Landslide Stabilization Along The Ohio River Using Cantilevered Stub Piers

Swaminathan Srinivasan
Terracon Consultants, Inc., Cincinnati, OH

Jess A. Schroeder
Terracon Consultants, Inc., Cincinnati, OH

LANDSLIDE STABILIZATION ALONG THE OHIO RIVER
USING CANTILEVERED STUB PIERS

Swaminathan Srinivasan, P.E., M. ASCE
Terracon Consultants, Inc.
611 Lunken Park Drive
Cincinnati, OH 45226

Jess A. Schroeder, P.E., M. ASCE
Terracon Consultants, Inc.
611 Lunken Park Drive
Cincinnati, OH 45226

ABSTRACT

Landslide activity along U.S. 50 in Cincinnati, Ohio has caused roadway damage for decades. After a necessary closure of 3 lanes due to slope movements, emergency stabilization measures were undertaken to protect the roadway by providing a "pseudo" short-term solution (target 3 to 5 years) necessitated by ODOT budget constraints.

The landslide shear plane was near the top of a sloping bedrock surface as much as 50 feet below grade. "Stub Piers" were installed 40 feet downslope of the roadway shoulder. The shafts were heavily reinforced across the deep shear plane but steel reinforcing did not extend the full length of the shafts and was stopped well short of the ground surface. The goal was to provide shear resistance across the failure plane, forcing the theoretical failure surface higher into the overburden soil profile, resulting in a comparatively higher safety factor against slope failure. These "Stub Piers" were installed and found to meet all of the project goals.

The stub piers and surrounding ground were instrumented and analyses of collected data to date showed earth pressures and horizontal deflections were over-predicted in the original design.   Instrumentation by means of inclinometers, vibrating wire earth pressure cells, and strain gages has been monitored over a period of several years since construction of the Stub Piers and results indicate this option offers an attractive alternative to conventional drilled piers or tieback drilled pier solutions.

INTRODUCTION

Landslide activity has occurred along U.S. Rt. 50 in western Cincinnati, Ohio for many decades. The site is located between North Bend and Addyston, OH, on the right descending (cutting) bank of the Ohio River, at about river mile 485. The landslide activity along this area has been on-going for many years. Slope and road movements have required periodic repairs over recent decades. Railroad tracks located downslope of the roadway also showed signs of horizontal displacement and periodic repair. Visual evidence suggested the shear plane extended below the roadway at deep levels and out into the Ohio River.

In brief review, the road elevation at the time of the geotechnical study was at about 508 to 516

ft., increasing in an east-northeast direction. A weed and brush-covered slope extended southwest and downward toward the Ohio River at about 3H:1V. The slope rose more than 100 feet above the roadway. On the downhill side of U.S. 50, grade sloped down about 15 to 20 feet in elevation to a railroad right-of-way at about elevation 490 ft. The riverbank then sloped down at about 2.5H to 3H:1V to the water's edge. Normal pool elevation of the Ohio River is 455 ft.

In 2005, Terracon was retained by the Ohio Department of Transportation (ODOT) to perform a geotechnical study that included 17 test borings and inclinometer monitoring at 4 locations.

After only a few weeks of monitoring, the inclinometer casings sheared off about 50 feet below grade, near the soil /

Fig. 1: Pre-repair road distress (2005).

bedrock interface (see Figure 1). Soon after, the roadway distress worsened, causing ODOT to close 3 of the 4 lanes to

traffic and reroute traffic onto the remaining lane and shoulder

Fig. 2: Road distress and lane closure (2005).

(Figure 2). Terracon was asked to develop a stabilization design under emergency repair conditions. However, funds were limited at the time, necessitating a direction by ODOT that the solution be at least "pseudo" short-term (3 to 5 years).

The on-going landslide displayed deep-seated movement extending down to the top of bedrock, about 40 to 50 feet below present grade. The toe of the slide most likely extended out into the Ohio River.

The use of a toe berm or MSE-type retaining wall was not considered practical or feasible for remediation due to the ODOT right-of-way limitations and also because such a repair would add unwanted load and driving forces to the landslide. Such a load could possibly accelerate slope movements.

The use of a "soil nail launcher" was also discussed with ODOT. This method of remediation was not considered feasible either. The slide plane extends to bedrock and the soil nails installed by this launching technique would not extend deep enough nor provide the level of shear and passive restraint needed.

