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ABSTRACT

A case history is presented for slope stabilization of channels using piles. The Kansas City District, Corps of Engineers has used various pile designs and types of piles for projects along the Blue River Channel in Kansas City Missouri to stabilize sections of unstable channel slopes in previous slide areas, since June of 1986. Low existing soil shear strengths and limited rights-of-way precluded use of more conventional stabilizing methods. Although a pile analysis method had been used for initial projects, with several piles instrumented with slope indicator devices, an improved analysis method was needed. In July of 1994, through a design analysis, the Kansas City District, Corps of Engineers developed a pile design procedure for the stabilization of a failed slope, along the Blue River Channel, known as the Gregory Blvd Project. The method utilizes both Limit Equilibrium slope stability methods and P-Y curve methods for pile analysis, and takes into account earth pressure theory. The method includes a trial and error procedure for determining the driving forces from a sliding mass of soil. The driving forces were then calculated at a concentrated point on the slide plane, coincident with the location of the piles, or where the piles intersect the slide surface, using limit equilibrium procedures with the most critical slide surface and at the desired factor of safety for the stabilized slope. For projects discussed, shear strengths, were determined from a back analysis of the existing slopes at failure using F=1.0. Laboratory test results generally did not yield usable values. The slope stability computer programs UTEXAS2 and UTEXAS3, using Spencer's Procedure was used in all analyses. The driving forces, thus determined, are then input into the pile analysis (LPILE) program, using both a triangular and a uniform load distribution along with the appropriate soil strength parameters for generating the P-Y curves within the program. Depth of pile embedment below the slip surface, size of the piles, position of the piles in the slope and the required spacing are then determined. Actual instrumented field data located on piles are presented as well as comparisons with predicted results, using the pile procedure analysis technique. The case studies demonstrate the importance of proper formulation of the analysis and of modeling the soil using the correct P-Y soil parameters for the pile.

KEYWORDS

Unstable slopes, Slope stability analyses, Shear strengths, Driving forces, Piles, Design procedure

INTRODUCTION

The Kansas City District, Corps of Engineers became interested in slope stabilization using piles as a result of slope stability difficulties on one of our large civil works projects, the Blue River Channel. This channel improvement project has an estimated completion cost of $200,000,000 and is located in an industrial area on Kansas City's east side. The project calls for channel improvements on about 12.5 miles of the lower Blue River Channel beginning at its confluence with the Missouri River. The channel improvements include channel widening and deepening to increase flow capacity and provide a 30 year level of flood protection for the industrial district. With commercial development that began in the area in the early 1900's, land owners adjacent to the river placed landfill immediately adjacent to the river bank to provide some measure of flood reduction. The fill included all manner of construction debris and waste ranging up to large, high density steel slag "boulders", wood, bricks, steel balls, etc. As a result many of the river banks have failed in the past and remained in failed or marginally stable configurations. Project construction began in 1983 with an initial contract on a short reach of the river, however it was not until the second contract that the true nature of the channel bank stability was encountered. The District attempted to construct designed rockfill toe sections extending below the channel flowline, using a technique of limited excavation followed by immediate replacement with rockfill. Slope failures continued to occur even with open excavation lengths limited to less than 50 feet. With extremely limited right-of-way, slopes could not be flattened further. Piles were required to be introduced as a means to stabilize the channel slopes that had undergone slide movements in the natural state many times prior to construction. The Kansas City District had to have a design quickly, because a contractor was being delayed of completing construction of a Contract and the channel bank slides were encroaching on buildings and existing railroads. The development of a practical analysis method to determine optimum design and placement is discussed below.

