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THE DESIGN OF RAILWAY OVERHEAD LINE EQUIPMENT (OLE) MAST FOUNDATIONS

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ABSTRACT

Railway electrification offers significant benefits in terms of decarbonisation at the point of use and reduced traction costs. However to realise these benefits, the fixed infrastructure must be provided at an affordable cost. Recent schemes in the UK have seen the cost of railway electrification soar: one of a number of reasons for this has been the substantial increase in mast foundation pile lengths compared with historic practice. The paper explores this through a comparative review of traditional and modern pile design methods. In addressing the ultimate limit state, the various approaches are shown to give broadly consistent results in terms of pile length. However, increased pile lengths will be calculated if three-dimensional effects are not allowed for in limit equilibrium (ultimate limit state) calculations, or if a serviceability limit state calculation is carried out using unrealistically low soil stiffness. The results of the comparative analyses should give designers the confidence to use the traditional empirical approach, or a limit equilibrium calculation without the need for an explicit serviceability limit state check (as permitted by EC7) using potentially inappropriate soil stiffness parameters.

Keywords: Piles & piling; limit equilibrium methods; standards and codes of practice

INTRODUCTION

Railway electrification offers benefits for both the environment (zero CO_2 and particulate emissions at the point of use) and traction operation (reduced complexity and cost), but requires investment in fixed infrastructure. If this can be provided economically, the life cycle benefits of electrification should outweigh the costs, especially on an intensively used railway. However, recent experience with the UK Great Western mainline Electrification Project (GWEP) has not been encouraging, with projected costs rising from £900M in 2013 to £2.8bn in 2016 (National Audit Office, 2016). Costs of this magnitude make the economic argument for electrification challenging, but in the case of GWEP political and contractual commitments (particularly to a new fleet of electric trains) had already been made. The specification of the trains was changed from electric-only to bi-mode (with diesel engines capable of powering the trains for main line running) when it became clear that electrification would be delayed – possibly indefinitely on parts of the route (e.g. Bristol Temple Meads and west of Cardiff).

The design and installation of the overhead line masts, and in particular their foundations, seems to have been a key factor in the GWEP cost over-run (RIA, 2019). A High Output Piling System factory train (HOPS) was specified and built on the basis of previous experience with UK main line electrification, which suggested a maximum pile length of 5.5 m. Unfortunately, the simultaneous development of a revised approach to pile foundation design (Krechowiecki-Shaw and Alobaidi 2015) led to an apparently significant increase in design foundation depths. The development of both the HOPS and the revised foundation design process occurred in advance of the specification of the OLE itself. In the event, the more massive “Series 1” superstructure added to the problem by increasing the loads (compared with previous norms) that the foundations were required to carry.

While all of these changes were individually well-intentioned, aiming variously to

- give a design framework clearly founded on modern soil mechanics principles and unequivocally compliant with Eurocodes
- minimise the number of track possessions needed for installation
- generally de-risk the installation process and
- reduce the potential for failure in service, and the associated disruption,

their combined effect on the credibility and affordability of the UK electrification programme has been catastrophic (e.g. curtailment of GWEP at Cardiff, and cancellation of the Midland Main Line electrification scheme). A common thread has been a loss of awareness and / or confidence in the knowledge gained through previous experience, particularly in the design of Over Line Electrification (OLE) mast foundations.

This paper

- summarises the traditional and revised approaches to OLE pile foundation design
- discusses their philosophical differences
- compares the results of calculations for OLE structural and foundation loads typical of the GWEP “Series 1” designs
- discusses the implications of the foundation design approach adopted for future railway overline electrification.

BACKGROUND

This section summarises the loads associated with the GWEP OLE masts, and the fundamental basis and key attributes of the various methods for calculating foundation pile length.

Loads applied to OLE mast foundations

Three standard types of new “Series 1” cantilever OLE support structure were developed; the single track cantilever (STC), the twin track cantilever (TTC), and the extra-large twin track cantilever (XL-TTC). The regular twin track cantilever (TTC), designed to support electrification equipment over two adjacent tracks, is shown in Figures 1a and 1b.

The principal loads on an OLE mast foundation are

- the weight of the mast, W_M and of the foundation itself, W_F, acting vertically downward through the centroid of the foundation

- the weight of the cantilever boom and the OLE it supports (plus snow / ice loading as appropriate), w, acting downward at an eccentricity x from the centroid of the foundation
- a horizontal load H, representing cross-track wind and potentially an in-plane component of wire tension on curved track, acting at a height y above ground level

At the top of the foundation (assumed here to be at ground level, GL), the generalized forces are statically equivalent to

- a vertical force ($W_M + W_F + w$)
- a horizontal force H, and
- a moment of either ($H.y + w.x$) towards the track, or ($H.y - w.x$) away from the track, depending on the net direction of H.

The key design load for an OLE foundation is the cross-track moment at ground level, M_{GL} = ($H.y + w.x$) towards the track or M_{GL} = ($H.y - w.x$) away from the track. The larger moment, with H acting towards the track and increasing the moment due to w - which may or may not be the worst case depending on the slope of the terrain - is shown in Figure 2. This paper focuses on M_{GL}; it does not address the vertical capacity check that would also need to be carried out for design according to standard geotechnical engineering principles.

For simplicity, it is assumed that there is no upstand to the foundation and, if the pile is installed in an embankment, the downslope commences at the side of the pile further from the track so there is no loss of embedment. The total embedded length of the foundation is l metres, of which the uppermost h' metres may be considered to be ineffective owing for example, to disturbance during pile installation.

The most onerous characteristic loads, expressed as moment at ground level (M_{GL}) were calculated for the three standard types of Series 1 OLE structure by Buro Happold (2015a, 2015b, 2015c) and are summarised in Table 1. These define the range of interest for comparative calculations for the foundation analysis methods. The structure must fulfil serviceability limits on variable-load deflection at the wire height under a 1 in 3 year wind load but not under a 1 in 50 wind load, when the structure must fulfil only the ultimate limit state criterion.

The symbols shown in Figure 2 mainly follow Fleming et al (1994, 2009). Other authors use different symbols and sometimes different terms. In the description and discussion of each method that follow, the original symbols are retained to facilitate reference back to the source documents. Later, where the results of different methods are compared, the Fleming et al (1994, 2009) nomenclature and symbols are generally adopted. A full list of symbols is provided in Appendix 1.

Empirical analysis: the UIC-ORE and OLEMI methods

The UIC-ORE method (UIC-ORE 1957) is based on a series of formulae derived from the results of reduced-scale model tests on square, rectangular and circular section foundations in dry sand carried out and reported by Ramelot and van Deperre (1950) and subsequently modified following a series of full scale tests. It is based purely on observation, and does not explicitly distinguish between drained and undrained conditions. The relevant formulae are as follows.

$$M_B = (Mr)_p . (1 - Ep) \tag{1}$$

where

$$(Mr)_p = K_1''. e. Nr + K_2''. \Delta. b. h^3 \tag{2}$$

$$K_1'' = \left(0.5136 - \frac{0.175}{0.54 + \frac{b}{e}}\right) \tag{3}$$

$$K_2'' = \left(2.8 - \frac{96.5}{68.5 + 3.375\left(\frac{Nr}{10.\Delta.b.e.a}\right)^3}\right) . \left(1 + 0.45\frac{e}{b}\right) \tag{4}$$

M_B is the “moment limit” at ground level
h is the embedded depth of the foundation block (interpreted as the total, i.e. including any ineffective or unconsolidated depth, h')
e is the dimension of the block, viewed on plan, parallel to the overturning force
b is the dimension of the block, viewed on plan, perpendicular to the overturning force
a is the smaller of the two dimensions, e and b
Nr is the total vertical load (the weight of the block, the mast and equipment)
Δ is the “specific weight” of the soil, “specific weight” being the term used in the 1957 UIC-ORE report. The units are given as kgm^{-3}; it seems that this is an approximation for decaNewtons per cubic metre ($daN.m^{-3}$), because (although it is not obvious in the UIC-ORE report) the forces are assumed to be in decaNewton (daN) and moments in decaNewton-metres (daNm).

(1-*Ep*) is a correction factor to allow for a depth h' of replaced, unconsolidated or otherwise ineffective soil at the surface of the foundation, and is calculated using the expression:

$$(1 - Ep) = 3.44\left(1 + \left(\frac{h'}{h}\right)^3\right) - 2.44. \sqrt{\left(1 + \left(\frac{h'}{h}\right)^2\right)^3} \tag{5}$$

If the foundation is a cylindrical block of diameter D, $e = b = a = 0.8D$. Although M_B is described as a "moment limit", the UIC-ORE (1957) report states that the foundation movements were small and remained stable at this load.

The full scale tests carried out under the auspices of the ORE investigated the effect of three different configurations of ground (in a cutting; on the level; and on an embankment) and different degrees of support from the track (close to the track and further from the track, with pull towards the track; and pull in the direction away from the track). These led to a final equation of the form

$$(M)_{ult} = 27.45\, K.(M_B)^{2/3} \tag{6}$$

where M_{ult} in decaNewton-metres (daNm) is the ultimate moment of resistance of the foundation, measured at ground level. (The original UIC-ORE report expresses M_{ult} as $(T.H)_{limit}$, where T is the equivalent lateral load in decaNewton (daN) and H (m) is the height above the top of the foundation at which it acts). K is a numerical multiplier ("K factor"), which accounts for the slope of the ground and the degree of support from the track; values are given in Table 2. M_B is in daNm. i is the distance from the track to the foundation (it is not clear how this is measured, e.g. from the centreline, or from the nearest rail). Because the two sides of Equation 6 are dimensionally inconsistent, the constant 27.45 has units of $(\text{daNm})^{1/3}$. With T in kN and H in metres (M_{ult} and M_B in kNm), the constant becomes numerically equal to 5.194 and has units of $(\text{kNm})^{1/3}$.

The allowable value of moment at ground level, M_{GL}, is obtained by dividing the value of M_{ult} from Equation 6 by a factor of 3. This is equivalent to applying a partial factor to the ultimate resistance of the foundation, expressed as an overturning moment. In addition to the terrain "K factor", foundation lengths may be further modified according to a series of corrections that account for the proximity to the crest of a slope and / or a drainage or cable trench (filled or unfilled). These corrections may either increase or reduce the foundation lengths, and are influenced by the shape of the foundation cross-section on plan (parallelpiped or circular). Circular foundations are slightly more sensitive to topographical features, as summarised in Appendix 2.

OLEMI

Inspection of Equations 2 and 4 shows that neglecting the vertical loads W_M, W_F and w (Nr in Equations 2 and 4) will lead to a conservative design. The simplification resulting from not having to consider the effect of the foundation weight enables generic look-up charts or tables to be developed. Neglecting the weight of the foundation also removes any distinction in the calculation between foundations made of steel and foundations made of concrete. Thus a relationship between total foundation depth (h) and factored ultimate moment resistance, $M_{ult}/3$, may be developed that is a function of only the foundation cross-sectional dimensions (e.g. the diameter d), the effective soil unit weight γ* (= $\gamma - \frac{du}{dz}$), and the disturbed or unconsolidated soil depth h'. ($\frac{du}{dz}$ is the rate of increase in pore water pressure u with depth z. In hydrostatic conditions, $\frac{du}{dz} = \gamma_w$ and γ* = ($\gamma - \gamma_w$).

This is the basis of the Over Line Equipment Master Index (OLEMI) method of design, a modified and more conservative version of the ORE approach in which the beneficial effect of the weight of the foundation and OLE structure is neglected (Mootoosamy et al., 2015) For the Series 1 loads (Table 1), OLEMI gives foundations that are typically 10 to 20% longer than those determined using the original ORE method. This paper reports pile lengths derived using the OLEMI methodology.

In a further development, the UK Master Series (Network Rail, 2015) provides "strength depth" tables for allocating concrete and tubular steel pile foundations based on an OLEMI design using an ineffective near-surface depth h' = 0.3 m, a terrain factor K = 1.3 or K = 1 (i.e. level ground with across track or away from track loading respectively), and a soil effective unit weight γ* = 15 kN/m^3. Appendix 2 summarises the factors and additional corrections that must be applied to extend the foundation "strength depth" tables to foundation allocation for different foundation types, terrain conditions and loading directions.

Range of applicability of ORE and OLEMI methods

The ORE method is empirical, in that Equation 6 was determined directly from experimental results in a range of terrain types, rather than by considering either a limiting or an in-service ("working") stress distribution – something that is difficult to address in sloping terrain such as an embankment. The authority of the method derives from the international collaborative research that went into developing it, together with the fact that it has been used extensively and successfully by railway administrations in Europe for decades – including on Network Rail and HS1 (AMEC SPIE Rail, 2000).

Given the empirical nature of the method, it is important that it should not be used outside its current evidence base. In particular, the tests described in the original UIC-ORE (1957) report

- were for concrete foundations up to 3 m in length
- involved design moments at ground level of up to 130 kNm away from the track and 170 kNm towards the track for flat terrain (K = 1), and design moments at ground level of up to 190 kNm away from the track and 230 kNm towards the track in a cutting (K = 1.8).

The report also states that the unconsolidated depth h' would not normally be expected to exceed 0.5 m; and cautions that the full effect of the foundation weight, although appearing in the factor K_2" defined in Equation 4, is not completely captured by the equations.

A review of Network Rail's experience of the OLEMI method (Mootoosamy, et al., 2015) highlighted its use in the design of a range of foundations since 1984, with no reported instances of subsequent associated failure (collapse or loss of serviceability). Electrification of the East Coast Main Line between 1986 and 1992 represents the most extensive use of ORE based foundation design in the UK, in which the Master Series "strength depth" tables were applied to both concrete and 610 mm dia CHS steel piles between Hitchin and Carstairs (via Edinburgh), supporting the Mk3B 25 kV AC OLE and associated masts. Historical records also reveal the successful application of the Master Series method to the design of foundations in alluvium and chalk, and to a more limited extent, peat. These geologies are excluded from the range of applicability of the ORE method given in the original UIC-ORE document.

Analysis based on allowable lateral stresses: the Balfour Beatty Power Construction Limited (BBPCL) Method

The BBPCL Foundation Design Manual (1990) describes methods of design analysis based on allowable lateral stress distributions on piles in terms of both effective and total stresses (corresponding to drained and undrained conditions, respectively). These are illustrated in Figure 3. The effective stress distribution was originally proposed by Sulzberger (1945).

Explanatory notes relating to the BBPCL approach are given in Table 3.

In both cases (drained and undrained), the net lateral stress distribution is automatically in horizontal equilibrium; hence an additional average lateral pressure in front of the foundation of $H/(L.D)$ (where L is the width of the foundation perpendicular to the direction of H, and D is the effective depth of the foundation) would be needed to resist the net horizontal force H.

Although the BBPCL method has been used successfully in the design of OLE mast pile foundations, it has fallen out of favour in recent times. Being based on permissible stresses, it does not readily fit within the framework of EC7 and uses soil parameters whose determination is to an extent subjective. It will therefore not be considered further in this paper.

Analysis based on limiting lateral stresses (following Brinch Hansen 1961, Broms 1964a and 1964b, Fleming et al 1994 and 2009)

i) effective stresses

The design approach developed for GWEP (Atkins, 2010) is based on factored limiting net lateral stress distributions. It broadly follows the approach given in textbooks such as

Fleming et al (2009), which is itself based on principles suggested by Brinch Hansen (1961) or Broms (1964a, 1964b).

The form of the limiting lateral stress distribution assumed for drained (effective stress) conditions is illustrated in Figure 4.

There are differences in the detail of the assumed limiting lateral stress distribution proposed by the various authors.

Brinch Hansen (1961) bases the equivalent lateral stress near the surface on an assumed three-dimensional passive failure wedge, and includes an allowance for a contribution from friction on the sides. At great depth, failure is assumed to correspond to a bearing failure of the pile in the horizontal plane, hence involves a term in $K_o.K_p.e^{\pi\tan\phi'}$, where $K_p.e^{\pi\tan\phi'}$ is the bearing capacity factor, N_q. To avoid a discontinuity in the lateral stress at failure between the two regimes, Brinch Hansen (1961) proposed an empirical formula to achieve a smooth transition between the values of net normalised lateral resistance, $\sigma'_h/\gamma z$, at the surface and at great (infinite) depth.

Broms (1964b) proposes the use of a net normalised lateral resistance, $\sigma'_h/\gamma z$, of K_p over the uppermost 1.5 pile diameters. Failure occurs by the formation of a conventional shallow passive wedge intersecting the soil surface; the apparent neglect of side friction on the wedge is conservative. At depths greater than 1.5 pile diameters, where failure occurs by the flow of soil around the pile, the net limiting lateral pressure is taken as $3K_p\gamma z$. On the basis of data from Barton (1982), Fleming et al (1994) propose an approach similar to Broms (1964b), but with a net normalised lateral resistance of K_p^2 below a depth of 1.5 pile diameters. As explained below, the approach proposed by Broms (1964b) was adopted for the comparative analyses presented in this paper.

The key features of the effective stress limit equilibrium analyses carried out for the present study were that

- the loading system was taken as statically equivalent to a moment $M_{GL} = H_e.e$ and an equivalent lateral load H_e, both acting at the top of the foundation (ground level)
- the pile is of diameter d and total length l, and rotates about a “pivot point” at a depth z_p below ground level
- where the pile moves into the soil (i.e., in front of the pile above the pivot point and behind the pile below it), passive pressures are developed. The net passive pressure is given by K_p times the nominal vertical effective stress ($\gamma^*.z$, where $\gamma^* = \gamma - \frac{du}{dz}$), down to a depth of 1.5 times the pile diameter d; and $3K_p$ times the nominal vertical effective stress below this depth, where K_p is the passive pressure coefficient

- the equations of moment and horizontal force equilibrium were used to find the two unknown depths, l and z_p
- active pressures in zones where the pile is supposed to be moving away from the soil were assumed to be small and were ignored
- the vertical load on the foundation is assumed to be carried in base bearing.

There may be some uncertainty concerning the appropriate value of passive pressure coefficient K_p – in particular, whether it should take a numerical value that reflects possible interface friction between the pile and the soil. This could make a very significant difference. For example, for ϕ' = 30°, K_p without friction is 3 whereas K_p with full interface friction, $\delta = \phi'$, is 5.67.

Brinch Hansen (1961) used values of passive pressure coefficient for a frictional pile having $\delta = \phi'$. In contrast, Fleming et al (1994) used values of K_p based on the classical expression

$$K_p = \frac{(1+\sin\varphi')}{(1-\sin\varphi')} \qquad (11)$$

for a pile / soil interface friction angle $\delta = 0$.

