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Guidelines for Anchored Sheet Piles Design using the Fixed Earth Support Method

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Introduction

A common use of anchored sheet piles in permanent structures is when only a small difference exists between the water levels on the two sides of the piling. Such applications of sheet piles prevail in marine and harbor structures, such as those in wharves, bulkheads, quays, piers, docks, jetties, breakwaters, river and canal walls, water front retaining walls, levees and reclamation walls. If the difference in the water levels between the two sides of the sheet pile wall becomes relatively large, high net water pressures as well as considerable seepage effects, wherever applicable, develop which usually increase the cost of the wall significantly. To maintain equal water levels on both the sides of the wall it is often recommended to provide a drainage system in the form of weep holes in the piling system along with graded filters.

This article presents the results from a detailed parametric analysis of anchored sheet pile walls in sands, using the fixed earth support method, by keeping the same water levels on both the sides of the sheet pile. The results have been plotted in a non-dimensional fashion so as to determine (i) the embedment depth of the piling below the dredge line; (ii) the design force for the anchorage; and (iii) the maximum bending moment for finding the section modulus of the piling. The effects of changes in (i) the level of the water table; (ii) the location of the anchor tie rod; (iii) the friction angle of the soil mass; and (iv) the surcharge pressure on the ground surface, on the results have been explicitly brought out.

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Conventional Design Methods

The conventional procedures for designing anchored sheet piles, depending on the relative penetration of the piling below the dredge line, can be broadly classified into two major groups. The first type is referred to as free-earth support, and second the fixed earth support (Terzaghi, 1943). In the free earth support method, the sheet pile has a lesser penetration depth, and is assumed to be like a vertical beam spanning two supports, those being (i) the anchorage system, and (ii) the passive soil cover in front of the piling below the dredge line. While using this method, depending on the flexibility of the piling, reduction factors for the computed maximum bending moments are usually recommended for finding the section modulus of the sheet pile (Rowe, 1952, 1955; Nataraj and Hoadley, 1984; and Bowles, 1988). Clayton et al. (1993) provided the charts for the free earth support method. Rowe's modification of the free earth support method usually provides more economical design than the other methods.

In the case of the fixed earth support, the sheet piling has a greater embedment depth, and it is treated equivalent to a vertical beam fixed at the lower end and propped at the point of anchorage (Terzaghi, 1954). While a sheet pile for any site and soil conditions can generally be designed either for free or fixed earth support (Tsinker, 1983), the fixed earth support design automatically provides sufficient penetration to ensure an adequate margin of safety against possible outward movement of the piling (Terzaghi, 1954). The practical relevance of the problem of anchored retaining wall with a fixed earth support is proven, e.g. by the British Steel Piling Handbook, page E10-E11 (1988), and is not disputed. The computational procedure involved in the implementation of the fixed earth support method is, however, more tedious than that of the free earth support method. The exact solution can only be obtained through an iterative procedure so as to attain zero (or required) deflection at the anchor level with reference to the assumed fixed support below the dredge line. To simplify the computational effort associated with the use of fixed earth support design, this problem is often tackled by assuming the location of the point of contra-flexure (Blum, 1931; and Tschebotarioff, 1973); this assumption, however, results in a loss of the accuracy of the results (Azizi, 2000).

Finite Element Method

On similar lines with regard to the design of various important civil engineering structures, the finite element method is also becoming popular in sheet piling design (Bowles, 1997). It has obvious merits over the conventional methods that, in addition to drawing the bending moment and shear force diagram, at any stage, the complete deformed profile of the sheet pile wall and the displacement of the anchorage-tie rod system can also be

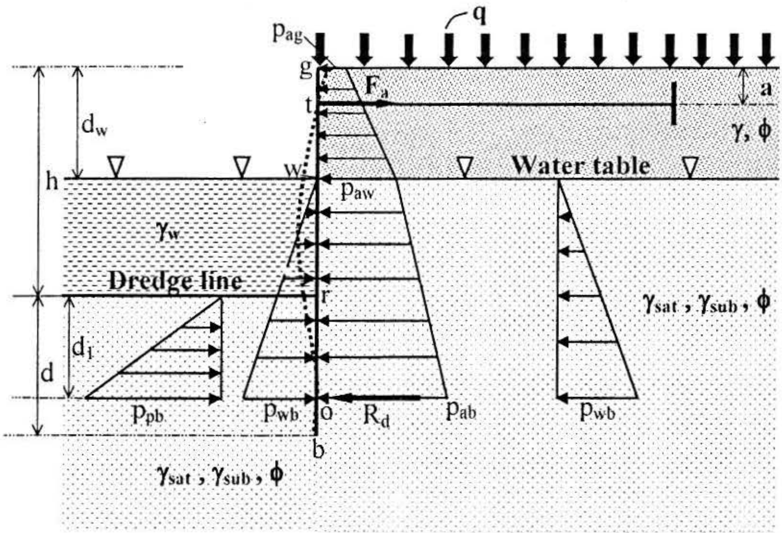

FIGURE 1 : Pressure Distribution on the Sheet Pile

obtained as a part of the output. On account of the availability of the displacements of the sheet piling and the anchor, a better design can therefore, be carried out.

However, for making the computational runs with the FEM, in addition to defining the material properties of soil, piling and anchor-tie rod, it is also necessary to provide (i) the values of the penetration depth and the sectional modulus of the piling; (ii) the position of the anchor tie rod system. The design charts from the conventional methods can here be used so as to frame guidelines for providing the initial values of various required parameters, which later on can be updated after analyzing the output of the FE program. In the present paper, the design of the sheet pile wall with the use of the FEM is not included. The reader may refer the text book of Bowles (1997) for this purpose.

