obtained at the location and time of corrosion rate measurements reduce uncertainty, and improve our ability to model the performance MSES constructed with marginal quality fills. Results presented in this report demonstrate that the resistivity of in situ materials surrounding earth reinforcements can be determined at the time and location of corrosion rate measurements. The technique employs measurements of resistance via the three-electrode technique, and a simple relationship between the measured resistance and resistivity. Measurements of corrosion rate and resistivity in the three-electrode configuration are most useful, since the majority of measurements in the database compiled as part of NCHRP Project 24-28 utilize this technique. Thus, resistivity is computed using the measured resistance, and the width and length of the reinforcement. This technique is verified via measurements under known conditions within a test embankment, and data collected from five different sites in California with access to MSE reinforcements and fill materials for sampling and testing.

2. The fieldwork included the oldest MSE wall in the United States, which was constructed in 1971, along Route 39 through the San Gabriel Mountains near Los Angeles, and 39 years old at the time when observations were made. Measurements of corrosion rate obtained from San Gabriel are extremely interesting since they are some of the first confirmed observations of the rate of steel consumption subsequent to depletion of zinc for galvanized reinforcements used in MSE construction in the United States. These rates are much less than those used in design and appear to be closer to the corrosion rates used for zinc, implying that a discrete change in corrosion rates as zinc is depleted, as implied by the current AASHTO model for metal loss of MSE reinforcements, does not necessarily occur.

3. The impulse response test (IR) was implemented for probing the lengths of rock bolts to access details of the installation and conditions surrounding the structural elements (i.e. steel rods). The IR test is useful to locate and identify the size and shape of anomalies along rock bolt installations. This is considered an improvement over the sonic echo (SE) technique, which is the existing practice for condition assessment of rock bolts. Results from the IR test were verified with respect to tests performed with dummy rock bolts that included both typical and deliberately distressed installations. Results from this study indicate that the IR test is more robust and renders additional information for condition

assessment of rock bolts compared to what is achievable from the SE test. However, the SE test is adequate to assess remaining levels of prestress, and to identify the free lengths of rock bolt installations. Knowledge of the free length is useful if the total length is known, and general details of the installation are needed for reconciling data from further electrochemical testing (e.g., lengths are needed to assess surface areas of the test elements and reconcile corrosion rates from LPR measurements).

RECOMMENDATIONS

1. The techniques for measuring fill resistivity implemented in this study should be applied to the database developed for NCHRP Project 24-28 to develop service life models that directly incorporate fill resistivity as a variable. Current service life models used to calibrate LRFD strength reduction factors for NCHRP Project 24-28 apply to relatively broad ranges of fill types (resistivities) leading to wide scatter and variability within the range of marginal quality fills. Service life models that directly incorporate fill quality will render improved correlations between corrosion rate and fill resistivity, and improvements to reliability-based calibration of LRFD strength reduction factors for MSE constructed with marginal quality fills.

2. More data should be collected to document the performance of MSE constructed with marginal quality fills. The characteristics of marginal quality fill including salt contents and resistivities may vary randomly with respect to location within a given source. Fill resistivity may change over time due to changes in moisture content, salt concentration, etc., and these changes are not described from measurements from fill sources obtained prior to construction. Thus, data should be collected from selected sites at prescribed intervals to monitor spatial variations and the effects of changes in conditions that may occur over time. These data will be useful to provide guidance, recommendations, specifications and limitations on use of marginal quality fills for construction involving earth reinforcements.

3. The IR test is recommended for condition assessment of rock bolts when more detailed information from within the bonded zone is necessary. Further evaluations should be conducted considering rock bolt lengths greater than twenty feet.

4. Additional data on the performance of rock bolts should be collected and used to develop fragility curves to describe the time dependant vulnerability and safety of rock cuts that are supported with rock bolts.

CHAPTER 1 - BACKGROUND

As part of NCHRP Project 24-28 we assessed and improved the predictive capabilities of existing computational models for corrosion potential, metal loss, and service life of metal-reinforced systems used in retaining walls and highway cuts and fills. Methodology was developed that incorporates the improved predictive models into an LRFD approach for the design of metal reinforced systems. Recommended additions and revisions were prepared to incorporate the improved models and methodology into the AASHTO LRFD Bridge Design Specifications.

MSES reinforcements consist of galvanized steel strips, mats or grids. The possibility of metal loss in design is considered with respect to selection of fill materials that are relatively nonaggressive, and by including sacrificial steel in the reinforcement cross section. Data archived for NCHRP Project 24-28 incorporate a range of fill conditions generally classified as high, good and marginal quality. These different classifications fall into ranges defined by threshold values of resistivity including 10,000 Ω-cm, 3000 Ω-cm and 1000 Ω-cm. High and good quality fills both meet AASHTO criteria for electrochemical parameters including resistivity (>3000 Ω-cm), pH (5-10), chloride (<100 ppm) and sulfate (<200 ppm) content; but for high quality fills $\rho_{min} \geq 10{,}000$ Ω-cm, and ρ_{min} is between 3000 Ω-cm and 10,000 Ω-cm for good quality fills. Marginal quality fills are described as having $5 < pH < 10$, and 1000 Ω-cm < $\rho_{min} < 3000$ Ω-cm.

It was found that estimates of sacrificial steel requirements and remaining strength were reliable considering reinforcements surrounded by high or good quality fill materials. However, the performance estimates for reinforcements in marginal quality fills are highly uncertain. Also, the estimated performance of galvanized reinforcements after base steel that has been exposed subsequent to the depletion of zinc is subjective due to a lack of data from older installations. This may mean that service life estimates are overly conservative, rendering recommendations of resistance factors for use in LRFD that are too low.

Rock bolts, soil nails and ground anchors include features that differ from MSES reinforcements including components of single or double corrosion protection systems, and use of high strength steel elements that are often prestressed during installation (soil nails are not prestressed). Thus, performance and service lives of these types of earth reinforcements are related to the details of the installation, the existing conditions of elements incorporated into the corrosion protection systems; and, forms of corrosion that may include localized rather than uniform or general corrosion including stress crack corrosion or hydrogen embrittlement. Better information on the in service conditions of rock bolts, soil nails and ground anchors is needed to assess factors that may have significant effects on performance. Construction records are often not available from rock bolt installations, so information from in service reinforcements is needed relative to the geometry and quality of these installations.

Project 24-28(a) is to further validate results from NCHRP 24-28 and predictive models for corrosion potential, metal loss, and service life of metal reinforced systems. Measurements are collected at an independent set of field sites across the United States, supplemented with results from controlled field experiments with prototype installations of earth reinforcements. Sites with

rock bolt installations and mechanically stabilized earth (MSE) walls are included in this follow-up study. Testing includes both NDT techniques (e.g., ultrasonic testing, sonic echo, impulse response and electrochemical testing) and comparisons with direct measurement after exhumation of in situ reinforcements, or with known details from installations of "dummy" reinforcements. The comparisons serve to validate the NDT methods as well as the predictive models based upon these methods.

OBJECTIVES

For NCHRP Project 24-28 and this study, reinforced systems are broadly categorized as two types. Type I reinforcements are passive elements used in the construction of metallically reinforced earth (MSE) structures that may consist of steel strips, welded wire fabric, wire mesh, or soil nails. Type II reinforcements are active elements that are prestressed during installation including ground anchors (strands and bars), and rock bolts. In general, Type II reinforcements consist of relatively high strength steel, and a higher level of corrosion protection compared to Type I reinforcements.

Type I Reinforcements

The following objectives apply to Type I reinforcements and the need for data to validate the performance models, and supplement data to address limitations inherent to the database compiled as part of NCHRP Project 24-28.

- Evaluate effect of marginal fills on performance and service life
- Assess the corrosion rate of steel after zinc has been consumed from galvanized elements

Type II Reinforcements

The following objectives apply to Type II reinforcements and the need to substantiate use of electrochemical test techniques for corrosion monitoring and integrity testing of corrosion protection systems; and to extract more information on existing condition from the results of dynamic testing (e.g. sonic echo and impulse response).

- Study application of corrosion monitoring with LPR techniques. Seek measurements and observations that can render the surface area in contact with the surrounding earth material, and knowledge of the influence of grout and other components of the corrosion protection system.
- Refine data analysis techniques for dynamic tests (wave propagation techniques).
- Verify results obtained with these techniques, and evaluate the limitations of these NDT's for probing earth reinforcements.

CHAPTER 2 – RESEARCH APPROACH

Marginal quality fills are classified according to minimum resistivity (AASHTO T 288), which can be very different from the in situ values. Samples of fill for resistivity testing are often collected prior to construction from stockpiles representing potential sources of fill material. However, sources actually used for construction are uncertain. Therefore, the variability inherent to the observed performance of marginal fills is likely due to uncertainty with respect to the fill properties, which may also be inherently variable. These effects are not as prevalent for good or high quality fills wherein sources of materials are more certain, variability is less, and the rate of metal loss is not as sensitive to changes in fill resistance. Therefore, the approach to reducing the uncertainty with respect to performance of marginal quality fills is to obtain better measurements for fill resistivity, and corresponding correlations with measurements of corrosion rate. Thus, there is a need to identify and implement methods to measure fill properties at the time and location of the corrosion rate measurements.

For rock bolts, soil nails and ground anchors there is a need to extract more information on existing condition. Testing and data analysis techniques must be refined to meet this objective. Others have applied the impulse response (IR) technique to study the condition of deep foundation elements, and these results have been useful to locate and identify the size and shape of anomalies along concrete drilled shafts. This is considered an improvement over the sonic echo (SE) technique, which is the existing practice for condition assessment of rock bolts. The SE technique is only useful to identify the locations of sources of reflections, and to provide qualitative information about prestress levels. Thus, the suitability of the IR test for rock bolt installations is explored as part of NCHRP Project 24-28(a).

The following tasks were performed consistent with the stated objectives:

1. Evaluate the effectiveness of the impulse response technique to identify and render more information on the in situ condition of rock bolt installations. An important component of this task is installation and testing of "dummy" rock bolts incorporating planned anomalies to provide a basis for comparison.

2. Identify methods to determine resistivity for in situ materials surrounding earth reinforcements at the time and location of corrosion rate measurements. An important part of this task is to demonstrate the veracity of these methods by constructing an earth embankment that incorporates earth reinforcements, testing and comparison with known conditions.

3. Further verify the test methods and relationships developed in Task 2 via field-testing. This is accomplished via measurements of corrosion rates and resistivity from sites with marginal fills where fill samples can be retrieved and tested for resistivity using a standard test box as a basis of comparison.

4. Field data are obtained from older sites to evaluate the long-term performance of base steel subsequent to depletion of zinc from older reinforcements that are in service.

CHAPTER 3 – FIELD EVALUATION OF NDT FOR ROCK BOLTS

More than 200 rock bolts have been in service since 1974 at the site of the Barron Mountain rock cut along I-93 near Woodstock, NH. This site provides a unique opportunity to evaluate test protocols and gather data on the condition and performance of various types of rock bolt installations. We performed condition assessments including NDT on selected rock bolts beginning in the fall of 2003, and some of these data were collected as part of the fieldwork for NCHRP 24-28. Based on the recommendations resulting from these studies, the NHDOT implemented a project during the summer and fall of 2009 including measurement of loads via "lift-off" tests, and retrofit or installation of replacement rock bolts as necessary. Due to the ongoing maintenance and construction activities, contractors were mobilized at the site, and site access and necessary traffic control were available. We took advantage of this opportunity to cooperate with NHDOT, share data, and perform additional testing. Specific activities performed as part of NCHRP project 24-28(a) included (1) installation of "dummy" rock bolts with planned anomalies, (2) nondestructive testing of dummy rock bolts, and (3) interpretation of data, and comparison with known conditions along the dummy rock bolts.

INSTALLATION OF DUMMY ROCK BOLTS

Janod Inc. completed installation of six "dummy" rock bolts at the site on September 21 and 22, 2009. Rockbolts were one-inch nominal diameter, Grade 150, galvanized steel threaded rods manufactured by Dywidag. The installation included a stressing length (L_f = free length) surrounded by grease and a plastic sheath, and a bonded length (L_b) in direct contact with grout. Anchor heads included a 1-½ -inch thick, twelve-inch square, bearing plate with a spherical nut and seat. Lengths of the individual rock bolts ranged from 10 to 20 feet incorporating bonded lengths of three to five feet. The rods were inserted into 3.5 inches diameter drill holes advanced with a pallet drill outfitted with a pneumatic, down-the-hole, hammer. Holes were drilled at a downward inclination of approximately 15° from the horizontal. Centralizers were placed along each rod within approximately two feet from the lowest end. Rock bolts were grouted from the bottom up using a grout tube, and a mixture of Sitka 300 PT Grout and approximately 1.5 gallons of water per 50 lb bag. The grout was allowed to cure for approximately three days after which the installations were proof tested and locked-off at loads ranging from 10 to 40 kips. Lock-off loads were verified via lift-off testing. Appendix I includes photographs depicting the site, and installation of the dummy rock bolts. Table 1 is a summary of the installations describing the total lengths (L_T), free lengths (L_f), bonded lengths (L_b) and the bolt condition. Sketches of these elements are shown in Figures 1(a) to 1(c). Figure 2 shows an elevation view and approximate relative locations of Dummy Bolts #1 through #6.

See Table 1. Details of "Dummy" Rockbolts Installed at the Barron Mountain Rock Cut

See Figure 1(a) Sketch of 10' long dummy rock bolt installations.

See Figure 1(b) Sketch of 15' long dummy rock bolt installations.

See Figure 1(c) Sketch of 15' long dummy rock bolt installations.

See Figure 2. Locations (Elevation View) of dummy rock bolts 1-6.

Anomalies were incorporated into three of the installations, and three were installed normally. Anomalies include voids in the grout along the bonded or free lengths, compromised sheaths along the free lengths or partial cuts into the cross sections of the steel rods. Voids were created by taping a small plastic bottle or other soft object to the perimeter of the rod prior to grouting. Partial cuts into the threaded rods were advanced with a chop saw creating an approximately 3/8 inch deep slot.

NONDESTRUCTIVE TESTING

Testing performed at the site includes sonic echo (SE), impulse response (IR), ultrasonic, (UT), and electrochemical measurements including half-cell potential and linear polarization resistance. Mechanical tests such as SE, IR and UT are useful for condition assessment and identifying levels of prestress, reinforcement geometries, and the locations of voids in the grout, gaps at the free end, and loss of steel section. Measurements of half-cell potential and linear polarization resistance are electrochemical tests used to indicate surface condition (metal type and/or presence of corrosion) and corrosion rates.

Both the SE and IR tests are impact type tests wherein the faces of the reinforcement elements are impacted with a hand-held hammer that generates elastic compression waves. The traveling waves are reflected whenever a change in material properties or geometry is encountered along the length of the element. The arrival of these reflected waves at the face of the element produces accelerations, which are measured with an accelerometer mounted to the face. Results from the SE test are interpreted in the time domain, and the acceleration waveform can be used to determine the travel time from the initiation of impact to the arrival of the wave reflection. If the compression wave velocity of the test element is known the distance to the reflector can be calculated. The locations of the reflectors correspond to changes in material properties or geometry that may be related to anomalies, distress or various as-built details.

The IR test is similar to the SE test in many respects, but an additional set of measurements and more sophisticated data processing are inherent to the IR test. An instrumented hammer is necessary for the IR test, which is not required for the SE test, rendering measurement of the impact force. In difference to the time domain used to process SE test results, IR test results are interpreted in the frequency domain in terms of the response spectrum from the accelerometer attached to the face of the element, and the force spectrum from the impact hammer. Compared to the SE test, the additional instrumentation and effort associated with the IR test provides increased resolution and more information relative to the size and shape of the sources of reflections. This is considered an improvement over the SE technique, which is the existing practice for condition assessment of rock bolts (Withiam et al., 2002). The IR test has been applied to evaluate deep foundation such as drilled shafts (e.g., Finno et al., 1998), but this is the first application to testing for rock bolt integrity.

SONIC ECHO

Details and application of the sonic echo test for condition assessment of rock bolts are described by Withiam et al. (2002) that includes Appendix E, "Recommended Test Method for Impact

Echo Test of Bar-Type Rock Bolts, Ground Anchors and Soil Nails." Salient details of the equipment required for the test and the test procedure are as follows.

Equipment for the SE test includes a light hammer, an accelerometer for measuring the element response, a signal conditioner, a data acquisition system, and a portable computer for recording and storing data. Other necessary equipment includes the connections required to link all of these items and a portable power supply. Specific recommendations and desirable attributes are as follows:

- A light (0.5 lb to 1.0 lb) tack hammer or small ball peen hammer works well. A hand-held punch may also be used with the hammer to direct impact energy to a smaller area over the face of the test element.
- A general-purpose shock accelerometer with a frequency range from 0.4 Hz to 7500 Hz, a capacity of ± 5000 g, a sensitivity of a least 10 mV/g, and a relatively high resonant frequency of at least 25 kHz is recommended. An integrated-electronics piezoelectric, shear structured accelerometer with very low sensitivity to transverse motion is desirable.
- A signal conditioner is needed to (a) provide constant current excitation to power the accelerometer microelectronics, (b) remove DC bias voltage from the analog signal and provide a drift free AC coupled output compatible with standard readout instrumentation, and (c) provide a selectable gain to amplify the output signal from the accelerometer.
- A portable computer with a high-speed digital data acquisition system should be used to capture the transient output of the accelerometer, store the digital waveform, and perform signal analysis.

The accelerometer is attached to the face of the rock bolt by drilling and tapping a hole to receive the threaded base of the accelerometer, or mounting a threaded base to the face of the rock bolt with adhesive. After connecting the accelerometer to the signal conditioner and data acquisition system perform the test as follows:

1. Set the parameters for data acquisition in terms of the sample frequency (number of measurements per second), duration of measurement and trigger levels. A sample frequency of at least 20 kHz is recommended and the duration of measurements depends on the length of the rock bolt, but should be long enough to receive reflections from the end of the bonded zone. The trigger level should be set as low as possible, but high enough to overcome background noise. A pre-trigger feature that continuously buffers data prior to the trigger is also desirable to capture the initial portion of the impact.
2. Strike the face of the reinforcement with the hammer to generate compression waves along the length of the element. The impact should be administered at or near the center of the reinforcement cross section.
3. Observe the reflected waveform from the accelerometer output.
4. Store the data.
5. Repeat the impact until three repeatable signals are observed and recorded.
6. Plot the time history of the acceleration measurements and identify reflections apparent in the time history.

7. Determine the arrival time of each of the observed reflections and compute the reflector locations using the arrival times and a known or assumed value of compression wave velocity.

Evaluation of the SE test at the Baron Mountain test site included use of different hammers, and methods of attaching the accelerometer to the end of the element. Two different lightweight hammers (1 lb) were evaluated including a tack hammer available from most any hardware store, and a modally tuned hammer available from PCB Piezotronics (Model # 086D05). Various methods of attaching the accelerometer to the end of the rock bolts include a) drilling and tapping a hole in the end of the rock bolt and engaging the male threads at the base of the accelerometer, (b) using a base to engage the threads of the accelerometer and gluing the base to the end of the rock bolt, and c) using a magnetic base to engage the threads of the accelerometer that is then secured to the end of the rock bolt via a magnetic bond.

Element responses are affected by the lengths of the stick-up, which cause reflections at intervals ranging between 0.166 ms and 0.240 ms; also corresponding to predominant frequencies of approximately 2000 Hz to 3000 Hz. These reflections are present throughout the duration of the observed response, and other reflections from changes in geometry, materials and stiffness along the lengths of the rock bolts are superimposed on these oscillations. Notable reflections include the interfaces between the free lengths and the bonded zone at intervals between 1 ms and 2 ms, and the far ends of the rock bolts at intervals ranging from 1.5 ms to 3 ms. These reflection intervals are computed using the lengths in Table 1, and compression wave velocities of 16,000 ft/s along the free lengths and 12,000 ft/s along the bonded (grouted) zones.

The modally tuned hammer rendered signals with the sharpest reflections corresponding to the interfaces between the free lengths and the bonded zones, and the best representation of signal attenuation within the first millisecond of response. However, responses observed with the tack hammer compared reasonably well with those from the modally tuned hammer.

The method of attaching the accelerometer affected signal quality in terms of the clarity of the signal reflections. Securing the accelerometer by drilling and tapping a hole at the end of the rock bolts renders the best signal responses, however this is a very difficult and time consuming method of attachment. The glued base rendered adequate results compared to the drilled and tapped attachments. Use of the magnetic base produced signals with a lot more noise, and sometimes signals that were difficult to interpret.

Conclusions regarding the ability of the SE test to render accurate condition assessments are discussed from results obtained with the modally tuned hammer, and attachment of the accelerometer with the glued base. Appendix I includes time histories of the responses from dummy rock bolts 1 – 6. These results are plotted to different scales to depict the responses within the first millisecond, and responses for time intervals up to five milliseconds. The attenuation within the first millisecond is useful to assess levels of prestress/tension in the reinforcements, and longer time intervals depict reflections from probing along the lengths of the reinforcements. The strengths of signals at later time intervals are related to the quality of the grout along the lengths of the elements, and changes in cross section along the rock bolts. SE test results show:

- More attenuation of energy within the first millisecond is evident from testing more highly stressed elements. Signal attenuation is much more apparent for specimens with prestress levels of 20 kips and 40 kips compared to those with the lower prestress levels of 10 kips.
- Reflections indicate the locations of the interfaces between the free lengths and the bonded/grouted zones. Lower wave velocities are apparent for the distressed elements.
- For distressed elements clear reflections are apparent at times less than those corresponding to the far ends of the element. These reflections are caused by the presence of defects created within the grouted zone as described in Table 1 and Figures 1(a), 1(b) and 1(c).

IMPULSE RESPONSE

The equipment and procedures for the IR test are similar to the SE test, but use of an instrumented hammer is also required. A modally tuned impact hammer, model # 086D05 manufactured by PCB Piezotronics is employed for the IR test. The hammer consists of an integral quartz force senor mounted on the striking end of the hammer head and has a measurement range of ± 5000 lb, sensitivity of 1 mV/lb, and a resonant frequency ≥ 22 kHz. The hammer output was calibrated at the factory prior to testing for this project.

The hammer impulse consists of a nearly constant force over a broad frequency range, and is therefore capable of exciting all resonances within that range. The hammer size, length, hammer tip material and velocity at impact determine the amplitude and frequency content of the force impulse. The hammer weighs 0.7 lb with a head diameter of 1.0 in, and a 9-inch length. The striking end of the hammer has a threaded hole for installation of various impact tips. The impact cap material generally determines energy content, and we recommend using a steel tip to generate higher frequencies and energy suitable for testing rock bolts.

The methods used to process, present and interpret measurements are mainly what distinguish the IR from the SE test. Data analysis for the IR test is completed in the frequency domain, rather than the time domain as described for the SE test. A Fast Fourier Transform (FFT) is performed on both the applied force and acceleration response signals to convert them from the time to the frequency domain, and the velocity spectrum is derived from the acceleration spectrum. After this is completed, the velocity spectrum is divided by the force spectrum to obtain a plot of the mobility versus frequency. The mobility includes peak responses, which are due to wave reflections from changes in geometry or material properties. These changes occur at the end of the bolt stick-up, free lengths and bonded zones, and other features that may include anomalies such as loss of cross section, or voids and cracks along the grouted column.

The necessary calculations may be performed using functions commonly available via an Excel spreadsheet with the Data Analysis Package installed with the Microsoft Excel software. Three sets of data measured at each time increment are acquired from the IR test including time, impact force, and acceleration response. These are organized into the first three columns of the spreadsheet and the lengths of the columns depend on the sampling frequency and duration. These columns must then be padded to render lengths that are an even power of two to allow

application of the FFT. The following steps and corresponding Excel functions are applied to compute a mobility curve from the IR test measurements.

1. The applied force and acceleration responses are transformed to the frequency domain using the Fourier analysis tool. The analysis tool analyzes periodic data via the Fast Fourier Transform Method to transform the data. Thus, a column of complex numbers is computed for the impact force and the acceleration response representing the amplitude and phase associated with the frequency contents for each of these signals. Each cell of the column corresponds to a frequency that depends on the sample rate, duration of sampling, and time of measurement.

2. The frequencies corresponding to each cell are computed in the next column as

$$f(n) = t(n) \times (\text{sample rate})^2/\text{total number of samples}$$

   where f(n) and t(n) are the frequency (in Hertz) and time (in seconds) corresponding to the nth cell, and the sample rate is in terms of the number of measurements acquired per second.

3. The mobility is the ratio of the velocity and force response functions, where the velocity response is the acceleration response divided by $2\pi f$. Excel functions to perform the arithmetic in terms of the complex functions for each cell are employed as follows:

   =improduct(FFT accel, impower(complex(0,2πf),-1), impower(FFT force,-1))

   where "FFT accel" and "FFT force" are the transforms of the acceleration and force measurements described in step 1, and f are the frequencies determined in step 2.

4. The mobility spectrum is computed as the absolute value of the complex number for each cell as:

   =imabs(mobility), where "mobility" corresponds to each of the cells created in Step 3.

5. Following the calculations in step 1-4, the mobility curve may be plotted using the frequencies computed in step 2 along the abscissa, and the mobility computed in step 4 along the ordinate.

Figures 3(a) to 3(e) are typical results corresponding to Rock Bolt #1 (15' long, normal installation), and depict results from data processing and analysis as described in Steps 1-5. Figures 3(a) and (b) depict the time history of the impact force and the force spectrum determined from the time history, respectively. Figure 3(a) indicates that the duration of the impact is approximately 0.2 ms, and the majority of the energy is incorporated within frequencies between 0 and 4000 Hz as depicted in the force spectrum in Figure 3(b). Figures 3 (c) and (d) depict typical responses in the time and frequency domains from the accelerometer measuring the responses at the free end of the rock bolt.

See Figure 3. Typical IR Test Results

Figure 3(e) depicts the mobility plot, which is obtained by dividing the velocity response spectrum by the force spectrum. The mobility plot renders more information on the condition of the rock bolt in addition to the locations of anomalies or features of the installations. Additional details include the stiffness of the elements, and the impedances associated with cross sections where reflections occur.

The stiffness is determined from the reciprocal of initial slope of the mobility plot (Figure 3(e)). The impedance of the rock bolt is related to the geometric mean of the heights of the resonant peaks in the portion of the mobility curve where the shaft response is in resonance. The geometric mean, N, of the resonant peaks depicted in Figure 3(e) is equal to

$$N = \sqrt{P \times Q} = \sqrt{0.0052 \times 0.0012} = 0.0025 \qquad (1)$$

where P and Q correspond to the local maximum and minimum, respectively of the resonant peaks. Mobility is related to impedance as (Davis and Robertson, 1975)

$$N = \frac{1}{\rho \times V_p \times A} \qquad (2)$$

where,
ρ is mass density = 4.66 ($lb*s^2/ft^4$) for grouted rock bolts,
V_p is compression wave velocity = 12,000 ft/s for grouted rock bolts, and
A is the cross sectional area corresponding to the resonant portion of the mobility plot.

Thus the mobility plot of Figure 3(e) renders

$$A = \frac{1}{\rho \times V_p \times N} = \frac{1}{4.66 \frac{lb \times s^2}{ft^4} \times 12{,}000 \frac{ft}{s} \times 0.0025 \frac{ft}{s \times lb}} = 0.00714\, ft^2 = 1.03 in^2$$

This area corresponds to a diameter of 1.14 inches consistent with the diameter of the Dywidag rods, and resonant peaks at 800 Hz correspond to the interface between the free length and unbonded zone. At this location the Dywidag rod is isolated from the surrounding grout by grease and a plastic sheath such that the mobility is related to the area of the rod.

Mobility plots obtained from normal installations and those with defects are compared in Appendix I considering 10 feet, 15 feet and 20 feet long rock bolts. The following conclusions are apparent for these data:

- The resonant portions of the mobility plots from distressed installations contain addition peaks due to reflections from the anomalies compared to the mobility plots from normal installations.
- Lower predominant frequencies are noted for the distressed rock bolts. Lower predominant frequencies are a manifestation of lower prestress and stiffness inherent to the distressed elements.
- Resonance corresponding to the far ends of the rock bolts is more distinct for the installation where a soft zone or void is created in the grout at the end of the drill hole; e.g. Bolt #5, fifteen feet long, distressed installation. This is due to the higher change in impedance associated with the void/rock bolt interface compared with the interface between the rock bolt and the surrounding rock.

Predominant and resonant frequencies observed in the mobility curves are useful to locate changes in geometry or material properties. These may be correlated with as-built details and comparisons made to locate anomalies and identify locations where the elements may be distressed. Three features of the installations are considered including the lengths of the stick-ups (L_{stick}), bonded lengths (L_{bond}), and free lengths (L_{free}). Predominant and resonant frequencies corresponding to L_{stick}, L_{bond} and L_{free} are identified from the mobility curves for each element and summarized in Table 2. Lengths corresponding to the frequency contents are computed as:

$$L_{Stick}(ft) = \frac{16{,}000(\frac{ft}{s})}{4 \times f_1(Hz)} \tag{3}$$

$$L_{bond}(ft) = \frac{12{,}000(\frac{ft}{s})}{2 \times f_2(Hz)} \tag{4}$$

$$L_{free}(ft) = \frac{16{,}000(\frac{ft}{s})}{2 \times \Delta f(Hz)} \tag{5}$$

where f_1 and f_2 and Δf are predominant and intervals between resonant frequencies. The compression wave velocities of 16,000 ft/s or 12,000 ft/sec in the numerators correspond to the steel rod or grout column, respectively.

See Table 2. Predominant and Resonant Frequencies Observed from Mobility Curves

The lengths depicted in Table 2 can be compared to the geometries of the installations described in Table 1. Measurements of L_{stick} are close to those observed from the installations. Careful inspection of the measurements of L_{bond} and L_{free} indicates that these measurements are indeed affected by the presence of voids placed within the grout of the distressed elements. The bonded and free lengths measured from the normal installations correspond to the lengths described in Table 1. The measurements of L_{bond} and L_{free} from the distressed elements differ due to the

locations of the voids placed in the grout along the bonded zone. Voids placed near the leading edge of the bonded zone create a response that indicates the bonded zone is shorter and the free length is longer compared to the normal installations. The void in the middle of the bonded zone for the 20 feet long distressed rock bolt installation appears as a significant reduction in the measured bonded length, which is due to the loss of energy at this discontinuity. Thus, the data depicted in Table 2 demonstrate how characteristics of the mobility curves can be compared to those expected for normal installations to locate anomalies that may be due to distress or problems with the installation.

ULTRASONIC

Results from ultrasonic testing from samples 4,5 and 6 are compared in Figures 4(a), 4(b) and 4 (c), respectively. Results from samples 4 and 5 exhibit reflections at approximately 0.2 ms that correlate well with the length of the stick-up ((1.7 ft x 2)/16000 ft/sec ≈ 0.2 ms). For sample #6 reflections at approximately 0.35 ms are apparent in addition to reflections corresponding to the stick-up (≈ 0.2 ms). These additional reflections appear to correspond to the location of the cut slot in the bar near the anchor head, ≈ 1 ft. behind the bearing plate. Thus, the ultrasonic test successfully identified the presence of a defect in the steel near the anchor head, however appearance of the reflection is very subtle.

See Figure 4. UT Results for Bolts #4, #5 and #6

ELECTROCHEMICAL TESTS

Electrochemical tests, including measurements of half-cell potential and liner polarization resistance (LPR), were performed on each sample. Results from testing are summarized in Table 3. These results reflect conditions within the bonded zone of the rock bolts where there is electrical continuity between the steel rods, the grout and the surrounding rockmass. Half-cell potential measurements range from –1186 mV to –947 mV, which is consistent with the half-cell potential of zinc present on the surface of the galvanized steel rods. Corrosion rates computed from measurements of LPR are less than 6 μm/yr.

