INVESTIGATION OF LOAD TRANSFER MODELS FOR RECYCLED PLASTIC REINFORCEMENT FOR SLOPE STABILIZATION

A Thesis presented to the Faculty of the Graduate School
University of Missouri – Columbia

In Partial Fulfillment
Of the Requirements for the Degree

Master of Science
Civil & Environmental Engineering

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May 2011

The undersigned, appointed by the Dean of the Graduate School, have examined the thesis entitled

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ACKNOWLEDGEMENTS

I would like to take this opportunity to thank my academic advisor, Dr. Erik Loehr, for his academic guidance for my duration at the University of Missouri – Columbia.

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ABSTRACT

Slope failures are not only hazardous to the public, but they are also costly to maintain and repair. A field testing program involving five test sites has been executed in an effort to develop better design practices for slopes reinforced with slender reinforcement. This thesis is directed at three of these sites, including a slope located along Interstate Highway 70 near Emma, Missouri, a slope located along US Highway 36 near Stewartsville, Missouri, and a slope located along Interstate Highway 435 (at Wornall Road) in Kansas City, Missouri.

This thesis describes analyses performed to evaluate current analysis models and to develop recommendations for future design of slopes stabilized with slender reinforcement. The analysis models were evaluated by comparing measured bending moments from the field test sites with predicted bending moments calculated using conventional soil-structure interaction models implemented in the commercial software, LPile, Version 5.0®. The models and input parameters for the soil-structure interaction analysis were varied to produce matches between the measured and predicted response of the reinforcement. The models that produced the best results for each site were then collectively assessed to develop recommendations for use in slope designs with slender reinforcement.

Results of the analyses described suggest that the "API Sand (O'Neill)" model should be used when modeling reinforcement for long-term, drained loading conditions, regardless of the type of soil present. This model should be used with a p-multiplier selected based on the relative pile batter angle. The soil movement profile should be input as anticipated soil movements down to the sliding depth, and then zero below the sliding depth.

CHAPTER 1 – INTRODUCTION

Slope failures and landslides constitute significant hazards to all types of both public and private infrastructure. Total direct costs for maintenance and repair of landslides involving major U.S. highways alone (roughly 20 percent of all U.S. highways and roads) were estimated in 1996 to exceed $100 million annually (TRB, 1996). In the same study, indirect costs attributed to loss of revenue, use, or access to facilities as a result of landslides were conservatively estimated to equal or exceed direct costs. Costs for maintaining slopes for other highways, roads, levees, and railroads maintained by government and private agencies such as county and city governments, the U.S. Forest Service, the U.S. Army Corps of Engineers, the National Parks Service, and the railroad industry significantly increase the total costs for landslide repairs.

A significant, but largely neglected, toll of landslides is the costs associated with routine maintenance and repair of "nuisance" slope failures. Costs for repair of these small slides were not explicitly included in the above referenced study because of limited record keeping for these types of slides by most state departments of transportation. However, the authors of the TRB study conservatively estimated that costs for repair of nuisance slides equal or exceed costs associated with repair of major landslides. This estimate is supported by the Missouri Department of Transportation's (MoDOT) experience with nuisance slide problems, which are estimated to cost on the order of $1 million per year on average. Many other state departments of transportation have similar problems with similarly high, or even higher annual costs. All available evidence clearly indicates that the cumulative costs for repair of many nuisance slides can become extremely large, despite the fact that costs for repair of individual slides are generally low. In addition, nuisance failures can constitute significant hazards to infrastructure users (e.g. from damage to guard rails, shoulders, or portions of road surface) and, if not properly maintained, often progress into more serious problems requiring more extensive and costly repairs.

A field testing program involving five field test sites has been executed in an effort to develop better design practices for slopes reinforced with slender reinforcement. The five field test sites were monitored for periods of up to five years (Loehr and Bowders, 2007). The research described in this thesis is based on analyses of data from three of the five sites. The three sites include a slope located along Interstate Highway 70 near Emma, Missouri, a slope located along US Highway 36 near Stewartsville, Missouri, and a slope located along Interstate Highway 435 at Wornall Road in Kansas City, Missouri. Each of the three sites studied in this report were monitored with various instrumentation (Loehr & Bowders, 2007; Chandler, 2005). The instrumentation allowed the loading induced in the reinforcement from slope movements to be measured over a period of several years.

The primary objective of this research is to improve current design methods for slopes reinforced with slender structural members based on the field performance data acquired in the field testing program and to evaluate current p-y models for the soil types present at these sites to develop recommendations for applications of the p-y method. These objectives were addressed by comparing the bending moments measured in the field with predicted bending moments calculated using several conventional soil-structure interaction models implemented in the commercial software, LPile® Version 5.0. The models and input parameters for the soil-structure interaction analysis were varied to produce reasonable matches between the measured and predicted response of the reinforcement. The models that produced the best comparisons for each site were then collectively assessed to develop recommendations for use in slope designs with slender reinforcement. This approach will lead to designers being able to take less conservative approaches to designing slopes reinforced with slender reinforcing members.

This thesis is organized into five chapters. The locations and characteristics for the respective test site are described in Chapter 2. The LPile® computations are described in Chapter 3, along with required LPile® inputs and iterations. A step by step example of how the LPile® analyses were completed is also provided in Chapter 3.

Chapter 4 describes the specific parameters that were input into LPile® for each site. Chapter 4 also describes comparisons between the predicted response of the reinforcement from the LPile® analyses and the measured response derived from field instrumentation measurements. Results from all three sites are compared to show general trends. Finally, Chapter 5 presents a summary of this thesis along with conclusions reached based on the results of this work, and recommendations for future work.

CHAPTER 2 – BACKGROUND

The three field test sites considered in this work are described in this chapter. These sites include the "I70-Emma site", the "US36-Stewartsville site", and the "I435-Wornall Road site". For each of the three sites, the location of the sites, site and slope characteristics, and the stabilization and instrumentation schemes are described.

2.1 I70-Emma Site

The I70-Emma site is located on Interstate 70, approximately 65 miles west of Columbia, Missouri and approximately 1 mile north of Emma, Missouri at the intersection with Route VV. The embankment slope forms the entrance ramp for eastbound Interstate 70 traffic and has experienced recurring slides in four areas, designated areas S1, S2, S3 and S4, in the past. Figure 2.1 shows a plan view of the site indicating locations of the four slide areas. The embankment is approximately 22 feet tall with slopes varying from 2.2:1 to 2.5:1 (H:V) (Loehr and Bowders, 2003).

The embankment soil is composed of mixed lean and fat clays with scattered gravel, cobbles, and construction rubble (Loehr and Bowders, 2003). Loehr and Bowders divided the slope into two clay layers; an upper clay and a lower clay. The upper clay layer was found to have an effective stress cohesion intercept of 100 psf and an effective stress friction angle of 23 degrees. The lower clay layer nominally had an effective stress cohesion intercept of 350 psf and an effective stress friction angle of 24 degrees.

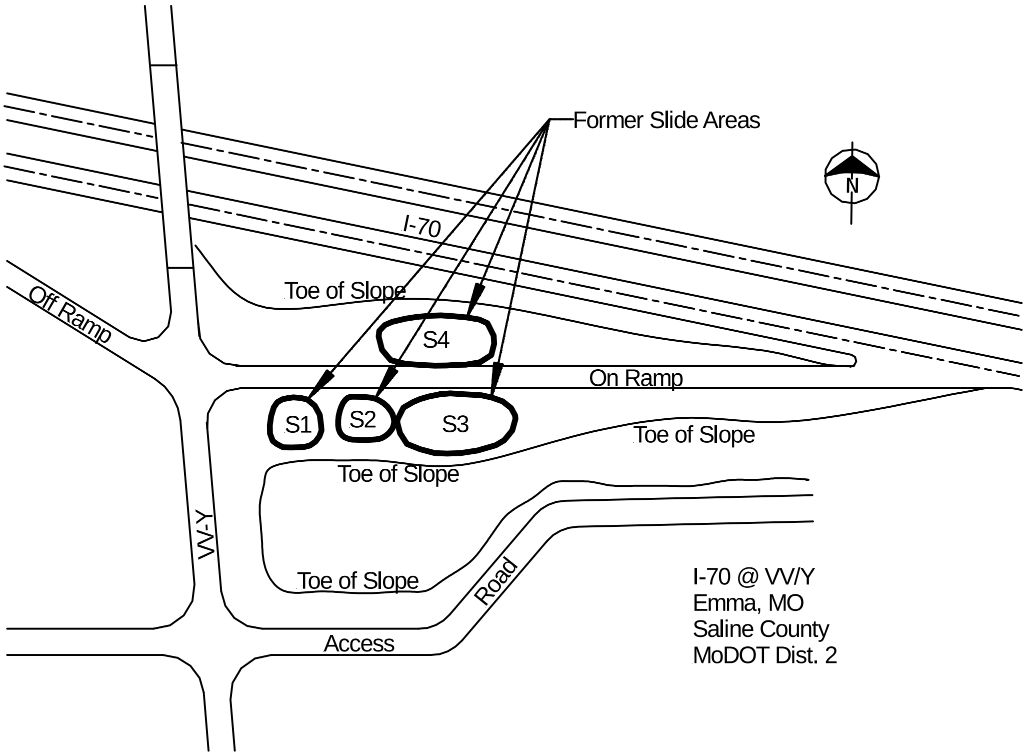


Figure 2.1: Plan view of I70-Emma site showing slide areas.

Slide areas S1 and S2 were stabilized using recycled plastic reinforcing members during the months of October and November, 1999. Details regarding the specific stabilization for slide areas S1 and S2 are provided in Loehr and Bowders (2003). This work focuses on slide area S3. Slide area S3 was divided into 4 sections, denoted Sections A through D, each having a different stabilization scheme as shown in Figure 2.2. In Section A, members were placed on a 4.5 ft by 3.0 ft longitudinal by transverse staggered grid. A 4.5 ft by 6.0 ft grid arrangement was used in Section B, a 6.0 ft by 6.0 ft. grid was used in Section C, and a 6.0 ft by 4.5 ft grid was used in Section D. All grids were “staggered” grids, meaning that adjacent rows of reinforcement were offset by one-half of the member spacing as shown in Figure 2.2.

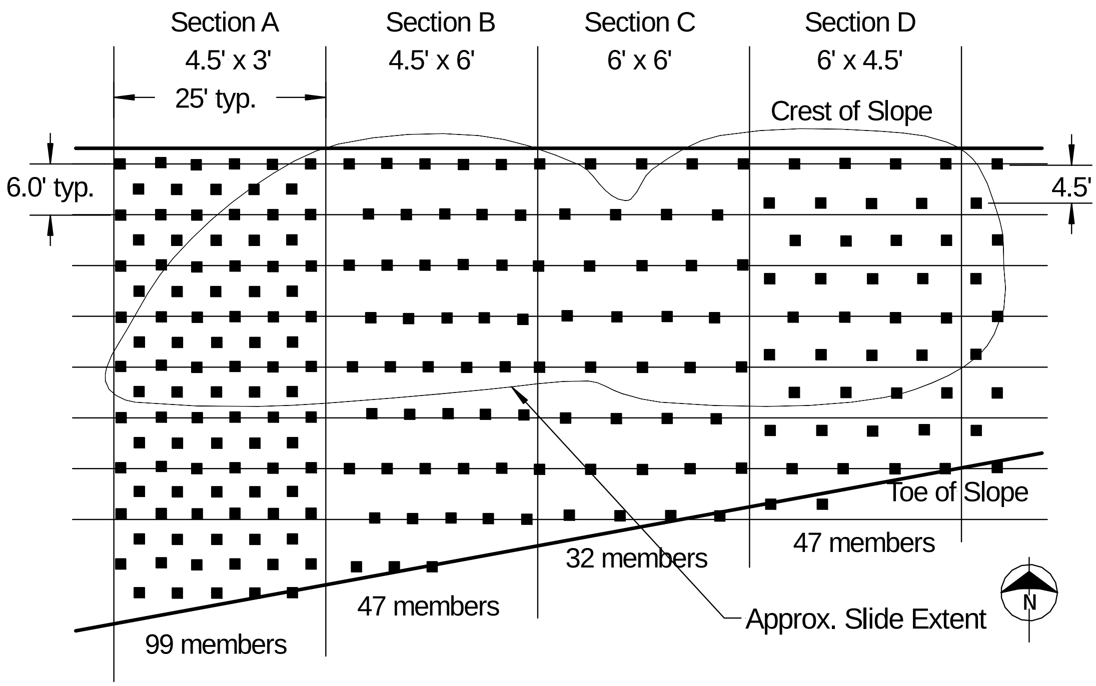


Figure 2.2: Plan view of selected stabilization schemes for slide area S3 at the I70-Emma test site.

A total of 199 recycled plastic reinforcing members were installed in slide area S3 in January, 2003. Members installed near the toe of the slope generally met refusal at depths ranging from 3-6 ft, while members near the top of the slope were driven to the full depth of 8 ft. Typical dimensions of the reinforcing members are 3.5 inches long, 3.5 inches wide, and 8 feet in length. This corresponds to a moment of inertia (I) of 11.75 inches^4. The modulus of elasticity (E) for the members was nominally 145 ksi (Loehr and Bowders, 2003).

Among the reinforcing members installed at slide area S3 were instrumented reinforcing members, slope inclinometers, piezometers, and soil moisture sensors, as shown in Figure 2.3. Six instrumented reinforcing members were installed: one in each of Sections A and D, and two each in Sections B and C. The strain gages attached on the upslope and downslope sides of the members allowed the bending moment developed in the members to be calculated at the depth of the gages. One slope inclinometer was installed in each of the four sections, in close proximity to an instrumented reinforcing member to measure the amount of slope movement. The inclinometers were installed in 6 inch diameter holes that extended 19 ft below grade. Two clusters of three piezometers were installed between Sections B and C to measure the depth of ground water in the slope. Piezometers 1-3 were placed just below the center of the slide area and screened at depths of 14.5 ft, 9.5 ft, and 4.5 ft, respectively. Piezometers 4-6 were located near the top of the slide area and were screened at depths of 14.5 ft, 9.5 ft, and 4.5 ft, respectively. Two soil moisture sensors were installed near the crest and toe of the slope in sections A, C and D (Loehr and Bowders, 2007).

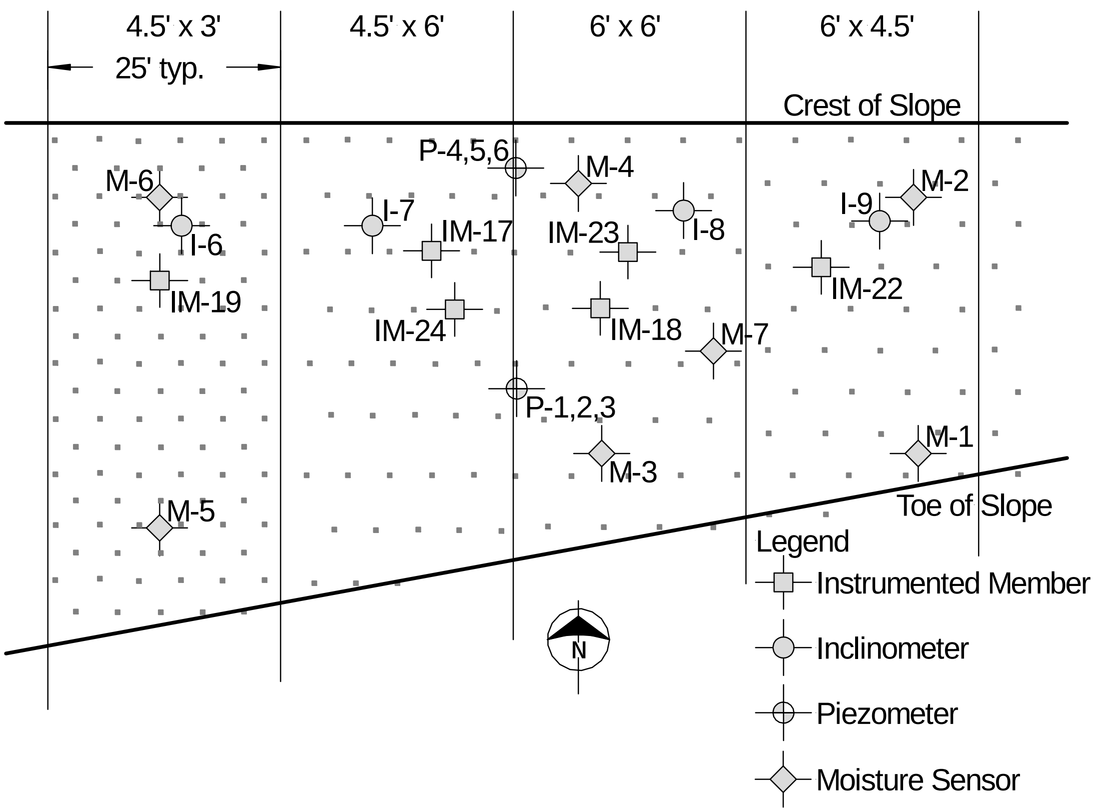


Figure 2.3: Plan view of slide area S3 at the I70-Emma test site showing locations of instrumentation.

2.2 US36-Stewartsville Site

The US36-Stewartsville site is located in northwest Missouri on U.S. Highway 36, approximately two miles west of the city of Stewartsville. The slope lies in the median of US36 between the eastbound and westbound roadway. The slope is approximately 29 ft high and is inclined at 2.2:1 (H:V) (Loehr and Bowders 2003). The slope experienced a failure involving approximately 150 ft of the slope in March of 1997, shown in Figure 2.4. A second, smaller slide occurred approximately 100 feet west of the main slide. This smaller slide was used as a control site for the main slope.

The slope is generally composed of a surficial layer of soft to medium clay overlying stiff to hard fat clay (Loehr and Bowders 2003). The upper clay layer was found to have an effective stress cohesion intercept of 0 psf and an effective stress friction angle of 29 degrees. The lower clay layer nominally had an stress effective cohesion intercept of 100 psf and an effective stress friction angle of 35 degrees.

Figure 2.4: Photograph of slide at the US36-Stewartsville site taken after the slide in March, 1997.

The slide area was divided into 4 sections, denoted Sections A through D, for the field testing program. Different stabilization schemes were used in each section, as shown in Figure 2.5. In Section A, members were placed on a 4.5 ft by 3.0 ft staggered grid. A 6.0 ft by 6.0 ft grid was used in Section B, a 6.0 ft by 4.5 ft grid was used in Section C, and a 4.5 ft by 6.0 ft grid in was used in Section D.

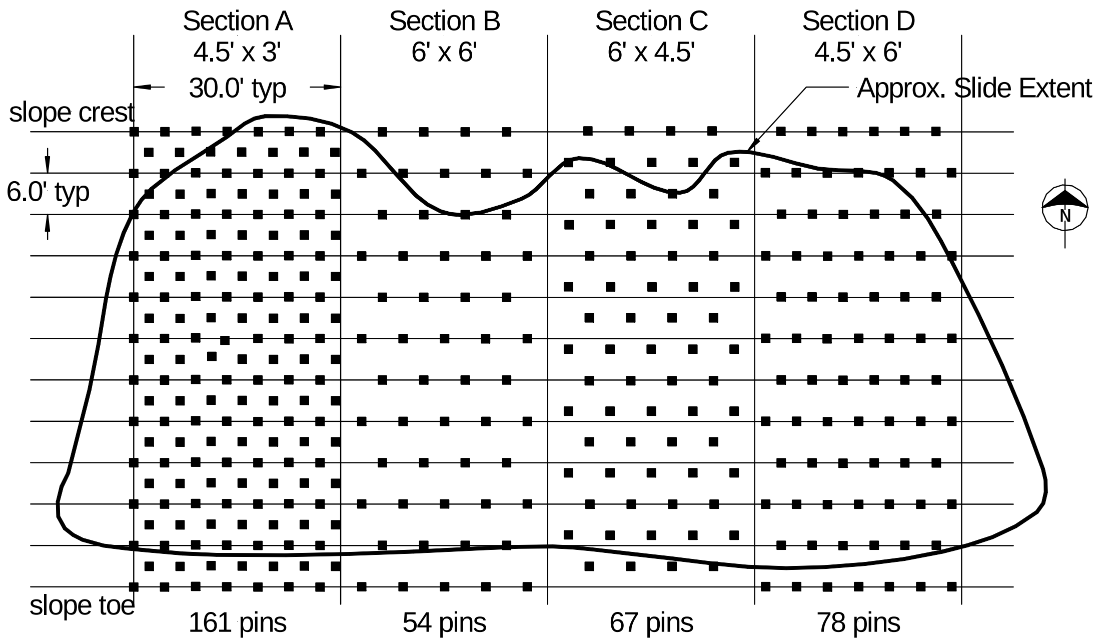


Figure 2.5: Plan view of selected stabilization schemes for the US36-Stewartsville test site.

A total of 360 recycled plastic reinforcing members were installed at the site in the first week of May in 2002. Members installed near the toe of the slope generally met refusal at depths ranging from 4-5 ft, while members installed nearer the crest of the slope were driven to the progressively deeper depths. Only 59 members were installed to the full 8 ft depth. The properties for the recycled plastic reinforcing members installed at this site were identical to those installed at the I70-Emma site (Section 2.1).

Instrumentation installed at the US36-Stewartsville site was similar to that installed at the I70-Emma site, as shown in Figure 2.6. Five instrumented reinforcing members were installed: two in Section A and one in Sections B through D. Two additional instrumented members were installed approximately 10 ft apart near the center of the control slide. Of these two members, one was recycled plastic, and the other was a 3.5” diameter steel pipe. One slope inclinometer casing was installed in each of the four test sections in close proximity to an instrumented reinforcing member. One inclinometer casing was also installed at the control site. The inclinometer casings were installed to a depth of 19 ft below grade. Two clusters of three piezometers were installed between Sections B and C to monitor piezometric levels. Piezometers 1-3 were placed in the upper third of the slide area and screened at depths of 14 ft, 9 ft, and 4 ft, respectively. Piezometers 4-6 were near the lower third of the slide area and were screened at similar depths. Soil moisture sensors were also installed at 7 locations across the main slide area to monitor negative pore water pressures (Loehr and Bowders, 2007).