Definition of the Problem

Given a sheet pile wall to retain the cohesionless backfill of height h above the horizontal dredge line in Fig.1. The sheet pile is driven in and resting against the same soil mass with friction angle ϕ. The water table on both the sides of the sheet pile is at the same level with a depth d_w below the ground surface. The soil mass has saturated unit weight, γ_{sat} and submerged unit weight, γ_{sub}; the unit weight of the soil above the water table is equal to γ. The ground surface is horizontal and is loaded with surcharge pressure q. The tie rod for the anchor is located at a depth *a* from the ground surface as shown in Fig.1. It is required to estimate (i) the embedment depth d of the piling below the dredge line; (ii) force F_a for the design of anchorage system; and (iii) maximum bending moment M_{max} for determining the section modulus of the sheet pile.

Fixed Earth Support Method

In this method, the sheet piling is considered flexible but penetrating to a sufficient depth so that it can be assumed to be fixed at its toe. The resultant of active and passive soil pressures on the lower portion ob (Fig.1) of the sheet pile is assumed to be replaced by a single concentrated force R_d, with no bending moment, acting at the point o, at a depth d_1 below the dredge level (Terzaghi, 1943). For a given distribution of active and passive earth pressures on the beam gtwro, there are three basic unknowns (i) depth d_1; (ii) anchor force F_a; and (iii) the reaction R_d. These three unknowns can be computed from the satisfaction of (i) horizontal force equilibrium; (ii) moment equilibrium; and (iii) the condition that the horizontal displacement of the beam at the point of anchorage (t) is zero relative to the point o; in the fixed earth support method it is assumed that the slope of the deformed sheet pile at the point o is zero. The last condition is satisfied by equating to zero the moment of the M/EI area diagram between t and o about the point t; where M is the bending moment, E is the elastic modulus of the sheet pile material and I is the moment of inertia about the vertical neutral axis of the sheet pile. Starting from the assumed value of the depth d_1, the true magnitude of the d_1 can be found by repetitive calculations until the horizontal displacement of the sheet pile at the anchor level becomes zero relative to the point o. The cross section of the sheet pile was assumed to be uniform, as a result, the EI term does not appear in the results. The maximum moment M_{max} can subsequently be determined by locating the point of zero shear force. The total penetration depth d of the piling below the dredge line is normally recommended 20% higher than the computed value of d_1.

Distribution of Earth Pressures

While the analysis developed in this paper is more rigorous than the simplified analysis, with the assumption of the point of contra-flexure, commonly used in practice, it sill involves simplifying assumptions regarding the actual pressure distribution acting on a complex wall-anchor system. By assuming the sheet pile to be smooth, Rankine earth pressure theory was utilized to generate the distribution of active and passive earth pressures along either side of the piling. Below the water table, the lateral earth pressures on both the sides of the sheet pile wall can be established with the consideration of the submerged unit weight of the soil mass. The pressure distribution as shown in Fig. 1 corresponds to a situation when the water table is in between the ground surface and the dredge line ($d_w \leq h$). The values of the pressures at the different levels are given below:

$$p_{ag} = k_a q \tag{1}$$

$$p_{aw} = k_a\left(q + \gamma d_w\right) \tag{2}$$

$$p_{ab} = k_a\left[q + \gamma d_w + \gamma_{sub}\left(h + d_1 - d_w\right)\right] \tag{3}$$

$$p_{pb} = k_p \gamma_{sub} d_1 \tag{4}$$

$$p_{wb} = \gamma_w\left(h + d_1 - d_w\right) \tag{5}$$

wherein, γ_w = unit weight of water

$$k_a = \frac{(1-\sin\phi)}{(1+\sin\phi)} \tag{6}$$

$$k_p = \frac{(1+\sin\phi)}{(1-\sin\phi)} \text{ and} \tag{7}$$

Likewise, the lateral active and passive earth pressure distribution can be established with $d_w > h$

Expressions for the Horizontal Deflection

As mentioned before, the horizontal deflection (δ_h) at the point of anchorage (t) can be determined by taking the moment of the M/EI area diagram between t and o about the point t. The resultant pressure distribution on the sheet pile can be obtained by superimposing, with proper sign conventions, three different types of loading, namely, (i) concentrated load; (ii) the triangular distributed pressure; and (iii) the uniform distributed pressure. Three different possible types of loading on a beam are shown in Fig. 2. The appropriate formula for the moment of M/EI diagram between t and o about the point t for each type of loading was obtained from the integration, the final expressions are summarized below:

(i) concentrated load,

$$\delta_h = \frac{FL^3}{6EI} \tag{8}$$

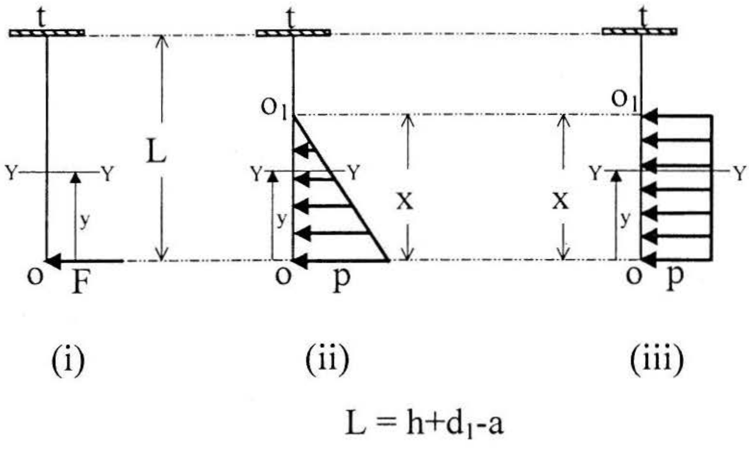

FIGURE 2 : Different Loading Conditions for computing horizontal deflection at the anchor level.

(ii) triangular distributed loading,

$$\delta_h = \frac{px}{120EI}\left[10L^3 - 10L^2x + 5Lx^2 - x^3\right] \quad (9)$$

(iii) uniform distributed pressure,

$$\delta_h = \frac{px}{24EI}\left[4L^3 - 6L^2x + 4Lx^2 - x^3\right] \quad (10)$$

The positive value of the δ_h indicates that the deflection at the anchorage point will take place in a direction same as that of the direction of the respective loading. The derivation of the above expressions is given in Appendix I.