The original work that led to the expressions for limiting lateral resistance of $3K_p$ and K_p^2 was based on tests carried out in sandy soils, with values of angle of shearing resistance ϕ' likely to have been in excess of 30° (i.e., for which $K_p^2 > 3K_p$). Pan et al (2012) reported three dimensional finite element analyses of laterally loaded piles in a soils having ϕ' = 20°. Computed limiting lateral pressures were bracketed by K_p^2 with K_p calculated using a pile/soil interface angle of friction $\delta = \phi'$, and $3K_p$ calculated using $\delta = 0$. This suggests that the conventional approach of taking a limit defined by K_p^2 with K_p calculated using $\delta = 0$ may be conservative for lower strength soils. In general, however, the work of Pan (2013) supports the use of Equation (11) – which assumes a pile / soil interface friction angle $\delta = 0$ – to calculate K_p in the empirical formulae for determining limiting lateral pile pressures, as in Fleming et al (1994).

The use of Equation (11) ($\delta = 0$) is further supported by the likely need to invoke shear stresses acting tangentially on the pile (to carry torque), and / or vertically (to help carry vertical load, if the base bearing pressure is not sufficient). It could thus be unsafe to assume that soil / pile interface shear stresses in possibly a different direction will be available to enhance the lateral stresses on the pile. It was therefore decided to calculate values of K_p using Equation (11), without any enhancement for soil/pile friction effects.

Where the pile is being pushed towards the outer face of an embankment of slope β, Equation (11) may be modified to take account of the rotation in principal stresses that occurs between the pile and the soil surface as a result of the slope. If the ratio of the slope angle β to the effective soil friction angle ϕ' is high, the modified value of K_p may be less than 1. For this reason, three dimensional effects were represented in the current calculations by an effective lateral pressure coefficient of $3K_p$ (following Broms 1964b), rather than K_p^2 (following Fleming et al 1994).

A further point concerns the relationship between the surface zone soil of 1.5 times the pile diameter d (in which the limiting lateral stresses are taken as K_p, rather than $3K_p$ or K_p^2 times the vertical effective stresses) and the UIC / ORE ineffective depth of disturbed, replaced or otherwise unconsolidated soil near the top of the foundation. It seems reasonable that the ineffective surface layer would at least act as a surcharge on the soil below it, enabling the "at depth" failure mechanism to develop below a depth of 1.5 times the pile diameter d. Thus provided that the thickness of the ineffective layer is at least $1.5d$, the lateral stress may reasonably be taken as $3K_p$ times the vertical effective stress at all depths below it. This assumption has been made here; it also has the benefit of simplifying the calculations. The ground in the vicinity of a railway line is likely to be well compacted by trafficking, in which case the assumption of an ineffective zone is conservative.

ii) total stresses

Limiting net total lateral stress distributions for laterally loaded piles, following Fleming et al. (1994, 2009), are shown in Figure 5 for three different ranges of pile length to diameter ratio, l/d.

As with the effective stress analysis,

- the pile is subject to an equivalent horizontal load H_e acting at a height e above the top of the foundation (ground level), giving a moment at ground level $M_{GL} = H_e.e$
- the pile is of diameter d and total length l, and rotates about a "pivot point" at a depth z_p below ground level
- where the pile moves into the soil (i.e., in front of the pile above the pivot point and behind the pile below it), at limiting equilibrium passive pressures are developed. Active pressures where the pile is moving away from the soil are ignored as being insignificant (and possibly tensile)
- the implied failure mechanism in the soil changes from a three-dimensional passive wedge near the surface to soil flowing horizontally around the pile at greater depths, where the vertical stress is sufficient to suppress upward movement of the soil
- the equations of moment and horizontal force equilibrium are used to find the two unknown depths, l and z_p
- a separate check would need to be carried out on the vertical load capacity of the pile.

In this case, the limiting lateral load per unit pile depth, p_u (that is, the limiting lateral total stress, σ_h, multiplied by the pile diameter, *d*) is taken as zero over the assumed ineffective depth of 1.5*d*, then

$$p_u = \left[2 + \frac{7(z-1.5d)}{3d}\right].c_u.d \; for \; depths \; 1.5d \leq z \leq 4.5d \tag{12}$$

and

$$p_u = 9.c_u.d \; for \; depths \; z \geq 4.5d \tag{13}$$

(following Fleming et al 1994).

The maximum value of p_u is just less than that determined analytically for a frictionless pile by Randolph and Houlsby (1984), leaving vertical and tangential shear stress capacity to carry vertical load and torque.

iii) implementation within EC7

In both the effective and total limiting lateral stress analyses, it is necessary in design to apply partial factors as required by EC7 (BSI, 2004). Design Approach 1 (DA1), in which partial factors are applied to actions (A) and ground strength parameters (M), is adopted in the UK. For structures other than axially loaded piles, the designer must verify that "a limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors.

Combination 1: A1 + M1 + R1
Combination 2: A2 + M2 + R1."

The numerical values of Sets A1, A2, M1 and M2 of these partial factors are reproduced in Table 4; the partial factors in Set R1 are numerically equal to unity.

It is generally held that Combination 2 will usually give the most onerous geotechnical conditions (greatest pile length, depth of embedment etc.). However, both combinations must be checked.

Applying the relevant partial factors to the characteristic permanent (self-weight) and variable (wind etc.) loads identified in Table 1 for a wind with a 50 year return period gives the factored loads for each type of structure indicated in Table 5. The corresponding characteristic loads for use in the ORE / OLEMI calculation are also shown.

In the case of OLE support masts, the major variable load is due to wind, which is assessed on a statistical basis as (for example) having a 50 year return period (i.e., a probability of 2%

of being exceeded in any given year). That trains are unlikely to be running in such conditions is reflected in guidance on wind loading of OLE structures (Network Rail 2015b) by the relaxation of the need for the structure to meet the serviceability (displacement) requirement under the 50 year return period wind loading.

COMPARATIVE CALCULATIONS

Calculations have been carried out to compare the results of the OLEMI and limit equilibrium approaches; the latter in both drained (effective stress) and undrained (total stress) conditions (ORE / OLEMI does not draw a distinction). The characteristic loads, topography and soil conditions investigated are summarised in Table 6.

In the effective stress limit equilibrium approach, values of K_p were determined using Equation 14, to account for the effect of the embankment slope angle β ($\beta \leq \phi'$), assuming a soil/pile friction angle $\delta = 0$:

$$K_p = \frac{\sigma'_h}{\gamma z} = \frac{cos^2\beta.(1+sin\theta).e^{2\theta tan\varphi'}}{\left(1-sin\varphi'.cos(\Delta_1-\beta)\right)} \quad (14)$$

where

$$\theta = -\left(\frac{\Delta_1+\beta}{2}\right) \quad (15)$$

and

$$sin\Delta_1 = \frac{sin\beta}{sin\varphi'} \quad (16)$$

Equation (14) was derived using the principles of stress analysis set out in, for example, Powrie (2014). It is approximate, as there may in reality be insufficient space within the embankment to accommodate the implied rotation in principle stress direction between the pile and the soil surface.

In the total stress analysis, the effect of the embankment slope in reducing the limiting lateral earth pressures is more difficult to account for simply, as the geometry of the Mohr circle of stress associated with the undrained shear strength failure criterion leads to a non-linearity with depth. For a horizontal force in the direction away from the track, finite element analyses by Georgiadis and Georgiadis (2010) suggest that the slope may reduce the ultimate lateral resistance to pile movement by a maximum of about 20%, while field tests by Nimityongskul et al (2018) show that the slope has no noticeable effect for piles installed on the crest at or beyond a horizontal distance of eight pile diameters from the top of the slope. However, for OLE structures, the greater horizontal force is usually in the direction towards the track. Finite element analyses by Kanagasabai et al (2011) suggest that in this case, where the net lateral stress on the pile is acting downslope, a slope angle of up to at least 22° makes little difference to the ultimate lateral pressure on the pile. Hence results of total stress analyses are presented only for level ground, β = 0.

In all calculations, the pile was circular in cross section with an outside diameter d = 0.61 m, and an ineffective depth of 0.915 m (1.5 times the pile diameter) was assumed.

The limit equilibrium calculations were carried out using the factored loads (C1 or C2) indicated in Table 5, and factored soil strengths (C2) as required by EC7. In all cases, C2 gave greater pile lengths than C1. Calculations were carried out for across-track wind loads acting both towards and away from the track. In most cases, the first (where the wind moment acts with the structural load) gave the greater pile length. However at steeper embankment slopes, the second (with a smaller applied moment) gave the greater pile length, because the dominant effect was the reduced ground resistance due to the presence of the slope when the pile is pushed outward towards the surface of the embankment.

No further factors were applied in the OLEMI calculations, as the required load factors are already included in the equations.

Results are presented for the effective stress limit equilibrium analyses in drained ground with zero pore water pressures, as graphs of the required pile embedment depth l against the embankment slope angle β, for values of ϕ' of 20°, 32.5° and 45°, for the STC, the TTC and the XL-TTC in Figures 6, 7 and 8 respectively. The corresponding results for waterlogged ground with hydrostatic pore water pressure conditions ($\frac{du}{dz} = 10kPa/m$) are shown in Figures 9, 10 and 11 respectively.

Results for the undrained (total stress) limit equilibrium analyses are presented non-dimensionally in Figure 12 as graphs of the required pile length to diameter ratio (l/d) against normalised horizontal load $H_e/c_u.d^2$, for normalised heights of action of H_e (e/d) of 3.5, 8.5 and 13.5, (which covers the range for the OLE masts under consideration: see Table 5). Numerical values of pile length l, including the ineffective depth of 1.5d, for the STC, TTC and XL-TTC with the appropriate partial factors applied to loads and soil strengths are compared with the OLEMI-derived lengths for level ground (β = 0) in Table 7. In the limit equilibrium calculations in level ground, towards-track loading with the partial factors associated with DA1 Combination 2 (including a factor of 1.4 on the undrained shear strength of the soil) was always the most onerous condition.

COMMENTARY

Generally, the results of the two methods of calculation are, perhaps surprisingly, broadly consistent.

Effective stress analyses

For the effective stress analyses,

- in level ground (β = 0) with zero pore water pressures, the OLEMI calculation gives an embedment depth just less than the limit equilibrium analysis (LEA) with ϕ' = 32.5° for the STC (Figure 6), just greater than the LEA with ϕ' = 32.5° for the TTC (Figure 7), and greater than the LEA ϕ' = 20° for the XLTTC (Figure 8)
- in waterlogged level ground (β = 0) with hydrostatic pore water pressures, the difference between the two calculations for the STC (Figure 9) is increased slightly (i.e. the OLEMI calculation is relatively more unconservative), but for the TTC (Figure 10) and the XL-TTC (Figure 11) there is no real relative change
- as the slope angle is increased, the OLEMI calculation (which is insensitive to slope angles β < 20°) gives increasing more unconservative (i.e., relatively shorter) pile lengths (Figures 6-11)
- at a slope of 20°, there is a step change in the OLEMI pile length which makes it greater than that calculated by the limit equilibrium analysis for ϕ' = 32.5°and ϕ' = 45° (Figures 6-11)
- as the slope angle is increased towards 45°, the OLEMI calculation for the STC (Figures 6 and 9) gradually becomes unconservative again, first relative to the limit equilibrium calculation for ϕ' = 32.5° and then to that for ϕ' = 45°. However, this does not occur for the TTC (Figures 7 and 10) or the XL-TTC (Figures 8 and 11). For these structures, the OLEMI calculation consistently gives pile depths greater than the limit equilibrium analyses for ϕ' = 32.5°and ϕ' = 45°, except for the TTC with slope angles greater than about 25° (waterlogged, Figure 10) or 26° (zero pore water pressures, Figure 7) and ϕ' = 32.5° (Figures 7 and 10). Also, it must be recognised that the maximum slope using the OLEMI approach of 45° is greater than the maximum slope permitted with any of the effective stress analyses having $\phi' \leq$45° (Figures 6-11).

It is notable that, for the XL-TTC (Figures 8 and 11), the OLEMI calculation always gives a greater pile depth than the limit equilibrium analysis except for moderate slopes with ϕ' = 20°.

Total stress analyses

For the total stress analyses in level ground (Table 7),

- the OLEMI calculation gives pile lengths between those calculated using the LEA for undrained shear strengths of 60 kPa and 30 kPa; the equivalent undrained shear strength reduces from a little less than 60 kPa for the STC through perhaps 48 kPa for the TTC to less than 30 kPa for the XL TTC.

Thus the OLEMI approach becomes relatively less unconservative or even more conservative compared with the LEA as the foundation moment is increased, in both the undrained and drained analyses.

Summary

Overall, the degree of unconservatism of the OLEMI approach compared with the limit equilibrium analysis is only really a potential concern for relatively small loads (i.e., the STC) in soil having a relatively low effective angle of shearing resistance ($\phi' \sim 20°$). These soils are likely to be clays, for which an undrained analysis of their response to short term, transient loading is likely to be appropriate. In these conditions, the OLEMI method is generally conservative, unless the undrained shear strength of the ground is low (less than ~57 kPa for the STC, and less than ~27 kPa for the XL-TTC). It must also be recalled that the Broms-type limit equilibrium calculation is itself far from rigorous, especially for the effective stress analysis; it involves significant assumptions, approximations, empiricism and uncertainty, as has already been discussed in this paper.

DISCUSSION AND FURTHER ANALYSIS IN THE CONTEXT OF THE GREAT WESTERN ELECTRIFICATION PROJECT

Given the broad consistency between the results of the limit equilibrium and OLEMI methods, in both drained (zero pore water pressure) and waterlogged (hydrostatic) conditions, the question arises – although not the main thrust of this paper – as to why the early design calculations associated with the Great Western Electrification Project seemed to result in excessive pile embedment depths. For example, as-built drawings and records for OLE foundations installed at Langley, Berkshire indicate installed pile lengths of 6.5 m to 7.0 m for 0.762 m dia. CHS piles subject to a characteristic across track moment (associated with a TTC) of 140 kNm. Based on γ*=18 kN/m^3, an ineffective depth h' = 1.143 m (1.5d), the OLEMI-calculated lengths for piles located on the crest of an embankment (k = 0.95); level ground (k = 1.3) and in a cutting (k = 1.8) are 3.87 m, 3.24 m and 2.89 m respectively. For waterlogged ground with γ*= 8 kN/m^3, the OLEMI calculated lengths are 4.75 m on an embankment (k = 0.95); 3.99 m in level ground (k = 1.3); and 3.51 m in a cutting (k = 1.8).

Calculations seen by the authors of this paper suggest two main reasons for the specification of such long pile lengths. These were overconservatism in (i) the specified limiting lateral earth pressure coefficient (we have suggested the use of 3K_p, calculated using a soil/pile friction angle δ =0); and (ii) the equivalent linear soil stiffness parameters used in SLS pile-soil interaction analyses.

i) limiting lateral earth pressure coefficient

Calculations carried out using the program WALLAP for structures between Stockley and Maidenhead appear to have used a limiting passive earth pressure coefficient K_p based on a pile / soil friction angle δ/ϕ' of $\approx$ 0.45, but without the enhancement factor of 3 to allow for three-dimensional effects. The results of further limit equilibrium calculations summarised in Figures 13-16 show that, in level ground, using δ/ϕ' = 0.45 makes very little difference compared with using $\delta = 0$, but that the neglect of the empirical factor of 3 on K_p to allow for three dimensional effects generally increases the calculated pile length by about 50% (compare Figure 6 with Figure 13 for a STC with a low GWL, Figure 8 with Figure 14 for an XLTTC with a low GWL, Figure 9 with Figure 15 for a STC with a high GWL, and Figure 11 with Figure 16 for an XLTTC with a high GWL).

ii) effective (linear) soil stiffness

The second issue arose from an attempt to carry out SLS assessments aimed at limiting the calculated structural deflection at wire height. The serviceability limit state was in the first initially rather poorly defined, owing to uncertainty about the permissible deflection (given the liveliness of the catenary itself, and the lateral stagger that is designed-in to even out pantograph wear).

A further problem is that the calculated deflection and rotation at the pile head will often depend on the equivalent linear elastic soil stiffness chosen for use in the calculation. For example, Krechowiecki-Shaw and Alobaidi (2015) report the results of serviceability limit state (SLS) calculations carried out using an effective Young's modulus of 15 MPa for what is described as a medium dense sand. Young's moduli used in the calculations for structures between Maidenhead and Stockley Park (other than in the surface zone) ranged from 12 MPa for a loose sandy gravel to 60 MPa for a medium dense gravelly sand.

It is difficult to find representative stiffnesses for granular soils in the literature. Nonetheless, recognising the EC7 requirement for the parameter value used in calculations to reflect the limit state under consideration (in this case, the SLS and therefore at small or moderate strains), some of these values do seem to be more appropriate to much less competent materials; for example, Duley (2018) measured an equivalent small strain Young's modulus of 12 MPa for a very soft organic silt of bulk density 1.2 Mg/m^3 at a confining stress of 17 kPa.

To investigate the influence of foundation pile length and assumed soil stiffness on this potential serviceability limit state, the component of the horizontal deflection of the track support structure at wire height resulting from the deflection / rotation of the pile head was estimated using the OASYS program ALP (Analysis of Laterally loaded Piles), as summarised in Appendix 3. Both an STC and an XLTTC were considered, with soil Young's moduli of 15, 30 and 60 MPa and pile lengths equal to 1$\times$ and 2$\times$ those calculated using the OLEMI method for hydrostatic (i.e., worst case) pore water pressures: these loading and ground conditions are summarised in Table 8.

The results of the ALP calculations, summarised in Figures 17 and 18 with the calculated wire height deflections given in Table 9, show that

- In all cases, the calculated across-track wire height deflection due to the variable component of load is less than the limit of 50 mm specified in Network Rail Standard NR/SP/ELP/27215 (Network Rail)
- for the STC and a 3.5 m pile length, the calculated deflection is approximately linearly dependent on the assumed soil stiffness. This is unsurprising, as the 3.5 m pile is effectively "short", so that the mode of deformation is essentially by rotation about a pivot point near the toe. Doubling the length to 7 m changes the pile behaviour from "short" to "long" and the mode of deformation from rigid body rotation to bending with the bottom end effectively fixed. This brings a significant reduction in the calculated wire height deflection.
- for the XL-TTC, the calculated wire height deflection is relatively insensitive to changes in soil stiffness and an increase in the pile length. This is because the pile is already "long", with its lower end largely fixed. The effectiveness of the fixity is improved slightly by the increase in soil stiffness from 15 MPa to 30 MPa, but deformation is predominantly due to pile bending and the effect of the soil stiffness is small.

For simple structures such as the majority of OLE mast foundations, an attempt to carry out a separate SLS calculation with inappropriate soils parameters is almost certainly misconceived. EC7 (2014a) Clause 2.4.8 (4) does not require an explicit SLS calculation provided:

> "It may be verified that a sufficiently low fraction of the ground strength is mobilised to keep deformations within the required serviceability limits, provided this simplified approach is restricted to design situations where established comparable experience exists with similar ground, structures and application method."

