1st North American Landslide Conference

June 3-8, 2007
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1st North American Landslide Conference

Vail, Colorado - June 2007

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LANDSLIDE STABILIZATION USING PILES

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Abstract: This paper presents a combined geotechnical and structural approach for the design of large diameter piles or drilled shafts to stabilize landslides with discrete, deep-seated shear zones. This approach provides two major improvements over earlier design methodologies: i) the point of application of the slide force acting on the pile is modeled in a manner that, based on successful case histories, more realistically reflects the deformation pattern of deep-seated slide movement; and ii) the factors of safety for geotechnical and structural design are addressed iteratively to minimize compounding design conservatism between the two disciplines.

BACKGROUND

Piles stabilize landslides by mobilizing available passive resistance in the underlying stable ground mass and transmitting that resistance into the overlying slide mass. The design of the piles for this application is usually controlled by the bending moments developed in the pile. Only in the rare instance of a rock mass sliding over another rock mass on a thin, weak interbed would shear forces control pile design. Furthermore, if the underlying stable mass is stronger than the overlying slide mass, as is usually the case, the maximum bending moments will occur in the stable mass. In this situation, development of a plastic hinge in the portion of the pile within the stable mass is the most likely potential failure mode (Viggiani, 1981).

Existing design methods for landslide stabilization using large-diameter piles typically place the resultant design force acting on the pile at a distance of one-third to one-half the depth of the slide above the shear zone (Anagnostopoulos, et. al, 1991; Fukuoka, 1977; Ito, Matsui, and Hong, 1981). Many of these methods are based on lateral earth pressure theory for retaining structures. Unfortunately, given the relatively large size of landslides and the calculated lateral force required to improve stability, this approach can lead to high bending moments and unfeasible or expensive solutions.

The proposed methodology outlined in this paper provides an integrated geotechnical and structural approach for the design of shear piles to stabilize landslides. This paper will discuss: (1) the point of application of the slide force acting on the pile; (2) an LPILE technique to evaluate the bending moments, shear forces, soil reactions, and displacements that develop as the pile reacts to landslide loading; and (3) an interative approach for structural and geotechnical design that allows appropriate FS design for pile capacity under the normal landslide loading conditions, but also ensures adequate structural capacity under extreme landslide loading scenarios.

DESIGN CONSIDERATIONS

Landslide Movement

The approach discussed in this paper is most applicable for landslides that have deep-seated, discrete shear zones of finite thickness. Typically, the shear strength of the soil in the failure zone is much lower than the strengths above and below the shear zone, having been subjected to large strains and development of residual strengths. For these types of landslides, which can have relatively stiff soil above the shear zone, the applied loading on the piles will occur close to the shear zone. An extreme example is the case of a rock mass sliding on top of another rock mass, separated by a weak, very thin interbed. In this case the point of application of loading on a pile would occur at the top of the interbed. The other end of the spectrum is a slow-moving, earth flow slide. These slides typically have very soft to medium stiff soil consistency throughout the depth of the slide mass. In this case, the slides typically will develop resultant slide forces acting on piles that are closer to the ⅓ to ½ point above the shear zone (i.e. similar to existing design methods for shear piles). While the proposed methodology can be applied to earth-flow slides, existing design methodologies are also applicable.

Pile Location

The approach in this paper assumes that the slide mass on the downhill side of the pile remains in contact with the pile. Therefore the location of the piles within the slide mass is a critical design consideration. Piles located near the top of a slide mass may be successful in stabilizing the landslide mass upslope of the piles; however, the remainder of the slide mass may continue to move. If this occurs in clayey soils, relatively small amounts of slide movement will be enough to develop a tension crack on the downhill side of the piles, resulting in a loss of lateral earth support on this side. If locating piles in the upper or middle portion of the slide mass is unavoidable or preferable due to project considerations, then piles should be designed to resist active or at-rest lateral earth pressures in addition to the slide forces.