The most appropriate and effective long-term remedial measure appeared to be the construction of a soldier pile or drilled pier wall containing multiple rows of tieback anchors. The anchor installation would likely involve substantial excavation for equipment access to install multiple tiers of tieback anchors. While effective, this method would involve significant cost. After discussions with ODOT, it was our understanding that a sufficient budget was not currently available for "permanent" repair. Instead, ODOT requested a recommendation from Terracon for a "temporary" repair. The primary goal was to allow U.S. 50 to be reopened and maintained open for some period of time (3 to 5 years). This period of time would allow for budget and plans to proceed with a more permanent solution.

Due to the significant depth to bedrock and the deep shear plane, the use of "stub piers" was proposed by Terracon as the

"pseudo-temporary" repair. A series of heavily-reinforced drilled piers were designed and constructed. The pier reinforcement was somewhat unique when considering more standard practice in the Cincinnati local area. Details are presented in the following paragraphs, as well as instrumentation results.

GEOLOGIC SETTING

The overburden profile consists of cohesive embankment fill, alluvium, colluvium, and residuum. Fill ranges from 10 to 25 feet deep and is underlain by alluvium that is interbedded and sometimes lying atop colluvium. Colluvial clays are formed by action of gravity and have slickensides with random orientation. Residuum is also present in some areas at a thickness of about 3 feet. Residuum is a soil formed from the n-place weathering of the underlying parent bedrock.

Bedrock lies between 31 and 50 feet deep. Typically, gray shale and limestone occurs. However, about 3 feet of brown weathered shale with limestone occurs in some locations above the gray shale. The horizontally-bedded shale and limestone belongs to the Kope Formation (Ordovician System) and includes shale that rates as very soft to soft in terms of bedrock hardness. There are numerous documented landslides in this local geologic setting. Shale comprises about 90% of the Kope's mass. Very hard limestone makes up the remainder, occurring in layers up to about 1.5 inches thick. Figure 3 provides a general subsurface profile illustration.
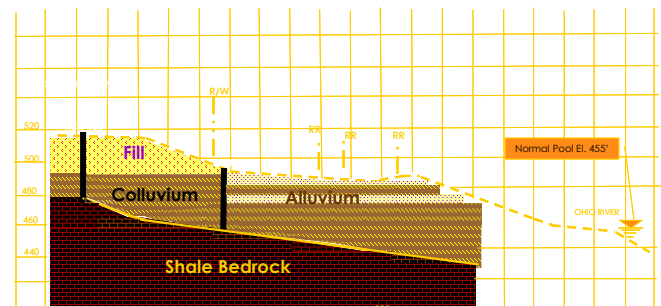

Fig. 3: Typical subsurface profile.

The Ohio River in this area has a normal pool elevation of 455 feet and official flood elevation of 485 feet. The 100-year flood elevation is 501 feet while the highest recorded river level in Cincinnati occurred during the 1937 flood at elevation 512 feet. With the U.S. 50 roadway elevation at 508 to 516 feet and the railroad at 490 feet, at least the lower portions of this slope are subject to periodic flooding and river drawdown conditions. These conditions worsen the overall slope instability.

STUB PIER DESIGN APPROACH

The assumed repair method included a row of straight-sided drilled piers socketed into bedrock. Due to the thickness of overburden, a tieback anchor system would be required to

support these shafts as a more permanent solution. However, as directed by ODOT, the primary goal here was to develop a temporary repair scheme within a limited budget. Therefore, it was assumed that the reinforced concrete piers only extend part of the way upward through the overburden soils. These "stub piers" were assumed to be closely spaced where soil arching could be assumed to make the piers behave as a continuous wall. The piers would therefore force a theoretical shear plane upward from the bedrock surface to above the pier butt (steel) elevation.

The selected design consisted of a single row of cantilevered drilled shafts located within the right-of-way about 40 feet downslope of the roadway shoulder. The shafts would be socketed into bedrock. The innovative and cost-effective aspect of this scheme involved the steel-reinforcing length. Only the zone near the deep shear plane would be heavily reinforced, thus creating shear pin-type support across the deep shear plane. The structural steel would be terminated as much as 35 feet short of the ground surface.