Achieving this criterion was the original purpose of the "load factor" – usually applied as a strength factor in geotechnical engineering analysis of laterally loaded piles and retaining walls – in a plasticity-based design (see, e.g., Baker and Heyman, 1969). It is the authors' opinion that for simple embedded retaining walls and laterally loaded piles, the partial factors specified in EC7 Combination 2 as interpreted through the UK National Annex were intended to give results broadly compatible with earlier approaches, whose sufficiency has been demonstrated by many decades of experience of satisfactory structural performance in both stability and serviceability. By comparing the results of ULS calculations to the empirical OLEMI approach, the work described in this paper then justifies the use of Clause 2.4.8(4) for the design of OLE foundations.

CONCLUSIONS

In level or sloping ground, factored limit equilibrium analyses in terms of total stresses, and in terms of effective stresses with zero or hydrostatic pore pressures, give results that are broadly comparable with the OLEMI method in terms of the required pile embedment depth, for a variety of soil strengths.

1. The fact that the limit equilibrium ULS analyses give broadly similar results and are sometimes more conservative than the proven OLEMI method shows that for these types of structure, the limit equilibrium calculation is very robust.
2. The OLEMI approach is likely to give shorter piles than the factored limit equilibrium calculation for smaller applied loads (i.e., a single track cantilever, STC), soils of lower effective angle of shearing resistance, and intermediate slope angles in the range 10° - 20°. For soil strengths greater than about 30°, higher loads (i.e., a twin track cantilever, TTC, and an extra-large twin track cantilever, XL-TTC), and slope angles less than about 10°or greater than 20°, the OLEMI method may give slightly longer piles than the factored limit equilibrium analysis.
3. The effect of pore water pressures on the conservatism or otherwise of the OLEMI approach relative to the factored effective stress limit equilibrium analysis is negligible.
4. Undrained limit equilibrium analysis, which for transient and short-term loading in clay soils having relatively low angles of effective shearing resistance is likely to be more appropriate than effective stress analysis, gives shorter pile lengths than the OLEMI method for soils having undrained shear strengths greater than ~57 kPa for the loads associated with the STC, and greater than about ~27 kPa for the loads associated with the XL-TTC.
5. The apparent overdesign of the Great Western Electrification Project foundations appears to have arisen largely because of an attempt to carry out an explicit serviceability limit state (SLS) calculation using over-conservative soil stiffnesses, and / or carrying out limit equilibrium ULS calculations that made no allowance for three-dimensional effects.
6. The satisfactory performance of a large number of OLEMI-designed foundations provides further evidence that a specific SLS check for this type of relatively simple structure is not required.
7. The comparative calculations should give designers the confidence to use the OLEMI method, or limit equilibrium analysis with the partial factors specified in EC7. This should result in shorter pile lengths that will perform adequately, helping to reduce electrification costs back towards historic levels.

ACKNOWLEDGEMENTS
The work described in this paper was funded and carried out under the Network Rail – University of Southampton Strategic Partnership in Future Infrastructure Systems. The authors are grateful to colleagues including Dr Anthony Blake, Dr Tony Lock and Rob Stacey for their discussions, insights and help with various parts of the paper, and to Dr Joel Smethurst for carrying out the ALP analyses.

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APPENDIX 1: SYMBOLS AND NOMENCLATURE

The key geometrical parameters and symbols used in the source documents for each method are summarised in Table A1.1. Symbols given in square brackets [] do not feature in the source document but are shown for completeness and/or overall clarity.

Other key terms and symbols are defined in Table A1.2.

APPENDIX 2: OLEMI & ALLOCATION DESIGN METHODOLOGY

It was stated in the main text that the OLEMI method gives foundations that are approximately 10 to 20% longer than those determined using the UIC-ORE design method as a result of simply neglecting the vertical loads (*Nr* in equations 2 and 4). All other corrections for topographical features and foundation type are the same in both methods.

The updated Network Rail specification for the Design and Installation of Overhead Line Foundations (NR/L2/CIV/074, 2018) uses the UK Master Series (Network Rail, 2015) foundation allocation method derived from OLEMI (ie no vertical load is considered).

A series of 'strength depth' tables for allocating concrete 'grabbed' (i.e., excavated and cast in-place) side bearing foundations (parallelepiped) and augured tubular steel pile foundations are provided in drawing MS/B80/L00/A3. These tables provide the foundation lengths in increments of 0.1 m and the corresponding level ground moment based on a non-effective near-surface depth h' = 0.3 m and a soil effective unit weight γ^* = 15 kN/m^3. For grabbed side bearing foundations a towards track moment ie level terrain factor K = 1.3 is used to calculate the foundation 'strength depth' (D). For the tubular steel pile foundations, an away from track moment (ie level terrain factor K = 1) is used resulting in increased foundation 'strength depths' for a given level ground moment. This apparent discrepancy accommodates the likely ground disturbance in pre-auguring the ground to facilitate tubular steel pile (CHS) installation. "Augured" is a term used only in the title of drawing MS/B98/K08/A3.

To facilitate foundation allocation according to in-service foundation type, terrain condition and loading direction, two allocation schedules for grabbed and hand excavated (parallelepiped) and augured side bearing (tubular steel) foundations are provided in drawings MS/B98/K05/A3 and MS/B98/K08/A3 respectively. These allocation schedules introduce a modification Factor F that converts the level ground moment to an equivalent level ground moment that reflects the actual direction of loading and terrain type. This equivalent level ground moment and its corresponding foundation 'strength depth' is derived from the appropriate table in drawing MS/B80/L00/A3. The additional corrections to foundation length based on slope angle, direction of loading, proximity to topographical features related to foundation type are also depicted in MS/B98/K05/A3 and

MS/B98/K08/A3 and are in accordance with UIC-ORE rules. These are replicated in Tables A2.2(a) and A2.2(b) respectively.

Table A2.3 shows that the application of the Factor *F* values provided in Table A2.2(a) (MS/B98/K05/A3: Grabbed) converts the across track level ground moment to the appropriate ORE *K* factor for each terrain type and loading direction as presented in Table 2. The Factor *F* values provided in Table A2.2(b) (MS/B98/K08/A3: Augured) are identical but because the 'strength depth' table in MS/B80/L00/A3 has been derived using a *K* = 1 (away from track moment), the equivalent ORE *K* factor is reduced resulting in foundation lengths approximately 17% and 23% longer for 0.610 m and 0.762 m CHS foundations respectively.

APPENDIX 3: USE OF THE LATERAL PILE / SOIL INTERACTION PROGRAM ALP TO ILLUSTRATE THE EFFECT OF THE ASSUMED SOIL STIFFNESS ON CALCULATED WIRE HEIGHT DEFLECTIONS

To investigate the influence of foundation pile length and assumed soil stiffness on this potential serviceability limit state, the deflection at wire height was estimated using the OASYS program ALP (Analysis of Laterally loaded Piles).

ALP uses rudimentary soil-structure interaction to calculate the deflected PILE shape (lateral displacement and rotation), shear forces and bending moments and lateral soil pressures, in response to the application of loads or the imposition of soil displacements. The soil is represented by non-linear springs and the pile as elastic beam elements. The program is limited to laterally loaded piles in level ground. Load-deflection behaviour is modelled either by assuming elastic-plastic soil behaviour or by specifying load-deflection (*P*-*y*) curves: in the current analyses, the former approach was adopted. Analyses are very quick to undertake as only two stiffness matrices are developed, representing the pile in bending and the soil stiffness.

In the current analyses, soil stiffnesses were taken as integer multiples (×1, ×2 and ×4) of the value of 15 MPa used by Krechowiecki-Shaw and Alobaidi (2015).

The lateral displacement at contact wire height (assumed to be 5.2 m above ground level) was determined by multiplying the calculated rotation of the top of the pile by the distance from the effective top of the pile to the contact wire (5.5 m), and adding this to the calculated lateral displacement of the effective top of the pile.

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interface friction angle δ = 0. All necessary load and strength factors applied. Soil unit weight 18 kN/m^3 (effective unit weight 8 kN/m^3). *Note: the soil strengths indicated apply only to the limit equilibrium calculations - the OLEMI method does not use the soil strength, only the unit weight / density*

Figure 10: Pile length *l* as a function of slope angle β, for soil effective friction angles of 20°, 32.5° and 45°. Twin Track Cantilever, effective stress limit equilibrium and OLEMI analyses with hydrostatic pore water pressures below a water table at the ground surface. Soil / pile interface friction angle δ = 0. All necessary load and strength factors applied. Soil unit weight 18 kN/m^3 (effective unit weight 8 kN/m^3). *Note: the soil strengths indicated apply only to the limit equilibrium calculations - the OLEMI method does not use the soil strength, only the unit weight / density*

Figure 11: Pile length *l* as a function of slope angle β, for soil effective friction angles of 20°, 32.5° and 45°. Extra Large Twin Track Cantilever, effective stress limit equilibrium and OLEMI analyses with hydrostatic pore water pressures below a water table at the ground surface. Soil / pile interface friction angle δ = 0. All necessary load and strength factors applied. Soil unit weight 18 kN/m^3 (effective unit weight 8 kN/m^3). *Note: the soil strengths indicated apply only to the limit equilibrium calculations - the OLEMI method does not use the soil strength, only the unit weight / density*

Figure 12: Dimensionless pile length l/d as a function of dimensionless load $H_e/c_u.d^2$, for normalised heights of action of H_e (e/d) of 3.5, 8.5 and 13.5; total stress limit equilibrium analyses (slope β= 0, ineffective pile depth of 1.5 pile diameters included).

Figure 13: Pile length *l* as a function of slope angle β, for soil effective friction angles of 20°, 32.5° and 45°. Single Track Cantilever, effective stress limit equilibrium analyses with the empirical enhancement factor of 3 to allow for three dimensional effects omitted. Soil / pile interface friction angle δ = 0. All necessary load and strength factors applied. Zero pore water pressures, soil unit weight 18 kN/m^3. The results of calculations for a slope angle β = 0 and δ/ϕ' = 0.45 are also shown.

Figure 14: Pile length *l* as a function of slope angle β, for soil effective friction angles of 20°, 32.5° and 45°. Extra Large Twin Track Cantilever, effective stress limit equilibrium analyses with the empirical enhancement factor of 3 to allow for three dimensional effects omitted. Soil / pile interface friction angle δ = 0. All necessary load and strength factors applied. Zero pore water pressures, soil unit weight 18 kN/m^3. The results of calculations for a slope angle β = 0 and δ/ϕ' = 0.45 are also shown.

Figure 15: Pile length *l* as a function of slope angle β, for soil effective friction angles of 20°, 32.5° and 45°. Single Track Cantilever, effective stress limit equilibrium analyses with

hydrostatic pore water pressures below a water table at the ground surface, with the empirical enhancement factor of 3 to allow for three dimensional effects omitted. Soil / pile interface friction angle $\delta = 0$. All necessary load and strength factors applied. Soil unit weight 18 kN/m^3 (effective unit weight 8 kN/m^3). The results of calculations for a slope angle $\beta = 0$ and $\delta/\phi' = 0.45$ are also shown.

Figure 16: Pile length *l* as a function of slope angle β, for soil effective friction angles of 20°, 32.5° and 45°. Extra Large Twin Track Cantilever, effective stress limit equilibrium analyses with hydrostatic pore water pressures below a water table at the ground surface, with the empirical enhancement factor of 3 to allow for three dimensional effects omitted. Soil / pile interface friction angle $\delta = 0$. All necessary load and strength factors applied. Soil unit weight 18 kN/m^3 (effective unit weight 8 kN/m^3). The results of calculations for a slope angle $\beta = 0$ and $\delta/\phi' = 0.45$ are also shown.

Figure 17: Calculated pile deflections using ALP analysis - Single Track Cantilever. Pile lengths and soil stiffnesses as indicated. Limiting lateral earth pressure coefficient = 9 (=3K_p; $\phi' = 30°$ and $\delta = 0$).

Figure 18: Calculated pile deflections using ALP analysis – XL-Twin Track Cantilever. Pile lengths and soil stiffnesses as indicated. Limiting lateral earth pressure coefficient = 9 (=3K_p; $\phi' = 30°$ and $\delta = 0$).