PREVIOUS ANALYSIS METHODS

Early analysis attempts, used the Kansas City District force equilibrium slope stability program, known as "KC Wedge", and with graphical hand solutions, using the modified Swedish circle and non-circular Block and Wedge procedures, in accordance with the Corps EM-1110-2-1902, dated 1970. The earlier analysis employed only adding a cohesion value for the shear strength resistance provided by a steel H-Pile section placed in the slide surface to resist slide movement. This approach, with merely using cohesion, allowed the spacing of the piles to be excessive and soil flow between piles was a concern. An alternative approach which took into account bending stresses in the pile, used the Corps of Engineers Sheetwall Program Classical method , that used both active and passive soil pressures with a cantilever deflected shape, but neglected true soil support above the slip surface, by using net pressures to load the piles . The design approach led to a 4 foot center to center pile spacing for an HP 14x102 size pile, being used in the second contract phase in 1986. The design assumed each pile had 4-times the driving load per foot of channel reach developed from net pressures calculated in the sheetpile analysis, which used maximum bending stresses as the controlling factor in design. There were pipe piles as well as sheet piles and H-piles designed with the Sheetwall Program design method, which led to piles being constructed at 4 foot spacing in the second contract phase of the Blue River Channel Project, using drained soil strengths for the long-term condition that controlled design. A weak layer of CH-material with Liquid Limits greater than 70, and blue-gray in color, was typically found in the Blue River Channel banks in thin layers, at or near the channel flow-line elevation. Phi angle results from residual S-tests, reported values of phi as low as 7.5 degrees. Later a backed-in slope stability analysis was completed, using the UTEXAS2 computer program (developed by Dr. Steven Wright, University of Texas) that combined all slide area locations in the Stage II Blue River Channel reach, which resulted in a phi angle equal to 11.86 degrees for the weakest CH-material, and was the adopted strength parameter used in all analyses on the Blue River Channel Project, for the weak material identified in slide zones. Joint efforts by structural and geotechnical disciplines were made during design of the next contract reach of the Project in 1988 and 1989 for the "paved reach", to further improve the procedure. The UTEXAS2 slope stability computer program, was used in conjunction with the LPILE program (by Lymon Reese). Unbalanced forces (ie, those forces required to provide a desired safety factor for slope stabilization) were calculated along the shear surface using the UTEXAS2 computer program output. The unbalanced forces were input as a shear force at the top of the pile ("cut" at the shear surface) in the LPILE program. The problem was thus split into two analysis pieces, one above the shear surface and one below. For the upper portion of the pile the P-Y curves were inverted in the analysis from the slide surface to the top of the pile. For the zone below the shear surface another set of P-Y curves were input separately for analysis. Then both LPILE computer runs were compared and adjusted to establish deflection continuity across the shear surface. Structural designers believed the pile deflection should be S-shaped, (Ref. paper by Kirt Mitscher, presented in 1988 at the Structural Conference, St. Louis, Missouri). To achieve this deflected shape and generate "acceptable forces and displacements" in the program the P-Y curves for soft clays had to be input as "low ϕ angle sands".

IMPROVED DESIGN APPROACH

Because of continued uncertainty in the analysis method several steel H-piles in the "paved reach" were instrumented with slope indicator devices. This data proved to be key in developing an improved approach to slope stabilization using piles. Once the deflected shape and magnitude of deflection became known, it became possible to more closely assess how the soil was loading the pile and what P-Y curves should be used in the analysis. Also fundamental to the slope stabilization was assessment of areas of previous failed slopes and performing back analysis for shear strength determination. In all cases for reaches of the Blue River channel laboratory shear strengths over predicted factors of safety and did not identify marginally stable or failed slopes.

In developing the improved procedure we sought a simple approach that could be easily understood and applied in a practical manner. We believed the problem to be bounded by the classic sheet pile cantilever loading (Fig. 1) and that of a rigid sliding block pinned by a single pile (Fig. 2). Neither of these approaches however represent what is believed to be the actual pressure distribution on the pile. In the latter case the soil is not rigid, nor fixed at the top, as would be required to get an "s" shaped deflection as proposed by (Mitscher, 1988). In the case of the sheet pile, the wedge of soil above the shear surface and down slope from the pile is not considered. In actuality this wedge does provide some resistance load against the pile as the pile deflects (Fig. 3a). Additionally the pile must be able to transfer a stabilizing force down to the foundation below the slip surface from the active wedge above (equal to $\Sigma F_U\text{-}F_L$). A free body diagram of the pile and of the "passive block" are shown in Fig. 3b. If a driving force of the magnitude consistent with that required to stabilize the slope is applied to the pile then the LPILE program can be used to compute stress distributions on the pile and thus the associated forces shown in