Design Approach

The design approach consists of the following steps:

- Determine the unfactored and factored landslide loads for global landslide stability
- Determine the thickness of landslide shear zone
- Develop strength parameters for materials above and below shear zone
- Determine resultant location of slide force
- Develop p-y curves for the underlying stable mass
- Determine soil-pile interaction using numerical methods
- Design structural reinforcement to code using the unfactored landslide load
- Confirm that the structural design does not result in unacceptable pile performance when subjected to the factored landslide load calculated for the desired global FS

Unfactored and Factored Landslide Loads

Determining the cause of landslide movement is an important first step in the analysis. Many marginally stable landslides exhibit seasonal reactivation of movement as a result of increased groundwater levels. Other slides develop as a result of natural or manmade changes to landslide geometry, either by removal or erosion of ground from the toe area, or by addition or deposition of material in the upper portion of the slide. These conditions or potential future changes must be considered for long-term stability. For example, continued erosion at the toe of a landslide could cause the ground downslope from the shear piles to become unstable and move away from the piles, resulting in higher earth pressures acting on the piles.

Unfactored Load. Limit-equilibrium stability analyses can be performed to quantify the forces that are needed to maintain equilibrium when the global FS of the landslide drops below one without the use of piles. The maximum force that is reasonably expected to be needed to maintain equilibrium during the life of a project is termed the "unfactored load". Another way to consider the unfactored load is that it represents the best estimate of the actual load that the piles would be subjected to. For example, seasonal monitoring indicates that a marginally stable landslide reactivates when groundwater levels rise 5 feet within the slide mass. In this example the unfactored load would be force required to maintain a FS of unity under the 5-foot rise in groundwater level. However, it should be noted that if seasonal groundwater data is limited, the unfactored load should take into consideration an estimate of the highest groundwater level that could conceivably occur in the slide during the life of the project.

Factored Load. Stability analyses are also performed to determine the force required to achieve a desired global improvement in the FS, termed the "factored load". The desired FS improvement for landslide stabilization is dependent on several considerations including size of the slide mass, geologic and engineering uncertainties, and the importance or criticality of structures affected by the slide. Typically for medium to large landslides, where the geometry and groundwater levels can be accurately determined with field instrumentation, a global FS of 1.2 to 1.3 is adequate to account for potential uncertainties.

Thickness of Shear Zone

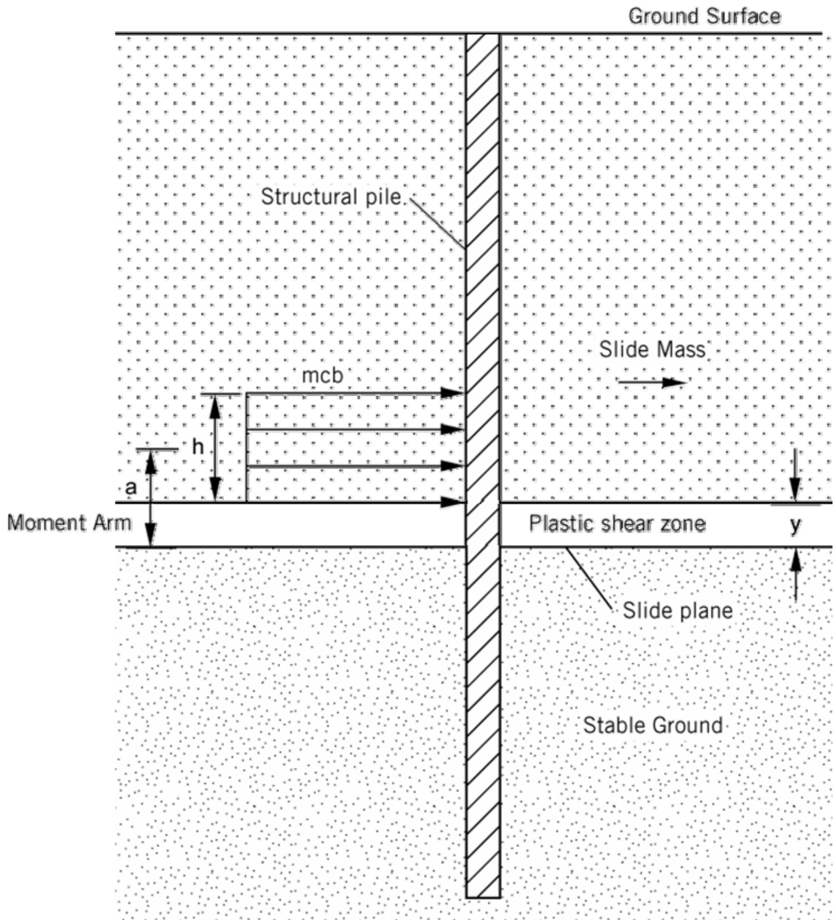
The thickness of the shear zone directly influences the magnitude of bending moment developed in the pile. Slope inclinometers can provide detailed measurements of the shear zone thickness. Measurements from several inclinometers can be used to develop an average thickness, or for conservatism, a slightly larger value can be used for design. Inclinometer measurements are commonly obtained at 2-foot intervals. Usually, greater accuracy is desired for evaluation and it is possible to obtain inclinometer measurements at smaller intervals. Cornforth (2005) has summarized a procedure to obtain 3-inch inclinometer readings for a more precise determination of the shear zone thickness.