See Table 3. Summary of Electrochemical Test Results for Dummy Rock Bolts

Details of the installations for the dummy rock bolts are well known, and therefore the surface areas needed to compute corrosion rates from measurement of LPR are readily available. For in service reinforcements installation details are less certain, and surface areas in the bonded zone are estimated. However, results from impact testing indicate that the length of the bonded zone may be observed based on the clear reflections and/or resonance associated with the interface between the free length and the bonded/grouted zone. Thus, the bonded length can be computed if the total length of the rock bolt is also known or observed.

Table 1. Details of "Dummy" Rockbolts Installed at the Barron Mountain Rock Cut

#	Lengths[1] (ft)				Preload (kips)	Description
	L_T	L_f	L_b	Stick -up		
1	15	10	5	1.50	40	Normal installation.
2	20	15	5	1.92	20	(1) 5 inch long void in grout at beginning of bonded zone. (2) 9 inch long void in grout near middle of bonded zone.
3	10	7	3	1.33	10	Normal installation.
4	20	15	5	1.77	40	Normal installation.
5	15	10	5	1.78	20	(1) 9 inch long void in grout at beginning of bonded zone. (2) 10 inch long void in grout, 3 inch from low end of bonded zone
6	10	7	3	1.72	10	(1) Cut slot in bar near anchor head, ≈ 1 ft. behind bearing plate. (2) Cut 3" inch notch in sleeve along free length, ≈ 5 ft. behind bearing plate.

[1] L_T = total length, L_f = free length, L_b = bonded length, stick-up refers to the length of the threaded rod protruding beyond the bearing plate to facilitate lift-off and proof testing.

Table 2. Predominant and Resonant Frequencies Observed from Mobility Curves

L_T (ft)	Condition	Stick-up		Bonded Length		Free Length	
		f_1 (Hz)	L_{stick} (ft)	f_2 (Hz)	L_{bond} (ft)	Δf (Hz)	L_{free} (ft)
10	Normal	3000	1.33	2200	2.73	1400	5.71
10	Distressed	2400	1.66	2400	2.50	1000[1]	8.00
15	Normal	2800	1.42	1200[2]	5.00	800	10.00
15	Distressed	2400	1.66	1800	3.33	600	13.33
20	Normal	2200	1.82	1400	4.29	500	16.00
20	Distressed	1800	2.22	2200	2.73	600	13.33

[1] also observed a predominant frequency at 1400 Hz corresponding to a length of 5.71 ft and the approximate location of the notch created along the sleeve for the 10 feet long, distressed element as described in Table 1.

[2] very subtle break evident in mobility plot that appears much more clearly in the mobility curve from the corresponding 15 feet long, distressed installation.

Table 3. Summary of Electrochemical Test Results for Dummy Rock Bolts

L_T (ft)	L_b (ft)	Condition	Sample #	CR (μm/yr)	E_{corr} (mV)
10	3	Normal	3	5.4	-1064
10	3	Distressed	6	5.7	-965
15	5	Normal	1	2.5	-1147
15	5	Distressed	5	3.0	-947
20	5	Normal	4	2.2	-1186
20	5	Distressed	2	3.3	-1009

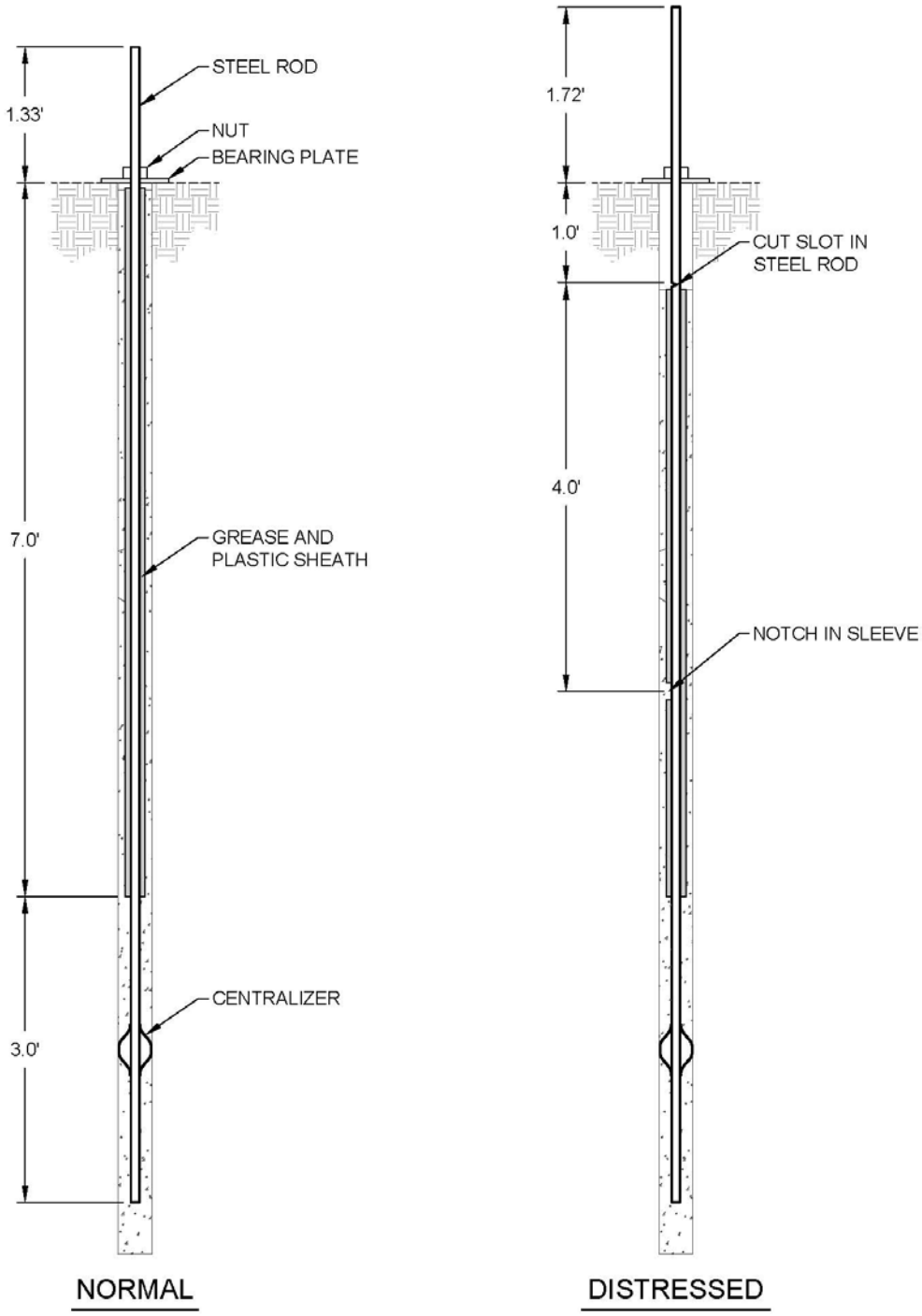


Figure 1(a) Sketch of 10’ long dummy rock bolt installations.

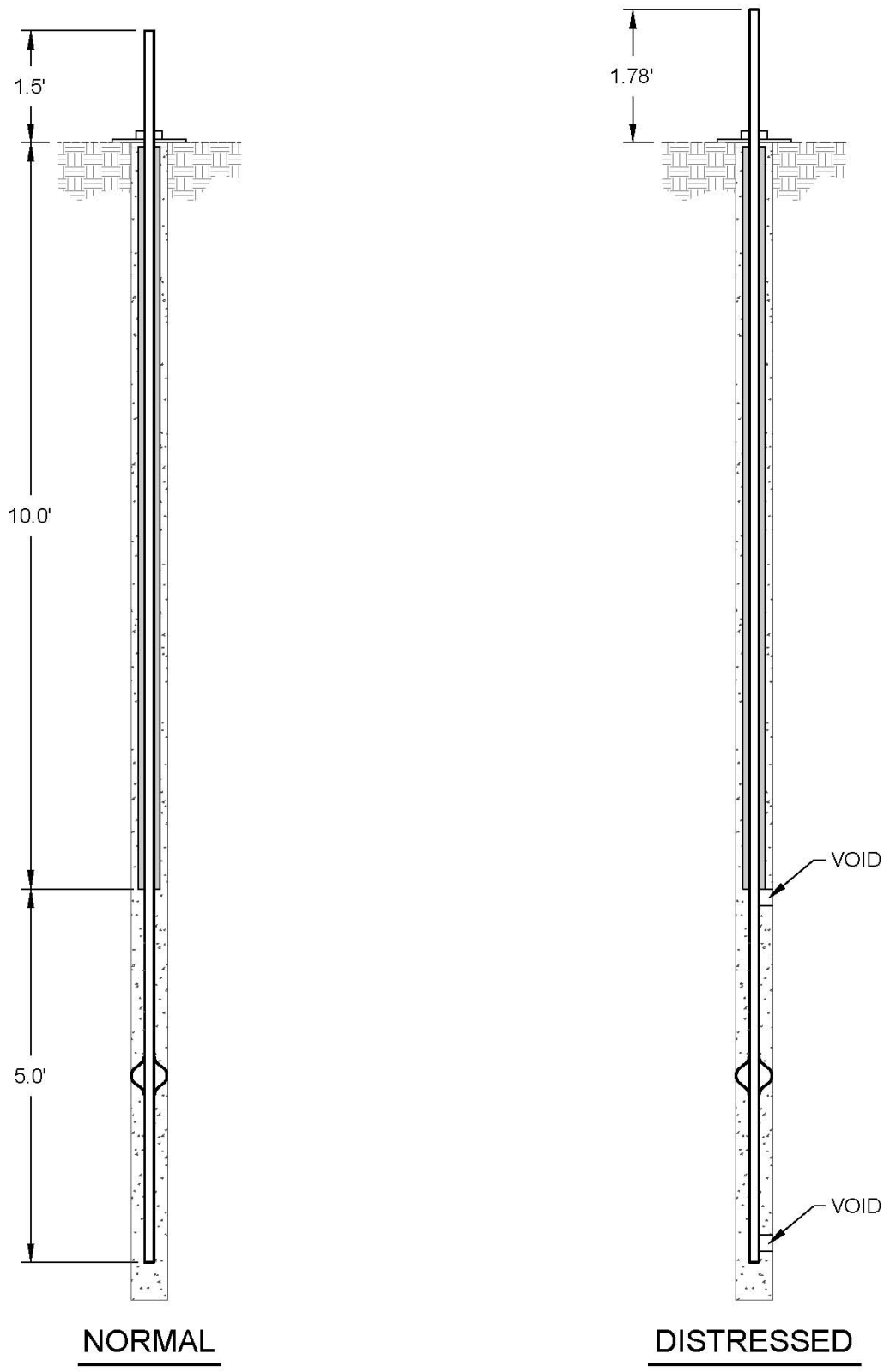


Figure 1(b) Sketch of 15’ long dummy rock bolt installations.

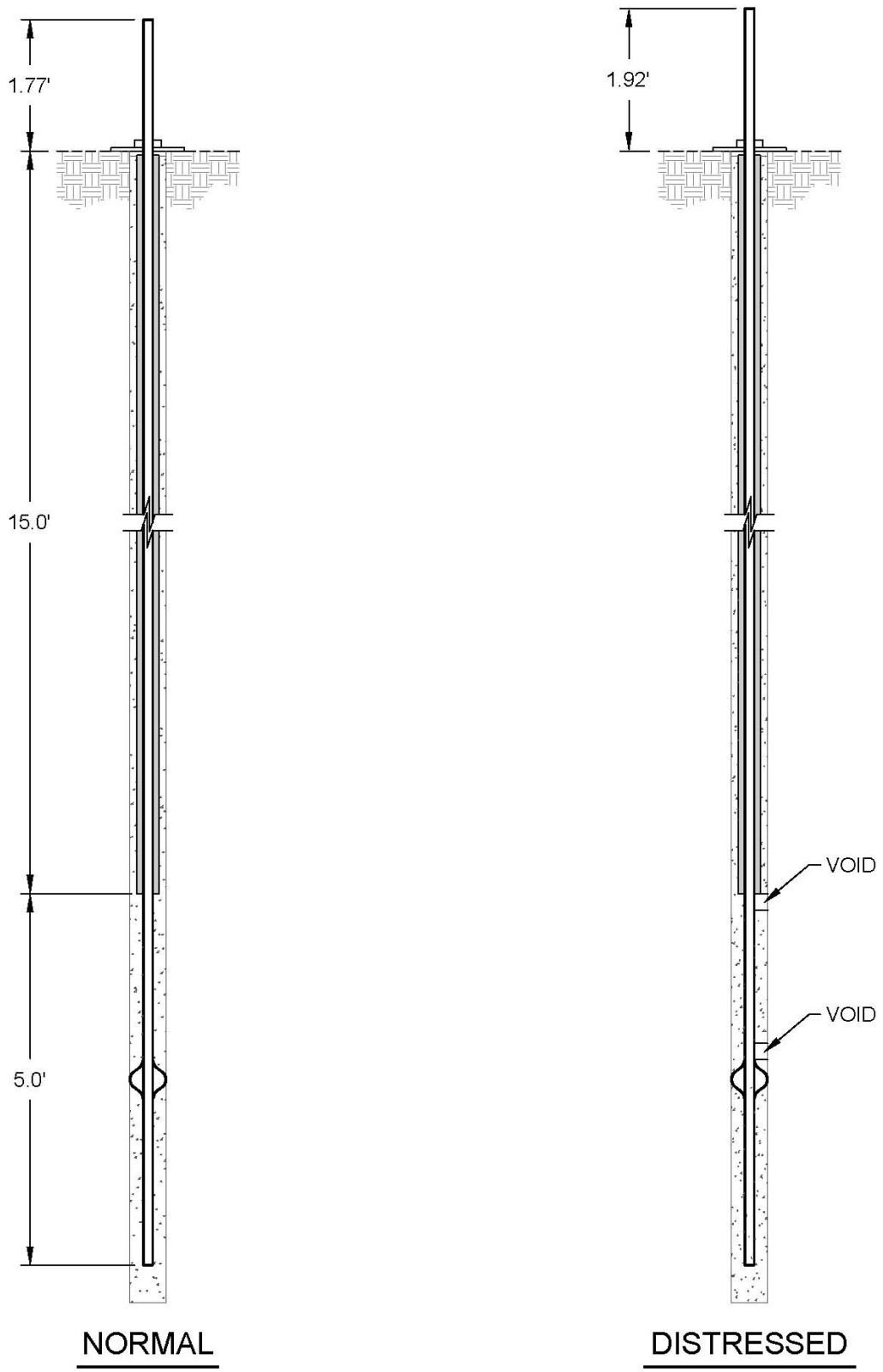


Figure 1(c) Sketch of 20’ long dummy rock bolt installations.

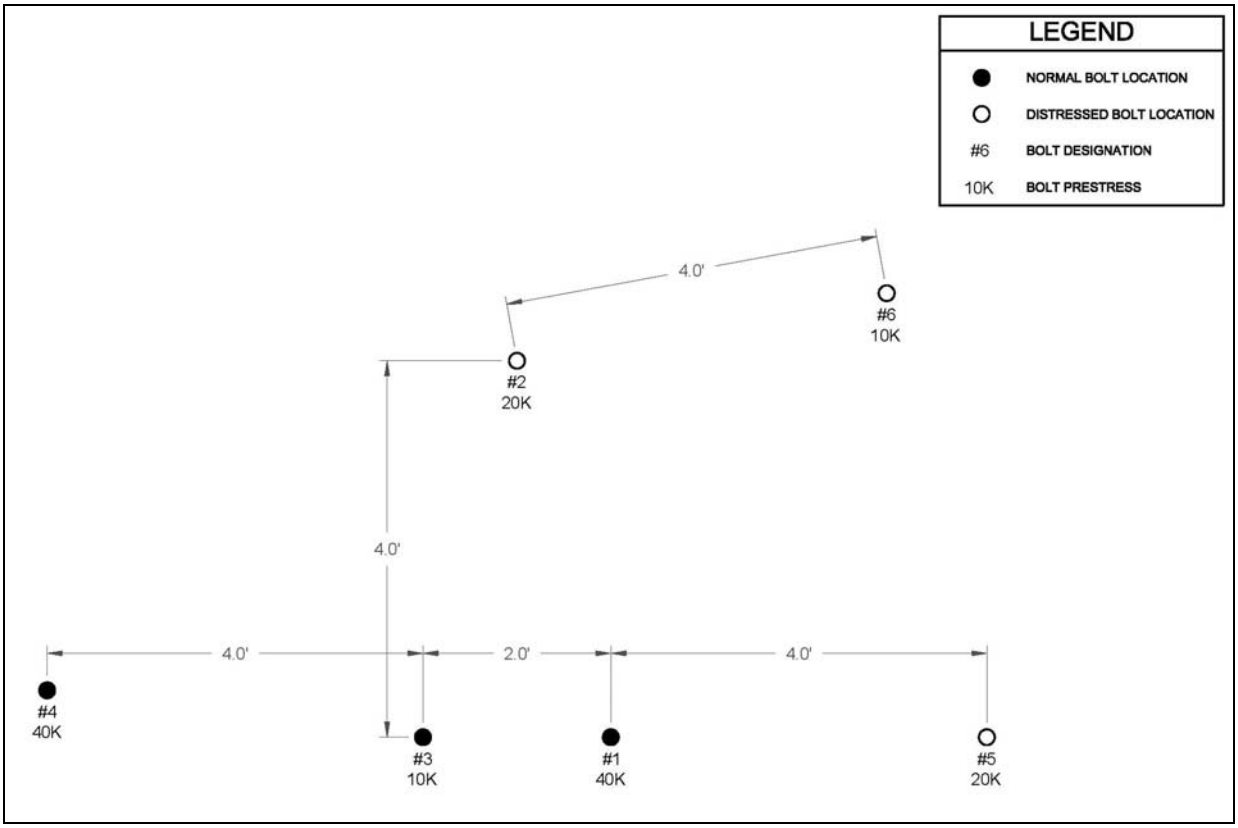


Figure 2. Locations (Elevation View) of dummy rock bolts 1-6.

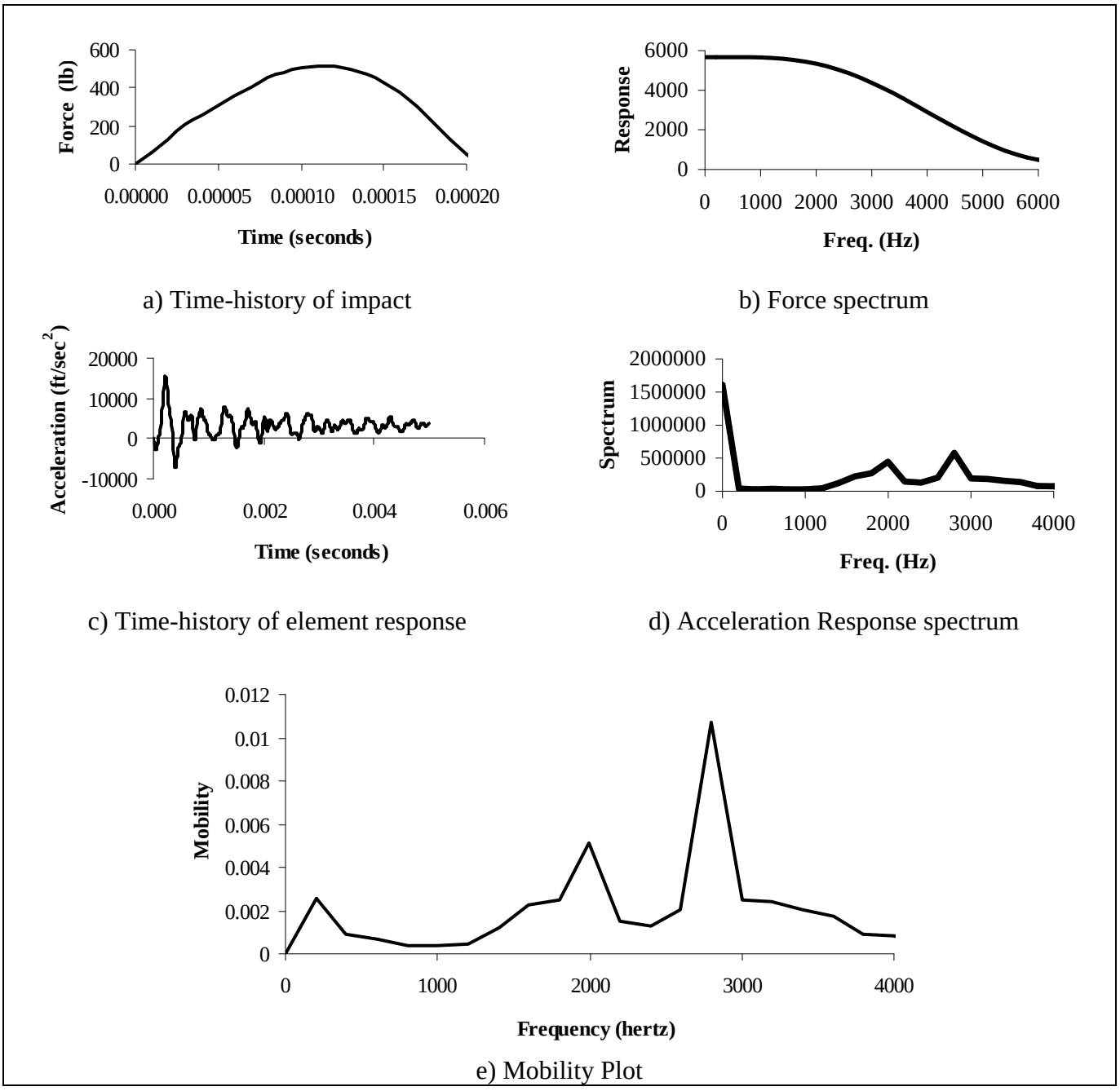


Figure 3. Typical IR Test Results

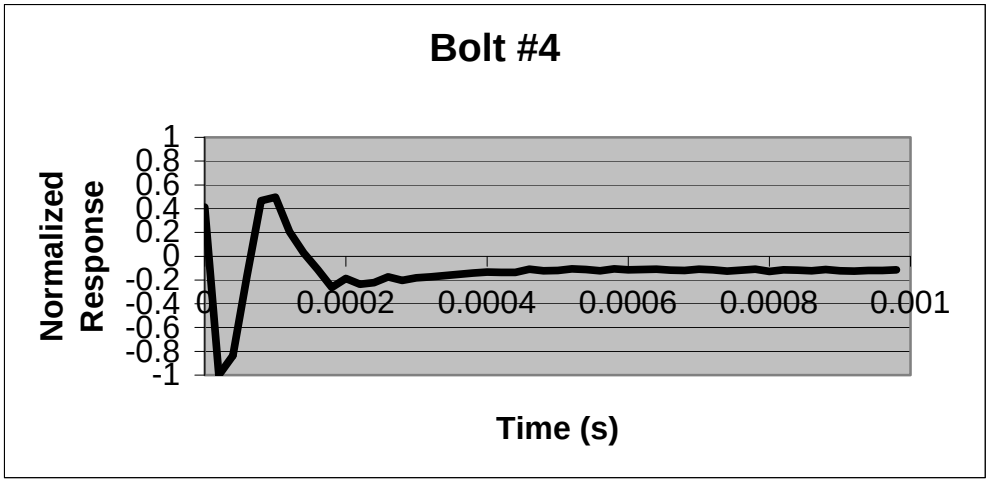


Figure 4a) UT Results for Bolt #4

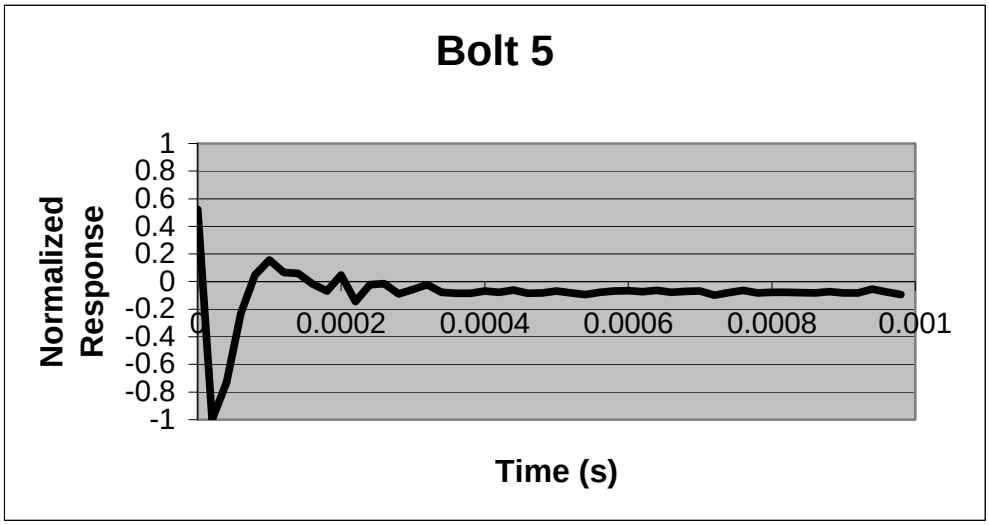


Figure 4(b). UT Test Result for Bolt #5

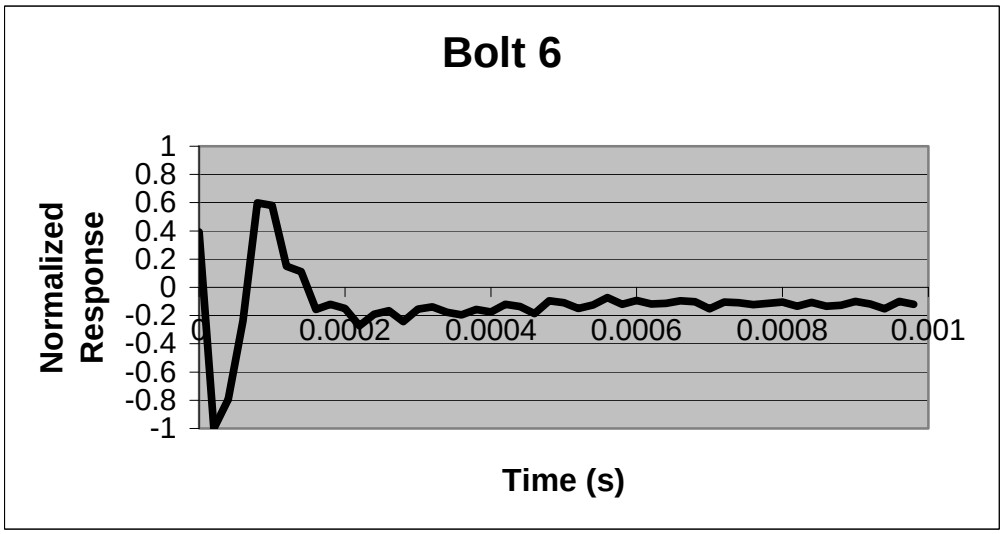


Figure 4(c). UT Test Result for Bolt #6.
Figure 4. UT Test Result for Bolts #4, #5 and #6

CHAPTER 4. IN SITU RESISTIVITY MEASUREMENTS

This task is to develop the ability to obtain information on backfill conditions at the time and locations of corrosion rate measurements. Soil resistance is measured as part of the electrochemical test protocol, and these measurements may be related to resistivity by means of an appropriate factor, which is related to the geometry and configuration of the test electrodes; similar to the manner in which soil resistivity is measured in the field via the Wenner probe test (ASTM G57). Resistivity is an important electrochemical property of reinforced fill that may vary with respect to fill conditions including moisture content, salt intrusion etc. Reliable, targeted, and timely measurements of resistivity are necessary to assess the impact of marginal backfill materials on the service life and performance of earth reinforcements. The database developed for NCHRP 24-28 needs additional, reliable, data on backfill condition for correlation with measured corrosion rates; particularly to identify and quantify the quality of reinforced fill in the "marginal quality fill" category.

We studied measurement of soil resistance during electrochemical testing under controlled conditions in an MSE test embankment using a variety of test parameters, techniques and geometric configurations. The goal of the study is to identify appropriate factors to relate resistance measurements to resistivity for geometries representative of typical MSE installations. Additional field verification of these factors is addressed as part of Task 3.

CONSTRUCTION OF EDI TEST EMBANKMENT

Figure 5 depicts the geometry of the test embankment and placement of replicate soil reinforcements and coupons for testing. Appendix II includes photographs depicting construction of the embankment. The test embankment was constructed on a lot owned by Earth Dimensions, Inc. (EDI), located in Elma, New York.

See Figure 5. Cross Section of EDI Test Embankment and Placement of Dummy Reinforcements and Coupons

Different fill materials were used on either side of the embankment centerline; both meeting AASHTO requirements for MSE wall fill. These materials were obtained from a nearby sand and gravel supplier and are representative of fill materials used for MSE construction. Fill materials included a finer material often used as mortar sand (Material #1), and a courser material often used as a base course for driveways (Material #2). Samples of Materials #1 and #2 were submitted to Geotechnics Geotechnical Engineering Laboratory for grain size analysis, determination of optimum moisture content and maximum density, and electrochemical testing including measurement of pH, resistivity and salt content. Material #1 is a brown, poorly graded, medium grain, sand classified as SP according to the unified soil classification system; and Material #2 is a dark gray, well graded, coarse grain, silty sand with gravel classified as SW-SM.

Figure 6 present results from grain size analysis of each material, compared with AASHTO specified limits. Table 4 is a summary of optimum moisture content, maximum density and electrochemical test results from samples of each material. Detailed data and results from

Geotechnics are presented in Appendix II. Although Table 4 presents the minimum resistivity, ρ_{min}, the relationships between moisture contents and resistivities, depicted in Appendix II are of particular interest to compare with in situ moisture contents measured during construction of the test embankment.

See Figure 6. Grain Size of Materials #1 and #2 for EDI Test Embankment

See Table 4. Parameters for Fine Sand and Fine Gravel Used for the Test Embankment

Samples of reinforcements and coupons were placed within the embankment during construction at locations depicted in Figure 5 and detailed in Appendix II. Reinforcement samples were ten feet long, and one-foot long coupons were prepared from the same sources of reinforcements. Both strip and grid type reinforcements were obtained from the Reinforced Earth Company. Ribbed strip reinforcements were manufactured from galvanized steel meeting the requirements of ASTM A572 Grade 65, with a width of 50 mm and thickness of 4 mm. Grids were W11 x W11 x 6 in x 6 in. (longitudinal wire x transverse wire x longitudinal spacing x transverse spacing) manufactured from galvanized, cold drawn wire meeting the requirements of ASTM A82. Grids included five longitudinal wires, and were two-feet wide. Coupons from the grids were 1 feet square, and included three, one-foot long longitudinal wires, and three one-foot long transverse wires. Additionally two, ten-feet long single wire samples were cut from the grids, to replicate the single wire inspection elements used by Caltrans at many sites in California.

The embankment was constructed in six-inch lifts that were compacted with a self-propelled, vibrating plate tamper. 3rd Rock Inc. performed in situ measurements of moisture content and density during construction using a nuclear density gage. At least two measurements were obtained from each lift for both Materials #1 and #2. Detailed data from construction observations are presented in Appendix II, and Table 5 is a summary of the in place density measurements.

See Table 5. Summary of In-place Density measurements.

MMCE collected samples from each lift and performed laboratory measurements of moisture contents in accordance with AASHTO Test Method T-265. Laboratory measurements of moisture contents ranged from 7.1 % to 11.5% and 5% to 11.7% for Materials #1 and #2, respectively. These ranges which are consistent with the ranges of in-situ measurements as shown in Table 5.

