CHAPTER IV.

APPLICATION OF A FINITE ELEMENT METHOD TO THE PROBLEM OF CAPACITY OF AXIALLY LOADED SWAY PILES

Introduction

There are two stages to predict the capacity of axially loaded sway piles:

i. At application of horizontal soil movements

ii. At application of vertical pressure on the pile head

At the first stage, failure of the pile will occur if the horizontal soil movements are sufficiently large. However, if that movements are small, reduction of the axial load capacity from that predicted for a straight pile under concentric loading will occur and, then, the arising problem at the second stage is to predict the adequacy of the pile in regard to the ultimate load capacity.

This chapter describes the analytical procedures which demonstrate how the method of analysis developed in the previous chapter can be used to predict the capacity of axially loaded piles at the two stages mentioned above. In obtaining a suitable finite element model, parametric solutions for a wide range of idealized cases are presented and their advantages and limitations are discussed. Then, some comments on the significant parameters are given and applicability of the proposed method to practical problems is illustrated and discussed. Finally, a comparison among the pile movements predicted by the proposed method and by Poulos method (16) and those measured is presented.

Analytical procedure

Application of the proposed method to practical problems can be described by the following procedures:

i. Calculate a range of possible failure loads and moments for a pile section using fundamental structural theories based on pile material. This can be obtained by means of either moment-curvature-axial compression (M-Ψ-P) relationship or interaction diagram.

ii. Measure the horizontal movement of the pile head at selected sites. The distribution of horizontal pile movement near the ground surface, the position of crack occurance, the reduction of pile section and the pile load test may be required to confirm the solutions predicted by the proposed method.

iii. Analyze the problem by using the proposed method.

In analyzing the problem by using the proposed method, the following comments on the step of analysis are made:

In the Run No.2, the horizontal soil movement distribution with depth equalled to the soft soil layer below the ground surface is applied incrementally until reasonably good agreement between predicted and measured horizontal movements of the pile head has been obtained prior to proceeding to the Run No.3. If failure of the pile does not occur, the analysis may be proceeded to the next load increment for predicting the limiting lateral deflection of the pile.

In the Run No.3, the vertical pressure on the pile head representing the design axial load of a straight pile under concentric loading is applied to predict the capacity of axially loaded sway pile. Similarly, if failure of the pile does not occur, the analysis may be proceeded to the next load increment for predicting the ultimate load capacity.

Parametric Studies of Pile Behaviour

To obtain a suitable finite element model for practical problems, parametric solutions for a wide range of idealized cases are examined and investigations into the effect of various factors on the development of pile movements and curvatures resulting from horizontal soil movements are made. In most of such solutions, a standard soil movement profile (16) as shown in Figure 4.1 is assumed.

1. Effect of Soil Mass on Both Sides of a Pile

For idealized finite element models SYM.104, SYM.88, SYM.72, SYM.56, SYM.40 and SYM.24 which have symmetrical soil mass on both

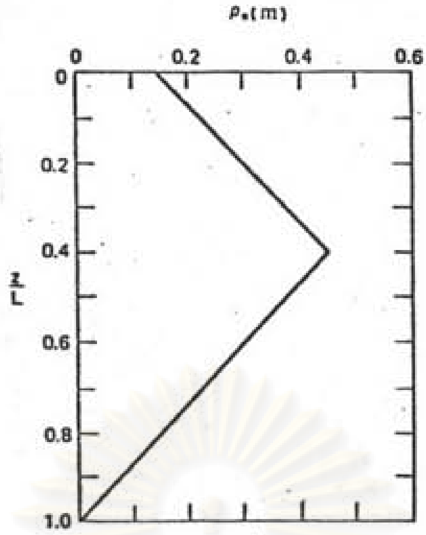

FIGURE 4.1 Standard Soil Movement Profile (16)

sides of a pile and the typical pile and soil parameters as shown in Figure 4.2 subjected to standard soil movement profile within the soft soil layer, the solutions for distributions of pile movement and curvature are shown in Figure 4.3. It is found that the pile movements and curvatures increase as the soil mass on both sides of a pile decreases. For model SYM.40 and SYM.24 which have small soil mass on both sides of the pile, the pressures developed in the soil mass may exceed its lateral resistance. In such case, the proposed method may give poor solutions since the responses of pile in elastic soil are different from those in elastic-plastic soil as investigated by Poulos (16).

2. Effect of Soil Mass on Each Side of a Pile

In the idealized finite element model, the soil mass on induced face is used to transfer the specified displacements on the boundaries to the pile, while the soil mass on reacting face is used

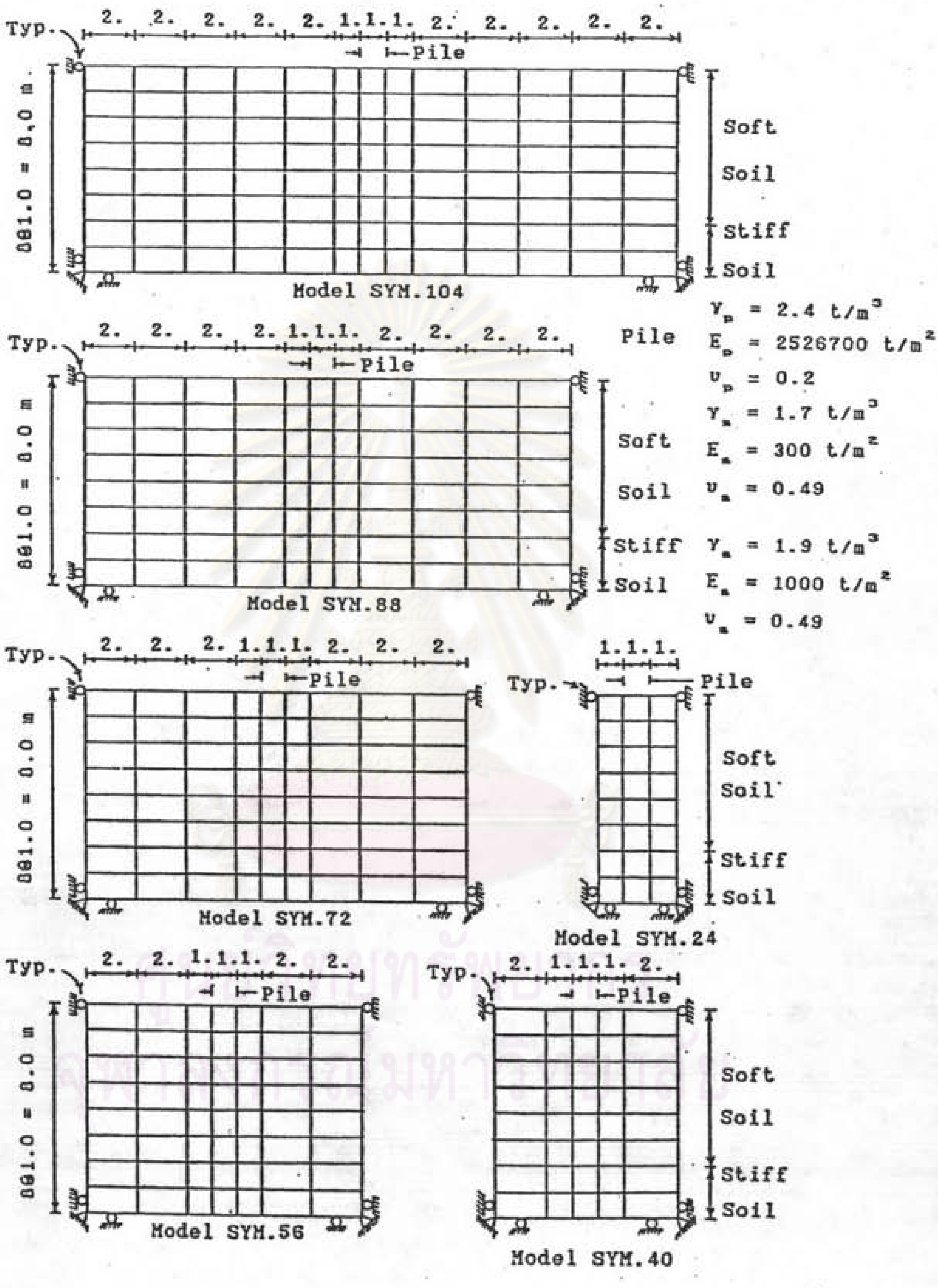

FIGURE 4.2 Idealized Finite Element Models used for Analysis of Effect of Soil Mass on Both Sides of a Pile