Strength Parameters

Develop undrained strength parameters for the stable mass below the shear zone and the landslide mass immediately above the shear zone. As discussed below, the shear zone can be modeled as a plastic zone with no shear strength.

Resultant Landslide Loading

The maximum force that the slide mass can impart to the pile is equal to the passive resistance of the soil immediately above the shear zone (at which point, assuming adequate pile strength, the soil would fail around the pile). The resultant location of the slide force acting on a pile is dependent on the strength of the slide mass above the shear zone. The stronger the soil, the closer the point of application is to the shear zone, and vice-versa. In the case of rock sliding on rock (separated by a thin shear zone interbed), the point of load application would be near the top of the shear zone interbed. As the strength of the slide mass decreases, the distance of the resultant slide force above the shear zone increases. At the other end of the spectrum, a very soft soil would ooze around the piles and the resultant slide force would be ⅓ to ½ the distance from the shear zone to the ground surface. For a single pile, the maximum resistance in cohesive soils at depth is equal to 9cb, where c is the undrained shear strength and b is the pile width (Broms, 1964). It is assumed that the maximum pressure that soil in the slide mass can impart on the shear pile is equal to its maximum resistance. The point of load application can then be determined from the design load and pile spacing (see Figure 1). Reese et al. (1992) has shown that the maximum soil resistance decreases if pile center-to-center spacing is less than 3b perpendicular to the direction of slide movement. The maximum resistance decreases to 4cb for the case of a contiguous wall.



Slide Loading Equation

$m \cdot c \cdot b \cdot h = F_p \cdot S$

Where: c = undrained shear strength above shear zone
b = pile diameter
h = height over which slide force acts
F_p = pile force per unit width of slide
S = pile center-to-center spacing

$$m = \begin{cases} 9 \text{ for } S/b \geqslant 3 \\ 7 \text{ for } S/b = 2 \\ 6 \text{ for } S/b = 1.5 \end{cases}$$