MMCE also performed resistivity measurements on samples of material retrieved during construction using a test box, similar to that described in AASHTO T-288, and samples of material retrieved during construction. Corresponding measurements of moisture contents and dry density were also determined from the same samples as placed in the test box. Results are summarized in Table 6, which indicates that the densities of materials placed in the test box were less than the in situ density. The objective of the onsite testing was to obtain samples and test as close to the as-placed moisture content as possible. Therefore samples were not air dried prior to testing, and distilled water was not used to adjust the moisture as described in AASHTO Test

Method T-288. Wet samples could not be effectively separated with a No. 10 sieve, so samples were separated into fractions for testing with a No. 4 sieve.

See Table 6. Resistivity Measurements from Samples Retrieved During Construction.

For Material #2 the measurements shown in Table 6 compare well with the measurements made by Geotechnics with respect to determining the minimum resistivity of the material. Although ρ_{min} for Material #2 is 3400 Ω-cm, this corresponds to a moisture content of 25% percent with the material compacted in the test box to a unit weight of approximately 100 pcf. Detailed data from Geotechnics included in Appendix II indicates that laboratory measurements of resistivity at moisture contents between 9.6 and 11.7 percent ranges between 7000 Ω-cm and 11,000 Ω-cm. The comparison is less favorable for Material #1, which was tested by Geotechnics at a density of approximately 107 pcf. At moisture contents between 8.4 and 9.5 percent, Geotechnics measured resistivity in the range of 45,000 Ω-cm to 50,000 Ω-cm, and this does not compare as well with the data presented in Table 6.

RESISTANCE MEASUREMENTS

Different test techniques and parameters were evaluated to assess their impact on in situ measurement of soil resistivity. Different test techniques are distinguished by the frequency of excitation and electrode configuration. Tests were performed at an excitation frequency of zero corresponding to DC measurements, and AC measurements employing square waveforms at frequencies of 97 Hz, similar to the Wenner probe test, and at higher frequencies up to 100 kHz corresponding to zero phase shift between current and voltage measurements (i.e., in the absence of effects from capacitance at the interfaces between the fill and metal surfaces) as determined from electrochemical impedance spectrometry (EIS).

These test techniques were applied using either two or three electrode configurations similar to corrosion rate measurement techniques described by Elias et al. (2009), and Fishman and Withiam (2011). For the two electrode configurations, currents are impressed between a pair of reinforcements or coupons. One element serves as the working electrode and the second serves a both a counter and a reference electrode. A copper/copper sulfate half-cell is added to the circuit, and serves as the reference electrode in the three-electrode configuration.

Test parameters including the sizes and shapes of electrodes, depths to the electrodes from the ground surface (i.e., boundary effects) and spacings between electrodes (elements under test) were also varied. Various sizes and shapes are intended to represent different types of elements installed in the test embankment. Ten feet lengths represent “dummy” reinforcements or inspection elements, and one-foot lengths correspond to coupons. Various combinations of working and counter electrodes, and half-cell placements allowed the effects of electrode spacings and boundary effects to be evaluated. The orientation of current path could also be varied with respect to vertical or horizontal directions. Detailed results from measurement of in situ resistivities are presented in Appendix II.

Equations describing current flow through a homogeneous, isotopic, semi-infinite half space provide a theoretical basis to compute factors that relate direct measurements of total resistance

to resistivity of the surrounding fill material. The purpose of the test embankment is to verify the validity of the solution as applied to configurations of reinforcements and different fill materials used for MSE construction. The validation is performed over the range of parameters previously described, and incorporated into the test embankment.

CALCULATION AND VERIFICATION OF RESISTIVITY

Analytical solutions are available from the literature to model either the three electrode or two electrode configurations. Resistivity may be computed when the resistance of a single element, idealized as a rod, and its length and width are known. The three-electrode configuration is a means to measure the resistance of the working electrode with respect to a stable reference, thus rendering the resistance of a single element. For the two-electrode configuration solutions are available describing the response of a pair of parallel rods using the length and width of the working electrode, the spacing between the working and counter electrodes, and the measured soil resistance as input. Given the width to spacing ratio of the ten feet long grid elements installed in the test embankment, the flow of current between these elements may be considered to occur in the vertical direction only, and modeled using the formula for current flow in a single direction, similar to the way results from testing soil in a test box are interpreted. Equations relating measurements of resistance to resistivity and results from the test embankment for the two and three electrode configurations are summarized in the following paragraphs.

For the three electrode configuration and the resistance of a single element (AEMC, 2003):

$$\rho = \frac{2 \times \pi \times L \times R}{\left[\ln\left(\frac{8 \times L}{D} \right) - 1 \right]} \tag{6}$$

where L and D are the electrode length and diameter, respectively, and R is the measured resistance. The resistance of one element is measured with respect to a copper/copper sulfate half-cell placed near the base of the test embankment. The half-cell is placed at a sufficient distance such that its presence does not affect the resistance measurement. Resistance is determined from the slope of the voltage vs. applied current in response to the impressed current applied between the working and counter electrodes. For single wire elements the diameter corresponds to the width, and the widths of the strip or grid elements and the lengths of all the elements are also known.

The resistance measurements presented in Appendix II indicate that resistance is nearly constant considering the same length and diameter of the test element with respect to different placements of the half-cell at distances from the working electrode ranging between approximately 250 cm and 675 cm. Furthermore, for the same geometry (i.e. element shape or idealized diameter and length) the measured resistances between Material #1 and Material #2 appear to scale with respect to the known resistivities of these two materials.

Similar results were obtained from testing at different frequencies; therefore only results from testing at the lower frequency of 97 Hz are presented in the following discussion. Table 7 is a summary of resistance measurements from the three-electrode configuration and computations of

resistivity using Equation (6) for elements located below a depth of two feet from the surface of the embankment, and not affected by the boundary conditions inherent to the test embankment. The means of the computed resistances relative to Materials #1 and #2 are 19,715 Ω-cm and 10,126 Ω-cm, respectively; which compare very well with results via the test box as presented in Table 6.

See Table 7. Calculations of ρ from Measured Resistance of a Single Element –Three Electrode Configurations

For grid elements and resistance measured from the two-electrode configuration:

$$\rho \ (\Omega\text{-cm}) = R\ (\Omega) \times [A_s\ (cm^2)/\ S\ (cm)] \qquad (7)$$

where A_s is the surface area of the grid serving as the working electrode, and S is the spacing between the grids serving as working and counter electrodes. Results obtained from the two-electrode technique (2-Point Method) and ten feet long grid reinforcement are presented in Table 8. Results from Table 8 compare very well with those presented in Table 7. This is a valuable comparison because although the measurements were made via different techniques and resistivities computed with different equations, similar result are obtained. Thus, methods to measure resistivity via resistance measurements from corrosion monitoring appear to be robust.

See Table 8. Calculations of ρ from Measured Resistance Between Wide Grids – Two Electrode Configurations

Task 2 included measuring in situ resistance under various conditions and with various techniques, and comparison with direct measurements of resistivity via soil samples tested in the test box to identify applicable relationships between measured resistance and soil resistivity. Results from this study indicate that in situ measurement techniques are robust, and demonstrate application of analytical techniques to discern resistivity from resistance measurements. Task 3 will further demonstrate the robustness of this technique using measurements from inspection elements and in situ reinforcements from in service MSE walls. Task 3 is important to study application of measurement techniques and data interpretation at large scale compared to the test embankment, and with respect to dimensions and spatial relationships inherent to in service MSE walls. Also, additional variations with respect to fill types and in situ conditions are available from the field.

APPLICATION OF IN SITU RESISTIVITY MEASUREMENTS

Some data on the field performance of earth reinforcements from sites in California were obtained as part of NCHRP 24-28. However, opportunities to collect performance data from sites within California have not been fully exploited. This task is to obtain further data in cooperation with Caltrans, and to take advantage of the inspection elements that Caltrans has installed as part of MSE construction since the mid 1980's. These inspection elements are dispersed throughout California, and present opportunities to collect data from different climates, site conditions, fill materials etc. Many of the regions in California do not have readily available sources of fill

material that meet AASHTO's stringent electrochemical requirements. Therefore, marginal quality fills were used at many of these locations. This provides an opportunity to extend the database developed for NCHRP 24-28 considering marginal fills. Caltrans has developed procedures to retrieve inspection rods subsequent to electrochemical testing and this provides opportunities to verify results from NDT. Samples of reinforced fill may be obtained after inspection rods are exhumed. Laboratory resistivity measurements from these samples are used to further verify the methods and equations that relate resistance measurements to resistivity as described in Task 2.

CALTRANS SITES

MMCE visited five sites in California, as described in Table 9, in cooperation with Caltrans during the winter and spring of 2010. Figure 7 is a map depicting the site locations and pictures of each site are included in Appendix III. The first four sites listed in Table 9 are MSE walls that incorporate galvanized steel reinforcements, and the fifth site (Cypress Bunker) is a corrosion test site used for NCHRP Project 21-06 (Corrpro, 2009).

See Table 9. Summary of Sites in California with Observations from 2010

See Figure 7. Locations of Sites in California Listed in Table 9.

Three of the MSE walls, including those located at Gilroy, San Luis Obispo and San Gabriel, were constructed with strip type reinforcements, and grids were installed at the Los Olivos site. All of the walls, except the oldest wall in San Gabriel included ten feet long inspection elements that were either 4 mm thick, 50 mm wide, ribbed, strips representing strip type reinforcements, or W11 cold drawn wire at Los Olivos where grid type reinforcements are used. The fourth site (San Gabriel) did not include inspection elements, but we were able to access a few reinforcements near the top of the wall by excavating. The Cypress Street Bunker includes specially designed corrosion monitoring probes that were installed at various depths as part of NCHRP Project 21-06. The ages of the retaining walls ranged between 14 and 39 years old at the time of monitoring, and the corrosion monitoring probes at the Cypress Street Bunker had been in the ground for approximately five years.

The retaining wall along SR 39 in San Gabriel CA was constructed in 1971, making this one of the oldest MSE walls in North America. We unearthed three reinforcements that were located within the top five feet of the wall. We made visual observations on the exposed portions of the reinforcements that included the first five feet from the wall face. Photographs of the exposed portions of the reinforcements are depicted in Appendix III. Ferrous oxide was observed on the outside surface of the steel skin face, but the facing was not perforated.

We isolated the reinforcements and performed electrochemical testing (linear polarization resistance technique) to probe the lengths of the reinforcements that were not unearthed, and observed corrosion rates between 2 and 3 microns per year. Samples of fill were retrieved and tested for resistivity in the field using a soil box.

DATA COLLECTION

MSE Walls

Detailed data from LPR and soil resistance measurements are included in Appendix III. Table 10 is a summary of results from corrosion monitoring and measurement of soil resistance at the four sites with MSE walls. Table 10 provides a comparison between laboratory measurements of resistivity from samples of fill retrieved from the site during monitoring, measurements of resistivity performed onsite using a test box and the same fill samples, calculations of fill resistivity based on resistance measurements from the three-electrode configuration, and corrosion rates determined from LPR measurements.

See Table 10. Comparison of Lab, Field and In Situ Resistivity Measurements

Results from testing samples of fill obtained from the Gilroy site using a test box include minimum resistivity (ρ_{min}) determined by Caltrans in the laboratory, and measurements performed by MMCE in the as-received moisture content. These results compare reasonably well considering that the as-received moisture content is less than the moisture content at, or near saturation, corresponding to the measurement of ρ_{min}. In situ measurements using the three-electrode configuration, which included measurements from eleven inspection elements, rendered a range of results that compare reasonably well with the measurements obtained using the test box and the as-received moisture content.

Measurements are not available from fill samples in the as-received condition from the San Luis Obispo site. However, given the low in situ moisture content of 1.7%, the range of in situ measurements of resistivity are higher than the measurement of ρ_{min}, so this appears to be a reasonable comparison. For the San Gabriel site measurements of ρ_{min} are not available, but the comparison between measurements from the test box at the as-received moisture content and in situ measurements is excellent. Samples of fill are not available from the Los Olivos site but the in situ measurements of resistivity are useful for comparison with measurements at other sites, and corresponding measurements of corrosion rate.

In situ measurements of resistivity were also obtained using the two-electrode configuration. Results of resistivity from the two-electrode configuration do not compare very well with results from the three-electrode configuration. This may be due to the presence of in service reinforcements between the inspection elements.

Table 10 also provides a comparison between in situ measurements of resistivity and corrosion rate for the four sites. Corrosion rates are negatively correlated with in situ resistivity, and higher corrosion rates are associated with sites having lower ranges of in situ fill resistivity measurements. Thus, higher corrosion rates were observed at the Los Olivos site, which included the fill with the lowest in situ measurement of fill resistivity.

Corrosion rate measurements at the first three sites listed in Table 10 appear to correspond to rates of zinc loss. The ages of the reinforcements/inspection elements at these sites range between 14 and 20 years at the time measurements were taken. Given the range of fill

resistivities and corrosion rates that were measured, one would not expect the zinc coating with a minimum thickness requirement of 86μm to be consumed within 20 years. However, at San Gabriel, which is the oldest site where reinforcements were approximately 39 years old at the time of measurement, the zinc coating has been consumed along portions of the surface. This condition is consistent with observations of half-cell potential depicted in Table 11. The half-cell potentials of reinforcements at San Gabriel are higher than those from the other three sites and are at levels corresponding to the base steel. Photographs of the surfaces of reinforcements that were exposed at San Gabriel, and presented in Appendix III, indicate that blotches of ferrous oxide are present on the surface where the zinc layer has been consumed and corrosion of the base steel has been initiated. Furthermore, measurements of remaining zinc performed by Caltrans on samples of inspection strips retrieved from San Luis Obispo, and included in Appendix III, indicate that 6.4 oz/ft^2 of zinc remains on the surface corresponding to an average thickness of approximately 275 μm; well above the minimum requirements for the initial thickness. This confirms that indeed zinc is present on the surface of the inspection elements at San Luis Obispo, and correlates well with the low half –cell potential measured at this site.

See Table 11. Reinforcement Age and E_{corr} Measurements from Sites in California

Thus, measurements of corrosion rate obtained from San Gabriel are extremely interesting and are some of the first confirmed observations of the rate of steel consumption subsequent to depletion of zinc for galvanized reinforcements used in MSE construction in the United States. These rates are much less than those used in design and appear to be closer to the corrosion rates used for zinc, implying that a discrete change in corrosion rates as zinc is depleted, as implied by the current AASHTO model for metal loss of MSE reinforcements, does not necessarily occur.

Cypress Street Bunker

At the Cypress Street Bunker (Emeryville), corrosion monitoring and measurement of fill resistance were performed using corrosion probes that were specially designed and installed as part of NCHRP Project 21-06. Two types of probes were employed including disposable multipurpose probes (DMP) and cylinder probes as depicted in Figures 8 and 9. The DMPs include four, two-inch long, 0.2 inch thick, carbon steel strips cast into an epoxy block such that the strips are parallel with a spacing of 0.25 in (0.635 cm). Cylinder probes include four, 1.6-inch diameter, 0.25-inch long, rings spaced at 0.25 inches along the axis of a cylindrical probe. DMPs are placed in excavations advanced to depths of four and eight feet, and the cylinder probes are driven to depths of thirty feet. A DMP is also attached to the cylinder probe for redundant measurements at the thirty-foot depth. Both instruments allow measurements of corrosion rate and soil resistance by using one strip, or ring, as a working electrode, and the nearest parallel strip or ring as a counter electrode. Half-cells are placed on the soil surface to act as a reference electrode in the three-electrode configuration.

See Figure 8. Illustration of Cylinder Probes Developed for NCHRP Project 21-06
(Corrpro, 2009)

See Figure 9. Illustration of DMP Developed for NCHRP Project 21-06
(Corrpro, 2009)

The Cypress Street Bunker is located in Emeryville, CA in a marine environment near the shore of San Francisco Bay. The site is covered with fill that includes slag to a depth of approximately eight feet. Below the fill the site is underlain by soft clay soil known as San Francisco Bay Mud. Groundwater is brackish and subject to tidal fluctuations. The four-foot depth is above groundwater, the eight-foot depth is subject to periodic inundation, and the thirty-foot depth is below the groundwater table. Table 12 is a summary of laboratory measurements of soil properties performed on soil samples retrieved during installation of the corrosion monitoring probes as reported for NCHRP Project 21-06. Table 12 depicts how the moisture contents, salt contents and resistivity measurements are affected by depth and the presence of brackish groundwater.

See Table 12. Soil Properties Measured at the Cypress Bunker, Emeryville, CA

Measurements of corrosion rate and soil resistance were performed using a variety of instruments including the Aquamate (Corrpro, 2009), FHWA PR monitor, the Gamry 300 and Gamry 700 Potentiastats, and a Nilsson meter. Tests were performed with two- and three-electrode configurations. Detailed data from these measurements are included in Appendix III. Table 13 is a summary of data and analysis of results from the two-electrode configuration.

See Table 13. Comparison of Laboratory and In situ Measurements of Resistivity from Coupons at the Cypress Corrosion Test Site

At the time of measurements the site was relatively dry and this is reflected in the relatively high resistivity and low corrosion rates measured at the four-foot elevation. Resistivity is lower at the depth of eight feet and the corresponding corrosion rate is considerably higher compared to the four-foot elevation. At depths of 30 feet, corresponding to the submerged Bay Mud, resistivity is in the range of salt water and measured corrosion rates are extremely high. These trends are consistent with the laboratory measurements of resistivity in the as-received condition displayed in Table 12; considering the in situ moisture content at the depth of 4 feet is probably lower then 5.9%. Thus, the in situ measurements of corrosion rates reflect the site conditions and demonstrate that the corrosion monitoring probes can render reasonable results that make sense compared to laboratory measurements of resistivity performed on soil samples retrieved during installation of the probes.

Table 4. Parameters for Fine Sand and Fine Gravel Used for the Test Embankment

Fill Type	Modified Proctor		Electrochemical Properties			
	γ_{max} (lb/ft^3)	w_{opt} (%)	pH	ρ_{min} @ w%[1] (Ω-cm)	SO_4 (ppm)	Cl^- (ppm)
Mat #1	127.7	10.2	8.2	17,500@20%	<50	<50
Mat #2	139.6	6.9	8.4	3400@25%	140	< 50

[1] minimum resistivity and corresponding moisture content

Table 5. Summary of In-place Density measurements.

Mat. #	Description	Unit Wgt. (lb/ft^3)	Percent Compaction	w (%)	Comments
1	medium grained sand	110.6 to 118.2	87 % to 93 %	8.1 to 11.6	within approximately 2 percentage points of optimum; with most points slightly dry of optimum
2	coarse grained sand	123.8 to 133	89 % to 95 %	5.2 to 9.5	within approximately 2 1/2 percentage points of optimum; even distribution between dry and wet of optimum

Table 6. Resistivity Measurements from Samples Retrieved During Construction.

Mat. #	Description	Unit Wgt. (lb/ft^3)	w %	Resistivity, ρ at w% (Ω-cm)
1	medium grained sand	107 to 109	8.4 to 9.5	17,500 to 20,110
2	coarse grained sand	107 to 113	9.6 to 11.7	6943 to 7490

Table 7. Calculations of ρ from Measured Resistance of a Single Element – Three Electrode Configurations

Material #1 - Finer Sand - In Situ Measurements With Half-Cell				
Element - Type	Measurements			Computed
	L (m)	d (mm)	R (Ω)	ρ (Ω–cm)
A1 - Single wire	3.05	9.5	47.3	13231
B1- Grid	3.05	610	16	11403
B2 - Grid coupon	0.305	305	150	26630
B3 - Strip coupon	0.305	50	340	22563
B4 - Strip	3.05	50	51	18830
B5 - Strip Coupon	0.305	50	317	21037
B6 - Strip Coupon	0.305	50	402	26678
C1- Single wire –W11	3.05	9.5	79	22098
D1 - Grid	3.05	610	21	14967
Material #2 - Coarser Sand - In Situ Measurements With Half-Cell				
Element - Type	Measurements			Computed
	L (m)	d (mm)	R (Ω)	ρ (Ω–cm)
B1 - Grid	3.05	610	7.5	5345
B2 - Grid coupon	0.305	305	58	10297
B3 - Strip coupon	0.305	50	159	10552
B4 - Strip	3.05	50	21	7754
B5 - Grid coupon	0.305	305	57	10119
B6 - Grid coupon	0.305	305	94	16688

Table 8. Calculations of ρ from Measured Resistance Between Wide Grids – Two Electrode Configurations

WE	Fill #	Reinf. Type	As (cm^2)	S (cm)	Rs (Ω)	Rs X As/L (Ω-cm)
Material #1						
B1F	1	Grid	8047	26	34	10,532
D1F	1	Grid	8047	26	34	10,532
E1F	1	Grid	4024	30	43	11,637
Material #2						
B1C	2	Grid	8263	59	31	4355
E1C	2	Grid	8263	59	31	4355

Table 9. Summary of Sites in California with Observations from 2010

Location					Construction Details			Coordinates	
City	County[1]	Dist #	Rte #	Bridge #	Year Const.	Hgt (ft)	Type	Lat.	Long.
Gilroy	SCL	4	101	37-475G	1990	15	Strip	37.0100	-121.5596
Los Olivos	SB	5	154	51-313M	1996	15	Grids	34.5044	-119.8124
San Luis Obispo	SLO	5	101	49-0231	1992	30	Strip	35.3175	-120.6216
San Gabriel	LA	7	39	NA	1971	55	Strip	34.3389	-117.8532
Emery-ville	SF	4	I-880 S	Cypress Bunker B	2005	NA	DMP[3] & CYL	37.8308	-122.2933

[1]SCL - Santa Clara, SB- Santa Barbara, SLO – San Luis Obispo, LA – Los Angeles, SF – San Francisco
[2] Type refers to reinforcements
[3] DMP – disposable multipurpose probe, CYL – cylinder probe

Table 10. Comparison of Lab, Field and In Situ Resistivity Measurements

Site	As-received MC[1]	Resistivity – (Ω-cm)				Measured Corrosion Rate (μm/yr)	
	w%	Test Box (Test Box)		In situ[2]		Range	Mean
		Caltrans Lab (ρ_{min})	MMCE (onsite)	Range	Mean		
Gilroy	11.6 to 17.3	3536	5181	5169 to 13,661	9600	0.4 to 1.3	0.71
Los Olivos	NA[4]	NA	NA	5035 to 6983	5874	0.8 to 4.6	2.32
San Luis Obispo	1.7	1518	NA	3692 to 13,292	8808	0.5 to 1.4	0.84
San Gabriel	3.0 to 4.0	NA	30,000	20,417 to 43,497	28296	1.9[3] to 2.5[3]	2.28[3]

[1] MC - moisture content
[2] $\rho_{in\ situ}$ from 3-electrode configuration
[3] represents corrosion rate of steel after zinc is consumed
[4] NA – not available

Table 11. Reinforcement Age and E_{corr} Measurements from Sites in California

Site	Age (Years)	E_{corr} (mV)
San Gabriel	38	-410
Gilroy	20	-706
San Luis Obispo	18	-803
Los Olivos	14	-686

Table 12. Soil Properties Measured at the Cypress Bunker, Emeryville, CA

Depth (ft)	w%	pH	CL^- ppm	Resistivity (Ω-cm)	
				As-is	ρ_{min}
4	5.9	8.4	37	12,000	1600
8	6.5	6.0	81	10,000	2000
30	28	7.5	2600	120	120

Table 13. Comparison of Laboratory and In situ Measurements of Resistivity from Coupons at the Cypress Corrosion Test Site

Depth (ft)	Shape	Lab	In Situ				
		ρ_{min} (Ω-cm)	A_s (cm^2)	L (cm)	R_s (Ω)	R_s x A_s/L (Ω-cm)	CR (μm/yr)
4	DMP	1600	2.58	0.635	53,500	217370	2.3
8	DMP	2000	2.58	0.635	1212	4924	87
30	DMP	120	2.58	0.635	6.1	25	>>1000
30	CYL	120	8.11	0.635	2.4	31	>>1000

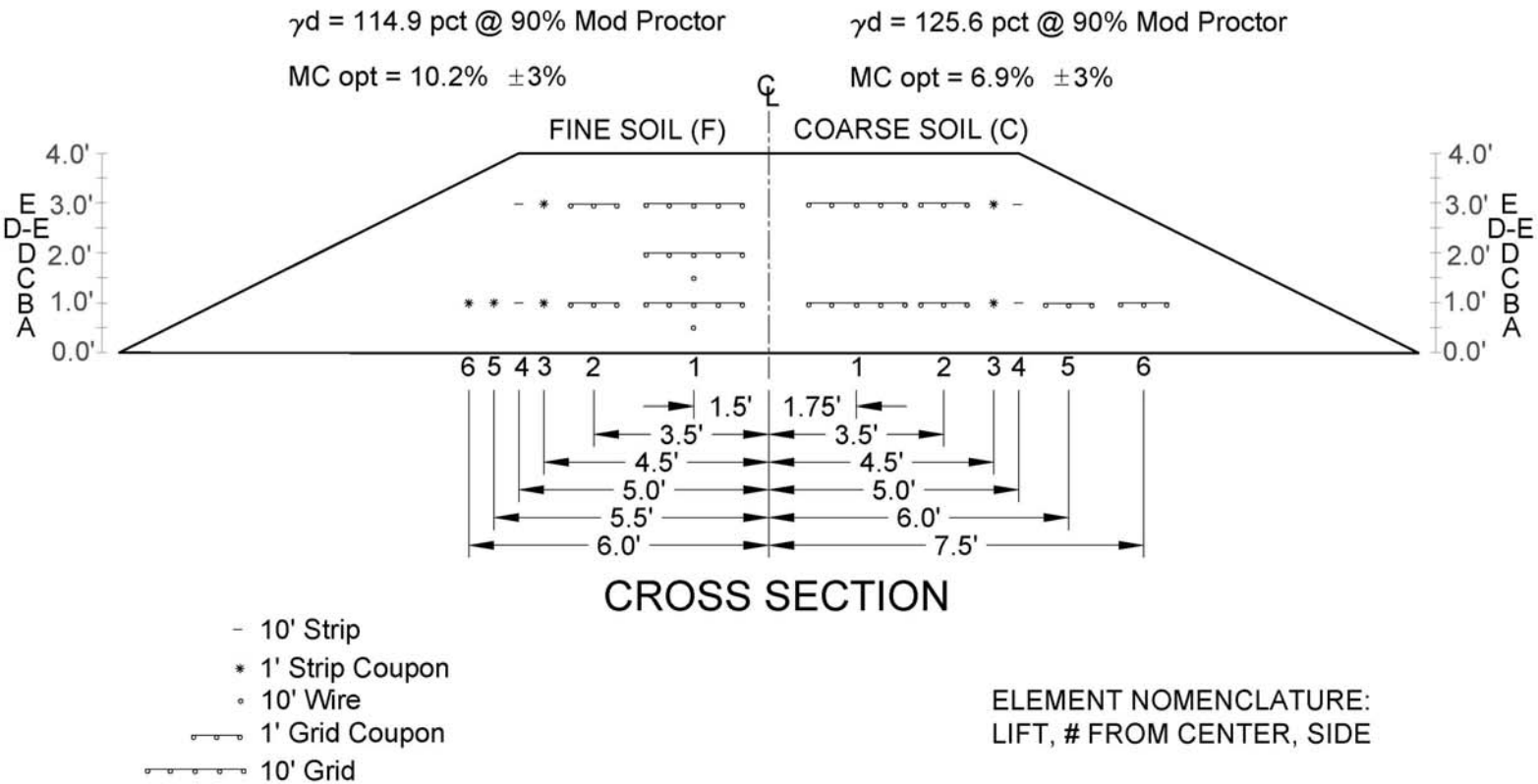


Figure 5. Cross Section of EDI Test Embankment and Placement of Dummy Reinforcements and Coupons

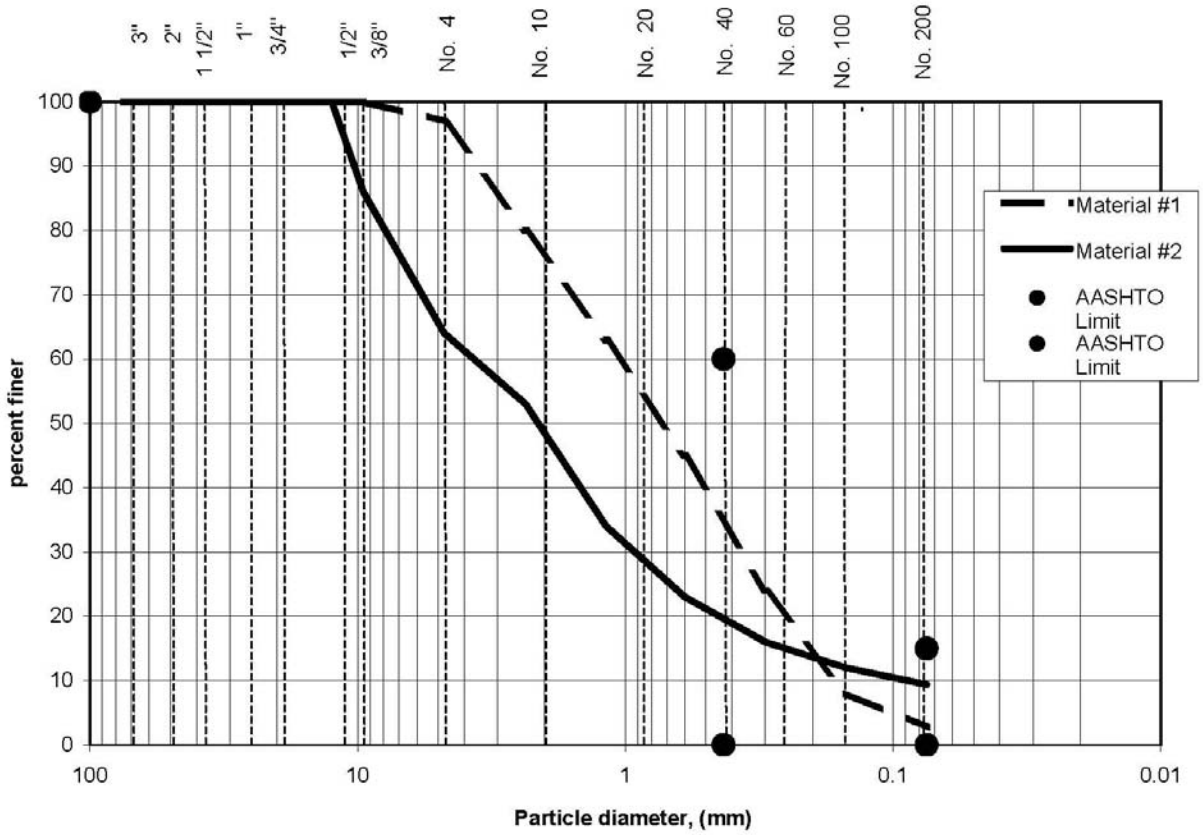


Figure 6. Grain Size of Materials #1 and #2 for EDI Test Embankment

Figure 7. Locations of Sites in California Listed in Table 9.