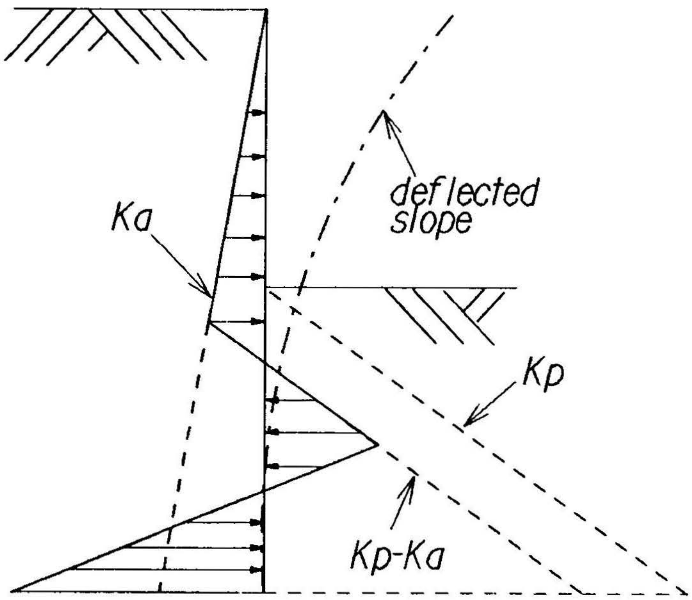

Fig. 1 Classic Sheet Pile Cantilever Loading

the free body diagram in Fig. 3b (ie. P, F_U, and F_L). However, P is an internal force in the free body diagram shown in Fig. 3a and cannot be used to provide an external stabilizing force to the slope; furthermore it cannot exceed the resistance provided by the passive wedge as shown by the free body diagram in Fig. 3b. In order to use the LPILE program correctly a means had to be found which did not use the unbalanced force as determined in the previous method to load the pile. In doing so, one would in effect, use the soil resistance in the passive block twice. To enable correct determination of the external stabilizing force developed by the pile (Σ F_U-F_L) the driving force, R, is determined from limit equilibrium procedures without taking into account any shear resistance on the passive wedge, as shown in Fig. 3a. This is done by trial and error using the computer program UTEXAS3, by setting the shear strength to zero from point c to point e in Fig. 3a. A concentrated point load required to bring the stabilized slope to the desired safety factor is calculated at the pile location. Using the LPILE program this load is input as a distributed load (shape is assumed) and applied to the pile. With P-Y curves generated, the program is able to compute the soil response pressures acting on the pile, the moment and shear developed in the pile, and its deflection. The LPILE output must be checked to assure that P does not exceed the shear resistance available (from the passive wedge), that the depth of embedment is sufficient to develop the resisting force necessary, and does not allow the pile to translate through the soil and that moment and shear in the pile is satisfactory. Several trials are necessary to determine the pile location, size, depth of embedment, and the required pile spacing along the slope. In some cases one row of piles has been found to be insufficient and a second row must be added. Fig. 3c shows a free body diagram for the "load sharing" design procedure.

