REVIEW OF DESIGN METHODS FOR SEISMIC LATERAL RESISTANCE OF PILES IN SENSITIVE FINE-GRAINED SOILS

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ABSTRACT

This paper raises some issues surrounding the seismic design of piles and pile groups in soft, sensitive silts and clays. The discussion is confined to piles which are end-bearing in a very dense layer below the sensitive soil. The stress-strain behaviour of sensitive soils during static and cyclic loading and pile design procedures are discussed. A simplified case history is presented for a site typical of the Still Creek Basin to illustrate the important factors in design. The results of the analyses suggest that, based on a soil stratigraphy typical of the Still Creek Basin, the pile performance for a fixed head pile under lateral loading appears to be relatively insensitive to the P-y curves assumed for the clay. Where significant pile head lateral loading exists, the form of the P-y curve in the fill is of some importance. The most important consideration in seismic pile design in these soil conditions is the estimation of lateral movements of the fills on the site and the structural properties of the pile or pile group. The estimation of the magnitude of movements depends on the accurate characterisation of the soil deposits; in particular, the degree of strain-softening to be expected during shaking. An approximate but conservative approach is suggested for consideration.

INTRODUCTION

In the Lower Mainland of B.C., seismic design of piles requires consideration of the effects of ground displacements on the response of piles and pile groups. Much of the effort has concentrated upon the case of soil performance in liquefiable cohesionless deposits. This paper discusses some of the factors important to the performance of piles in soft, sensitive soils. A case typical of the soils in the Still Creek Basin is used to illustrate the factors important to the pile performance.

STRESS-STRAIN BEHAVIOUR OF STRAIN-SOFTENING CLAYS

Sensitive clays suffer a loss of strength when they are remoulded, the greater the strength reduction the greater the sensitivity. The ratio of peak undrained shear strength to the undrained shear strength at large strain is used to quantify sensitivity, the measured sensitivity varying depending on the strength test used, e.g. fall cone, unconfined compression test, vane test, etc. The classification of sensitivity presented by Rosenquist (1953) is shown in Table 1. Quick clays have a metastable fabric and, in the extreme, turn into viscous fluids when remoulded. Sensitive soils typically strain-soften during shear, i.e. are stiff at low strains up to peak shear resistance and, beyond a threshold or yield strain, the stiffness and available shear resistance drop dramatically. The degree of strength drop is dependent upon soil fabric which depends on mineralogy, depositional environment, post-depositional environment (e.g. leaching) and stress-history. Mitchell (1993) gives a useful discussion of the causes of sensitivity.

for both static and dynamic conditions. A detailed discussion of design of piles and pile groups under earthquake shaking is given in Gohl (1991, 1993).

LATERALLY LOADED PILES IN SENSITIVE CLAYS

Selection of P-y Curves

For piles and pile groups in sensitive clays, the response of the soil immediately adjacent to the pile will depend primarily on the effects of installation. Randolph et al. (1979) used cavity expansion theory and critical state soil mechanics to show theoretically that installation of large displacement piles of diameter, D, caused failure of the soil out to 2.5D from the pile wall. Matlock (1970) suggested that the zone of influence during large displacement lateral loading of single piles was mainly concentrated within 2.5D from the pile wall, although the zone of influence has been shown to be dependent upon the amplitude of the lateral pile deflection (Gohl, 1991).

For single large-displacement piles, the soil being stressed has likely been destructured and re-consolidated due to pile installation and the selection of P-y curves can follow accepted procedures. For low-displacement piles, much less remoulding of the soil will have occurred and the soil-pile response will be more dependent upon the in situ state of the soil. For pile groups experiencing large strains,intact soil beyond the zone affected by pile installation may be stressed and so pile performance could be more susceptible to the effects of strain-softening. It should be noted that even for strain-softening soil, the P-y curve will increase to an ultimate resistance, i.e. will not display strain-softening behaviour. The soil response will reflect a combination of intact and destructured soil behaviour. The applicable ultimate resistance is likely to be less than for insensitive soil.