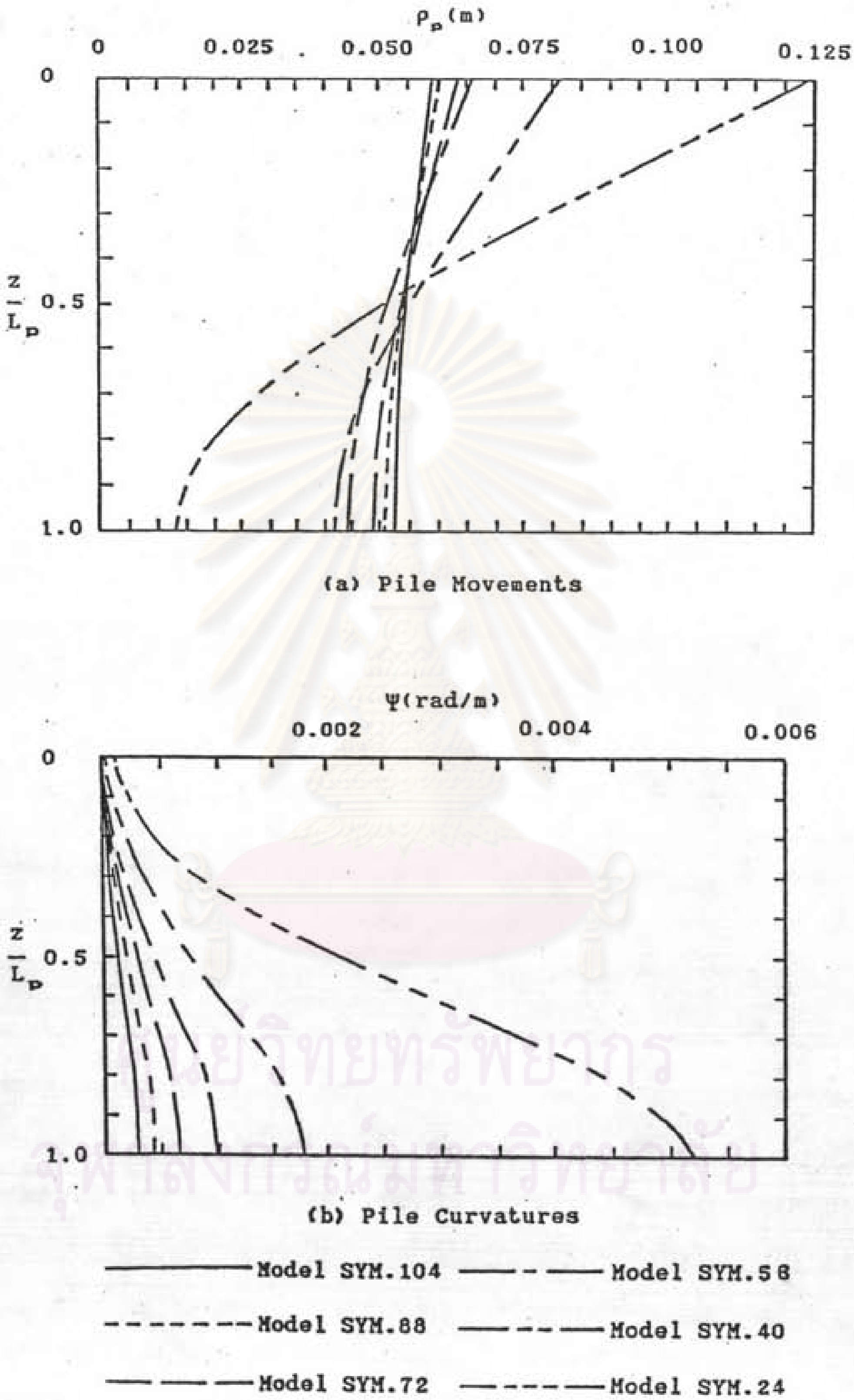

FIGURE 4.3 Effect of Soil Mass on Both Sides of a Pile

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to represent the soil resistance. In the analysis, the soil movement profile in the vicinity of the pile should be applied near the pile. In addition, the soil mass on reacting face should be sufficiently large, so that there is no effect of the boundaries on the pile behaviour. Therefore, a suitable finite element model for practical problem should have unsymmetrical soil mass on both sides of a pile.

For models UNSYM.48 and UNSYM.40, which have unsymmetrical soil mass on both sides of a pile, and the typical pile and soil parameters as shown in Figure 4.4 subjected to standard soil movement profile within the soft soil layer, the solutions for pile displacements and curvatures are shown in Figure 4.5. This figure also shows the solutions obtained previously from models SYM.56 and SYM.40. As the soil mass on induced face decreases and on reacting face increases, the pile displacements and curvatures increase. It is found

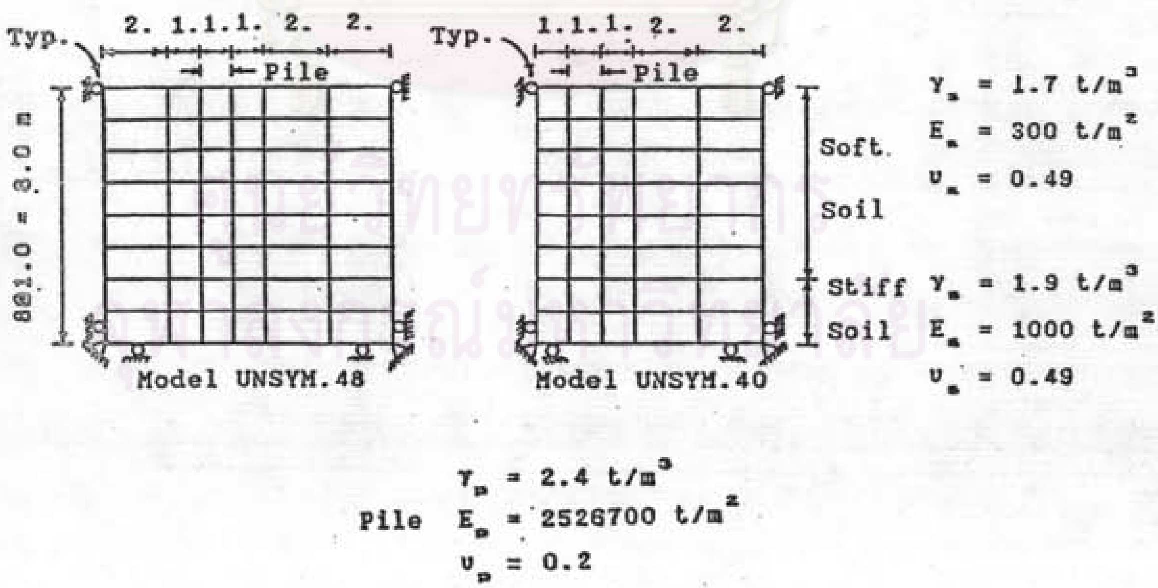

FIGURE 4.4 Idealized Finite Element Models used for the Analysis of Effect of Soil Mass on Each Side of a Pile

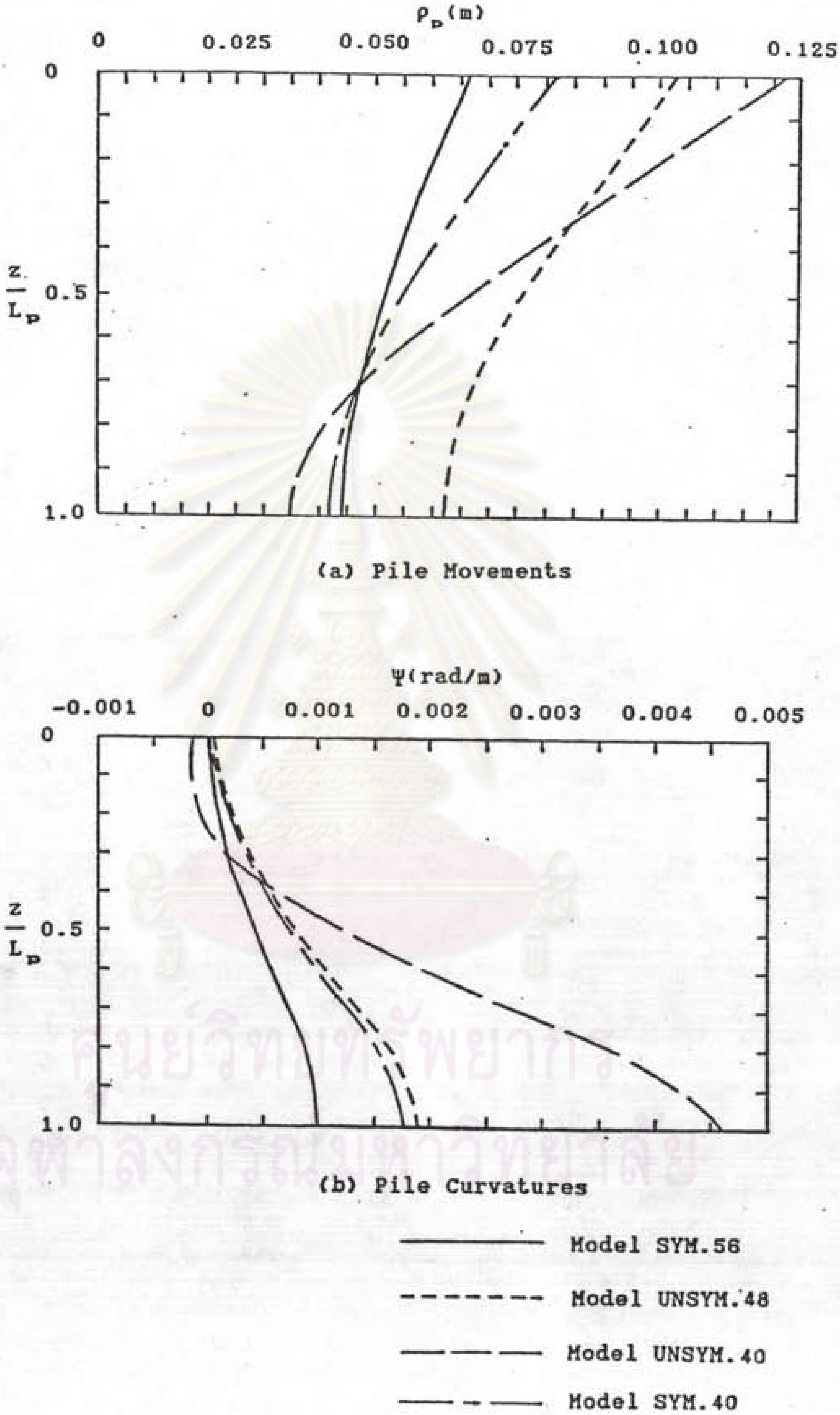

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FIGURE 4.5 Effect of Soil Mass on Each Side of a Pile that the pile curvatures obtained from model UNSYM.40 become negative near the ground surface that is slightly different from those obtained from models SYM.56 and UNSYM.48. It should be observed that a more flexible model can be obtained by extending the soil mass on reacting face and reducing that on induced face.

It can be suggested that the soil mass on induced face should not greater than three times the width or diameter of the pile and that on reacting face should not less than five times the width or diameter of the pile.

