

REDUCTION OF UNDRAINED LATERAL PILE CAPACITY IN CLAY DUE TO AN ADJACENT SLOPE

D.P. Stewart

*Lecturer, Department of Civil and Resource Engineering
The University of Western Australia*

ABSTRACT

A practical method exists for calculating the undrained lateral load capacity of piles in level ground. However, the reduction in capacity brought about by constructing piles or piers adjacent to a slope is more problematic. In this paper, an upper bound plasticity method has been used to estimate the undrained collapse load of laterally loaded piles. For the case of level ground, the method yields results that are very similar to those from the conventional approach. The effect of a slope adjacent to the pile was incorporated by truncating part of the assumed collapse mechanism. The results of the analysis are presented in chart form as reduction factors on level ground collapse loads. The reduction factor in any case depends largely on the distance of the pile from the crest of the slope, the slope gradient, and the length to diameter ratio of the pile.

1 INTRODUCTION

A method of calculating the ultimate lateral load capacity of piles in clay was presented by Broms (1964), and is generally well accepted for practical purposes. This method was developed for a level ground surface, and was verified against a number of field and laboratory tests. However, there are many practical situations where piles are constructed near to the edge of a slope, particularly to support signs, light towers and noise barriers around roads and railways. There have been several publications dealing with the effect of sloping ground on the lateral capacity of piles in sand or cohesive-frictional materials, although very little information is available about the same problem in clays under undrained conditions.

In this paper, an upper bound plasticity method developed by Murff and Hamilton (1993) for calculating the undrained lateral capacity of piles in clay is used to assess the effect of an adjacent slope on the ultimate load capacity. It is first shown that the method yields results that are comparable to simpler calculations based on Broms (1964) work. The results are then summarised in chart form as reduction factors on level ground calculations.

2 BACKGROUND

In the method developed by Broms (1964), a distribution of the limiting lateral soil pressure with depth was defined, and this distribution used in a static calculation of equilibrium to determine the ultimate load capacity. The limiting lateral soil pressure was defined as equal to $2s_u$ at the surface, increasing linearly to $9s_u$ at a depth of three pile diameters and remaining constant below that depth, Figure 1(a). To simplify the calculations, it was assumed that the lateral resistance was zero from the surface to a depth of $1.5D$, and equal to $9s_u$ below that depth, Figure 1(b). Obviously for very short piles or piers this simplification is not appropriate, and the original limiting pressure distribution shown in Figure 1(a) should be used. This approach represents some form of benchmark, since it is in widespread use in design and is easy to implement.

Poulos (1976) presented some results of an analysis of piles adjacent to a vertical cut in clay. To account for the cut, the limiting lateral resistance was reduced in the vicinity of any free surface in the same way as shown in Figure 1(a). Once any point along the pile is greater than $3D$ away from either the ground surface or the face of the cut, the limiting lateral resistance is equal to $9s_u$. The results of this analysis were shown to compare favourably with data from small scale laboratory tests. It was concluded that once the pile is greater than $4D$ away from the crest of the cut, the cut has no effect on the ultimate capacity.

Gabr and Borden (1990) presented a method of analysis of short rigid piers based on a limiting equilibrium calculation of a three dimensional wedge on one side of the pier, with active earth pressures acting on the other side of the pier. The wedge originates from the base of the pier. The method requires iteration to reach a minimum solution by varying the angle of the base of the wedge from the horizontal, and the angle of the sides of the wedge from the direction of loading. It is believed that the method is only suitable for short piers, where the lateral capacity might reasonably be represented by the wedge mechanism originating from the base of the pier. Limited results were presented in chart form for soils possessing both cohesion and friction, although there are no results shown for $\phi = 0$. The charts suggest that the

reduction in capacity is largely unaffected by the length to diameter ratio of the pier. This seems physically unrealistic, but is not surprising given the constraint on the failure wedge originating from the base of the pier.