Moment arm: $a = y + h/2$
y = thickness of shear zone

Figure 1. Landslide loading on a pile

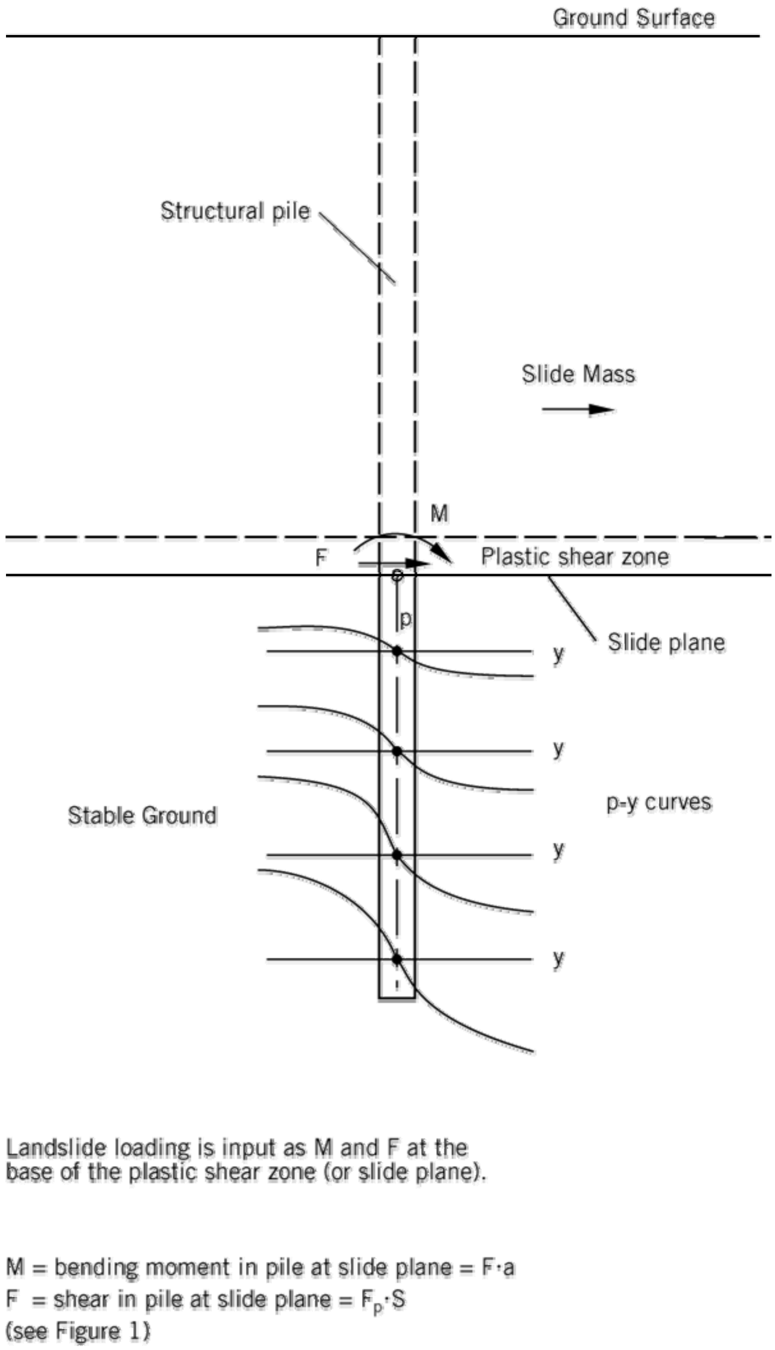


Figure 2. LPILE analysis of a shear pile

As landslide movement loads and deforms the pile, the stable mass provides resistance through passive reaction. The maximum passive force of the stable mass is also equal to 9cb; however, unlike the soil resistance used to locate the distribution of the slide force, this ultimate value should be reduced to an allowable level to prevent a punching failure of the pile into the stable ground. A typical FS of 2.5 to 3 should be used for design purposes, also modified when pile spacing is less than 3b.

p-y Curves

Numerical analyses can be performed to determine the deflection, bending moments, and shear in the pile, and soil reaction with depth for a laterally loaded pile. LPILE is an example of a well-known computer program for this type of analysis (Reese and Wang, 1989). As mentioned earlier, if the strength of the stable mass is higher than the strength of the slide mass, the controlling stresses and moments, and the soil reactions of concern, will be located in the stable mass below the landslide. Accordingly, the evaluation of soil-pile interaction in the stable ground can be performed by modeling a pile with the top of the pile located at base of the shear zone; i.e. the portion of the pile above the shear zone is not considered in the analysis (Figure 2).

The factored and unfactored slide forces are then modeled as applied bending moment and shear acting at the top of the pile. The applied bending moment is equal to the resultant slide force times the arm above the stable mass. The critical step for this analysis is to use the correct p-y curves for the stable mass below the landslide. These curves need to be developed using the in-situ effective stresses for the subsurface profile *including* the slide mass.

Soil-Pile Interaction

An initial pile spacing and pile diameter are assumed for analysis. The slide force is multiplied by the pile spacing to determine the applied moment acting at the top of the stable ground (i.e. immediately below the shear zone) for factored and unfactored slide loads. The program LPILE allows for input of initial steel reinforcement. The analysis provides estimates of pile deflection, soil reaction, bending moments, and shear force with depth. Spacing and reinforcement can be modified as needed based on the results. Once the preliminary layout gives feasible results, a detailed structural design can be accomplished.