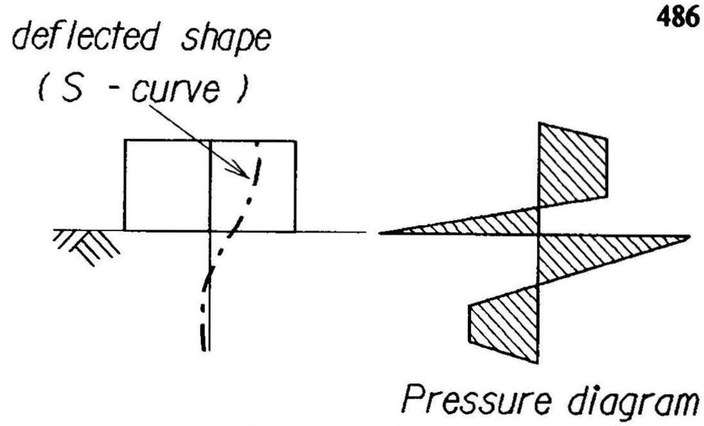

Fig. 2 Rigid Sliding Block Pinned By A Single Pile

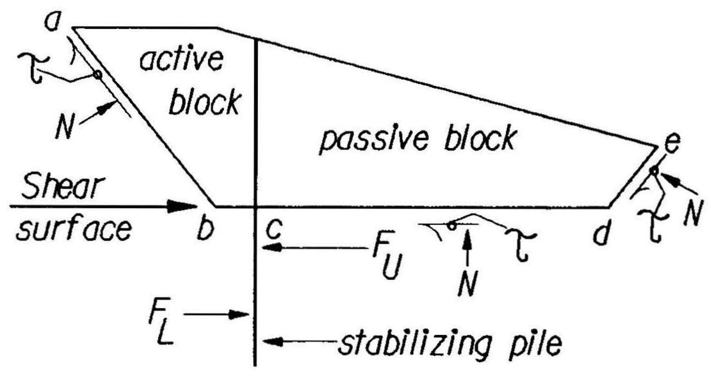

Fig. 3a Schematic Of "Real" Problem

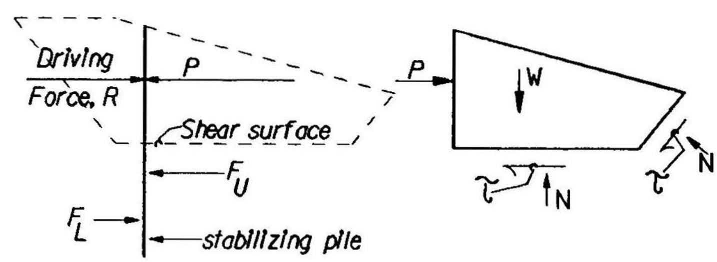

Fig. 3b. Free Body Diagrams

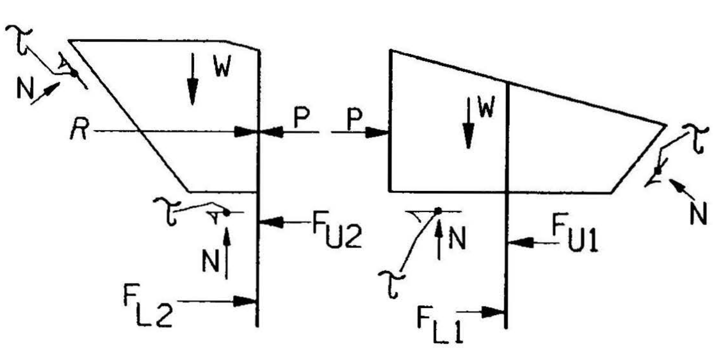

Fig. 3c. Free Body Diagram For "Load Sharing"

PILE DESIGN PROCEDURE (GREGORY BOULEVARD SLIDE)

The Gregory Boulevard slide stabilization project presented the opportunity for the development of the improved approach described above. Numerous solutions other than pile stabilization had been suggested for this area, but none had met the right of way requirements and other special requirements of the city. A photo of the failed Blue River channel bank along Gregory Boulevard, prior to design, is shown in Fig 4.
A slope stability back analysis was first completed for the slide surface at failure, and with an F=1.0, using the UTEXAS2 computer program to find the appropriate design shear strength parameters to use in the analysis.

Fig. 4 Existing Channel Bank Slide Location Gregory Blvd. (Blue River Channel)

See Fig. 5 for backed in slope analysis details. A back analysis condition that was considered to control design was the "End of Construction Case" which used undrained strength parameters, as shown in Fig. 5. Undrained strength parameters of c=240 psf, phi=0 were found to be applicable along the slide plane. The design section used for the analysis, is as shown in Fig. 6. Rockfill as shown was placed on a 1 vertical to 1.5 horizontal finished slope. The design slope shown in Fig 6, without piles in the slope, resulted in an F=0.69. For the pile stabilization analysis, a trial and error procedure with the UTEXAS3 computer program was then used with a horizontal concentrated force option to represent pile resistance at a point on the slide surface between the pile rows, that calculates a driving force acting on the pile required to give a safety factor of F=1.25. Zero shear strength was used below the pile rows along the slide surface so not to use passive resistance twice in the LPILE analysis. The driving force was determined to be 20 kips per foot of channel reach. The driving force was then input into the LPILE program as both a triangular load and a uniform load distribution on the pile, using P-Y curves for soft cohesive type soils. See Fig. 7 for loading diagram schematic for determining the driving forces on the pile. The piles, as designed, were 24-inch dia. Augercast, 32-feet long, spaced at 5-feet center to center, with the pile rows 6-feet apart. The pile rows were offset from each other perpendicular to the direction of the driving forces, which resulted in an equivalent load for each pile at 2.5 feet of channel reach, or 50 kips lateral load per pile. The piles were designed to be socketed into shale 5-feet. The uniform distributed load resulted in the maximum deflection at the top of the pile being 0.51-inches as a cantilever shape with the maximum moment at 63.0 kip-feet. Shear results were -18.5 kips. See Fig. 8 for the deflection plot of the pile with a uniform load distribution. The triangular load distribution resulted in the highest of the two loadings at a maximum moment of -90 kip-feet with a shear of -21 kips. The deflection resulted in the lowest value at 0.24-inches.