From an analytical point, the short-term solution criterion was quantified by slope stability analyses. Laboratory tests were conducted and soil parameters were then adjusted slightly for the failed slope condition (safety factor of 1.0) and observed shear plane depths. Then, the shear plane was forced upward to the planned top-of-steel elevation of the stub piers. This process resulted in a theoretical safety factor increase from the original 1.0 to about 1.2 (see Figure 4). ODOT agreed with this potential improvement, as a short-term solution.
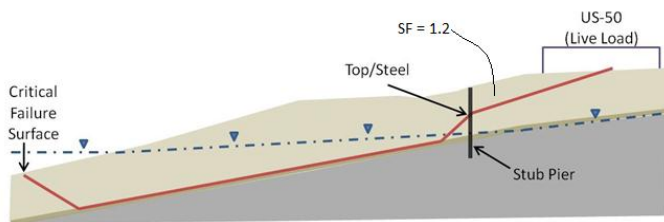

Fig. 4: Slope stability schematic.

Stub pier design details were then developed. The lateral earth pressure was estimated assuming triangular earth pressure distribution from the ground level to the shear plane. This resulted in a trapezoidal-shaped earth pressure diagram acting on the piers. For potential arching effects above the steel, it was assumed that the contributing pressure extended to one pier diameter above the top-of-steel. This estimated earth pressure was also checked using slope stability analysis to compute the resisting pressure required to generate a theoretical safety factor of 1.2. Refer to Figure 5 for schematics of the assumed earth pressure diagram.
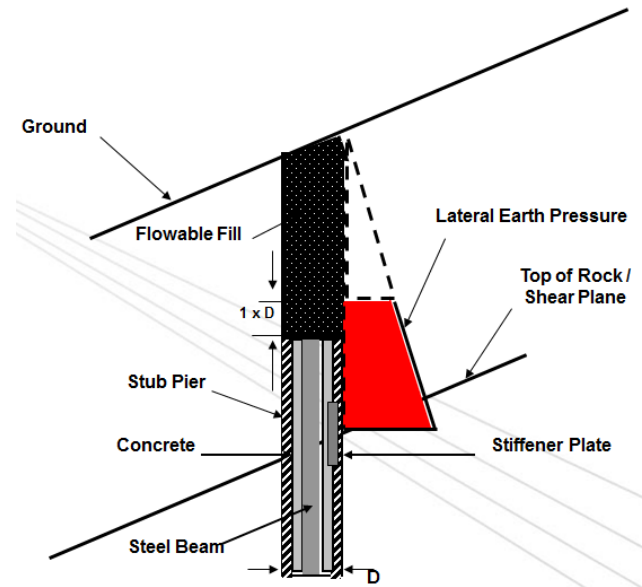

Fig. 5: Stub pier schematic.

Stub pier design was developed using the LPILE computer program. The drilled shafts included 30 and 36-inch diameter units and were socketed 10 to 15-ft. into gray unweathered shale bedrock. The steel reinforcement within the drilled shafts consisted of rolled steel sections that included HP14X73, W18X119, and W24X117. In some cases, additional bending resistance was necessary and developed by welding a steel plate to the uphill face of the beam. The steel extended to the bottom of the hole; however, it was limited in length and only extended about 20-ft. above the top-of-rock. Therefore, steel beam lengths ranged from 30 to 35-ft. and stopped well short of the ground surface. The top-of-steel was essentially determined to be the top-of-shaft, thereby assuming that slope shear failure could occur at the top-of-steel. The shaft opening above the steel was backfilled with either unreinforced structural concrete or a lean concrete fill, as determined by ODOT and contractor in the field.

Due to the limited height of the reinforced section of these shafts (with their tops occurring well below grade), they were essentially deemed to act as shear pins installed across the deep failure plane. For the presentation purposes these shafts have been termed "Stub Piers."

CONSTRUCTION

The 154 Stub Piers were installed from July to September 2005 under an emergency repair contract. The roadway was repaved on October 6 and 7, 2005, adding upwards of 2 feet of new asphalt in some areas to relevel the road. Traffic was reopened on October 7, 2005.

ODOT indicated the cost for stub pier installation was about $500,000.00 (in 2005 dollars). This cost included drilling, reinforcing, and backfilling 154 stub piers. As-built quantities

included 8386 feet of shaft drilling, 1485 cu. yds. of concrete backfill, 553 cu. yds. of flowable fill backfill, and 273 tons of structural steel beams plus stiffening plates.