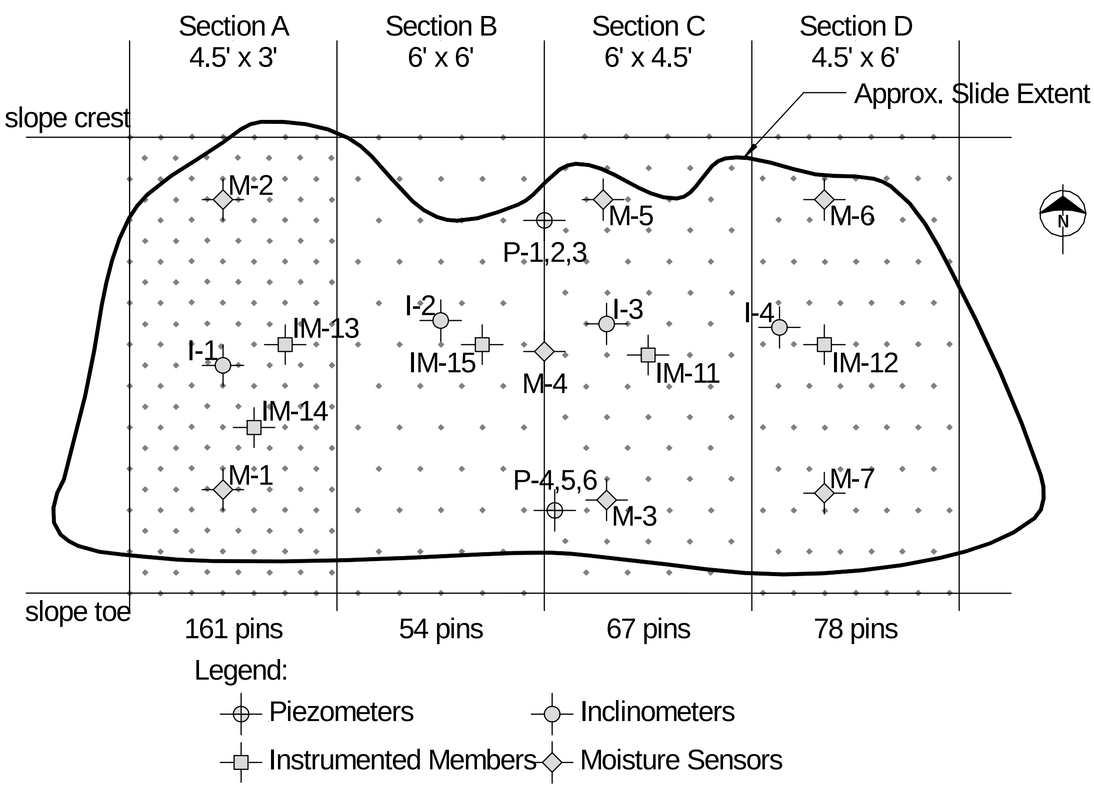


Figure 2.6: Plan view of slide area at the US36-Stewartsville test site showing locations of instrumentation.

2.3 I435-Wornall Road Site

The I435-Wornall Road site is located in the northeast quadrant of the intersection of I435 and Wornall Road between I435 and the westbound exit ramp, in Kansas City, Missouri. The embankment is a bridge approach embankment that serves to support I435 as it passes over Wornall Road. The slope is approximately 32 ft high with 2.2:1 (H:V) side slopes (Loehr and Bowders, 2003). In the past, the embankment had experienced surficial slides along the interface between the upper and lower clay, shown in Figure 2.7.

The slope generally consists of a 3-5 ft thick surficial layer of lean to fat clay with soft to medium consistency overlying stiffer compacted clay shale (Loehr and Bowders, 2003). The upper clay layer was found to have an effective stress cohesion intercept of 0 psf and an effective stress friction angle of 27 degrees. The lower clay layer was found to have an effective stress cohesion intercept of 0 psf and an effective stress friction angle of 29 degrees.

Figure 2.7: Photograph of slide at the I435-Wornall Road site taken after a slide on June 20, 2001.

The I435-Wornall Road slide was stabilized using recycled plastic members installed on a 3.0 ft by 3.0 ft staggered grid over the entire area where the previous slide had occurred, as shown in Figure 2.8. Additional reinforcing members were placed on a 3.0 ft by 6.0 ft grid above the slide area to reduce the potential for future sliding in the upper portion of the slope.

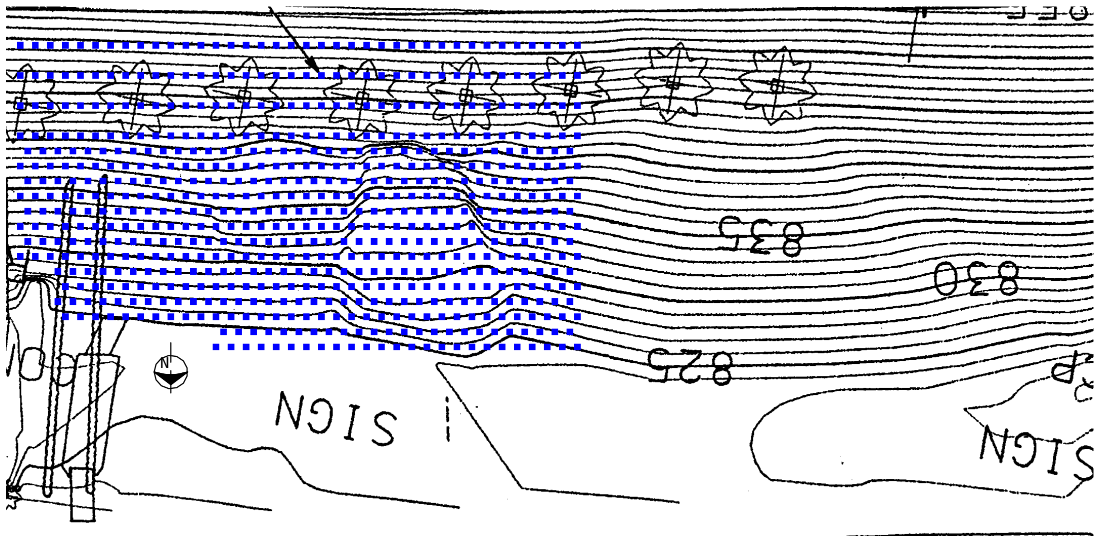


Figure 2.8: Plan view of selected stabilization scheme superimposed on contours for the I435-Wornall Road test site.

A total of 620 recycled plastic reinforcing members were installed between October and December, 2001. The members were either driven to full depth, or until penetrating at least 6 inches into the stiff clay layer. The properties for the recycled plastic reinforcing members installed at this site were similar to those installed at the other test sites.

Instrumentation installed at the I435-Wornall Road site was similar to that installed at the other test sites, as shown in Figure 2.9. Four instrumented reinforcing members were also installed near the center of the slide area. Four slope inclinometer casings were installed. The inclinometer casings were installed to depths ranging from 14.5 ft to 26 ft below grade. Two clusters of two piezometers were installed at the western half of the site. Piezometers 1 and 2 were placed near the top of the slide area and were screened at depths of 11.6 ft and 5.0 ft, respectively. Piezometers 3 and 4 were placed near the lower third of the slide area and were screened at depths of 11.0 ft and 4.0

ft, respectively. Soil moisture sensors were installed at 7 locations across the main slide area, as shown in Figure 2.9.

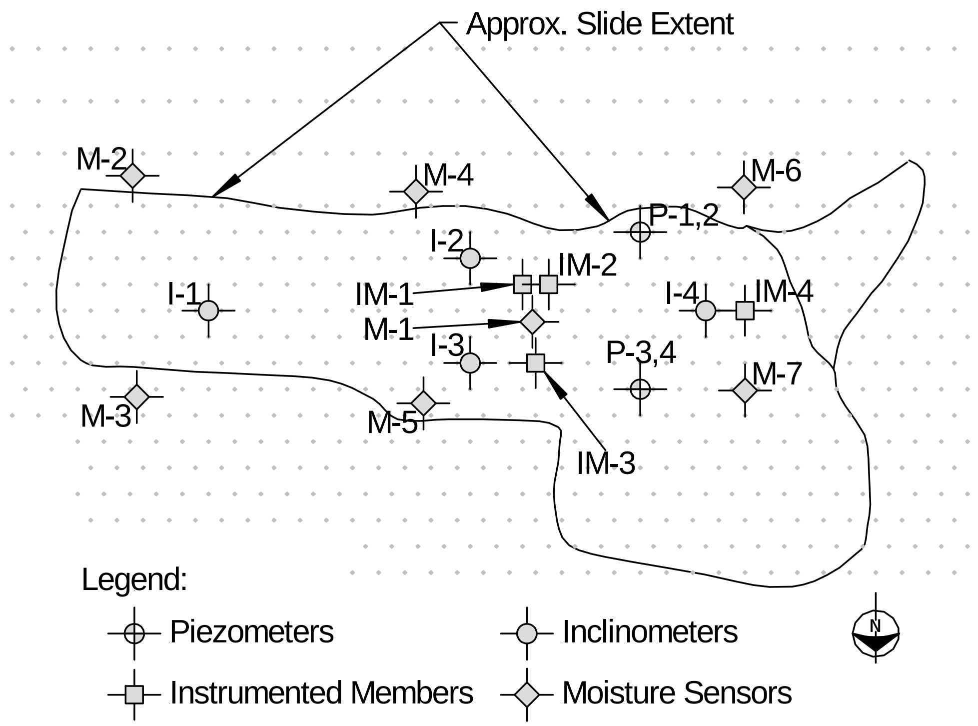


Figure 2.9: Plan view of slide area at the I435-Wornall Road test site showing locations of instrumentation.

2.4 Summary

The general characteristics of the three field test sites considered in this study have been described in this chapter. The characteristics of the slopes for each site, including approximate slope height, slope angle and slope composition were also described. The slope stabilization schemes used at the respective sites were also presented along with a summary of the instrumentation used to monitor performance at each site.

CHAPTER 3 – ANALYTICAL APPROACH

The procedure used to evaluate load transfer in reinforcing members used for slope stabilization is described in this chapter. Also described are the inputs and variables used in the soil-structure interaction analysis software. An example of the procedure used to analyze each reinforcing member is also provided.

3.1 Analysis Procedure

The software used to perform the soil-structure interaction analyses was Ensoft's LPILE 5.0®. LPILE® is software that models the behavior of laterally loaded piles in soil or rock. The analyses include the effects of the stiffness of the pile and the non-linear stiffness of the soil surrounding the pile, as represented by "p-y curves", when subjected to lateral loads. LPile® also has the capability to model piles subject to loading from moving soil, where the applied loads are derived from the magnitude of lateral soil movement and the appropriate p-y curves. It is this capability that is used directly for this research.

The primary question regarding use of LPile®, and other similar software, is whether the p-y curve models appropriately reproduce observed pile response when subjected to loading from moving soil. Current p-y models were generally empirically derived from tests on "actively loaded" piles with a lateral load applied at the pile head. This loading condition is substantially different from that of pile loading from moving soil. The objective of this research is, thus, to utilize the available field measurements of pile response to evaluate current p-y curves for the soil types present at these sites and to develop recommendations for application of the p-y method in such cases.

The procedure used for the analyses involved first establishing the measured soil movement profiles from field instrumentation data obtained over several years. The soil movement profile was then input into LPILE® along with appropriate soil properties (which establish the specific p-y curves) and reinforcing member properties (which establish the member stiffness) to compute the predicted response of the reinforcement due to the measured soil movements. The predicted reinforcement response was then compared to the measured reinforcement response to assess the validity of the models. Comparisons of measured and predicted reinforcement response were generally based on comparisons of measured and predicted bending moments. When the measured and predicted bending moments matched, the soil-structure interaction model was deemed to be appropriate. When comparisons of measured and predicted bending moments were not reasonably close, the model was deemed inappropriate. In such cases, modifications were made to the model and the analyses repeated until good comparisons of measured and predicted response were achieved.

The two primary methods for modifying the soil-reinforcement interaction model were to vary the sliding depth and to vary the p-y curve used to predict reinforcement response. A trial and error procedure was used to find the soil movement profile and p-y curve that produced predicted bending moments resembling the measured moments.

Soil movements were measured with inclinometers every two feet vertically. Therefore, determining the sliding depth inevitably required some judgment. The sliding depth for all three sites analyzed was approximately 4 feet. Adjustments of the sliding depths analyzed were consistent with the measured soil movements as described in Section 3.4. All measured data, including the bending moments, were taken from

Chandler, 2005. The p-y curves were also adjusted, using "p-multipliers", until the predicted moments compared with the measured moments, as described in Section 3.5. “y-multipliers” were set equal to 1 and were not varied during these analyses.

3.2 Reinforcement Properties

The reinforcement properties input into LPILE® were taken from data provided by Chen, 2003. The reinforcement properties that were input include the total embedment length, reinforcement dimensions, moment of inertia, and modulus of elasticity. Typical dimensions of the reinforcing members are 3.5 inches long, 3.5 inches wide, and 8 feet in length. This corresponds to a moment of inertia of 11.75 $inches^4$. The modulus of elasticity used was 145 ksi (Chen, 2003).

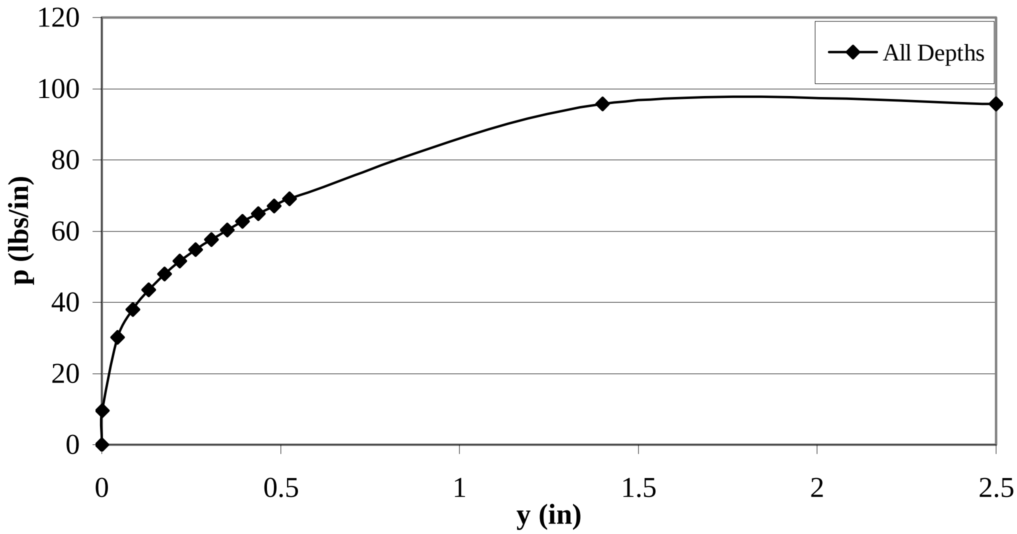
3.3 Soil Properties

Soil properties were input into LPILE® according to data provided by Chandler, 2005. The specific properties input into LPILE® depended on the specific type of p-y curve used. These properties included the thickness, effective unit weight, undrained shear strength, the strain at which one half of the undrained shear strength is mobilized, effective stress friction angle, and p-y modulus for each soil layer. Specific values used for each of these properties are provided with descriptions of the analysis for specific sites in Chapter 4.

The specific form of p-y curves used was established using an iterative process. First, a trial p-y curve was selected, and the problem was analyzed to produce a predicted response (bending moment) for the member being considered. The resulting predicted moment distribution was then compared to the measured moment distribution. If the predicted moment distribution did not compare well with the measured moment distribution, then a different p-y curve was chosen and the problem reanalyzed. This process was repeated until the general shape of the predicted moments compared to the shape of the measured bending moments.

Two different forms of p-y curve models were used in the analyses: the “soft clay (Matlock)” curve (Matlock, 1970) and the “API sand (O’Neill)” curve (O’Neill and Murchison, 1983). These two different types of p-y curves were used to reflect the degree that the soil was permitted to drain (dissipate excess pore water pressures) during loading rather than to reflect the specific soil type(s) present at the respective field test sites (as the model names imply). As mentioned in Section 3.1, p-y curves in current use were empirically derived from load tests performed over a matter of hours (relatively rapid loading). Such loading can be considered as being completely undrained, with no dissipation of excess pore water pressures, when the tests are performed in clays since clays drain slowly. In contrast, such tests can be considered as fully drained (complete dissipation of excess pore water pressures) when performed in sands. It is postulated for this work that the differences in p-y curves for the Soft Clay (Matlock) and API Sand (O'Neill) models are predominantly attributed to differences in drainage conditions rather than to specific properties of sands and clays themselves. This is important for the slope stabilization application because loading is frequently induced at rates that can be considered as fully drained. No p-y models currently exist to reflect drained loading in clays (as is believed to have occurred at the field test sites) so p-y curves for "sand" were used to reflect drained loading.

Examples of p-y curves for the soft clay and API sand curves are shown in Figure 3.1. As shown in the figure, the p-y curves for the soft clay model are independent of depth, a feature that is consistent with undrained loading in saturated soils. The p-y curves for the API sand model, in contrast, vary substantially with depth to reflect the increasing strength and stiffness of the soil with increasing confining stress. Such behavior is also expected to occur for silty or clayey soils under drained loading conditions.



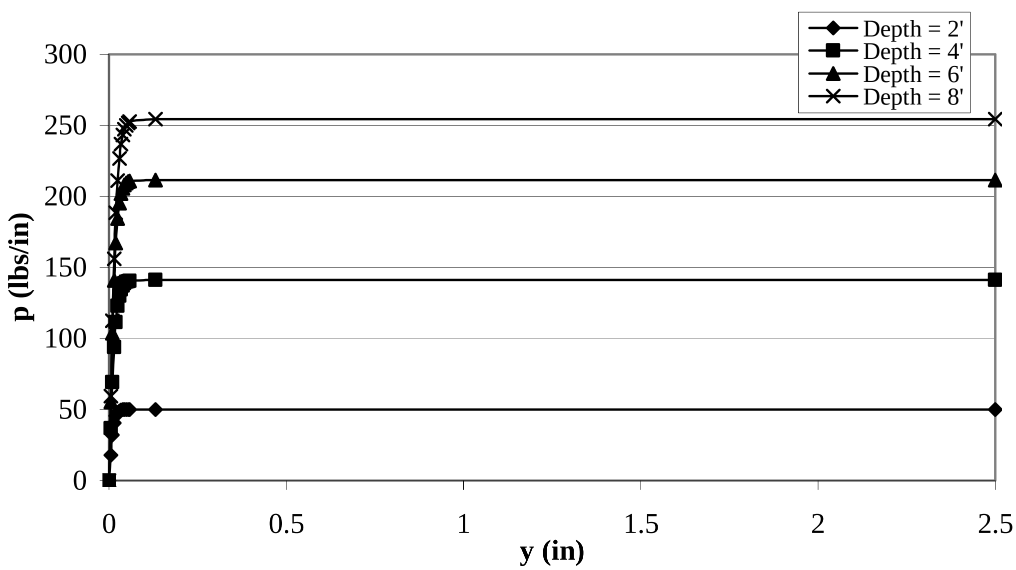


Figure 3.1: Sample p-y curve for soft clay (top) and sample p-y curve for API sand (bottom).

Once the general shape of the predicted moments compared well with the general shape of the measured moments, a p-multiplier was applied to the p-y curves and varied until the magnitude of the predicted moments generally matched the magnitude of the measured moments. P-multipliers are a simple means for scaling of the p-y curves, a feature that was used to simplify the analyses used throughout this work. As shown in Figure 3.2, increasing the p-multiplier increases the resistance of the soil for any given soil deflection.

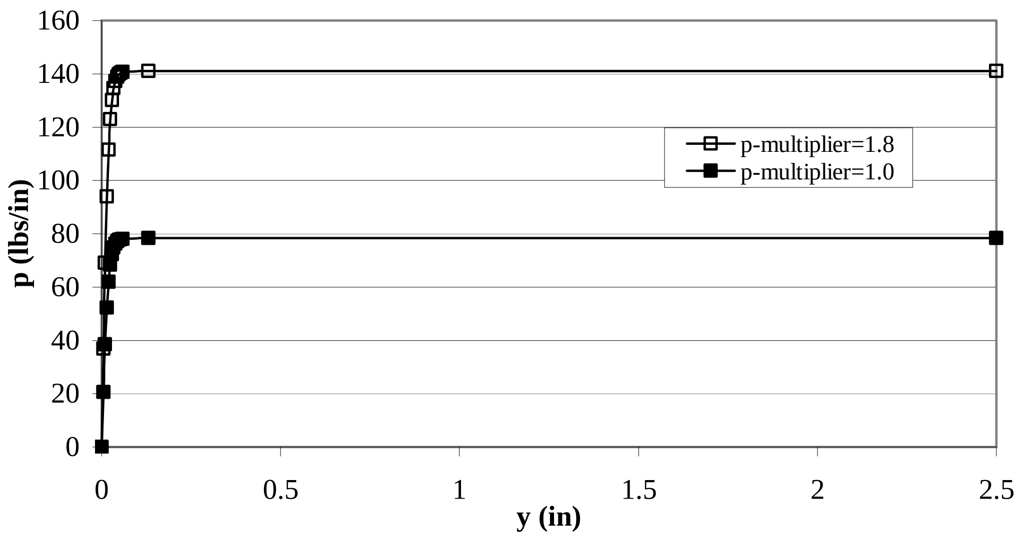


Figure 3.2: p-y curves at the same depth, varying the p-multiplier.