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Figure 1a

Click here to access/download;Figure;Fig 1A.jpg

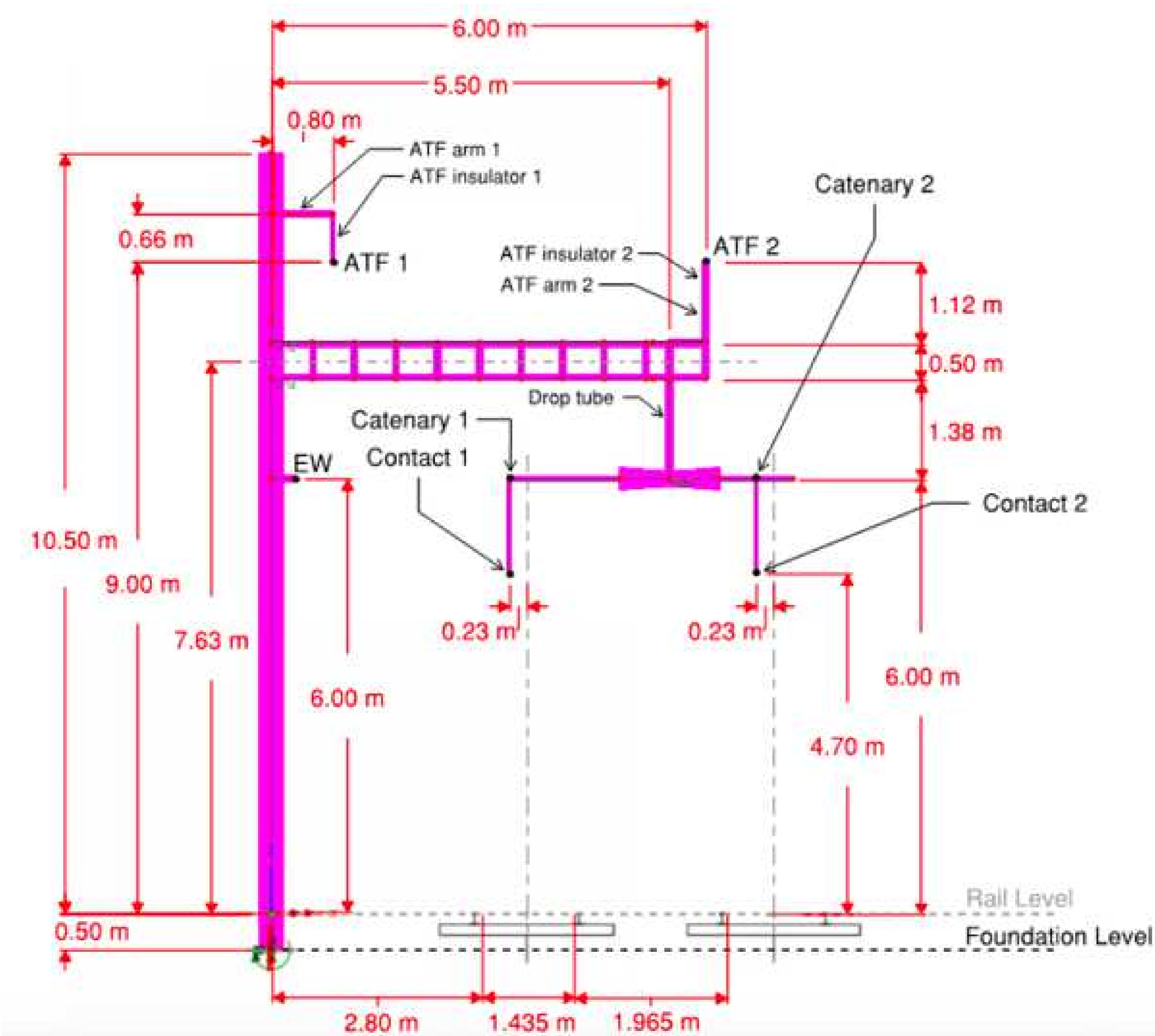


Figure 1b

Click here to access/download;Figure;Figure 1b.JPG

Figure 2

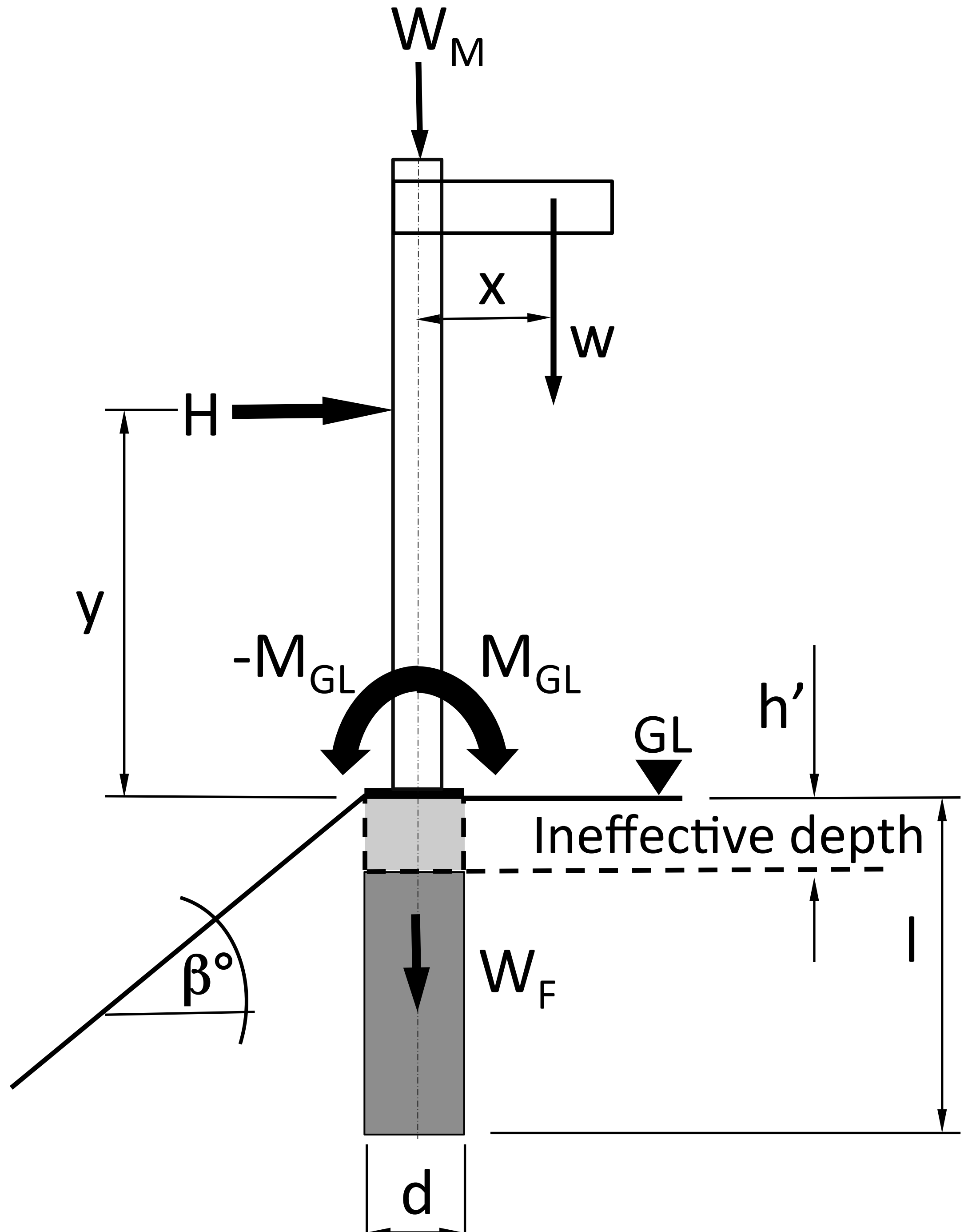


Figure 3a

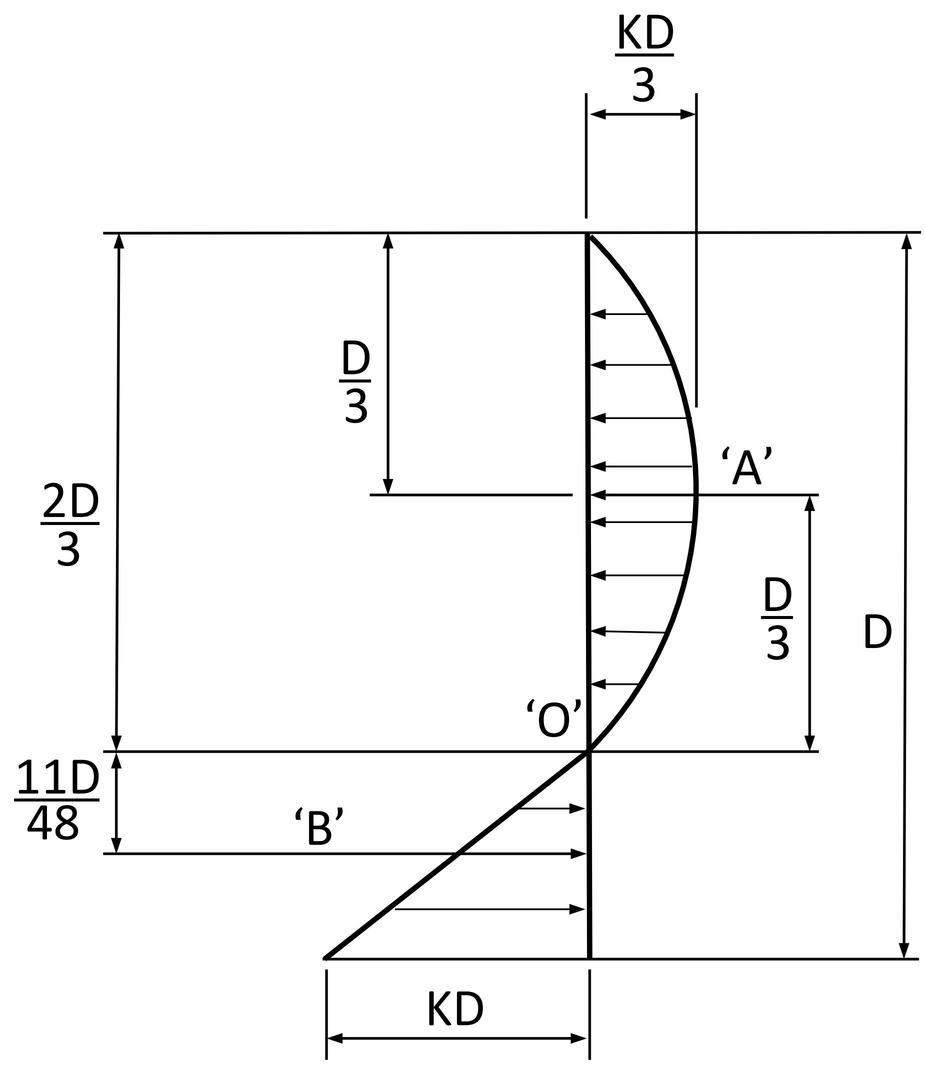


Figure 3b

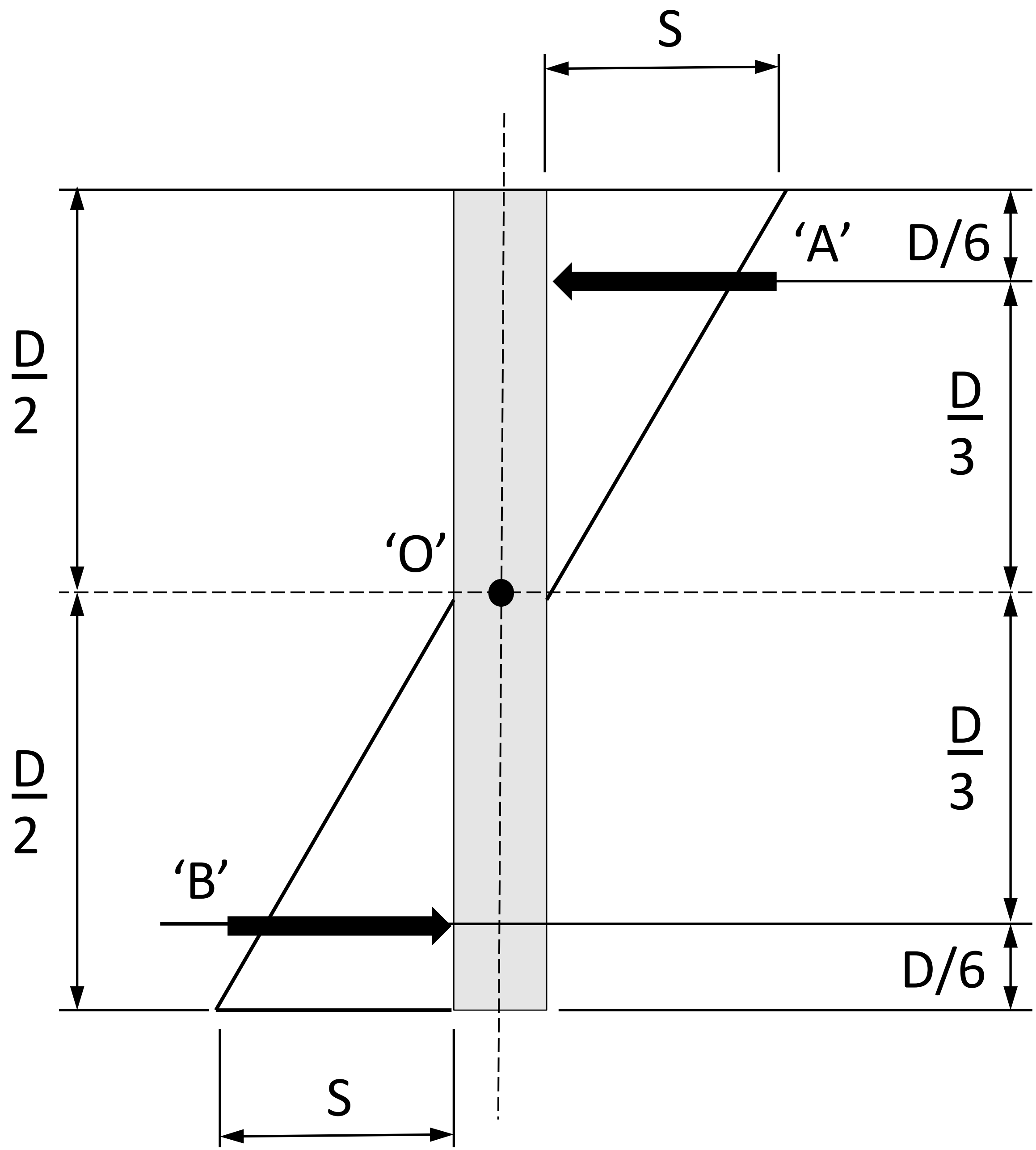


Figure 4

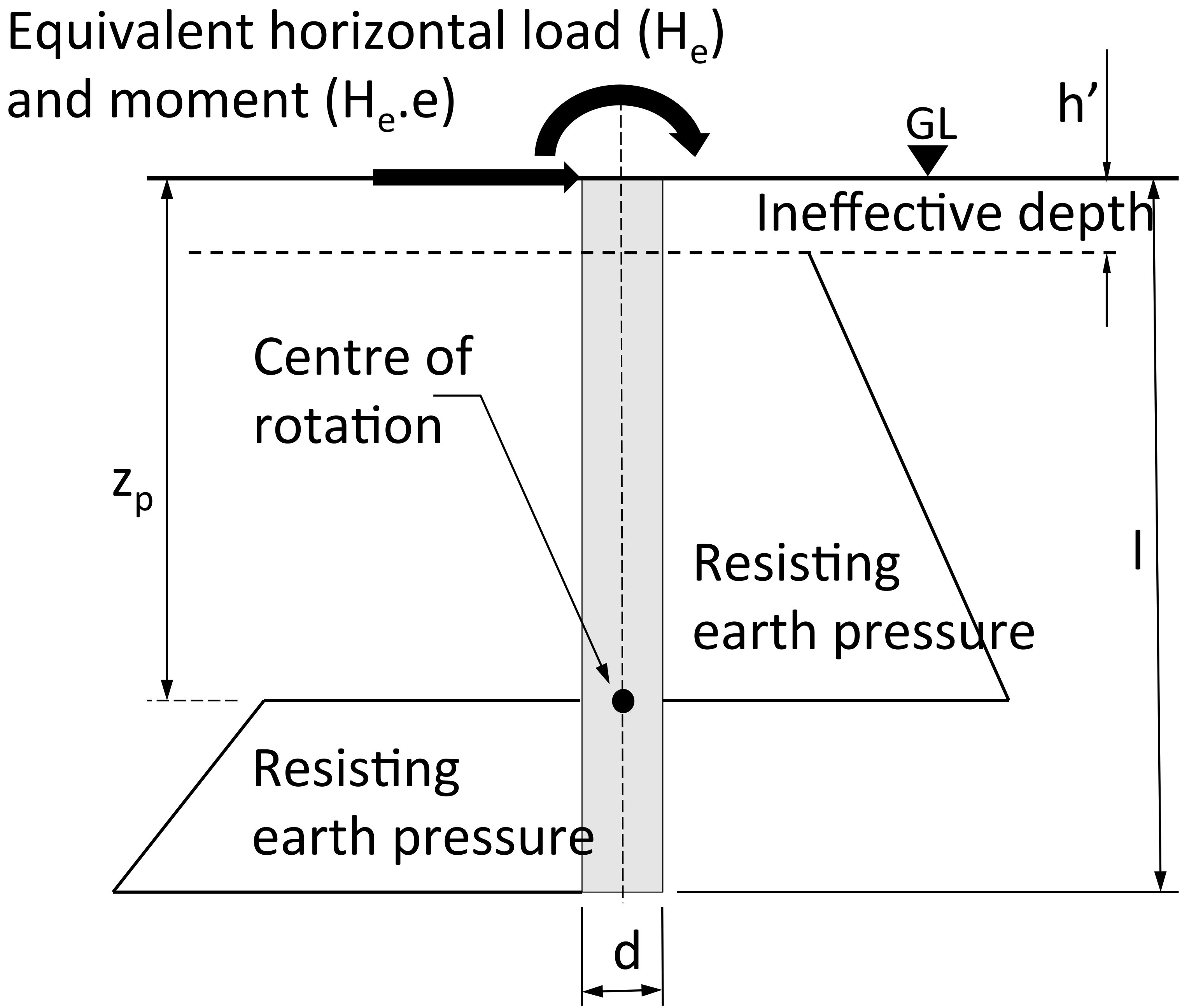
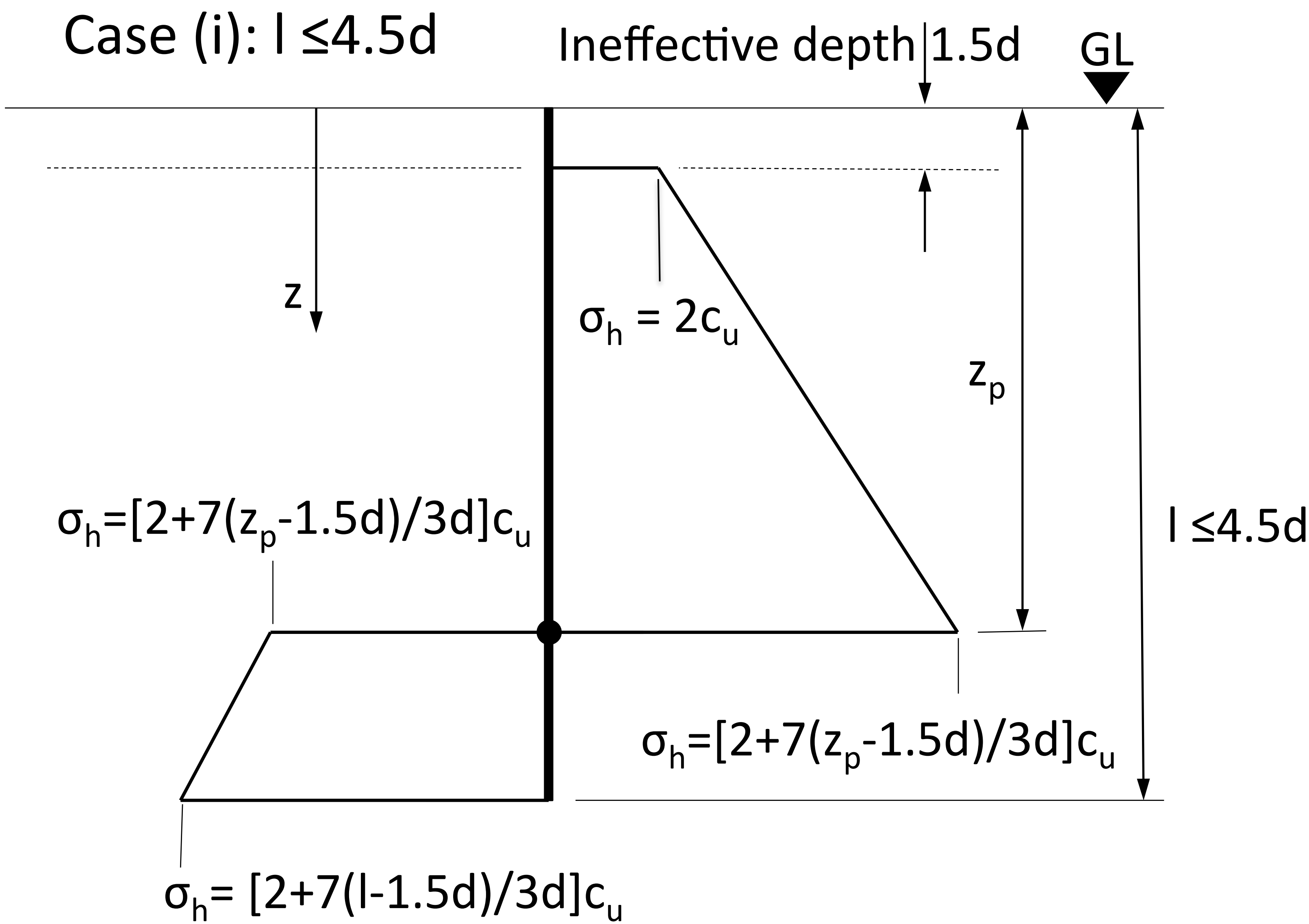
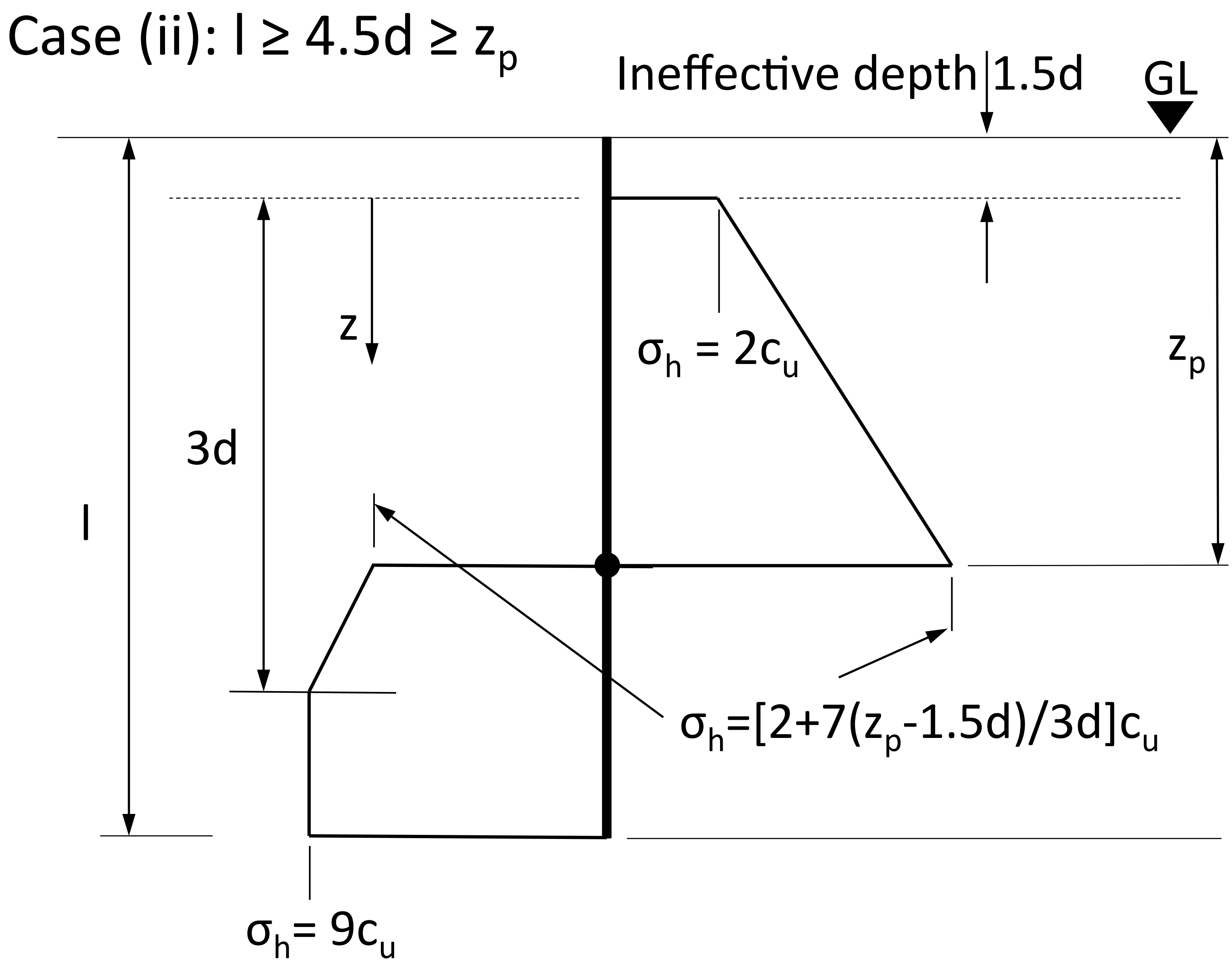