The proposed methodology provides an upper bound for shears and moments in the pile. It does not provide any information relative to potential shear and moment demands in the region of the pile located in the slide mass. Conservatively the same moment and shear reinforcement could be used for that region of the pile, or by judgment and/or soil-pile interaction analysis the reinforcement used in this region of the pile could be reduced to be more in line with expected shear and moment demands.

STRUCTURAL DESIGN

Interaction of Geotechnical and Structural Disciplines

Soil-structure interaction problems present unique challenges with respect to structural safety. This is because soil loads are often a function of structure displacement and because the soil strength variability can influence the loads imposed on the structure. The objective is to arrive at an economical structural design without compounding conservatisms entailed in both disciplines. In this paper, discussion is limited to shear piles consisting of reinforced concrete.

In slope stability calculations it is common practice to apply a factor of safety to the ultimate soil strength to obtain a mobilized strength that is considered to produce an acceptable FS. This factor of safety with respect to the stability of a soil mass ($FS_{GEOTECH}$) would be defined as:

$$FS_{GEOTECH} = \text{ultimate shear strength} \div \text{mobilized shear strength}$$

In structural design the practice is to use load and resistance factor design (LRFD) where the resistance, modified by a prescribed strength reduction factor, must be greater than the load modified by a prescribed load factor, or:

strength reduction factor • ultimate resistance ≥ load factor • service load

And as such the factor of safety for structural design (FS_{STRUCT}) would be:

FS_{STRUCT} = load factor ÷ strength reduction factor

The different methods used by geotechnical engineers and structural engineers to define factors of safety must be recognized when designing shear piles or other systems influenced by soil-structure interaction.

Structural Design

Present day reinforced concrete design is based on load and resistance factor design (LRFD) methods. LRFD uses load factors to account for the probability that maximum load effects occurring during the lifetime of the structure may be greater than those assumed in the design. LRFD also uses resistance factors to account for the probability that the strength of various structural components may be less than that assumed for design. Load effects can vary due to variability of the load itself, its distribution on the structure, and the structural analysis method used. Resistance can vary due to the variability in material strengths and variability in member dimensions. The probability approach to reinforced concrete design has significant influence on modern day building codes.

It should be noted that in LRFD the pressure from soil is considered to be a load effect and therefore a load factor is used to account for its potential variability. In the limit equilibrium approach, an approach commonly used by geotechnical engineers to evaluate earth retaining systems and ground mass stability, a factor of safety is applied to the shear strength of the soil. This approach impacts resistance directly and loads indirectly. The differences in the LRFD approach and the limit equilibrium approach suggest that shear piles designed by LRFD methods be checked by limit equilibrium methods to assure all performance objectives are met. To avoid compounding factors of safety, the limit equilibrium check should be made with respect to the nominal strength of the pile.

ACI 318 - Building Code Requirements for Reinforced Concrete - determines the required strength (U) using earth pressures (H) based on available soil strength parameters (i.e., FS of 1.0 is applied to the shear strength of the soil). The resulting earth pressures are multiplied by a load factor equal to 1.6 to determine the required strength (U).

$U = 1.6\ H$

The FS would be equal to the load factor (1.6) divided by the appropriate strength reduction factor (ϕ). For flexural members controlled by tension, the strength reduction factor (ϕ) is equal to 0.90, and therefore the factor of safety is equal to 1.6 ÷ 0.90, or 1.8.

For crack control purposes the load factor may be multiplied by a hydraulic factor equal to 1.3, resulting in a required strength equal to (1.3) (1.6), or 2.08 times the applied earth pressure loading. The higher level of required strength reduces stress in reinforcing steel at service load levels, thereby resulting in improved crack control. This follows ACI guidance used for the design of environmental engineering of concrete structures (ACI, 1989). Controlling concrete cracking is especially important when the piles are exposed to aggressive environments (salt-water or other corrosive conditions). The additional margin of safety inherent in the hydraulic factor also helps to offset some of the uncertainty involved in selecting the load exerted on shear piling due to landslide movement. In this case, the corresponding factor of safety is 2.08 ÷ 0.90, or 2.3.