3.4 Soil Movement Profile

Measured soil movement profiles for each instrumented reinforcing member considered were established from inclinometer data provided by Chandler, 2005. Analyses were performed for each member using soil movements derived from the inclinometer located nearest to the member.

Inclinometer measurements provide deformations at two foot intervals along the length of the inclinometer casing. Because of the limited precision provided by such measurements, the soil movement profile was varied in a trial and error process until the general shape of the predicted moments best matched the general shape of the measured moments, within the bounds of deformations indicated by inclinometer measurements. Since the failure plane for all three sites was approximately 4 feet deep, the soil movement variations were selected keeping the sliding depth in consideration. As illustrated in Figure 3.3, soil movement profile trials included:

1) Actual inclinometer values, as measured between 0-4 feet.
2) Actual inclinometer values, as measured between 0-8 feet.
3) Average soil movements between 0- 4 feet.
4) Average soil movements between 0- 8 feet.
5) Average movements between 0-4 feet and average movements between 4-8 feet.

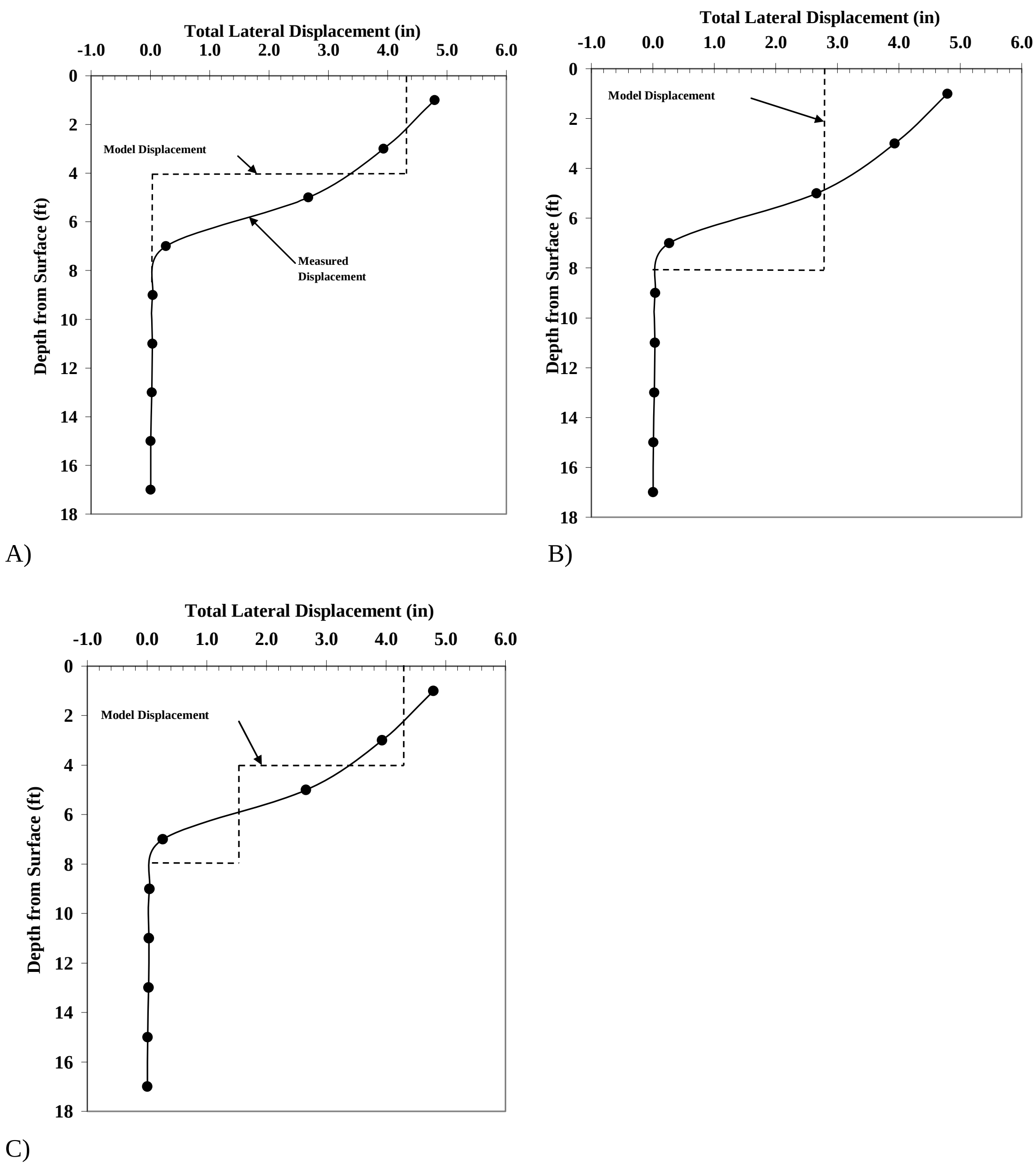


Figure 3.3: Alternative soil movement profile assumptions for measured displacements from inclinometer I-7 located at the site in Emma, Missouri taken on January 27, 2005: A) Data averaged from 0-4 feet. B) Data averaged from 0-8 feet. C) Data averaged from 0-4 feet and 4-8 feet.

Analyses were performed for each of these options. The soil movement profile that produced predicted moments that most closely agreed with the measured moments was chosen for subsequent analyses of that site. In general, the soil movement profile was varied to produce a distribution of bending moments along the member that was similar in shape to the measured bending moments. These distributions were not always similar in magnitude. However, the magnitudes were generally matched by varying the p-y curve as described in Section 3.3 and illustrated in Section 3.5.

3.5 Example of Analysis

To demonstrate the procedure used to back-calculate an appropriate p-y curve for each instrumented member, analyses performed for instrumented reinforcing member IM 24, from the Emma, Missouri site are presented here. IM 24 was a plastic member with dimensions 3.5 inches long, 3.5 inches wide, and 8 feet in length. This corresponds to a cross section area of 12.25 square inches and a moment of inertia of 11.75 inches^4. The modulus of elasticity used was 145 ksi (Chen, 2003). These characteristics are generally representative of all modeled plastic members used in this research unless noted otherwise.

Six different trial cases were performed to establish the most appropriate model for the measured data. For all of these analyses, pile head boundary conditions were unrestrained with zero shear and zero moment. The soil profile was divided into three layers that correspond to the depths of piezometers installed at the site. The effective unit weights for each layer in the respective models were calculated using measured pore water pressures and a total unit weight of 120 pcf.

Two different p-y curves were used in the analyses: the “soft clay (Matlock)” curve and the “API sand (O’Neill, 1983)” curve. The profile of soil movement also varied among the different trials.

3.5.1 Trial 1

The first model considered utilized the soft clay p-y curve and the soil movement profile was established by using the actual measured movements over a depth of 0-8 feet. All movement was nominally zero deeper than 8 feet, as illustrated in Figure 3.4.

The soil profile for the model was divided into three layers. Layer 1 was 0-7.5 feet, layer 2 was 7.5-12.5 feet, and layer 3 was 12.5 feet and deeper. The properties input for these three layers are summarized in Table 3.1. Undrained shear strength values were derived from torvane measurements provided in Loehr and Bowders (2003). The E_{50} values used were the default values provided in LPILE® based on undrained shear strength. The p- and y-multipliers were both set to 1 for the first trial.

Table 3.1: Properties used for the soft clay model in LPILE® for the Emma, Missouri site.

Layer	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Strain Factor (E_{50})
1	73.2	800	0.01
2	71.5	1498	0.007
3	102.2	800	0.01

Figure 3.4 shows a comparison of measured bending moments from the January 27, 2005 readings with predicted moments from the Trial 1 model. The trend in predicted moments do not compare well with the measured moments, which suggests that this model is not likely to be appropriate.

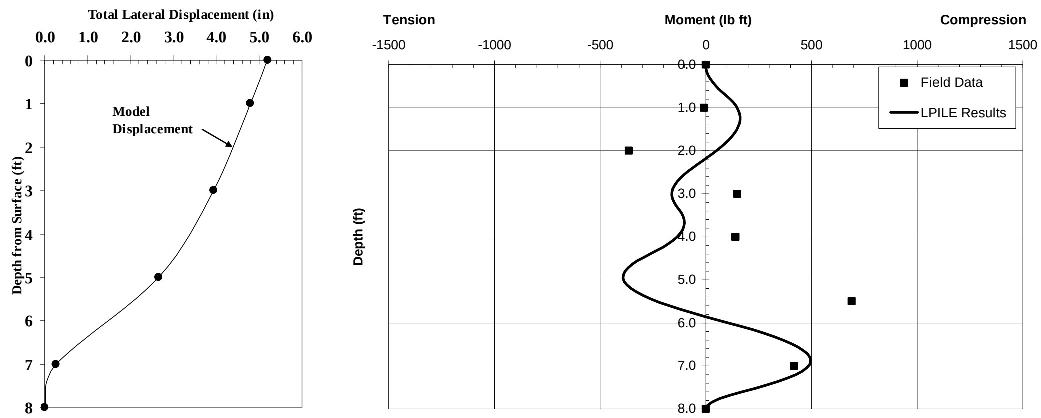


Figure 3.4: Comparison of predicted and measured bending moments for Trial 1 analyses for IM24 using measured soil movements for 0-8 feet. Measured moments compared to LPILE® predicted moments for the IM 24 data taken on January 27, 2005.

3.5.2 Trial 2

For Trial 2, the soil movement profile corresponding to the actual measured soil movement between 0-4 feet and shown in Figure 3.5 was employed along with the same p-y curves used for Trial 1. The p- and y-multipliers were set equal to 1. Figure 3.5 shows a comparison of measured and predicted moments for IM 24 based on the January 27, 2005 readings. As shown in the figure, measured and predicted moments from the Trial 2 analyses compare more favorably than those from Trial 1. Measured moments do not compare well with predictions at a depth of 1 foot, where the predicted moment is positive, while the measured moment is negative. Similarly, predicted moments at a depth of 3 feet are negative, while the measured moment is positive.

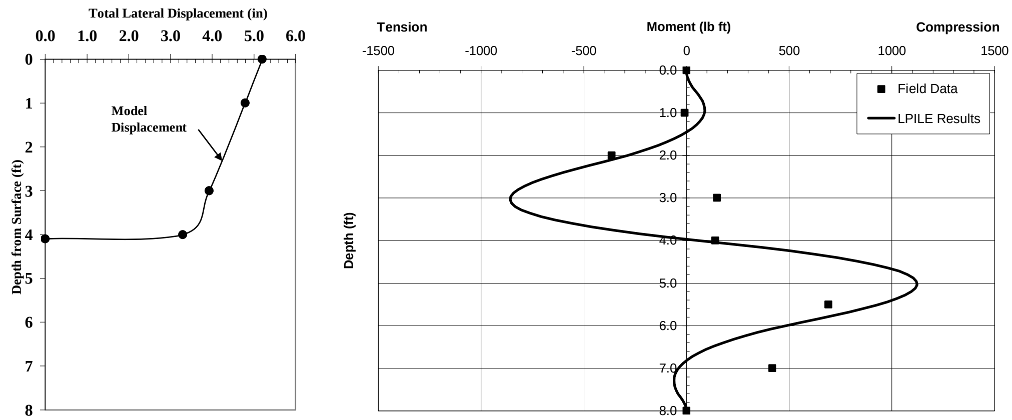


Figure 3.5: Comparison of predicted and measured bending moments for Trial 2 analyses for IM24 using measured soil movements for 0-4 feet. Measured moments compared to LPILE® predicted moments for the IM 24 data taken on January 27, 2005.

3.5.3 Trial 3

In Trial 3, the soil movement profile was taken as the average soil movement between 0-4 feet as shown in Figure 3.6. The soft clay p-y curve was also used for Trial 3, with the p- and y-multipliers set equal to 1. Figure 3.6 shows a comparison of measured and predicted moments for Trial 3 for IM 24 based on the January 27, 2005 readings. The comparison of moments for Trial 3 analyses is also good. Predicted moments from Trial 3 tend to more closely match with measured moments near the top of the member than was observed for Trial 2. However, the predicted moment at a depth of 3 feet still negative while the measured moment is positive.

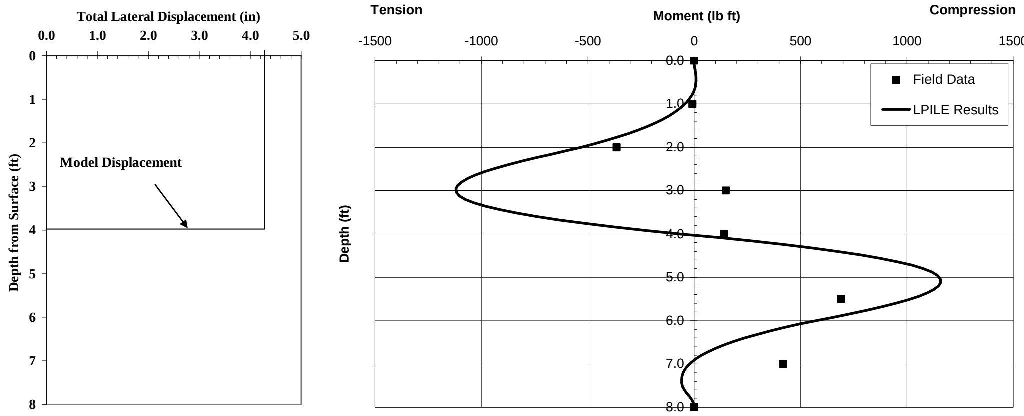


Figure 3.6: Comparison of predicted and measured bending moments for Trial 3 analyses for IM24 using averaged soil movements between 0-4 feet. Measured moments compared to LPILE® predicted moments for the IM 24 data taken on January 27, 2005.

3.5.4 Trial 4

For Trial 4, the soil movement profile was taken as the average soil movement between 0-8 feet as shown in Figure 3.7. The soft clay p-y curve was also used for Trial 4, with the p- and y-multipliers set equal to 1. Figure 3.7 shows a comparison of measured and predicted moments from Trial 4 analyses for IM 24 based on January 27, 2005 readings. The comparison of the predicted moments and the measured moments for Trial 4 analyses are similar to those obtained Trial 3 analyses.

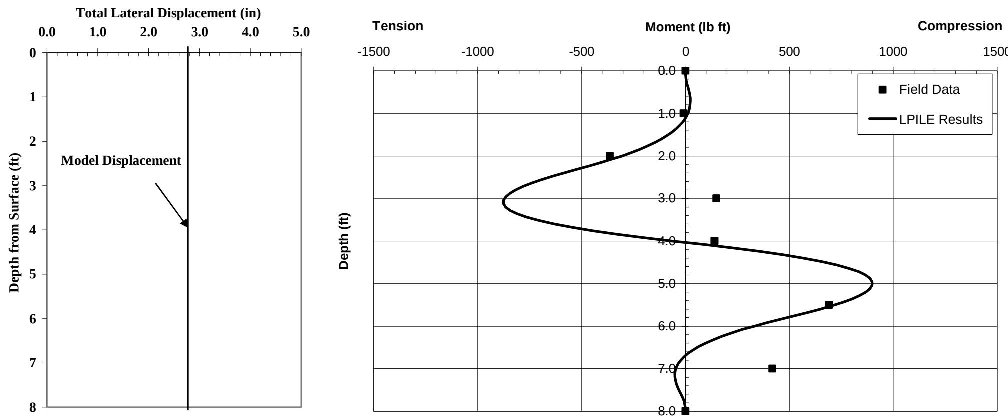


Figure 3.7: Comparison of predicted and measured bending moments for Trial 4 analyses for IM24 using averaged soil movements between 0-8 feet. Measured moments compared to LPILE® predicted moments for the IM 24 data taken on January 27, 2005.

3.5.5 Trial 5

Analyses for trial 5 used soil movements determined by averaging measurements from 0-4 feet and from 4-8 feet as shown in Figure 3.8. Trial 5 also used the soft clay p-y curve with the p- and y-multipliers set equal to 1. The predicted moments for Trial 5 do not compare with the measured moments as well as the other trials, particularly along deeper segments of the member.

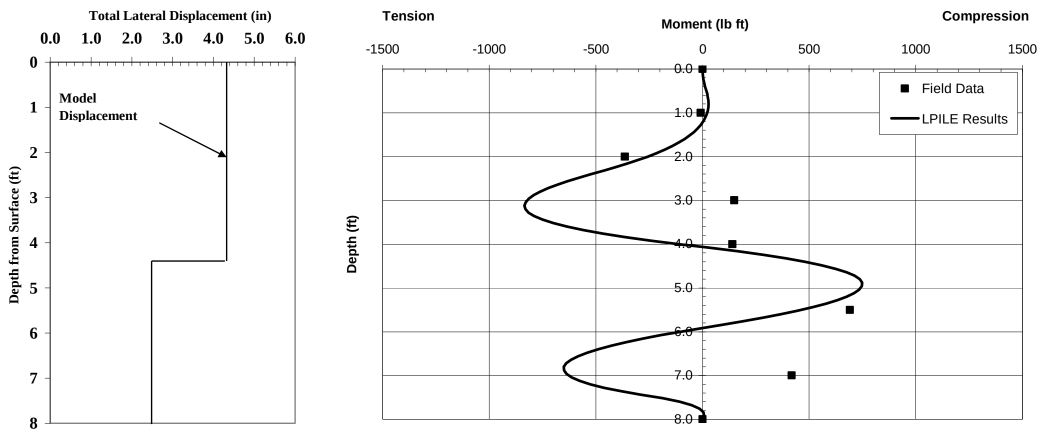


Figure 3.8: Comparison of predicted and measured bending moments for Trial 5 analyses for IM24 using averaged soil movements between 0-4 feet and the soil movements averaged between 4-8 feet. Measured moments compared to LPILE® predicted moments for the IM 24 data taken on January 27, 2005.

3.5.6 Trial 6

The "API Sand (O'Neill, 1983)" curve was used for Trial 6 with the p- and y-multipliers set equal to 1. The soil movement profile was established by using the actual measured soil movements over a depth of 0-4 feet, as illustrated in Figure 3.9. The input parameters for the API sand curve include the effective unit weight, friction angle, and p-y modulus, *k*. Similar to what was done for the soft clay model, the soil profile was divided up into three layers. The effective unit weight remained the same as calculated for the soft clay model and summarized in Table 3.1. The friction angles used in the

LPILE® model were 23 degrees for layer 1 and 24 degrees for layers 2 and 3. The p-y modulus was estimated from the LPILE® "medium sand" default value to be 90 lb/in^3 for all layers. The p-y modulus was varied from 90 lb/in^3 to 225 lb/in^3, but had little effect on the outcome of the moments predicted. The properties input for the API sand model are shown in Table 3.2.

Table 3.2: Properties used for the API Sand (O'Neill, 1983) model in LPILE® for the Emma, Missouri site.

Layer	Effective Unit Weight (pcf)	Effective Friction Angle	p-y Modulus, k (lbs/in^3)
1	73.2	23	90
2	71.5	24	90
3	102.2	24	90

Figure 3.9 shows results of analyses for Trial 6 for IM 24 based on the January 27, 2005 readings. This trial produced the best comparison of measured and predicted bending moments among the six different trial analyses performed. The results are similar to trial 2 with the soft clay model; however, for this trial measured moments compare better with predicted moments better at a depth of 1 foot.

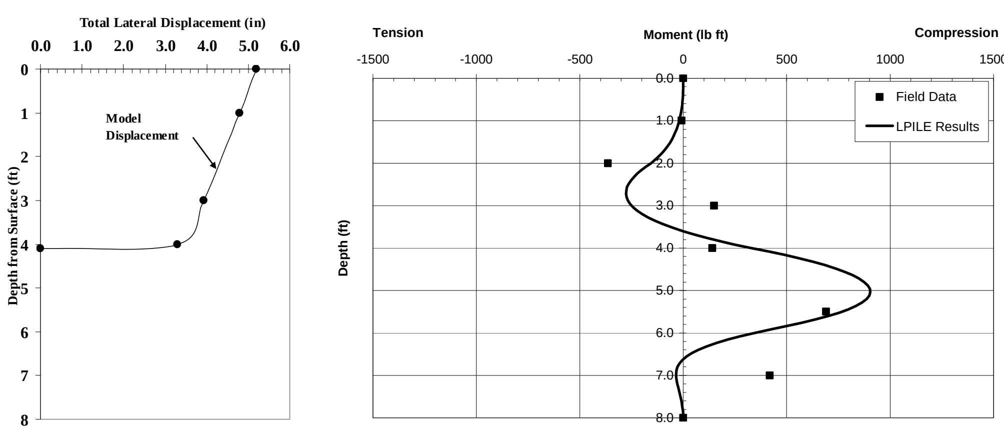


Figure 3.9: Comparison of predicted and measured bending moments for Trial 6 analyses for IM24 using measured soil movements for 0-4 feet. Measured moments compared to LPILE® predicted moments for the IM 24 data taken on January 27, 2005.

3.5.7 p- and y-multipliers

The first five trials were analyzed using the soft clay p-y model with p- and y-modifiers set equal to 1. However, other values of p-multipliers are needed in other instances to produce good matches of the predicted and measured moments. For example, if the magnitude of the predicted moments was less than the magnitude of the measured moments, then the p-multiplier would be increased until the predicted moment magnitudes matched the measured moment magnitudes. Likewise, if the predicted moment magnitudes were more than the measured moment magnitudes, then the p-multiplier would be decreased.

3.5.8 Summary of Trials

The soil movement profiles were varied as described in Section 3.4 to compare the predicted moments with the measured moments. Trials 1-5 utilized the soft clay p-y curve. The predicted moments in Trials 1 and 5 do not match the measured moments. However, the predicted moments compared well with the measured moments in Trials 2-4. Trial 6 used the API sand p-y curve and provided the best comparisons with the measured moments at all depths.