Computational Procedure

For starting the computational run, a value of d_1 needs to be assumed. For each assumed value of d_1, the distributions of active and passive earth pressures as shown in Fig.1 were drawn by using the Eqns.1 to 7. The magnitude of the force F_a in the anchor is then obtained by equating the moment of all the forces acting on the beam gtwro about the point o equal to zero; it should be mentioned that in the fixed earth support method it is assumed that there is no bending moment at the point o. The magnitude of the reaction R_d acting at the point o is then subsequently determined from the overall horizontal force equilibrium of the beam gtwro. From the superimposition of the known appropriate pressure distributions and the reaction R_d, the magnitude of the deflection of the sheet pile at the anchorage point (t) was then obtained by using the corresponding deflection equations 8, 9 and 10. This computed value of deflection at the anchorage point will become equal to zero only for the correct magnitude of d_l.

The value of the d_l was gradually varied in a do-loop by first assigning the minimum and maximum values of d_l, which are termed as d_{lmin} and d_{lmax}, respectively. In the beginning the values of d_{lmin} and d_{lmax} were arbitrarily kept equal to zero and 3h respectively. For each chosen value of d_l, if the direction of the computed deflection of the sheet pile at the anchorage point was found to be (i) towards the waterfront side, then the value of d_{lmin} was replaced with the computed d_l; (ii) towards the backfill side, then the value of d_{max} was replaced with the obtained d_l; (iii) almost zero (it was taken equal to 1.0×10^{-10} h) then the chosen value of d_l becomes the correct solution of the problem. The procedure was continued until the deflection at the anchorage point becomes equal to zero. With the present algorithm, the convergence was obtained very rapidly in all the cases.

Results and Interpretations

Computations were performed by varying (i) the location of the water table; (ii) the depth of anchor a from 0 to 0.4 h; (iii) the value of ϕ in between 30 and 45 degrees; and (iv) surcharge pressure q from 0 to $0.4\gamma_{sub}h$. All the calculations were carried out with two different values of the ratio γ/γ_{sub} namely, 1.5 and 2.0, respectively. The results were non-dimensionalised with respect to parameters h and γ_{sub}, and the corresponding plots are shown in Figs.3 to 11. All these results correspond to unit length of sheet pile wall. Following observations were made:

1. In all the cases that the values of d_l, F_a and M_{max} become maximum for d_w = h, that is, when the water table lies at the dredge level. An increase in the value of the ratio γ/γ_{sub} results in a continuous increase in the maximum values (at the dredge level) of d_l/h, $F_a/\left(\gamma_{sub}h^2\right)$ and $M_{max}/\left(\gamma_{sub}h^3\right)$.

2. The magnitudes of d_l and M_{max} decrease with increase in the distance a. On the other hand, the magnitude of F_a becomes greater as the value of a is increased. In other words if the location of anchorage is shifted downwards, the required penetration depth and the section modulus of the sheet piling tend to be smaller; whereas at the same time the anchor need to be designed for greater pullout resistance.

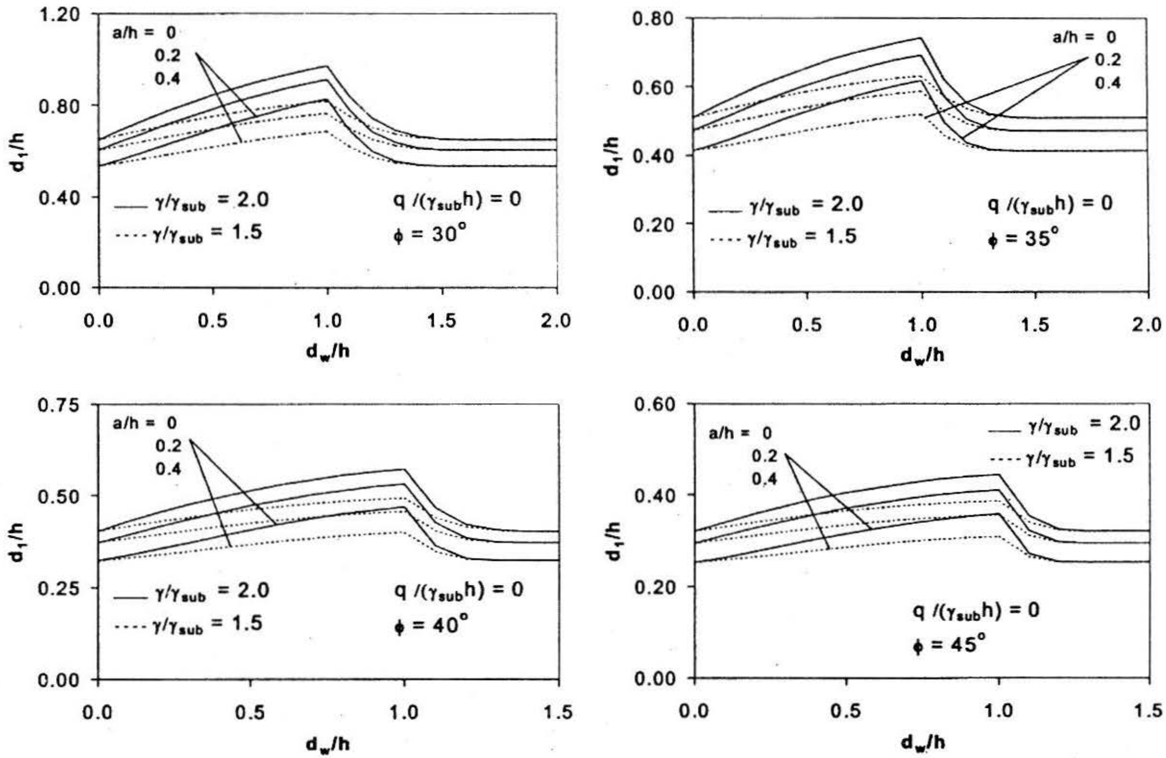

FIGURE 3 : The Variation of d_1/h **with** d_w/h, a/h **and** ϕ **for** $q/(\gamma_{sub}h) = 0$