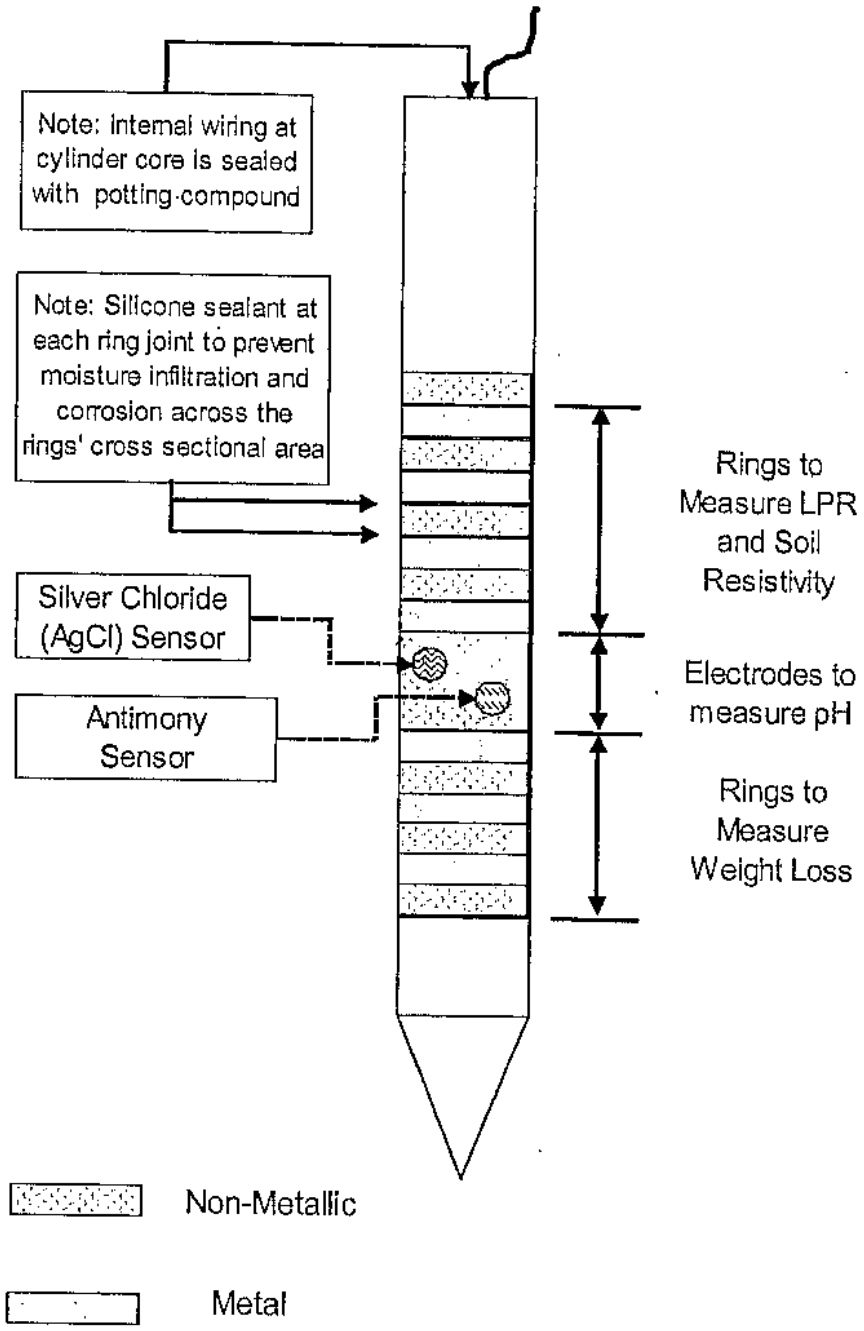


Figure 8. Illustration of Cylinder Probes Developed for NCHRP Project 21-06 (Corrpro, 2009)

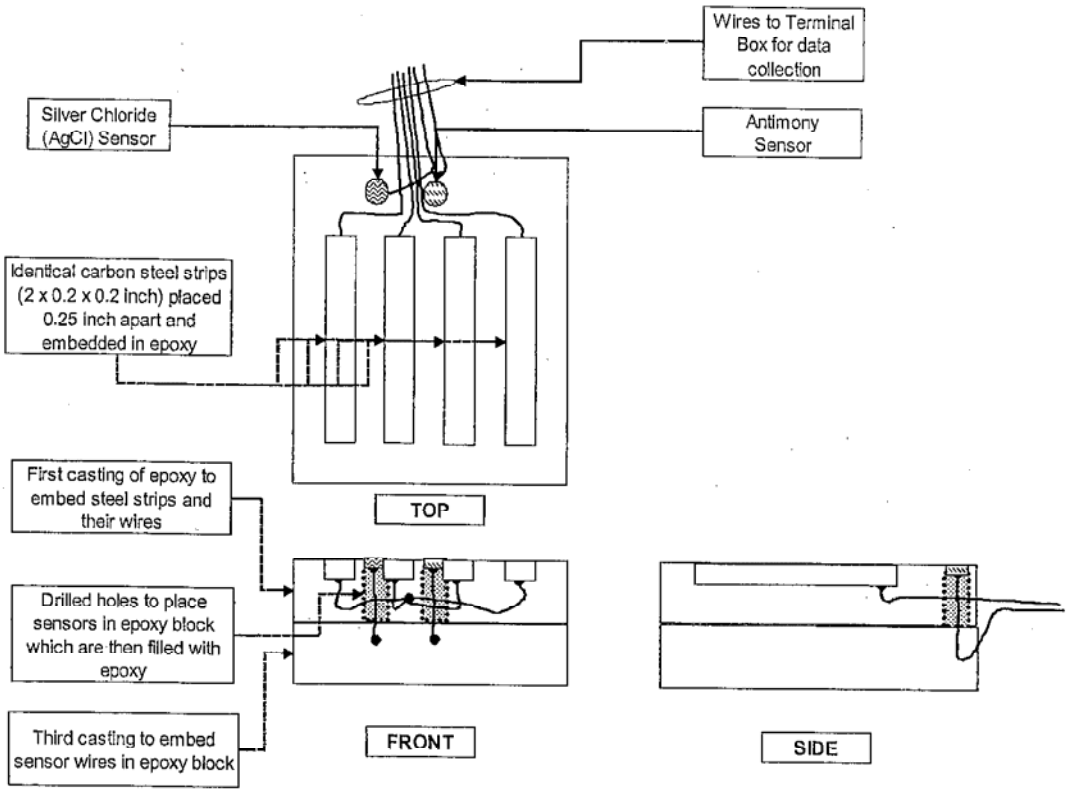


Figure 9. Illustration of DMP Developed for NCHRP Project 21-06 (Corrpro, 2009)

CHAPTER 5 – CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

This study addresses recommendations resulting from NCHRP Project 24-28 to (1) obtain more reliable data and reduce uncertainty with respect to the performance of MSES constructed with marginal quality fills, (2) obtain additional performance data from older installations of MSES reinforcements, and (3) implement more robust test techniques to evaluate the existing conditions of rock bolts, soil nails and ground anchors. Results presented in this report serve to further validate results from NCHRP 24-28 and predictive models for corrosion potential, metal loss, and service life of metal reinforced systems.

1. The variability of the observed performance of marginal fills resulting from NCHRP Project 24-28 is due to uncertainty with respect to the fill properties, which may also be inherently variable. Measurements of fill resistivity obtained at the location and time of corrosion rate measurements reduce uncertainty, and improve our ability to model the performance of MSES constructed with marginal quality fills. Results presented in this report demonstrate that the resistivity of in situ materials surrounding earth reinforcements can be determined at the time and location of corrosion rate measurements. The technique employs measurements of resistance via the three-electrode technique, and a simple relationship between the measured resistance and resistivity. Measurements of corrosion rate and resistivity in the three-electrode configuration are most useful, since the majority of measurements in the database compiled as part of NCHRP Project 24-28 utilize this technique. Thus, resistivity is computed using the measured resistance, and the width and length of the reinforcement. This technique is verified via measurements under known conditions within a test embankment, and data collected from five different sites in California with access to MSE reinforcements and fill materials for sampling and testing.

2. The fieldwork included the oldest MSE wall in the United States, which was constructed in 1971, along Route 39 through the San Gabriel Mountains near Los Angeles, and 39 years old at the time when observations were made. Measurements of corrosion rate obtained from San Gabriel are extremely interesting since they are some of the first confirmed observations of the rate of steel consumption subsequent to depletion of zinc for galvanized reinforcements used in MSE construction in the United States. These rates are much less than those used in design and appear to be closer to the corrosion rates used for zinc, implying that a discrete change in corrosion rates as zinc is depleted, as implied by the current AASHTO model for metal loss of MSE reinforcements, does not necessarily occur.

3. The impulse response test (IR) was implemented for probing the lengths of rock bolts to access details of the installation and conditions surrounding the structural elements (i.e. steel rods). The IR test is useful to locate and identify the size and shape of anomalies along rock bolt installations. This is considered an improvement over the sonic echo (SE) technique, which is the existing practice for condition assessment of rock bolts. Results from the IR test were verified with respect to tests performed with dummy rock bolts that

included both typical and deliberately distressed installations. Results from this study indicate that the IR test is more robust and renders additional information for condition assessment of rock bolts compared to what is achievable from the SE test. However, the SE test is adequate to assess remaining levels of prestress, and to identify the free lengths of rock bolt installations. Knowledge of the free length is useful if the total length is known, and general details of the installation are needed for reconciling data from further electrochemical testing (e.g., lengths are needed to assess surface areas of the test elements and reconcile corrosion rates from LPR measurements).

RECOMMENDATIONS

1. The techniques for measuring fill resistivity implemented in this study should be applied to the database developed for NCHRP Project 24-28 to develop service life models that directly incorporate fill resistivity as a variable. Current service life models used to calibrate LRFD strength reduction factors for NCHRP Project 24-28 apply to relatively broad ranges of fill types (resistivities) leading to wide scatter and variability within the range of marginal quality fills. Service life models that directly incorporate fill quality will render improved correlations between corrosion rates and fill resistivity, and improvements to reliability-based calibration of LRFD strength reduction factors for MSE constructed with marginal quality fills.

2. More data should be collected to document the performance of MSE constructed with marginal quality fills. The characteristics of marginal quality fill including salt contents and resistivities may vary randomly with respect to location within a given source. Fill resistivity may change over time due to changes in moisture content, salt concentration, etc., and these changes are not described from measurements from fill sources obtained prior to construction. Thus, data should be collected from selected sites at prescribed intervals to monitor spatial variations and the effects of changes in conditions that may occur over time. These data will be useful to provide guidance, recommendations, specifications and limitations on use of marginal quality fills for construction involving earth reinforcements.

3. The IR test is recommended for condition assessment of rock bolts when more detailed information from within the bonded zone is necessary. Further evaluations should be conducted considering rock bolt lengths greater than twenty feet.

4. Additional data on the performance of rock bolts should be collected and used to develop fragility curves to describe the time dependant vulnerability and safety of rock cuts that are supported with rock bolts.

REFERENCES

1. AEMC Instruments (2003). Understanding Ground Resistance Testing, Workbook Edition 8, AEMC Instruments, Foxborough, MA.

2. Corrpro Companies, Inc. (2009). Corrosion in the Soil Environment: Soil Resistivity and pH Measurements, NCHRP Project 21-06, Final Report, submitted to the National Cooperative Highway research Program, Washington, D.C.

3. Davis, A.G. and Robertson, S.A. (1975). "Economic Pile Testing," Ground Engineering, London, UK, May, 40-43.

4. Elias, V., Fishman, K.L., Christopher, B.R., and Berg, R.R., 2009, Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, FHWA Report No. FHWA-NHI-09-087, NTIS, Springfield, VA.

5. Finno, R.J. and Gassman, S.L. (1998). "Impulse Response Evaluation of Drilled Shafts," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 124(10), Reston, VA.

6. Fishman, K.L. and Withiam, J.L., 2011, LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems, NCHRP Report 675, National Academy Press, Washington, D.C., 50 pp.

7. Withiam, J.L., Fishman, K.L. and Gaus, M.P., 2002, Recommended Practice for Evaluation of Metal-Tensioned Systems in Geotechnical Applications, NCHRP Report 477, National Academy Press, Washington, D.C., 93 pp.

APPENDIX I

INSTALLATION AND NDT OF DUMMY ROCK BOLTS AT THE BARRON MOUNTAIN TEST SITE WOODSTOCK, NH

1. Pallet drill used to advance the drill hole.

2. Down-the-hole pneumatic hammer advances 3.5” diameter drill hole.

3. Prestressing steel rod inserted into drill hole.

4. Six dummy rock bolts with gout tubes

5. Close-up of grout tube

6. Typical rock bolt prepared for “normal” installation. Anchor head is on ground with plastic sheath covering the free length and leading to the bonded length resting on top of the Jersey Barrier.

7. Sample #6 showing notch cut into prestressing steel rod near anchor head location.

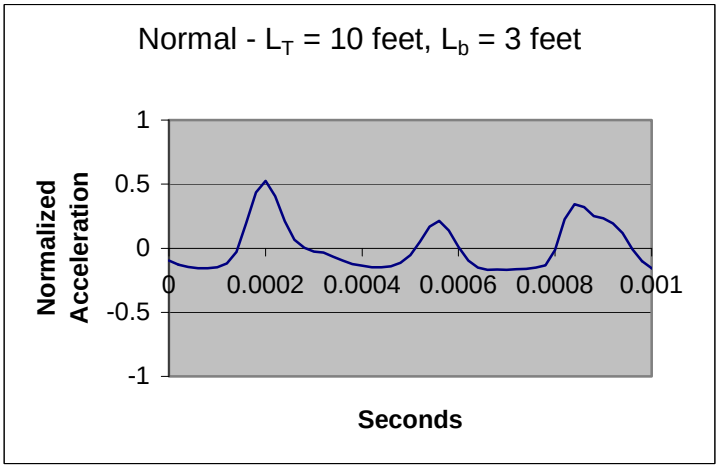
8. Sample #2 - 15 feet long rock bolt with inclusions near each end of the bonded zone.

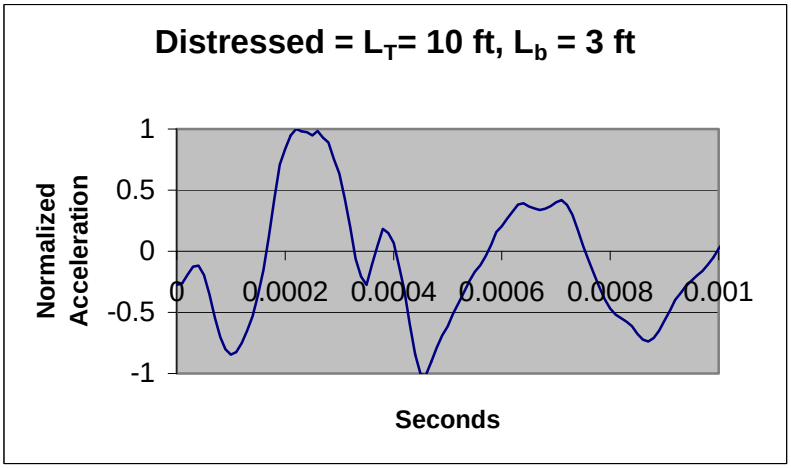
9. Sample #5 – 20 feet long rock bolt with inclusions near the beginning and middle of the bonded zone.

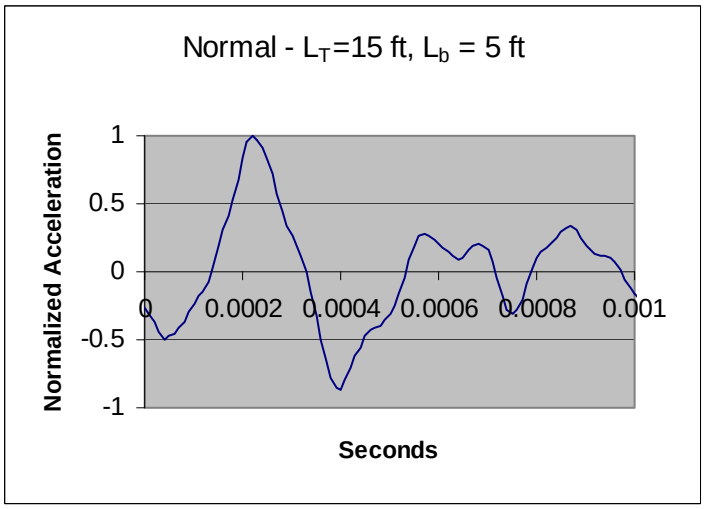
10. View of installations ready for NDT

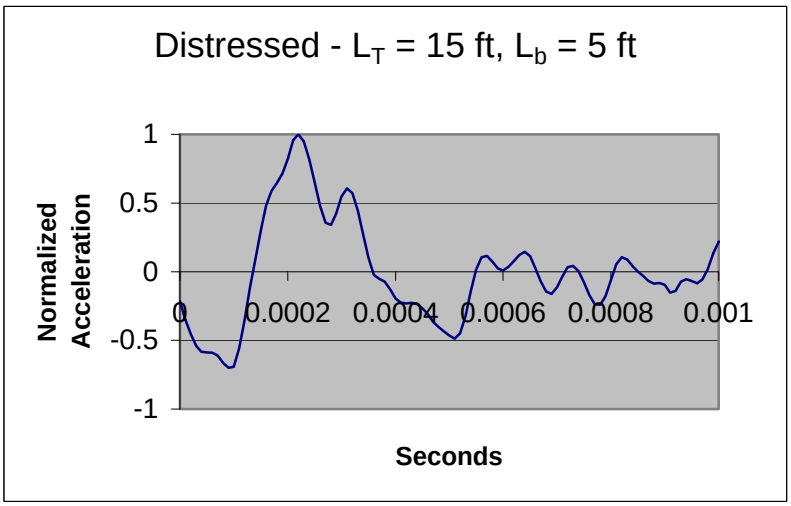
11. Accelerometer attached to end of rock bolt.

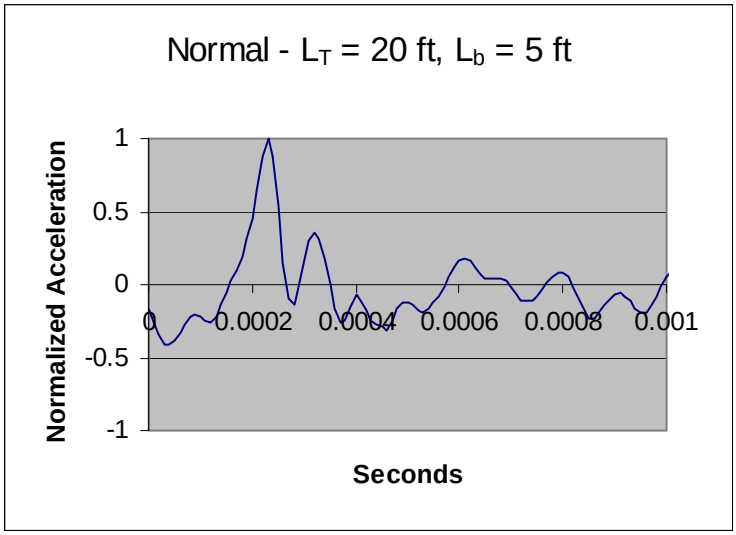
12. Signal conditioners and data acquisition equipment.

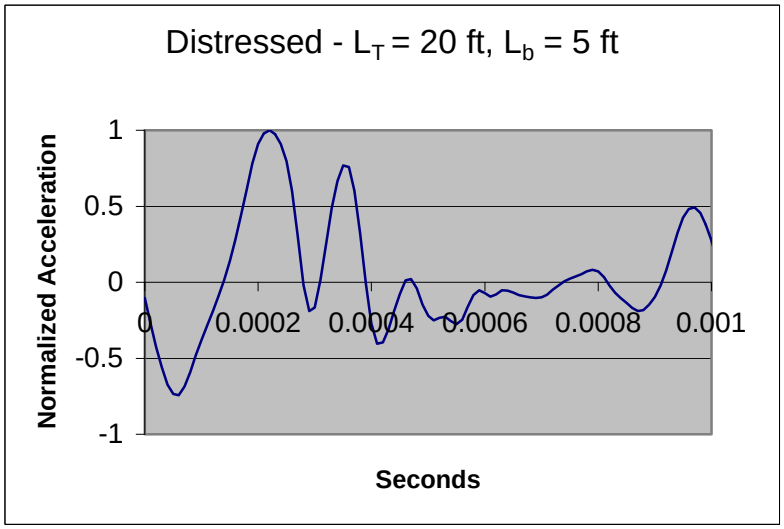






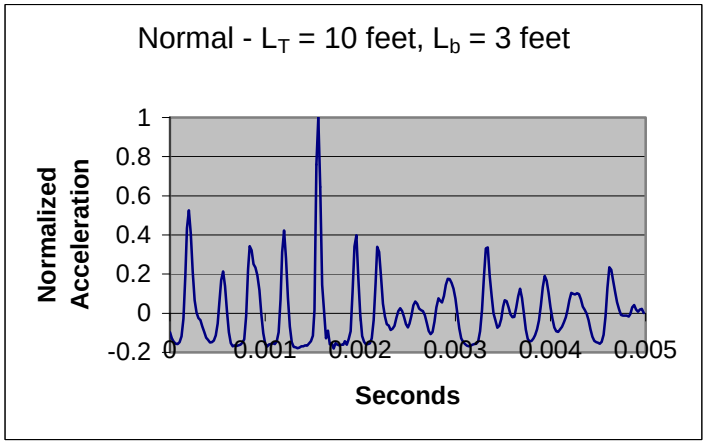


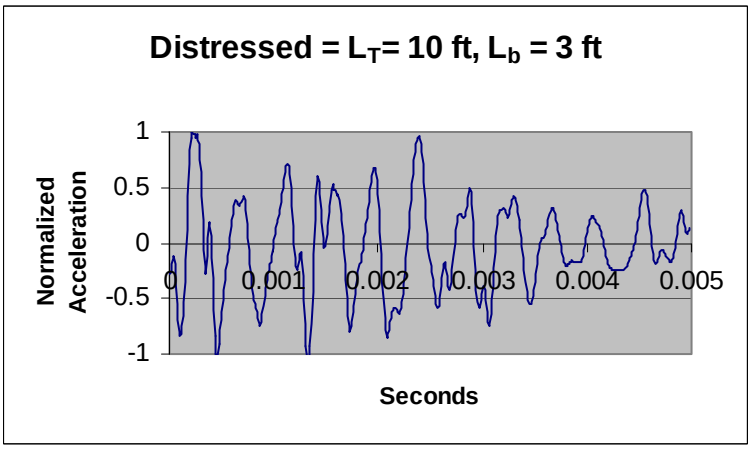


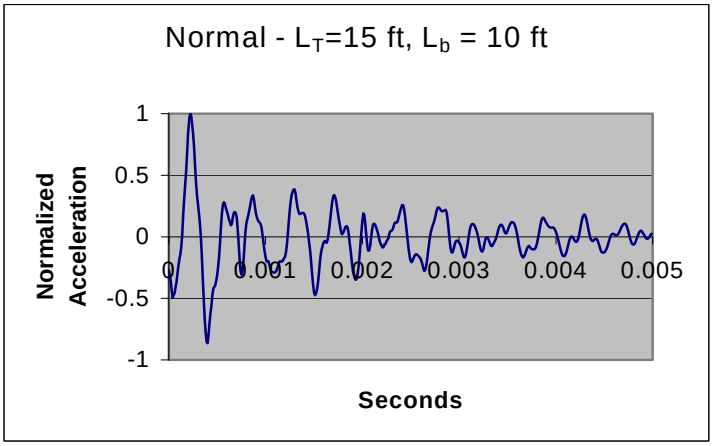


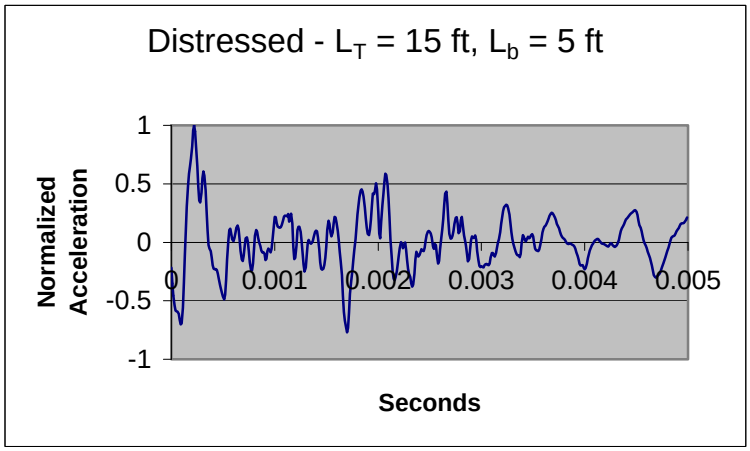
Attenuation of Impact Force Related to Prestress (preload values are cited in Table 1)