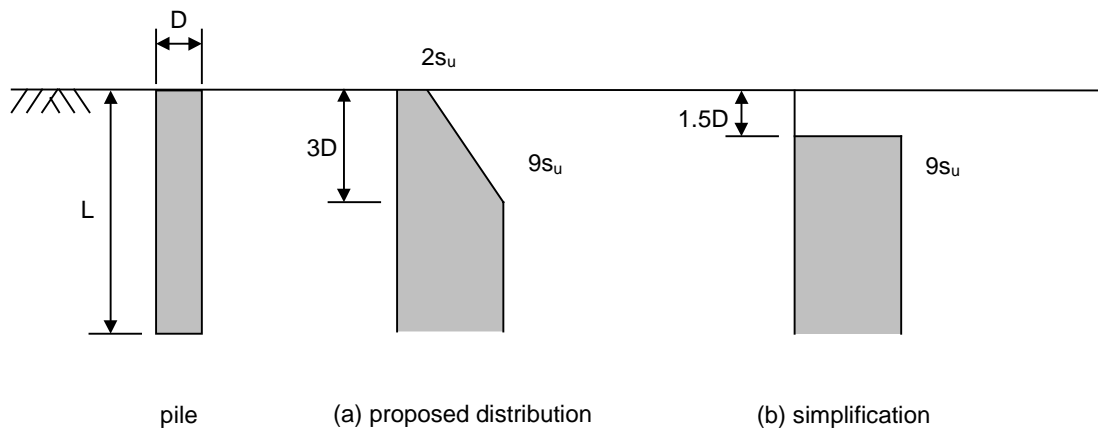


Figure 1 Limiting lateral soil resistance proposed by Broms (1964)

McDonald (1999) described work that was aimed at developing reduction factors for undrained lateral pile capacity adjacent to a slope. It was postulated that the pile is analogous to a strip footing, and that the extent of the stress bulb below the footing (that is, adjacent to the pile) could be used to factor down the limiting lateral soil resistance. Using this reasoning, it was proposed that the slope had no effect once the pile was greater than $13D$ from the slope face, and the limiting resistance was progressively reduced where the slope was closer than this distance. This clearance is significantly greater than that suggested by Poulos (1976), and is believed to be rather conservative. Viewing the strip footing analogy in a different way, the collapse load of the footing is governed primarily by the soil strength within only one or two diameters, which would suggest a more localised effect.

3 METHOD OF ANALYSIS

The method of analysis adopted here is a modification of the method developed by Murff and Hamilton (1993) for undrained analysis of the collapse of laterally loaded piles. This has also been used by Randolph et al. (1998) for analysing the collapse load of suction anchors in clay. The method uses a three-dimensional collapse mechanism and is based on the upper bound method of plasticity theory. In brief, the upper bound method requires a collapse mechanism to be assumed that comprises a complete velocity field. The external work done by the imposed (collapse) load is then set equal to the total energy dissipated within the collapse mechanism, and the collapse load solved for. By varying a number of parameters that describe the geometry of the mechanism, the minimum collapse load can be found.

The approach will not be described in detail, since a good description and development of the equations is given by Murff and Hamilton (1993), but the basic mechanism and components of resistance that are assumed is given below. With reference to Figure 2, it is assumed that a deforming conical wedge of soil forms near the surface, and below the base of the wedge the soil is assumed to flow horizontally around the pile. It has been assumed that a gap develops behind the pile above the base of the wedge. The pile is allowed to rotate about a point that is located anywhere below the base of the wedge. In the analysis presented here, it is assumed that the pile fails by rigid body rotation and translation. It is quite straightforward to include the development of a plastic hinge, but for simplicity this has not been considered in the results presented here. Resistance to lateral displacement and rotation of the pile is assumed to be comprised of energy dissipation due to:

- 1 deformation of soil within the wedge;
- 2 work done by the soil weight within the deforming wedge as it moves upward;
- 3 shear along the wedge-soil interface;
- 4 shear along the pile-soil interface as the wedge moves upwards;
- 5 flow of soil around the pile below the base of the wedge with a limit pressure of $9s_u$; and
- 6 shear of soil over the base of the pile.