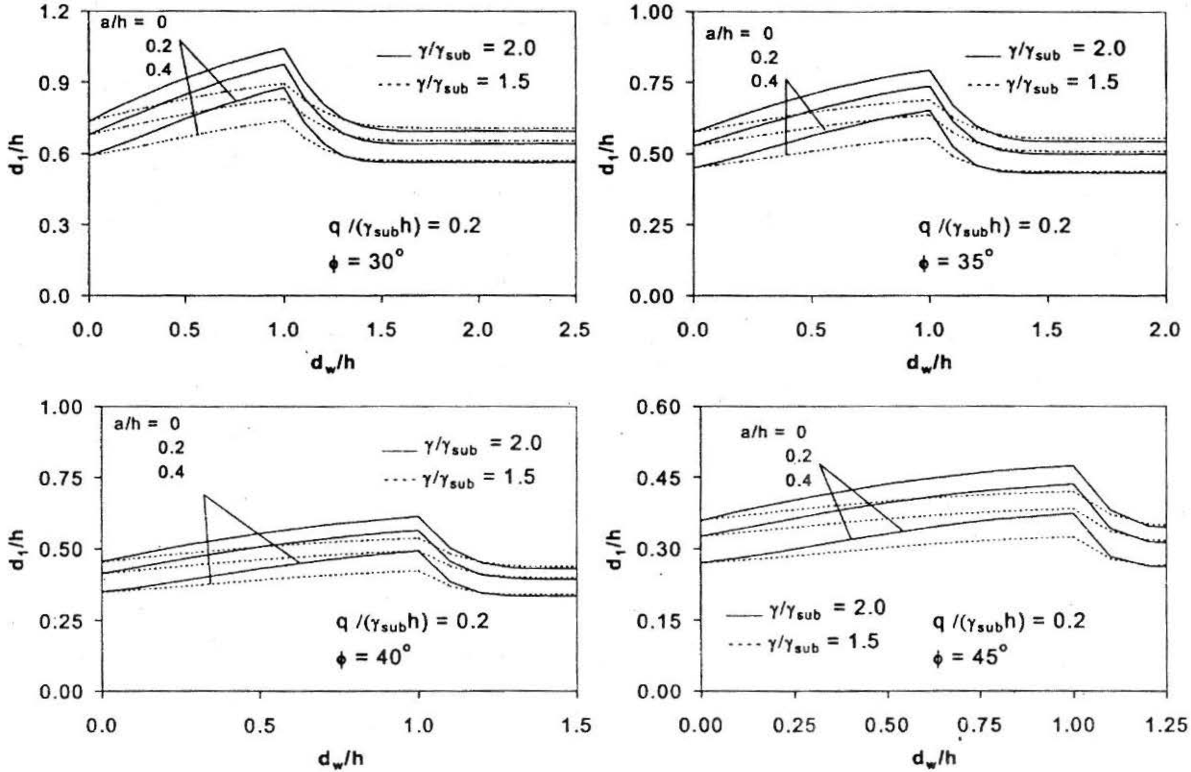

FIGURE 4 : The Variation of d_1/h with d_w/h, a/h and ϕ for $q/(\gamma_{sub}h) = 0.2$

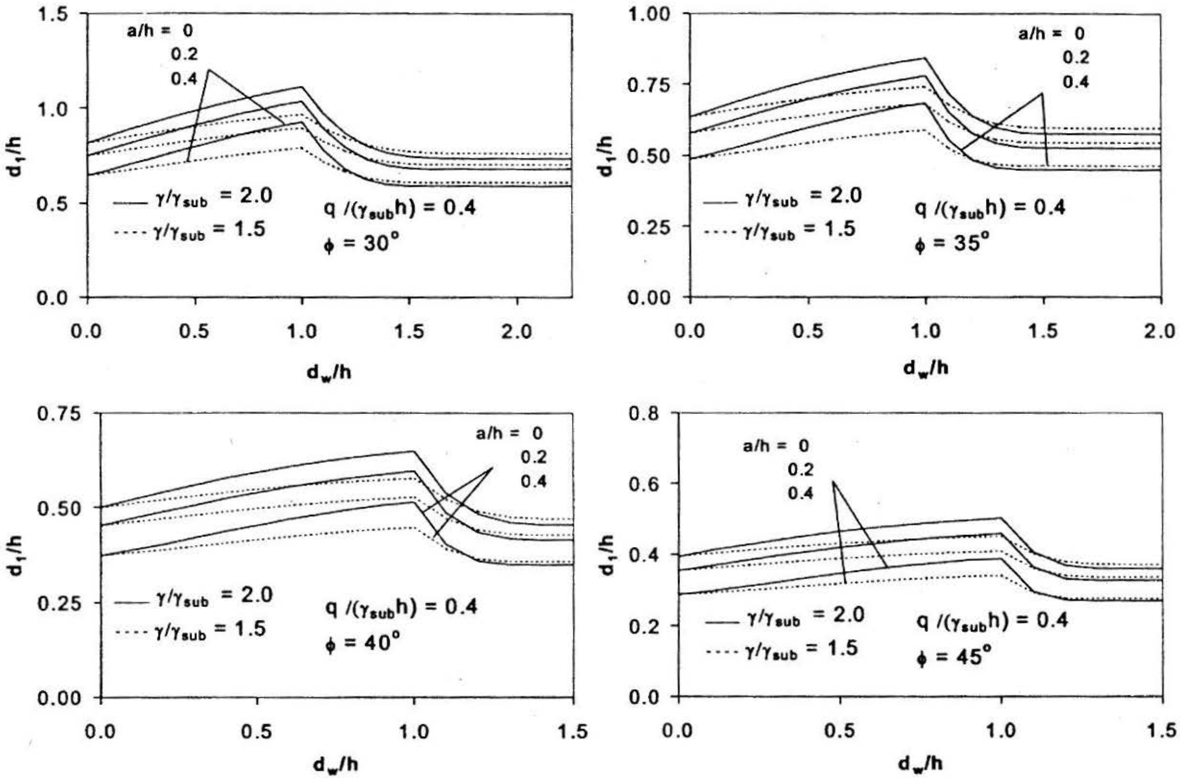

FIGURE 5 : The Variation of d_1/h with d_w/h, a/h and ϕ for $q/(\gamma_{sub} h) = 0.4$