Equivalent horizontal load (H_e)
and moment (H_e.e)
GL
h'
Ineffective depth
Centre of rotation
z_p
Resisting earth pressure
l
Resisting earth pressure
d

Figure 5 (i)



Case (i): l ≤4.5d
Ineffective depth 1.5d
GL
z
σ_h = $2c_u$
z_p
σ_h=[2+7(z_p-1.5d)/3d]c_u
l ≤4.5d
σ_h=[2+7(z_p-1.5d)/3d]c_u
σ_h= [2+7(l-1.5d)/3d]c_u

Figure 5 (ii)



Case (ii): l ≥ 4.5d ≥ z_p
Ineffective depth 1.5d
GL
z
3d
l
σ_h = 2c_u
z_p
σ_h=[2+7(z_p-1.5d)/3d]c_u
σ_h= 9c_u

Figure 5 (iii)

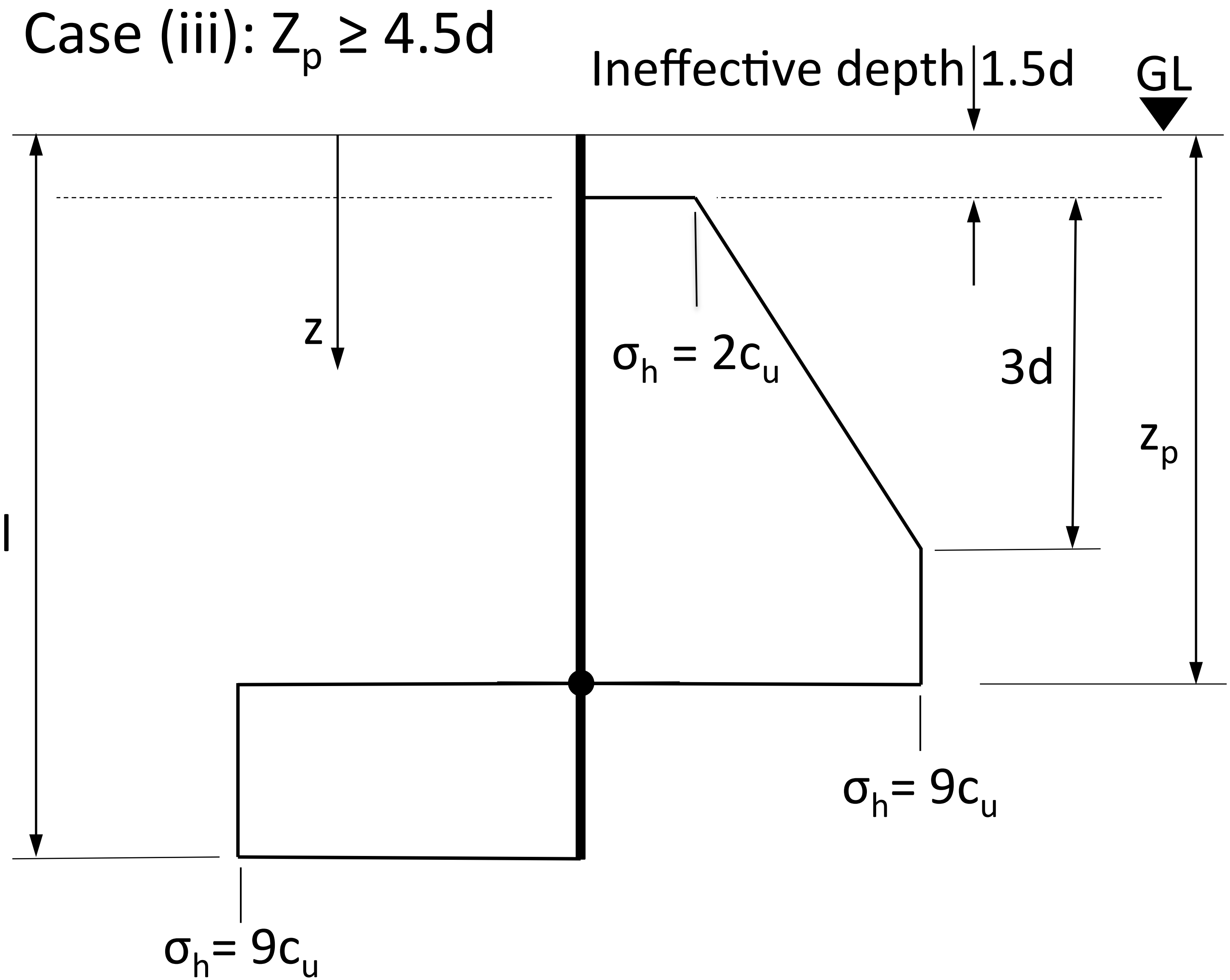
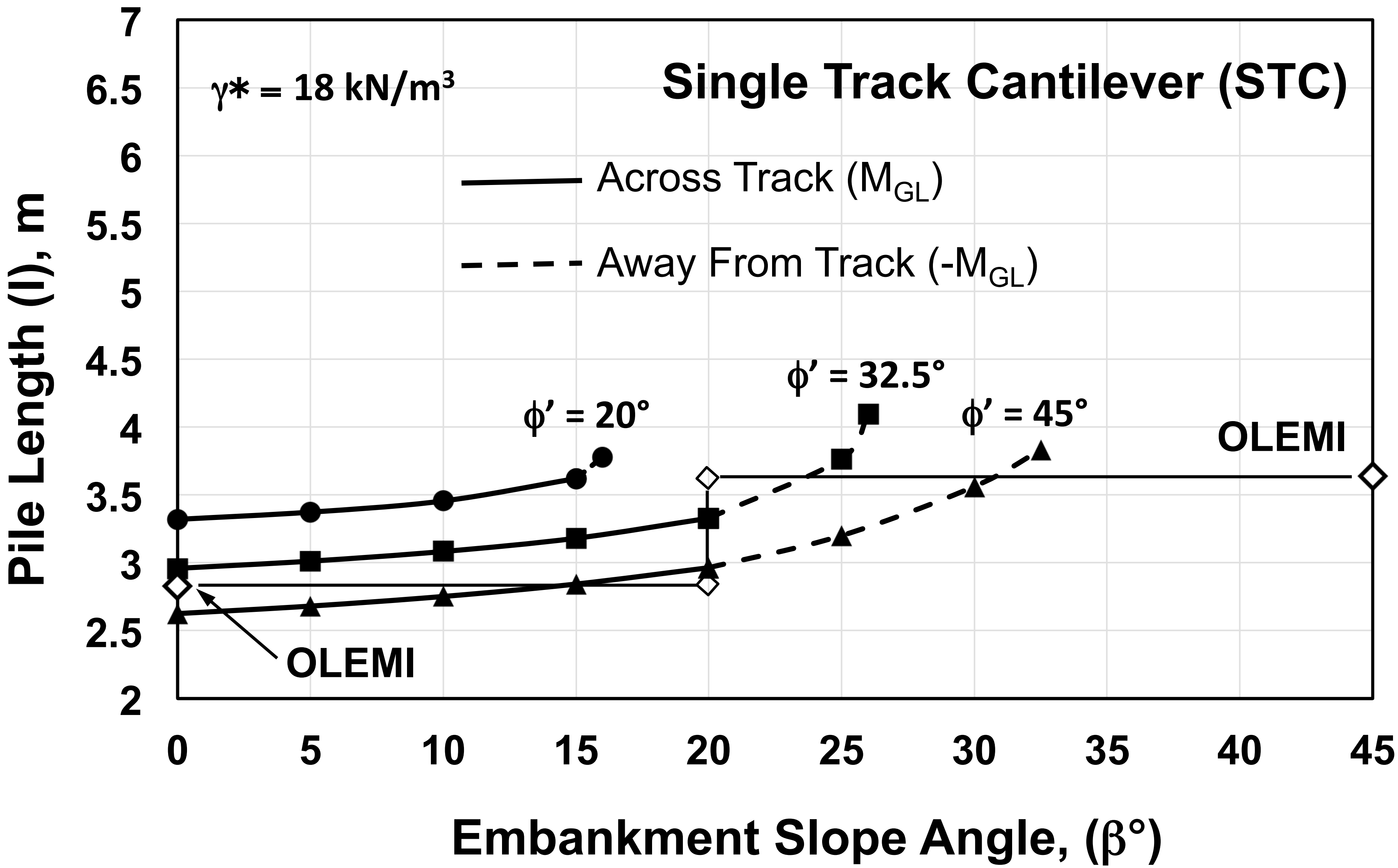
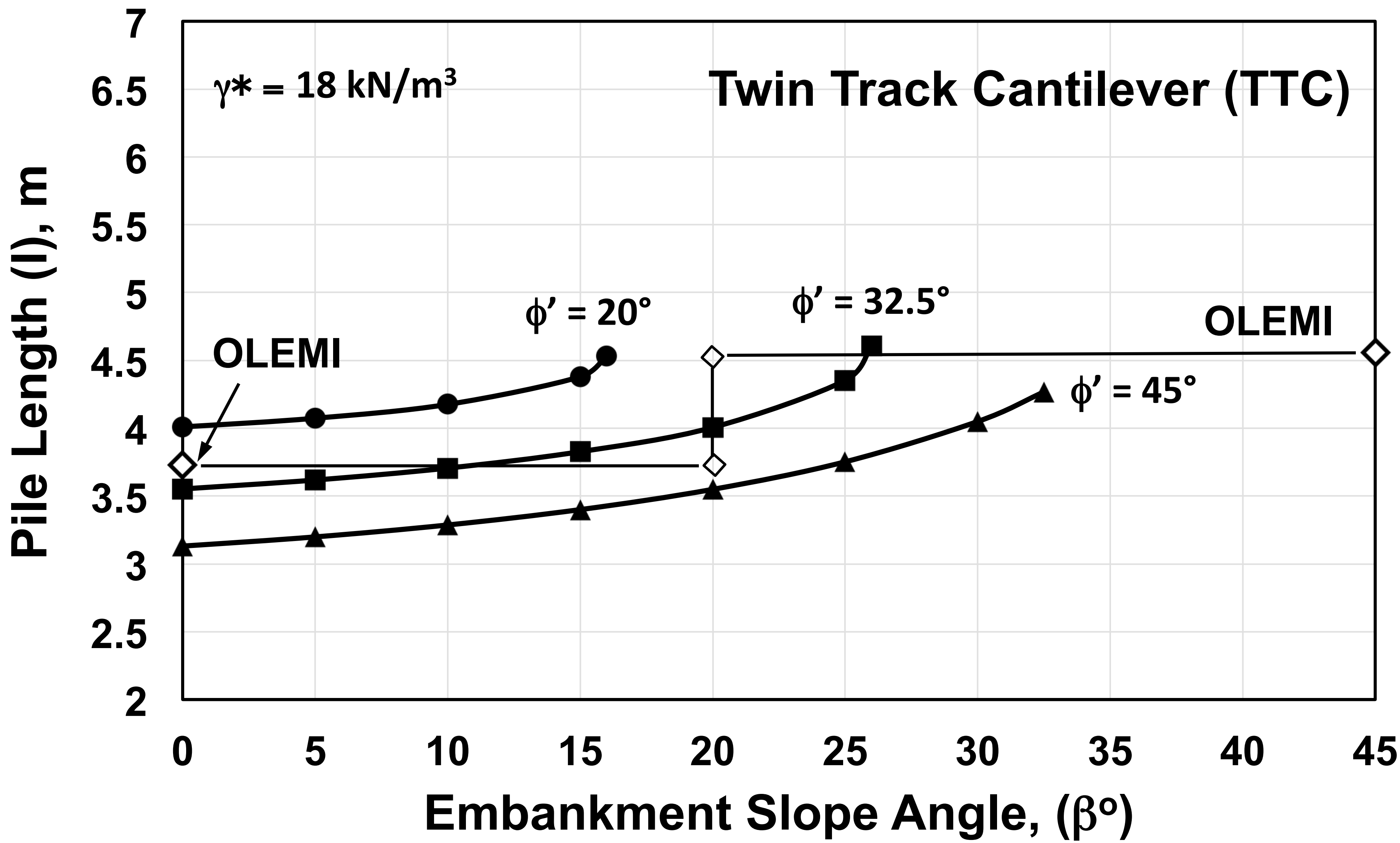

Case (iii): Z$_p$ ≥ 4.5d
Ineffective depth 1.5d
GL
z
σ$_h$ = 2c$_u$
3d
z$_p$
l
σ$_h$= 9c$_u$
σ$_h$= 9c$_u$

Figure 6



Single Track Cantilever (STC)
γ* = 18 kN/m³
Across Track (M_{GL})
Away From Track ($-M_{GL}$)
φ' = 20°
φ' = 32.5°
φ' = 45°
OLEMI
OLEMI
Pile Length (l), m
Embankment Slope Angle, (β°)
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
0
5
10
15
20
25
30
35
40
45

Figure 7



Twin Track Cantilever (TTC)
γ* = 18 kN/m³
φ' = 20°
φ' = 32.5°
φ' = 45°
OLEMI
OLEMI
Pile Length (l), m
Embankment Slope Angle, (β°)
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
0
5
10
15
20
25
30
35
40
45