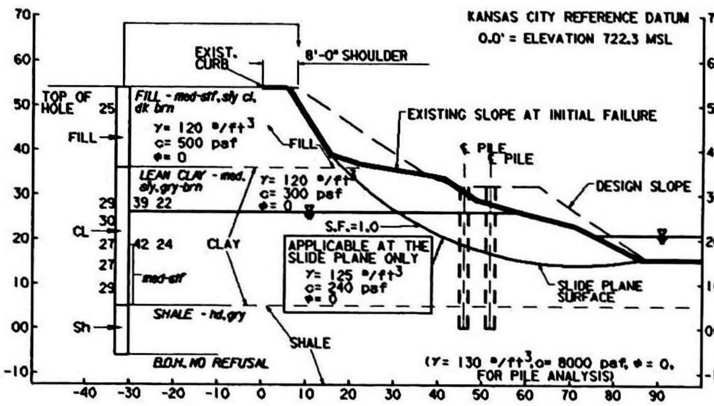

Fig. 5 Backed-in Strengths Analysis For End of Construction Case-Gregory Blvd (Blue River Channel)

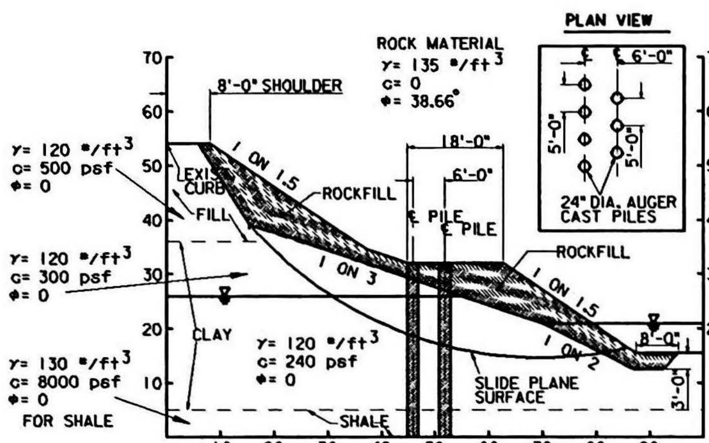

Fig. 6 Design Section Used For Analysis Gregory Blvd (Blue River Channel)

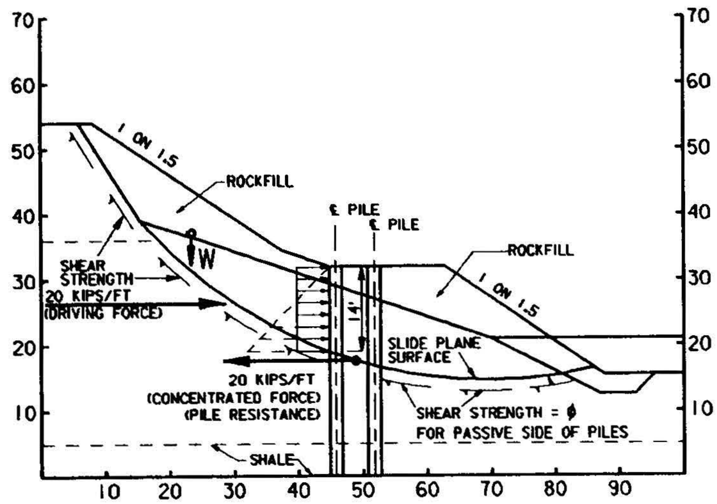

Fig. 7 Loading Diagram Schematic For Determining The Driving Forces-Gregory Blvd (Blue River Channel)

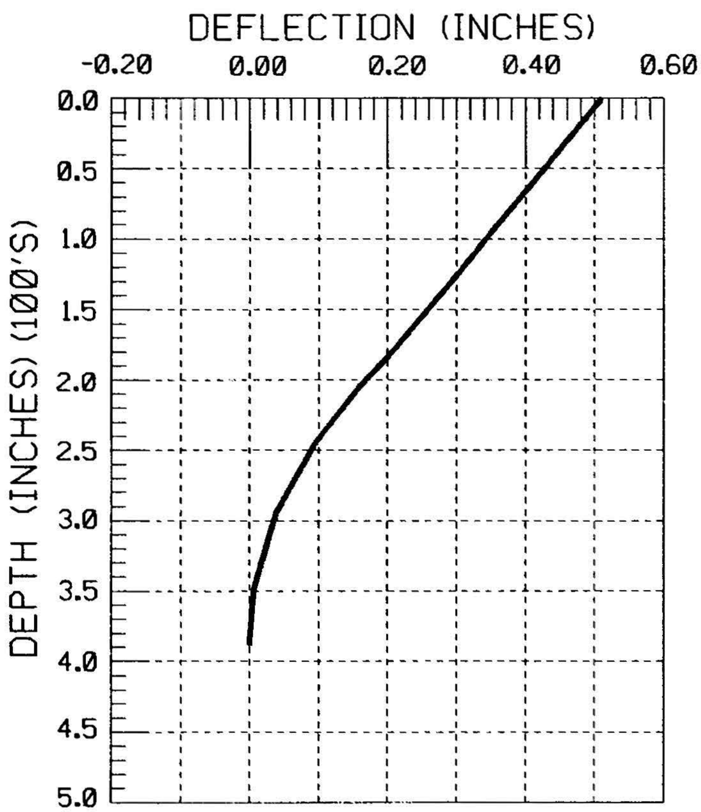

Fig. 8 Deflection Plot Of Pile With Uniform Load Distribution Gregory Blvd (Blue River Channel*)*