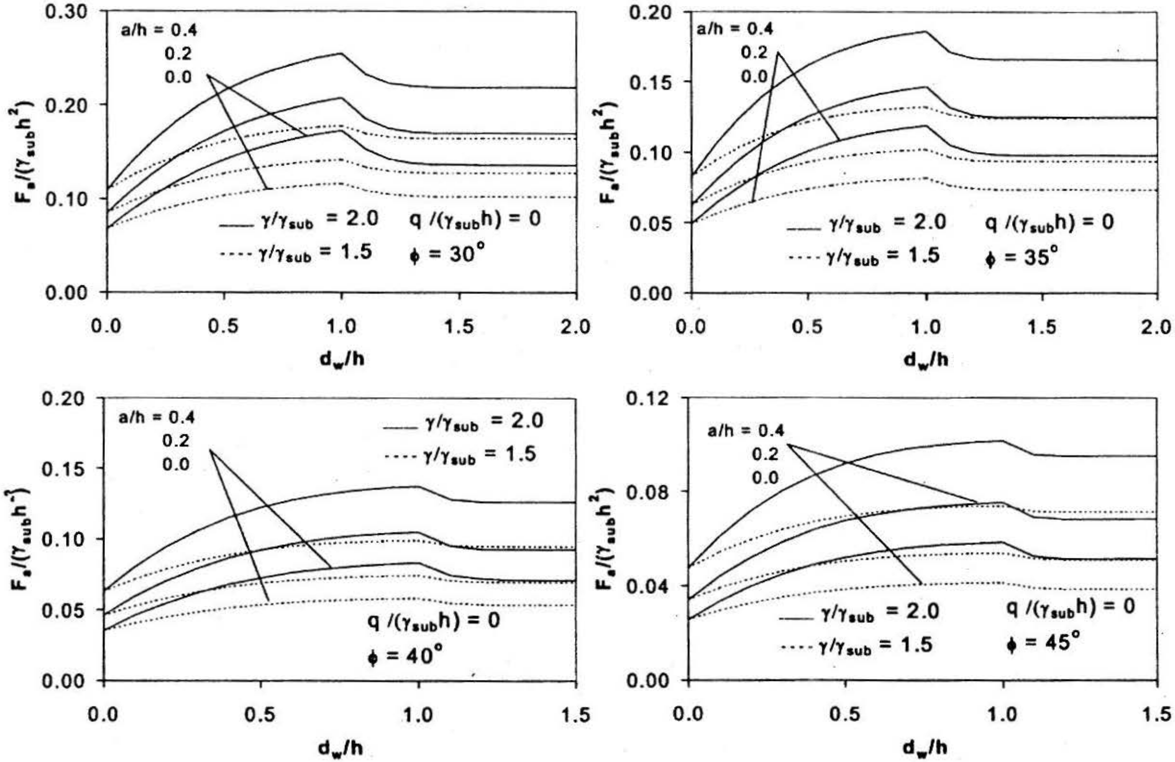

FIGURE 6 : The Variation of $F_a/(\gamma_{sub}h^2)$ with d_w/h, a/h and ϕ for $q/(\gamma_{sub}h) = 0$