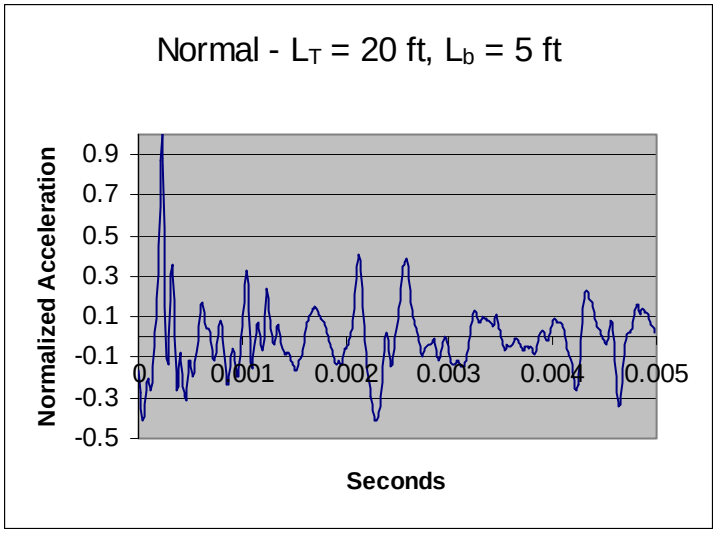
Instrumented Hammer

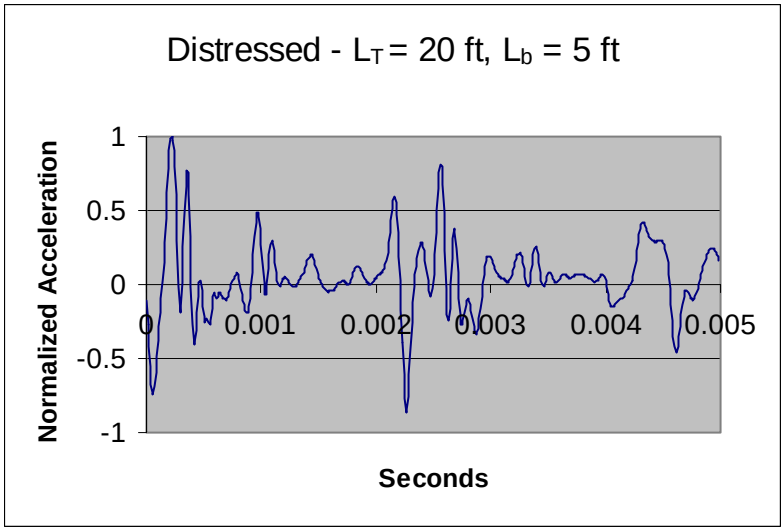






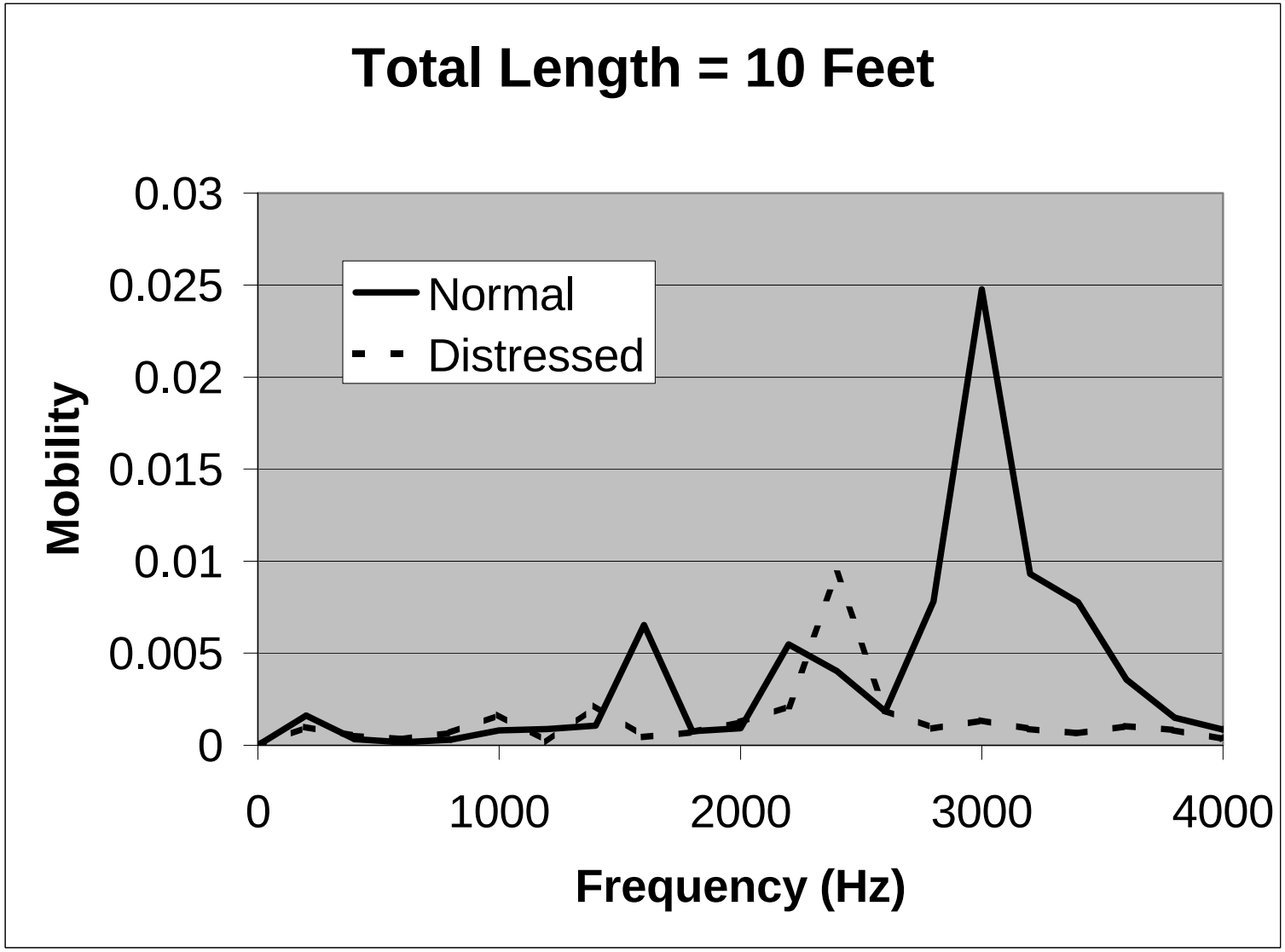


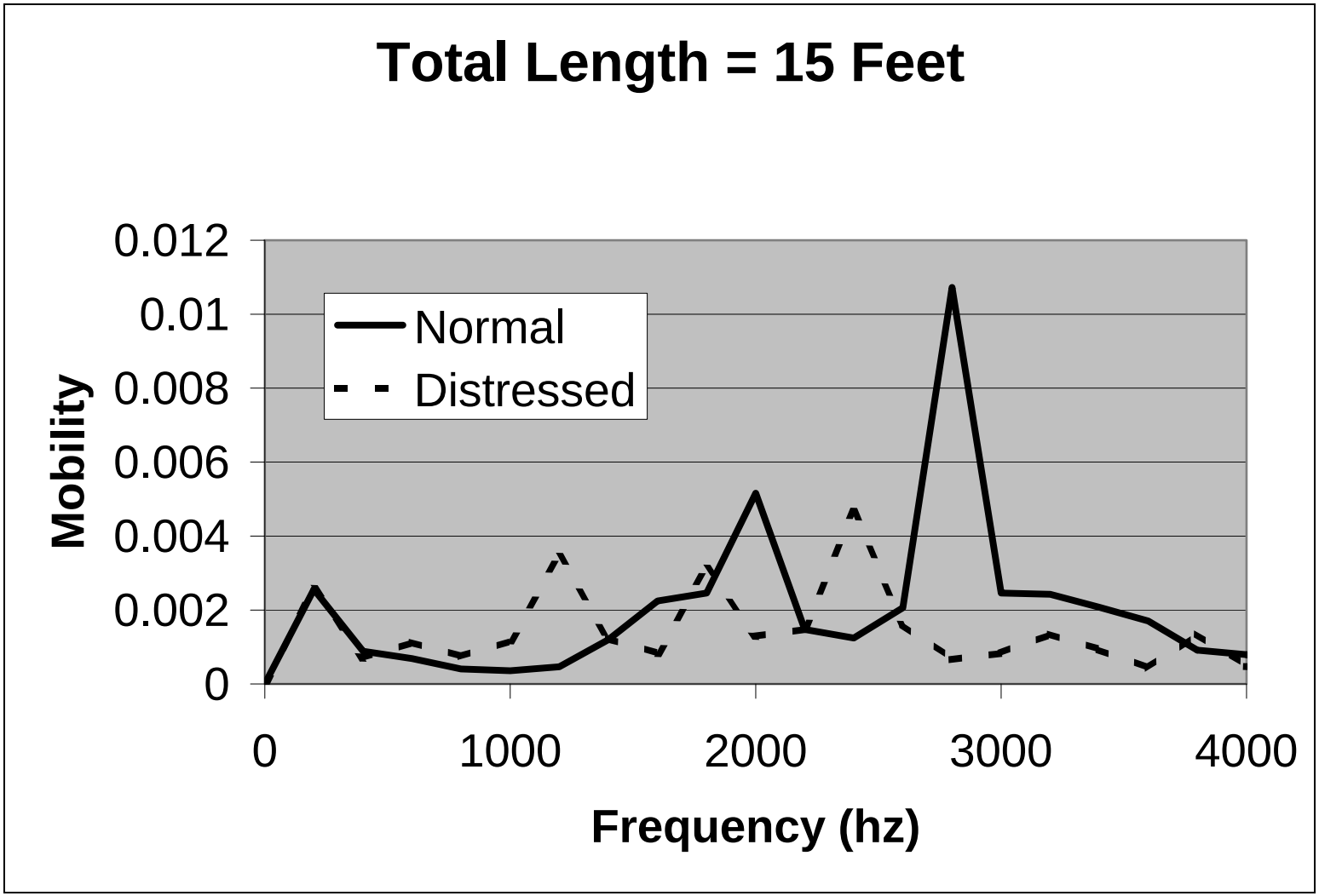


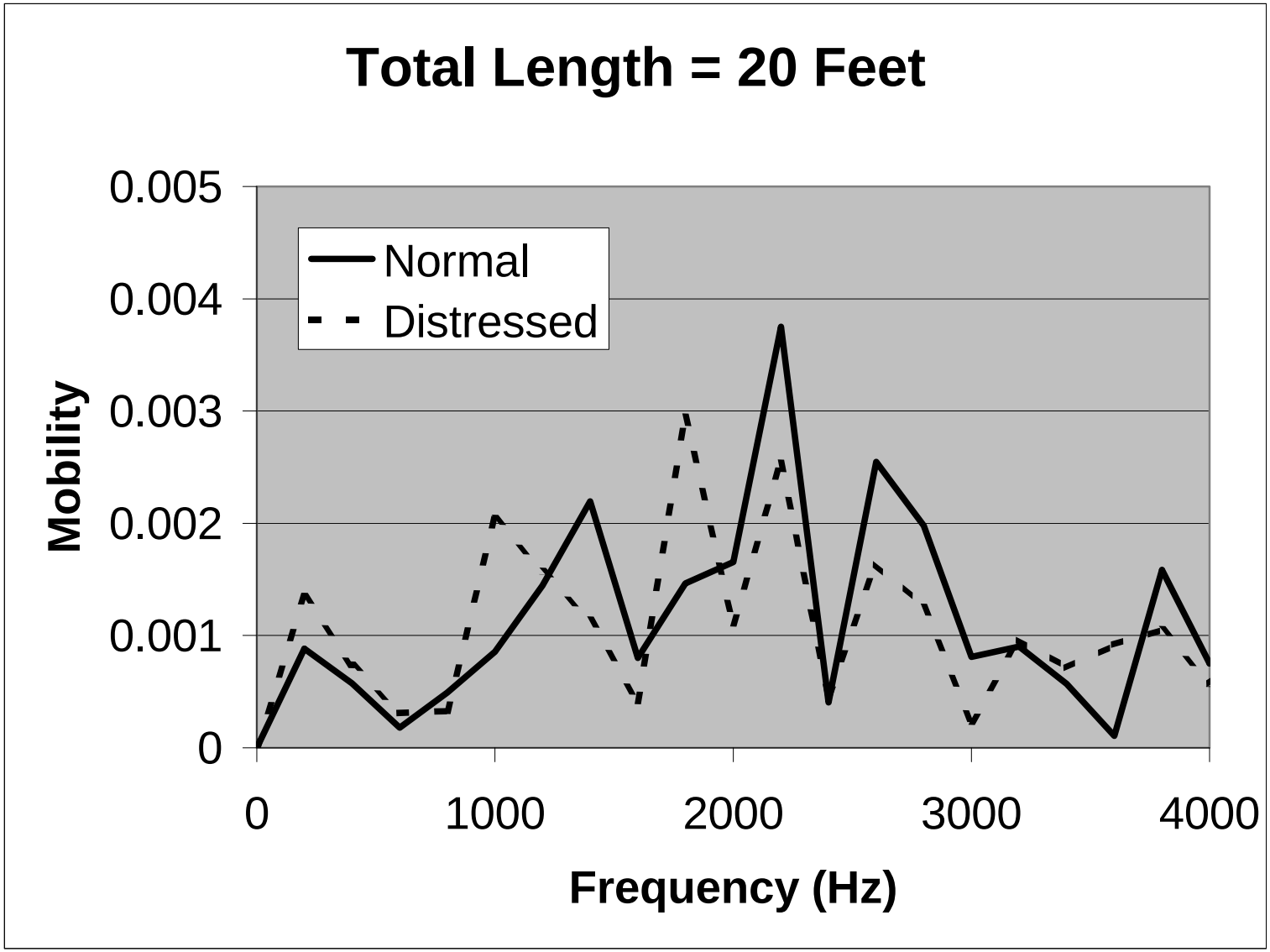


Time Histories of Impacts from Sonic Echo Tests

Instrumented Hammer


Total Length = 10 Feet
Mobility
0.03
0.025
0.02
0.015
0.01
0.005
0
Normal
Distressed
0
1000
2000
3000
4000
Frequency (Hz)


Total Length = 15 Feet
0.012
0.01
0.008
0.006
0.004
0.002
0
Mobility
Normal
Distressed
0
1000
2000
3000
4000
Frequency (hz)


Total Length = 20 Feet
Normal
Distressed
Mobility
0.005
0.004
0.003
0.002
0.001
0
0
1000
2000
3000
4000
Frequency (Hz)

Units	A_s (cm^2)	B (V)	Mat	Name	State	Loc	WE	CE	Notes	R_s (Ω)	P_r (Ω)	CR (mm/yr)	E_{corr} (V)	Coef	CR (μm/yr)	E_{corr} (mV)
Metric	1216	0.05	Zn	NH-10-27-09-BM-1-3	NH	Barron	1	3	Dummy Rods	809.084	251.24	0.00245051	-1.146866	0.998608	2.45	-1146.866
Metric	1216	0.05	Zn	NH-10-27-09-BM-1-3-2e	NH	Barron	1	3	Dummy Rods 2 electrode	1712.568	319.6299	0.00192618	-0.075328	0.999485	3.85	
Metric	1216	0.05	Zn	NH-10-27-09-BM-2-1	NH	Barron	2	1	Dummy Rods	941.7698	186.6221	0.003299	-1.008873	1.000426	3.30	-1008.873
Metric	1216	0.05	Zn	NH-10-27-09-BM-2-1-2E	NH	Barron	2	1	Dummy Rods 2 electrode	1644.579	324.9906	0.00189441	0.141453	0.999358	3.79	
Metric	730	0.05	Zn	NH-oct-27-09-3-one-repeat	NH	Barron	3	1	Dummy Rods	867.6667	191.4724	0.00535611	-1.064184	0.999792	5.36	-1064.184
Metric	730	0.05	Zn	NH-oct-27-09-3-one-2e	NH	Barron	3	1	Dummy Rods 2 electrode	1841.67	192.7682	0.00532011	0.086658	0.999577	10.64	
Metric	1216	0.05	Zn	NH-10-27-09-BM-4-1	NH	Barron	4	1	Dummy Rods	898.1115	279.0239	0.0022065	-1.185938	0.998301	2.21	-1185.938
Metric	1216	0.05	Zn	NH-10-27-09-BM-4-1-2e	NH	Barron	4	1	Dummy Rods 2 electrode	1830.327	290.7146	0.00211777	-0.039391	0.99974	4.24	
Metric	1216	0.05	Zn	NH-10-27-09-BM-5-1	NH	Barron	5	1	Dummy Rods	1021.823	208.203	0.00295705	-0.946683	0.999254	2.96	-946.6829
Metric	1216	0.05	Zn	NH-10-27-09-BM-5-1-2e	NH	Barron	5	1	Dummy Rods 2 electrode	1862.218	353.858	0.00173987	0.204681	0.999628	3.48	
Metric	730	0.05	Zn	NH-oct-27-09-6-one	NH	Barron	6	1	Dummy Rods	766.2807	180.0773	0.00569504	-0.964876	0.99999	5.70	-964.8759
Metric	730	0.05	Zn	NH-oct-27-09-6-one-2e	NH	Barron	6	1	Dummy Rods 2 electrode	1583.374	340.8019	0.00300922	0.186457	0.999706	6.02	

APPENDIX II

CONSTRUCTION, INSTALLATION OF DUMMY REINFORCEMENTS AND COUPONS, AND TESTING AT THE EDI TEST EMBANKMENT ELMA, NY

1. Place level working platform.

2. Set divider to separate Materials #1 and #2.

3. Add moisture to achieve near optimum moisture content

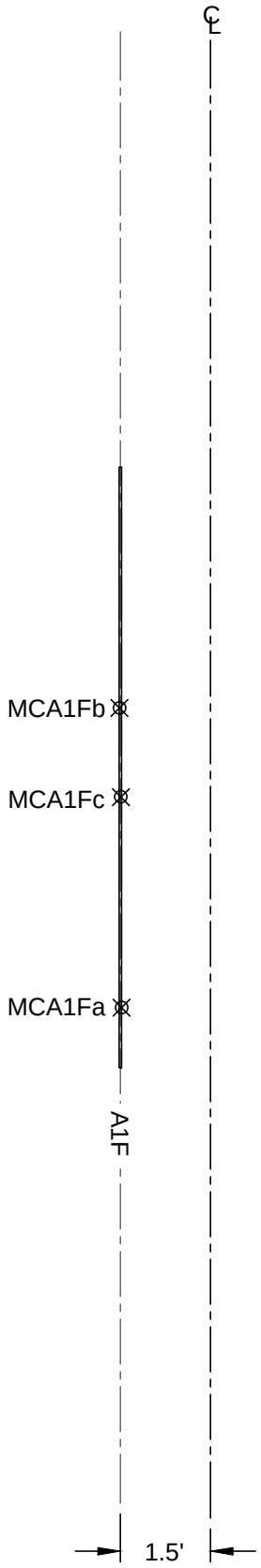
4. Compacting Finer Material (#1)

5. Set Dummy Reinforcements and Coupons on Material #1 side at Elevation 1'.

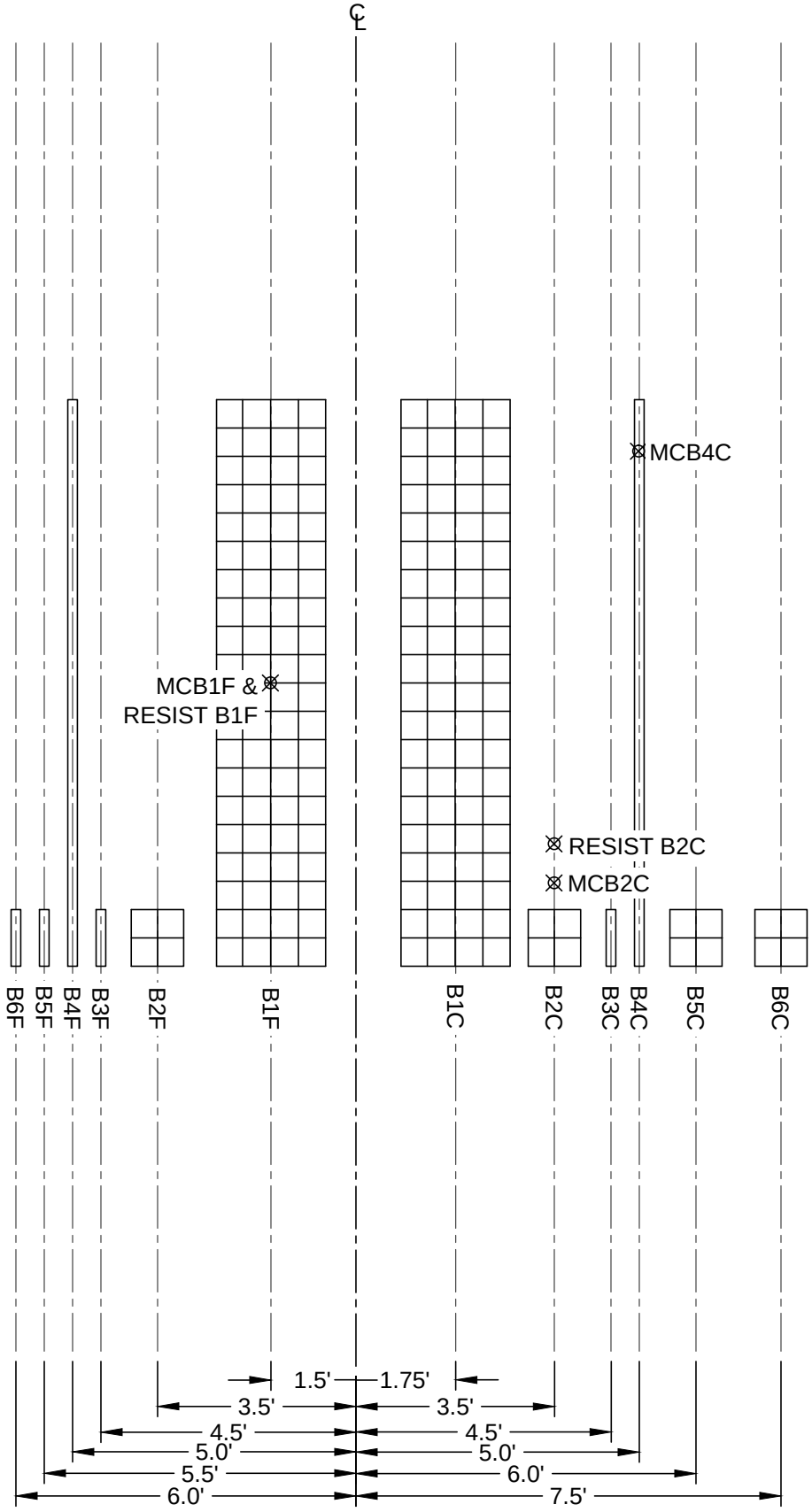
6. Nuclear density testing for in situ unit weight and moisture content.

7. Near top of test embankment – Material #1 on left and Material #2 on right.

8. Prepared for electrochemical testing and soil resistance measurements.

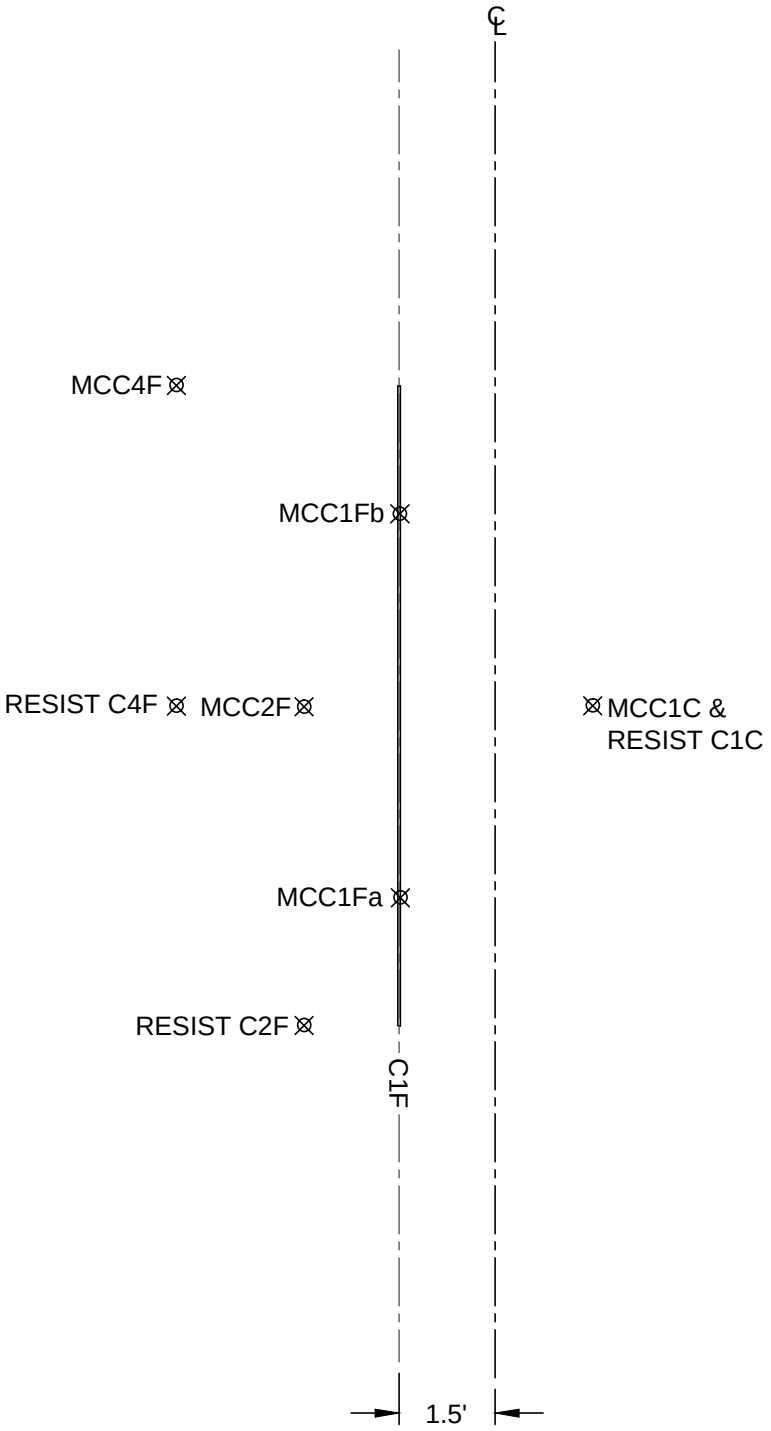


LIFT A AT ELEVATION 0.5'
PLAN VIEW

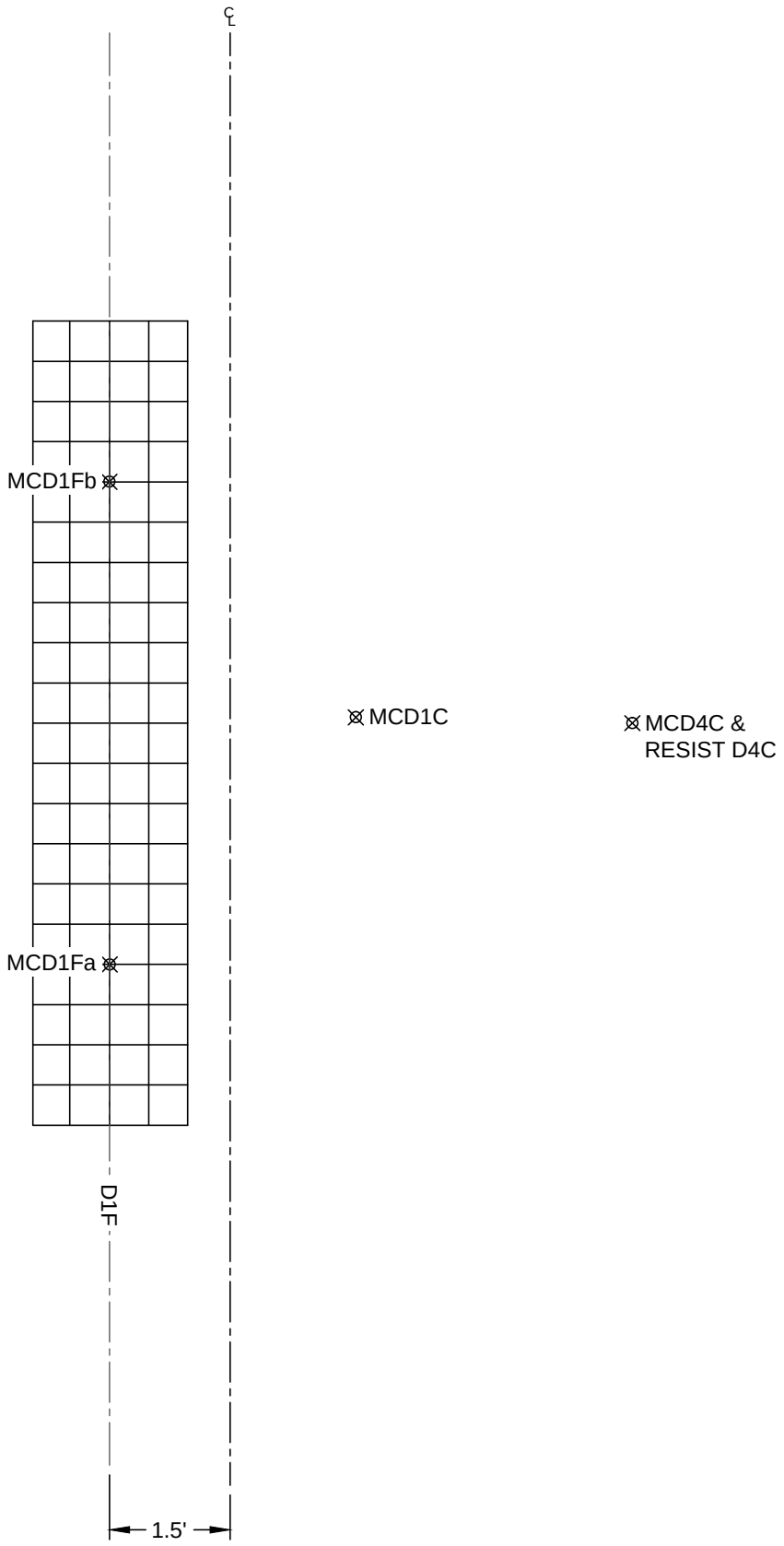


LIFT B AT ELEVATION 1.0'
PLAN VIEW

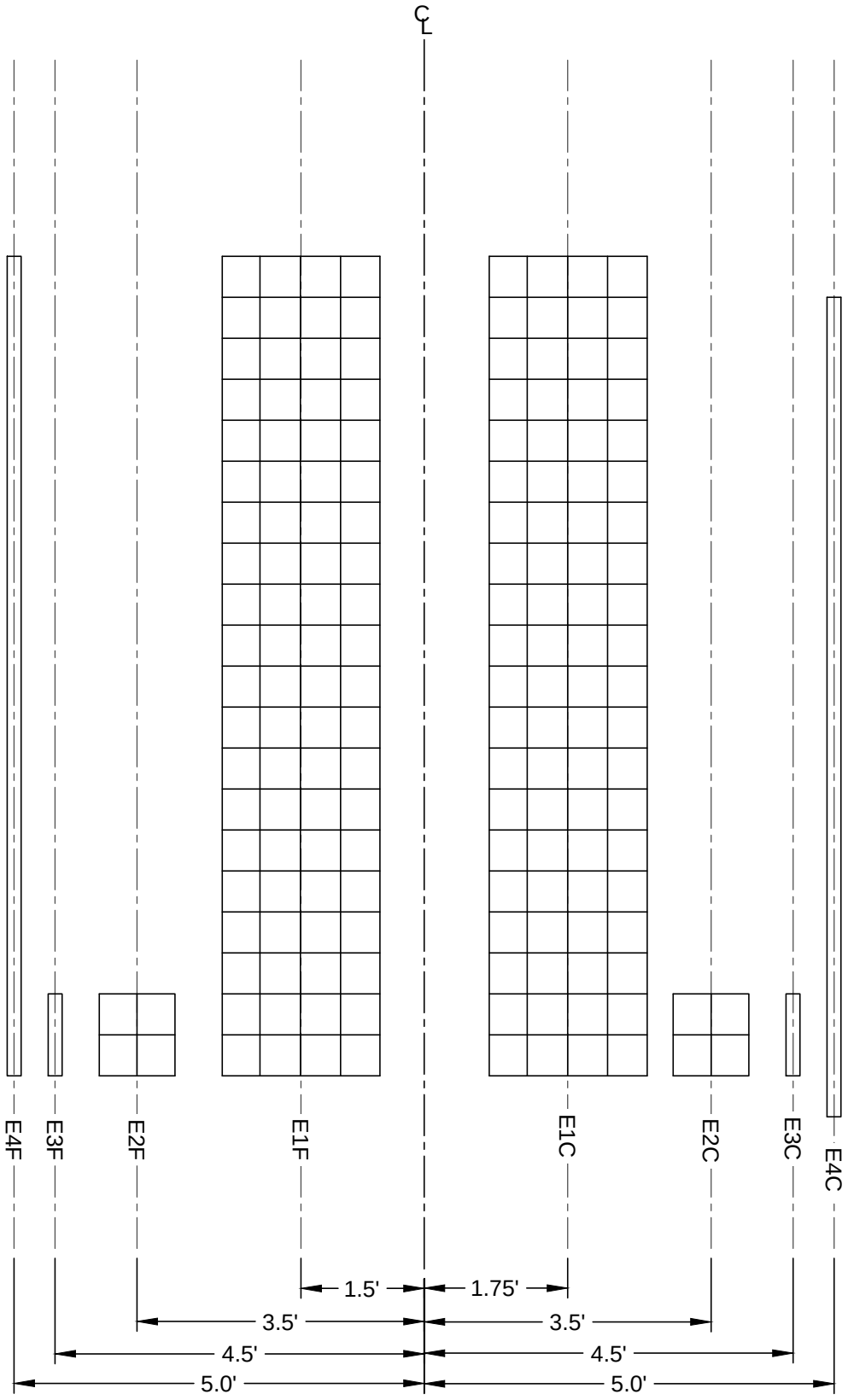
LIFT D AT ELEVATION 2.0'
PLAN VIEW



LIFT C AT ELEVATION 1.5'
PLAN VIEW



LIFT D AT ELEVATION 2.0'
PLAN VIEW



LIFT E AT ELEVATION 3.0'
PLAN VIEW

Geotechnics
.....GEOTECHNICAL AND GEOSYNTHETIC SERVICES

April 22, 2010

Project No. 2010-146-01

Mr. Jim Bojarski
McMahon & Mann Consulting Engineers
2495 Main Street, Suite 432
Buffalo, NY 14214

RECEIVED
APR 26 2010
McMahon & Mann
Consulting Engineers, P.C.

Transmittal
Laboratory Test Results
NHCRP SOIL BERM 06-018

Please find attached the laboratory test results for the above referenced project. The tests were outlined on the Project Verification Form that was faxed to your firm prior to the testing. The testing was performed in general accordance with the methods listed on the enclosed data sheets. The test results are believed to be representative of the samples that were submitted for testing and are indicative only of the specimens which were evaluated. We have no direct knowledge of the origin of the samples and imply no position with regard to the nature of the test results, i.e. pass/fail and no claims as to the suitability of the material for its intended use.

The test data and all associated project information provided shall be held in strict confidence and disclosed to other parties only with authorization by our Client. The test data submitted herein is considered integral with this report and is not to be reproduced except in whole and only with the authorization of the Client and Geotechnics. The remaining sample materials for this project will be retained for a minimum of 90 days as directed by the Geotechnics' Quality Program.

We are pleased to provide these testing services. Should you have any questions or if we may be of further assistance, please contact our office.

Respectively submitted,
Geotechnics, Inc.

David R. Backstrom
Laboratory Director

We understand that you have a choice in your laboratory services and we thank you for choosing Geotechnics.