A 5-foot center to center spacing was considered a minimum for a 24-inch diameter pile. The Corps of Engineers EM-1110-2-2906, dated January 1991, states that for a pile row center to center spacing equal to 2.5B, or more, perpendicular to the driving forces, that no P-Y reduction is required for overlapping soil wedges (B=pile width). Two inclinometers were installed with the tips into shale, within the 24-inch diameter augercast piles, during construction to monitor movement of the slope and gather pile deflection data to further validate the analysis procedure for the future. Both inclinometers were read in May of 1997 and typical results are plotted as total displacement toward the direction of the channel in Fig. 9 and Fig. 10.

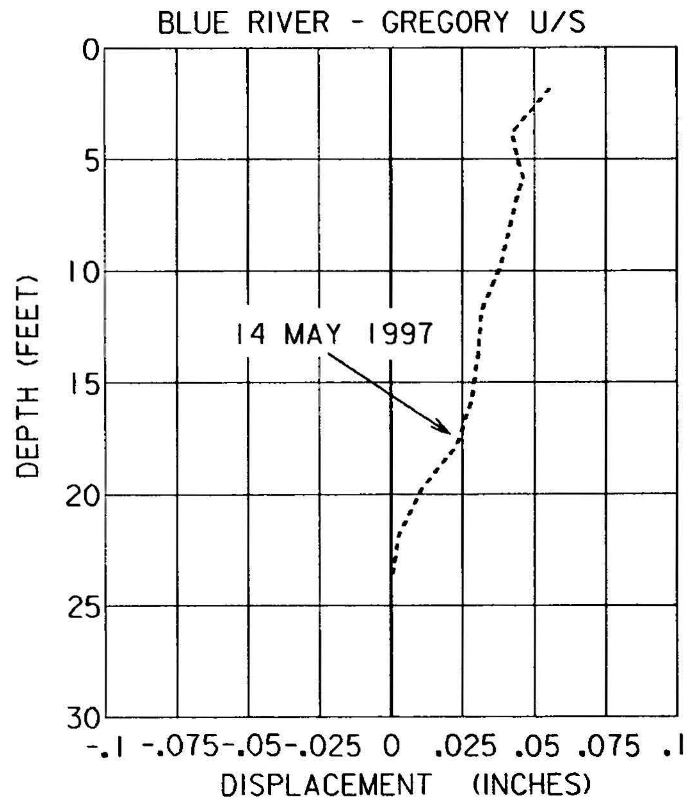

Fig. 9 Upstream Inclinometer On Upper Row Of Piles Gregory Blvd (Blue River Channel)