Figure 8

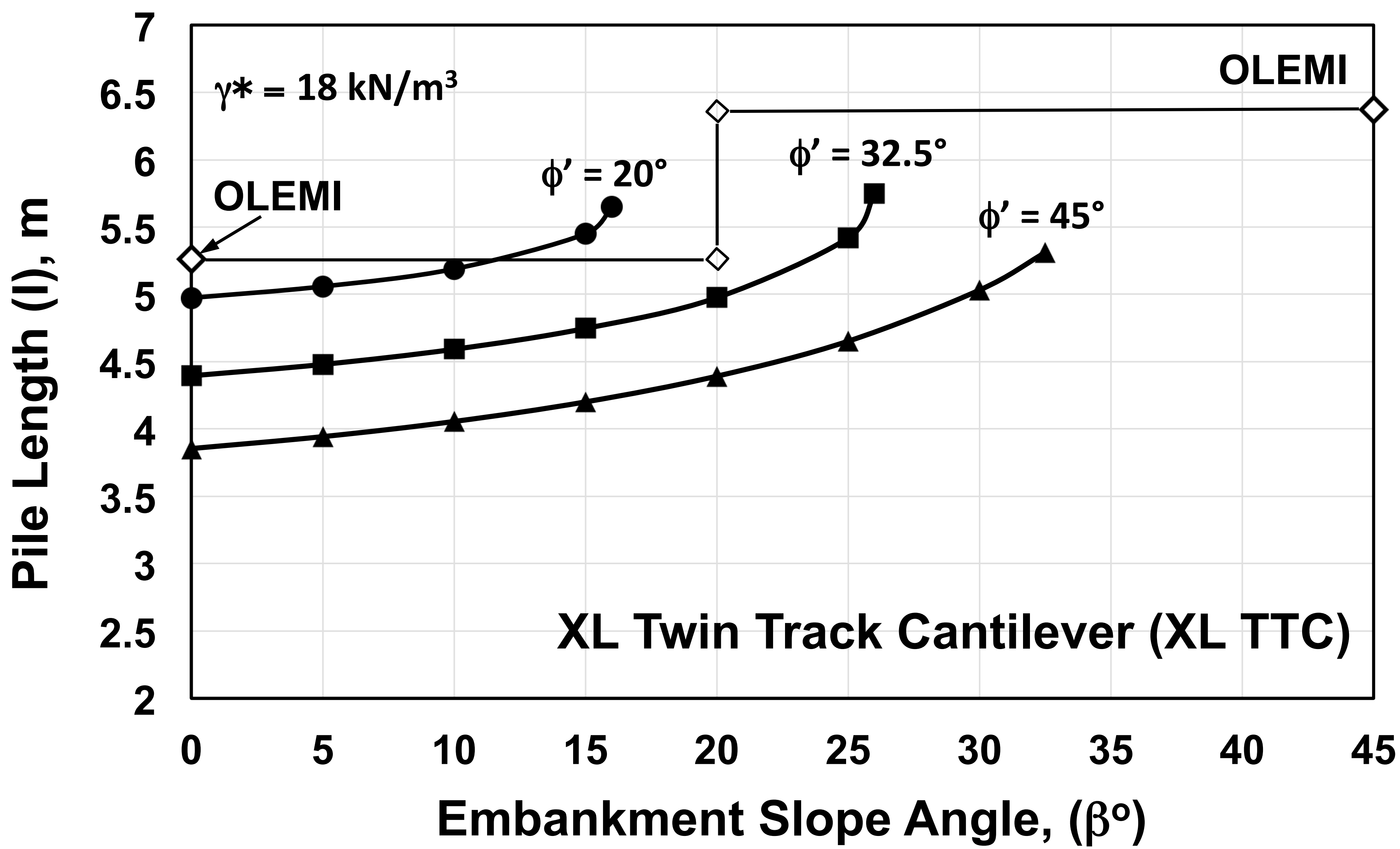
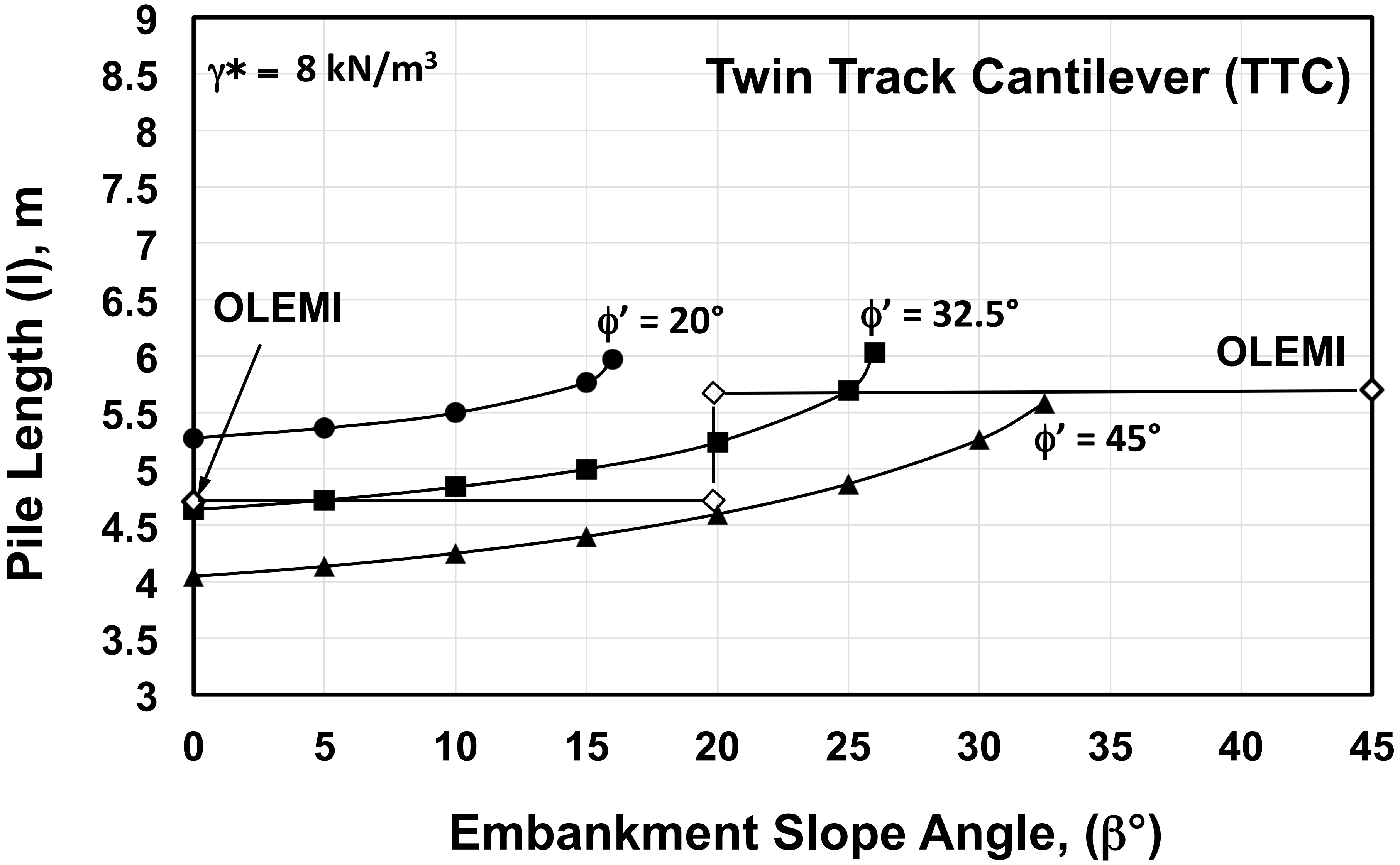
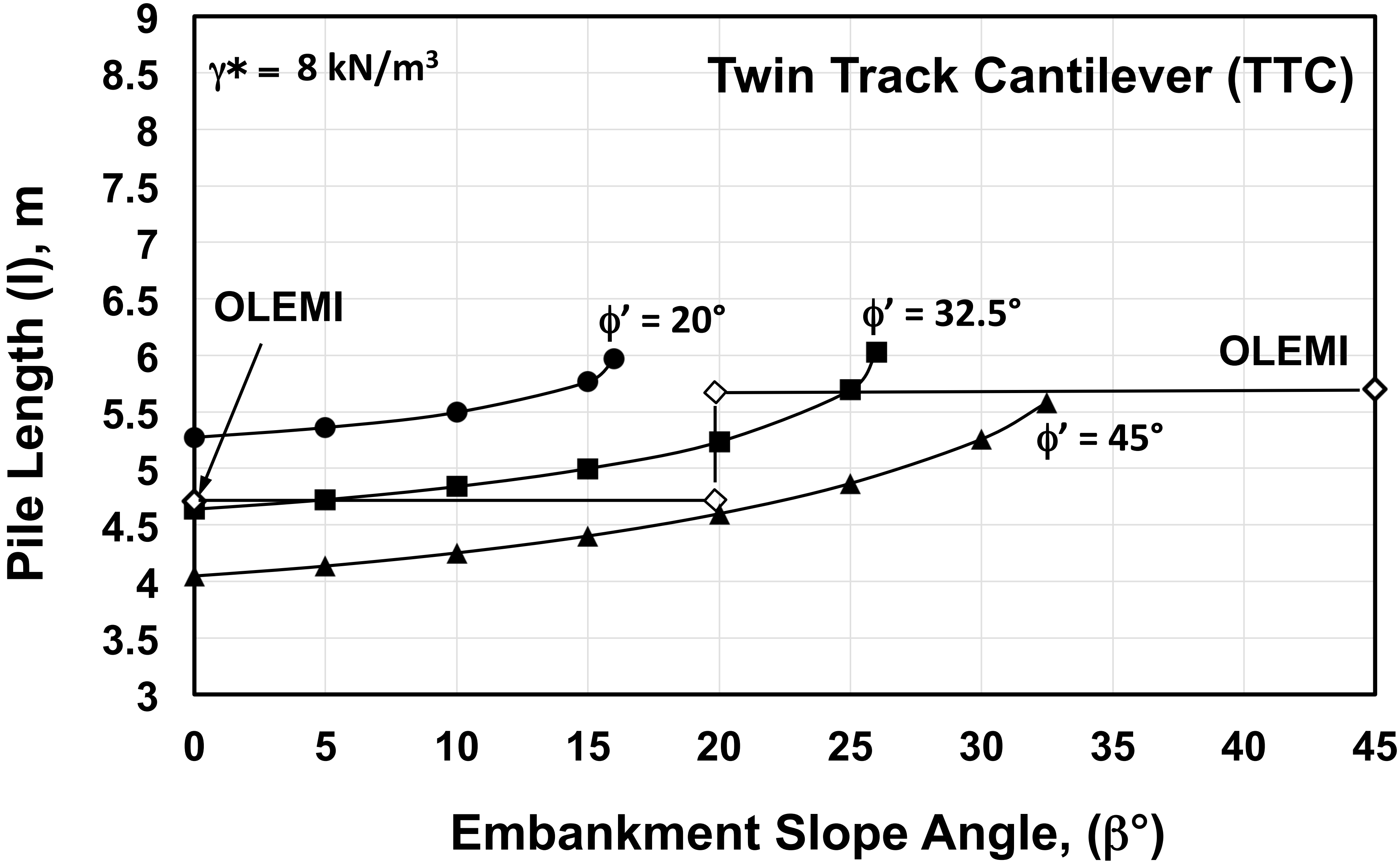

γ* = 18 kN/m³
OLEMI
OLEMI
ϕ' = 20°
ϕ' = 32.5°
ϕ' = 45°
XL Twin Track Cantilever (XL TTC)
Pile Length (l), m
Embankment Slope Angle, (β°)
7
6.5
6
5.5
5
4.5
4
3.5
3
2.5
2
0
5
10
15
20
25
30
35
40
45

Figure 9



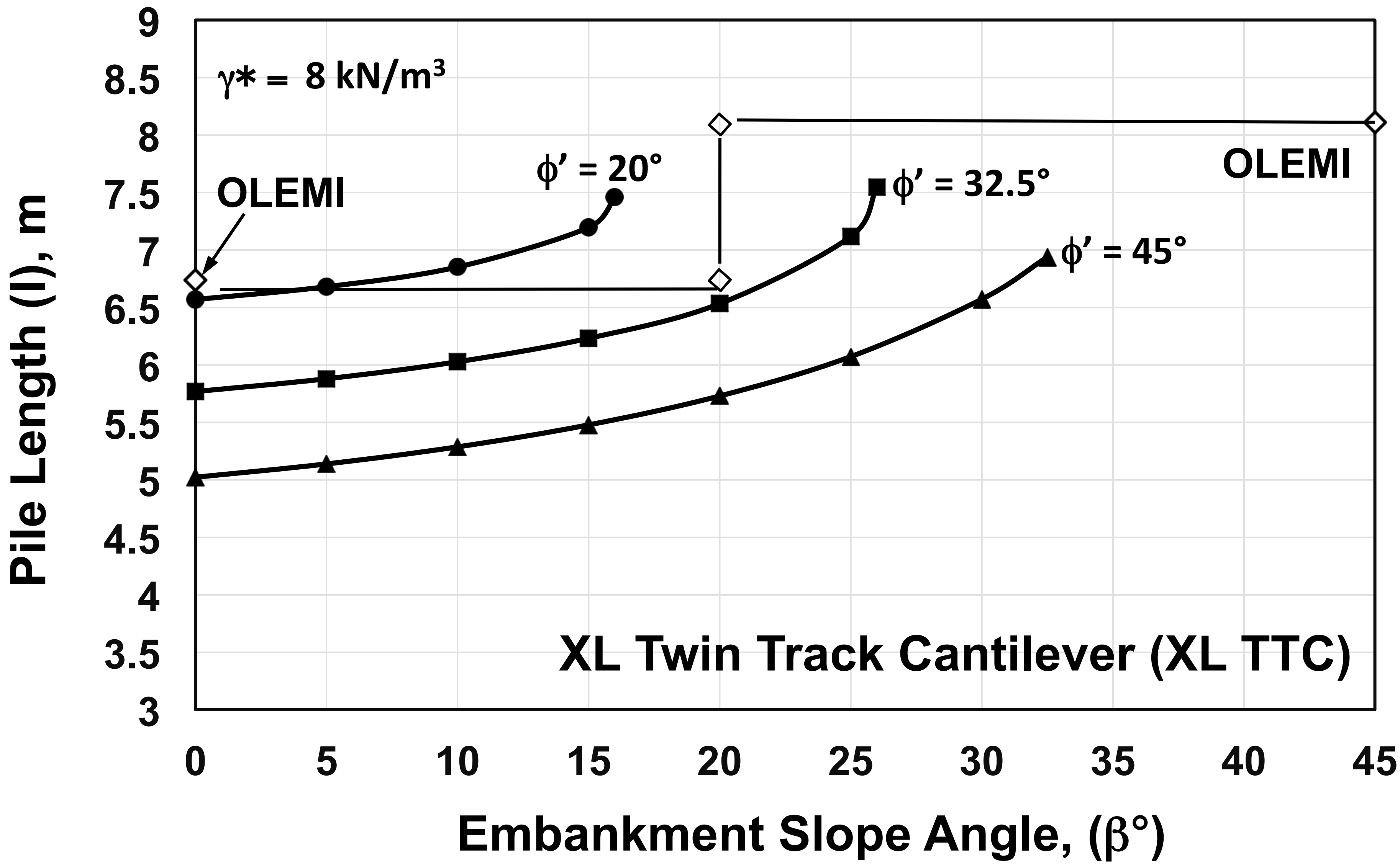
Twin Track Cantilever (TTC)
γ* = 8 kN/m³
OLEMI
OLEMI
φ' = 20°
φ' = 32.5°
φ' = 45°
Pile Length (l), m
Embankment Slope Angle, (β°)
9
8.5
8
7.5
7
6.5
6
5.5
5
4.5
4
3.5
3
0
5
10
15
20
25
30
35
40
45

Figure 10



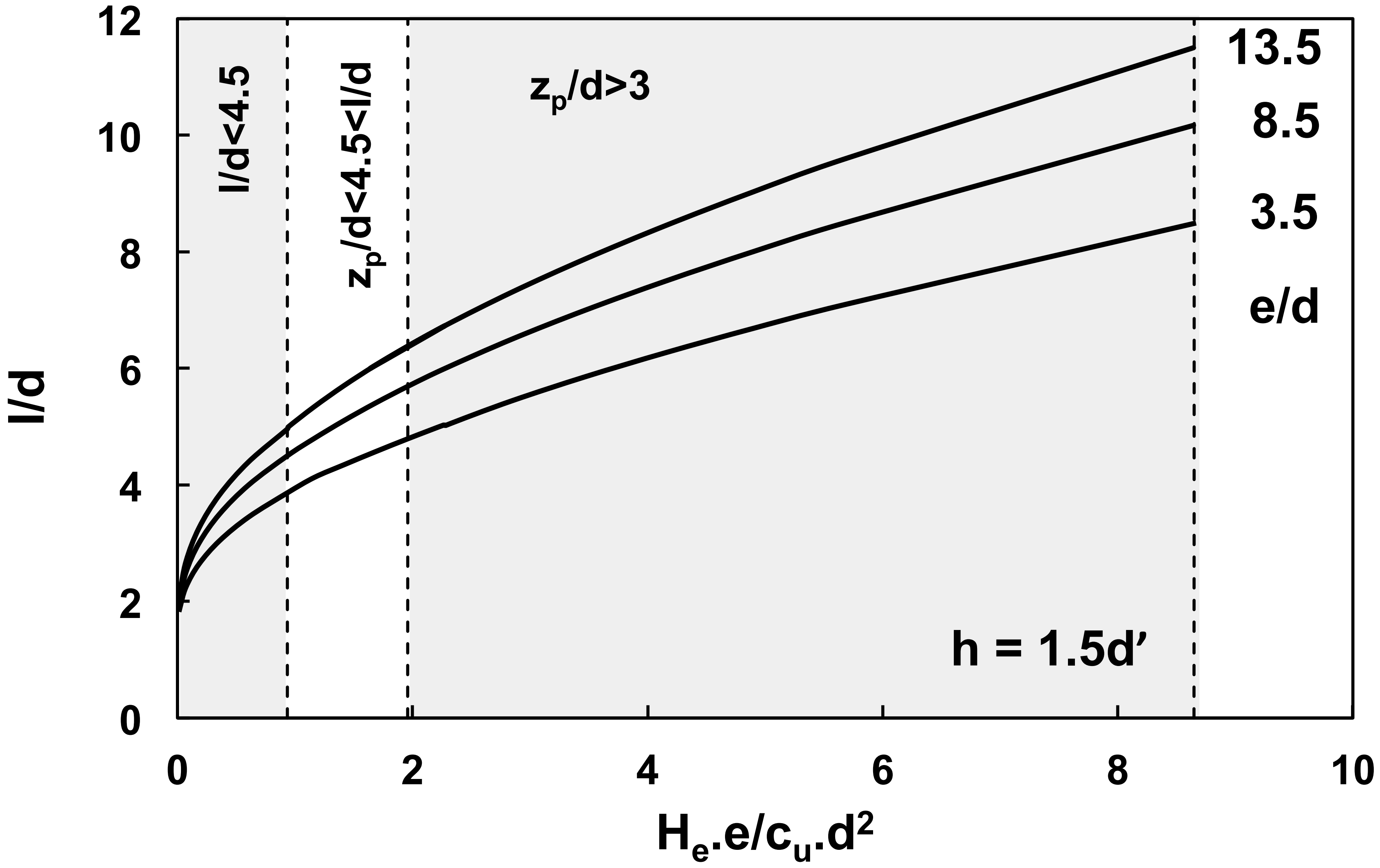
γ* = 8 kN/m³
Twin Track Cantilever (TTC)
OLEMI
ϕ' = 20°
ϕ' = 32.5°
OLEMI
ϕ' = 45°
Pile Length (l), m
Embankment Slope Angle, (β°)
9
8.5
8
7.5
7
6.5
6
5.5
5
4.5
4
3.5
3
0
5
10
15
20
25
30
35
40
45

Figure 11



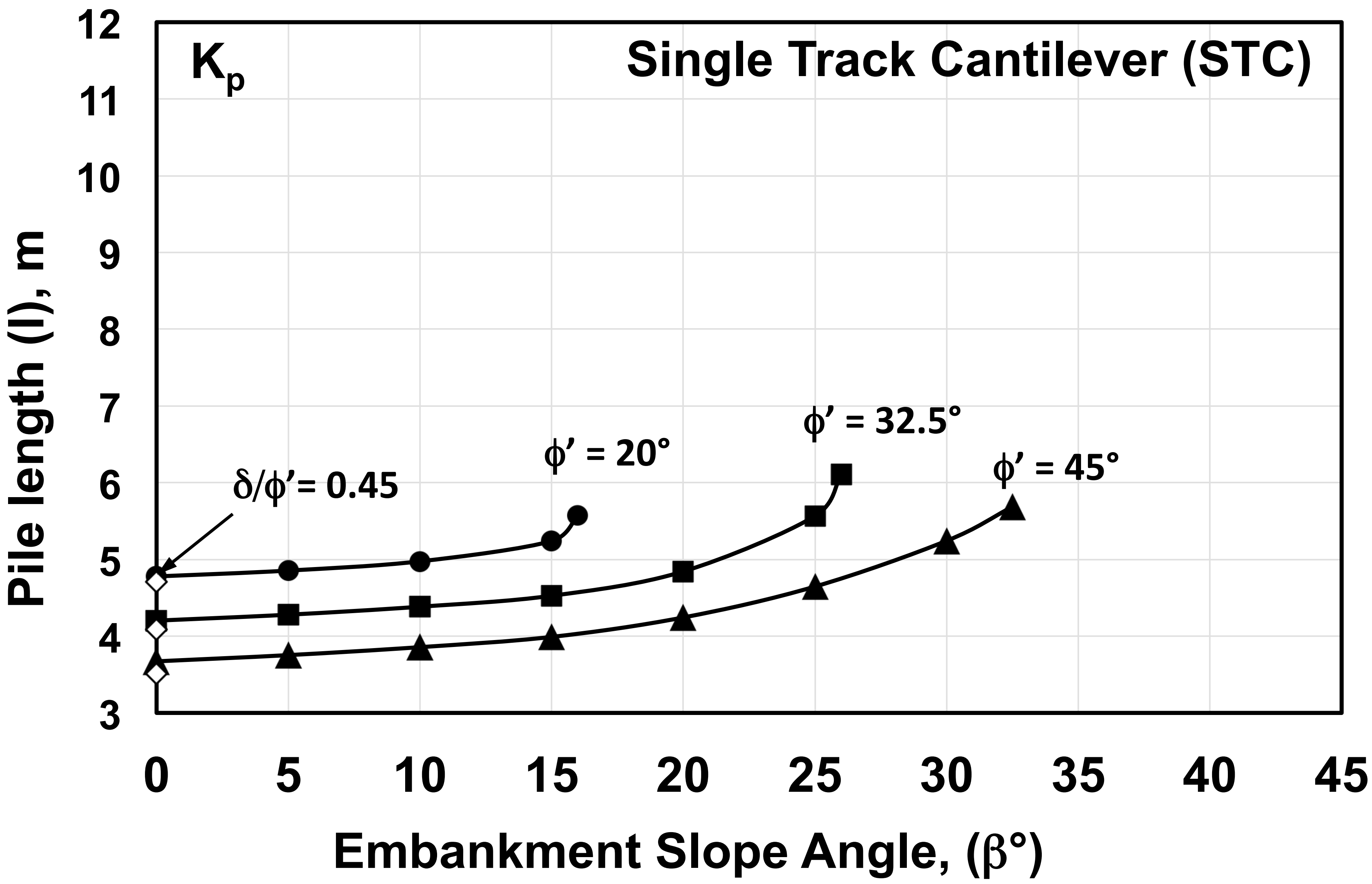
γ* = 8 kN/m³
OLEMI
ϕ' = 20°
ϕ' = 32.5°
ϕ' = 45°
OLEMI
XL Twin Track Cantilever (XL TTC)
Pile Length (l), m
Embankment Slope Angle, (β°)
9
8.5
8
7.5
7
6.5
6
5.5
5
4.5
4
3.5
3
0
5
10
15
20
25
30
35
40
45