3.6 Summary

In this chapter, the general approach taken to evaluate the p-y method for prediction of pile response due to moving soil was described. The input parameters used for the respective field test sites were provided, including the reinforcement properties, the stratigraphy for each site, the soil properties for each site, and the soil movement profiles for each reinforcing member. Finally, an example of the trial and error procedure used to establish the most appropriate model was presented.

CHAPTER 4 – MODELING RESULTS

Results and discussion of modeling conducted as part of this work are presented in this chapter for reinforcing members from the I70-Emma site, the US 36-Stewartsville Site, and the I435-Wornall Road Site. Moments predicted from analyses using different p-multipliers are compared to measured moments calculated from field instrumentation measurements for each of the aforementioned sites. Results of collective interpretation of results from these analyses are then presented to provide guidance on selection of input parameters (p-y curve) for prediction of loading on similar slope reinforcement in the future.

4.1 I70-Emma Results

Analyses were performed for measured soil movements from several different measurement dates to establish the p-y curve that produced predicted moments that best compared to the measured moments. These p-y curves were collectively compared to establish the best p-y curve for prediction across the range of movements observed.

Analyses were performed for several instrumented members at the I70-Emma, site including: IM 24, IM 19, IM 18 and IM 23. Typical dimensions of the reinforcing members are 3.5 inches long, 3.5 inches wide, and 8 feet in length. This corresponds to a moment of inertia of 11.75 inches^4. The modulus of elasticity used was 145 ksi (Chen, 2003). Pile head boundary conditions were unrestrained with zero shear and zero moment for all analyses. The p-y curve curve used for all analyses performed for the Emma site was the “API Sand (O’Neill, 1983)” curve.

As described in Section 3.1, the sliding depth and p-multipliers were varied until the magnitude of predicted and measured bending moments were similar. The first set of analyses used p- and y- multipliers set equal to 1. The second set of analyses used p-multiplier set equal to 1.8 and the y- multiplier set equal to 1. A p-multiplier of 1.8 was chosen to account for the relative batter of the reinforcing members relative to the direction of soil movement based on Bozok (2009). In all cases analyzed, the reinforcing members where installed vertically, while the soil movement occurred essentially parallel to the slope face. The combined effect is therefore to have an "effective" batter angle for the reinforcing members that is equal to the slope angle. Figure 4.1 shows the recommendations of Bozok (2009), based on large-scale laboratory tests. The slope angle for the I70-Emma site is nominally 23 degrees (or 2.4 H: 1 V), which corresponds to a p-multiplier of 1.8.

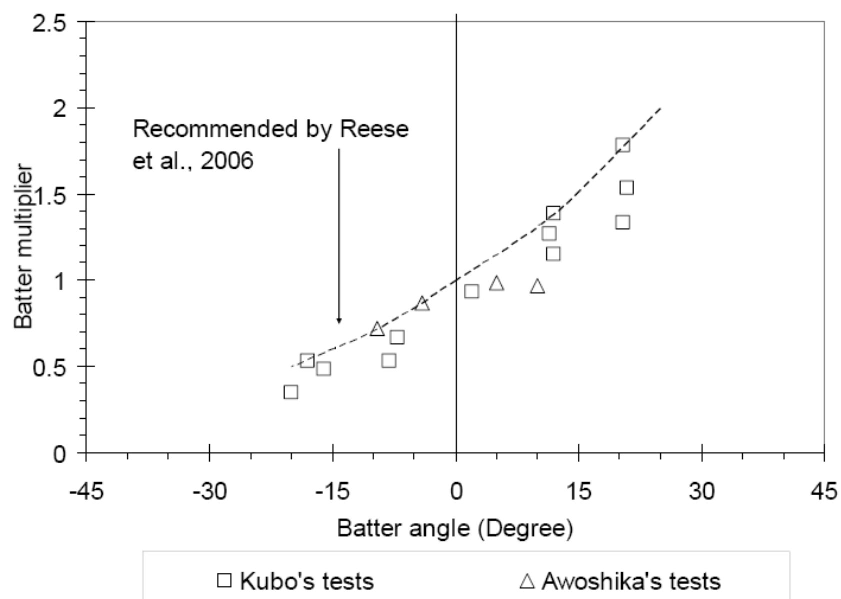


Figure 4.1: Effect of batter angle on soil resistance (Bozok, 2009).

Analyses were performed for several different observation dates, generally corresponding to different magnitudes of soil movement. These dates include: April 23, 2003; July 22, 2003; October 16, 2003; September 16, 2004; and January 27, 2005. The soil properties input into LPILE® for the aforementioned dates are shown in Table 4.1. The low effective unit weights for the September 16, 2004 and January 27, 2005 readings are due to high pore pressures measured on those dates.

Table 4.1: Properties input into LPILE® for the Emma, Missouri site.

		Effective Unit Weight (pcf)			Friction Angle	P-Y Modulus
Layer	Depth (ft)	4/23/03	7/22/03 & 10/16/03*	9/16/04 & 1/27/05*	(degrees)	(lb/in3)
1	0-7.5	88.8	88.8	73.2	23	90
2	7.5-12.5	109.6	99.2	71.5	24	90
3	12.5-beyond	115.5	108.9	102.2	24	90

*Pore pressures on these dates were equal; therefore the effective unit weights are equal.

4.1.1 Results for Member IM 24, Emma site

Analyses were performed for member IM 24 located at the Emma site. Soil movement profiles for this member were taken from inclinometer I-7. Soil movement profiles used for the aforementioned dates are shown in Figure 4.2. Numeric values for each of these movements are listed in Table 4.2. The movements utilized correspond to movements determined from inclinometer measurements at the depths specified to the depth of sliding. Therefore, all movements below 4 feet were input as zero.

The predicted moments are compared to the corresponding measured moments for the respective measurement dates in Figures 4.3-4.7 for analyses performed using p-multipliers of 1.0 and 1.8. The shape and magnitude of the moments predicted by LPILE® generally match with the measured moments for both values of p-multiplier. However, the predicted moments analyzed using a p-multiplier equal to 1.8 appears to match the magnitude of the measured moments better.

Table 4.2: Soil movements input into LPILE® for member IM 24 at the Emma, Missouri site.

Date	4/23/2003	7/22/2003	10/16/2003	9/16/2004	1/27/2005
Depth from Surface (ft)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)
0	0.25	0.42	2.00	2.65	5.20
1	0.25	0.41	1.68	2.33	4.79
3	0.18	0.30	1.04	1.60	3.93
4	0.14	0.24	0.80	1.32	3.30

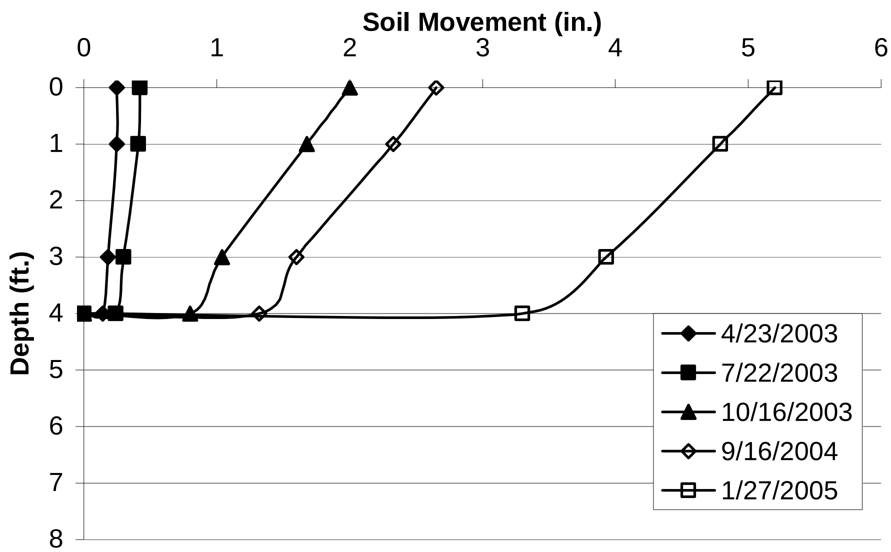


Figure 4.2: Soil movement profiles used for member IM 24 at the Emma, Missouri site.

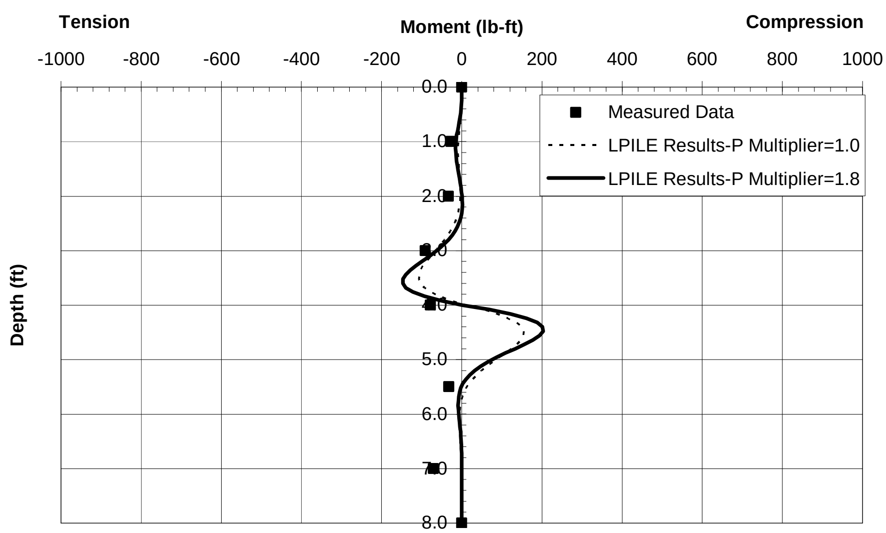


Figure 4.3: Comparison of measured and predicted bending moments for member IM 24 at Emma based on measured soil movements from April 23, 2003.

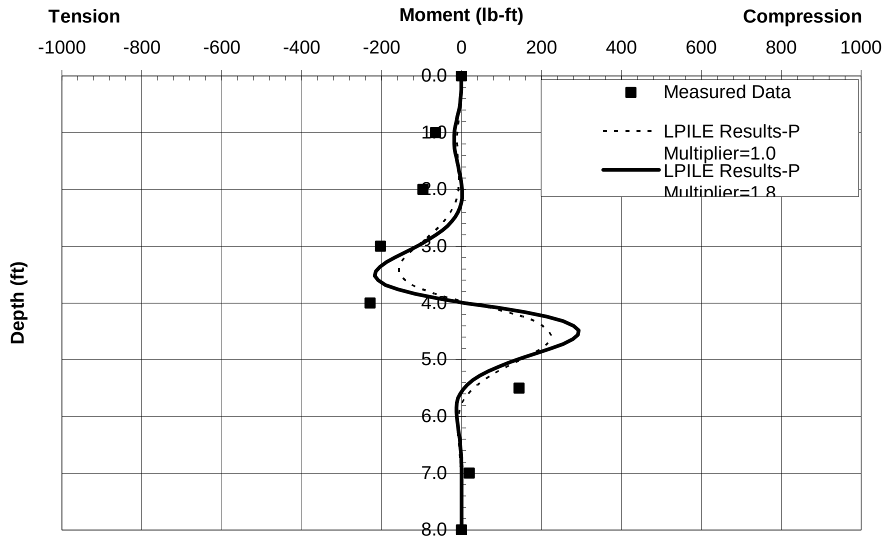


Figure 4.4: : Comparison of measured and predicted bending moments for member IM 24 at Emma based on measured soil movements from July 22, 2003.

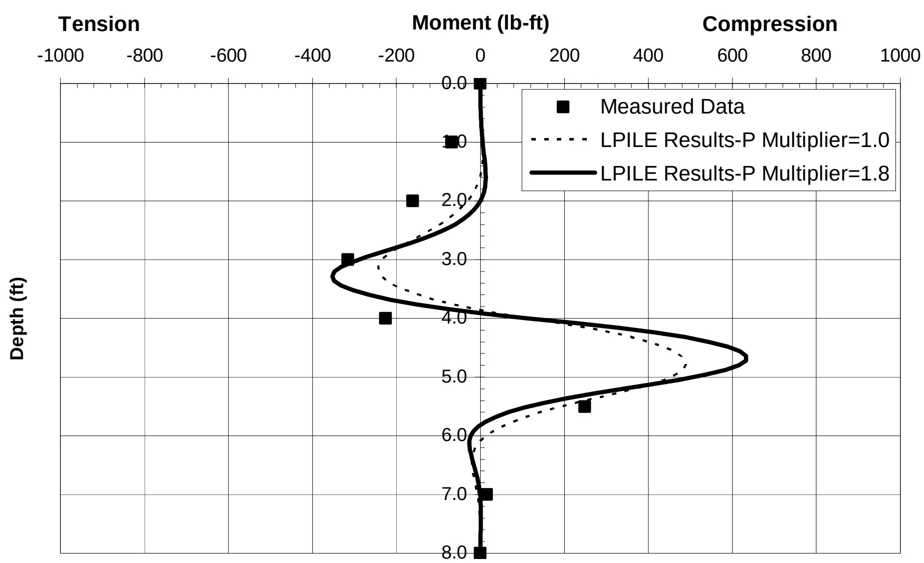


Figure 4.5: : Comparison of measured and predicted bending moments for member IM 24 at Emma based on measured soil movements from October 16, 2003.

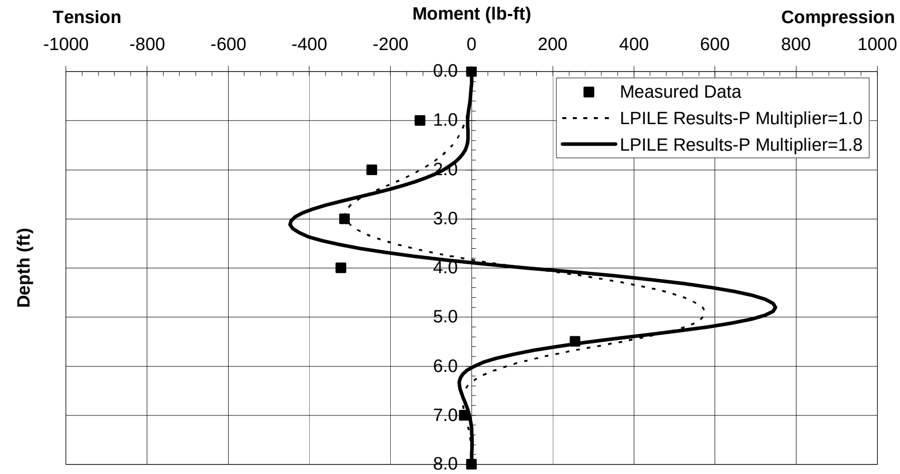


Figure 4.6: : Comparison of measured and predicted bending moments for member IM 24 at Emma based on measured soil movements from September 16, 2004.

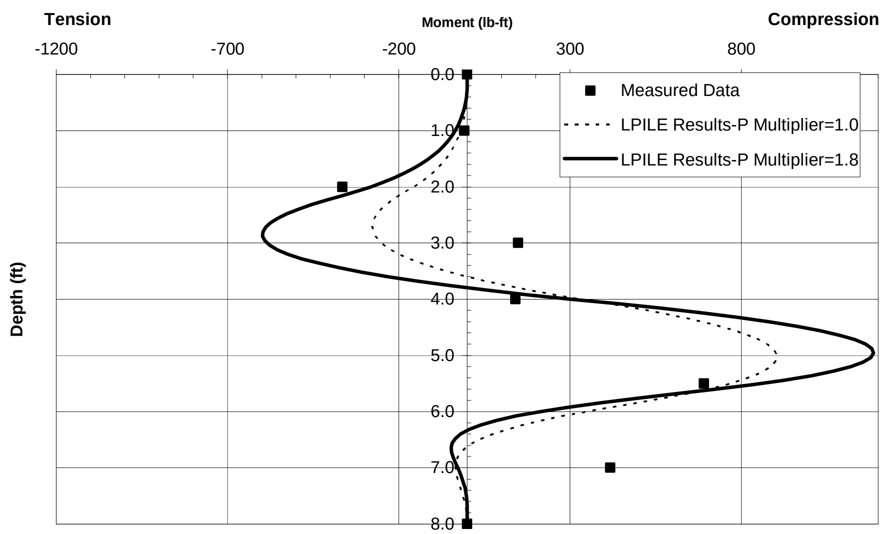


Figure 4.7: : Comparison of measured and predicted bending moments for member IM 24 at Emma based on measured soil movements from January 27, 2005.

Figure 4.8 shows the maximum predicted moments from the LPILE® analyses for the respective measurement dates and model conditions as well as the maximum measured moments for those same dates plotted versus the average soil movement for those dates. The figure shows that the predicted maximum moments consistently exceed the maximum measured moment for all soil movement values. However, the trends in maximum moment are generally consistent. It is not surprising that the maximum predicted moments exceed the maximum measured moments because strain gages were located at specific locations, which may or may not coincide with the location of the actual maximum moment. The fact that the maximum predicted moments exceed the maximum measured moments, therefore, does not imply that the predictions are erroneous. As expected, the predicted maximum moments are greater for analyses performed using a p-multiplier of 1.8 as compared to results for analyses performed using a p-multiplier of 1.0. Considering the comparisons plotted in Figures 4.3 through 4.7, the results of analyses performed using both p-multipliers of 1.8 and 1.0 suggest that both values can be used to produce predictions of the measured reinforcement response.

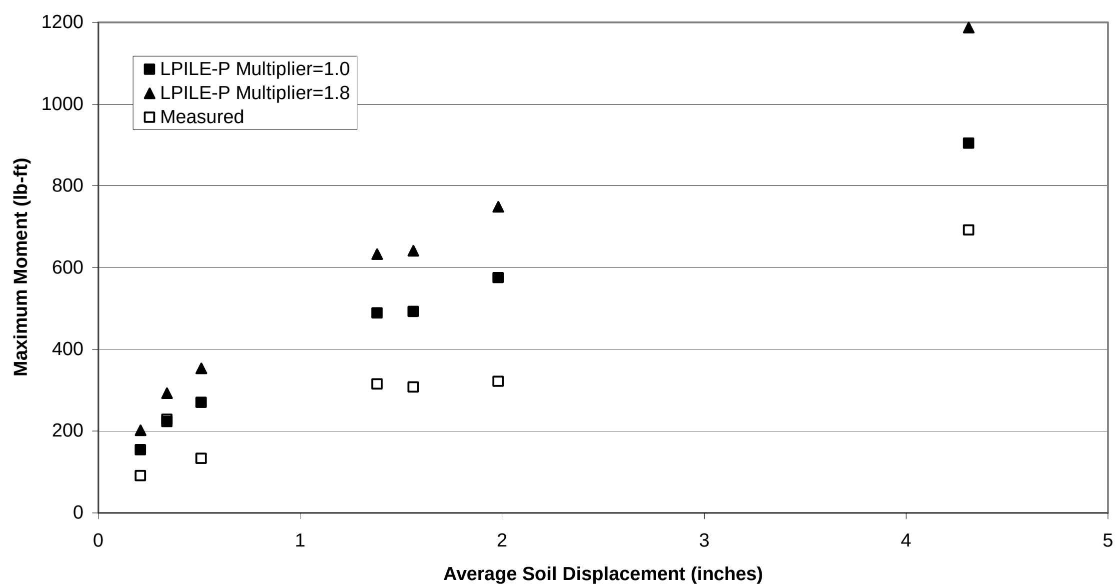


Figure 4.8: Comparison of maximum predicted moments and maximum measured moments for IM 24 at Emma.

4.1.2 Results for Member IM 19, Emma site

Soil movement profiles for member IM 19 were taken from readings from inclinometer I-6. Strain gages located at depths of 2 and 7 feet were not included in analyses for all measurement dates because the gages appeared to have shorted out, causing erroneous readings. The strain gage located at 5.5 feet was excluded from analyses for dates after July 22, 2003 for similar reasons. Soil movement profiles used for the aforementioned dates are shown in Figure 4.9; specific values used are summarized in Table 4.3. The movements represent deformations as recorded by the inclinometers at the depths specified. All movements below a depth of 4 feet were input as zero.

Predicted moments from the analyses are compared to the measured moments for the dates analyzed in Figures 4.10-4.14. Again, the shape and magnitude of the moments predicted by LPILE® generally match with the measured moments for both values of p-multiplier. The predicted moments analyzed using a p-multiplier equal to 1.8 appears to match the magnitude of the measured moments better, particularly for the dates of September 16, 2004 and for January 27, 2005.

Table 4.3: Soil movements input into LPILE® for member IM 19 at the Emma, Missouri site.

Date	4/23/2003	7/22/2003	10/16/2003	9/16/2004	1/27/2005
Depth from Surface (ft)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)
0	0.00	-0.15	0.62	1.40	2.00
0.8	0.02	-0.06	0.59	1.26	1.68
2.8	0.12	0.18	0.43	0.87	0.96
4	0.12	0.20	0.31	0.71	0.75

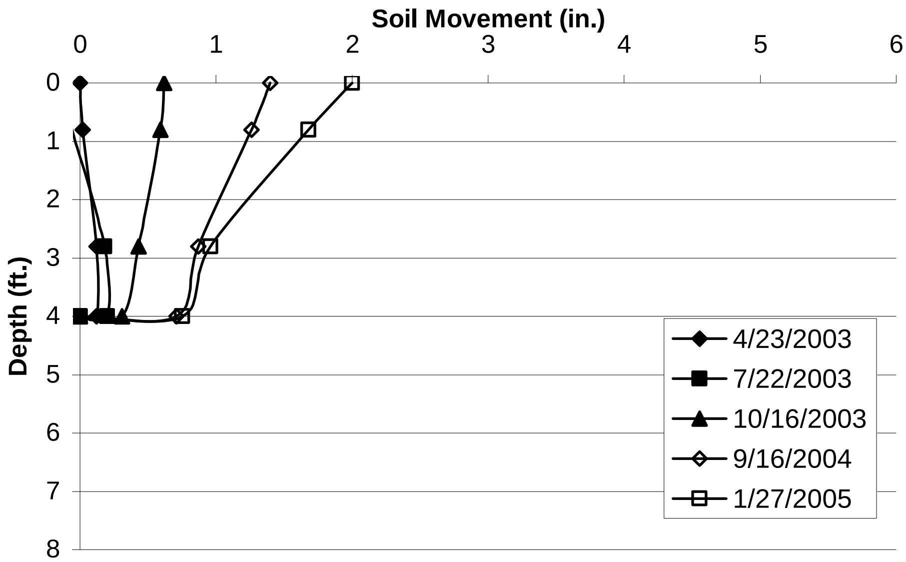


Figure 4.9: Soil movement profiles used for member IM 19 at the Emma, Missouri site.