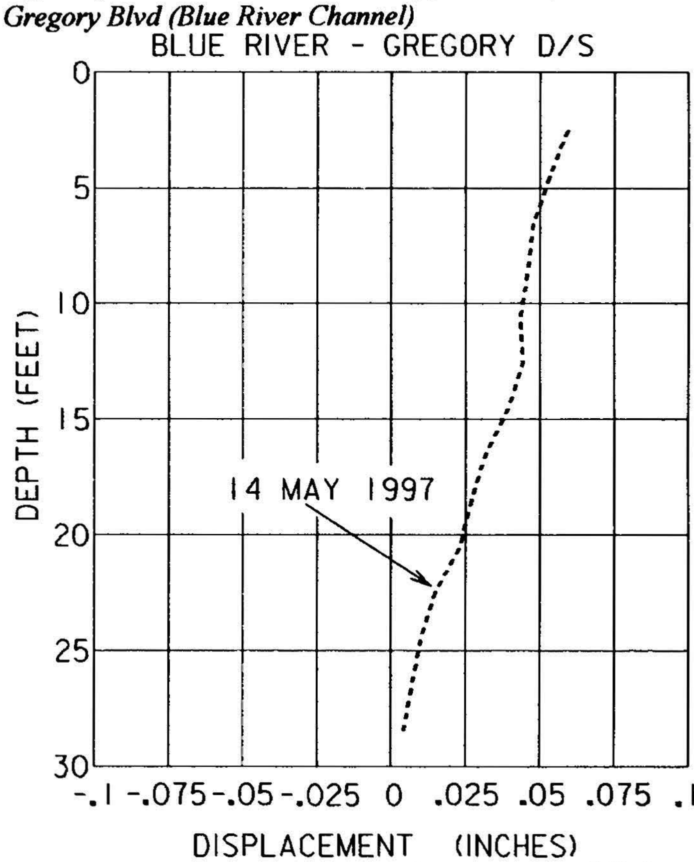

Fig. 10 Downstream Inclinometer On Lower Row Of Piles Gregory Blvd (Blue River Channel)

VALIDATION OF METHOD

An after the fact pile validation study was also completed on a instrumented H-pile in the Second Contract Reach of the Blue River Channel. The instrumented pile had inclinometer readings taken, since 1986 after the inclinometer was installed. The UTEXAS2 computer program was used to first check the slope without the H-piles for two slide surfaces which gave results of F=0.78 and F=0.84 for the overall slide and lower slide surface below the top row of piles (Row no. 1). The UTEXAS3 computer program was then used to calculate the driving forces for the load on the pile (Row no. 1) at a F=1.0 using a concentrated resisting force at the intersection of the pile and the slide surface with the same procedures used in the Gregory Blvd pile design. See Fig. 11 for the design section used in the validation study. A residual drained strength of 11.86 degrees had been used along the majority of the slide surface. The driving force was determined to be 35.9 kips per foot of Channel reach at F=1.0. The pile spacing for a HP 14X102 has been constructed at 4-feet center to center. Each pile had to take 4-times the driving force calculated per foot of channel reach or 143 kips of driving force. Zero shear strength was used below the pile along the slide surface, so as not to use the passive resistance twice in the LPILE analysis. It must be emphasized that an unbalanced force is not used in this procedure, only a total driving force on the pile is considered to be valid. Also note in Fig. 11 the second slide surface shown below pile (Row no. 1) that is loading a second pile row is included to indicate that the geometry of this slope required two rows of piles with separate driving forces calculated for each row of piles. Pile row no. 2 is not part of the validation study, since there is no instrumentation located on these piles.

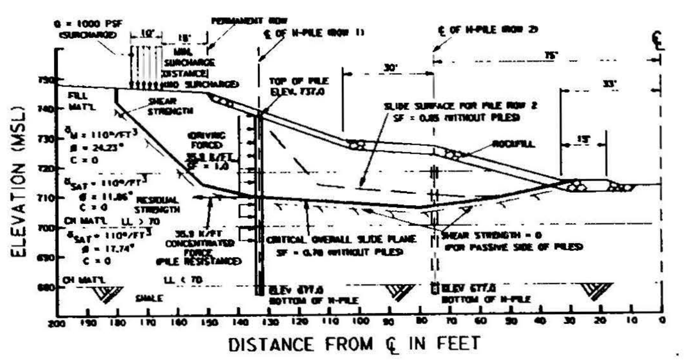

Fig. 11 Design Section For The Validation Study Blue River Channel Project (2nd Contract Reach)

P-Y curves were originally generated for the weak slide zone area, defined in Fig. 11, using a loose sand, since long-term soil strengths with phi=11.86 degrees was the adopted design strengths along the slide surface in the weak zone. Using sand P-Y curves to match the instrumented pile inclinometer shape was not successful. Two different distributed load cases above the slide plane were used. The uniform distribution gave a pile head deflection that more nearly equaled the instrumented pile. The analysis showed a cantilever shape, but since P-Y curves for sand increased with depth, the pile would only deflect within the top 25-feet of pile depth. It was then evident that clay type P-Y curves could only be used when you have clay type soils. The P-Y curves were then modeled as a plastic clay material using undrained strengths (phi=0) equivalent to, approximently the average shear strength along the slide surface, calculated from the UTEXAS3 computer program. The undrained strengths used for the P-Y curves were as follows:

<u>Fill-Material:</u>
c=450psf
<u>CH-Material With LL>70:</u>
c=320psf (soft slide zone)

<u>CH-Material With LL<70:</u>
c=425psf (Strength curves in this zone were adjusted slightly higher during analysis to match instrumented pile)

The final validation analysis used the same driving forces, as used previously of 35.9 kips per foot of channel reach, or with a total driving force of 143 kips per pile, but the load was distributed uniformly over 42-feet of the pile length, which was 5-feet lower than what was defined as the weak zone from our earlier soil borings. The adjustments to the validation, show that the validation results were remarkably close to the instrumented pile, as plotted in Fig. 12, showing both comparisons. The calculated pile head deflection was 2.5-inches with a maximum moment resulting in 75.1 kip-ft. Results of this pile validation indicated that actual bending stresses, as calculated from our latest procedure were less than we had anticipated using a previous design procedure.