Estimation of Cyclic Free Field Displacements

In order to estimate the free field displacements, it is necessary to consider the following:

- select the appropriate stiffness degradation curve for the soil mass
- consider the effect of strain-softening on the ability of the soils to transmit the seismic waves from depth to the surface. It has been noted [Sun et al. (1988)] that high PI clays may remain essentially elastic and will exhibit low-damping. However, it may be expected that where cyclic shear stresses exceed the available shear resistance (e.g. in soft clays or peats), there may be a high degree of damping of the ground motions.
- the program SHAKE [Schnabel et al.(1972)] typically used for level ground free-field response analysis, is essentially an elastic analysis and tends to under-predict strains and gives a conservative over-prediction of stresses and accelerations. It is thus difficult to use the estimated strains to estimate the likely degree of strain-softening. A program which includes a true non-linear soil model, e.g. DESRA, Finn et al. (1977), would be likely to give more realistic displacements.
- although the soft clay may limit the transmission of the seismic waves to the surface, the piles penetrate to the underlying firm ground bearing layer. The pile-soil system acts as a filter of high-frequency input. The pile response will be amplified at the natural frequency

of the soil, the natural frequency of the soil-pile system and the natural frequency of the superstructure.

Estimation of Permanent Free Field Displacements

Cyclic straining will result in a reduction of the available stiffness and shear strength as shaking progresses. Permanent displacements will occur on level ground due to the non-symmetrical nature of earthquake shaking. Additional permanent displacements would be induced where a static shear stress or static bias exists, e.g. due to a sloping site or a body of fill on the site. The latter could lead to a potential for "decapitation" when the inertia forces on the fill exceed the shear resistance of the underlying material. The magnitude of the displacement will depend upon the response of the soil to the induced shear strains.

For the estimation of permanent displacements, two cases exist:

1. Cyclic stresses exceed the available strength but the post-cyclic softened strength exceeds the static shear stresses. The displacements in this case can be estimated using pseudo-static methods such as Newmark's Method (Newmark, 1965). Moriwaki et al.(1989) illustrated the use of this method to explain large slope movements in clay during seismic loading by assuming that the available shear resistance reduced as strain levels increased.

2. The static bias exceeds the post-cyclic shear strength. In this case, large deformations occur and a shear failure results.

These considerations are also applicable to the estimation of movements due to liquefaction of sands.

The estimation of post-cyclic strength requires careful consideration. For stability analysis of embankments on soft clays, Mesri (1975, 1989) has shown that the mobilized shear strength, s_u, at failure can be taken to be $0.22\sigma_p'$, where σ_p' is the pre-consolidation pressure. This was based on a reconsideration of the work of Bjerrum (1972) which summarized the results of back-analyses of many static slope and embankment failures. The applicable strength is an average which takes into account the range in stress paths on the failure surface (see Figure 3). The triaxial compression (TC) strength is typically greater than the Direct Simple Shear (DSS) and Triaxial Extension (TE) strengths. In the case of very flat slides, the direct simple shear (DSS) strength may be more appropriate.

For insensitive normally-consolidated soils, it is reasonable to assume that shear strains will not reduce the post-cyclic shear strength in the free field below $0.7s_u$, i.e. a 30% reduction. Therefore, a conservative approach to estimation of displacements would be to assume a post-cyclic shear strength, s_{upc}, of:

$$s_{upc} = 0.7 \text{x} 0.22\sigma_p' = 0.15\sigma_p'.$$

This should give a conservative estimate of movements as the mobilized shear strength over the estimated number of cycles in a Newmark analysis will fall somewhere between s_u and s_{upc}.

For sensitive clays, the choice of s_{upc} is more difficult and depends upon the shear strains induced by shaking. As long as the driving dynamic or static shear stress is greater than the available shear resistance, then strain-softening will continue. For very high liquidity indices, the remoulded strength could drop lower than 1 kPa (see Mitchell, 1993, Figure 11.35). However, based on the work of Ladanyi (1965) and Thiers and Seed (1968), it seems unlikely that for level ground sites sufficient shear strain would occur during shaking (12 significant cycles assumed for the 1 in 475 year earthquake) to reduce s_{upc} below $0.5s_u$, i.e. the use of:

$$s_{upc} = 0.11\sigma_p'$$

should give a conservative estimate of permanent displacements in the free field during cyclic loading for very sensitive clays. Where a static bias exists or for quick soils, the strength reduction could be greater and greater displacements will occur. The level of strength drop should be assessed using laboratory tests.

Effect of Displacements on Piles

The implications of the estimated free field displacements depend on the degree of fixity at the pile head and the pile tip. For single free head piles, the piles will generally track with the soil. Pile groups or fixed head piles will tend to resist ground movements [Lam et al. (1987), Gohl(1993)]. For pile groups, prediction of pile response including interaction requires a full structural analysis using 2-D or 3-D frame analysis programs including the best estimate of P-y curves (with modifications for pile to pile interaction) and free field ground displacement. The design process involves calculation of the pile response at various stages during the cycling to evaluate the conditions of maximum curvature which does not necessarily occur at the point of maximum deflection.