INSTRUMENTATION

A limited instrumentation program was implemented to monitor slope movements, verify that the stub piers were meeting design goals, and to help confirm design assumptions. This program began shortly after construction was underway. Locations for instrumentation devices were selected for their critical locations, as well as to coordinate with the contractor's activities and schedule.

The instrumentation program consisted of the following:

1. Five Inclinometers installed within selected Stub Piers.

2. Four Inclinometers installed upslope of selected Stub Piers.

3. Two Inclinometers installed about 10 feet downslope of selected Stub Piers.

    An inclinometer consists of a grooved PVC pipe that is socketed into bedrock or another fixed reference. Readings are taken by lowering the inclinometer probe down the pipe to obtain a profile of the horizontal displacement from its original position.

4. Three Push-In Earth Pressure Cells (Geokon Model 4830; see Figure 6) were installed within boreholes located about 8 to 10 feet upslope of selected Stub Piers. These devices were located about 40 to 45 ft. below grade and were installed with the intent of being just above the bedrock surface (close to the interpreted shear plane). These devices measure total horizontal pressure in the soil.

Fig. 6: Push-in earth pressure cell.

5. At two piers, six vibrating wire strain gages were installed per pier (four on the tension side and two on the compression side). The strain gages (Geocon Model 4000 Strain Gages, weldable mounting blocks, plucking coil and thermistor) were welded directly to the steel beam; see

Figure 7. A thermistor is integrated into the strain gages to account for temperature induced strain. Individual pieces of angle iron were welded over the strain gages to prevent damage during concrete placement.

Fig. 7: Strain gage.

The strain gage cables were extended up the two respective Stub Piers to the ground surface. These cables, as well as the earth pressure cell cables, were routed laterally to a terminal box, which was installed on a post embedded within the top of a nearby Stub Pier. Figures 8 and 9 show the cables, protective steel angle iron over the strain gages, and fully instrumented pile before installation. Figure 10 shows installation of an instrumented steel beam.
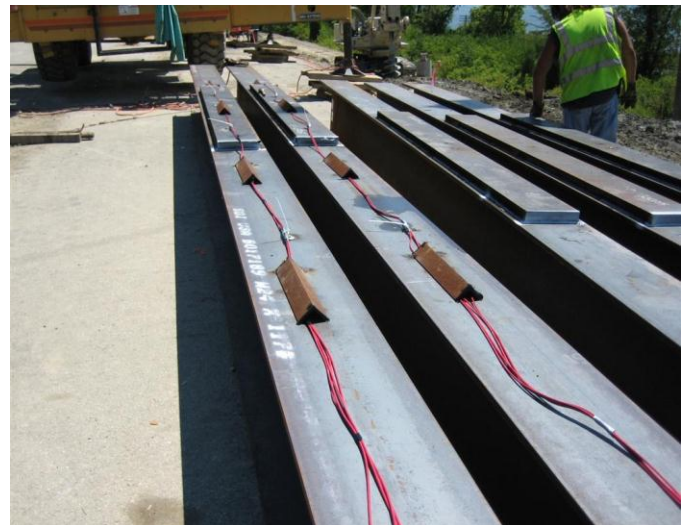

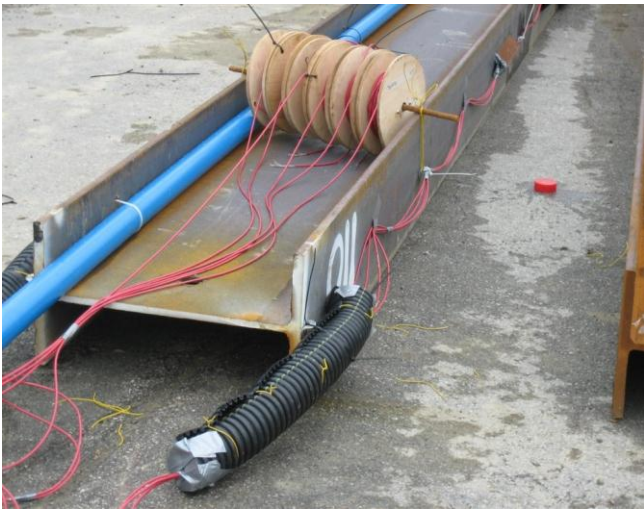*Fig. 8: Strain gage and cable assembly.*

Fig. 9: Pile instrumented with strain gages, cables, and inclinometer casing.