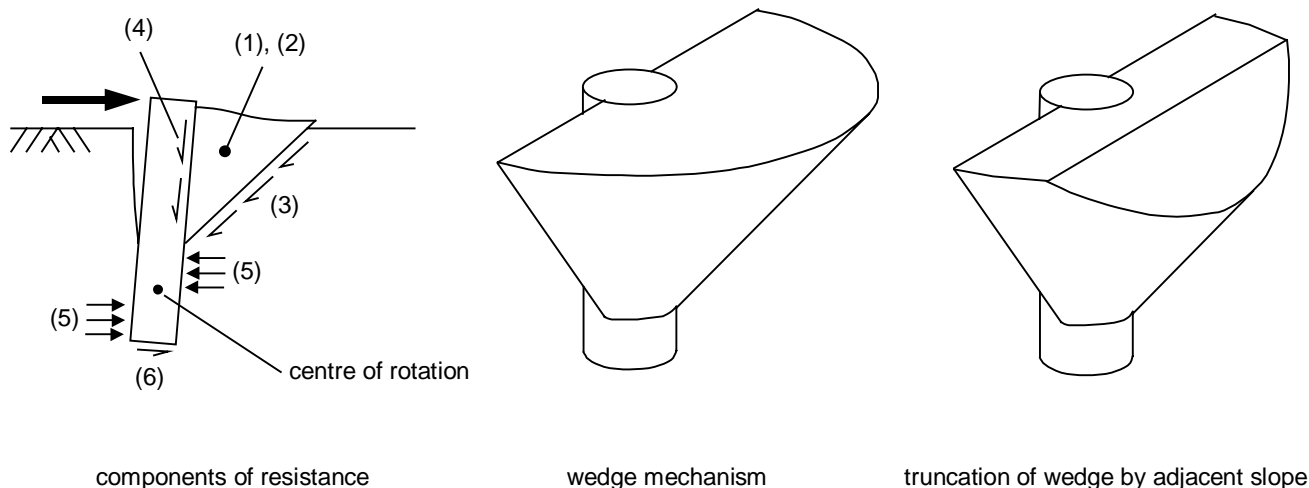


Figure 2 Collapse mechanism assumed for upper bound calculation

To assess the effect of a slope adjacent to the pile, part of the wedge was simply truncated in the analysis, thus leading to a reduction in the energy dissipated in the first three components listed above. It is assumed that the presence of the slope does not affect the magnitude of the limiting soil resistance below the wedge, since this failure mechanism will be confined to within one or two diameters of the pile (Randolph and Houlsby, 1984). This would appear to be a reasonable assumption, except for very steeply inclined slopes in close proximity to the pile. However, in this situation it is unlikely that the collapse mechanism proposed here is appropriate.

In this analysis, the following parameters were varied to find the optimum lower bound:

the depth to the base of the wedge, constrained to be at or above the toe of the pile;

- the radius of the wedge at the surface;
- the centre of rotation of the pile, constrained to be at or below the base of the wedge;
- a parameter describing the velocity field within the deforming wedge.

Murff and Hamilton (1993) have shown that this approach yields predictions of ultimate lateral soil resistance that are consistent with the work of both Broms (1964), for soils of uniform strength, and Matlock (1970), for soils whose strength increases with depth. Randolph (1998) and Randolph et al. (1998) show that the method provides excellent correlation with the results of model tests on suction anchors in clay.

4 ULTIMATE CAPACITY IN LEVEL GROUND

The ultimate capacity of piles in level ground is first analysed here for the purposes of:

- 1 examining the contribution of the various components of resistance to pile movement,
- 2 providing a reference from which to assess the effect of a slope adjacent to a pile, and
- 3 illustrating the similarity between this analytical method and the approach of Broms (1964) that is commonly adopted in design.