3. Effect of Depth of Pile Tip Embedment in Stiff Soil Stratum

An example of the effect of depth of pile tip embedment in stiff soil stratum is demonstrated using models UNSYM.48 as shown in Figure 4.4, UNSYM.36, UNSYM.60 and UNSYM.72 and the typical pile and soil parameters as shown in Figure 4.6 subjected to standard soil movement profile within the soft soil layer. These models have the soil mass on each side of a pile as those suggested previously. The solutions for pile movements and curvatures are shown in Figure 4.7, which reveal that the depth of pile tip embedment in stiff soil stratum plays an important role on pile movements near the tip. As that depth increases, the pile tip curvature decreases significantly and its boundary condition tends to be a more fixity. The provision of pinned tip in the model UNSYM.36 increases pile movements near the head and curvatures near the tip. This effect may be used to represent a partially fixed condition at the pile tip.

It can be suggested that the actual length of the pile should be used in the analysis.

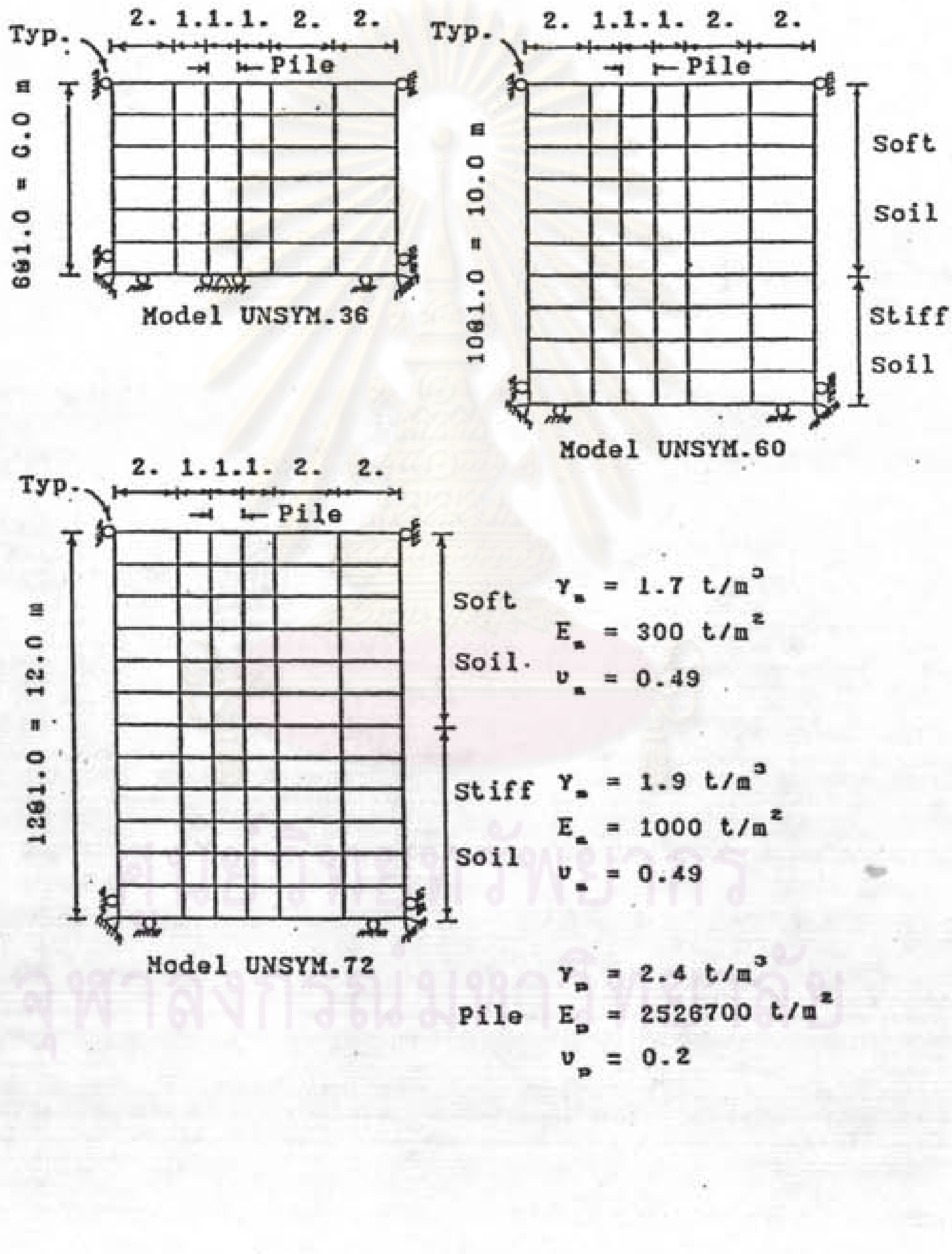

FIGURE 4.6 Idealized Finite Element Models used for the Analysis of Effect of Depth of Pile Tip Embedment in Stiff Soil Stratum

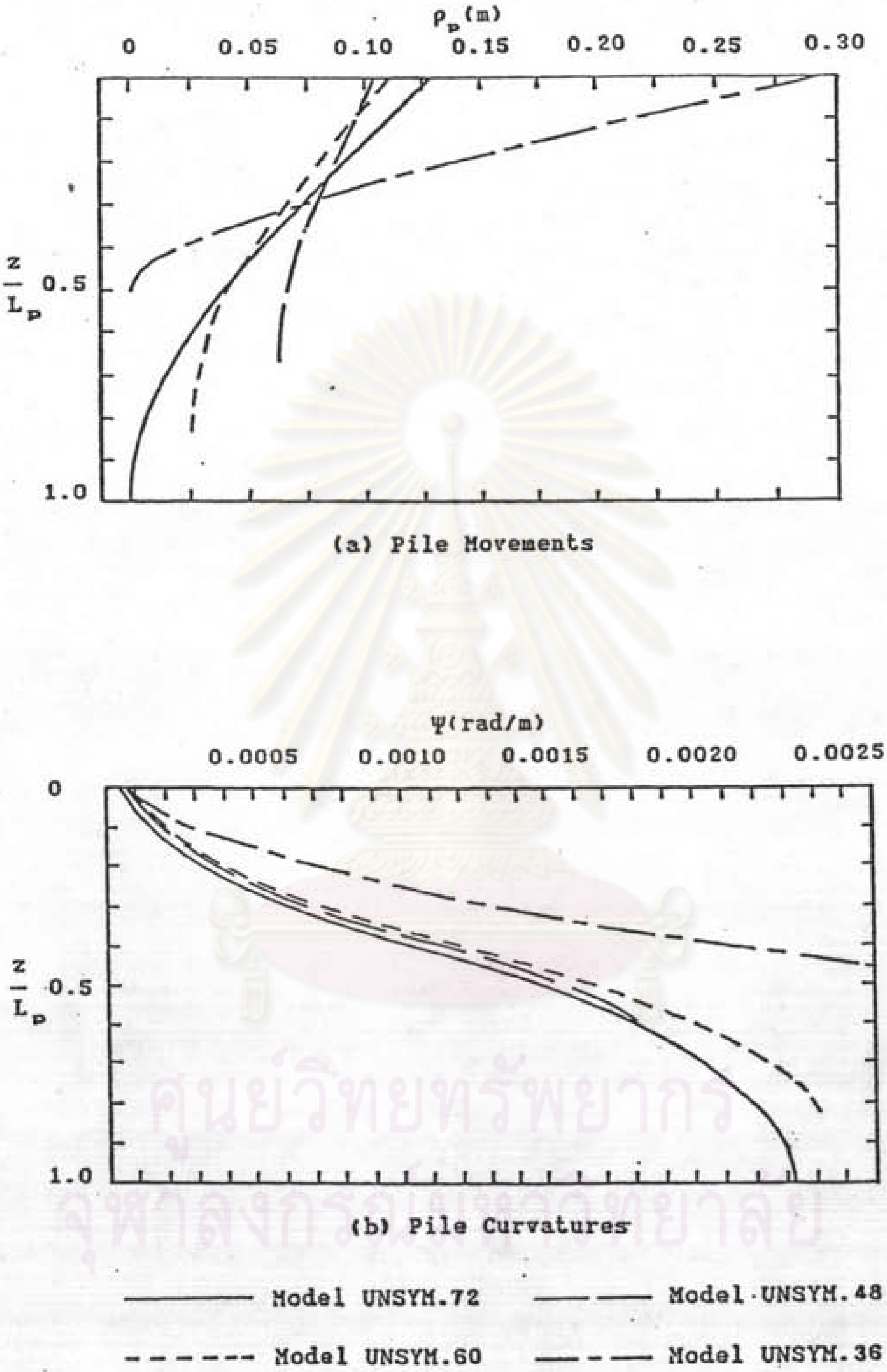

FIGURE 4.7 Effect of Depth of Pile Tip Embedment in Stiff Soil Stratum

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4. Effect of Aspect Ratio of Finite Element and Mesh Refinement

Based on finite element concept, the best aspect ratio of finite element for analysis is 1:1 and the mesh refinement tends to soften the finite element model. In addition, for compatible or conforming element as used in the analysis the solutions approach their exact values as the mesh is refined. For models UNSYM.96, UNSYM.64.R1.5, UNSYM.48.R2, UNSYM.32.R3 and UNSYM.16.R6 having aspect ratio of 1:1, 1.5:1, 2:1, 3:1 and 6:1, respectively and the typical pile and soil parameters as shown in Fugure 4.8 subjected to standard soil movement profile within the soft soil layer, the solutions for pile movements and curvatures are shown in Figure 4.9. It is found that the solutions obtained from aspect ratios of 1:1, 1.5:1, 2:1 and 3:1 are in close agreement and those of 1:1, 1.5:1 and 2:1 agree with the finite element concept mentioned above.

It can be suggested that the aspect ratio of element should not exceed 2:1 for the pile and 4:1 for the soil, and that the model should be more refined in the vicinity of the pile and between the head and twice the depth of soil movement profile.