SEISMIC DESIGN OF PILES IN THE STILL CREEK BASIN

The importance of the above issues will be illustrated by considering the design of piles in the soft soils of the Still Creek Basin in Burnaby, B.C.

Soil and Groundwater Conditions

The soil conditions generally consist of fibrous to amorphous peat overlying very soft to soft silty clay to clayey silt which is underlain by dense to very dense till. The fine-grained soils were deposited in a marine environment. Over time, the elevation of the land surface increased relative to sea level, and the area changed to a fresh water system. As a result of this change, leaching of the salts in the original marine silt and clay deposits likely occurred. Hoy et al. (1967) note that the salt contents in the pore water are no greater than for fresh water. Thick sequences of peat formed in the freshwater environment. The peat has typically been overlain either by alluvial soils deposited by streams or by fills placed to allow development to proceed.

The Still Creek Basin is a topographic low and is a groundwater discharge area. Due to the low permeability of the soil deposits, artesian water pressures exist at the base of the soft soils. The artesian pressures at the base of the clay layer and the low consolidation stresses imposed by the peat result in low shear strengths in the clays.

Figure 5 shows a typical Cone Penetration Test (CPT) log and soil profile. The high moisture contents in the upper portion of the clay are due to the 1 to 3% organic content [Hoy et al.(1967)]. The liquid limit and plasticity index decreases with depth with the liquidity index reaching a peak in the middle of the layer. Figure 4 shows liquidity indices for the soft clay deposit plotted against effective stress. Also shown are contours of sensitivity. Based on this figure, the soil would be expected to have a sensitivity mainly greater than 10 and up to 30 or greater. However, close to the top of the stratum, the in situ stress is very low and so the possible degree of strain softening is limited. The low PI, high liquidity index material in the lower portion of the stratum would appear the most susceptible to strain-softening when disturbed.

Construction Procedures

Development in the Still Creek Basin is most commonly achieved by placement of fills over the peats to precompress the soft soils and raise site grade. Structures are typically supported on driven piles bearing on the till stratum. The soils have a history of large deformations and instability when fill is placed on the peat. In some cases, these large displacements result in failure at the interface between the peat and the underlying clays [Lea and Brawner (1963), Hoy et al.(1967)]. In 1978, at a development site just east of Willingdon, rapid placement of fill resulted in a "Block and Graben" failure with almost fluid clay extruded up to the surface between "blocks" of fill.

Even where failure does not occur, the placement of significant thicknesses of fill result in the fine-grained soils being loaded past their maximum past pressure. Dissipation of the resulting pore pressures is hindered by continuing collapse of the soil fabric. Pore pressures thus remain above hydrostatic conditions, long term vertical and lateral movements continue and shear strength gains due to consolidation occur slowly. Provided dissipation of the pore pressures created by the placement of fill is achieved, the sensitivity of the soils should have been greatly reduced. Full consolidation is difficult to achieve and experience suggests that it may require the use of artificial drainage measures such as wick drains.

Where a significant density difference exists between the fill on a site and an adjacent peat site, or where the site is bordered by a topographic low such as Still Creek or drainage ditches, there will be a tendency towards long term lateral creep movements.

Response of Piles to Lateral Loading

The likely response of piles to earthquake shaking will now be illustrated by consideration of the simplified soil stratigraphies shown in Figure 6. The peat has been ignored. It is assumed that the piles extend 15 metres through fill and soft clays to bear on till at 15 metres depth and that the site is bordered by Still Creek which results in a static bias towards the creek. The pore pressure profile has been assumed to vary from 1 m below ground level to 2 metres above ground level at the till surface. In the fill, the pore pressure profile is assumed to be hydrostatic. At one location, the pile goes through 1 metre of fill and at the other 4.6 metres of fill. It has been assumed that full consolidation has been achieved under the fill loading which, as noted above, is very difficult to achieve. The resulting shear strength profiles used for derivation of P-y curves are shown on the figure.