For the purposes of the current investigation, the pile shaft and base were assumed to be frictionless and the soil was assumed to be weightless. All of these are conservative, and the weightless soil assumption is necessary to remove the need for a separate variable relating the ratio of soil strength and weight. The effect of including these factors will be considered separately later. The results of a series of analyses are summarised in Figure 3, with curves for different values of loading eccentricity, e , above the ground surface. The "modified Broms" solutions were derived by applying Broms' (1964) method, except that the pressure distribution shown in Figure 1(a) was used, without the simplifying assumption of zero resistance over the upper 1.5 diameters. The comparison between the two methods is very good for pile lengths greater than about 4 diameters. The relative difference between the failure loads from the two methods is illustrated in Figure 4, with the maximum difference of about 15 % at small L/D ratios.

The relative contribution of the six components of resistance is illustrated in Table 1 for two different L/D ratios and different loading eccentricity. In each case the optimum solution was found and the components expressed as a percentage of the total energy dissipated. For the soil strength used (50 kPa), the self weight of the soil is of minor

influence, although for weaker materials the contribution of self weight would become proportionately larger. Shear on the pile base is also a relatively minor component, being most significant for very short piles. Shear along the pile-soil interface may also be significant for short piles. The flow past mechanism below the base of the wedge is potentially a large component of the total resistance. In these analyses the limit pressure in this region has been taken as $9s_u$, corresponding to the plasticity solution for a frictionless pile (Randolph and Houlsby, 1984). Where the pile shaft is fully rough the lateral limit pressure could be as high as $12s_u$. The effect of the geometry of the collapse mechanism on the contribution of the various components is also apparent from the data presented in Table 1.