5. Effect of Magnitude of Soil Movement

A model UNSYM.72 and the typical pile and soil parameters as shown in Figure 4.6 have been examined for which the distribution of soil movement remains the same, but its magnitude increases. Solutions for pile displacements and curvatures are shown in Figure 4.10. As would be expected, pile displacements and curvatures increase as the

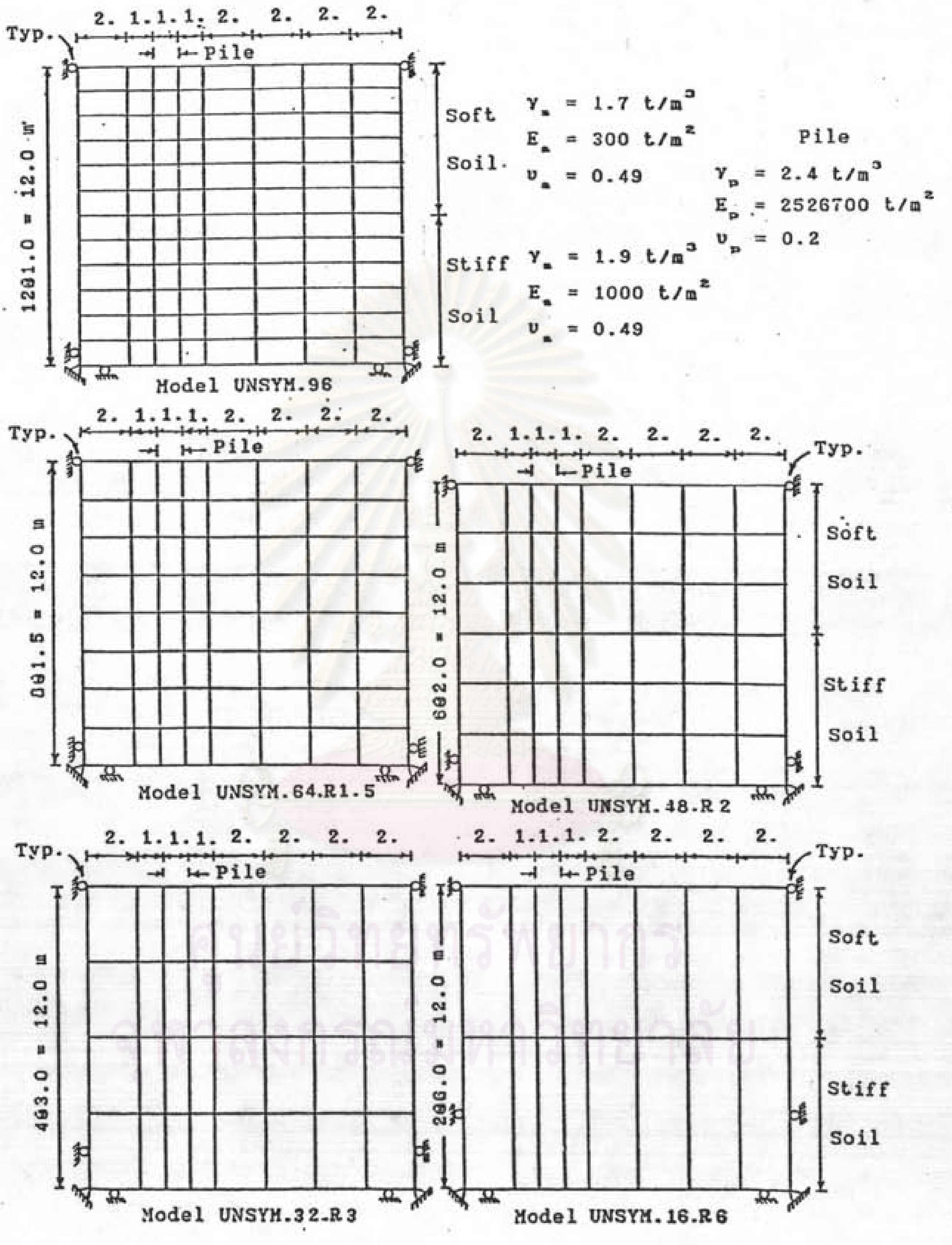
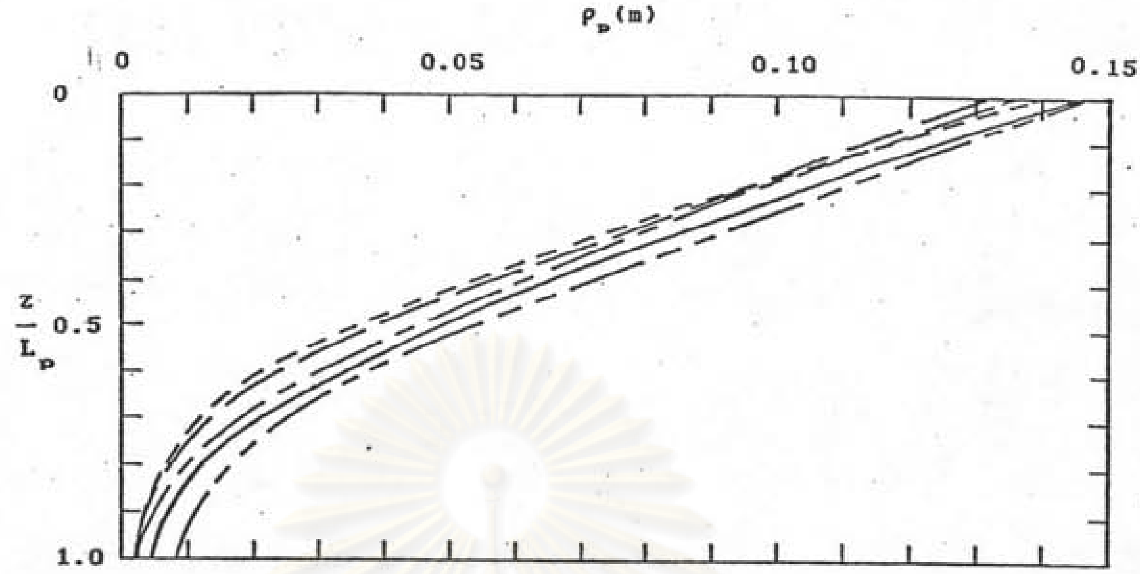
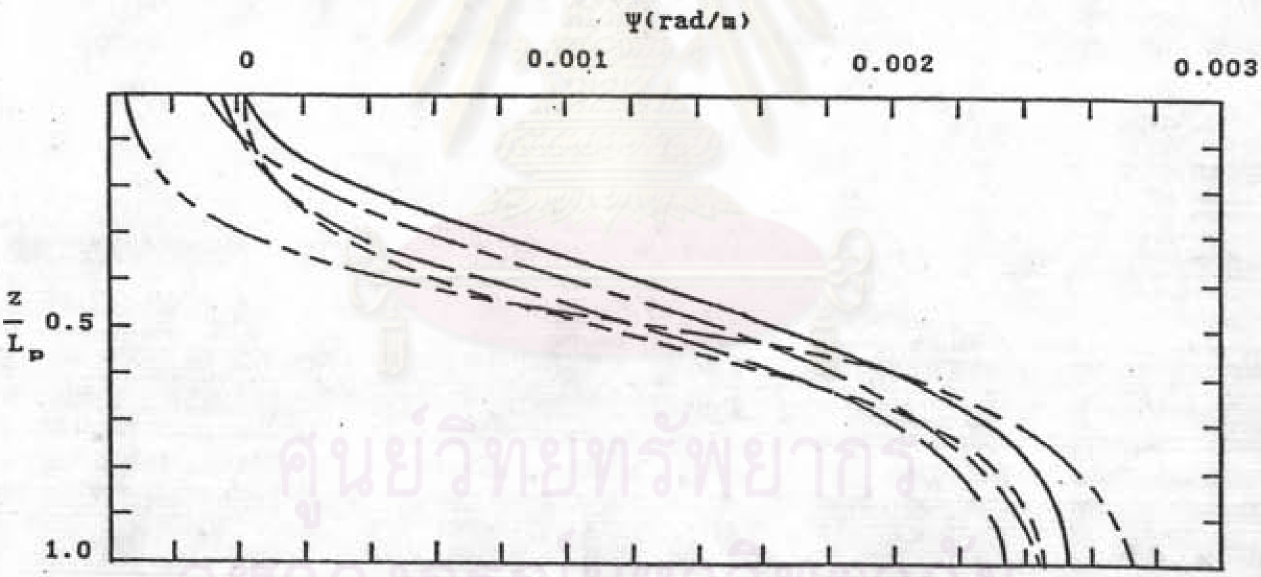


FIGURE 4.8 Idealized Finite Element Models used for the Analysis of Effect of Aspect Ratio of Finite Element and Mesh Refinement