INSTUMENTATION DATA REVIEW

Strain gage and earth pressure devices were monitored over a period of six years before the cables were vandalized. Inclinometers have been monitored over a period of seven years.

Comparisons were made between the maximum bending moments and average earth pressures between original theoretical design analyses and those estimated from measured strain gage data.

Strain gages installed at two Stub Piers allowed the conversion of measured or "apparent" strain to bending strain by subtracting the calculated compressive strain due to the weight of the pier above (carried by steel and concrete) from the measured apparent strain. The bending stress and bending moment were then computed from the bending strain value at each strain gage location. The computed bending moments based on these measured strains were only 25 percent of the values generated by the original LPILE analysis. Additionally, the strain gage data generated bending moments significantly higher on the tension side than the compression side of the steel. One potential explanation could be that the concrete contribution in resisting bending is neglected in the analysis.

Fig. 10: Setting instrumented steel beam into shaft excavation.

One inconsistency in the strain gage data occurs when earth pressures are back-calculated from the computed bending moments. These earth pressures are a fraction of those generated by earth pressure theory and are also well below those measured in the three earth pressure cells. There is no clear explanation for these results.

The earth pressure cells were installed at relatively close spacing and similar depths. We suspect two of the devices may have rotated before being seated at the bottom of the borehole where the sensors may not have been perpendicular to the slope forces. The maximum measured value of the three devices compared closely to the assumed earth pressure.

Inclinometer data clearly shows the deep-seated shear plane has been successfully cut off by the Stub Piers. Figure 11 shows a typical inclinometer before construction. The deep shear plane is clearly evident.
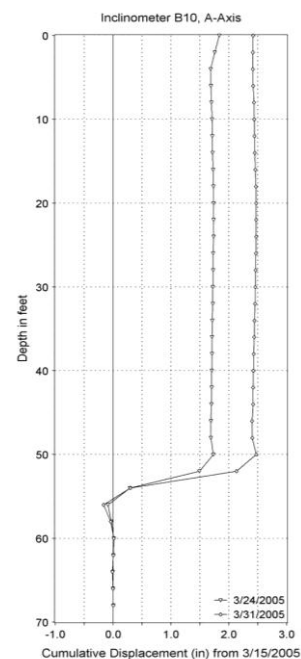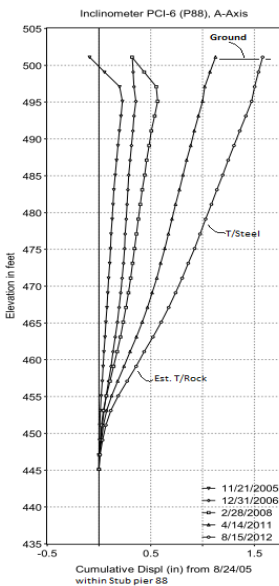

*Fig. 11: Inclinometer data before slope repair (2005).*

Figure 12 shows a typical inclinometer installed within a Stub Pier and monitored over seven years.

Ohio Department of Transportation
Hamilton-50-6.50 Landslide
Addyston, Hamilton County, Ohio

Fig. 12: Inclinometer data installed within a stub pier and monitored over seven years.

Figure 13 shows an inclinometer installed during construction and located just upslope of the Stub Pier referenced in Figure 12. As shown, slope movements have been slowed considerably. Slight continuing creep movements are evident.
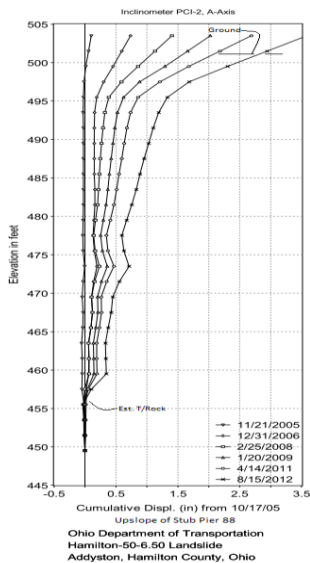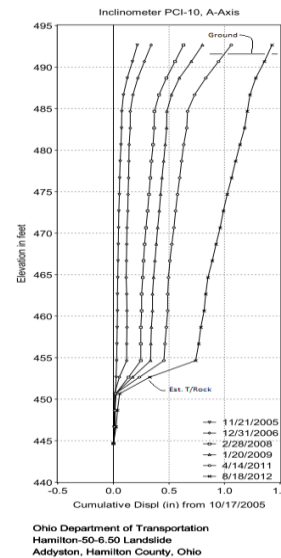

Ohio Department of Transportation
Hamilton-50-6.50 Landslide
Addyston, Hamilton County, Ohio

Fig. 13: Inclinometer data installed during construction and located upslope of stub pier.