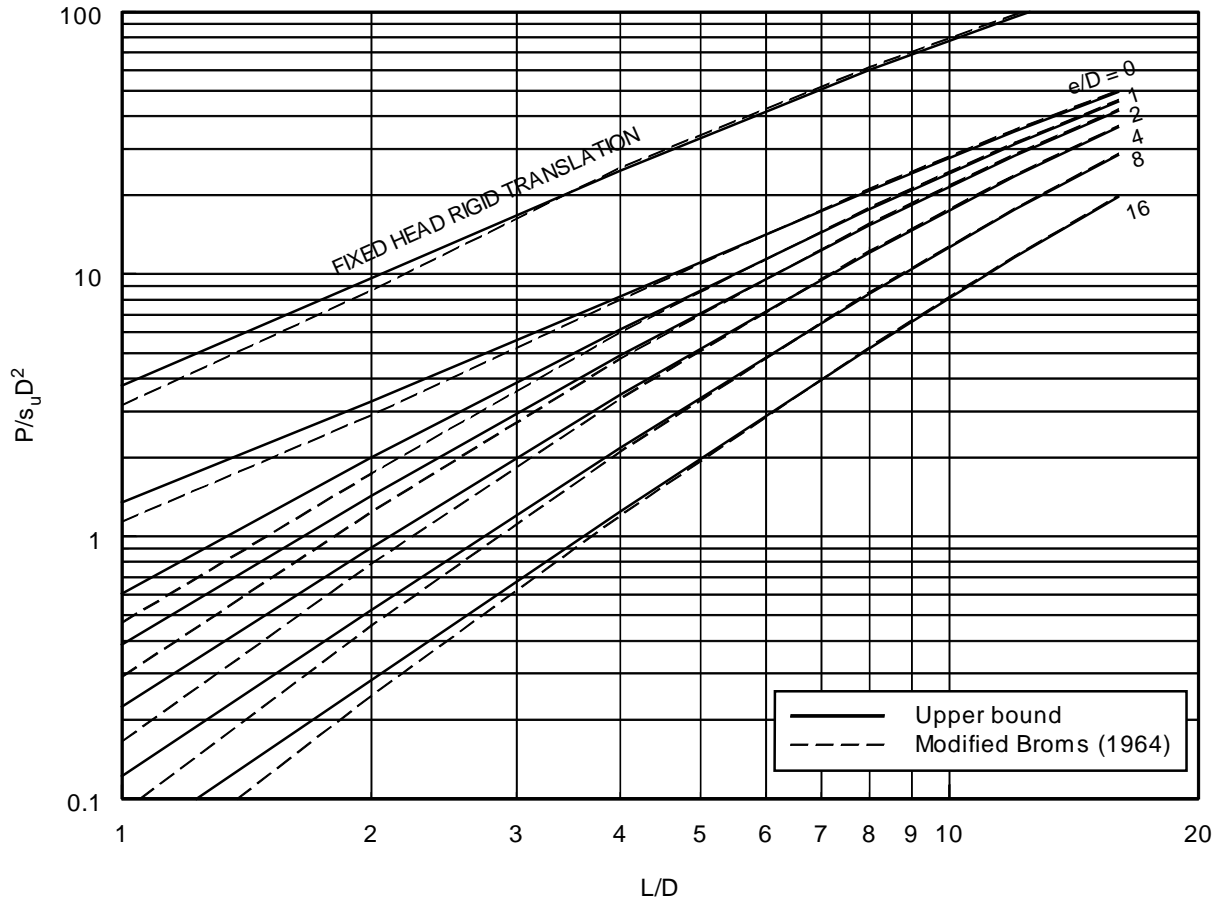


Figure 3 Pile capacity in level ground: frictionless pile, weightless soil

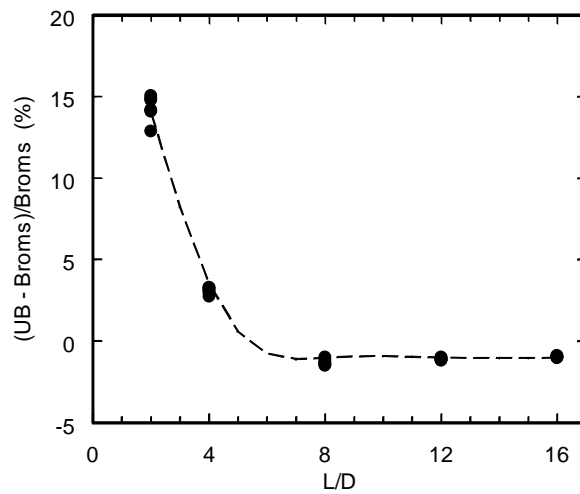


Figure 4 Relative difference between current method and that of Broms (1964) for level ground

The data presented in Table 1 show the contribution of the various components in an optimum solution. However, if one or several of the components are reduced (say by allowing for partial or zero adhesion on the pile shaft) then the optimum collapse mechanism will be altered, and the failure load may be reduced further than might be expected on the basis of the data in Table 1. The magnitude of this effect is shown in Table 2, where shaft and base friction and soil weight were progressively introduced and the new collapse load solved for. Interestingly, from these results it is apparent that shaft friction has potentially the largest effect, with an increase in the collapse load of up to about 35 % by raising shaft friction from zero adhesion to full adhesion. A further increase would be achieved by increasing the lateral limit pressure above $9s_u$ when the shaft adhesion is increased. While shaft friction may be quite low in many practical situations, there may be a significant benefit in terms of pile capacity if it can be relied on.

Component of resistance	Percentage of the total energy dissipated			
	L/D = 2		L/D = 8	
	e/D = 0	e/D = 16	e/D = 0	e/D = 16
deformation within wedge	37.2	26.8	19.0	28.1
soil self weight	2.9	1.9	2.5	4.1
wedge-soil interface	18.6	13.8	13.9	13.4
pile-soil interface	21.7	16.1	10.1	10.1
flow past mechanism ($p_{ult} = 9s_u$)	12.1	31.3	53.3	42.1
shear on pile base	7.6	10.1	1.2	2.1
Depth to base of wedge (m)	1.6	1.3	2.2	4.4
Depth to centre of rotation (m)	1.6	1.3	5.7	4.5

$$s_u = 50 \text{ kPa}, \gamma = 18 \text{ kN/m}^3, D = 1 \text{ m}, \alpha_{\text{shaft}} = \alpha_{\text{base}} = 1$$