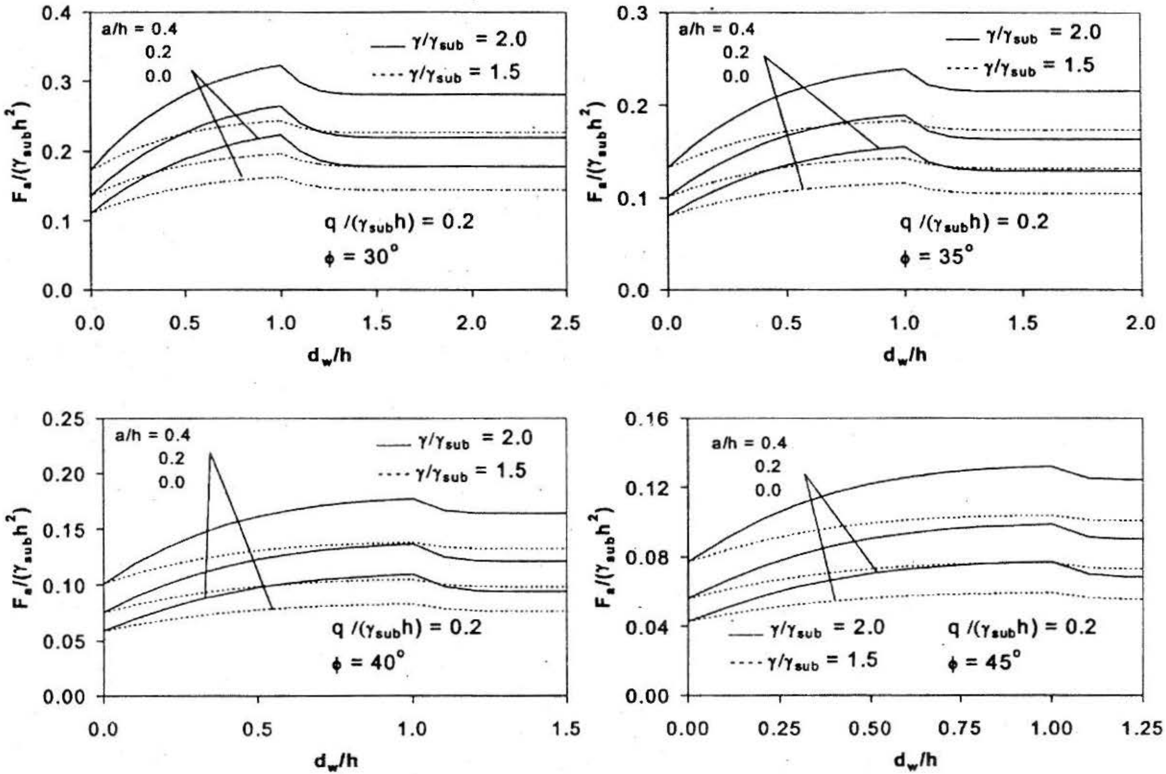

FIGURE 7 : The Variation of $F_a/(\gamma_{sub}h^2)$ **with** d_w/h, a/h **and** ϕ **for** $q/(\gamma_{sub}h) = 0.2$

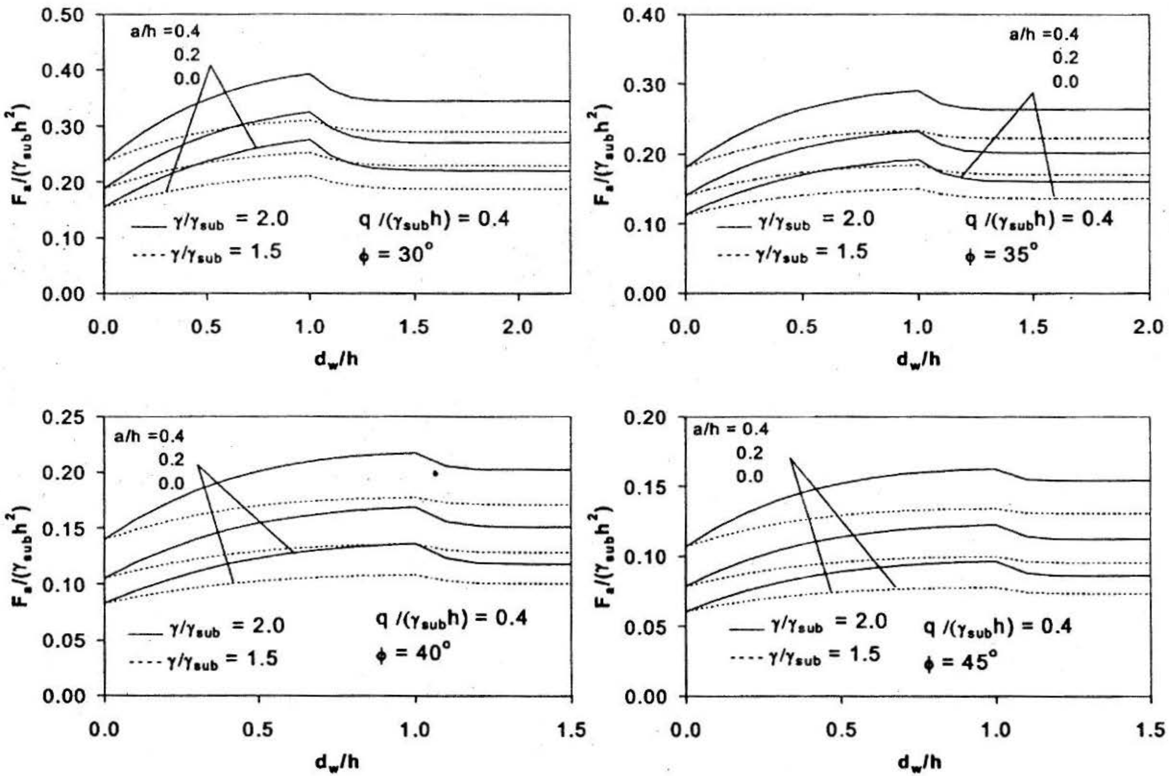

FIGURE 8 : The Variation of $F_a/(\gamma_{sub}h^2)$ **with** d_w/h, a/h **and** ϕ **for** $q/(\gamma_{sub}h) = 0.4$