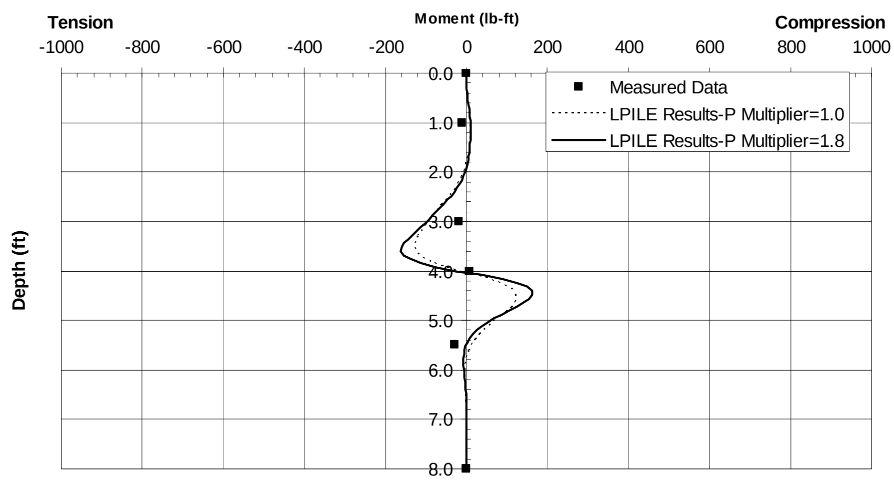


Figure 4.10: Comparison of measured and predicted bending moments for member IM 19 at Emma based on measured soil movements from April 23, 2003.

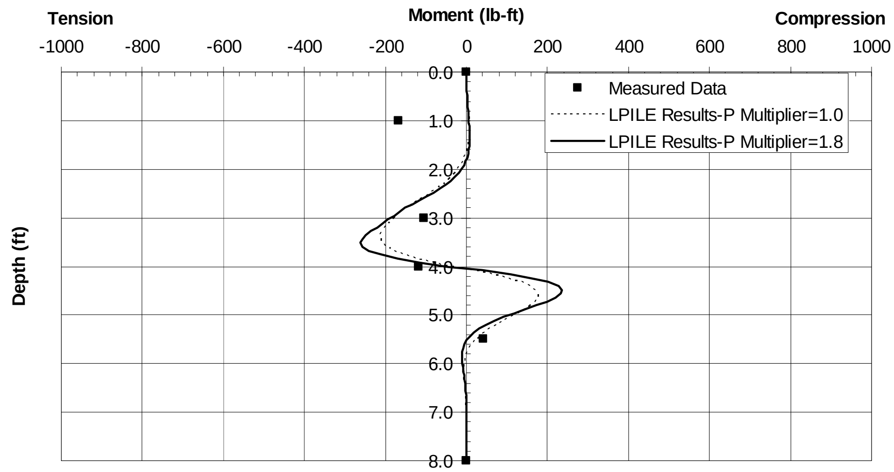


Figure 4.11: Comparison of measured and predicted bending moments for member IM 19 at Emma based on measured soil movements from July 22, 2003.

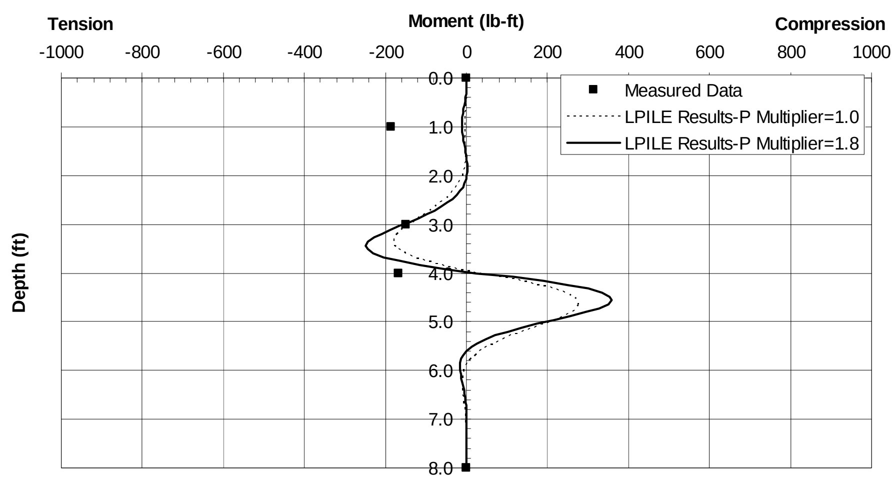


Figure 4.12: Comparison of measured and predicted bending moments for member IM 19 at Emma based on measured soil movements from October 16, 2003.

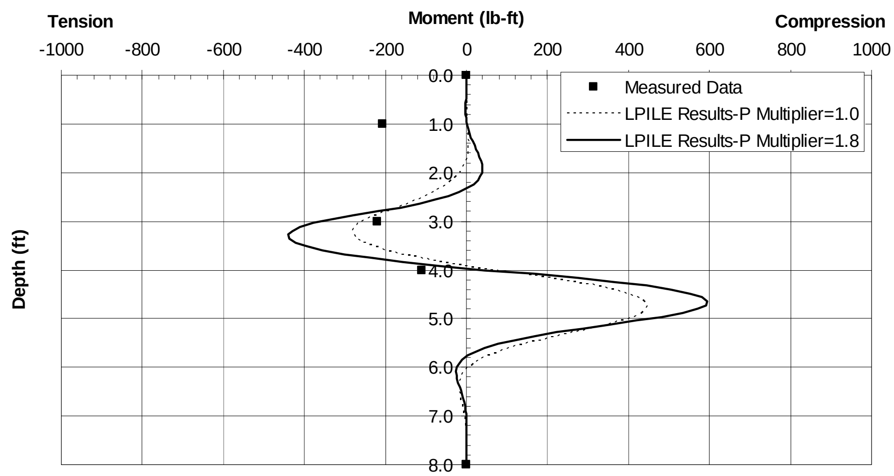


Figure 4.13: Comparison of measured and predicted bending moments for member IM 19 at Emma based on measured soil movements from September 16, 2004.

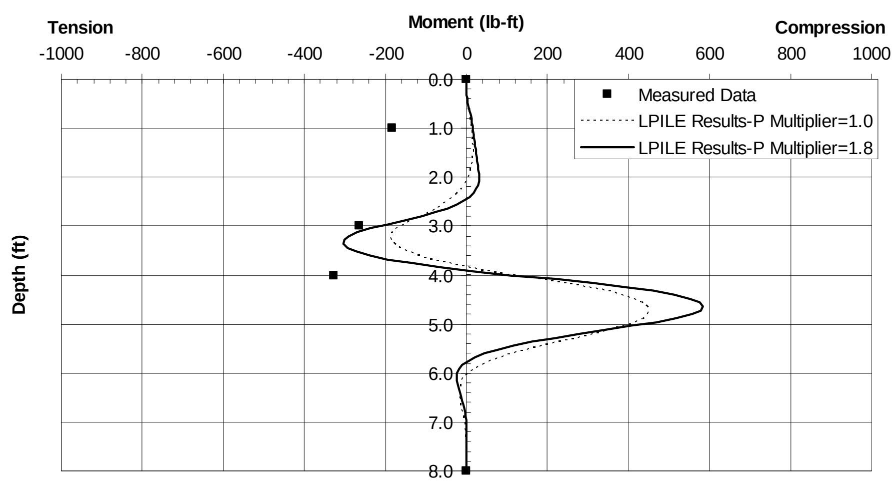


Figure 4.14: Comparison of measured and predicted bending moments for member IM 19 at Emma based on measured soil movements from January 27, 2005.

Figure 4.15 compares the maximum predicted moments for the respective measurement dates and model conditions as well as the maximum measured moments for those same dates, plotted as a function of average soil displacement. As was the case for member IM24, predicted maximum moments exceed the maximum measured moments for all soil movement values. The trends in the predicted and measured values are quite consistent, however, which suggests that the predictions are consistent with the actual response of the reinforcing members. Collectively considering the results of all analyses, it appears that good matches of predicted and measured member response can be achieved using both values for the p-multiplier.

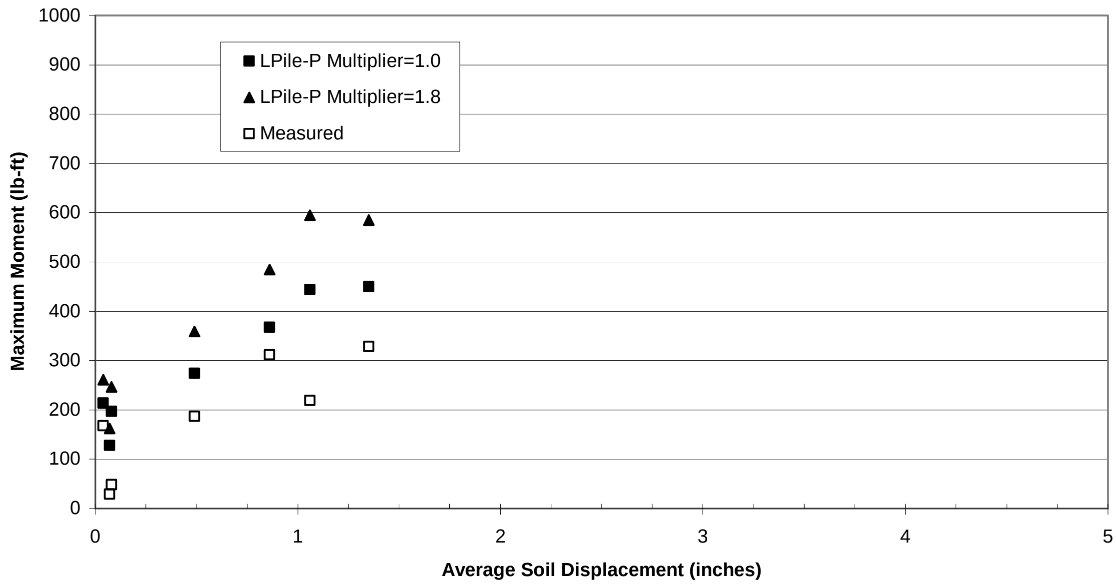


Figure 4.15: Comparison of maximum predicted moments and maximum measured moments for IM 19 at Emma.

4.1.3 Results for Member IM 18 and Member IM 23, Emma site

The response for members IM 18 and IM 23 were analyzed together because they are both in close proximity to inclinometer I-8. Soil movement profiles for these members were taken from inclinometer I-8. I-8 was "pinched off" at a depth of 7 feet after September 16, 2004 after which no measurements were taken. Therefore, no analyses were performed for measurements after that date. The soil movement profiles used are shown in Figure 4.16; specific values used are summarized in Table 4.4. The movements represent deformations as recorded by the inclinometers at the depths specified to the depth of sliding. Thus, all movements below a depth of 4 feet were input as zero.

Predicted moments are compared to the measured moments for the dates analyzed in Figures 4.17-4.20. The shape and magnitude of the moments predicted by LPILE® generally match with the measured moments for both values of p-multiplier. The predicted moments determined using a p-multiplier of 1.8 match the magnitudes of the measured moments, particularly at larger soil movements.

Table 4.4: Soil movements input into LPILE® for members IM 18 and 23 at the Emma, Missouri site.

Date	4/23/2003	7/22/2003	10/16/2003	9/16/2004
Depth from Surface (ft)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)
0	0.00	1.00	2.00	2.90
2	0.03	0.64	1.44	2.08
4	0.05	0.29	0.87	1.35

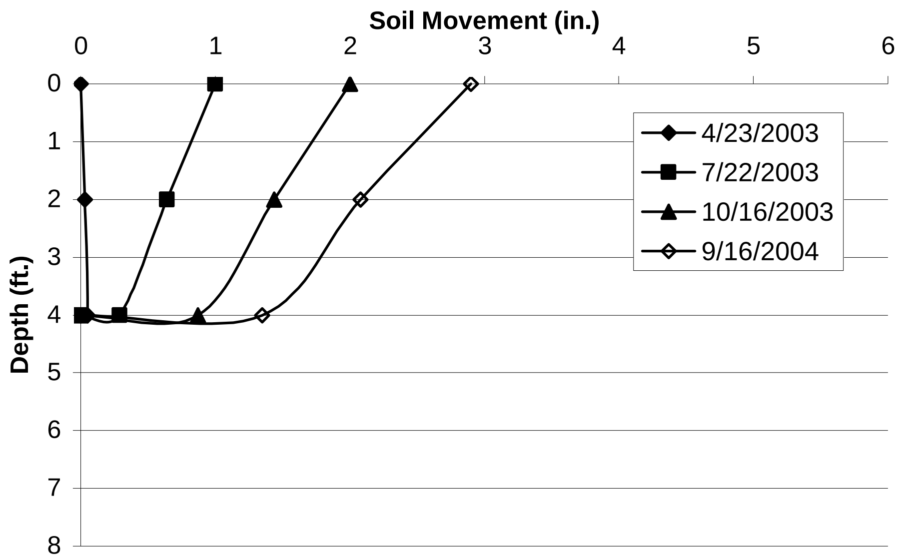


Figure 4.16: Soil movement profiles used for members IM 18 and 23 at the Emma, Missouri site.

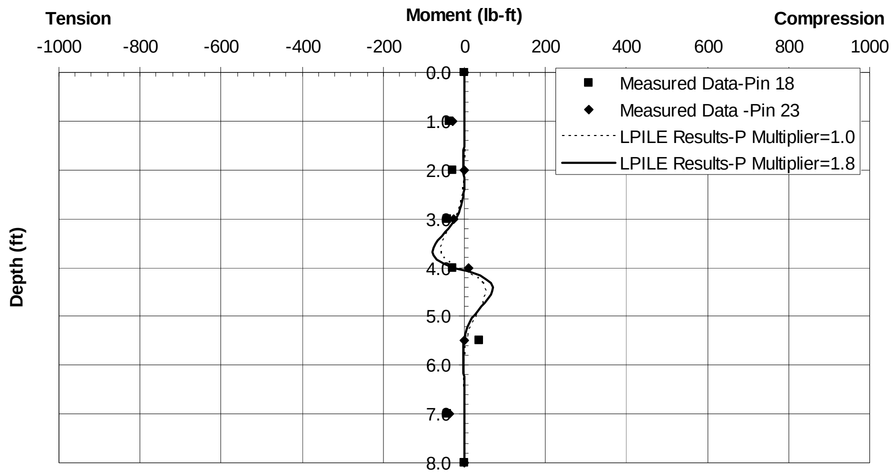


Figure 4.17: Comparison of measured and predicted bending moments for members IM 18 and IM 23 at Emma based on measured soil movements from April 23, 2003.

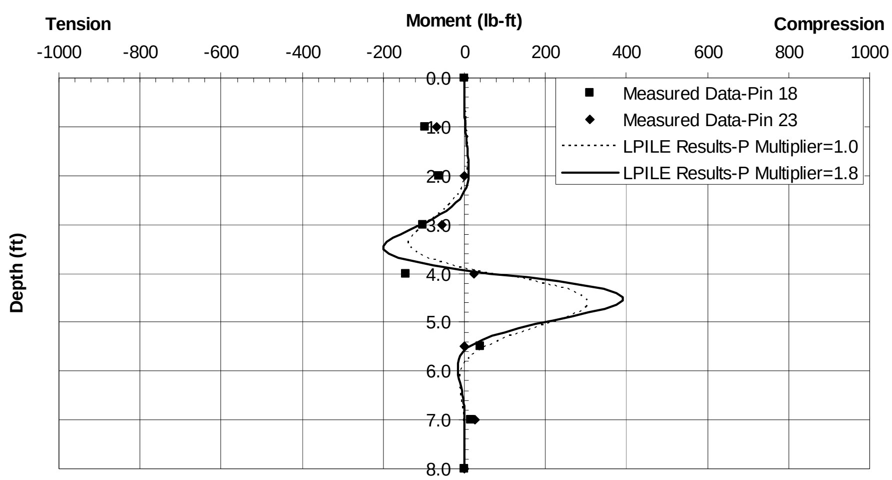


Figure 4.18: Comparison of measured and predicted bending moments for members IM 18 and IM 23 at Emma based on measured soil movements from July 22, 2003.

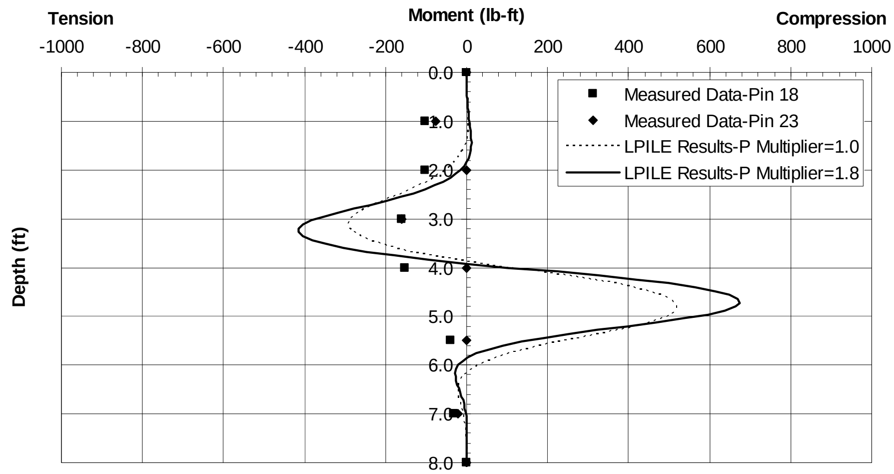


Figure 4.19: Comparison of measured and predicted bending moments for members IM 18 and IM 23 at Emma based on measured soil movements from October 16, 2003.

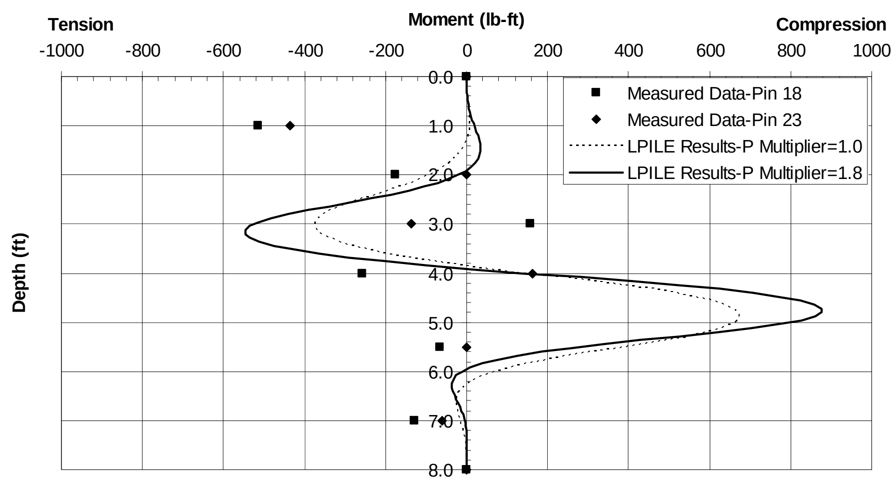


Figure 4.20: Comparison of measured and predicted bending moments for members IM 18 and IM 23 at Emma based on measured soil movements from September 16, 2004.

Figure 4.21 compares the maximum predicted moments for the respective measurement dates and model conditions as well as the maximum measured moments for those same dates, plotted as a function of average soil displacement. The predicted maximum moments again exceed the maximum measured moment for all soil movement values, as expected.

Considering the collective results of analyses performed for the instrumented members at the I70-Emma test site, it appears that closest comparisons among predicted and measured bending moments can be achieved using p-multipliers of 1.0 and 1.8. While the predicted distribution of bending moments are similar when using both of these values, the predicted distributions tend to match the measured values slightly better when 1.8 is used as the p-multiplier.

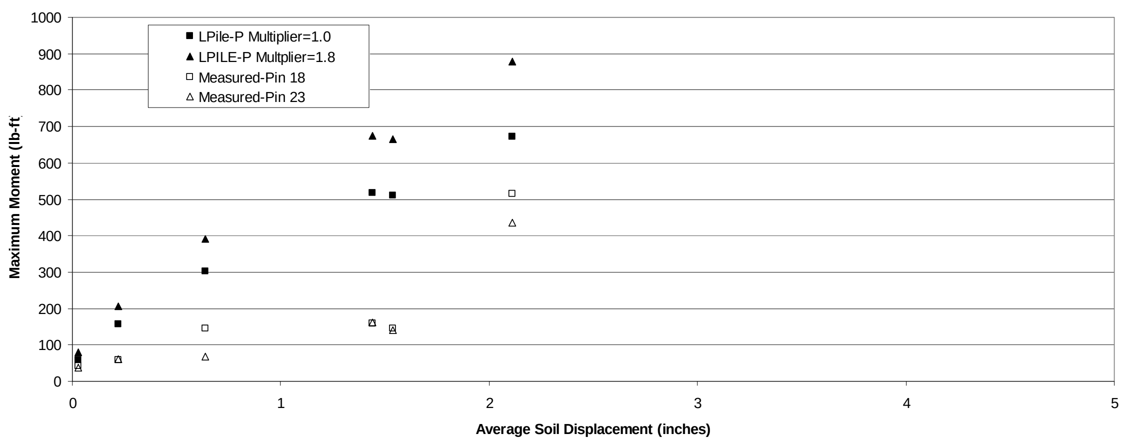


Figure 4.21: Comparison of maximum predicted moments and maximum measured moments for IM 18 and IM 23 at Emma.