Figure 12



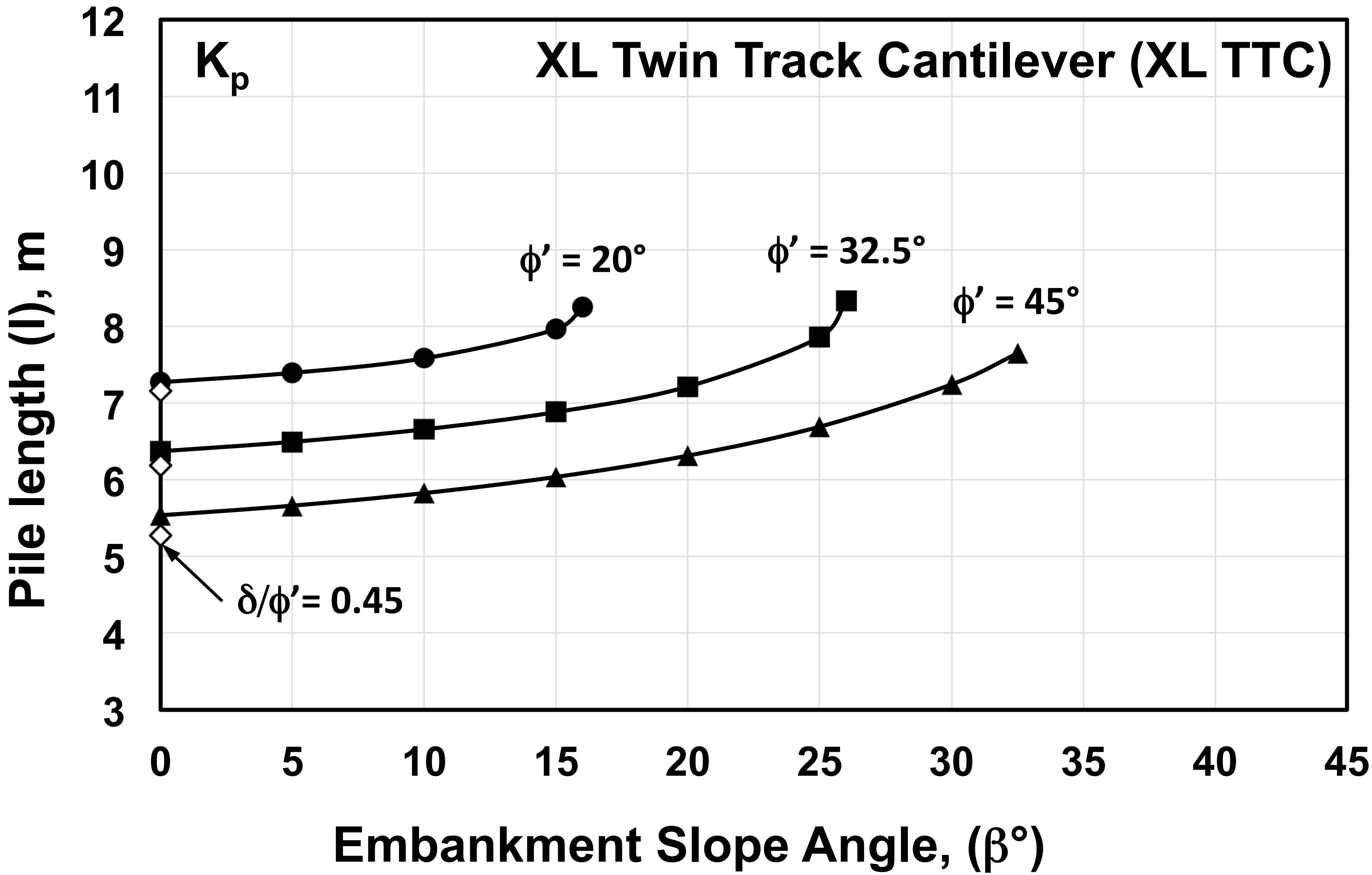
12
10
8
6
4
2
0
0
2
4
6
8
10
l/d
$H_e.e/c_u.d^2$
l/d<4.5
z_p/d<4.5<l/d
z_p/d>3
13.5
8.5
3.5
e/d
h = 1.5d’

Figure 13



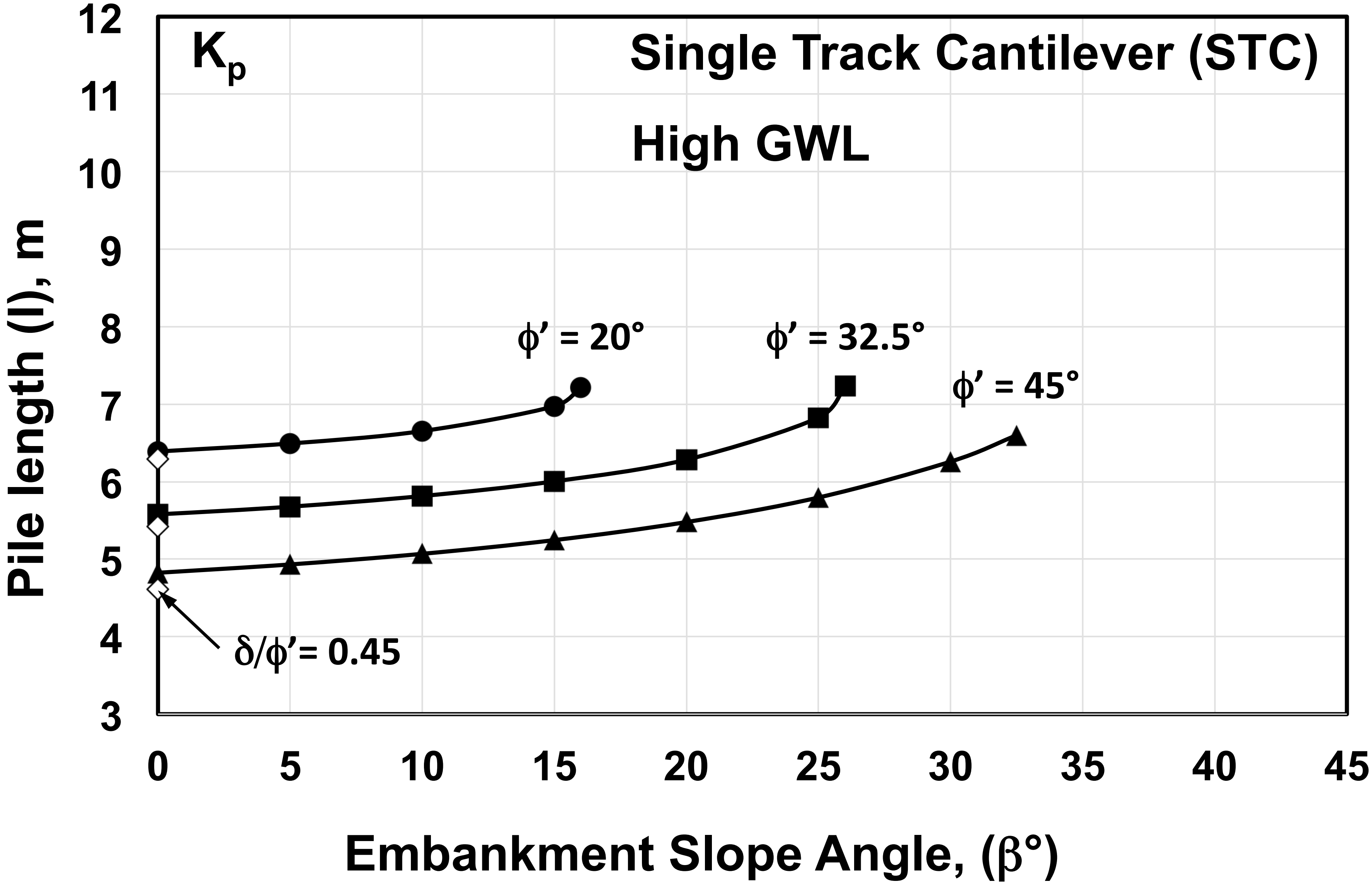
Kp
Single Track Cantilever (STC)
Pile length (l), m
Embankment Slope Angle, (β°)
δ/ϕ'= 0.45
ϕ' = 20°
ϕ' = 32.5°
ϕ' = 45°
3
4
5
6
7
8
9
10
11
12
0
5
10
15
20
25
30
35
40
45

Figure 14



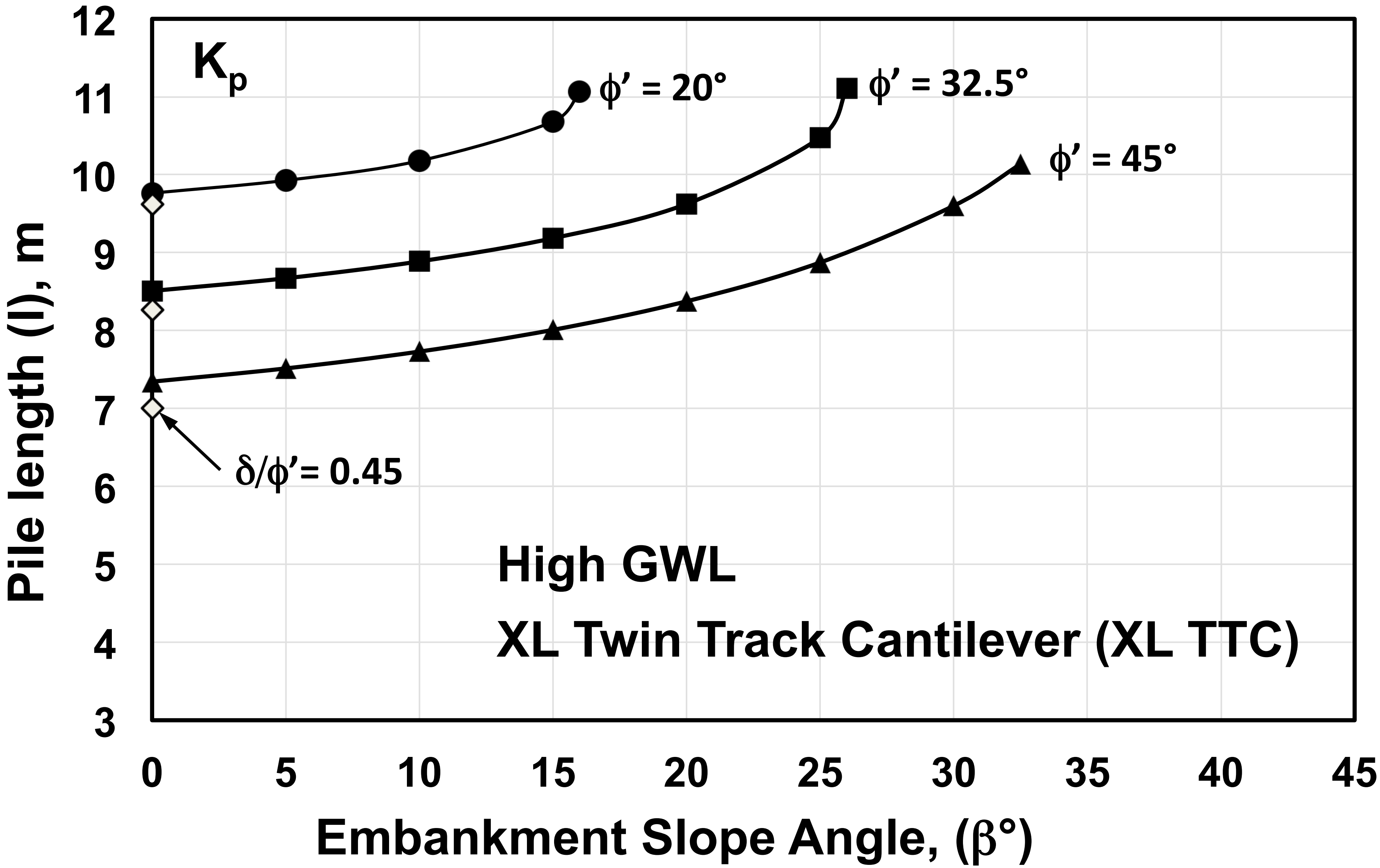
Kp
XL Twin Track Cantilever (XL TTC)
φ' = 20°
φ' = 32.5°
φ' = 45°
δ/φ'= 0.45
Pile length (l), m
Embankment Slope Angle, (β°)
12
11
10
9
8
7
6
5
4
3
0
5
10
15
20
25
30
35
40
45

Figure 15



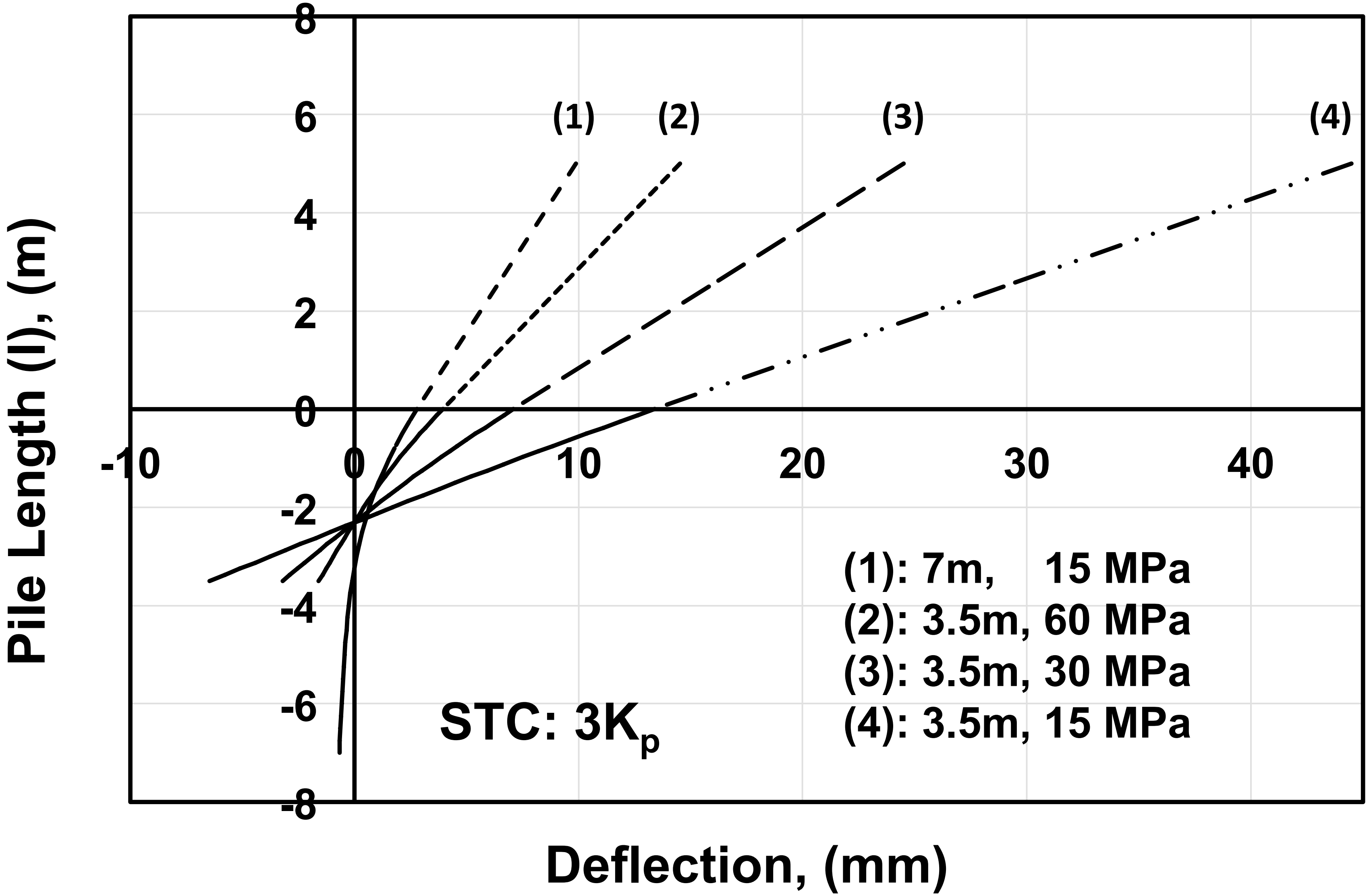
K_p
Single Track Cantilever (STC)
High GWL
ϕ' = 20°
ϕ' = 32.5°
ϕ' = 45°
δ/ϕ'= 0.45
Pile length (l), m
Embankment Slope Angle, (β°)
12
11
10
9
8
7
6
5
4
3
0
5
10
15
20
25
30
35
40
45

Figure 16



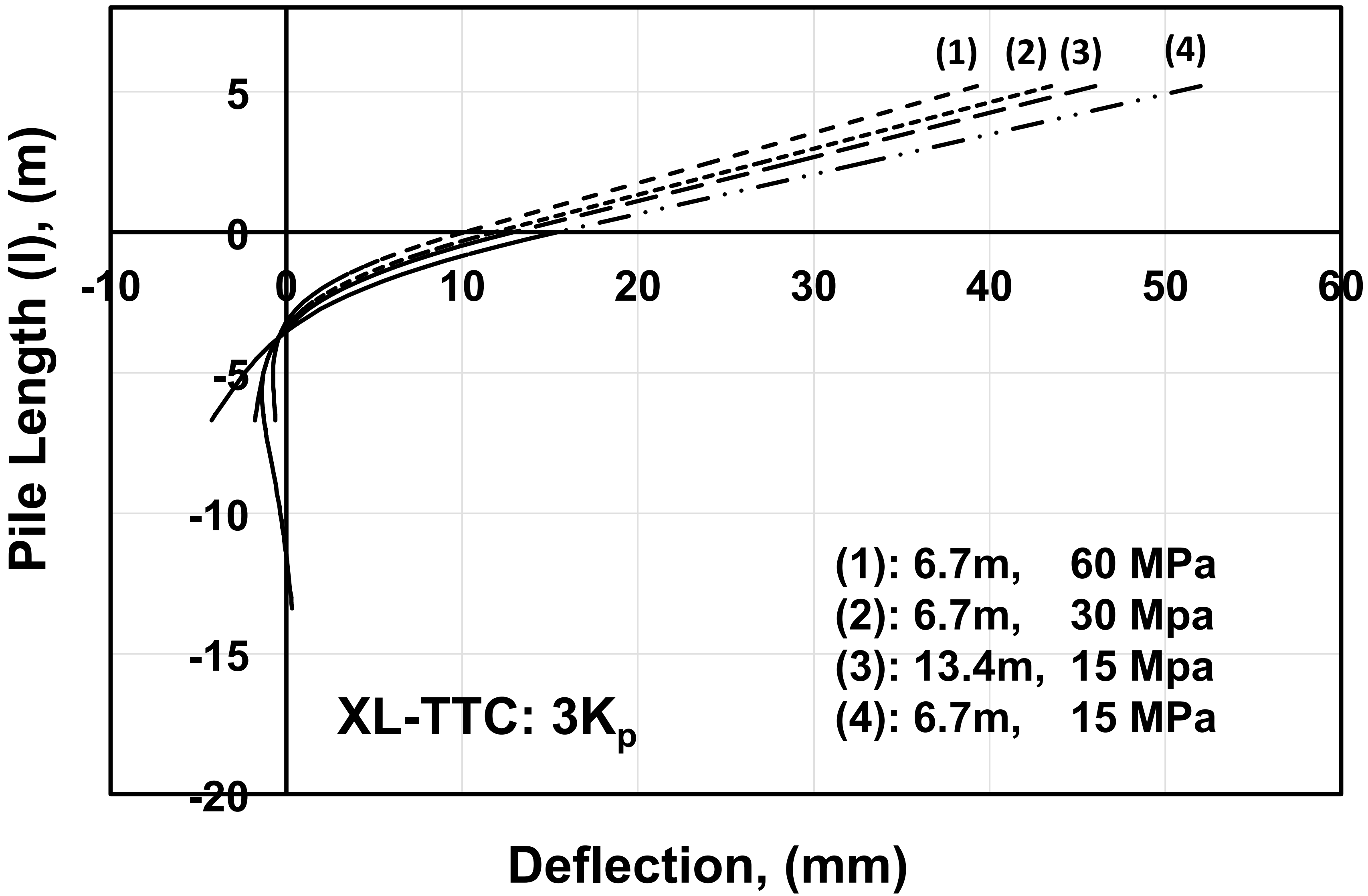
12
11
10
9
8
7
6
5
4
3
0
5
10
15
20
25
30
35
40
45
Pile length (l), m
Embankment Slope Angle, (β°)
Kp
ϕ' = 20°
ϕ' = 32.5°
ϕ' = 45°
δ/ϕ'= 0.45
High GWL
XL Twin Track Cantilever (XL TTC)