Load Factor for Landslide Loading

The unfactored landslide load, as determined from slope stability analyses, is provided to the structural engineer, who then applies the structural load factors discussed in the previous section to arrive at an initial reinforced concrete design in accordance with the ACI codes. Note that the factored landslide load should not be given to the structural engineer since this would result in compounding the conservatisms; first by the geotechnical FS in the slope stability analyses and second by the load factor in the structural design.

The earth pressures H discussed above are typically lateral earth pressures in conventional retaining wall type situations where the earth pressures result due to the Rankine or Coulomb wedge upslope of the wall. More commonly, the landslide mass upslope of the piles is much larger than the Rankine or Coulomb wedge and the loading mechanism is different as discussed earlier in this paper. Therefore, the load factors specified in the codes should be used with caution when applying to landslide loading.

Once the structural design is developed using the unfactored landslide load, the design should be checked against the factored landslide load. The structural capacity of the pile and the capacity of the rock/soil reaction below the slide plane must have some margin of safety above of failure under this factored landslide load. The criteria for the magnitude of this margin should be determined on a case by case basis depending on the level of conservatism entailed at various phases of the design process. The criteria that was used in a case history described below were FS = 1.0 for the structural capacity and FS = 1.5 for the capacity of the rock/soil reaction below the slide plane. If the initial structural design does not satisfy these criteria, the design needs to be modified by changing the diameter, spacing, and/or amount of steel until the criteria are satisfied.

A CASE HISTORY

This case history involves a shear pile stabilization of a translational landslide on the Oregon coast. A marina retaining wall, constructed in the 1950's, had been tilting and cracking for several decades as a result of the slow moving landslide. Geologic reconnaissance revealed that the landslide is about 450 wide and 300 feet long (in the sliding direction). The depth to the slide plane ranges from 30 to 40 feet. The landslide would move during the winter when groundwater levels became elevated due to seasonal rainfalls and would stop during the dry summer months. In recent years, inclinometers showed as much as ½ inch of movement during a single, wet winter season. A typical inclinometer deflection plot is shown on Figure 3.

The landslide was stabilized using a total of 71 shear piles installed near the toe of the landslide (see Figure 4). Construction occurred during October to November of 2003. The owner required a global factor of safety of 1.3 after the stabilization. The unfactored landslide load was 22 kips/ft width of landslide (i.e. 22 kips/ft were required to provide a FS = 1.0 under winter groundwater levels). The factored landslide load was 45 kips/ft (this load provided a global FS = 1.3 under winter groundwater levels). The piles had a diameter of 3 feet and a center-to-center spacing of 6 feet, and were typically 60 feet deep. The reinforcing steel arrangement for the piles consisted of twenty-four #11 bars equally spaced around the circumference with a 3-inch cover, augmented with a #4 spiral at

2.25-inch spacing (note that reinforcement was reduced slightly in the upper portion of the pile away from the shear zone). The design criteria used are shown in Table 1 below.