4.2 US 36-Stewartsville Results

Analyses were performed for several instrumented members at the US 36-Stewartsville site including members: IM 9, IM 12, IM 13 and IM 14. Typical dimensions of the reinforcing members are 3.5 inches long, 3.5 inches wide, and 8 feet in length. This corresponds to a moment of inertia of 11.75 inches^4. The modulus of elasticity used was 145 ksi (Chen, 2003). Member IM 9, however, was a 3.5 inch diameter steel pipe. This produced a cross section area of 1.96 square inches and a moment of inertia of 2.72 inches^4. The modulus of elasticity used was 29,000 ksi. Pile head boundary conditions for all members were unrestrained with zero shear and zero moment for all analyses. The p-y curve curve used for all analyses performed for the Stewartsville site was the "API Sand (O'Neill, 1983)" curve.

As was done for members from the I70-Emma site, the sliding depth and p-multipliers were varied until the predicted and measured moments compared closely. Three different p-multipliers were considered for the Stewartsville site. The first set of analyses were performed using p- and y- multipliers set equal to 1. A second set of analyses was performed using a p-multiplier equal to 1.8 with the y- multiplier of 1. A third analysis had the p-multiplier set to 8.0 and the y- multiplier set to 1. The first and third set of analyses were performed to establish lower and upper bounds for predicted moments. The analyses using a p-multiplier of 1.8 was performed to account for the effects of pile batter relative to the direction of soil movement as was the case for the I70-Emma site (Figure 4.1).

Analyses were performed for several different observation dates, generally corresponding to different magnitudes of soil movement. These dates include: November 15, 2002; June 19, 2003; July 26, 2004; September 28, 2004; and February 16, 2005. The soil properties used for the LPILE® analyses for the aforementioned dates are shown in Table 4.5.

Table 4.5: Properties input into LPILE® for the US 36-Stewartsville, Missouri site.

		Effective Unit Weight (pcf)			Friction Angle	P-Y Modulus
Layer	Depth (ft)	11/15/02	6/19/03	7/26/04 & 2/16/05*	(degrees)	(lb/in3)
1	0-6	120	116.5	109.6	29	90
2	6-Beyond	84.9	84.9	75.8	35	90

*Pore pressures on these dates were equal; therefore the effective unit weights are equal.

4.2.1 Results for Member IM 9, Stewartsville site

The soil movement profiles for member IM 9 were taken from readings of inclinometer I-5c. The soil movement profiles from these profiles for the aforementioned dates are shown in Figure 4.22; specific values used are summarized in Table 4.6. The movements represent deformations as recorded by the inclinometers at the depths specified. All movements below a depth of 4 feet (sliding depth) were input as zero.

Predicted moments are compared to the measured moments for the dates analyzed in Figures 4.23-4.26. The analysis using a p-multiplier equal to 8 predicts moments that match well with the measured moments for the readings taken on dates November 15, 2002 and September 28, 2004. The predicted moments determined using p-multipliers of 1.8 and 8 both match the measured moments for the date of July 26, 2004. None of the predicted moments match the measured moments for the date of June 19, 2003. Predicted moments determined using a p-multiplier equal to 1 are substantially less than the measured moments for all of the dates.

Table 4.6: Soil movements input into LPILE® for member IM 9 at the Stewartsville, Missouri site.

Date	11/15/2002	6/19/2003	7/26/2004	9/28/2004
Depth from Surface (ft)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)
0	-0.02	0.35	2.60	10.80
0.75	-0.21	0.29	2.29	9.45
2.75	-0.24	0.07	1.80	5.35
4	-0.20	0.00	0.80	2.40

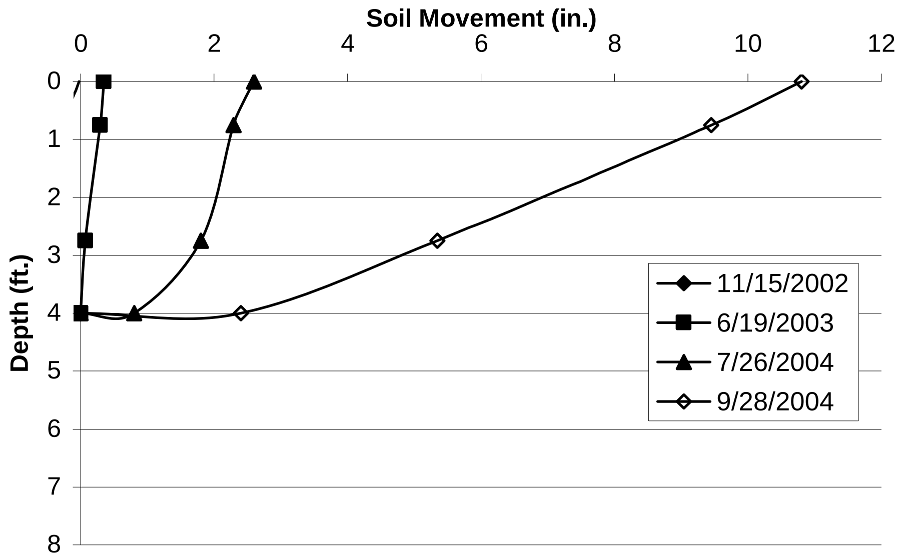


Figure 4.22: Soil movement profiles for member IM 9 at the Stewartsville, Missouri site.

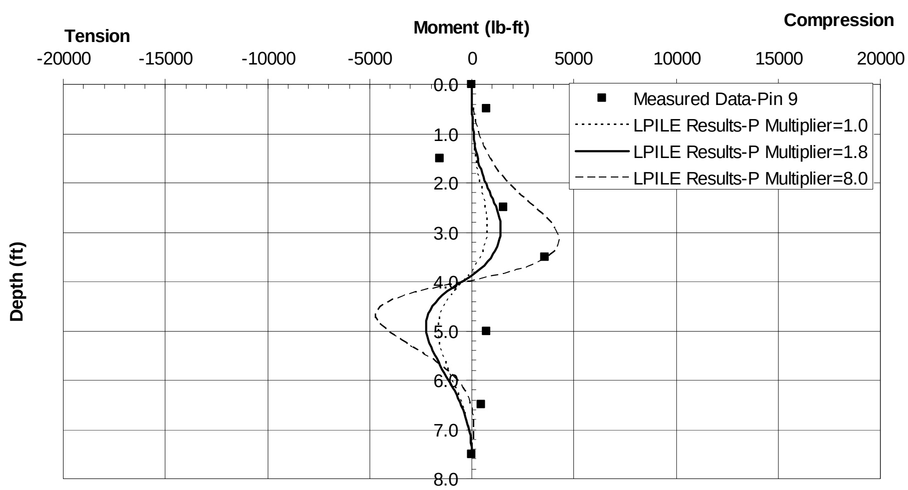


Figure 4.23: Comparison of measured and predicted bending moments for member IM 9 at Stewartsville based on measured soil movements from November 15, 2002.

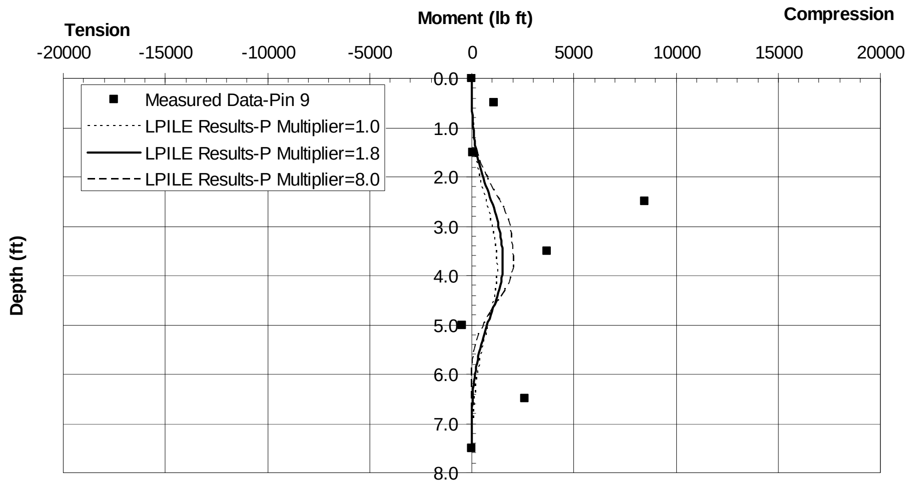


Figure 4.24: Comparison of measured and predicted bending moments for member IM 9 at Stewartsville based on measured soil movements from June 19, 2003.

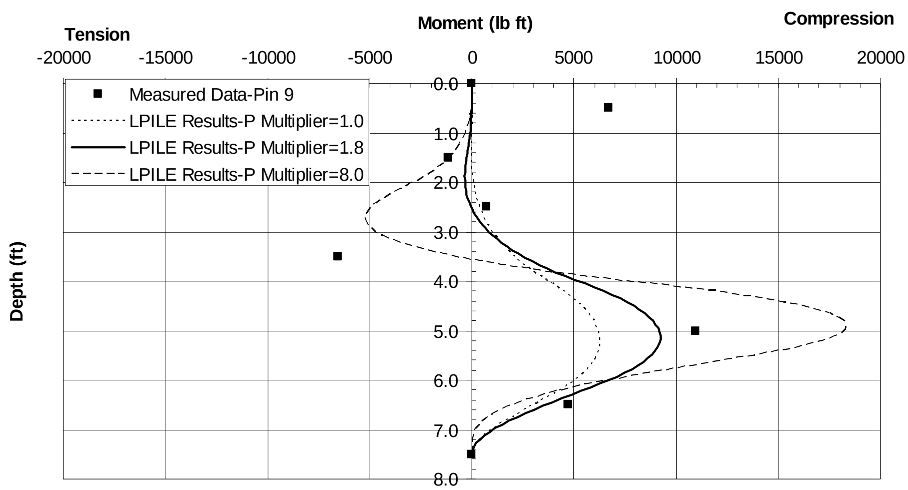


Figure 4.25: Comparison of measured and predicted bending moments for member IM 9 at Stewartsville based on measured soil movements from July 26, 2004.

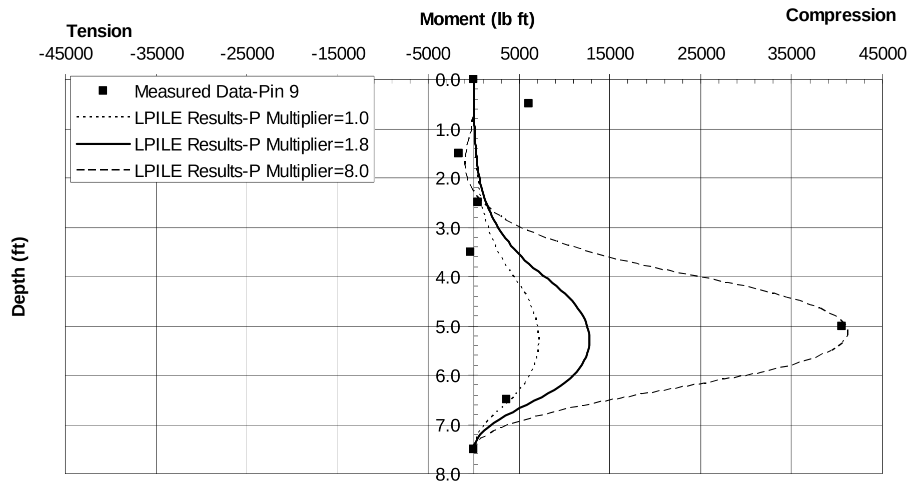


Figure 4.26: Comparison of measured and predicted bending moments for member IM 9 at Stewartsville based on measured soil movements from September 28, 2004.

Figure 4.27 compares the maximum predicted moments for the respective measurement dates and model conditions as well as the maximum measured moments for those same dates, plotted as a function of average soil displacement. As expected, the predicted maximum moments are greater for analyses performed using a p-multiplier of 1.8 as compared to results for analyses performed using a p-multiplier of 1.0. Likewise, the predicted maximum moments are greater for analyses performed using a p-multiplier of 8.0 as compared to results for analyses performed using a p-multiplier of 1.8. When viewed collectively, the moments predicted using a p-multiplier of 8.0 compared best with the measured moments for all dates.

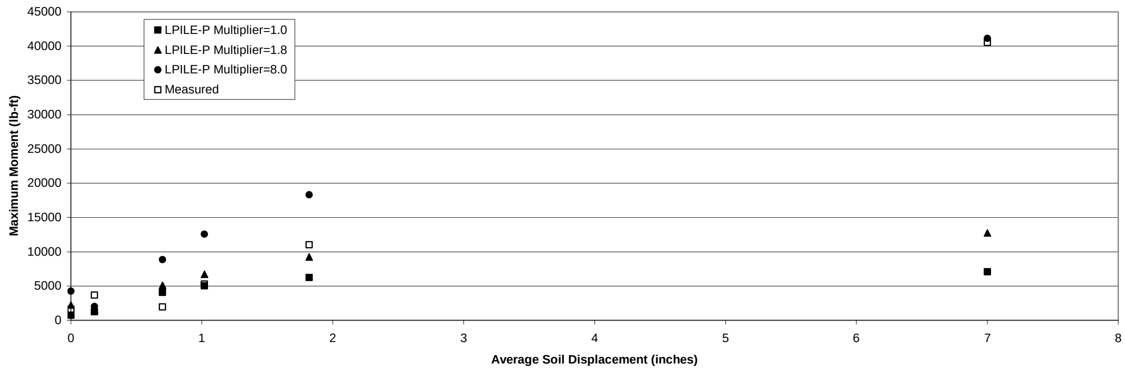


Figure 4.27: Comparison of maximum predicted moments and maximum measured moments for IM 9 at Stewartsville.

4.2.2 Results for Member IM 12, Stewartsville site

The soil movement profiles for member IM 12 were taken from readings from inclinometer I-4. Soil movement profiles used for the aforementioned dates are shown in Figure 4.28; specific values from these profiles are summarized in Table 4.7. The movements represent deformations as recorded by the inclinometers at the depths specified. All movements below a depth of 4 feet were input as zero.

Predicted moments are compared to the measured moments for the dates analyzed in Figures 4.29-4.33. Generally, the predicted moments match the measure moments for all p-multipliers. The magnitude of the p-multiplier has little effect on the predicted bending moments because the measured bending moments for member IM 12 are relatively small.

Table 4.7: Soil movements input into LPILE® for member IM 12 at the Stewartsville, Missouri site.

Date	11/15/2002	6/19/2003	7/26/2004	9/28/2004	2/16/2005
Depth from Surface (ft)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)
0	-0.11	0.50	1.15	1.58	2.35
2.75	-0.09	0.17	0.56	0.52	1.02
4	-0.09	0.02	0.12	0.15	0.50

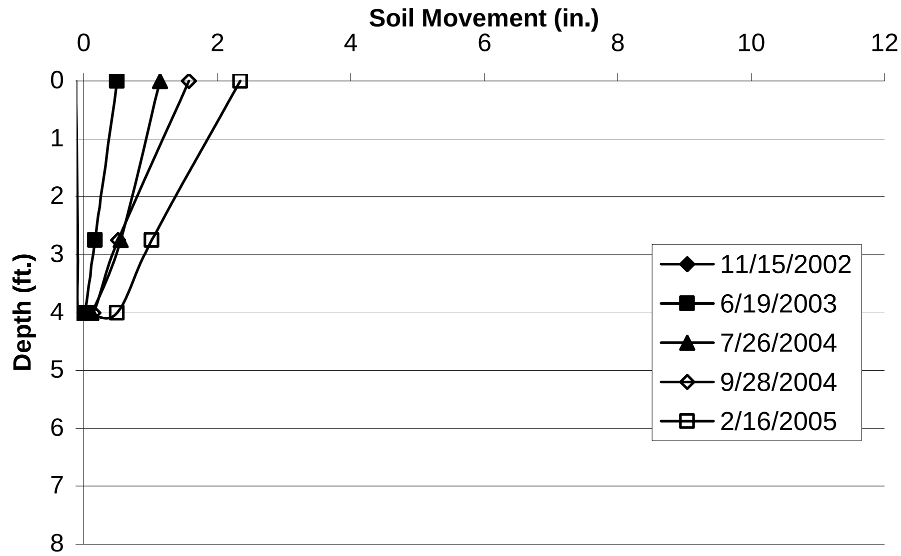


Figure 4.28: Soil movement profiles for member IM 12 at the Stewartsville, Missouri site.

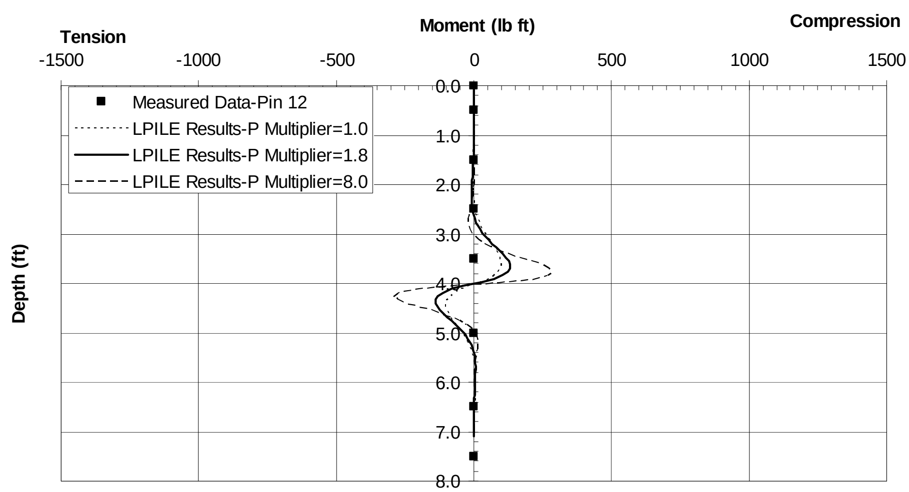


Figure 4.29: Comparison of measured and predicted bending moments for member IM 12 at Stewartsville based on measured soil movements from November 15, 2002.

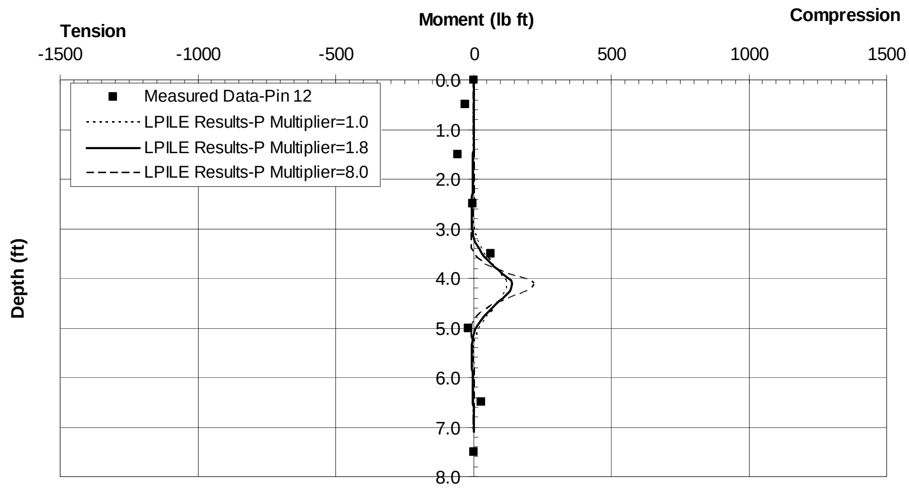


Figure 4.30: Comparison of measured and predicted bending moments for member IM 12 at Stewartsville based on measured soil movements from June 19, 2003.

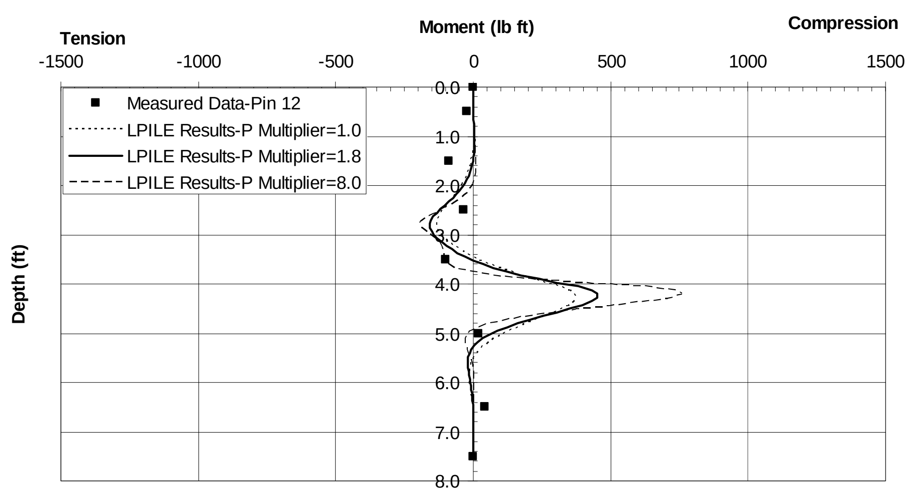


Figure 4.31: Comparison of measured and predicted bending moments for member IM 12 at Stewartsville based on measured soil movements from July 26, 2004.