A free field lateral displacement profile was selected to have a half-cosine distribution versus depth with a peak amplitude Δ_{max} at the top of the fill reducing to zero at the till surface. Permanent post-earthquake displacements of the fill due to inertia forces exceeding the available soil resistances during shaking are estimated to be 300 mm and 600 mm for the 1 metre and 4.6 metre fill cases, respectively. Steel pipe piles of 600 mm diameter have been assumed and a fixed head condition has been considered as this is most representative of the effects of structural constraints on a single pile or pile group. The plastic moment capacity of the pile section has been calculated to be 1220 kNm under an allowable axial load of 1900 kN.

The sensitivity of the computed pile response has been assessed using the computer program LATPILE [Byrne and Janzen (1984)]. The following factors were considered:

- the nature of the P-y curve in the clay
- the nature of the P-y curve in the fill
- the variation in input ground displacement.

Pile group interaction factors and variations in the effects of structural constraint (as exist for pile groups) have been neglected in order to isolate the effects of the above factors. Typical ranges of P-y curves used for the fill and clay in the 4.6 m fill depth case are shown in Figure 7(a) to (d). P-y curves were developed based on methods proposed by Brinch Hansen (1961), Reese et al.(1975), Kagawa and Kraft (1980) and Matlock (1970). A comparison of the results in terms of maximum computed bending moments, M_{max}, is presented in Table 2. Analyses were run using lateral pile loads, V_o, of zero and 53 kN.

For the soil stratigraphies and free field displacement profiles considered, the results suggest the following for <u>fixed head</u> piles:

- the maximum moment is most affected by the magnitude of the fill displacement. The magnitude of the free field displacement is governed by the thickness of the fill. Beyond a critical displacement, a plastic hinge forms at the pile head. For very large displacements, a second plastic hinge may form at greater depth along the pile making it incapable of carrying load.
- for no pile head lateral load and a constant fill thickness, the estimated maximum moment is insensitive to the input P-y curve for either fill or clay. Variations of a factor of two in the ultimate P-y resistance had little effect. Additional runs carried out to investigate the effect of the initial slope of the P-y curve indicated little effect.
- for a typical pile head lateral load and a constant fill thickness, the computed maximum bending moment is moderately influenced by the P-y curve in the fill and not in the clay.

It should be noted that where lateral pile response can be modelled assuming a <u>free head</u> condition, pile response is sensitive to the form of the p-y curve assumed. Lam and Martin (1984) indicate that the most important parameter in derivation of the P-y curves is the ultimate soil resistance which is a function of the mobilized shear strength. P-y curves calculated from the results of full scale field or centrifuge lateral load tests have been derived for free head piles. The above results suggest that for a <u>fixed head</u> pile, structural considerations and the shape and magnitude of the imposed free-field displacement profile dominates pile response.

TABLE 2
RESULTS OF ANALYSIS

FILL THICKNESS (m)	Δ_{max} (mm)	P-y CURVE		M_{max} (kN-m)	
		FILL	CLAY	V_o=53 kN	V_o=0 kN
1.0	300	LB*	UB*	-806	-665
		LB	Mid*	-815	-670
		LB	LB	-824	-676
		LB	Mid-k*	-800	-662
		UB	UB	-725	-665
4.6	600[(1)]	LB	UB	-1220	-1220
		LB	Mid	-1220	-1220
		LB	LB	-1220	-1220
		UB	UB	-1220	-1220

* LB-Lower Bound,
Mid-Mid-range,
UB-Upper Bound,
Mid-k - Mid range ultimate resistance, greater initial stiffness.

Note: (1) Since $\Delta_{max} > \Delta^{critical}$, M_{max} = Plastic moment at the pile head.

CONCLUSION

The above discussion raises issues important to the seismic design of piles in soft, sensitive clays. It is standard practice in the Lower Mainland of B.C. to place large thicknesses of fill to:

- raise grade
- precompress peat
- increase strength of clay.

For the softest sites, the structures are typically supported on piles driven to a bearing layer below the soft clay. This foundation scheme has the following disadvantages:

- fill placement can cause instability during construction

- fill placement causes breakdown of the soil structure which continues to generate pore pressures for considerable periods. This makes the achievement of full consolidation and shear strength gain difficult
- the presence of large thicknesses of fill increases the free field permanent movements caused by an earthquake
- large movements will increase the likelihood of yielding of piles in an earthquake. For very large movements beyond yield, the pile could develop additional plastic hinges and become unstable.