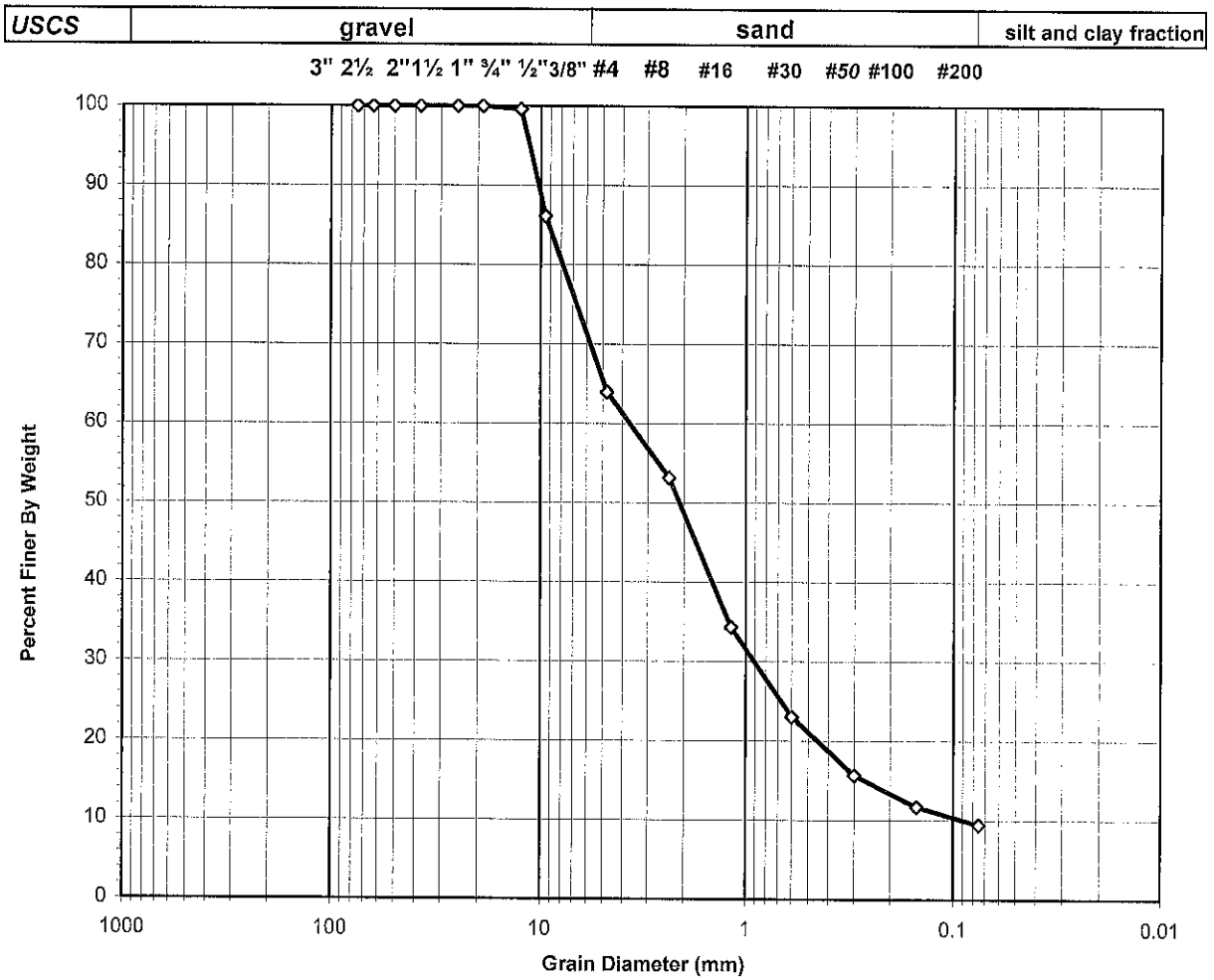
544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

WASH SIEVE ANALYSIS OF AGGREGATES
ASTM C117-04 / C136-06

Client	McMAHON & MANN	Boring No.	GRAVEL
Client Reference	NCHRP SOIL BERM 06-018	Depth(ft.)	NA
Project No.	2010-146-01	Sample No.	032910-1
Lab ID	2010-146-01-01		

Color **DARK GRAY**



USCS Symbol:	***sw-sm, ASSUMED***	D60 =	3.68		
		D30 =	0.91	CC =	2.54
Fineness Modulus:	***4.12***	D10 =	0.09	CU =	41.38

USCS Classification: ***WELL-GRADED SAND WITH SILT AND GRAVEL***

Tested By PC Date 4/6/10 Checked By HB Date 4-16-10

DCN: CT-S42A DATE 08/26/09 REVISION: 11

C:\MSOFFICE\Excel\PrintQ\[Y10.xls]Sheet1

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

WASH SIEVE ANALYSIS OF AGGREGATES
ASTM C117-04 / C136-06

Client	McMAHON & MANN	Boring No.	GRAVEL
Client Reference	NCHRP SOIL BERM 06-018	Depth(ft.)	NA
Project No.	2010-146-01	Sample No.	032910-1
Lab ID	2010-146-01-01		
		Color	**DARK GRAY**

Tare No.	687	Wt. of Dry Specimen (gm)	1158.99
Wt. Tare + DS.(Pre-Wash)	1256.40	Wt. of +#200 Specimen(gm)	1049.47
Wt.Tare + Dry,Washed specimen	1148.21	Wt. of -#200 Specimen(gm)	110.29
Wt Tare	97.41		
Wt. Dry,Washed specimen	1050.80		
		Total Wt. Retained After Sieving	1051.57

% Difference Wt. Dry,Washed specimen vs Total Wt. Retained After Sieving -0.1
Note: % Difference must not be more than 0.3

Sieve	Sieve Opening (mm)	Weight Retained (gm.)	Percent Retained	Accumulated Percent Retained	**Percent Finer**
3"	75	0.00	0	0	100
2 1/2"	63	0.00	0	0	100
2"	50	0.00	0	0	100
1 1/2"	37.50	0.00	0	0	100
1"	25.00	0.00	0	0	100
3/4"	19.00	0.00	0	0	100
1/2"	12.50	4.48	0	0	100
3/8"	9.50	156.45	13	14	86
#4	4.75	256.80	22	36	64
#8	2.36	125.58	11	47	53
#16	1.18	217.90	19	66	34
#30	0.60	131.30	11	77	23
#50	0.30	85.12	7	84	16
#100	0.15	45.86	4	88	12
#200	0.075	25.98	2.2	90.6	9.4
Pan	-	2.10	0	91	-

Tested By PC Date 4/6/10 Checked By KB Date 4-6-10

DCN: CT-S42A DATE 08/26/09 REVISION: 11
C:\MSOFFICE\Excel\PrintQ\Y10.xlsSheet1

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

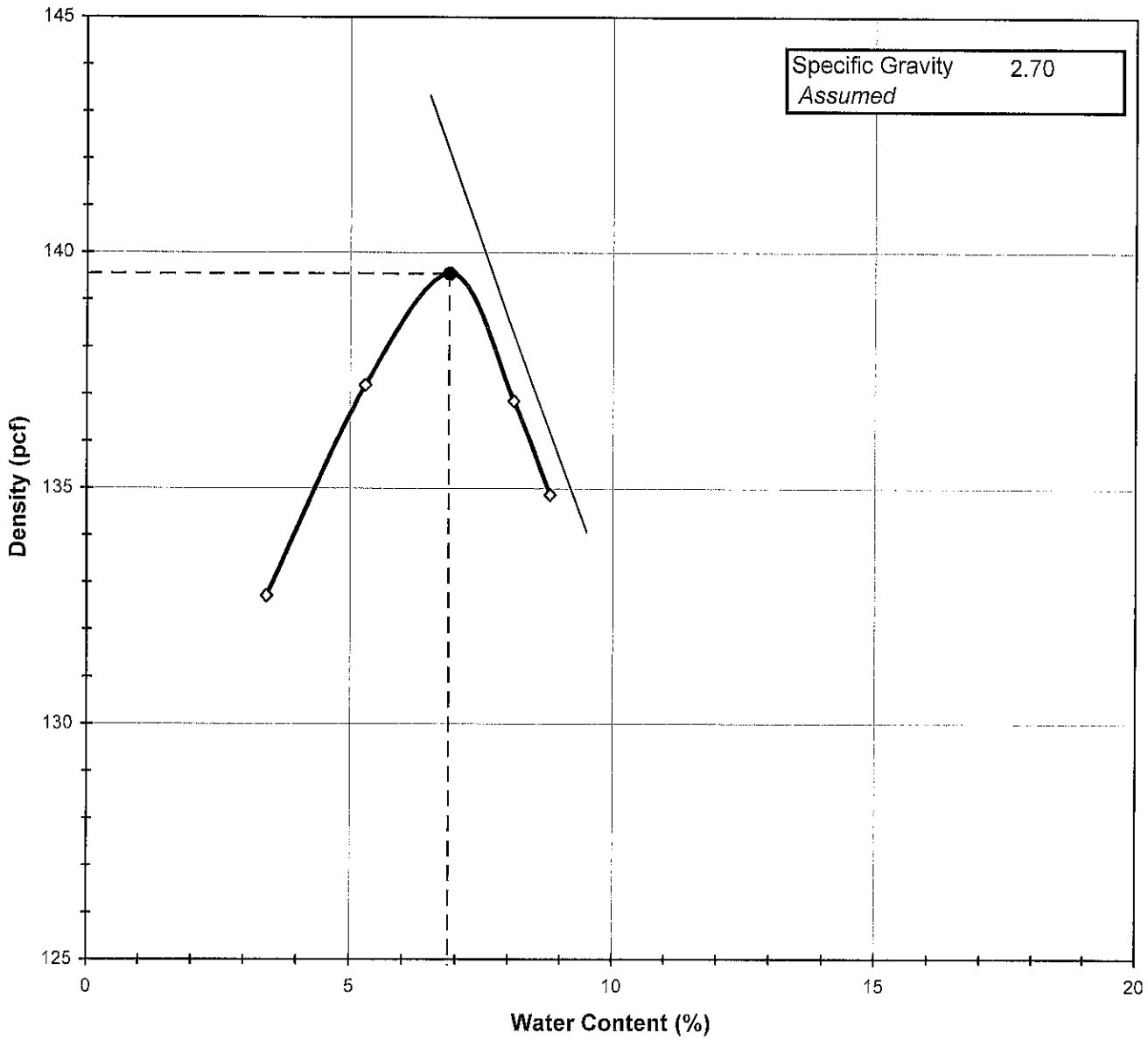
MOISTURE DENSITY RELATIONSHIP

ASTM D1557-07 SOP-S13

Client	McMAHON & MANN	Boring No.	GRAVEL
Client Reference	NCHRP SOIL BERM 06-018	Depth (ft)	NA
Project No.	2010-146-01	Sample No.	032910-1
Lab ID	2010-146-01-01	Test Method	**MODIFIED**

Visual Description GRAY CRUSHED STONE AND GRAVEL

Optimum Water Content **6.9**
Maximum Dry Density **139.6**



Tested By *MF* *Date* *4/13/10* *Checked By* KB *Date* 4-15-10

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

MOISTURE - DENSITY RELATIONSHIP

ASTM D1557-07 SOP-S13

Client	McMAHON & MANN	Boring No.	GRAVEL
Client Reference	NCHRP SOIL BERM 06-018	Depth (ft)	NA
Project No.	2010-146-01	Sample No.	032910-1
Lab ID	2010-146-01-01		

Visual Description GRAY CRUSHED STONE AND GRAVEL

Total Weight of the Sample (gm)	NA
As Received Water Content(%)	NA
Assumed Specific Gravity	2.70
Percent Retained on 3/4"	NA
Percent Retained on 3/8"	NA
Percent Retained on #4	NA
Oversize Material	Not included
Procedure Used	C

TestType		**MODIFIED**
Rammer Weight (lbs)		10.0
Rammer Drop (in)		18
Rammer Type		MECHANICAL
Machine ID	G	774
Mold ID	G	1154
Mold diameter		6"
Weight of the Mold		6358
Volume of the Mold(cc)		2128

Mold / Specimen

Point No.	**1**	**2**	**3**	**4**	**5**
Wt. of Mold & WS (gm)	11039	11284	11446	11403	11362
Wt.of Mold (gm)	6358	6358	6358	6358	6358
Wt. of WS	4681	4926	5088	5045	5004
Mold Volume (cc)	2128	2128	2128	2128	2128

Moisture Content / Density

Tare Number	602	1744	880	1693	870
Wt. of Tare & WS (gm)	456.80	425.90	565.30	575.80	744.40
Wt. of Tare & DS (gm)	444.50	408.70	535.80	538.90	693.10
Wt. of Tare (gm)	85.58	83.39	108.90	83.08	109.94
Wt. of Water (gm)	12.30	17.20	29.50	36.90	51.30
Wt. of DS (gm)	358.92	325.31	426.90	455.82	583.16
Wet Density (gm/cc)	2.20	2.31	2.39	2.37	2.35
Wet Density (pcf)	137.3	144.4	149.2	147.9	146.7
Moisture Content (%)	**3.4**	**5.3**	**6.9**	**8.1**	**8.8**
Dry Density (pcf)	**132.7**	**137.2**	**139.6**	**136.9**	**134.9**

Zero Air Voids

Moisture Content (%)	6.5	8.0	9.5
Dry Unit Weight (pcf)	143.3	138.6	134.1

Tested By *MF* *Date* *4/13/10* *Checked By* KB *Date* 4-15-10

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

pH OF SOILS

ASTM D 4972-01 / AASHTO T 289-91
(SOP- S36)

Client	McMAHON & MANN
Client Reference	NCHRP SOIL BERM 06-018
Project No.	2010-146-01

Lab ID		01	02
Boring No.		GRAVEL	SAND
Depth (ft)		NA	NA
Sample No.		032910-1	032910-2
Drying Tare No.		846	633
Testing Tare No.		D	E
Temperature (° C)		23	23
pH of Sample	***Test 1***	**8.4**	**8.8**
	Test 2	**8.4**	**8.8**
Agreement (+/- 0.2 units)		0.0	0.0

Meter Calibration

Buffer pH	Meter Reading	Meter Model
4.00	4.01	ORION 720A
7.00	7.01	
10.0	10.00	

pH of Distilled Water (Acceptable range 6.5-7.5)	6.9

Tested By KBL Date 4/14/10 Checked By KB Date 4-15-10

DCN: CT-S36 DATE 2/25/10 REVISION: 4

C:\MSOFFICE\Excel\PrintQ\[Y184.XLS]Sheet1

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

Minimum Resistivity

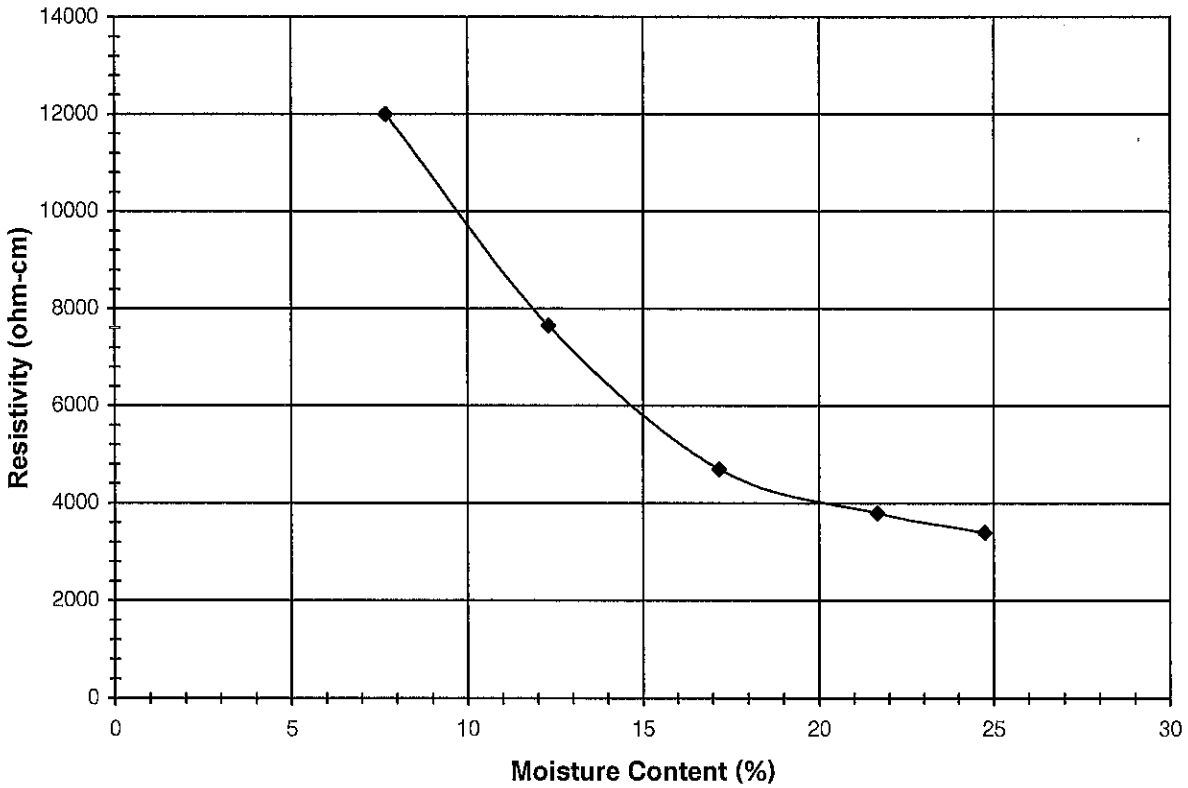
AASHTO T288 (SOP - S56)

geotechnics
INTEGRITY IN TESTING

Client	McMAHON & MANN	Boring No.	GRAVEL
Client Reference	NCHRP SOIL BERM 06-018	Depth (ft)	NA
Project No.	2010-146-01	Sample No.	032910-1
Lab ID	2010-146-01-01	Visual Description	DK GRAY COARSE SAND (- #10 Sieve material)

Tare No.	250	305	109	2184	120	
Tare & Wet Specimen (gm)	44.49	48.46	50.53	63.57	67.76	
Tare & Dry Specimen (gm)	42.57	45.25	45.40	55.86	57.86	
Tare Weight (gm)	17.57	19.14	15.53	20.27	17.86	
Moisture Content (%)	**7.7**	**12.3**	**17.2**	**21.7**	**24.8**	(Saturated)
Resistance (ohm)	**12000**	**7650**	**4700**	**3800**	**3400**	
Resistivity (ohm-cm)	**12000**	**7650**	**4700**	**3800**	**3400**	

Note: The ratio of Miller Box area versus distance between electrodes is equal to 1.



Soil Class	*Corrosion Resistance*	*Specific Resistivity (ohm-cm)*
1	Excellent	10,000 - 6,000
2	Good	6,000 - 4,500
3	Fair	4,500 - 2,000
4	Bad	2,000 - 0

Tested By PC *Date* 04/15/10 *Checked By* KB *Date* 4-16-10

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

Clark Laboratories LLC
IH and Environmental Laboratories
1801 Route 51 South - Bldg 9
Jefferson Hills, PA 15025
PHONE: 412-387-1001 FAX: 412-387-1028
AIHA Lab ID: 100355 *http://www.clarklabsllc.com*

Final Report
Tracking Sheet: 43646

Clark Laboratories LLC

Geotechnics, Inc. (PA) Geotechnics, Inc. 544 Bradock Avenue East Pittsburgh, PA 15112	Customer Code: 1300 - 0001 Attention: James Moyer Work Req By: Kevin Lichtenfels Customer P.O.: Project Number: NCHRP SOIL BERM 06-018	Phone: (412) 823-7600 Fax: (412) 823-8999 Email: klichtenfels@geotechnics.net Loc: McMahon & Mann Consulting Eng. Date Rcvd: 04/14/10	

Sample Id: **000328079** Client Sample Id: **GRAVEL 032910-1** Sampling Date: 04/13/10
Sampled Location: **2010-146-01-01**

Analyte	Method	Analysis Date	An. Init.	Reporting Limit	Results Total	
Chloride	Cal 422	04/21/10	RLH	**50 mg/kg**	< **50**	**mg/kg**
			Comments:	< 50 ppm		
Sulfate	Cal 417	04/21/10	RLH	**50 mg/kg**	**140**	**mg/kg**
			Comments:	140 ppm		

Sample Id: **000328080** Client Sample Id: **SAND 032910-2** Sampling Date: 04/13/10
Sampled Location: **2010-146-01-02**

Analyte	Method	Analysis Date	An. Init.	Reporting Limit	Results Total	
Chloride	Cal 422	04/21/10	RLH	**50 mg/kg**	< **50**	**mg/kg**
			Comments:	< 50 ppm		
Sulfate	Cal 417	04/21/10	RLH	**50 mg/kg**	< **50**	**mg/kg**
			Comments:	< 50 ppm		

NOTE: Sample sets with a supplied Field Blank have been blank corrected unless otherwise noted.

Analyst: Robin Hofrichter Date: 04/21/10

Robin Hofrichter - General Chemistry

Approved: _____ Date: 04/22/10

Rachelle Hergenroeder - Project Coordinator

Time, flow rate, and/or sample volume data are based on client supplied information, unless otherwise noted. All analytical quality control results for this tracking sheet met laboratory QC guidelines unless stated otherwise above.

***** END OF REPORT *****

NOTE: Sample sets with a supplied Field Blank have been blank corrected unless otherwise noted.

Geotechnics
.....GEOTECHNICAL AND
GEOSYNTHETIC SERVICES

RECEIVED
JUN - 3 2010
McMahon & Mann
Consulting Engineers, P.C.

June 1, 2010

Project No. 2010-146-02

Mr. Andre Klettke
McMahon & Mann Consulting Engineers
2495 Main Street, Suite 432
Buffalo, NY 14214

Transmittal
Laboratory Test Results
NCHRP Soil Berm 06-018

Please find attached the laboratory test results for the above referenced project. The tests were outlined on the Project Verification Form that was faxed to your firm prior to the testing. The testing was performed in general accordance with the methods listed on the enclosed data sheets. The test results are believed to be representative of the samples that were submitted for testing and are indicative only of the specimens which were evaluated. We have no direct knowledge of the origin of the samples and imply no position with regard to the nature of the test results, i.e. pass/fail and no claims as to the suitability of the material for its intended use.

The test data and all associated project information provided shall be held in strict confidence and disclosed to other parties only with authorization by our Client. The test data submitted herein is considered integral with this report and is not to be reproduced except in whole and only with the authorization of the Client and Geotechnics. The remaining sample materials for this project will be retained for a minimum of 90 days as directed by the Geotechnics' Quality Program.

We are pleased to provide these testing services. Should you have any questions or if we may be of further assistance, please contact our office.

Respectively submitted,
Geotechnics, Inc.

David R. Backstrom
Laboratory Director

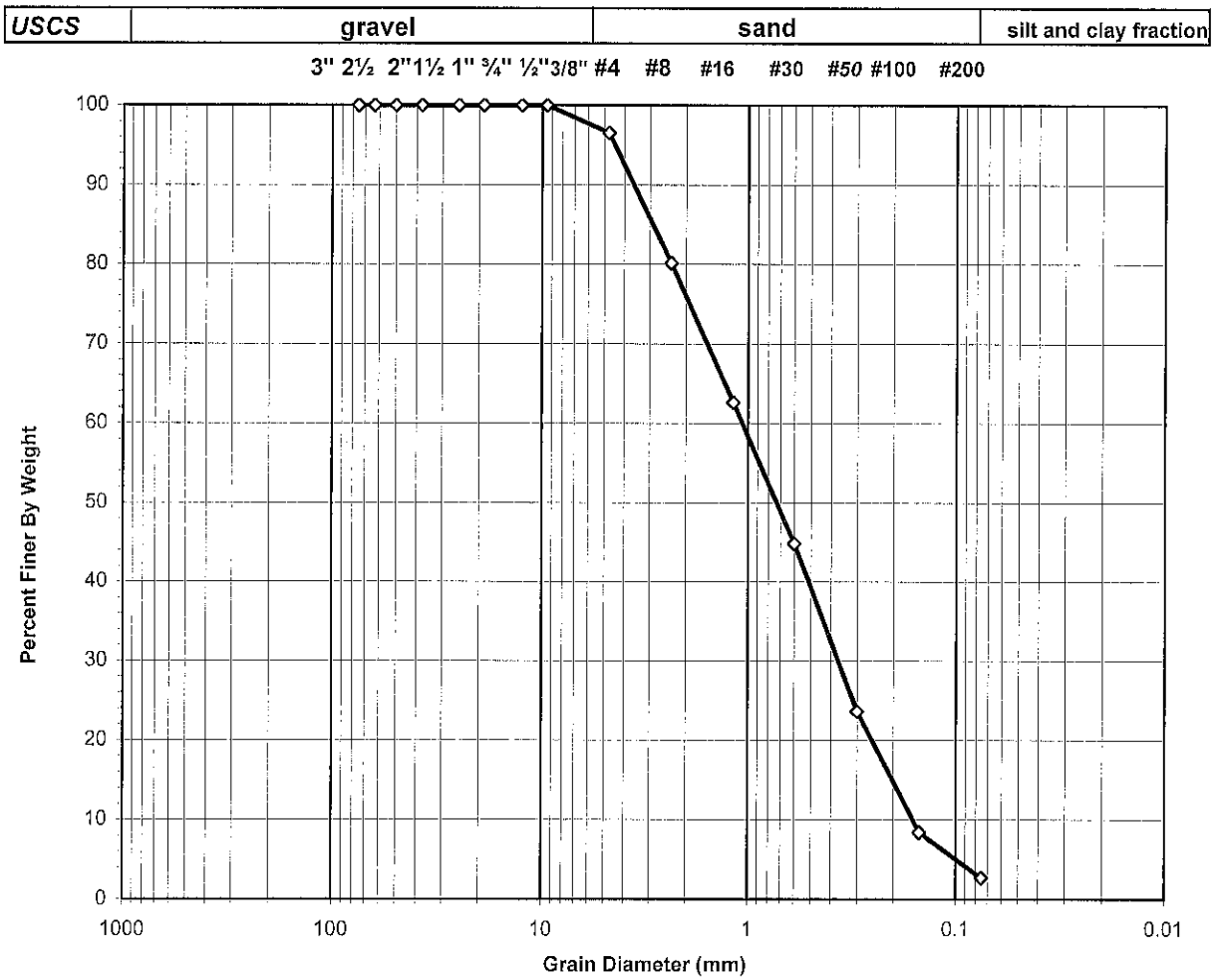
We understand that you have a choice in your laboratory services and we thank you for choosing Geotechnics.

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

WASH SIEVE ANALYSIS OF AGGREGATES
#REF!

Client	McMAHON & MANN	Boring No.	CONC. SAND
Client Reference	NCHRP SOIL BERM 06-018	Depth(ft.)	NA
Project No.	2010-146-02	Sample No.	042910-3
Lab ID	2010-146-02-01		
		Color	**BROWN**



USCS Symbol:	***SP***	D60 =	1.07		
		D30 =	0.37	CC =	0.79
Fineness Modulus:	***2.84***	D10 =	0.16	CU =	6.62

USCS Classification: ***POORLY GRADED SAND***

Tested By PC Date 5/7/10 Checked By KB Date 5-11-10

DCN: CT-S42A DATE 08/26/09 REVISION: 11
C:\MSOFFICE\Excel\PrintQ\jy703.xls]Sheet1

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

WASH SIEVE ANALYSIS OF AGGREGATES
AASHTO T11 and T27

Client	McMAHON & MANN	Boring No.	CONC. SAND
Client Reference	NCHRP SOIL BERM 06-018	Depth(ft.)	NA
Project No.	2010-146-02	Sample No.	042910-3
Lab ID	2010-146-02-01		
		Color	**BROWN**

Tare No.	2451	Wt. of Dry Specimen (gm)	909.78
Wt. Tare + DS.(Pre-Wash)	1012.60	Wt. of +#200 Specimen(gm)	885.24
Wt.Tare + Dry,Washed specimen	988.60	Wt. of -#200 Specimen(gm)	25.29
Wt Tare	102.82		
Wt. Dry,Washed specimen	885.78		

Total Wt. Retained After Sieving 886.53

% Difference Wt. Dry,Washed specimen vs Total Wt. Retained After Sieving -0.1
Note: % Difference must not be more than 0.3

Sieve	Sieve Opening (mm)	Weight Retained (gm.)	Percent Retained	Accumulated Percent Retained	**Percent Finer**
3"	75	0.00	0	0	100
2 1/2"	63	0.00	0	0	100
2"	50	0.00	0	0	100
1 1/2"	37.50	0.00	0	0	100
1"	25.00	0.00	0	0	100
3/4"	19.00	0.00	0	0	100
1/2"	12.50	0.00	0	0	100
3/8"	9.50	0.00	0	0	100
#4	4.75	31.69	3	3	97
#8	2.36	148.98	16	20	80
#16	1.18	159.30	18	37	63
#30	0.60	161.92	18	55	45
#50	0.30	193.20	21	76	24
#100	0.15	138.19	15	92	8
#200	0.075	51.96	5.7	97.3	2.7
Pan	-	1.29	0	97	-

Tested By PC Date 5/7/10 Checked By KB Date 5-11-10

DCN: CT-S42A DATE 08/26/09 REVISION: 11
C:\MSOFFICE\Excel\PrintQ\y703.xls]Sheet1

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

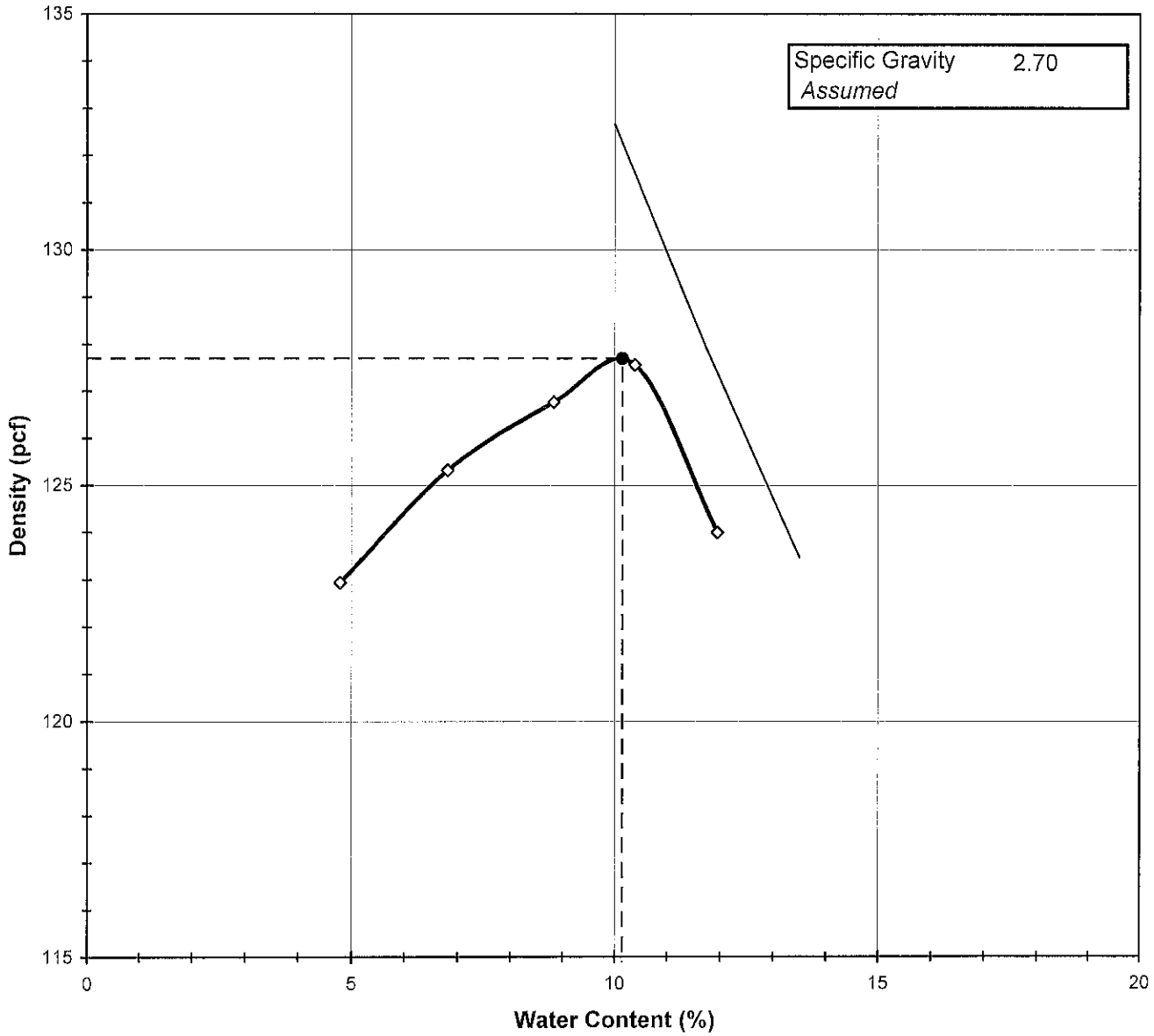
MOISTURE DENSITY RELATIONSHIP

AASHTO T 99

Client	McMAHON & MANN	Boring No.	CONC. SAND
Client Reference	NCHRP SOIL BERM 06-018	Depth (ft)	NA
Project No.	2010-146-02	Sample No.	042910-3
Lab ID	2010-146-02-01	Test Method	**MODIFIED**

Visual Description BROWN SAND

Optimum Water Content **10.2**
Maximum Dry Density **127.7**



Tested By *MF* *Date* *5/6/10* *Checked By* KB *Date* 5-11-10

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

MOISTURE - DENSITY RELATIONSHIP

AASHTO T 99

Client	McMAHON & MANN	Boring No.	CONC. SAND
Client Reference	NCHRP SOIL BERM 06-018	Depth (ft)	NA
Project No.	2010-146-02	Sample No.	042910-3
Lab ID	2010-146-02-01		

Visual Description BROWN SAND

Total Weight of the Sample (gm)	NA
As Received Water Content(%)	NA
Assumed Specific Gravity	2.70
Percent Retained on 3/4"	NA
Percent Retained on 3/8"	NA
Percent Retained on #4	NA
Oversize Material	Not included
Procedure Used	B

TestType		**MODIFIED**
Rammer Weight (lbs)		10.0
Rammer Drop (in)		18
Rammer Type		MECHANICAL
Machine ID	G	441
Mold ID	G	1031
Mold diameter		4"
Weight of the Mold		4263
Volume of the Mold(cc)		943