(a) Pile Movements



(b) Pile Curvatures

FIGURE 4.9 Effect of Aspect Ratio of Finite Element and Mesh Refinement

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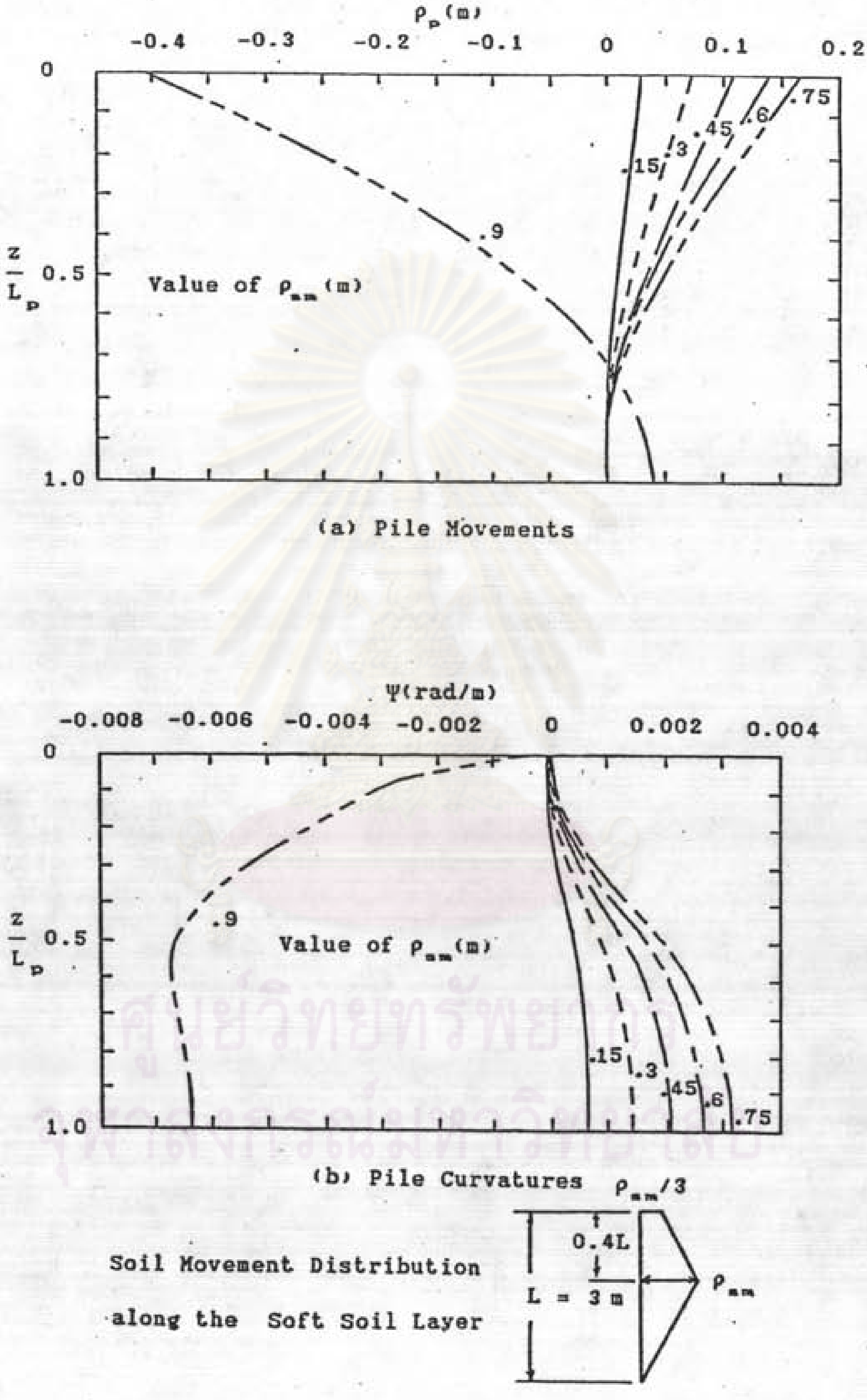

FIGURE 4.10 Effect of Magnitude of Soil Movement

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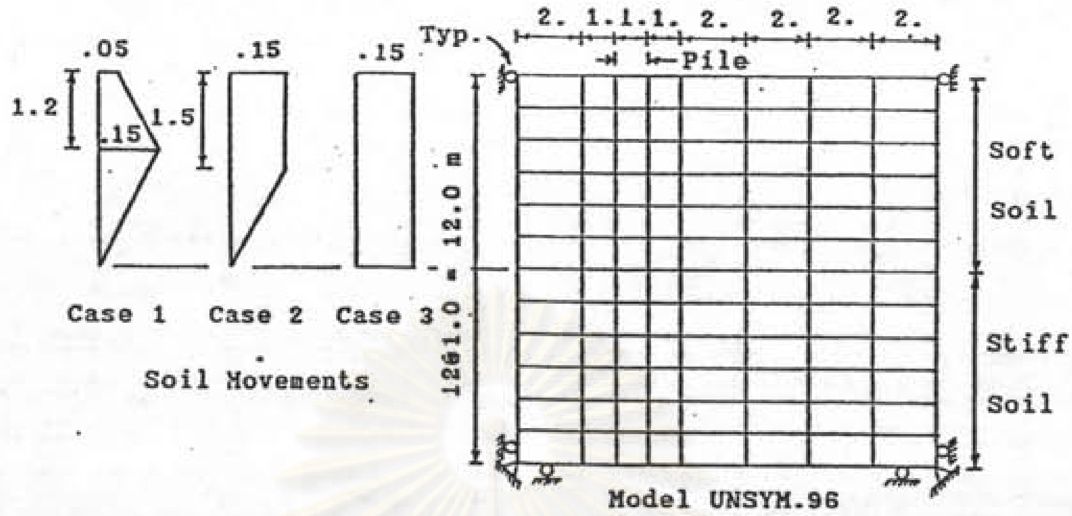
magnitude of soil movement increases. However, for very large magnitude of soil movements, in case of ρ_{sm} = 0.9 m, the pile displaces to the soil on induced face and the curvatures become negative. This shows that the effect of large magnitude of soil movements may lead to unreasonable solutions. Thus, it is interesting to note that the large magnitude of soil movements should be applied incrementally to confirm that the solutions obtained from the current load increment conform to those obtained from the previous load increment. In such a case, a large soil mass on reacting face may be employed to soften the finite element model as previously described.

6. Effect of Soil Movement Distribution

For the model UNSYM.96 and the typical pile and soil parameters subjected to three soil movement profiles within the soft soil layer as shown in Figure 4.11, pile movements and curvatures are shown in Figure 4.12. In all three cases, the maximum soil movement is the same. It is found that the pile head movement is largely dependent on the magnitude of the soil surface movement and that the maximum pile curvature is greatest for the uniform soil movement profile. As the soil movement profile tends to a triangular distribution with zero movement at the base of the soft soil layer, the pile movements and curvatures tend to decrease.

7. Effect of Depth of Soil Movement

For model UNSYM.72 and the typical pile and soil parameters subjected to three soil movement profiles within the soft soil layer as



Soft Soil

γ_s = 1.7 t/m³

E_s = 300 t/m²

υ_s = 0.49

Stiff Soil

γ_s = 1.9 t/m³

E_s = 1000 t/m²

υ_s = 0.49

Pile

γ_p = 2.4 t/m³

E_p = 2526700 t/m²

υ_p = 0.2

FIGURE 4.11 Idealized Finite Element Model used for the Analysis of Effect of Soil Movement Distribution

shown in Figure 4.13, the solutions are shown in Figure 4.14. In all three cases, the distribution of soil movement remains the same, but its depth decreases. The pile movements and curvatures increase as the depth of soil movement increases. This effect is similar in principle to the effect of soil movement distribution examined previously.

8. <u>Effect of Soft Soil Modulus</u>

The model UNSYM.72.D0.5 and the typical pile and soil parameters as shown in Figure 4.15 subjected to standard soil movement profile within the soft soil layer is used to analyze for values of E_s for soft soil of 100 t/m^2 and 300 t/m^2. For the same magnitude of soil