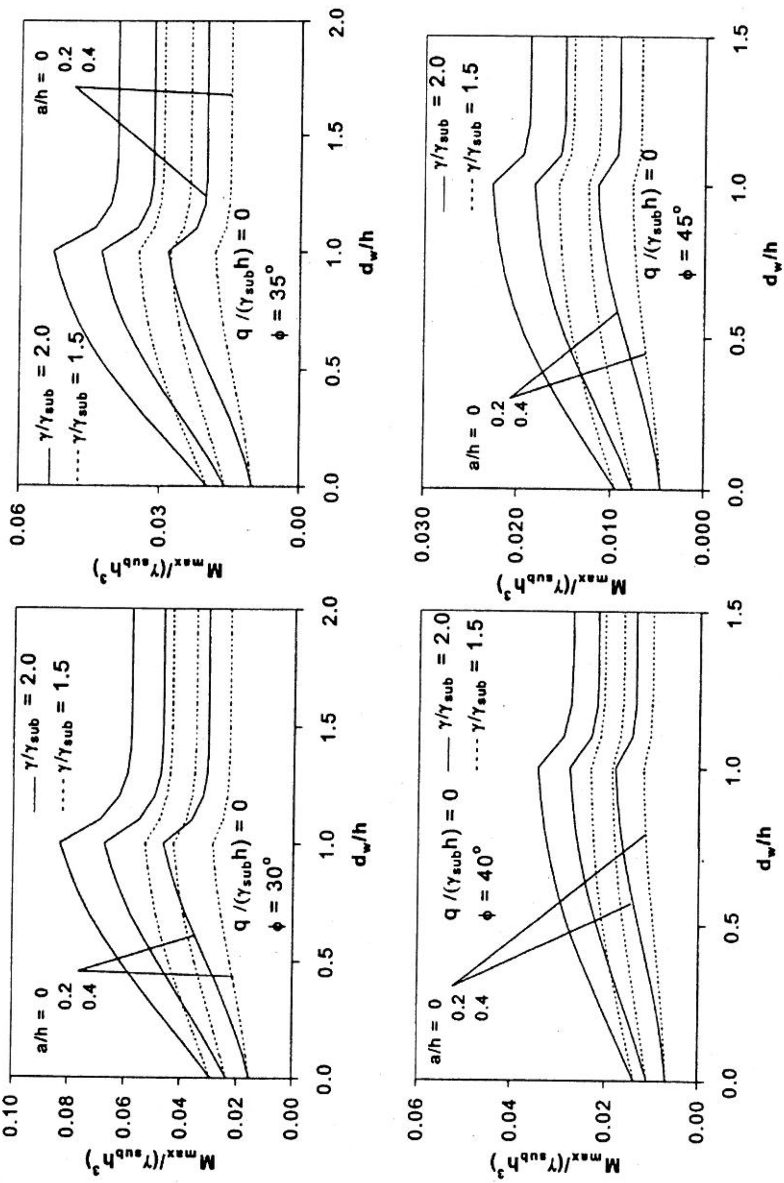

FIGURE 9 : The Variation of $M_{max}/(\gamma_{sub}h^3)$ with d_w/h, a/h and ϕ for $q/(\gamma_{sub}h) = 0$

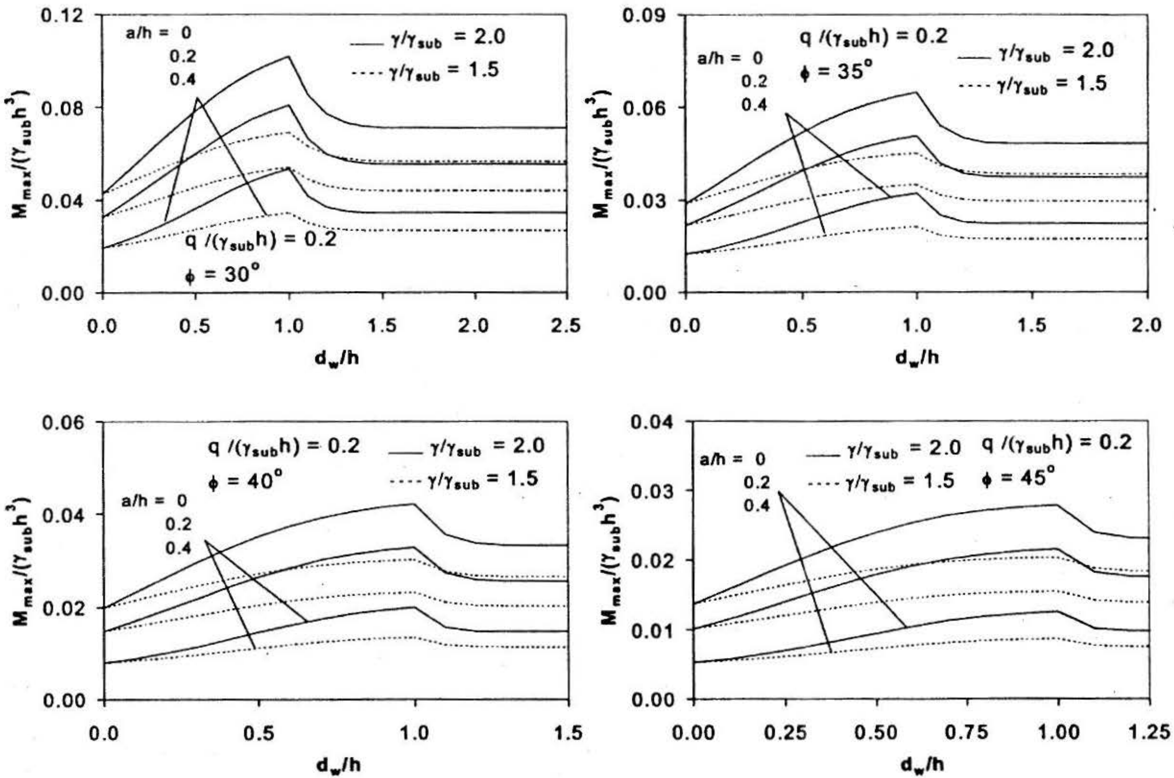

FIGURE 10 : The Variation of $M_{max}/(\gamma_{sub}h^3)$ **with** d_w/h, a/h **and** ϕ **for** $q/(\gamma_{sub}h) = 0.2$