Mold / Specimen

Point No.	**1**	**2**	**3**	**4**	**5**
Wt. of Mold & WS (gm)	6210	6286	6348	6391	6361
Wt.of Mold (gm)	4263	4263	4263	4263	4263
Wt. of WS	1947	2023	2085	2128	2098
Mold Volume (cc)	943	943	943	943	943

Moisture Content / Density

Tare Number	885	1703	623	876	589
Wt. of Tare & WS (gm)	313.64	326.33	347.65	457.40	656.10
Wt. of Tare & DS (gm)	304.32	310.84	326.21	424.70	594.90
Wt. of Tare (gm)	110.06	83.61	83.51	109.96	82.98
Wt. of Water (gm)	9.32	15.49	21.44	32.70	61.20
Wt. of DS (gm)	194.26	227.23	242.70	314.74	511.92

Wet Density (gm/cc)	2.06	2.15	2.21	2.26	2.22
Wet Density (pcf)	128.8	133.9	138.0	140.8	138.8
Moisture Content (%)	**4.8**	**6.8**	**8.8**	**10.4**	**12.0**
Dry Density (pcf)	**122.9**	**125.3**	**126.8**	**127.6**	**124.0**

Zero Air Voids

Moisture Content (%)	10.0	11.7	13.5
Dry Unit Weight (pcf)	132.7	127.9	123.5

Tested By MF *Date* 5/6/10 *Checked By* KB *Date* 5-11-10

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

geotechnics
INTEGRITY IN TESTING

pH OF SOILS
AASHTO T 289-91
(SOP- S36)

Client	McMAHON & MANN
Client Reference	NCHRP SOIL BERM 06-018
Project No.	2010-146-02

Lab ID		01
Boring No.		CONC. SAND
Depth (ft)		NA
Sample No.		042910-3
Drying Tare No.		662
Testing Tare No.		E
Temperature (° C)		23
pH of Sample	***Test 1***	**8.2**
	Test 2	**8.2**
Agreement (+/- 0.2 units)		0.0

Meter Calibration

Buffer pH	Meter Reading	Meter Model
4.00	4.01	ORION 720A
7.00	7.01	
10.00	10.00	

pH of Distilled Water (Acceptable range 6.5-7.5)	7.1

Tested By KBL Date 5/13/2010 Checked By HB Date 5-17-10

DCN: CT-S36 DATE 2/25/10 REVISION: 4
C:\MSOFFICE\EXCEL\PrintQ\[G581A.xls]Sheet1

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

Minimum Resistivity

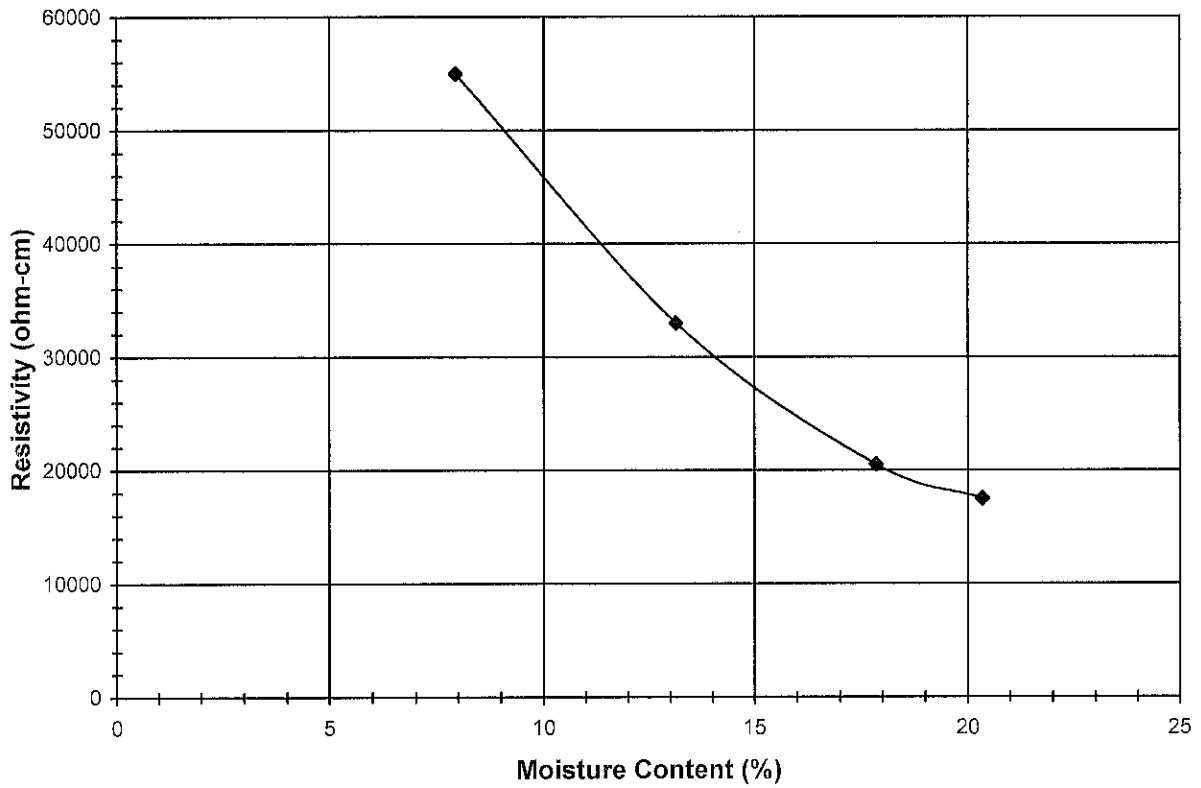
AASHTO T288 (SOP - S56)

geotechnics
INTEGRITY IN TESTING

Client	McMahon & Mann	Boring No.	CONC. SAND
Client Reference	NCHRP SOIL BERM 06-018	Depth (ft)	NA
Project No.	2010-146-02	Sample No.	042910-3
Lab ID	2010-146-02-01	Visual Description	BROWN SAND (- #10 Sieve material)

Tare No.	104	317	262	145	
Tare & Wet Specimen (gm)	42.85	46.75	44.56	58.89	
Tare & Dry Specimen (gm)	40.85	43.50	40.35	52.48	
Tare Weight (gm)	15.64	18.74	16.76	20.97	
Moisture Content (%)	**7.9**	**13.1**	**17.8**	**20.3**	(Saturated)
Resistance (ohm)	**55000**	**33000**	**20500**	**17500**	
Resistivity (ohm-cm)	**55000**	**33000**	**20500**	**17500**	

Note: The ratio of Miller Box area versus distance between electrodes is equal to 1.



Soil Class	*Corrosion Resistance*	*Specific Resistivity (ohm-cm)*
1	Excellent	10,000 - 6,000
2	Good	6,000 - 4,500
3	Fair	4,500 - 2,000
4	Bad	2,000 - 0

Tested By PC *Date* 5/10/10 *Checked By* KB *Date* 5-11-10

544 Braddock Avenue • East Pittsburgh, PA 15112 • 412-823-7600 • FAX 412-823-8999 • www.geotechnics.net

Clark Laboratories LLC
IH and Environmental Laboratories
1801 Route 51 South - Bldg 9
Jefferson Hills, PA 15025
PHONE: 412-387-1001 FAX: 412-387-1028
AIHA Lab ID: 100355 *http://www.clarklabsllc.com*

Preliminary Report
Tracking Sheet: 43812

Clark Laboratories LLC

Geotechnics, Inc. (PA) Geotechnics, Inc. 544 Bradock Avenue East Pittsburgh, PA 15112	Customer Code: 1300 - 0001 Attention: James Moyer Work Req By: Kevin Lichtenfels Customer P.O.: Project Number: NCHRP SOIL BERM 06-018	Phone: (412) 823-7600 Fax: (412) 823-8999 Email: klichtenfels@geotechnics.net Loc: Date Rcvd: 05/06/10	

Sample Id: **000329128** Client Sample Id: **CONC. SAND 042910-3** Sampling Date: 05/05/10

Analyte	Method	Analysis Date	An. Init.	Reporting Limit	Results		Total	
Chloride	Cal 422	05/12/10	RLH	**50 mg/kg**		<	**50**	**mg/kg**
			Comments:	< 50ppm				
Sulfate	Cal 417	05/12/10	RLH	**50 mg/kg**		<	**50**	**mg/kg**
			Comments:	< 50ppm				

NOTE: Sample sets with a supplied Field Blank have been blank corrected unless otherwise noted.

Analyst: Robin Hofrichter Date: 05/12/10

Robin Hofrichter - General Chemistry

Time, flow rate, and/or sample volume data are based on client supplied information, unless otherwise noted. All analytical quality control results for this tracking sheet met laboratory QC guidelines unless stated otherwise above.

***** END OF REPORT *****

NOTE: Sample sets with a supplied Field Blank have been blank corrected unless otherwise noted.

SAMPLE ID/TEST LOCATION	MOISTURE CONTENT (%)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	WET DENSITY (pcf)	RESISTANCE (Ω)	VOID RATIO	DEGREE OF SATURATION
	MMCE SAMPLES	THIRD ROCK NUCLEAR					
MCA1Fa	7.62						
MCA1Fb	8.96						
MCA1Fc	9.85						
A 1f		9.5	116.7	127.7		0.42	60.4
A 2f		9.6	112	122.8		0.48	53.4
MCB1F	7.9						
B 3f		9.3	114.7	125.4		0.44	55.8
B 4f		8.7	115.2	125.2		0.44	52.9
B 5f		8.7	114.5	124.5		0.44	51.9
B 6f		10.2	110.6	121.9		0.50	54.6
MCC1Fa	7.29						
MCC1Fb	7.06						
MCC2F	10.06						
MCC4F	10.55						
C 7f		8.4	113.3	122.8		0.46	48.4
C 8f		8.8	113.8	123.8		0.45	51.5
C 9f		8.1	115.5	124.5		0.43	49.7
MCD1Fa	11.48						
MCD1Fb	8.94						
D 10f		9	115	125.4		0.44	54.5
D 11f		10.2	118.2	130.2		0.40	67.7
D-E 12f		9.4	117.2	128.2		0.41	60.6
E 13f		11.6	114.4	127.7		0.45	69.0
E 14f		9.7	117	128.3		0.41	62.2
E 15f		9.5	112.5	123.3		0.47	53.6
F 16f		9	113.8	124.8		0.45	52.6
RESIST B1F	8.42		106.9	117.55	3015	0.55	40.8
RESIST C2F	9.47		108.1	119.61	2694.2	0.53	47.4
RESIST C4F	9.3		108.9	121.25	2621.8	0.52	47.5
A 3c		5.9	125.8	133.2		0.31	49.7
A 4c		3.7	124.8	129.4		0.33	30.2
MCB2C	10.1						
MCB4C	6.69						
B 5c		7.5	133	143		0.24	81.7
B 6c		9.2	130.2	142.1		0.27	90.3
B 7c		9.4	131.5	143.8		0.26	96.7
B 8c		9.5	132.3	144.9		0.25	100.7
MCC1C	5.04						
C 9c		6.3	127.3	135.2		0.30	55.8
C 10c		5.9	127.1	134.6		0.30	51.9
MCD1C	6.28						
MCD4C	7.46						
D 11c		8.7	130.7	142		0.27	86.9
D 12c		5.9	128.5	136.1		0.29	54.5
D 13c		7.7	129.4	139.3		0.28	73.4
D 14c		7.4	128.8	138.3		0.28	69.1
E 15c		8.2	132.2	143		0.25	86.6
E 16c		8.9	123.8	134.8		0.34	70.3
F 17c		6	126.7	134.3		0.31	52.1
F 18c		5.2	126.2	132.8		0.31	44.4
F 19c		5.4	129	135.9		0.28	50.8
RESIST B2C	10.53		107.7	121.25	1122.7	0.54	52.1
RESIST C1C	9.61		113.4	126.18	1041.3	0.46	55.6
RESIST D4C	11.73		106.9	122.07	1189	0.55	56.8

NUCLEAR DENSITOMETER FIELD LOG

Project:	Resistivity Experiment Test Pad@Earth Dimensions	Date:	06/02/10
Client:	McMahon and Mann	Report No.:	1
Job No.:		Inspector:	Robert Poczkalski
Contractor:	Earth Dimensions, Inc.	Page	1 of 4

PROCTOR DATA:

Type of Material	**Coarse (Gravel Mat'l)**	
Source Area	**On-Site Stockpile**	
Maximum Density	**139.6**	pcf
Optimum Moisture Content	**6.9**	%

NUCLEAR DENSITOMETER RESULTS:

STANDARD COUNTS

Density: 2174

Moisture: 692

GAUGE INFORMATION:

Troxler Model No.: 3440

Troxler Serial No.: 32534

TEST NUMBER	1c	2c	3c	4c	5c	6c	7c	8c	9c	10c	11c	12c
DEPTH OR ELEVATION (in.)	8	6	6	4	6	6	6	6	6	6	6	6
PERCENT COMPACTION (%)	85.6	85.1	90.1	89.4	95.3	93.3	94.2	94.8	91.2	91.1	93.6	92.1
DRY DENSITY (pcf)	119.5	118.8	125.8	124.8	133.0	130.2	131.5	132.3	127.3	127.1	130.7	128.5
WET DENSITY (pcf)	121.8	121.2	133.2	129.4	143.0	142.1	143.8	144.9	135.2	134.6	142.0	136.1
PERCENT MOISTURE (%)	1.9	2.0	5.9	3.7	7.5	9.2	9.4	9.5	6.3	5.9	8.7	5.9

LOCATION:

TEST NO. (from above)	**X**	**Y**	**Z**
1c	X1F		
2c	X1C		
3c	A1Ca	~6' From Post	
4c	A1Cb	~12' From Post	
5c	B1Ca	~6' From Post	
6c	B3Ca	~6' From Post	

TEST NO. (from above)	**X**	**Y**	**Z**
7c	B3Cb	~10' From Post	
8c	B1Cb	~14' From Post	
9c	C2Ca	~4' From Post	
10c	C1Ca	~9' From Post	
11c	D Ca	~6' From Post	~2' From Plywood
12c	D Cb	~10' From Post	~3' From Plywood

REMARKS:

SIGNED: Robert Poczkalski **DATE:** 6/02/2010

NUCLEAR DENSITOMETER FIELD LOG

Project:	Resistivity Experiment Test Pad@Earth Dimensions	Date:	06/02/10
Client:	McMahon and Mann	Report No.:	1
Job No.:		Inspector:	Robert Poczkalski
Contractor:	Earth Dimensions, Inc.	Page	2 of 4

PROCTOR DATA:

Type of Material	**Coarse (Gravel Mat'l)**	
Source Area	**On-Site Stockpile**	
Maximum Density	**139.6**	pcf
Optimum Moisture Content	**6.9**	%

NUCLEAR DENSITOMETER RESULTS:

STANDARD COUNTS

Density: 2174

Moisture: 692

GAUGE INFORMATION:

Troxler Model No.: 3440

Troxler Serial No.: 32534

TEST NUMBER	13c	14c	15c	16c	17c	18c	19c					
DEPTH OR ELEVATION (in.)	6	6	6	6	6	6	6					
PERCENT COMPACTION (%)	92.7	92.2	94.7	88.7	90.8	90.4	92.4					
DRY DENSITY (pcf)	129.4	128.8	132.2	123.8	126.7	126.2	129.0					
WET DENSITY (pcf)	139.3	138.3	143.0	134.8	134.3	132.8	135.9					
PERCENT MOISTURE (%)	7.7	7.4	8.2	8.9	6.0	5.2	5.4					

LOCATION:

TEST NO. (from above)	**X**	**Y**	**Z**
13c	D Cc	~6' From Post	~5' From Plywood
14c	D Cd	~14' From Post	~5' From Plywood
15c	E2Ca	~7' From Post	~3.5' From Plywood
16c	E1Cb	~7' From Post	~3.5' From Plywood
17c	F2Ca	~4' From Post	~3' From Plywood
18c	F2Cb	~8' From Post	~3' From Plywood

TEST NO. (from above)	**X**	**Y**	**Z**
19c	F1Cb	~8' From Post	

REMARKS:

SIGNED: Robert Poczkalski **DATE:** 6/02/2010

NUCLEAR DENSITOMETER FIELD LOG

Project:	Resistivity Experiment Test Pad@Earth Dimensions	Date:	06/02/10
Client:	McMahon and Mann	Report No.:	1
Job No.:		Inspector:	Robert Poczkalski
Contractor:	Earth Dimensions, Inc.	Page	3 of 4

PROCTOR DATA:

Type of Material	**Fine (Concrete Sand)**	
Source Area	**On-Site Stockpile**	
Maximum Density	**127.7**	pcf
Optimum Moisture Content	**10.2**	%

NUCLEAR DENSITOMETER RESULTS:

STANDARD COUNTS

Density: 2174

Moisture: 692

GAUGE INFORMATION:

Troxler Model No.: 3440

Troxler Serial No.: 32534

TEST NUMBER	1f	2f	3f	4f	5f	6f	7f	8f	9f	10f	11f	12f
DEPTH OR ELEVATION (in.)	6	6	6	6	6	6	6	6	6	6	6	6
PERCENT COMPACTION (%)	91.4	87.7	89.8	90.2	89.7	86.6	88.7	89.1	90.2	90.1	92.6	91.8
DRY DENSITY (pcf)	116.7	112.0	114.7	115.2	114.5	110.6	113.3	113.8	115.2	115.0	118.2	117.2
WET DENSITY (pcf)	127.7	122.8	125.4	125.2	124.5	121.9	122.8	123.8	124.5	125.4	130.2	128.2
PERCENT MOISTURE (%)	9.5	9.6	9.3	8.7	8.7	10.2	8.4	8.8	8.1	9.0	10.2	9.4

LOCATION:

TEST NO. (from above)	**X**	**Y**	**Z**
1f	A1Fa	~6' From Post	
2f	A1Fb	~12' From Post	
3f	B1Fa	~5' From Post	~2' From Plywood
4f	B2Fa	~5' From Post	~4' From Plywood
5f	B1Fb	~12' From Post	~2' From Plywood
6f	B2Fb	~12' From Post	~4' From Plywood

TEST NO. (from above)	**X**	**Y**	**Z**
7f	C1Fa	~6' From Post	
8f	C1Fb	~12' From Post	
9f	C4F		
10f	D1Fa	~4' From Post	
11f	D1Fb	~14' From Post	
12f	D/E F	~9' From Post	~3.5' From Plywood

REMARKS:

SIGNED: Robert Poczkalski **DATE:** 6/02/2010

NUCLEAR DENSITOMETER FIELD LOG

Project:	Resistivity Experiment Test Pad@Earth Dimensions	Date:	06/02/10
Client:	McMahon and Mann	Report No.:	1
Job No.:		Inspector:	Robert Poczkalski
Contractor:	Earth Dimensions, Inc.	Page	4 of 4

PROCTOR DATA:

Type of Material	**Fine (Concrete Sand)**	
Source Area	**On-Site Stockpile**	
Maximum Density	**127.7**	pcf
Optimum Moisture Content	**10.2**	%

NUCLEAR DENSITOMETER RESULTS:

STANDARD COUNTS

Density: 2174

Moisture: 692

GAUGE INFORMATION:

Troxler Model No.: 3440

Troxler Serial No.: 32534

TEST NUMBER	13f	14f	15f	16f								
DEPTH OR ELEVATION (in.)	6	6	6	6								
PERCENT COMPACTION (%)	89.6	91.6	88.1	89.1								
DRY DENSITY (pcf)	114.4	117.0	112.5	113.8								
WET DENSITY (pcf)	127.7	128.3	123.3	124.8								
PERCENT MOISTURE (%)	11.6	9.7	9.5	9.0								

LOCATION:

TEST NO. (from above)	**X**	**Y**	**Z**
13f	E1Fa	~14' From Post	
14f	E1Fb	~7' From Post	
15f	E3Fc	~5' From Post	~5' from plywood
16f	F1Fa	~8' From Post	

TEST NO. (from above)	**X**	**Y**	**Z**

REMARKS:

SIGNED: Robert Poczkalski **DATE:** 6/02/2010

Wednesday, June 23, 2010

DATE	WORKING ELECTRODE	COUNTER ELECTRODE	HALF CELL LOCATION	DISTANCE FROM WE TO CE	DISTANCE FROM HC TO WE	SURFACE AREA OF WORKING ELECTRODE	SOLUTION RESISTANCE	NOTE
				(cm)	(cm)	(cm^2)	(Ω)	
June 23, 2010	A1F	C1F		36.2712		903.482	130.3	2 electrode test
June 23, 2010	A1F	C1F	HC2	36.2712	252.3	903.482	47.3	L to front
June 23, 2010	A1F	C1F	HC2	36.2712	402.6	903.482	47.3	L to midpoint
June 23, 2010	B1F	D1F		25.908		8046.58	33.9	2 electrode test
June 23, 2010	B1F	E1F		55.7784		8046.58	54.5	2 electrode test
June 23, 2010	B1F	D1F	HC2	25.908	255.7	8046.58	11.9	L to front
June 23, 2010	B1F	E1F	HC2	55.7784	255.7	8046.58	16	L to front
June 23, 2010	B1F	D1F	HC2	25.908	404.8	8046.58	11.9	L to midpoint
June 23, 2010	B1F	E1F	HC2	55.7784	404.8	8046.58	16	L to midpoint
June 23, 2010	B2F	E2F		58.5216		566.321	447	2 electrode test
June 23, 2010	B2F	E2F	HC2	58.5216	252.3	566.321	146.7	L to front
June 23, 2010	B2F	E2F	HC2	58.5216	267.3	566.321	146.7	L to midpoint
June 23, 2010	B2F	E2F	HC3		343.3	566.321	150.4	L to front
June 23, 2010	B2F	E2F	HC4		672.3	566.321	150.9	L to front
June 23, 2010	B3F	E3F		58.8264		342.825	1028	2 electrode test
June 23, 2010	B3F	E3F	HC2	58.8264	256.2	342.825	334.9	L to front
June 23, 2010	B3F	E3F	HC2	58.8264	270.9	342.825	334.9	L to midpoint
June 23, 2010	B3F	E3F	HC3		368	342.825	347.1	L to front
June 23, 2010	B4F	E4F		57.912		3385.61	172.8	2 electrode test
June 23, 2010	B4F	E4F	HC2	57.912	259.8	3385.61	50.8	L to front
June 23, 2010	B4F	E4F	HC2	57.912	407.4	3385.61	50.8	L to midpoint
June 23, 2010	B5F	B6F		45.72		342.825	728.5	2 electrode test
June 23, 2010	B5F	B6F	HC2	45.72	264	342.825	317	L to front
June 23, 2010	B5F	B6F	HC2	45.72	278.3	342.825	317	L to midpoint
June 23, 2010	B5F	E3F		64.6		342.825	1103.3	2 electrode test
June 23, 2010	B6F	B5F		45.72		342.825	732	2 electrode test
June 23, 2010	B6F	B5F	HC2	45.72	269	342.825	401.9	L to front
June 23, 2010	B6F	B5F	HC2	45.72	283	342.825	401.9	L to midpoint
June 23, 2010	B6F	E3F		73.1		342.825	1224.6	2 electrode test
June 23, 2010	C1F	A1F		36.2712		903.482	115.3	2 electrode test
June 23, 2010	C1F	A1F	HC2	36.2712	259	903.482	78.8	L to front
June 23, 2010	C1F	A1F	HC2	36.2712	406.9	903.482	78.8	L to midpoint
June 23, 2010	D1F	B1F		25.908		8046.58	34.2	2 electrode test
June 23, 2010	D1F	E1F		29.8704		8046.58	41.5	2 electrode test
June 23, 2010	D1F	B1F	HC2	25.908	262.1	8046.58	21.1	L to front
June 23, 2010	D1F	E1F	HC2	29.8704	262.1	8046.58	14.4	L to front
June 23, 2010	D1F	B1F	HC2	25.908	408.8	8046.58	21.1	L to midpoint
June 23, 2010	D1F	E1F	HC2	29.8704	408.8	8046.58	14.4	L to midpoint
June 23, 2010	E1F	B1F		55.7784		8046.58	54.4	2 electrode test
June 23, 2010	E1F	D1F		29.8704		8046.58	43.2	2 electrode test
June 23, 2010	E1F	B1F	HC2	55.7784	272.3	8046.58	36.6	L to front
June 23, 2010	E1F	D1F	HC2	29.8704	272.3	8046.58	26.5	L to front
June 23, 2010	E1F	B1F	HC2	55.7784	415.5	8046.58	36.6	L to midpoint
June 23, 2010	E1F	D1F	HC2	29.8704	415.5	8046.58	26.5	L to midpoint
June 23, 2010	E2F	B2F		58.5216		566.321	429.1	2 electrode test
June 23, 2010	E2F	B2F	HC2	58.5216	269.8	566.321	277.4	L to front
June 23, 2010	E2F	B2F	HC2	58.5216	283.8	566.321	277.4	L to midpoint
June 23, 2010	E3F	B3F		58.8264		342.825	1055	2 electrode test
June 23, 2010	E3F	B3F	HC2	58.8264	273.3	342.825	704.7	L to front
June 23, 2010	E3F	B3F	HC2	58.8264	287.2	342.825	704.7	L to midpoint
June 23, 2010	E4F	B4F		57.912		3385.61	167	2 electrode test
June 23, 2010	E4F	B4F	HC2	57.912	276.8	3385.61	116.5	L to front
June 23, 2010	E4F	B4F	HC2	57.912	418.5	3385.61	116.5	L to midpoint

DATE	WORKING ELECTRODE	COUNTER ELECTRODE	HALF CELL LOCATION	DISTANCE FROM WE TO CE	DISTANCE FROM HC TO WE	SURFACE AREA OF WORKING ELECTRODE	SOLUTION RESISTANCE	NOTE
				(cm)	(cm)	(cm^2)	(Ω)	
June 23, 2010	B1C	E1C	2E	58.8264		8263.2	31	2 electrode test
June 23, 2010	B1C	E1C	HC3	58.8264	238.3	8263.2	7.5	L to front
June 23, 2010	B1C	E1C	HC3	58.8264	374.7	8263.2	7.5	L to midpoint
June 23, 2010	B2C	E2C	2E	58.8264		566.3	229	2 electrode test
June 23, 2010	B2C	E2C	HC3	58.8264	213.7	566.3	58	L to front
June 23, 2010	B2C	E2C	HC3	58.8264	227.9	566.3	58	L to midpoint
June 23, 2010	B3C	E3C	2E	58.8264		342.8	598	2 electrode test
June 23, 2010	B3C	E3C	HC3	58.8264	207	342.8	159	L to front
June 23, 2010	B3C	E3C	HC3	58.8264	221.7	342.8	159	L to midpoint
June 23, 2010	B4C	E4C	2E	58.8264		3385.6	88	2 electrode test
June 23, 2010	B4C	E4C	HCE	58.8264	205.4	3385.6	21	L to front
June 23, 2010	B4C	E4C	HC3	58.8264	354.7	3385.6	21	L to midpoint
June 23, 2010	B5C	B6C	2E	45.72		566.3	156	2 electrode test
June 23, 2010	B5C	B6C	HC3	45.72	205.4	566.3	57	L to front
June 23, 2010	B5C	B6C	HC3	45.72	220.1	566.3	57	L to midpoint
June 23, 2010	B6C	B5C	2E	45.72		566.3	158	2 electrode test
June 23, 2010	B6C	B5C	HC3	45.72	213.7	566.3	94	L to front
June 23, 2010	B6C	B5C	HCE	45.72	227.9	566.3	94	L to midpoint
June 23, 2010	B1C	E1C	HC2	58.8264	284.2	8263.2	7.4	L to front
June 23, 2010	B1C	E1C	HC2	58.8264	423.4	8263.2	7.4	L to midpoint
June 23, 2010	E1C	B1C	2E	58.8264		8263.2	31	2 electrode test
June 23, 2010	E1C	B1C	HC3	58.8264	254.7	8263.2	24	L to front
June 23, 2010	E1C	B1C	HC3	58.8264	385.3	8263.2	24	L to midpoint
June 23, 2010	E2C	B2C	2E	58.8264		566.3	239	2 electrode test
June 23, 2010	E2C	B2C	HC3	58.8264	231.7	566.3	178	L to front
June 23, 2010	E2C	B2C	HC3	58.8264	244.9	566.3	178	L to midpoint
June 23, 2010	E3C	B3C	2E	58.8264		342.8	606	2 electrode test
June 23, 2010	E3C	B3C	HC3	58.8264	225.7	342.8	460	L to front
June 23, 2010	E3C	B3C	HCE	58.8264	239.1	342.8	460	L to midpoint
June 23, 2010	E4C	B4C	2E	58.8264		3385.6	89	2 electrode test
June 23, 2010	E4C	B4C	HC3	58.8264	210.8	3385.6	70	L to front
June 23, 2010	E4C	B4C	HC3	58.8264	365.8	3385.6	70	L to midpoint

`

APPENDIX III

CALTRANS TEST SITES AND MSE WALLS FIELD MONITORING AND LABORATORY DATA

1. Gilroy - View of Site.

2. Gilroy – Inspection Elements and Monitoring Locations.

3. Gilroy – End of inspection element and clip for attaching wires for electrochemical testing.

4. Los Olivos – View of site.

5. Los Olivos – Inspection element location.

6. Los Olivos- Monitoring Station

7. San Luis Obispo – Site View.

8. San Luis Obispo – Access Inspection Elements

9. San Luis Obispo – Inspection element location.

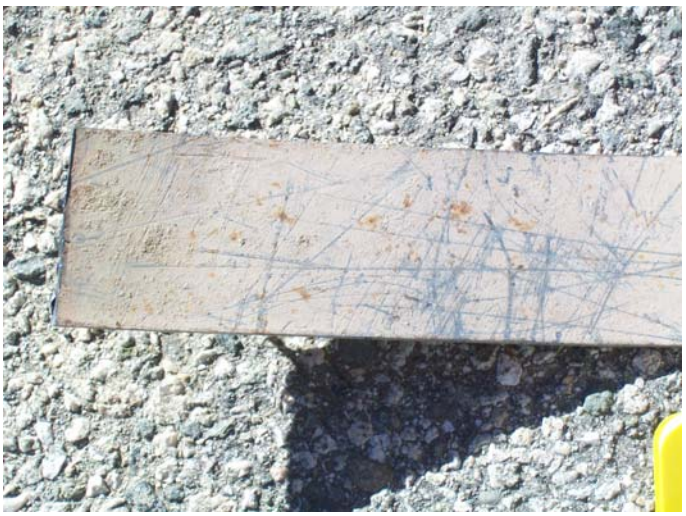
10. San Gabriel – Site view and steel skin facing.