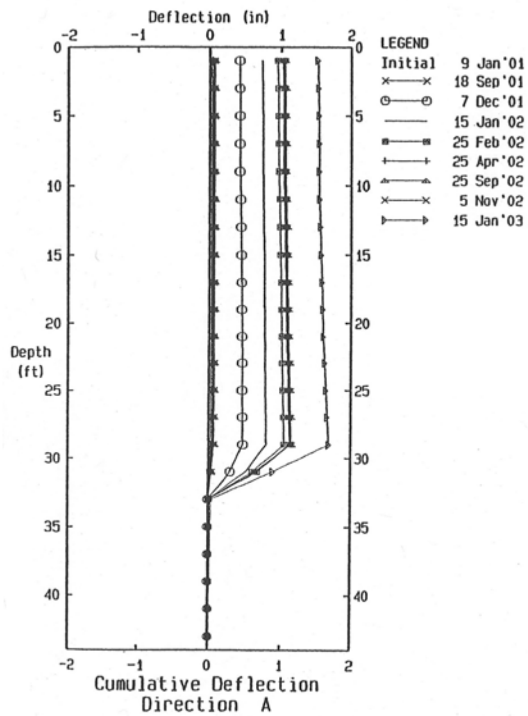


Figure 3. Inclinometer plot prior to shear pile stabilization

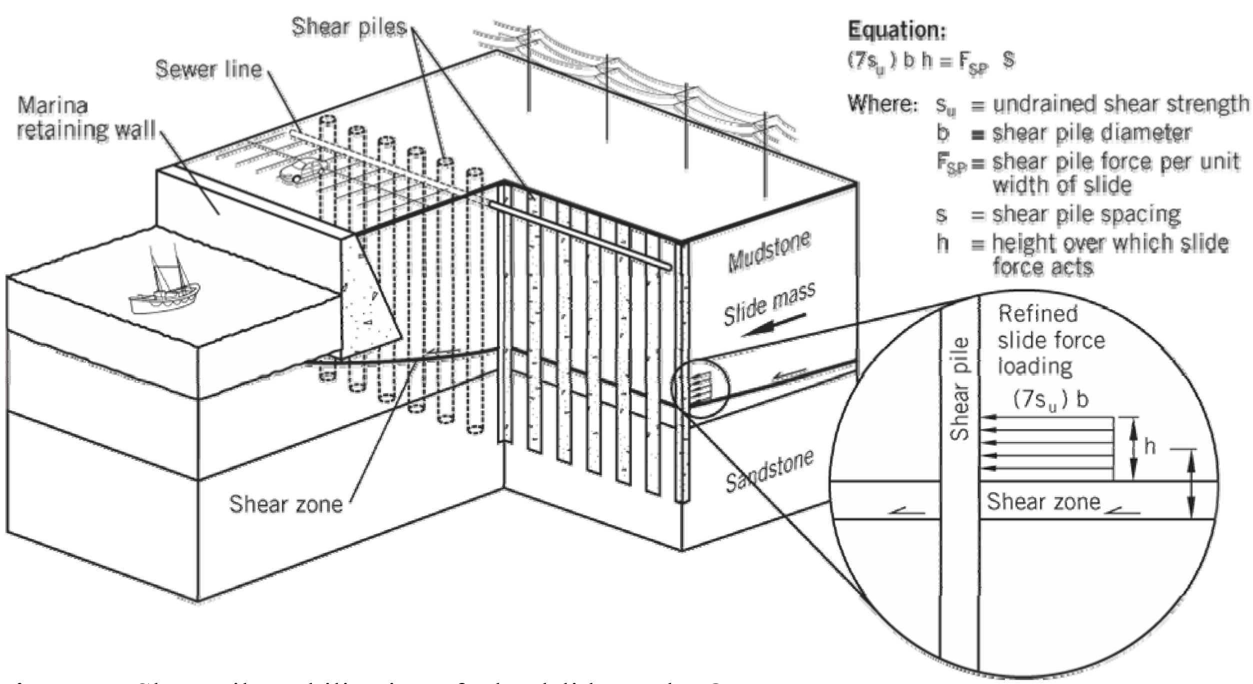


Figure 4. Shear pile stabilization of a landslide on the Oregon coast

Table 1. Design Criteria – Factor of Safety Values

Loading Condition	Under Unfactored Load (22 kips/ft)	Under Factored Load (45 kips/ft)
Global Landslide Stability	1.0	1.3
Pile Structural Capacity	2.5	1.0
Rock Reaction Capacity Below the Shear Zone	3.0	1.5

Prior to construction, LPILE analyses indicated that the maximum deflection of the shear piles (at the base of the shear zone) using the unfactored and factored loads would be 0.35 and 0.95 inch, respectively. Post-construction monitoring of inclinometers has shown that the piles have stabilized slide movements. Three years after construction, based on data from an inclinometer installed within a shear pile, the maximum deflection at the top of the shear zone is less than 0.1 inch. Furthermore, the deflection plot (Figure 5) shows maximum curvature near the shear zone and a relatively straight pile in the overlying slide mass, providing confirmation of the methodology proposed in this paper regarding the location of the resultant slide force. If the resultant slide force was closer to the 1/3 or ½ point within the slide mass, one would expect the defection plot to be curved from the shear zone to at least the ½ point.