Table 1 Relative contribution to collapse load when all components are included

	Relative failure load *			
	$\gamma = 0 \text{ kN/m}^3,$ $\alpha_{\text{shaft}} = \alpha_{\text{base}} = 0$	$\gamma = 18 \text{ kN/m}^3,$ $\alpha_{\text{shaft}} = \alpha_{\text{base}} = 0$	$\gamma = 18 \text{ kN/m}^3,$ $\alpha_{\text{shaft}} = 1, \alpha_{\text{base}} = 0$	$\gamma = 18 \text{ kN/m}^3,$ $\alpha_{\text{shaft}} = \alpha_{\text{base}} = 1$
L/D = 2, e/D = 0	1.00	1.05	1.40	1.52
L/D = 8, e/D = 0	1.00	1.08	1.24	1.25

* failure load divided by failure load when $\gamma = 0 \text{ kN/m}^3, \alpha_{\text{shaft}} = \alpha_{\text{base}} = 0$

Table 2 Effect of shaft and base friction and soil weight on collapse load

5 ULTIMATE CAPACITY NEAR TO A SLOPE

As described earlier, the effect of a slope adjacent to the pile was represented by truncating part of the wedge, assuming that the pile is loaded towards the slope. This can lead to a reduction in the energy dissipated through deformation of soil within the wedge, upward movement of the soil mass in the wedge, and shear along the wedge-soil interface. For the results presented here, the soil was assumed to be weightless, although on the basis of the data presented in Tables 1 and 2 and other analyses, this will lead to a very minor error in comparison to a real soil. The pile is also assumed to be frictionless. For this analysis, loading away from the slope would lead to the same capacity as in level ground, since the limiting soil pressure below the base of the wedge is assumed to be unaffected by proximity to the slope. As indicated earlier, this assumption is believed to be reasonable, except for very steeply inclined slopes located close to the pile where an approach similar to that used by Poulos (1976) might be more appropriate. The results presented below do not cover such a situation.

The results of the analyses are presented in Figure 5, as a slope correction factor to be applied to the pile capacity in level ground. The correction factor was derived by dividing the optimum collapse load for a given pile and slope geometry by the optimum collapse load for the corresponding pile in level ground. Given that for level ground the

method used here provides very similar capacities to those predicted by the Broms method, it is believed that these correction factors can be applied to analyses conducted using Broms approach.