Based on a soil stratigraphy typical of the Still Creek Basin, the pile performance for a fixed head pile under lateral loading appears to be relatively insensitive to the P-y curves assumed for the clay. Where significant pile head lateral loading exists, the form of the P-y curve in the fill is of some importance. The most important consideration in seismic pile design in these soil conditions is the estimation of lateral movements of the fills on the site and the structural properties of the pile or pile group. The estimation of the magnitude of movements depends on the accurate characterisation of the soil deposits; in particular, the degree of strain-softening to be expected during shaking. An approximate but conservative approach is suggested for consideration.

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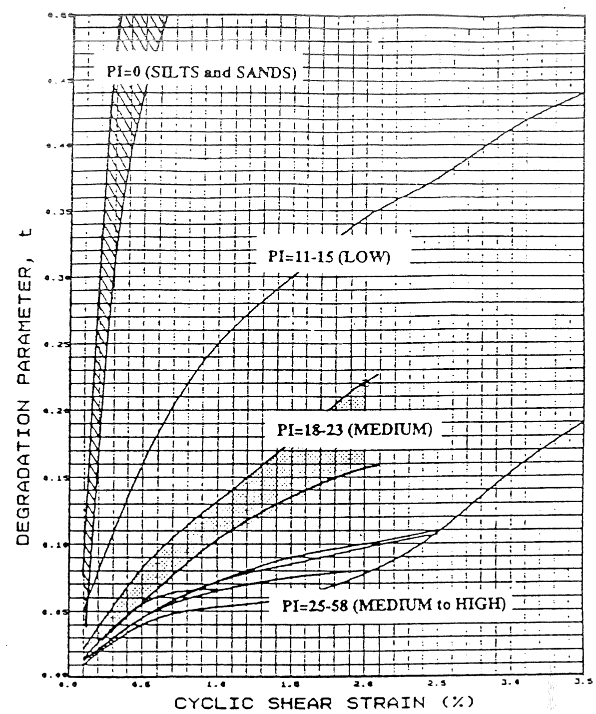

Figure 1. Influence of Plasticity Index, PI, on Rate of Cyclic Stiffness Degradation of Saturated Soils (Tan and Vucetic, 1989).

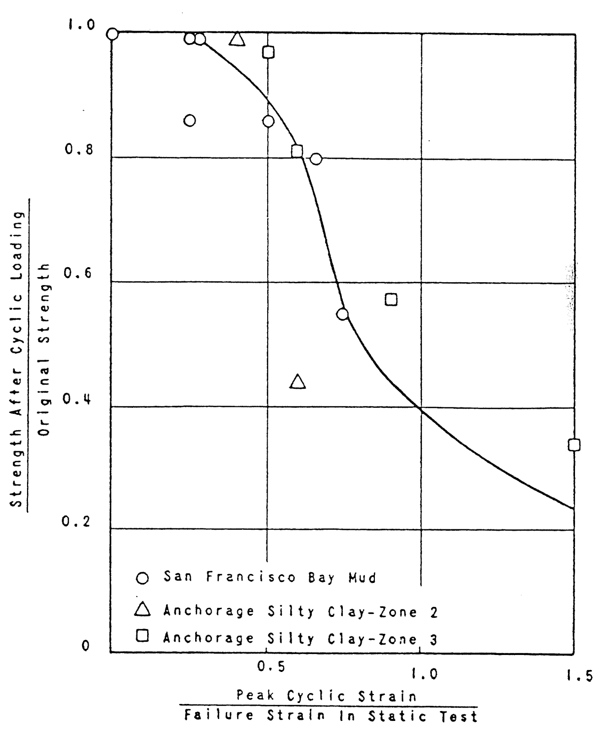

Figure 2. Variation of Static Strength After Cyclic Loading (Thiers and Seed, 1968).

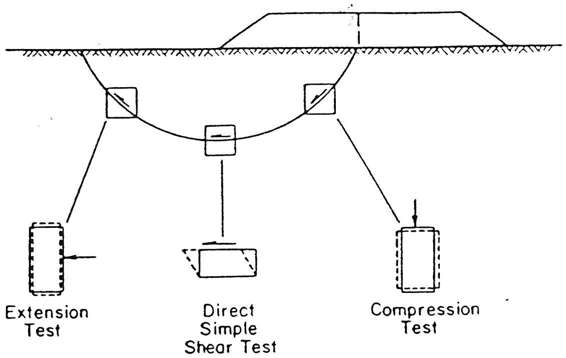

Figure 3. Relevance of Laboratory Shear Tests to Shear Strength in the Field (after Bjerrum 1972).