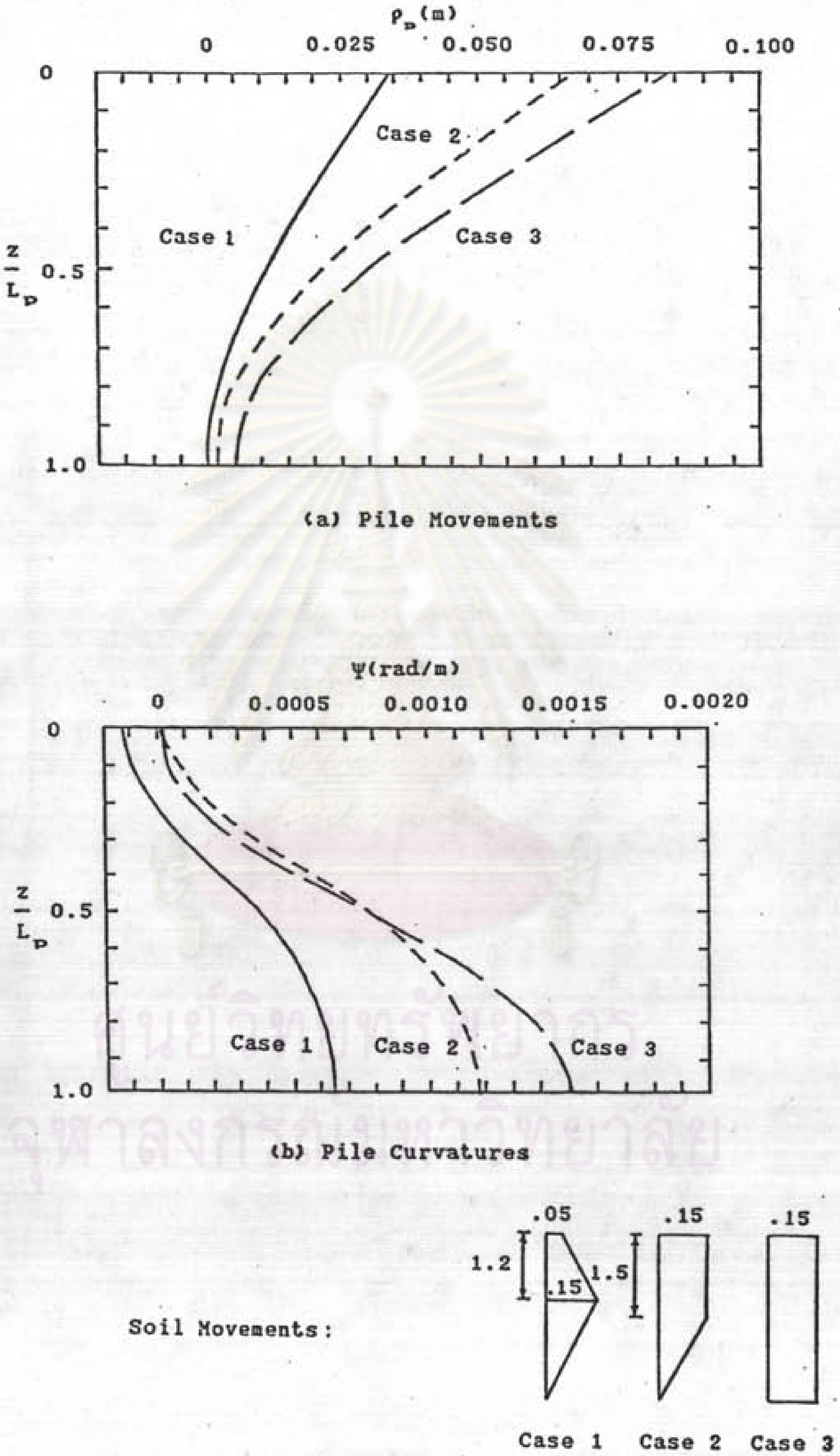


FIGURE 4.12 Effect of Soil Movement Distribution

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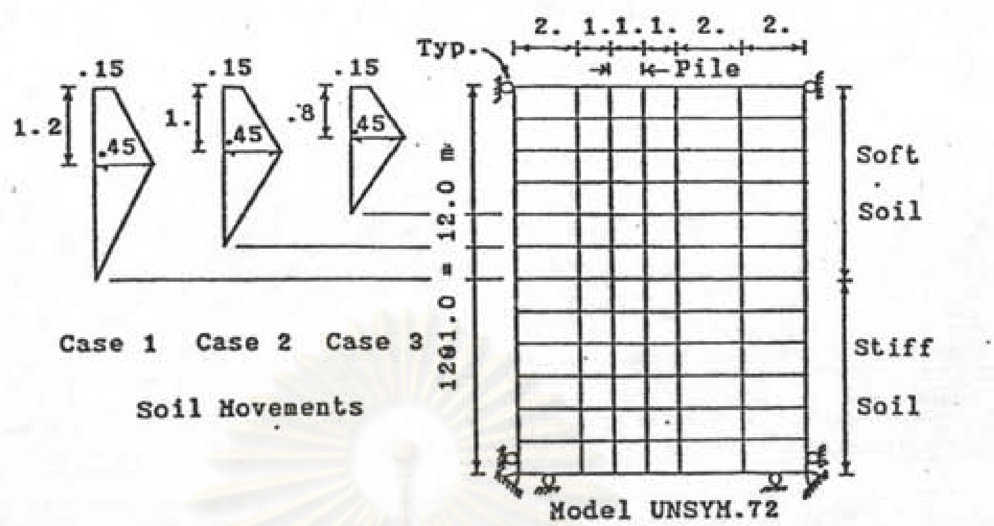


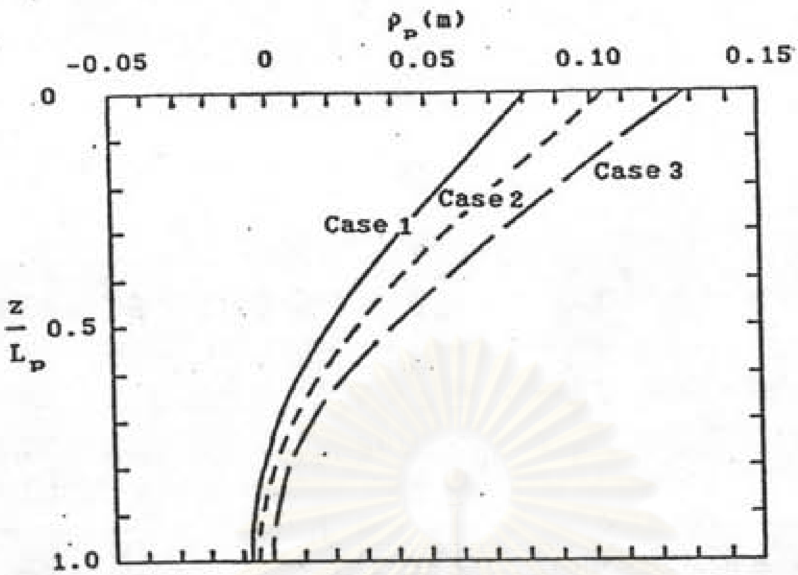
Soft Soil	Stiff Soil	Pile
$\gamma_s = 1.7\ t/m^3$	$\gamma_s = 1.9\ t/m^3$	$\gamma_p = 2.4\ t/m^3$
$E_s = 300\ t/m^2$	$E_s = 1000\ t/m^2$	$E_p = 2526700\ t/m^2$
$\upsilon_s = 0.49$	$\upsilon_s = 0.49$	$\upsilon_p = 0.2$

FIGURE 4.13 Idealized Finite Element Model used for the Analysis of Effect of Depth of Soil Movement

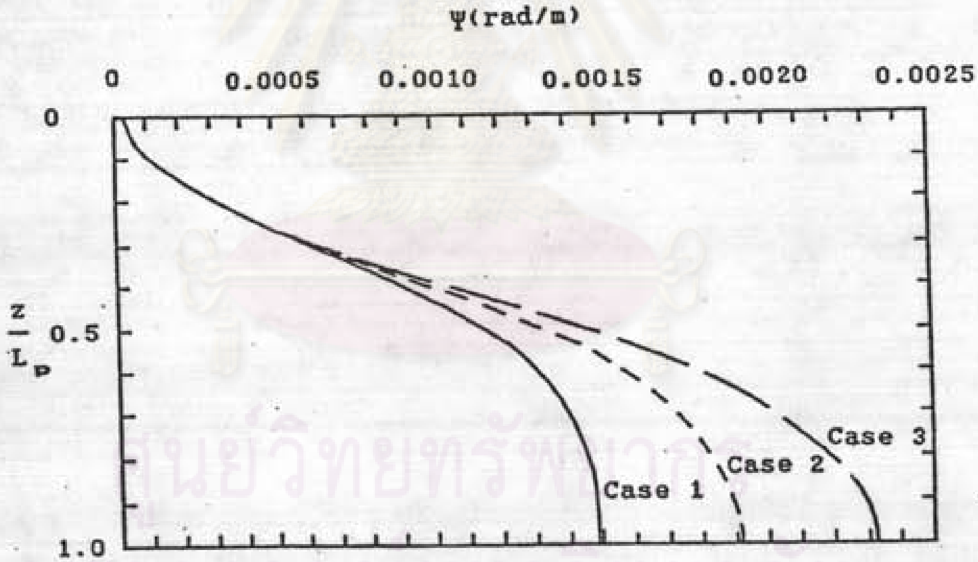
movement profile which is applied in terms of a set of specified displacements on the boundaries, that magnitude made at the face of the pile increases as the value of E_s for soft soil increases. Thus, increasing the value of E_s for soft soil leads to increasing pile movements and curvatures as shown in Figure 4.16.

9. Effect of Pile Modulus

To examined the effect of E_p, the model UNSYM.72 and the typical soil parameters as shown in Figure 4.6 subjected to standard soil movement profile within the soft soil layer is used to analyze for various values of E_p. The variation of E_p used in the analysis are



(a) Pile Movements



(b) Pile Curvatures

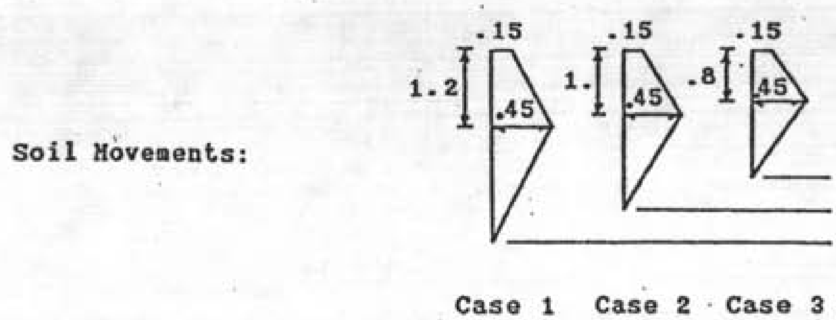


FIGURE 4.14 Effect of Depth of Soil Movement

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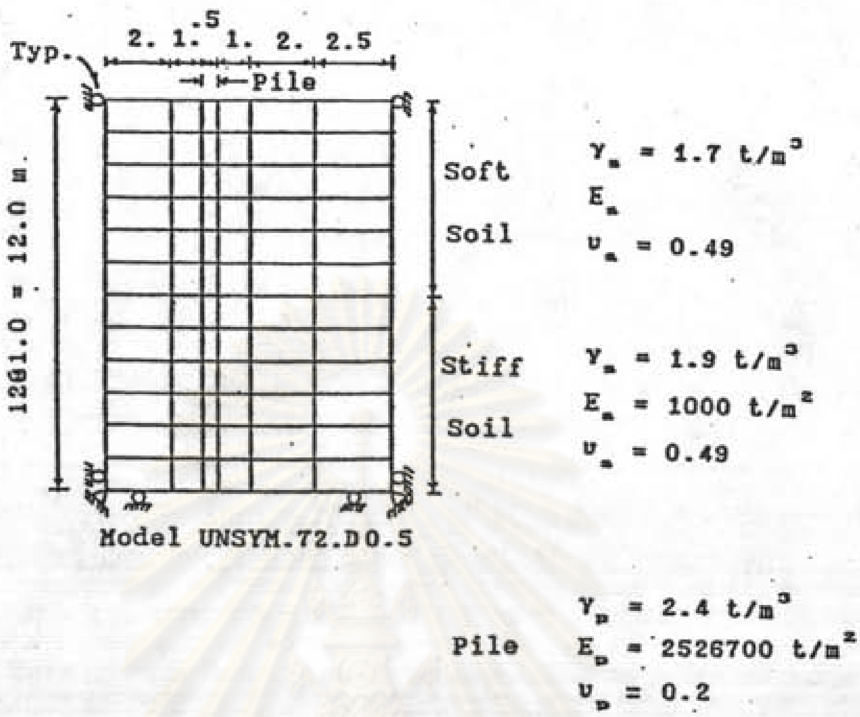