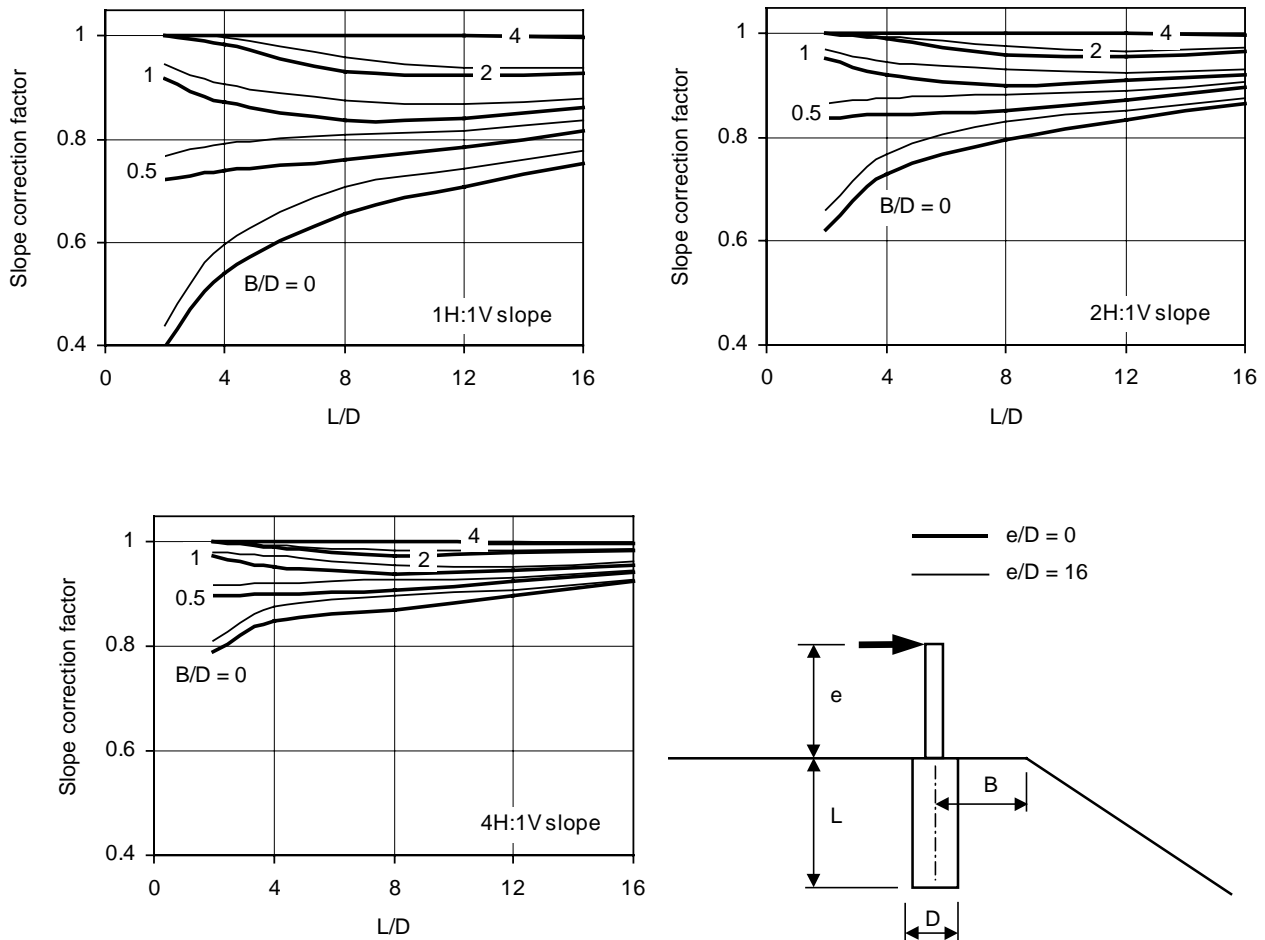


Figure 5 Reduction factors to account for the effect of a slope on pile capacity: frictionless pile, weightless soil.

The results show that the slope correction is strongly dependent on the L/D ratio for piles located at the crest of a slope. The eccentricity of loading has a minor effect. The correction factor is plotted against distance of the pile from the slope crest (B) in Figure 6. While the correction does depend on the length of the pile and the slope angle, the influence of the slope is minimal once the pile head is more than three diameters from the slope crest, and non-existent at four diameters distance. This concurs with the general conclusion of Poulos (1976).

The correction factor is plotted against slope gradient in Figure 7 for piles located at the crest of a slope. The reduction in capacity depends on the pile length, with a relatively minor reduction for long piles. However, for very short piles the reduction may be significant, even for relatively gentle slopes. For short piles, the inclusion of shaft and base resistance will offset this reduction to some extent, as these factors are virtually unaffected by the slope.