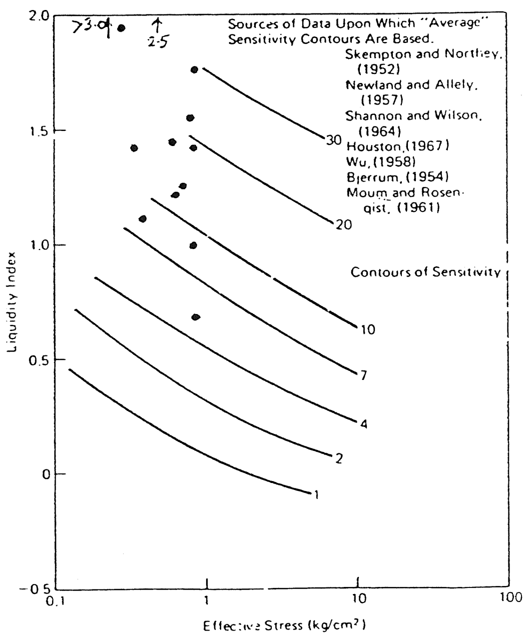

Figure 4. Comparison of Still Creek Soft Clay to General Relationship Between Sensitivity, Liquidity Index and Effective Stress (adapted from Mitchell, 1993)

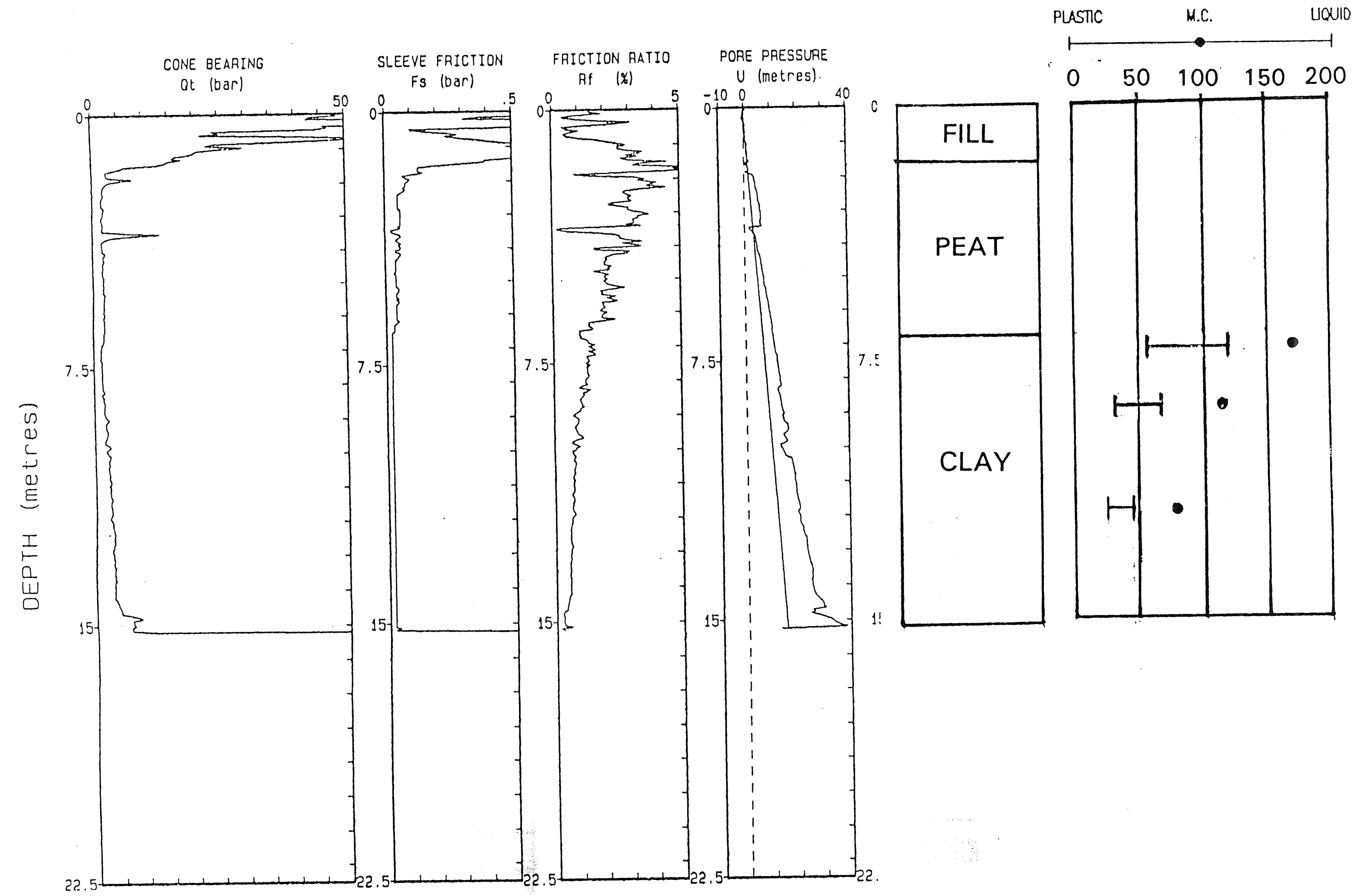

Figure 5. Typical Stratigraphy, Still Creek Basin

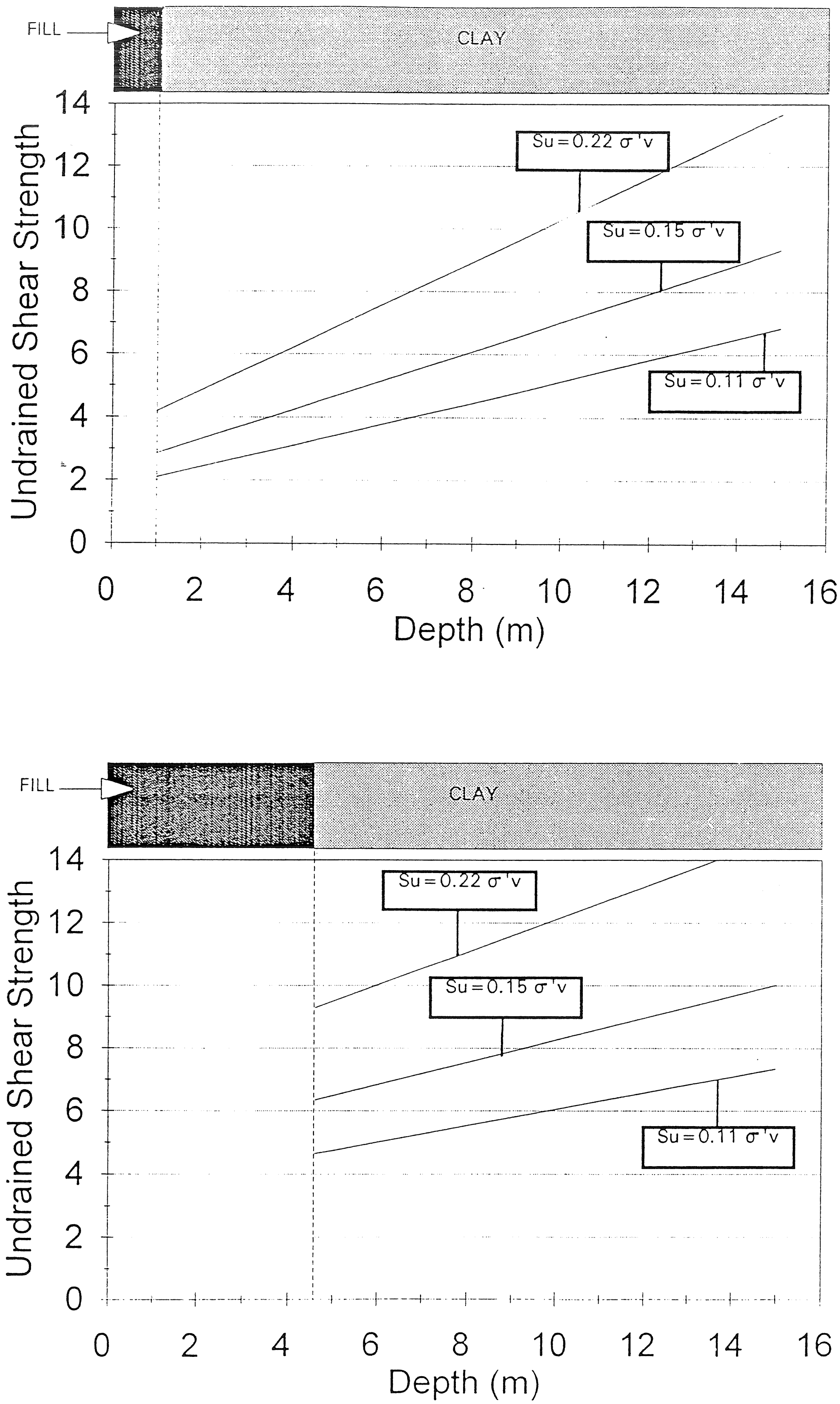

Figure 6. Simplified Soil Stratigraphy and Shear Strength Profiles Used for Analyses