11. San Gabriel – Locations of samples 1,2 and 3.

12. San Gabriel – Placement of half-cell for electrochemical testing.

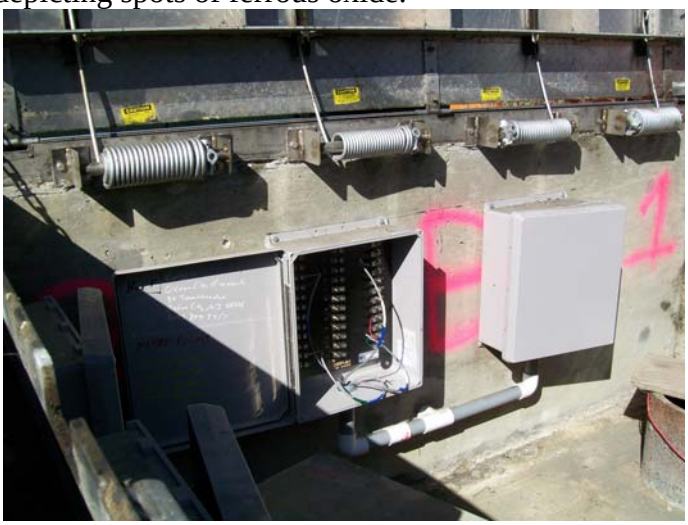
13. San Gabriel- Bottom of exposed reinforcement

14. San Gabriel – Reinforcement specimen cut and removed f rom behind the facing



15. San Gabriel – Close-up of reinforcement specimen depicting spots of ferrous oxide.

16. Emeryville – Cypress Street Bunker B



17. Emeryville – Junction box mounted in Bunker B

18. Emeryville – Half-cell placement

19. Emeryville – Corrosion monitoring and in situ measurement of fill resistance

GILROY

Units	Area (cm^2)	EnvCont	Matrial	Test Name	City	State	Location	Wall	WE	CE	Pr $_{uncomp}$(Ω)	Rs (Ω)	Pr $_{comp}$(Ω)	r (mm/yr)	r (µm/yr)	Ecor (V)	Ecor (mV)	Coef
Metric	3292	0.035	GC	CA-040610-37-0475G-10-4	GILROY	CA	I 25 & I 101	EAST ABUTMENT	10	4	195.0398254	30.77422	164.2656097	0.000752	0.752022	-0.66514039	-665.1403904	0.999251
Metric	3292	0.035	GC	CA-040610-37-0475G-11-5	GILROY	CA	I 25 & I 101	EAST ABUTMENT	11	5	160.6463318	16.50504	144.1412964	0.000857	0.857015	-0.640333712	-640.3337121	0.998315
Metric	3292	0.035	GC	CA-040610-37-0475G-13-7	GILROY	CA	I 25 & I 101	EAST ABUTMENT	13	7	279.3384399	36.92436	242.4140778	0.00051	0.509588	-0.679588914	-679.5889139	0.996051
Metric	3292	0.035	GC	CA-040610-37-0475G-16-4	GILROY	CA	I 25 & I 101	EAST ABUTMENT	16	4	270.2602539	32.33715	237.9230957	0.000519	0.519207	-0.66485554	-664.8555398	0.998882
Metric	3292	0.035	GC	CA-040610-37-0475G-2-8	GILROY	CA	I 25 & I 101	EAST ABUTMENT	2	8	115.4665375	17.55266	97.91387939	0.001262	1.261632	-0.741636157	-741.636157	0.996735
Metric	3292	0.035	GC	CA-040610-37-0475G-4-16	GILROY	CA	I 25 & I 101	EAST ABUTMENT	4	16	147.3901672	13.76341	133.6267548	0.000924	0.92445	-0.693915367	-693.9153671	0.998317
Metric	3292	0.035	GC	CA-040610-37-0475G-5-11	GILROY	CA	I 25 & I 101	EAST ABUTMENT	5	11	160.3448181	16.59679	143.7480316	0.000859	0.85936	-0.640740752	-640.7407522	0.997925
Metric	3292	0.035	GC	CA-040610-37-0475G-7-13	GILROY	CA	I 25 & I 101	EAST ABUTMENT	7	13	358.5027771	34.36831	324.1344604	0.000381	0.381111	-0.820187211	-820.187211	0.994988
Metric	3292	0.035	GC	CA-040610-37-0475G-8-2	GILROY	CA	I 25 & I 101	EAST ABUTMENT	8	2	258.4226074	30.58794	227.834671	0.000542	0.542197	-0.737342298	-737.342298	0.998079
Metric	3292	0.035	GC	CA-040610-37-0475G-8-9	GILROY	CA	I 25 & I 101	EAST ABUTMENT	8	9	221.019989	31.28424	189.7357483	0.000651	0.65107	-0.757631242	-757.6312423	0.999606
Metric	3292	0.035	GC	CA-040610-37-0475G-9-8	GILROY	CA	I 25 & I 101	EAST ABUTMENT	9	8	239.0592957	33.52029	205.5390015	0.000601	0.601011	-0.729731381	-729.7313809	0.995191
Metric	67.73999786	0.035	GC	CA-041210-GILROY-MILLER-1-2E	GILROY	CA	I 25 & I 101	EAST ABUTMENT			65375.04297	1028.896	64346.14844	9.33E-05	0.093297	0.01073559		0.991416
Metric	67.73999786	0.035	GC	CA-041210-GILROY-MILLER-3-2E	GILROY	CA	I 25 & I 101	EAST ABUTMENT			83666.14063	789.3232	82876.82031	7.24E-05	0.072437	0.185480267		0.990462

Los Olivos

Units	Area (cm^2)	EnvCont	Matrial	Test Name	City	State	Location	Wall	WE	CE	Pr $_{uncomp}$(Ω)	Rs (Ω)	Pr (Ω)	r (mm/yr)	r (µ/yr)	Ecor (V)	Ecor (mV)	Coef
Metric	876.2000122	0.035	GC	CA-041310-51-313M-1-2	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	1	2	121.1001968	20.91355	100.1866531	0.004633	4.632588	-0.623300791	-623.3007908	0.999581
Metric	876.2000122	0.035	GC	CA-041310-51-313M-2-1	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	2	1	182.0214081	24.7053	157.3161011	0.00295	2.950261	-0.649979651	-649.979651	0.999823
Metric	876.2000122	0.035	GC	CA-041310-51-313M-3-4	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	3	4	163.8708344	22.04321	141.8276215	0.003272	3.272448	-0.67004478	-670.0447798	0.998149
Metric	876.2000122	0.035	GC	CA-041310-51-313M-4-3	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	4	3	262.9848938	20.44077	242.5441284	0.001914	1.913563	-0.68634516	-686.34516	0.998669
Metric	876.2000122	0.035	GC	CA-041310-51-313M-5-6	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	5	6	171.170578	17.60901	153.5615692	0.003022	3.022393	-0.693325222	-693.3252215	0.99979
Metric	876.2000122	0.035	GC	CA-041310-51-313M-6-5	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	6	5	385.0565186	20.73076	364.3257446	0.001274	1.273924	-0.723789155	-723.7891555	0.99919
Metric	876.2000122	0.035	GC	CA-041310-51-313M-7-6	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	7	6	429.0067139	23.74287	405.263855	0.001145	1.145238	-0.697578371	-697.5783706	0.998737
Metric	876.2000122	0.035	GC	CA-041310-51-313M-8-9	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	8	9	596.0389404	19.2724	576.7665405	0.000805	0.804699	-0.721286118	-721.286118	0.998646
Metric	876.2000122	0.035	GC	CA-041310-51-313M-9-8	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	9	8	266.7879944	19.37734	247.4106445	0.001876	1.875924	-0.710622728	-710.6227279	0.998122

2 Electrode Tests Santa Barbara

Units	Area (cm^2)	EnvCont	Matrial	Test Name	City	State	Location	Wall	WE	CE	Pr $_{uncomp}$(Ω)	Rs (Ω)	Pr (Ω)	r (mm/yr)	r (µ/yr)	Ecor (V)	Ecor (mV)	Coef	r (µ/yr) 2E adj.
Metric	876.2000122	0.035	GC	CA-041310-51-313M-1-2-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	1	2	302.9606018	47.29711	255.6634979	0.001815	1.815369	0.029965404	29.96540442	0.99944	3.630737308
Metric	876.2000122	0.035	GC	CA-041310-51-313M-2-1-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	2	1	285.7024841	47.29546	238.4070282	0.001947	1.946769	-0.029566672	-29.5666717	0.997737	3.893538844
Metric	876.2000122	0.035	GC	CA-041310-51-313M-3-4-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	3	4	426.2686768	43.88471	382.3839722	0.001214	1.213763	0.017289886	17.28988625	0.998294	2.427525818
Metric	876.2000122	0.035	GC	CA-041310-51-313M-4-5-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	4	5	431.3095703	36.74543	394.5641479	0.001176	1.176294	0.004464603	4.464603495	0.997789	2.352588112
Metric	876.2000122	0.035	GC	CA-041310-51-313M-5-4-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	5	4	430.5817566	36.76836	393.813385	0.001179	1.178537	-0.004140275	-4.140275065	0.998347	2.357073128
Metric	876.2000122	0.035	GC	CA-041310-51-313M-6-7-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	6	7	843.6342163	45.05429	798.5799561	0.000581	0.581186	-0.023316035	-23.31603505	0.997848	1.162371947
Metric	876.2000122	0.035	GC	CA-041310-51-313M-7-6-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	7	6	828.3133545	45.03569	783.2776489	0.000593	0.59254	0.023813339	23.81333895	0.998811	1.185080386
Metric	876.2000122	0.035	GC	CA-041310-51-313M-8-9-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	8	9	746.1359253	39.48628	706.6496582	0.000657	0.656794	-0.006035371	-6.035370752	0.998401	1.313588582
Metric	876.2000122	0.035	GC	CA-041310-51-313M-9-8-2E	LOS OLIVOS	CA	ROUTE 154	SOUTHBOUND	9	8	798.3950806	39.43737	758.9577026	0.000612	0.611527	0.008135239	8.135238662	0.996157	1.223054947

San Luis Obispo

Units	Area (cm^2)	EnvCont	Matrial	Test Name	City	State	Location	Wall	WE	CE	Pr $_{uncomp}$(Ω)	Rs (Ω)	Pr (Ω)	r (mm/yr)	r (µ/yr)	Ecor (V)	Ecor (mV)	Coef
Metric	3292	0.035	GC	CA-041410-SANLUIS101-10-4	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	10	4	180.8402557	36.07104	144.7692108	0.000853	0.853298	-0.878428996	-878.4289956	0.998048
Metric	3292	0.035	GC	CA-041410-SANLUIS101-11-17	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	11	17	188.0506439	32.46714	155.5834961	0.000794	0.793987	-0.801994324	-801.9943237	0.997719
Metric	3292	0.035	GC	CA-041410-SANLUIS101-13-7	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	13	7	253.0862427	25.3404	227.7458344	0.000542	0.542409	-0.700610518	-700.6105185	0.998015
Metric	3292	0.035	GC	CA-041410-SANLUIS101-16-10	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	16	10	343.2347107	137.6521	205.5825806	0.000601	0.600884	-0.956288159	-956.2881589	0.995362
Metric	3292	0.035	GC	CA-041410-SANLUIS101-17-11	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	17	11	334.914978	96.67616	238.2388153	0.000519	0.518519	-0.902116418	-902.1164179	0.997042
Metric	3292	0.035	GC	CA-041410-SANLUIS101-18-12	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	18	12	366.1324463	129.686	236.4464569	0.000522	0.522449	-0.845075309	-845.0753093	0.99785
Metric	3292	0.035	GC	CA-041410-SANLUIS101-2-8	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	2	8	127.8134842	19.86949	107.9440002	0.001144	1.144402	-0.633821726	-633.8217258	0.997872
Metric	3292	0.035	GC	CA-041410-SANLUIS101-3-9	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	3	9	96.83451843	10.29117	86.54334259	0.001427	1.427392	-0.641717553	-641.7175531	0.998713
Metric	3292	0.035	GC	CA-041410-SANLUIS101-4-10	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	4	10	135.4272919	15.66796	119.7593384	0.001031	1.031496	-0.683129847	-683.129847	0.998915
Metric	3292	0.035	GC	CA-041410-SANLUIS101-7-13	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	7	13	199.3078461	18.46915	180.8386993	0.000683	0.683102	-0.906532347	-906.5323472	0.998644
Metric	3292	0.035	GC	CA-041410-SANLUIS101-8-2	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	8	2	124.5298386	23.8693	100.6605377	0.001227	1.227207	-0.863268197	-863.2681966	0.997985
Metric	3292	0.035	GC	CA-041410-SANLUIS101-9-3	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	9	3	201.595459	33.21084	168.384613	0.000734	0.733626	-0.832824588	-832.8245878	0.998509

2 Electrode Tests San Luis Obispo

Units	Area (cm^2)	EnvCont	Matrial	Test Name	City	State	Location	Wall	WE	CE	Pr_{uncomp}(Ω)	Rs (Ω)	Pr (Ω)	r (mm/yr)	r (µ/yr)	Ecor (V)	Ecor (mV)	Coef	r (µ/yr) 2E adj.
Metric	3292	0.035	GC	CA-041410-SANLUIS101-10-4-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	10	4	304.3050232	52.50085	251.8041687	0.000491	0.490585	-0.195171952	-195.1719522	0.999155	0.98116952
Metric	3292	0.035	GC	CA-041410-SANLUIS101-11-17-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	11	17	508.1337585	132.3702	375.7635498	0.000329	0.328747	0.104276046	104.2760462	0.999716	0.657494762
Metric	3292	0.035	GC	CA-041410-SANLUIS101-12-18-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	12	18	151.1568604	28.49261	122.6642532	0.001007	1.007068	-0.828917384	-828.9173841	0.999816	2.01413664
Metric	3292	0.035	GC	CA-041410-SANLUIS101-13-7-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	13	7	435.9222107	44.12136	391.8008423	0.000315	0.315291	0.208491042	208.4910423	0.998852	0.630582043
Metric	3292	0.035	GC	CA-041410-SANLUIS101-16-10-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	16	10	505.2610779	179.0309	326.2301636	0.000379	0.378663	-0.075849615	-75.84961504	0.999897	0.757325965
Metric	3292	0.035	GC	CA-041410-SANLUIS101-17-11-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	17	11	512.4816284	132.901	379.5806274	0.000325	0.325442	-0.099330992	-99.33099151	0.999531	0.650883012
Metric	3292	0.035	GC	CA-041410-SANLUIS101-18-12-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	18	12	516.7243042	163.8068	352.9175415	0.00035	0.350029	-0.016320717	-16.32071659	0.99926	0.700057484
Metric	3292	0.035	GC	CA-041410-SANLUIS101-2-8-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	2	8	305.3907471	44.05086	261.3398743	0.000473	0.472684	0.098941796	98.94179553	0.997978	0.94536884
Metric	3292	0.035	GC	CA-041410-SANLUIS101-3-9-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	3	9	287.9477844	44.19881	243.7489624	0.000507	0.506797	0.195024416	195.0244159	0.998608	1.01359433
Metric	3292	0.035	GC	CA-041410-SANLUIS101-4-10-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	4	10	304.4058838	52.44097	251.9649048	0.00049	0.490272	0.200269639	200.2696395	0.999326	0.980543671
Metric	3292	0.035	GC	CA-041410-SANLUIS101-7-13-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	7	13	433.2052612	44.06549	389.1397705	0.000317	0.317447	-0.209559426	-209.5594257	0.998626	0.634894182
Metric	3292	0.035	GC	CA-041410-SANLUIS101-8-2-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	8	2	247.8114624	43.06678	204.7446747	0.000603	0.603343	-0.181293249	-181.2932491	0.998914	1.206686255
Metric	3292	0.035	GC	CA-041410-SANLUIS101-9-3-2E	SAN LUIS OBISPO	CA	ROUTE 101 & Stagecoach Rd	West Wall	9	3	613.3330078	43.7023	569.6306763	0.000217	0.216862	-0.192755386	-192.7553862	0.993947	0.433724141

SAN GABRIEL - CA Rt. 39 - Constructed in 1972

Units	Area (cm^2)	EnvCont	Matrial	Test Name	City	State	Location	Wall	WE	CE	Pr_{uncomp}(Ω)	Rs (Ω)	Pr (Ω)	r (mm/yr)	r (µm/yr)	Ecor (V)	Ecor (mV)	Coef
Metric	11041.75977	0.035	Other	CA-06-09-10-RT 39-1C-2A	SAN GABRIEL	CA	ROUTE 39 WALL	METAL FACED	1C	2A	37.65568924	22.74294	14.91274452	0.00247	2.47	-0.420350015	-420	0.999656
Metric	11041.75977	0.035	Other	CA-06-09-10-RT 39-1C-2B	SAN GABRIEL	CA	ROUTE 39 WALL	METAL FACED	1C	2B	37.96604156	22.69872	15.26732445	0.002412	2.41	-0.424338639	-424	0.999667
Metric	11041.75977	0.035	Other	CA-06-09-10-RT 39-1C-2B-2E	SAN GABRIEL	CA	ROUTE 39 WALL	METAL FACED	1C	2B	90.55387115	53.1943	37.35957336	0.000986	1.97	-0.015670788		0.999842
Metric	10844.66992	0.035	Other	CA-06-09-10-RT 39-2A-1C	SAN GABRIEL	CA	ROUTE 39 WALL	METAL FACED	2A	1C	41.76624298	26.31094	15.45530319	0.002426	2.43	-0.40110907	-401	0.999622
Metric	10844.66992	0.035	Other	CA-06-09-10-RT 39-2A-1C-2E	SAN GABRIEL	CA	ROUTE 39 WALL	METAL FACED	2A	1C	83.06518555	49.42714	33.63804245	0.001115	2.23	0.02007911		0.999757
Metric	11041.75977	0.035	Other	CA-06-09-10-RT 39-2B-1C	SAN GABRIEL	CA	ROUTE 39 WALL	METAL FACED	2B	1C	48.76434326	29.86749	18.89685631	0.001949	1.95	-0.406155884	-406	0.999428
Metric	11041.75977	0.035	Other	CA-06-09-10-RT 39-2B-1C-2E	SAN GABRIEL	CA	ROUTE 39 WALL	METAL FACED	2B	1C	90.58361816	53.12572	37.45789719	0.000983	1.97	0.016151557		0.99971
Metric	67.73999786	0.035	Steel	CA-06-10-10-RT 39-MILLER-2-2E	SAN GABRIEL	CA	HWY 2 RT 39				83884.82813	5253.753	78631.07813	7.63E-05		-0.029401328		0.992575

Units	Area (cm^2)	EnvCont	Matrial	Test Name	City	State	Location	WE	CE	Pr_{uncomp}(Ω)	Rs (Ω)	Pr_{comp}(Ω)	r (mm/yr)	r (μm/yr)	Ecor (V)	Ecor (mV)	Coef	Device	Depth (ft)	Lab–ρ_{min} (Ω-cm)
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXA-1-2	EMERYVILLE	CA	BUNKER B	1	2	5.468197823	2.709042	2.759156227	42.43714	42437.1	-0.636773	-637	0.999537	DMP	30	110
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXA-1-2-2E	EMERYVILLE	CA	BUNKER B	1	2	8.651183128	4.962598	3.688585281	31.74407	63488.1	0.001047		0.999113	DMP	30	110
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXA-2-3	EMERYVILLE	CA	BUNKER B	2	3	2.242887497	1.770747	0.472140431	247.9997	247999.7	-0.636264	-636	0.998412	DMP	30	110
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXA-2-3-2E	EMERYVILLE	CA	BUNKER B	2	3	5.293726921	3.950968	1.342758656	87.2016	174403.2	0.001339		0.999227	DMP	30	110
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXA-3-4	EMERYVILLE	CA	BUNKER B	3	4	3.399574995	2.428119	0.971455574	120.5312	120531.2	-0.637688	-638	0.995808	DMP	30	110
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXA-3-4-2E	EMERYVILLE	CA	BUNKER B	3	4	9.307020187	5.454627	3.852393627	30.39427	60788.5	0.001325		0.99893	DMP	30	110
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXA-4-3	EMERYVILLE	CA	BUNKER B	4	3	5.672326565	2.981773	2.690553188	43.51919	43519.2	-0.641168	-641	0.999798	DMP	30	110
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXA-4-3-2E	EMERYVILLE	CA	BUNKER B	4	3	9.312129021	5.448908	3.863220692	30.30909	60618.2	0.000723		0.998753	DMP	30	110
Metric	8.11	0.026	Iron	CA-040710-CYPRESSB-BOXA-7-8	EMERYVILLE	CA	BUNKER B	7	8	2.61470437	1.11603	1.49867487	24.85501	24855.0	-0.622833	-623	0.999874	CYL	30	110
Metric	8.11	0.026	Iron	CA-040710-CYPRESSB-BOXA-7-8-2E	EMERYVILLE	CA	BUNKER B	7	8	7.235603809	2.227185	5.00841856	7.437392	14874.8	0.001026		0.998778	CYL	30	110
Metric	8.11	0.026	Iron	CA-040710-CYPRESSB-BOXA-8-9	EMERYVILLE	CA	BUNKER B	8	9	4.678863525	1.386369	3.292494774	11.31348	11313.5	-0.624481	-624	0.998708	CYL	30	110
Metric	8.11	0.026	Iron	CA-040710-CYPRESSB-BOXA-8-9-2E	EMERYVILLE	CA	BUNKER B	8	9	7.206372261	2.525511	4.680861473	7.957845	15915.7	0.001283		0.997905	CYL	30	110
Metric	8.11	0.026	Iron	CA-040710-CYPRESSB-BOXA-9-10	EMERYVILLE	CA	BUNKER B	9	10	2.490143776	1.396687	1.093456984	34.06588	34065.9	-0.625885	-626	0.994124	CYL	30	110
Metric	8.11	0.026	Iron	CA-040710-CYPRESSB-BOXA-9-10-2E	EMERYVILLE	CA	BUNKER B	9	10	3.51916647	2.109459	1.409707785	26.42361	52847.2	0.000648		0.998508	CYL	30	110
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXA-25-26	EMERYVILLE	CA	BUNKER B	25	26	4.129434586	2.651033	1.478401661	79.20087	79200.9	-0.616646	-617	0.999595	DMP	30	110
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXA-25-26-2E	EMERYVILLE	CA	BUNKER B	25	26	6.688337326	4.626015	2.062322617	56.77613	113552.3	0.000879		0.998962	DMP	30	110
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXA-26-27	EMERYVILLE	CA	BUNKER B	26	27	2.123045206	1.716728	0.406316876	288.1758	288175.8	-0.617013	-617	0.999911	DMP	30	110
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXA-26-27-2E	EMERYVILLE	CA	BUNKER B	26	27	3.970390081	3.245232	0.725157738	161.4693	322938.6	0.001213		0.998262	DMP	30	110
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXA-27-28-2E	EMERYVILLE	CA	BUNKER B	27	28	25.10587883	10.55179	14.55408669	8.045211	16090.4	-0.000445		0.998417	DMP	30	110
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXA-27-28	EMERYVILLE	CA	BUNKER B	27	28	6.157865524	3.067212	3.090653658	37.88542	37885.4	-0.618519	-619	0.999186	DMP	30	110
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXA-28-27	EMERYVILLE	CA	BUNKER B	28	27	19.16397285	7.217378	11.94659519	9.801178	9801.2	-0.61858	-619	0.997237	DMP	30	110
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXA-28-27-2E	EMERYVILLE	CA	BUNKER B	28	27	24.89691734	10.55978	14.33714104	8.166949	16333.9	0.001791		0.998644	DMP	30	110
Metric	8.11	0.026	Iron	CA-040810-CYPRESSB-BOXA-31-32	EMERYVILLE	CA	BUNKER B	31	32	8.949131012	1.402932	7.546198845	4.936203	4936.2	-0.613085	-613	0.995241	CYL	30	110
Metric	8.11	0.026	Iron	CA-040810-CYPRESSB-BOXA-31-32-2E	EMERYVILLE	CA	BUNKER B	31	32	11.06309605	2.578029	8.485067368	4.390015	8780.0	0.001528		0.999982	CYL	30	110
Metric	8.11	0.026	Iron	CA-040810-CYPRESSB-BOXA-32-33-2E	EMERYVILLE	CA	BUNKER B	32	33	6.845869541	2.42991	4.415959358	8.435216	16870.4	0.002687		0.995296	CYL	30	110
Metric	8.11	0.026	Iron	CA-040810-CYPRESSB-BOXA-33-32E	EMERYVILLE	CA	BUNKER B	33	34	5.838365078	2.335976	3.502388716	10.63548	21271.0	0.001769		0.998123	CYL	30	110
Metric	8.11	0.026	Iron	CA-040810-CYPRESSB-BOXA-34-33-2e	EMERYVILLE	CA	BUNKER B	34	33	5.809587002	2.331953	3.477633715	10.71118	21422.4	0.00232		0.997665	CYL	30	110
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-1-2	EMERYVILLE	CA	BUNKER B	1	2	90528.53906	26353.54	64174.99219	0.001825	1.8	-0.246851	-247	0.996396	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-1-2-2E	EMERYVILLE	CA	BUNKER B	1	2	173557.5313	41688.46	131869.0625	0.000888	1.8	-0.075323		0.995788	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-2-3	EMERYVILLE	CA	BUNKER B	2	3	94854.42969	20729.96	74124.47656	0.00158	1.6	-0.134972	-135	0.998163	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-2-3-2E	EMERYVILLE	CA	BUNKER B	2	3	126721	28583.27	98137.72656	0.001193	2.4	0.180907		0.998423	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-3-4	EMERYVILLE	CA	BUNKER B	3	4	29233.78906	9907.562	19326.22656	0.006059	6.1	-0.299858	-300	0.981523	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-3-4-2E	EMERYVILLE	CA	BUNKER B	3	4	67060.79688	16280.4	50780.39453	0.002306	4.6	-0.037958		0.999858	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-4-3	EMERYVILLE	CA	BUNKER B	4	3	30108.11133	6961.796	23146.31445	0.005059	5.1	-0.260073	-260	0.999653	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-4-3-2E	EMERYVILLE	CA	BUNKER B	4	3	66790.84375	17099.31	49691.53906	0.002356	4.7	0.0497		0.999708	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-7-8	EMERYVILLE	CA	BUNKER B	7	8	121918.6016	29714.59	92204.00781	0.00127	1.3	-0.265883	-266	0.99253	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-7-8-2E	EMERYVILLE	CA	BUNKER B	7	8	164769.2031	39228.68	125540.5234	0.000933	1.9	0.188792		0.997043	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-8-9	EMERYVILLE	CA	BUNKER B	8	9	36910.95703	24401.68	12509.2793	0.00936	9.4	-0.433974	-434	0.992793	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-8-9-2E	EMERYVILLE	CA	BUNKER B	8	9	277653.7813	143610.4	134043.375	0.000874	1.7	-0.266718		0.998415	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-9-10	EMERYVILLE	CA	BUNKER B	9	10	201530.7188	36936.43	164594.2969	0.000711	0.7	-0.209102	-209	0.991432	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-9-10-2E	EMERYVILLE	CA	BUNKER B	9	10	452700.6563	50427.05	402273.5938	0.000291	0.6	-0.012381		0.980358	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-10-9	EMERYVILLE	CA	BUNKER B	10	9	108503.4844	44403.41	64100.07031	0.001827	1.8	-0.182896	-183	0.997689	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-10-9-2E	EMERYVILLE	CA	BUNKER B	10	9	317321.25	91172.73	226148.5313	0.000518	1.0	-0.019319		0.998254	DMP	4	3500
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-25-26	EMERYVILLE	CA	BUNKER B	25	26	1019.30719	474.5919	544.71521	0.214958	215.0	-0.723382	-723	0.990599	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-25-26-2E	EMERYVILLE	CA	BUNKER B	25	26	2847.638672	1003.467	1844.171509	0.063492	127.0	-0.021638		0.999419	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-26-27	EMERYVILLE	CA	BUNKER B	26	27	1594.506592	555.7595	1038.74707	0.112723	112.7	-0.701058	-701	0.993934	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040710-CYPRESSB-BOXE-26-27-2E	EMERYVILLE	CA	BUNKER B	26	27	3178.970703	1004.144	2174.826172	0.053839	107.7	-0.053194		0.999707	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-28-27	EMERYVILLE	CA	BUNKER B	28	27	1790.258789	590.6215	1199.637329	0.097605	97.6	-0.680525	-681	0.997645	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-28-27-2E	EMERYVILLE	CA	BUNKER B	28	27	3545.326416	1150.788	2394.538086	0.048899	97.8	-0.041471		0.999829	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-31-32	EMERYVILLE	CA	BUNKER B	31	32	9337.833984	4681.782	4656.051758	0.025148	25.1	-0.41916	-419	0.994951	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-31-32-2E	EMERYVILLE	CA	BUNKER B	31	32	10936.08984	5200.447	5735.643066	0.020415	40.8	0.187358		0.999856	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-32-33	EMERYVILLE	CA	BUNKER B	32	33	2023.928345	768.7892	1255.13916	0.093289	93.3	-0.603297	-603	0.988696	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-32-33-2E	EMERYVILLE	CA	BUNKER B	32	33	4758.276855	1707.725	3050.551758	0.038383	76.8	-0.051583		0.999289	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-33-34	EMERYVILLE	CA	BUNKER B	33	34	3629.568115	957.8291	2671.739014	0.043826	43.8	-0.545747	-546	0.995777	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-33-34-2E	EMERYVILLE	CA	BUNKER B	33	34	4224.533691	1194.89	3029.643555	0.038648	77.3	0.009621		0.999496	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-34-33	EMERYVILLE	CA	BUNKER B	34	33	705.0900879	218.951	486.1390686	0.240858	240.9	-0.557896	-558	0.999464	DMP	8	1900
Metric	2.58	0.026	Iron	CA-040810-CYPRESSB-BOXE-34-33-2E	EMERYVILLE	CA	BUNKER B	34	33	4073.100342	1214.342	2858.758789	0.040959	81.9	-0.009431		0.999525	DMP	8	1900

State of California
Department of Transportation

Structural Materials Testing Laboratory
5900 Folsom Boulevard, Sacramento, CA 95819

TEST REPORT

ACCREDITED
CERTIFICATE NO. 2364.01

Remarks Test Results: Sample #1 Peak Load 32944 lbf (99229 psi) Reduction of Area = 39% Sample #2 Peak Load 32628 lbf (100178 psi) Reduction of Area = 47%

Sample No: SM-10-0739
Date Sampled: 06/24/10 **Date Rec'd:** 07/29/10 **Date Reported:** 08/19/10
Lot No: N/A **TL-101 / SIC No:** C721877
Contract/Permit No: Unknown
Material: Strap #18
Manufacturer: Unknown
Sampler: Joe Shanabrook

Results: **Report ONLY!**

SOURCE	DISTRICT	E.A.	SUB JOB	SPECIAL DESIGNATION	OBJECT
59318	05				1270

TL-101

Reviewed by: Glen Weldon
Lab Manager

Approved by:
Quality Manager

Print

Site	Sample Type	Location	Sample Box	Minimum Resistivity (ohm-cm)	As-Received Resistivity (ohm-cm)	pH	Chloride (ppm)	Sulfate (ppm)	As-Received Moisture (CT 226)	Grading Analysis CT 202, 203 (% Passing)						Remaining zinc (oz/ft^2)
										25 mm	19 mm	12.5 mm	9.5 mm	4.75 mm	2.36	
04-SCL-101-20/3.2	Soil	Coupon 1	Miller Box	5967	5996	6.92	9	11	12.3%				100	99		N/A
	Soil	Coupon 1	CT Soil Box	4534	5841	7.62	5	13	14.0%				100	99		N/A
	Soil	Coupon 2	CT Soil Box	2785	3196	7.99	9	28	17.1%			100	99	99		N/A
	Soil	Coupon 3	Miller Box	3413	3936	8.13	16	27	13.9%			100	99	97		N/A
	Soil	Coupon 3	CT Soil Box	3924	4948	8.19	15	119	11.6%	100	98	98	97	96		N/A
	Soil	Coupon 4	CT Soil Box	3299	3472	8.14	5	23	17.3%		100	99	98	96		N/A
05-SLO-101	Soil		CT Soil Box	1518	N/A	6.79	6	818	1.7				100	80	64	N/A
05-SLO-101	Strap Zinc Content	Random showing minimal loss	Sample1	-----		-----	-----	-----	N/A	N/A	N/A	N/A	N/A	N/A		6.28
			Sample2	-----		-----	-----	-----	N/A	N/A	N/A	N/A	N/A	N/A		6.43
			Sample3	-----		-----	-----	-----	N/A	N/A	N/A	N/A	N/A	N/A		6.42