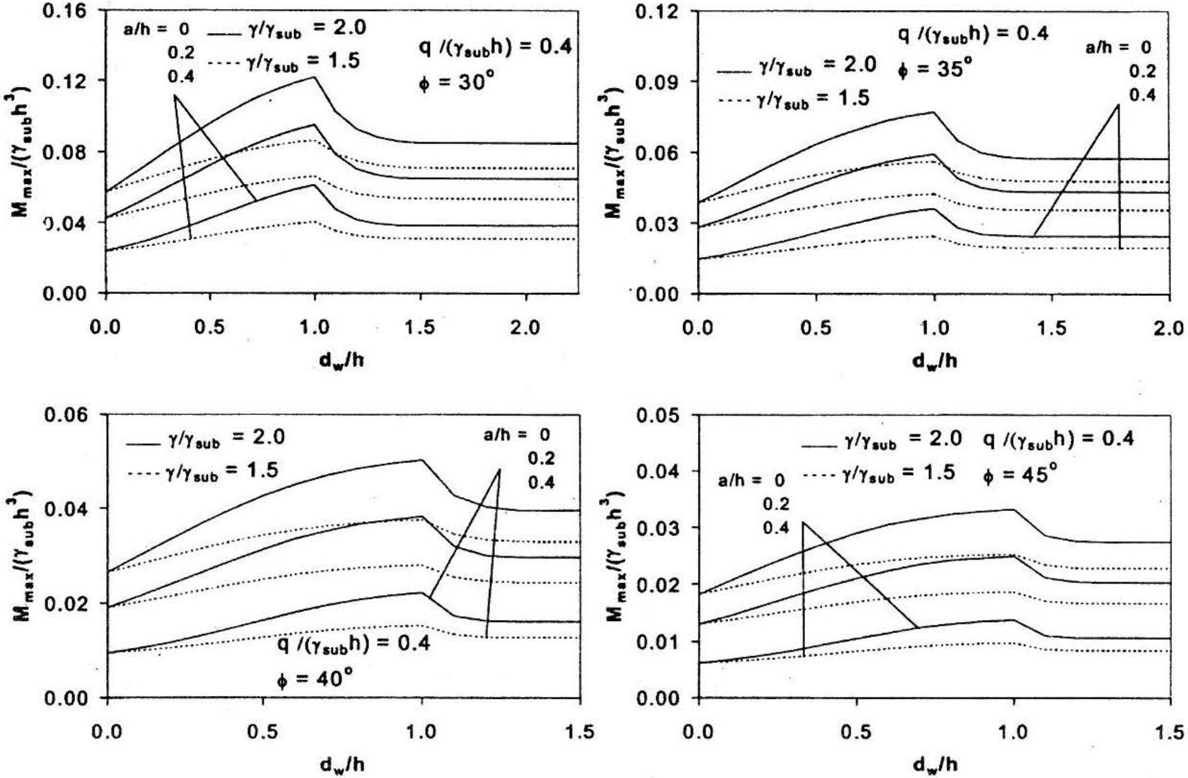

FIGURE 11 : The Variation of $M_{max}/(\gamma_{sub}h^3)$ with d_w/h, a/h and ϕ for $q/(\gamma_{sub}h) = 0.4$

Limitation of the Method

Cornfield (1969) from his extensive research on anchored sheet piles indicated that the fixed earth support method gives satisfactory results provided the sheet pile is buried in sands with a small water difference in the water heads on the two sides of the piling. Cornfield concluded that the method gives erroneous results if (i) a considerable difference in water levels exists between the two sides of the piling, and (ii) the magnitude of the surcharge pressure becomes very high. The results presented in this study have been given only for sandy material and with no difference of water levels on the two sides of the piling. Therefore, it is expected that the obtained design charts will provide useful guidelines for anchored sheet pile design.

Conclusions

For anchored sheet pile walls with equal levels of water on both the sides, it has been found that the position of the water table becomes critical, from the design point of view, when it lies at the dredge level; with this position of the water table the values of (i) the penetration depth, (ii) the section modulus of the piling, and the (iii) tensile force in the anchorage system, become maximum. As the location of the anchorage is shifted downwards, the penetration depth and the section modulus of the sheet pile become smaller, however, the anchor needs to be designed for greater pullout resistance. The magnitudes of d_1/h, $F_a/(\gamma_{sub}h^2)$ and $M_{max}/(\gamma_{sub}h^3)$ increase with increase in the value of ground surcharge pressure and decrease as the angle of internal friction of the soil mass is increased.

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Appendix I

In this appendix, for different types of loading conditions on a beam as shown in Fig.2, the expressions for the moment of the M/EI diagram between points t and o about the point t have been derived.

1. Concentrated Load

With reference to Fig.2a, the bending moment at section Y-Y,

$$M_y = Fy$$

where F = magnitude of the concentrated load, and

y = distance of the section Y-Y from the point of the application of the load.

The moment of M/EI diagram between the points t and o about t will be given by the expression,

$$\delta_h = \frac{1}{EI}\int_0^L M_y (L-y)dy = \frac{1}{EI}\int_0^L Fy(L-y)dy$$

On simplification, it can be indicated that the value of the δ_h will be given by the Eqn.8.

2. Triangular Distributed Loading

With reference to triangular loading as shown in Fig. 2b, it can be seen that the bending moment at section Y-Y,

$$M_y = \frac{p}{x}\left(\frac{xy^2}{2}-\frac{y^3}{6}\right) \text{ for } y = 0 \text{ to } x, \text{ and}$$

$$M_y = \frac{px}{2}\left(y-\frac{x}{3}\right) \text{ for } y = x \text{ to } L$$

where p = maximum intensity of the triangular loading, and

x = length of the loaded portion of the beam.

The moment of M/EI diagram between the points t and o about t will be given by the expression,

$$\delta_h = \frac{1}{EI}\int_0^x \frac{p}{x}\left(\frac{xy^2}{2}-\frac{y^3}{6}\right)(L-y)\,dy + \frac{1}{EI}\int_x^L \frac{px}{2}\left(y-\frac{x}{3}\right)(L-y)\,dy$$

On simplification, it can be shown that the value of the δ_h will be given by the Eqn.9.

3. Uniform Loading

With reference loading shown in Fig.2c, the bending moment at the section Y-Y,

$$M_y = 0.5py^2 \quad \text{for } y = 0 \text{ to } x, \text{ and}$$

$$M_y = px(y-0.5x) \quad \text{for } y = x \text{ to } L$$

where p = intensity of uniform distributed load, and

x = length of the loaded portion of the beam.

The moment of M/EI diagram between the points t and o about t will be given by the expression,

$$\delta_h = \frac{1}{EI}\int_0^x 0.5py^2(L-y)\,dy + \frac{1}{EI}\int_x^L px(y-0.5x)(L-y)\,dy$$

On simplification, it can be indicated that the value of δ_h will be given by the Eqn.10.