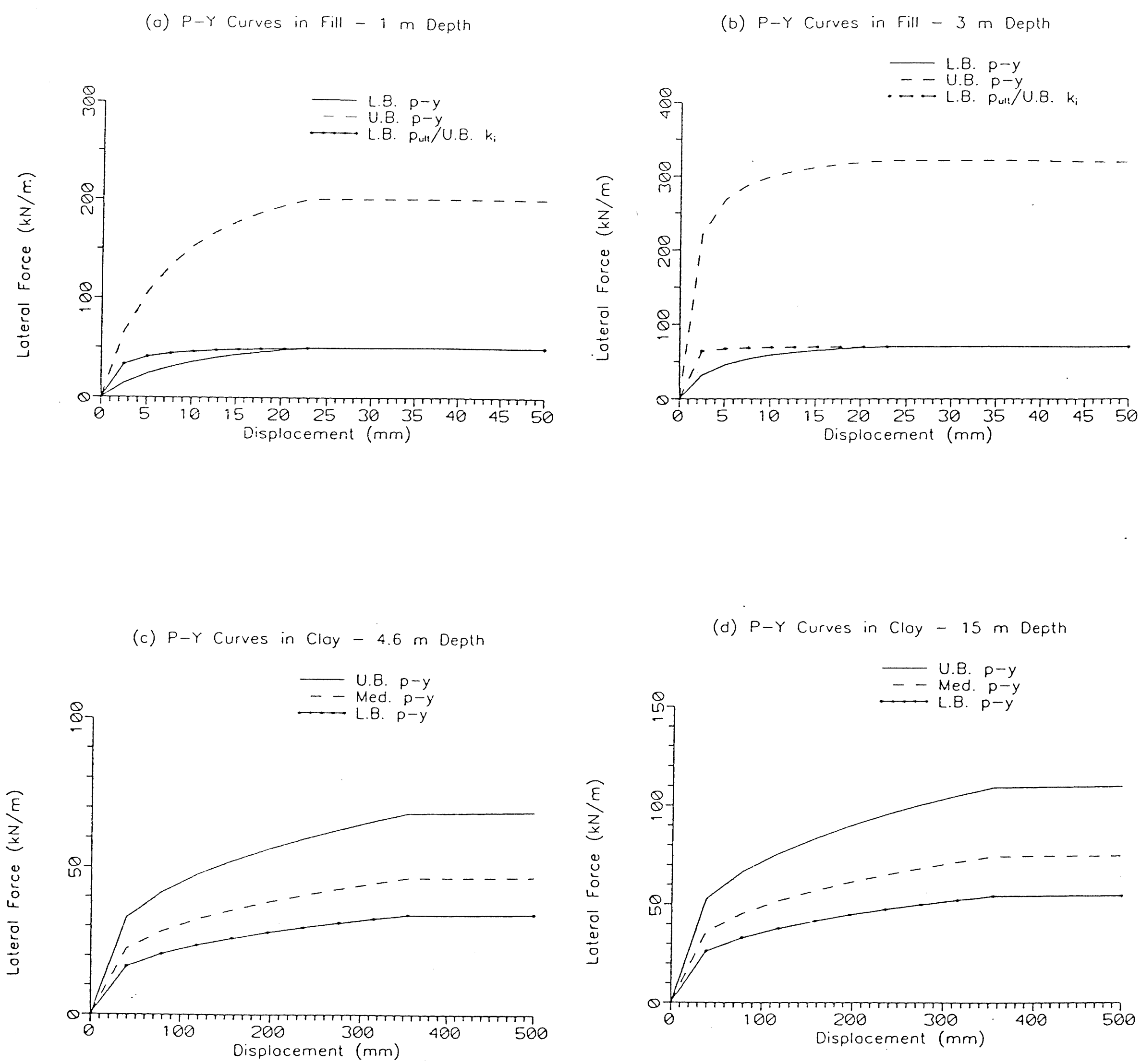

Figure 7. Typical P-Y Curves for 4.6 m Thick Fill Case