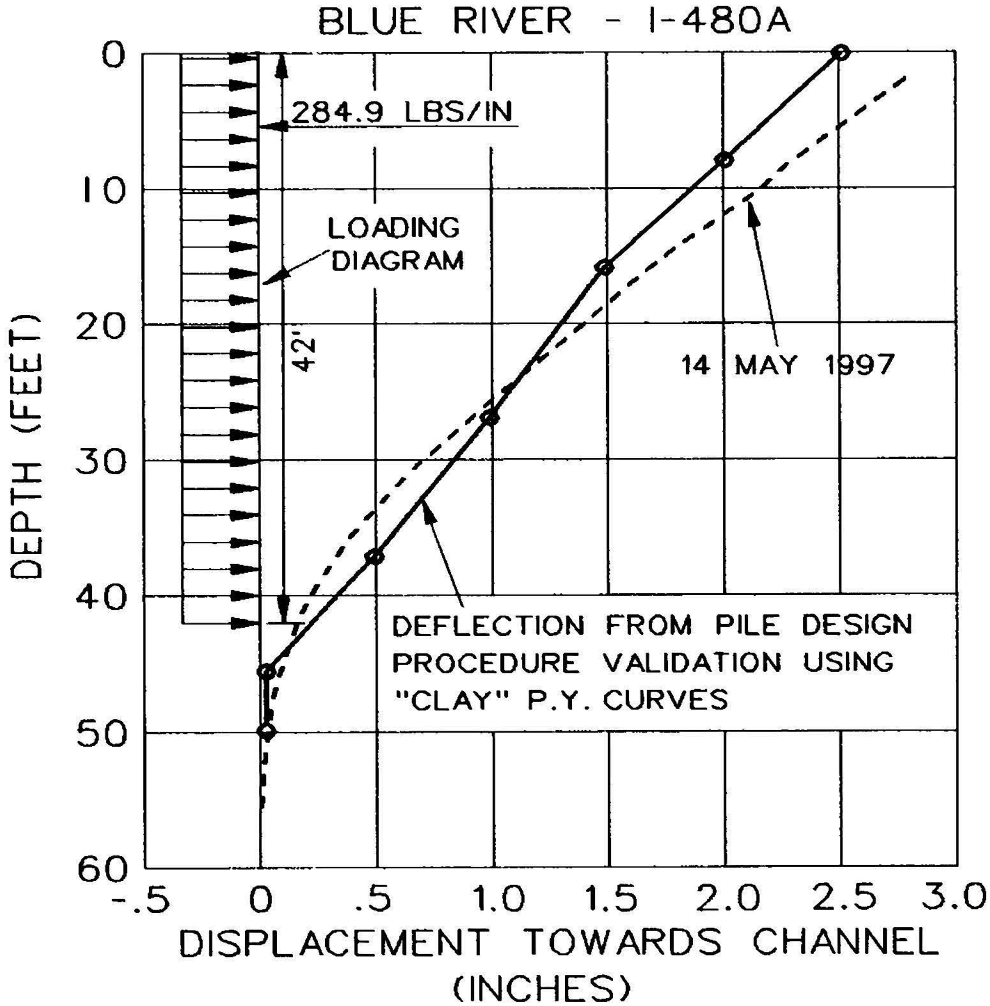

Fig. 12 Inclinometer Plot With Pile Validation Deflection Curve-Blue River Channel Project (2nd Contract Reach)

Additional analyses would be required to confirm that the weak zone was at a greater depth then was observed in our earlier field borings, however funds were not budgeted for additional drilling and testing for the validation study that was within the design scope of the Gregory Boulevard Project.

CONCLUSIONS

Limited validation has shown the improved analysis approach correctly predicts the shape of the pile deflection. The procedure generally should not predict actual displacements, because a safety factor greater than 1.0 is used and generally all the available shear strength in the "passive block" usually is not totally used up in the LPILE developed internal force "P". The procedure is also believed to be somewhat conservative, because all forces applied to the pile are assumed horizontal. With limited experience, thus forth, some conservatism is appropriate.