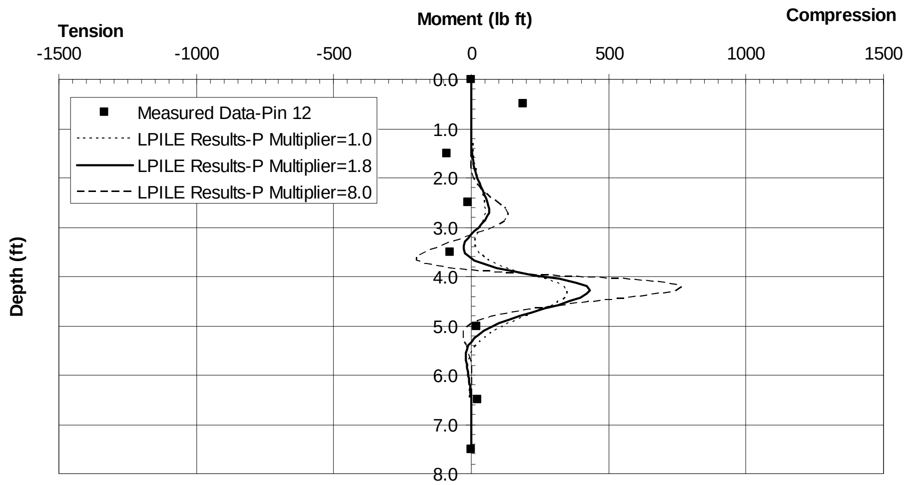


Figure 4.32: Comparison of measured and predicted bending moments for member IM 12 at Stewartsville based on measured soil movements from September 28, 2004.

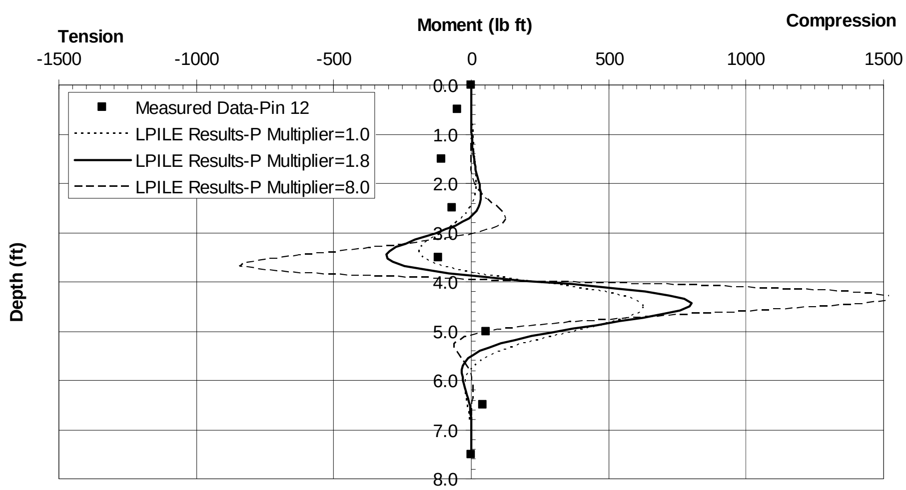


Figure 4.33: Comparison of measured and predicted bending moments for member IM 12 at Stewartsville based on measured soil movements from February 16, 2005.

Figure 4.34 compares the maximum predicted moments for the respective measurement dates and model conditions as well as the maximum measured moments for those same dates, plotted as a function of average soil displacement. The predicted moments determined using all p-multipliers were substantially greater than the measured moments. However, the predicted maximum moments are much greater for analyses performed using a p-multiplier of 8.0 as compared to results for analyses performed using a p-multiplier of 1.0 and 1.8. Since the magnitudes of the measured moments are small, predicted moments determined using any reasonable p-multiplier will produce results that will match the measured moments.

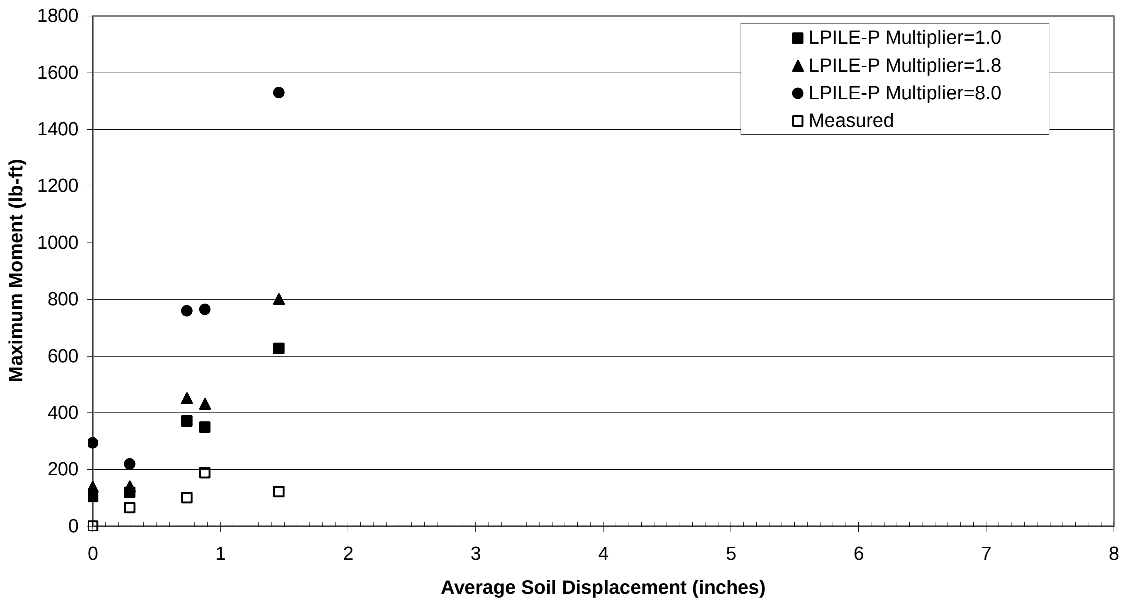


Figure 4.34: Comparison of maximum predicted moments and maximum measured moments for IM 12 at Stewartsville.

4.2.3 Results for Members IM 13 and IM 14, Stewartsville site

The soil movement profiles for members IM 13 and IM 14 were taken from readings of inclinometer I-1. The soil movement profiles used for the aforementioned dates are shown in Figure 4.35; specific values from these profiles are summarized in Table 4.8. The movements represent deformations as recorded by the inclinometers at the depths specified. All movements below a depth of 4 feet (sliding depth) were input as zero.

Predicted moments are compared to the measured moments for the dates analyzed in Figures 4.36-4.40. Similar to IM 12, the magnitudes of the measured moments are relatively small. Therefore, the predicted moments will reasonably match the measured moments with a wide range of p-multipliers. Predicted moments determined using a p-multiplier of 8 over predicts the moments at larger soil movements, as seen by Figures 4.38-4.40. Predicted moments determined using p-multipliers of 1.0 and 1.8 both match the measured moments with reasonable accuracy for all soil movement readings.

Table 4.8: Soil movements input into LPILE® for members IM 13 and 14 at the Stewartsville, Missouri site.

Date	11/15/2002	6/19/2003	7/26/2004	9/28/2004	2/16/2005
Depth from Surface (ft)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)
0	0.25	0.20	1.42	1.60	1.78
0.83	0.22	0.14	1.12	1.33	1.50
2.83	0.13	0.06	0.56	0.66	0.83
4	0.11	0.02	0.32	0.38	0.51

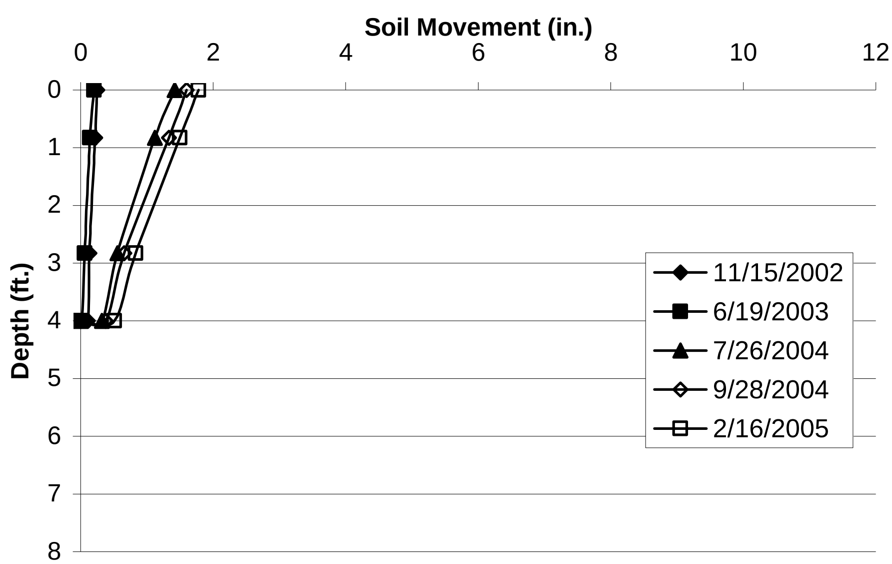


Figure 4.35: Soil movement profiles for members IM 13 and IM 14 at the Stewartsville, Missouri site.

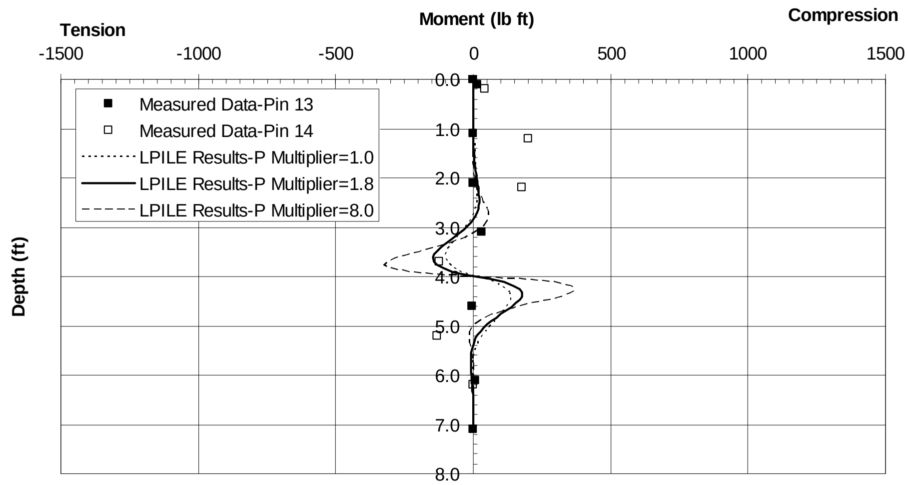


Figure 4.36: Comparison of measured and predicted bending moments for members IM 13 & IM 14 at Stewartsville based on measured soil movements from November 15, 2002.

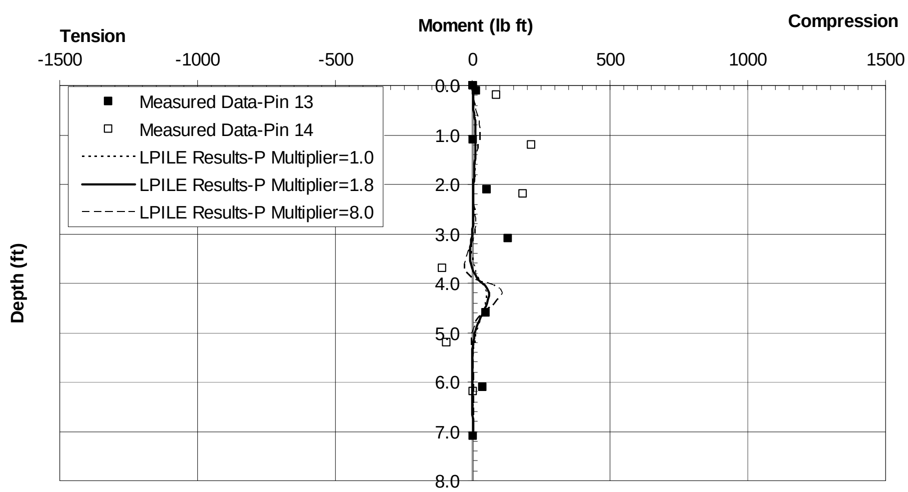


Figure 4.37: Comparison of measured and predicted bending moments for members IM 13 & IM 14 at Stewartsville based on measured soil movements from June 19, 2003.

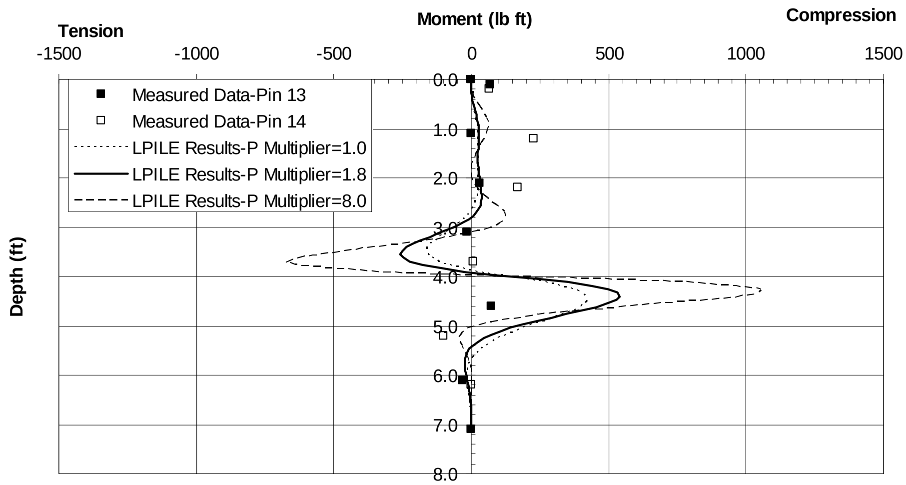


Figure 4.38: Comparison of measured and predicted bending moments for members IM 13 & IM 14 at Stewartsville based on measured soil movements from July 26, 2004.

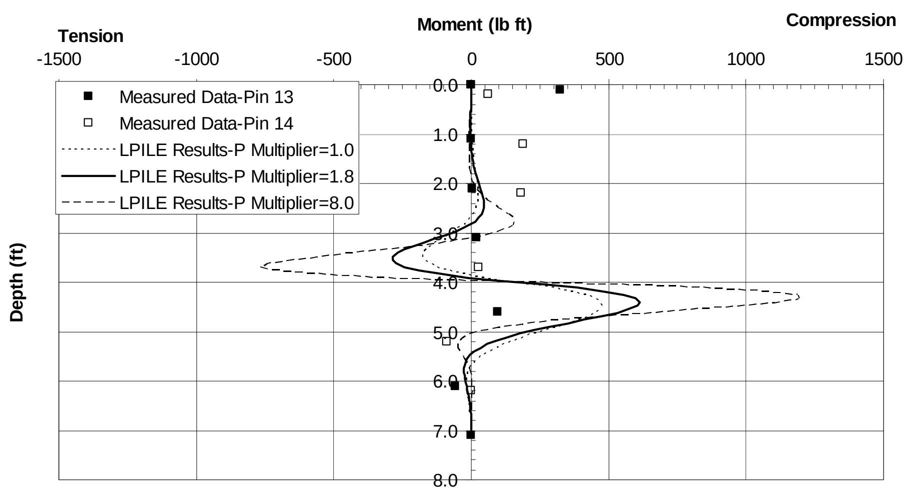


Figure 4.39: Comparison of measured and predicted bending moments for members IM 13 & IM 14 at Stewartsville based on measured soil movements from September 28, 2004.

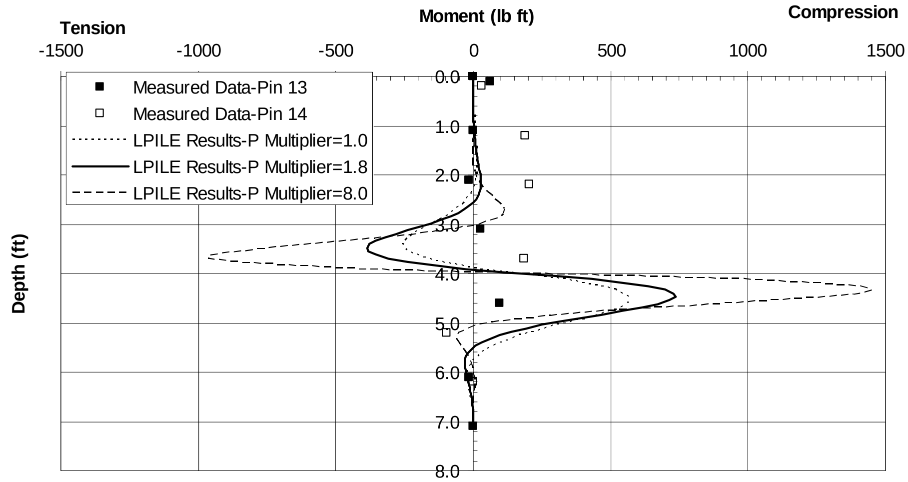


Figure 4.40: Comparison of measured and predicted bending moments for members IM 13 & IM 14 at Stewartsville based on measured soil movements from February 16, 2005.

Figure 4.41 compares the maximum predicted moments for the respective measurement dates and model conditions as well as the maximum measured moments for those same dates, plotted as a function of average soil displacement. The maximum predicted moments for p-multipliers equal to 1.0 and 1.8 compare well with the maximum measured moments for displacements less than 0.25 inches. At displacements greater than 0.25 inches, the maximum predicted moments are substantially greater than the maximum measured moments. Predicted moments determined using a p-multiplier equal to 8.0 greatly exceed the measured moments for all displacements.

The measured moments for the steel pipe member IM9 at the US36-Stewartsville test site did not have a consistent shape when compared with the measured moments for the plastic members at the Emma site. Predicted moments determined with a p-multiplier of 8 matched the measured moments best for member IM9.

Measured moments for members IM12-14 were small in magnitude at all depths for readings taken on all analyzed dates. Therefore, the predicted moments determined with both p-multipliers equal to 1.0 and 1.8 compared well with the measured moments.

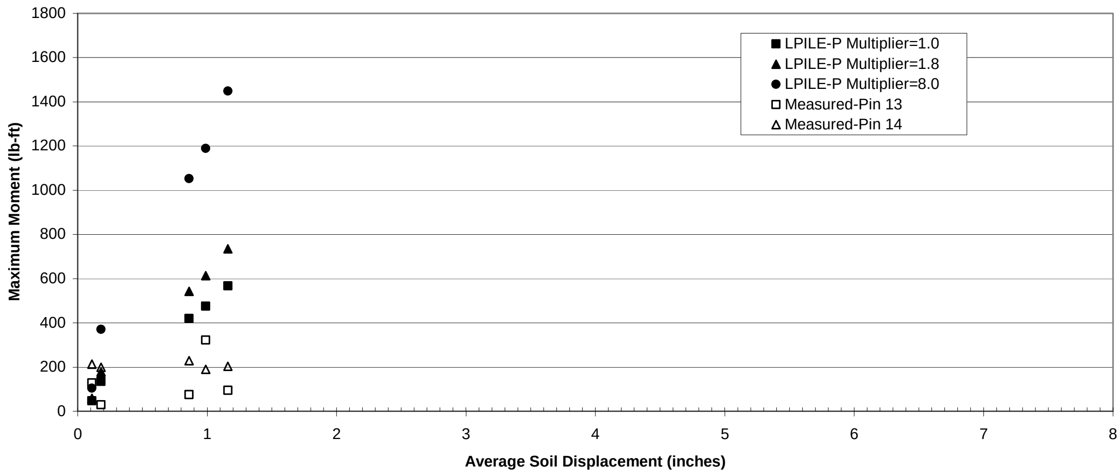


Figure 4.41: Comparison of maximum predicted moments and maximum measured moments for IM 13 and 14 at Stewartsville.

4.3 I435-Wornall Road-Kansas City Results

Analyses were performed for four instrumented members at the I435-Wornall Road site including: IM 1, IM 2, IM 3 and IM 4. Typical dimensions of the reinforcing members are 3.5 inches long, 3.5 inches wide, and 8 feet in length. This corresponds to a moment of inertia of 11.75 inches^4. The modulus of elasticity used was 145 ksi (Chen, 2003). Pile head boundary conditions were unrestrained with zero shear and zero moment for all analyses. The p-y curve curve used for all analyses performed for the site was the “API Sand (O’Neill, 1983)” curve.

Similar to the previous sites, the sliding depth and p-multipliers were varied until the predicted and measured moments compared closely. The first set of analyses were performed using p- and y- multipliers set equal to 1. A second set of analyses was performed using a p-multiplier equal to 1.8 with the y- multiplier of 1. Again, the analyses using a p-multiplier of 1.8 was performed to account for the effects of pile batter relative to the direction of soil movement as was the case for the I70-Emma site (Figure 4.1).

Analyses were performed for several different observation dates, generally corresponding to different magnitudes of soil movement. These dates include: April 13, 2002; July 17, 2002; September 7, 2002; and May 20, 2003. The soil properties input into LPILE® for the aforementioned dates are shown in Table 4.9.

Table 4.9: Properties input into LPILE® for the I435-Kansas City, Missouri site

		Effective Unit Weight (pcf)			Friction Angle	P-Y Modulus
Layer	Depth (ft)	4/13/02 & 7/17/02*	9/7/02	5/7/03	(degrees)	(lb/in3)
1	0-4	120	71.3	86.9	27	90
2	4-Beyond	84.6	84.8	91.2	29	90

*Pore pressures on these dates were equal; therefore the effective unit weights are equal.

4.3.1 Results for IM 1-4, Kansas City site

Instrumented members 1-4 were analyzed together due to their close proximity to the common inclinometers. The soil movement profiles for this pin were an average of the readings taken from inclinometer 2 and 3. The soil movement profile used for the aforementioned dates is illustrated in Figure 4.42. Numeric values for each of these movements are listed in Table 4.10. The movements utilized correspond to movements determined from inclinometer measurements at the depths specified. All movements below 4 feet were input as zero.