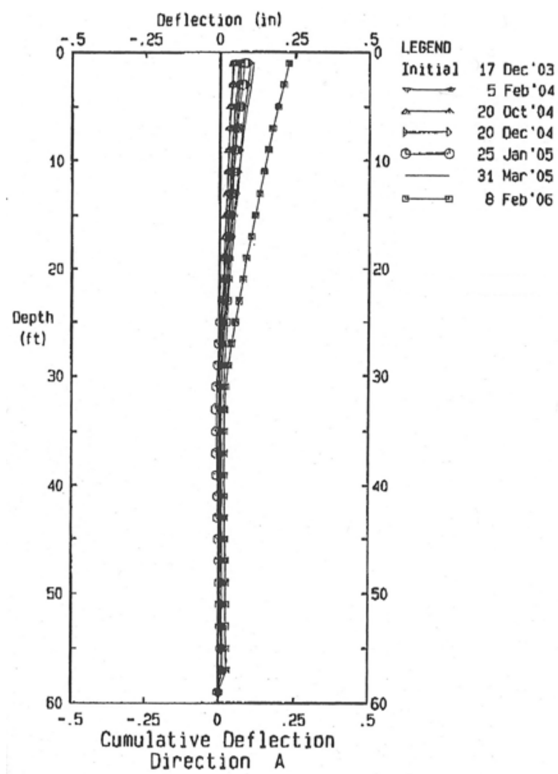


Figure 5. Inclinometer plot within structural shear pile

CONCLUSION

This paper provides a new methodology for designing piles to stabilize landslides. A major improvement over previous design methods is the location of the resultant slide force. For discrete, deep-seated shear zones bounded by stronger materials, the proposed methodology substantially reduces the calculated bending moments to be resisted by the piles. In many cases, this methodology could prove the difference between an economical and uneconomical design.

The second significant improvement presented in this paper is a coordinated approach between the geotechnical and structural engineers. Instead of a structural engineer designing the pile to resist landslide loads that include a global FS on the slide, the structural design is performed for the best estimate of the force required to maintain equilibrium of the slide mass under known or anticipated destabilizing conditions. The design of the pile is then evaluated under the factored landslide load to confirm that minimal structural capacity is still available. In addition, the reaction of the stable material below the shear zone is evaluated using different FS for the factored and unfactored landslide loads. An economic design requires avoiding the compounding of conservatisms entailed in the geotechnical and structural disciplines, and the geotechnical engineer must work closely with the structural engineer to achieve this goal.

REFERENCES

ACI (1989). Environmental Engineering Concrete Structures, *ACI Committee 350 Report,* American Concrete Institute.

ANAGNOSTOPOULOS, C., HADA, M., FUKUOKA, M. (1992). *Piles as Landslide Countermeasures – Model Study.* Proceedings of the 6th International Landslide Symposium, Christchurch, New Zealand, pp. 643-648

BROMS, B. (1964), *The Lateral Resistance of Piles in Cohesive Soils*. Journal of the Soil Mechanics Division, ASCE, Vol. 90, No. SM2, March, pp. 27-63.

CORNFORTH, D.H. (2005). *Landslides in Practice, Investigation, Analysis, and Remedial/Preventative Options in Soils.* John Wiley & Sons.

FUKUOKA, M. (1977). *The Effects of Horizontal Loads on Piles Due to Landslides*. Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, pp. 27-42.

ITO T., MATSUI, T., AND HONG, W.P. (1981). Design Method Stabilizing Piles Against Landslide – One Row of Piles. Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 21, No. 1, Mar.

REESE, L.C. AND WANG, S.T. (1989). LPILE PLUS Computer Program Documentation, Ensoft, Inc., Austin, Texas.

REESE, L.C., WANG, S.T., AND FOUSE, J.L. (1992). *Use of Drilled Shafts in Stabilizing a Slope*. Stability and Performance of Slopes and Embankments – II, Geotechnical Special Publication No. 31, ASCE, Vol. 2, pp. 1318-1332.

VIGGIANI, C. (1981). *Ultimate Lateral Load on Piles Used to Stabilize Landslides*. Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 3, pp. 555-560.