FIGURE 4.15 Idealized Finite Element Model used for the Analysis of Effect of Soft Soil Modulus

as follows:

Case 1: E_p of 25,267,000 t/m^2

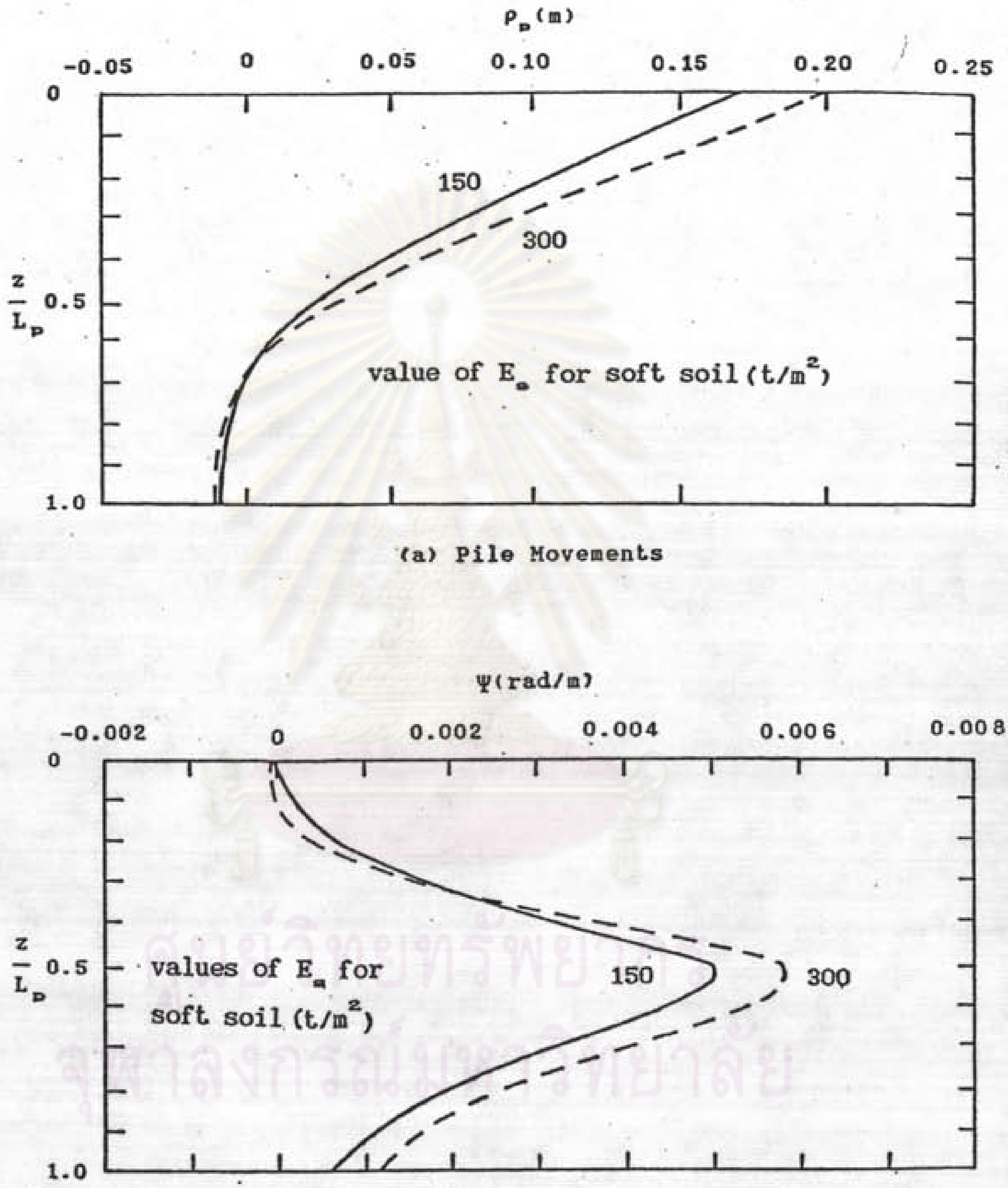
Case 2: E_p of 2,526,700 t/m^2

Case 3: E_p of 252,670 t/m^2

Case 4: E_p of 25,267 t/m^2

Case 5: No pile element

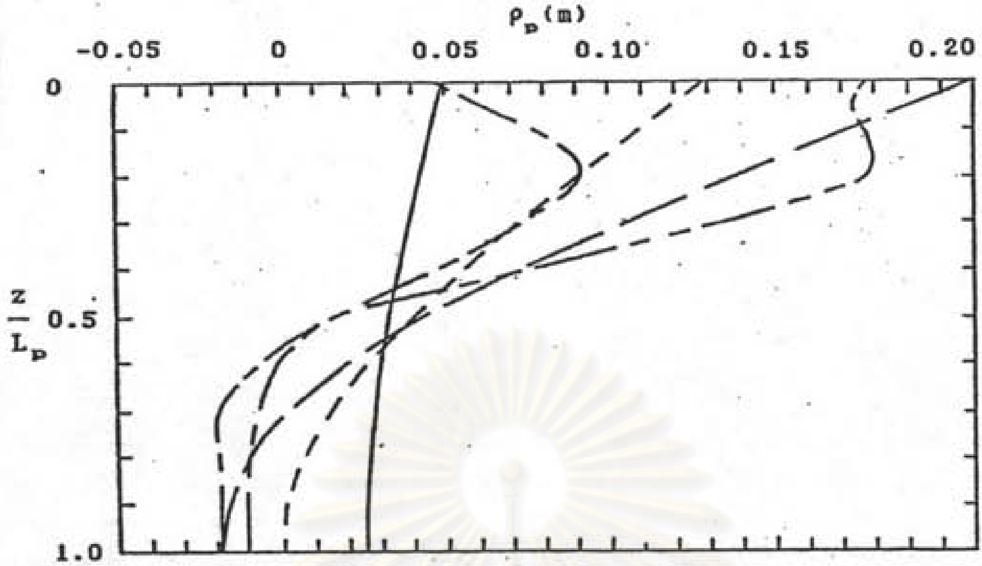
Case 5 having uniform stiff and soft soil stratums without pile is used to predict the soil movement distribution at face of the pile. Solutions for pile and soil movements and pile curvatures are shown in Figure 4.17. For a very flexible pile, case 4, pile movement



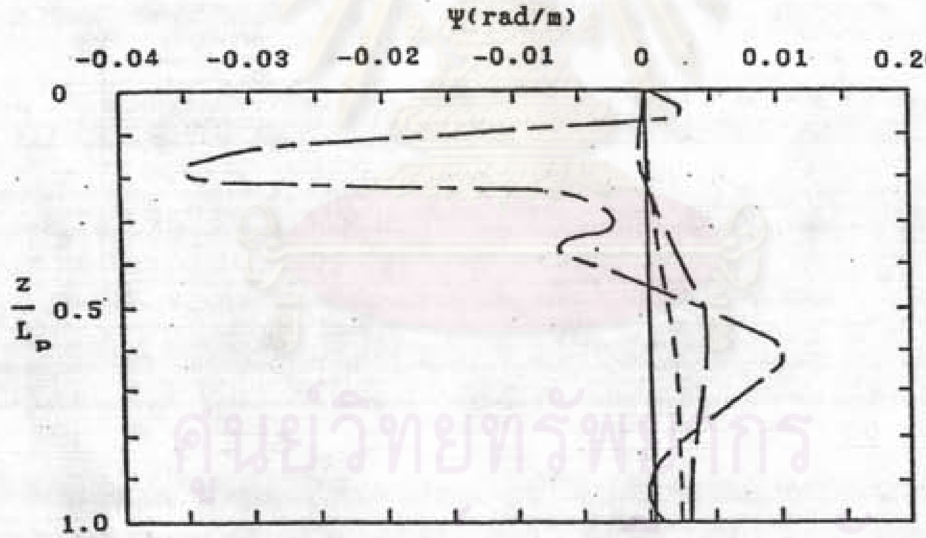
(a) Pile Movements

(b) Pile Curvatures

FIGURE 4.16 Effect of Soft Soil Modulus



(a) Movements



(b) Pile Curvatures

Case 1: E_p of 25,267,000 t/m^2

Case 2: E_p of 2,526,700 t/m^2

Case 3: E_p of 252,670 t/m^2

Case 4: E_p of 25,267 t/m^2

Case 5: No pile element

FIGURE 4.17 Effect of Pile Modulus

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distribution appears to be similar to the soil movement distribution at face of the pile. As the value of E_p increases, pile displacements and curvatures decrease and their distributions change. Although increasing E_p reduces the pile curvatures, it leads to greater moments.