The predicted moments are compared to the corresponding measured moments for the respective measurement dates in Figures 4.43-4.46 for analyses performed using p-multipliers of 1.0 and 1.8. The measured moment magnitudes for the dates of April 13, 2002 and July 17, 2002 are relatively small, and have quite a bit of scatter. Therefore it is difficult to determine which predicted moment matches best. However, for the latter two dates, both predicted moments match the shape of the measured moments, except at a depth of 3 feet. The predicted moments determined using a p-multiplier of 1.8 match the magnitude of the measured moments best for the readings taken on September 7, 2002 and May 20, 2003.

Table 4.10: Soil movements input into LPILE® for members IM 1-4 at the Kansas City, Missouri site.

Date	4/13/2002	7/17/2002	9/7/2002	5/20/2003
Depth from Surface (ft)	Deflection (in.)	Deflection (in.)	Deflection (in.)	Deflection (in.)
0	0.25	1.33	1.66	1.79
4	0.06	0.20	0.34	0.36

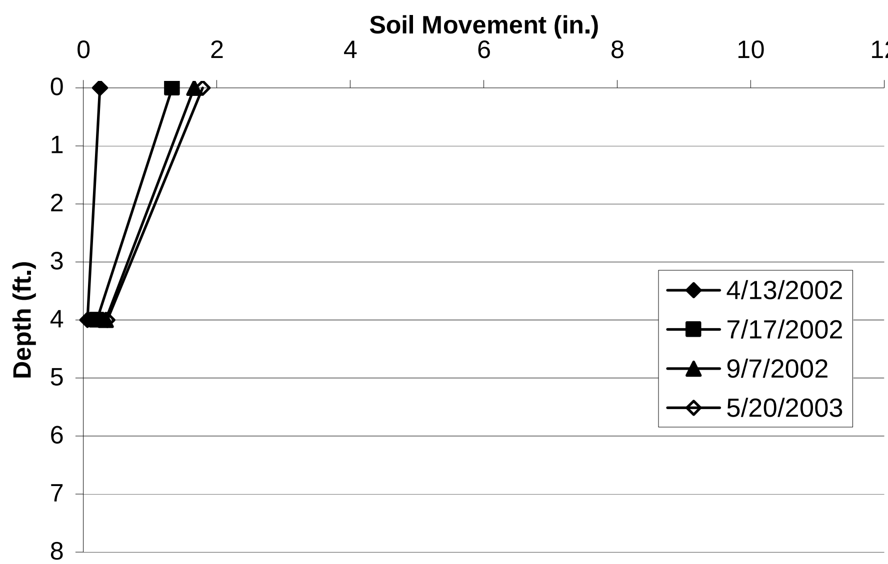


Figure 4.42: Soil movement profiles for members IM 1-4 at the Kansas City, Missouri site.

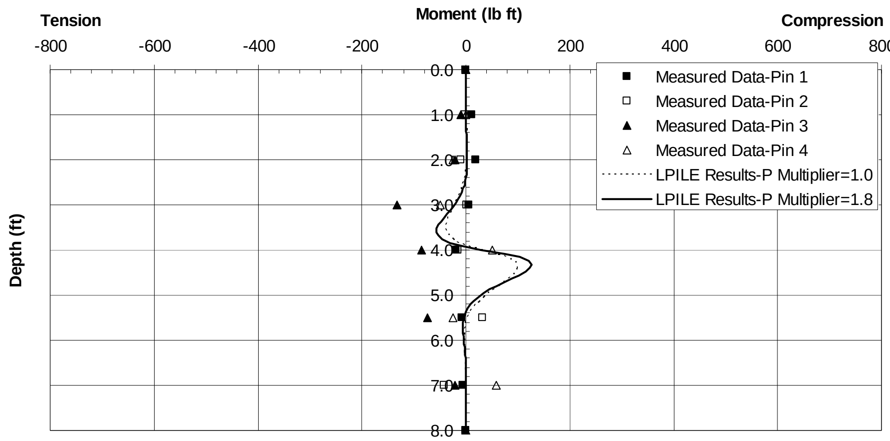


Figure 4.43: Comparison of measured and predicted bending moments for members IM 1-4 at Kansas City based on measured soil movements from April 13, 2002.

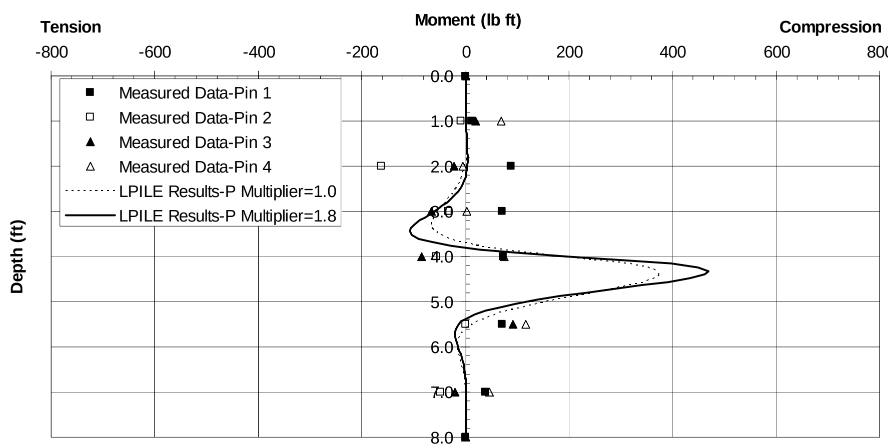


Figure 4.44: Comparison of measured and predicted bending moments for members IM 1-4 at Kansas City based on measured soil movements from July 13, 2002.

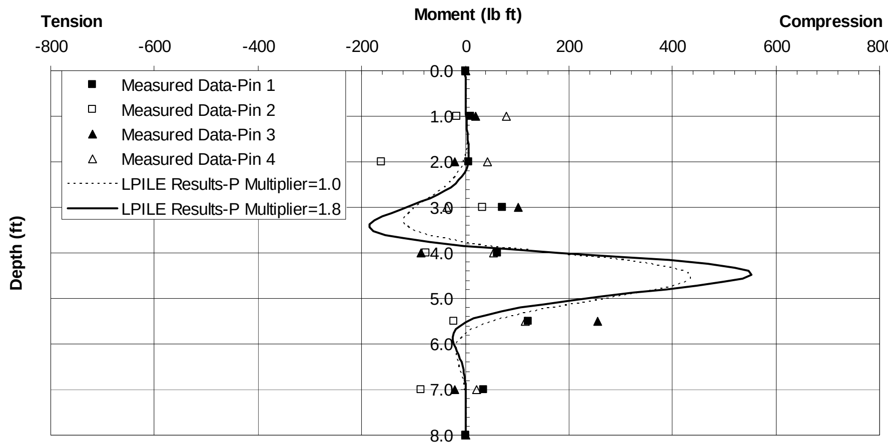


Figure 4.45: Comparison of measured and predicted bending moments for members IM 1-4 at Kansas City based on measured soil movements from September 7, 2002.

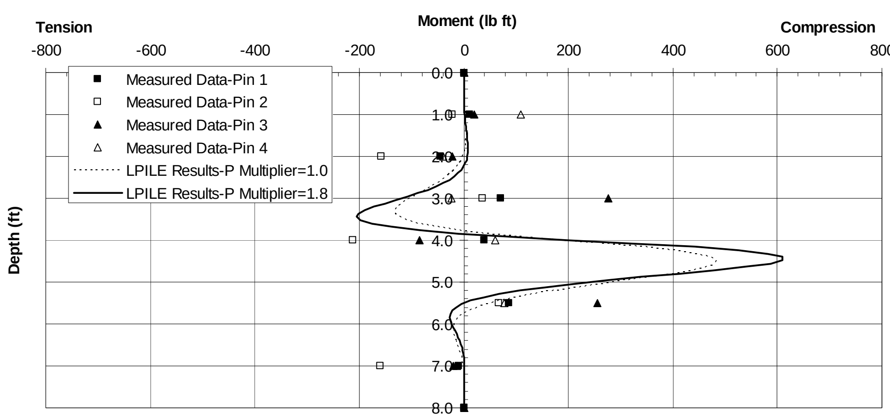


Figure 4.46: Comparison of measured and predicted bending moments for members IM 1-4 at Kansas City based on measured soil movements from May 20, 2003.

Figure 4.47 compares the maximum predicted moments for the respective measurement dates and model conditions as well as the maximum measured moments for those same dates, plotted as a function of average soil displacement. As expected, the predicted maximum moments consistently exceed the maximum measured moment for all soil movement values.

Considering the collective results of analyses performed for the instrumented members at the Kansas City test site, it appears that good comparisons among predicted and measured bending moments can be achieved using p-multipliers of 1.0 and 1.8. While the predicted distribution of bending moments are similar when using both of these values, the predicted distributions tend to match the measured values slightly better when 1.8 is used as the p-multiplier.

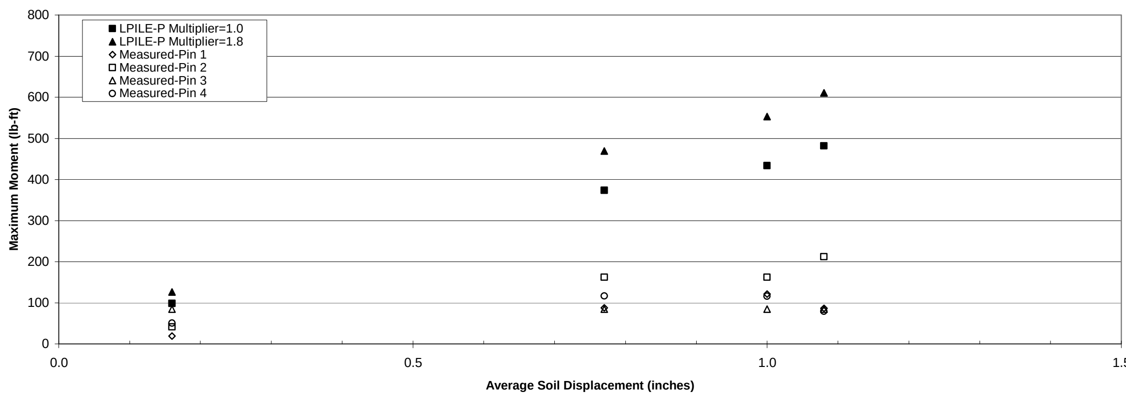


Figure 4.47: Comparison of maximum predicted moments and maximum measured moments for IM 1-4 at Kansas City.

4.4 Interpretation of Results

This section describes the p-y curve and p-multiplier that produce predicted bending moments that best compare with measured bending moments. Also provided is a comparison of the predicted location of maximum moments to the location of the measured moments for each instrumented member at each of the three test sites.

The predicted bending moments determined utilizing the API sand model with a p-multiplier of 1.8 provided the best comparison to the measured moments at the Emma test site. The predicted maximum moments for member IM 24 occurred at depths approximately 3 and 5 feet below the ground surface. The measured data also suggests a maximum moment at depths of 3 and 5 feet. The predicted maximum moments occurred at depths approximately 3.5 and 4.5 feet below the ground surface for member IM 19. The measured data supports the maximum moment at 3.5 feet. However, the gages located at depths of 2, 5.5 and 7 feet were disregarded for erroneous readings. Therefore, it is difficult to determine if the maximum moment at a depth of 4.5 feet compares with the measured maximum moment. The maximum predicted moment for members IM 18 and 23, occurred at depths of 3.5 and 4.5 feet below the ground surface. The maximum predicted moment located at a depth of 3.5 feet corresponds with the measured data. However, there is no indication of a maximum moments located at a depth of 4.5 feet. This is likely because strain gages were located at specific locations, which may or may not coincide with the location of the actual maximum moment. As illustrated in the figures presented in Section 4.1, the predicted moments using a p-multiplier equal to 1.8 compare well with the measured moments. These findings support the adjustment of p-multipliers based on pile batter angle.

The predicted bending moments determined utilizing the API sand model provided the best comparison to the measured moments at the Stewartsville test site. The predicted maximum moments for member IM 9 (steel pipe) occurred at depths consistent with the measured maximum moments. The measured data for member IM 9 is more inconsistent than what is seen in the plastic piles concerning the p-multiplier. The predicted moments determined with a p-multiplier equal to 8.0 compared the best with the measured moments for member IM 9. The predicted maximum moments for member IM 12 occurred approximately at depths of 3.5 and 4.5 feet below the ground surface. The measured data for member IM 12 indicated moments near zero for all depths. This could mean that strain gages were coincidentally placed at the depths where there were little moments measured, or that the measured soil deformations were small. Generally, the predicted moments reasonably compare with the measured moments. The predicted maximum moments for members IM 13 and 14 also occurred at a depth of 3.5 and 4.5 feet below the ground surface. The measured moments above a depth of 3 feet for member IM 14 do not compare well with the predicted moments. The measured moments for all depths of member IM 13 and the measured moments for member IM 14 for depths deeper than 3 feet compare reasonably well with the predicted moments. The figures presented in Section 4.2 illustrate that the predicted moments using both p-multipliers equal to 1.0 and 1.8 provide a good comparison for pins 12-14.

The predicted bending moments determined utilizing the API sand model with a p-multiplier of 1.8 provided the best comparison to the measured moments at the Kansas City test site. The predicted maximum moments for members IM 1-4 occurred approximately at depths of 3.5 and 4.5 feet below the ground surface. The maximum predicted moment located at depth of 3.5 feet does not agree well with the measured maximum moment. Considering the scatter in the data, the maximum predicted moment at 4.5 feet does fit with the measured maximum moment. The figures presented in Section 4.3 illustrate that the predicted moments using the p-multiplier of 1.8 compares well with the measured moments. These findings support the adjustment of p-multipliers based on pile batter angle.

4.5 Summary

Predicted moments determined utilizing the API sand curve and a p-multiplier equal to 1.8 compared the best with the measured moments in the recycled plastic reinforcing members at the Emma, Stewartsville, and Kansas City sites. However, the predicted moments determined with a p-multiplier of 8.0 compared the best with the measured moments in the steel pipe reinforcing member located at the Stewartsville site. Typically, the predicted maximum moments occurred at the same depth as the measured moments. The soil properties and soil movements used to predict the moments for each site were also presented. The moments predicted using different p-multipliers were compared to measured moments calculated from field instrumentation measurements for each of the aforementioned sites.

CHAPTER 5 – SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

A summary of the work performed for this thesis is provided in this chapter. The general characteristics of the three field test sites is provided along with a summary of results from analyses performed to compare the predicted response of reinforcing members with the response measured in the field. Several significant conclusions drawn from the results of these analyses are also provided along with recommendations for predicting reinforcement response for design of future slope stabilization measures.

5.1.1 I70-Emma Summary

This 22 foot tall slope, located 1 mile north of Emma, Missouri, is an embankment slope that forms the entrance ramp for eastbound I70 traffic at the intersection with Route VV. The side slopes inclinations range from 2.2:1 to 2.5:1 (H:V). The embankment is composed of lean and fat clays mixed with scattered gravel, cobbles, and construction rubble. The upper clay layer has an effective stress cohesion intercept of approximately 100 psf and an effective stress friction angle of 23 degrees. The lower clay layer has an effective stress cohesion intercept of 350 psf and an effective stress friction angle of 24 degrees.

The embankment was reinforced with recycled plastic reinforcing members placed in several different configurations. Reinforcement patterns included members on a 4.5 ft by 3.0 ft grid, a 4.5 ft by 6.0 ft grid, a 6.0 ft by 6.0 grid, and a 6.0 ft by 4.5 ft grid. All grids were “staggered,” meaning that adjacent rows of reinforcement were offset by one-half of the member spacing. Typical dimensions of the reinforcing members are 3.5 inches long, 3.5 inches wide, and 8 feet in length. This corresponds to a moment of inertia of 11.75 inches^4. The modulus of elasticity used was 145 ksi.

Strain gages were attached to several of the reinforcing members, which allowed the bending moment acting on the reinforcing to be measured. Slope inclinometers, piezometers and soil moisture sensors were also installed in the slope to allow monitoring of soil displacement and pore water pressure conditions over time.

Analyses were performed to compare the predicted member response using Ensoft's LPILE 5.0® to compare predicted and measured response according to several different models for several different members. The soil movement profiles utilized for these analyses were established from inclinometer measurements taken at the site. Pile head boundary conditions were unrestrained with zero shear and zero moment for all analyses. The p-y curve used for the analyses performed for the Emma site was the "API Sand (O'Neill, 1983)" curve. The p-multipliers were varied until the predicted moment magnitude compared with the measured moment magnitude. A p-multiplier of 1.8 was chosen particularly based on pile batter angle. A p-multiplier of 1.0 was used to compensate for the effects of pile spacing. Although comparisons were obtained from predicted moments determined using both p-multipliers equal to 1.0 and 1.8, the magnitude of moments predicted using a p-multiplier equal to 1.8 matched the magnitude of the measured moments the best.

5.1.2 US 36-Stewartsville Summary

The US36-Stewartsville site is located in northwest Missouri on U.S. Highway 36, approximately two miles west of the city of Stewartsville and lies in the median of US36 between the eastbound and westbound roadway. The excavated slope is approximately 29 ft high with a nominal inclination of 2.2:1 (H:V). The slope is composed of a surficial layer of soft to medium clay overlying stiff to hard fat clay. The upper clay layer had an effective stress cohesion intercept of 0 psf and an effective stress friction angle of 29 degrees. The lower clay layer nominally had an effective stress cohesion intercept of 100 psf and an effective stress friction angle of 35 degrees.

Reinforcing patterns included a 4.5 ft by 3.0 ft grid, a 6.0 ft by 6.0 ft grid, a 6.0 ft by 4.5 ft grid, and a 4.5 ft by 6.0 ft grid. Similar to the Emma site, the grid patterns were also staggered. The plastic reinforcing members for the Stewartsville site typically have the same physical properties as the Emma site and several of the reinforcing members were instrumented with strain gages. One of the instrumented members was a 3.5 inch diameter steel pipe. Slope inclinometers, piezometers and soil moisture sensors were also installed in the slope.

Analyses similar to those performed for the I70-Emma site were performed for the instrumented reinforcing members at the US36-Stewartsville site. The p-y curve used for the analyses was the “API Sand (O’Neill, 1983)” curve. The p-multipliers considered for this site were 1.0, 1.8, and 8.0. A p-multiplier of 1.8 was chosen particularly based on pile batter angle. p-multipliers of 1.0 and 8.0 were selected to represent an upper and lower bound trend. Predicted moments determined using both p-multipliers equal to 1.8 and 8.0 compared the best with the measured moments for the steel pipe. Moments predicted using a p-multiplier equal to 8.0 matched the measured moments for the steel pipe (IM9). Moments predicted using both p-multipliers equal to 1.0 and 1.8 match the measured moments for the plastic members because the measured moments were all small in magnitude.

5.1.3 I435-Wornall Road-Kansas City Summary

The I435-Wornall Road site is located in the northeast quadrant of the intersection of I435 and Wornall Road between I435 and the westbound exit ramp, in Kansas City, Missouri. The embankment is approximately 32 ft high with a slope inclination of 2.2:1 (H:V) slope. The slope consists of a 3-5 ft thick surficial layer of lean to fat clay with soft to medium consistency overlying stiffer compacted clay shale. The upper clay layer has an effective stress cohesion intercept of 0 psf and an effective stress friction angle of 27 degrees. The lower clay layer nominally has an effective stress cohesion intercept of 0 psf and an effective stress friction angle of 29 degrees.

The reinforcement pattern at the I435-Wornall Road site consisted of a 3.0 ft by 3.0 ft staggered grid was used over the entire area where the previous slide had occurred. The plastic reinforcing members for the Kansas City site have the same physical properties as the Emma site. Similar to the previous two sites, slope inclinometers, piezometers and soil moisture sensors were also installed in the slope.

Analyses to compare the predicted and measure reinforcement response were also conducted for the I435-Wornall Road site. The p-y curve used for the analyses was again the “API Sand (O’Neill, 1983)” curve. The p-multipliers considered for this site were 1.0 and 1.8. Moments predicted using both p-multipliers compared well with the measured moments. However, predicted moments using a p-multiplier set to 1.8 compared the best with the measured moment magnitudes for all dates.

5.2 Conclusions

Comparisons of predicted moments determined from numerical models to measured moments determined from several field test sites provide information that was used to establish conclusions and recommendations for design of slopes reinforced with recycled plastic members. The following conclusions can be drawn from this work.

1. Moments using the p-y method, as implemented in L-Pile®, generally compare well with the moments measured from field instrumentation for all of the instrumented members considered in this study. This conclusion suggests that the p-y method is an appropriate means for predicting the response of reinforcing members for design purposes.
2. The API Sand curve by O'Neill (1983) produced the best comparison between the measured and predicted response of the reinforcing members for all of the instrumented members considered. This suggests that this model is appropriate for predicting the response of reinforcing members subjected to long term, drained loading in spite of the fact that the soils present at each of the sites were clayey soils.
3. Results of the analyses presented in this thesis show that matches between measured and predicted performance could be achieved using several values for the p-multiplier. The "back-analysis" methods utilized in this work are inherently non-unique so this result is not surprising.
4. Despite the fact that the results of the analyses performed are non-unique, use of a p-multiplier of 1.8 tended to consistently produce good matches of measured and predicted response when considering the results of all analyses collectively. Since this value is also consistent with results from other research regarding the effects of pile batter (Bozok, 2009), it follows that p-multipliers appropriate for the relative pile batter present should be used.

5.3 Recommendations for Design of Recycled Plastic Reinforcement for Slope Stabilization

The “API Sand (O’Neill, 1983)” curve should be used when modeling reinforcement for long-term, drained loading conditions, regardless of the type of soil present. This model should be used with a p-multiplier selected based on the relative pile batter angle and pile spacing. The soil movement profile should be input as anticipated soil movements down to the sliding depth, and then zero below the sliding depth. Utilizing these criteria in predictive modeling software will predict bending moments that are similar to the bending moments that will be imposed on the reinforcing members in the field.

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