Figure 17



8
6
4
2
0
-2
-4
-6
-8
-10
0
10
20
30
40
(1)
(2)
(3)
(4)
Pile Length (l), (m)
Deflection, (mm)
STC: 3Kp
(1): 7m, 15 MPa
(2): 3.5m, 60 MPa
(3): 3.5m, 30 MPa
(4): 3.5m, 15 MPa

Figure 18



(1)
(2)
(3)
(4)
5
0
-5
-10
-15
-20
-10
0
10
20
30
40
50
60
Pile Length (l), (m)
Deflection, (mm)
XL-TTC: 3Kp
(1): 6.7m, 60 MPa
(2): 6.7m, 30 Mpa
(3): 13.4m, 15 Mpa
(4): 6.7m, 15 MPa

 Click here to access/download;Table;Table 1 Mar 2019.docx

Structure type	Loading direction	Permanent	Variable (3 year wind return period)	Variable (50 year wind return period)
STC	Vertical kN	10.47	-	-
	Horizontal kN	1.16	9.051	13.26
	Moment kNm	13.69	55.656	80.680
TTC	Vertical kN	26.97	-	-
	Horizontal kN	2.32	13.433	19.90
	Moment kNm	63.61	85.203	125.35
XL-TTC	Vertical kN	47.6	-	-
	Horizontal kN	-	21.274	29.4
	Moment kNm	180.7	171.608	235.8

 Click here to access/download;Table;Table 2.docx

Terrain (topography)	Direction of pull		
	Away from track	Towards track	
		i > 2 m	i < 2 m
Embankment	0.85	0.95	1.5
Level	1	1.3	2
Cutting	1.5	1.8	2

Stress distribution	Height of "centre of overturning" above base of foundation	Moment about centre of overturning	Soil stress parameter	Notes on soil stress parameter
Effective	*D*/3	$M = \frac{D^3.K.L}{12}$	*K*	*K* varies from 200 kPa/m for sandstone / limestone to 80 kPa/m for medium dense sand. Method not recommended for loose materials
Total	*D*/2	$P.D^2.(L+C)*$	*S*	*S* varies from 180 kPa for very stiff boulder clay to 84 kPa for firm clays. Method not recommended for medium / soft clays

* The width of the foundation *L* is increased by an amount *C* (*C* = 0.4 m for *L* > 1 m is suggested), to account for the fact that the supposed failure surface spreads out into the soil

 Click here to access/download;Table;Table 4.docx

ACTION		SYMBOL	SET A1	SET A2
Permanent	Unfavourable	γ_G	1.35	1.0
	Favourable	γ_G	1.0	1.0
Variable	Unfavourable	γ_Q	1.5	1.3
	Favourable	γ_Q	0	0

(EC7 Table A3, p130)

SOIL PARAMETER	SYMBOL	SET M1	SET M2
tan ϕ'	$\gamma_{\phi'}$	1.0	1.25
c'	$\gamma_{c'}$	1.0	1.25
τ_u (c_u)	γ_{cu}	1.0	1.4
Unit weight	γ_γ	1.0	1.0

(EC7 Table A4, p130)

 Click here to access/download;Table;Table 5.docx

		STC	TTC	XL-TTC
EC7 DA1 C1	Factored H_e, kN	21.456	32.982	44.1
	Factored M_{GL}, kNm	139.5015	273.899	597.645
	Equivalent height of load e, m	6.502	8.304	13.552
EC7 DA1 C2	Factored H_e, kN	18.398	28.19	38.22
	Factored M_{GL}, kNm	118.574	226.565	487.24
	Equivalent height of load e, m	6.445	8.037	12.748
ORE/OLEMI	Characteristic M_{GL}, kNm	94.37	188.96	416.5

(a): Across track

		STC	TTC	XL-TTC
EC7 DA1 C1	Factored H_e, kN	18.324	26.718	44.1
	Factored M_{GL}, kNm	102.539	102.1515	109.755
	Equivalent height of load e, m	5.596	3.823	2.489
EC7 DA1 C2	Factored H_e, kN	16.078	23.55	38.22
	Factored M_{GL}, kNm	91.194	99.345	125.84
	Equivalent height of load e, m	5.672	4.218	3.293
ORE/OLEMI	Characteristic M_{GL}, kNm	66.99	61.74	55.10

(b): Away from track

 Click here to access/download;Table;Table 6.docx

Structures considered	STC; TTC; XL-TTC
Loads (Horizontal load H_e and equivalent height of action e, or moment at ground level M_{GL})	See Table 5
Soil unit weight γ, kN/m^3	18
Soil effective unit weight $\gamma^* = \gamma - \frac{du}{dz}$, kN/m^3 (and corresponding pore water pressure gradient $\frac{du}{dz}$)	18 ($\frac{du}{dz} = u = 0$) (zero pore water pressures) 8 ($\frac{du}{dz} = 10\ kPa/m$) (hydrostatic)
Soil effective angle of shearing resistance, ϕ', degrees (effective stress limit equilibrium analyses)	20; 32½; 45
Soil undrained shear strength, c_u, kPa (total stress limit equilibrium analysis)	30; 60; 120
Embankment slope angle, β, degrees	0 to 40 (maximum)
Location of foundation	At embankment crest

 Click here to access/download;Table;Table 7.docx

Pile lengths, m	**Limit equilibrium analysis (DA1 C2, towards track)**			**OLEMI Pile lengths (m)**
	c_u = 30 kPa	**c_u = 60 kPa**	**c_u = 120 kPa**	
STC	3.46	2.76	2.29	2.83
TTC	4.24	3.31	2.67	3.73
XL-TTC	5.37	4.12	3.24	5.26

Parameter	**Value / assumptions**
Applied loading (moment at ground level): From Table 1	94.37 kNm (STC): 416.5 kNm (XLTTC)
Ratio Variable to Total moment at ground level (From Table 1)	0.855 (STC): 0.566 (XLTTC)
Pile diameter *d (m); wall thickness t (mm)*	0.61 m; 16 mm
Pile bending stiffness *EI*	263 MNm^2
Pile length	3.5 m and 7.0 m (STC), 6.7 m and 13.4 m (XLTTC): these correspond to the OLEMI calculated depth and 2 × the OLEMI calculated depth for the specified pile loading
Contact wire height above pile top	5.5 m
Ineffective soil depth	1.5 × pile diameter *d*
Soil strength ϕ'	30°
Soil Young's modulus E', kPa	15 MPa, 30 MPa and 60 MPa: these correspond to 1×, 2× and 4× the value of 15 MPa used by Krechowiecki-Shaw and Alobaidi (2015)
Limiting lateral soil stress below ineffective depth	$3K_p$ × vertical effective stress at the same depth
Soil unit weight γ	18 kN/m^3
Pore water pressures	Hydrostatic below a water table at ground level

 Click here to access/download;Table;Table 9.docx

Structure Type	Pile Length, m	Soil Young's Modulus, MPa	Wire Height Deflection, mm	
			Total	Variable
STC	3.5 m	15	45.7	39.1
		30	25.2	21.5
		60	15.0	12.8
	7.0 m	15	10.2	8.7
XL-TTC	6.7 m	15	52	29.4
		30	43.4	24.6
		60	39.2	22.2
	13.4 m	15	46.0	26.0

Table A1.1

Click here to access/download;Table;Table A1.1.docx

Description of parameter	**Symbol(s) used**		
	This report (broadly consistent with Fleming et al. 1994, 2009)	**BBPCL method**	**ORE**
Equivalent lateral load	H	[H]	T
Ineffective depth of soil	h'	[h']	h'
Height of action of lateral load above effective top of foundation	e	[e]	$H + h'$
Height of action of lateral load above ground surface	$(e - h')$	[$(e - h')$]	H
Effective depth of foundation	l	D	$(h - h')$
Foundation plan dimensions	diameter d	length L perpendicular to the overturning load	e parallel and b perpendicular to the overturning load. a is the smaller of e and b. If the foundation is a cylinder of diameter D, $e = b = a = 0.8D$
Depth from effective top of foundation to pivot point	z_p	Assumed equal to $2D/3$	N/A

 Click here to access/download;Table;Table A1.2.docx

Term	Definition	Symbol
Effective angle of shearing resistance, effective angle of friction	Apparent frictional strength of the soil, defining the Mohr-Coulomb failure envelope in effective stress terms, $(\tau/\sigma')_{max} = \tan\phi'$; ϕ' might be defined either at the peak stress ratio ϕ'_{peak}, or at the critical state, ϕ'_{crit}. The strength that needs to be mobilised to maintain a given equilibrium stress state is denoted ϕ'_{mob}	ϕ'
Effective soil unit weight	Weight of a unit volume of soil, adjusted for pore water pressure (buoyancy) effects, defined such that the vertical effective stress at depth $z = \gamma^*.z$, i.e. $\gamma^* = (\gamma - u/z)$ at depth z, where u is the pore water pressure	$\gamma^* = (\gamma - u/z)$ at depth z, where u is the pore water pressure. UIC-ORE (1957) uses the symbol Δ (although it assumes that the soil is not waterlogged, so that $u = 0$, and there is some ambiguity about the dimensions of Δ), and Fleming *et al*. (1994) use γ'

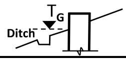
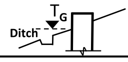
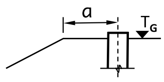
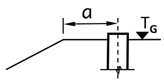
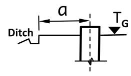
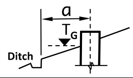
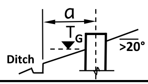
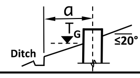
 Click here to access/download;Table;Table A1.3.docx

Parameter	**Symbol**
Resultant lateral load (wind plus or minus component of wire tension)	H
Height of action of resultant lateral load above ground level	y
Weight of mast	W_M
Weight of foundation	W_F
Weight of boom and OLE supported	w
Horizontal distance of line of action of *w* from centre of foundation	x
Disturbed or unconsolidated depth of soil	h'
Ultimate moment of resistance at ground level	M_{ult}
Design moment at ground level	M_{GL}
Design moment at the stanchion base	M_{SB}
Total depth of foundation	h
Foundation diameter	d

Click here to access/download;Table;Table A2.1.docx

Drg No.	Title	Purpose
MS/B98/K05/A3	Foundation allocation schedule: grabbed and hand excavated side-bearing foundations	To determine topographical 'location conditions' that will affect moment capacity and design depth
MS/B98/K08/A3	Foundation allocation schedule: augured side bearing foundations	To determine topographical 'location conditions' that will affect moment capacity and design depth
MS/B80/K73/A3	900 mm dia. Augured side bearing foundation	800 mm dia. Augured concrete foundations
MS/B80/L00/A3	Strength depth table for different OLE foundation types – based on ORE-UIC method	Strength depth tables for (a) 540 x 580 mm and 800 x 800 mm concrete grabbed side bearing foundations; (b) 610 mm and 762 mm dia tubular steel piles
MS/B98/K04/A4	Allocation method of bearing foundations	Details allocation method. Defines minimum overturning moment at ground level.

Table A2.2a Click here to access/download;Table;Table A2.2a.docx

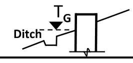
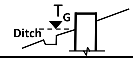
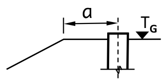
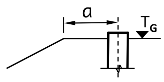
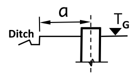
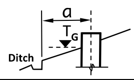
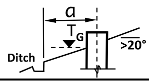
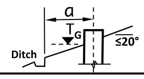
LOCATION	DIRECTION OF MOMENT	FACTOR F	ACCEPTABLE CONDITIONS	CORRECTIONS	SKETCH
SIDE OF CUTTING	UPHILL	1.37	ALL WITH CORRECTIONS	ADD 0.9 TO h	
	DOWNHILL	1.53			
SIDE OF EMBANKMENT	UPHILL	1.37	ALL WITH CORRECTIONS	ADD 0.9 TO h	
	DOWNHILL	1.53			
TOP OF CUTTING	UPHILL	1.37	$0.9 \le a \le h$	IF $a < 0.9$: ADD $(0.9 - a)$ TO h IF $a > h$: REGARD AS LEVEL GROUND	
	DOWNHILL	1.53	$0.9 \le a \le 0.7h$	IF $a < 0.9$: ADD $(0.9 - a)$ TO h IF $a > 0.7$ h: REGARD AS LEVEL GROUND	
TOP OF EMBANKMENT	UPHILL	1.37	$0.9 \le a \le h$	IF $a < 0.9$: ADD $(0.9 - a)$ TO h IF $a > h$: REGARD AS LEVEL GROUND	
	DOWNHILL	1.53	$0.9 \le a \le 0.7h$	IF $a < 0.9$: ADD $(0.9 - a)$ TO h IF $a > 0.7$ h: REGARD AS LEVEL GROUND	
LEVEL GROUND	AWAY FROM TRACK	1.3	$a \ge h$	IF $a < h$: REGARD AS SIDE OF EMBANKMENT	
	TOWARDS TRACK	1.0	$a \ge 0.7h$	IF $a < 0.7H$: REGARD AS SIDE OF EMBANKMENT	
BASE OF EMBANKMENT	UPHILL	1.0	$a \ge 0.7h$	IF $a < 0.7H$: REGARD AS SIDE OF EMBANKMENT	
	DOWNHILL	1.3	$a \ge h$	IF $a < h$: REGARD AS SIDE OF EMBANKMENT	
BASE OF CUTTING SLOPE >20°	UPHILL	1.3	$a \ge 0.9$	IF $a < 0.9$: ADD $(0.9 – a)$ TO h	
	DOWNHILL	1.0	$a \ge 0.9$	IF $a < 0.9$: ADD $(0.9 – a)$ TO h	
BASE OF CUTTING SLOPE ≤20°	UPHILL	0.867	$a \ge 0.7h$	IF $a < 0.7h$: REGARD AS BASE OF CUTTING > 20°	
	DOWNHILL	0.722	$a \ge h$	IF $a < h$: REGARD AS BASE OF CUTTING >20°	

$\mathbf{T_G}$: Lowest ground level in contact with foundation

All units of correction to a and **h** are in meters

Table A2.2b

Click here to access/download;Table;Table A2.2b.docx

LOCATION	DIRECTION OF MOMENT	FACTOR F	ACCEPTABLE CONDITIONS	CORRECTIONS	SKETCH
SIDE OF CUTTING	UPHILL	1.37	ALL WITH CORRECTIONS	ADD 0.6 TO h	
	DOWNHILL	1.53			
SIDE OF EMBANKMENT	UPHILL	1.37	ALL WITH CORRECTIONS	ADD 0.6 TO h	
	DOWNHILL	1.53			
TOP OF CUTTING	UPHILL	1.37	$0.9 \le a \le h$	IF $a < 0.6$: ADD $(0.6 - a)$ TO h IF $a > h$: REGARD AS LEVEL GROUND	
	DOWNHILL	1.53	$0.9 \le a \le 0.7h$	IF $a < 0.6$: ADD $(0.6 - a)$ TO h IF $a > 0.7$ h: REGARD AS LEVEL GROUND	
TOP OF EMBANKMENT	UPHILL	1.37	$0.9 \le a \le h$	IF $a < 0.6$: ADD $(0.6 - a)$ TO h IF $a > h$: REGARD AS LEVEL GROUND	
	DOWNHILL	1.53	$0.9 \le a \le 0.7h$	IF $a < 0.6$: ADD $(0.6 - a)$ TO h IF $a > 0.7$ h: REGARD AS LEVEL GROUND	
LEVEL GROUND	AWAY FROM TRACK	1.3	$a \ge h$	IF $a < h$: REGARD AS SIDE OF EMBANKMENT	
	TOWARDS TRACK	1.0	$a \ge 0.7h$	IF $a < 0.7H$: REGARD AS SIDE OF EMBANKMENT	
BASE OF EMBANKMENT	UPHILL	1.0	$a \ge 0.7h$	IF $a < 0.7H$: REGARD AS SIDE OF EMBANKMENT	
	DOWNHILL	1.3	$a \ge h$	IF $a < h$: REGARD AS SIDE OF EMBANKMENT	
BASE OF CUTTING SLOPE >20°	UPHILL	1.3	$a \ge 0.9$	IF $a < 0.6$: ADD $(0.6 – a)$ TO h	
	DOWNHILL	1.0	$a \ge 0.9$	IF $a < 0.6$: ADD $(0.6 – a)$ TO h	
BASE OF CUTTING SLOPE ≤20°	UPHILL	0.867	$a \ge 0.7h$	IF $a < 0.7h$: REGARD AS BASE OF CUTTING > 20°	
	DOWNHILL	0.722	$a \ge h$	IF $a < h$: REGARD AS BASE OF CUTTING >20°	

$\mathbf{T_G}$: Lowest ground level in contact with foundation
All units of correction to a and **h** are in meters

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FACTOR *F*	**ORE *K* Factors**	
	Grabbed	**Augured**
1.53	1.3/1.53 = 0.85	1/1.53 = 0.65
1.37	1.3/1.37 = 0.85	1/1.37 = 0.73
1.3	1.3/1.3 = 1	1/1.3 = 0.77
1.0	1.3/1 = 1.3	1/1 = 1
0.867	1.3/0.867 = 1.5	1/0.867 = 1.15
0.722	1.3/0.722 = 1.8	1/0.722 = 1.38