Figure 14 shows an inclinometer located just downslope of the Stub Pier referenced in Figure 12. As shown, creep movements along the original soil/bedrock shear plane have

continued since construction, but at a much lesser degree than pre-repair landslide conditions.

Ohio Department of Transportation
Hamilton-50-6.50 Landslide
Addyston, Hamilton County, Ohio

Fig. 14: Inclinometer located downslope of the stub pier.

The original Stub Pier design was based upon triangular earth pressure distribution from the ground surface. Also recall that vertical soil arching was assumed which added applied lateral pressure to a height of one pier diameter above the top-of-steel. LPILE analyses were conducted to determine the required pier size and steel reinforcement during design.

While there are some inconsistencies in the back-calculated bending moments and earth pressures, the overall monitoring program results suggest that the Stub Pier approach achieved the goal of creating short-term stabilization of the roadway embankment and may in fact provide much longer-term stabilization of this slope.

2012 SLOPE CONDITIONS (after seven years)

As referenced in Figures 12 through 14, new inclinometer readings were taken in August 2012, now reaching nearly 7 years after Stub Pier construction to provide a "pseudo-temporary" repair of the landslide. Earth pressure cells and strain gages could not be monitored as all cables have now been stolen / vandalized.

The hillside in August 2012 is heavily vegetated and difficult to see. However, sloughing just below the guard rail continues to be evident (as it was 7 years ago and deemed to be caused by poor backfilling of the upper bench of fill; i.e. not deep-seated).

A small sink hole observed in 2005 has reopened in the existing roadway (see Fig. 15). This feature was deemed to be caused by a leaking sewer. The sewer was not repaired during the 2005 construction and only the pavement hole had been sealed with concrete at the time.

Fig. 15: Sinkhole over leaking sewer.

Overall, the pavement appears to be in good condition seven years after construction. Some crack sealing is evident which is most likely the result of post-construction residual creep. Figures 16 through 18 compare conditions on the downhill edge of the road over a 7-year period.

*Fig. 16: Road damage in 2005
(pre-repair).*

Fig. 17: 2005, after Stub Pier construction.

Fig. 18: August 2012 conditions.

The 2012 inclinometer readings generally show about 1 to 2.5" of horizontal movement at a depth of 5 to 7 feet below ground surface near the top of the slope at a location just down slope of the guard rail. At the ground surface, these movements are more on the order of 1.5 to over 3 inches. This movement is apparently due to the poorly compacted wedge of backfill placed in the 2005 temporary access bench.

At the Stub Pier locations, lateral movements have been virtually stopped at the soil/bedrock interface, or original shear plane. One of the five instrumented Stub Piers shows 1.8-inches of movement at the top-of-steel, whereas the remaining four Stub Piers show are less than an inch of movement at the top-of-steel.

As one might expect, there are continuing creep movements within the deep soil profile at unsupported locations upslope and downslope of the Stub Piers. Inclinometers located about 10 feet upslope of the Stub Piers show small movements, but inclinometers 20 or more feet upslope of the Stub Piers show a greater amount of continuing movements, now on the order of two-inches at a depth of about five feet below grade. At the soil/bedrock interface, these uppermost inclinometers have shown about 0.5 to 1.4 inches of movement near the soil/bedrock interface, indicating continuing creep along the original failure plane. However, the greatest degree of movement at the failure plane over the past seven years is at a rate far less than when the 2005 landslide occurred, by a factor of 100 to 600 times slower.

Results are similar for the two inclinometers installed just downslope of the Stub Piers. Movements at the soil/bedrock shear plane range from about ¼ to ½-inch, but have shown continuing creep since the 2005 repair.