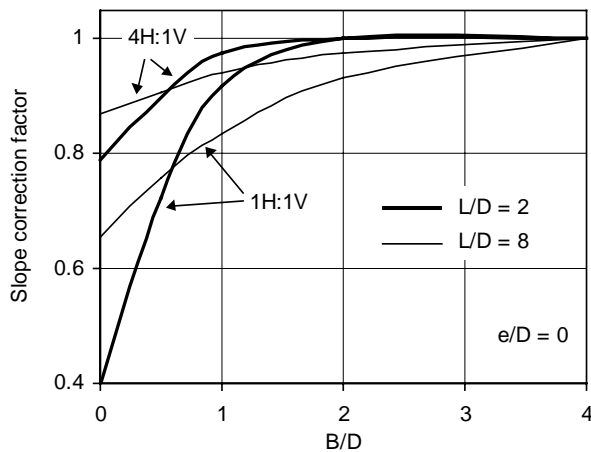


Figure 6 Reduction factor versus distance of the pile from the crest of the slope.

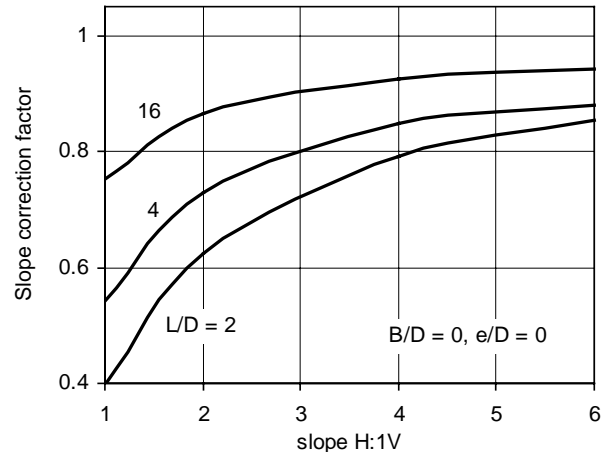


Figure 7 Reduction factor versus the slope angle.

6 CONCLUSIONS

An upper bound plasticity method has been used to estimate the undrained collapse load of laterally loaded piles in clay. For the case of level ground, the method yields results that are very similar to those from Broms (1964) approach. The effect of a slope adjacent to the pile was incorporated by truncating part of the assumed collapse mechanism. The results of this analysis are presented in chart form as reduction factors on level ground collapse loads. The reduction factor in any case depends largely on:

- the distance of the pile from the crest of the slope,
- the slope gradient, and
- the length to diameter ratio of the pile.

The effect of the slope was found to be negligible once the pile is more than three diameters from the slope crest, in general agreement with the work of Poulos (1976).

7 REFERENCES

- Broms, B.B. (1964) Lateral resistance of piles in cohesive soils, *Journal of the Soil Mechanics and Foundation Division, Proc. ASCE*, 90 (SM2), 27-63.
- Gabr and Borden (1990) Lateral analysis of piers constructed on slopes, *Journal of Geotechnical Engineering*, 116 (12), ASCE, 1831-1850.
- Matlock, H. (1970) Correlations for design of laterally loaded piles in soft clay, Proc. Offshore Technology Conference, Houston, Texas.
- McDonald, P. (1999) Laterally loaded pile capacity revisited, Proc. 8th Australia-New Zealand Conf. on Geomechanics, 1, Hobart, 421-427.
- Murff, J.D. and Hamilton, J.M. (1993) P-Ultimate for undrained analysis of laterally loaded piles, *Journal of Geotechnical Engineering*, 119 (1), ASCE, 91-107.
- Poulos, H.G. (1976) Behaviour of laterally loaded piles near a cut or slope, *Australian Geomechanics Journal*, G6 (1), 6-12.
- Randolph, M.F. and Houlsby, G.T. (1984) The limiting pressure on a circular pile loaded laterally in cohesive soil, *Geotechnique*, 34 (4), 613-623.
- Randolph, M.F. (1998) Modelling of Offshore Foundations, Part 2: Anchoring Systems, EH Davis Memorial Lecture, *Australian Geomechanics*, 33 (3), 3-15.
- Randolph, M. F., O'Neill, M. P., Stewart, D. P. and Erbrich, C. (1998) Performance of suction anchors in fine grained calcareous soils, Proceedings 30th Annual Offshore Technology Conference, paper 8831, Houston.