The original LPILE calculations have since been modified in an attempt to match field-observed horizontal deflections. For the model case, the deflection target was 0.8 inches at the top-of-steel (reduced from 4 inches, as originally predicted). The modified analysis required elimination of the vertical soil

arching effects above the steel. Soil shear strength in the overburden was also found to be slightly conservative in the original analyses.

Another more detailed approach to recreate field-measured conditions in the LPILE analysis would be to regenerate p-y curves using inclinometer data. That exercise has not been attempted here.

Moving further downslope from the Stub Piers, there are two inclinometers. Each of these continue to show creep movements at the original shear plain (near top-of-bedrock); however, the maximum deflection measured at the ground surface is on the order of 0.75 to 1.3 inches and well below pre-repair slope movements.

Comparing measured lateral displacements with time, it is evident that movements have continued steadily since the 2005 construction. Accelerated movements have also been evident during certain periods that have coincided with heavier than normal rain fall. For example, annual recorded Cincinnati precipitation varied from about 39 to 45 inches during the interim of 2005 to 2010. However, in 2011, annual precipitation increased to 75 inches. These values are based on published information and assumed snowfall equaling 10% rain.

In some cases, recorded inclinometer movements showed about half of the total occurred between 2005 and April 2011 (about 5.5 years) and the remaining half occurred within the monitoring period of April 2011 to August 2012. Figure 19 shows a typical rate of lateral deflection at the top-of-steel.
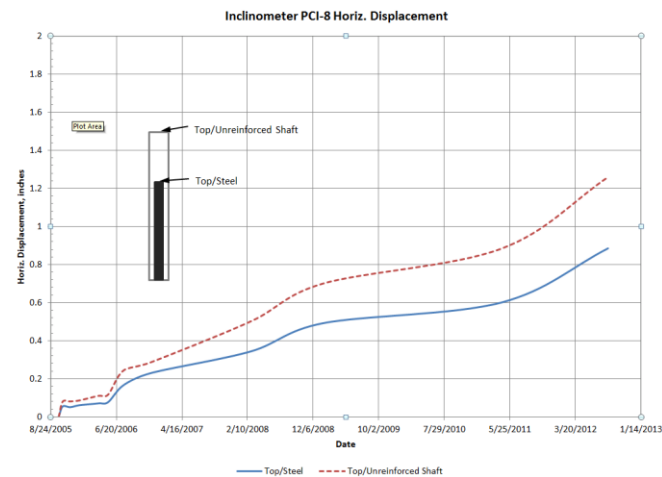

Fig. 19: Typical rate of lateral deflection at the top-of-steel.

LESSONS LEARNED AND CONCLUSIONS

❑ Stub Pier approach works for deep shear planes.
❑ Not suitable for all settings, as shallow landslide potential after construction must be quantified.
❑ Quantifying shallow landslide potential (by slope stability analysis) appears to be a valid basis for evaluating "longevity" of the system.

❑ Potential for significant cost savings and quick installation.
❑ Stub Pier installation with minimal specialty materials or equipment.
❑ Original design assumptions for active lateral earth pressure were conservative. For example, the original prediction for lateral displacement at the top-of-steel section was 4 inches. Measured values after 7 years are less than 1 inch. Assumed vertical soil arching effects for active earth pressure above the steel-reinforced zone do not appear to be necessary.
❑ In both cases (original and recent LPILE analyses), passive resistance on the downhill side of the Stub Piers between bedrock and the top-of-steel was included. The LPILE program computed this resistance using input soil properties.

The owner (ODOT) realized a successful repair solution because the repair was designed and constructed quickly, where the 154 stub piers were installed and the roadway repaved in under 3 months. The costs were significantly less than the alternative of a tieback-anchored drilled pier arrangement. A tieback approach would likely have involved excavating and installing multiple rows of tiebacks due to the deep shear plane (up to 56 feet deep). Excavation materials would have had to be removed from the site to avoid stockpile loads, only to be returned later for burying the deeper tiebacks. A much longer construction period would have been required at significant inconvenience to roadway users. A tieback anchor and drilled pier approach cost was estimated to be about 3 to 4 times the cost of the constructed stub pier approach.

Finally, the stub pier approach at this site appears to be functioning well after seven years and may provide many more years of support. Therefore, the original goal of providing a "short-term" solution appears to have been met and exceeded.