10. Effect of Width or Diameter of Pile

An example of effect of width or diameter of the pile, d, has been shown by using models UNSYM.72 as shown in Figure 4.6, UNSYM.72.D0.5 as shown in Figure 4.15 and UNSYM.72.D2 as shown in Figure 4.18 and the typical pile and soil parameters subjected to standard soil movement profile within the soft soil layer. The value of d in model UNSYM.72.D2, UNSYM.72 and UNSYM.72.D0.5 are equal to 2, 1 and 0.5 m,

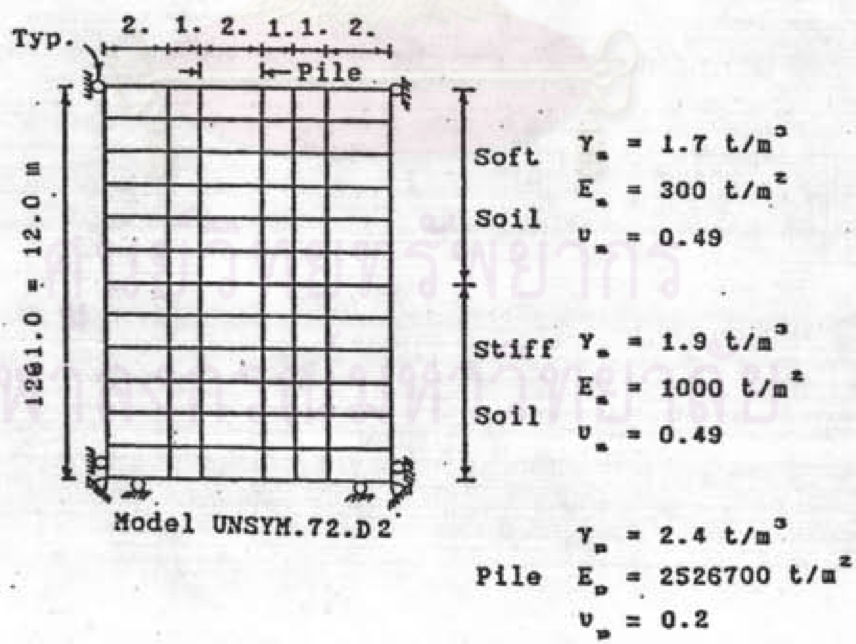
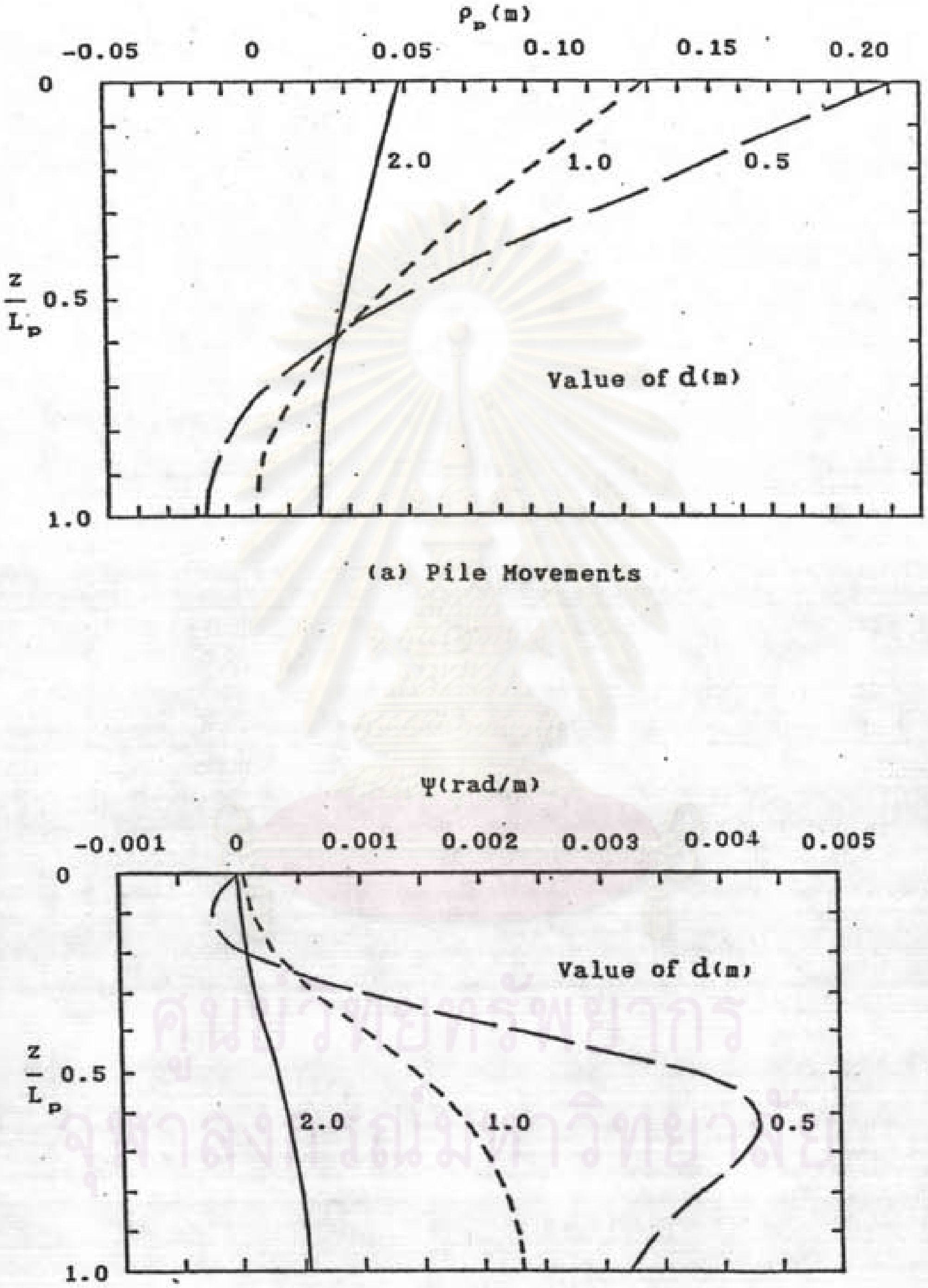


FIGURE 4.18 Idealized Finite Element Model used for the Analysis of Effect of Width or Diameter of pile



(a) Pile Movements

(b) Pile Curvatures

FIGURE 4.19 Effect of Width or Diameter of pile

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respectively. The solutions for three values of d are shown in Figure 4.19, which reveal that pile movements and curvatures increase substantially as the value of d decreases. As noted by Poulos (16), the effect of changing width or diameter of the pile is to change pile stiffness or modulus. This can be confirmed by comparing the solutions obtained from d of 2.0 and 0.5 m with those obtained from case 1 and 3 in the effect of E_p examined previously, which reveal excellent agreement.

11. Summary and Conclusions

Parametric studies on behaviour of the pile in a soil under horizontal soil movements have been made in order to obtain a suitable idealized finite element model for practical problems. Such studies have been carried out by investigating the effect of various factors on the development of moments and curvatures in the pile, which have revealed the following conclusions:

1. The most significant factors are the pile modulus and the distribution of horizontal soil movement. Variations of width or diameter of the pile and the soil modulus are reflected in a corresponding variation of the pile modulus.

2. It is an advantage to have a more flexible idealized finite element model by reducing the soil mass on the induced face and extending that on the reacting face. The soil mass on induced face should be less than three times the width or diameter of the pile and that on reacting face should be greater than five times the width or

diameter of the pile.

3. The actual length of the pile should be used in the analysis.

4. For analysis of the pile having a large measured horizontal movement of the head, the idealized finite element model should be a flexible one, in which a large horizontal soil movement profile is applied incrementally to confirm that the solutions obtained from the current load increment conform to those obtained from the previous load increment.

5. The aspect ratio of the element should not exceed 2:1 for the pile and 4:1 for the soil.

6. The idealized finite element model should be more refined in the vicinity of the pile, and between the head and twice the depth of the horizontal soil movement profile.

Application of The Method of Analysis to Practical Problems

1. Determination of Parameters

In the analysis, the required input data, in addition to pile geometry and boundary conditions, are pile and soil parameters. The values of pile Poisson's ratio υ_p and unit weight γ_p are based on its material. Those of the soil are as follows:

υ_s of 0.49 for cohesive soils and

of 0.3 for cohesionless soils.

γ_s of between 1.7 t/m^3 and 1.9 t/m^3 for cohesive soils and

of 2.0 t/m^3 for cohesionless soils.

The distribution of horizontal soil movement, the soil and pile moduli are commented on as follows:

(a) Horizontal Soil Movements: In principle, prediction of horizontal soil movements arising from embankment or excavation can be obtained either from elastic theory for uniform soil profile, or from a finite element analysis for more complicated cases. For soft soil stratum loaded by heavy embankments in which soil pressure in some areas may reach its yield pressure and elastic analysis may not be valid, the proposed program taking into account limiting shear stress in the element may be used to predict the horizontal soil movements. Nevertheless, a number of comparisons between predicted and measured horizontal soil movements made by Poulos (18) have revealed that the predicted values are generally considerably greater than the measured values, despite the fact that good agreement is obtained between predicted and measured settlements. In such cases, the discrepancy between the predicted and measured horizontal soil movements appears to be greater for stiff foundation soil than for soft foundation soil. Thus, if possible, it is desirable to use the horizontal soil movements observed by inclinometer in situ as the input data (16).

(b) Soil Modulus E_s: At present, the best means of obtaining E_s is to backfigure from a full scale in situ lateral load test and to use horizontal plate load tests at various depths in case of bored piles (12). However, in case of which such tests cannot be performed, some correlations have been suggested by a number of published load-deflection measurements on full-scale piles for constant values of E_s with depth as follows:

For cohesive soils, Poulos suggested values of E_s between 15 c_u and 95 c_u with an average value of 40 c_u, where c_u is the undrained shear strength (12). Marche and Lacroix suggested values of E_s between 25 c_u and 75 c_u (16). Later in 1978 Banerjee and Davies suggested values of E_s between 100 c_u and 180 c_u (1). For cohesionless soils, Poulos suggested values of E_s varying between 90 t/m^2 and 980 t/m^2 depending on initial relative density (16).

In this research, the values of E_s of between 100 c_u and 300 c_u are suggested for cohesive soils. The lower values tend to be associated with very soft clays and the higher values with very stiff clays. For cohesionless soils, the value of E_s of 2000 t/m^2 is suggested.

(c) <u>Pile Modulus E_p</u>: The value of E_p used in the analysis is not its actual but its idealized value since in the state of plane strain the finite element idealization of the actual cross section of the pile is transformed to a rectangular cross section of depth 1 m, in which the width or diameter remains the same. This change in pile cross section leads to a change in its moment of inertia, I_p. By assuming the actual pile stiffness to be equal to that idealized, the value of E_p used in analysis can be obtained.

2. Illustrative Application

To illustrate the applications of the proposed method described in the previous chapter and the parametric studies described in the previous section, the case of a cast in situ bored pile is considered herein.

A 0.6 m diameter bored pile section was reinforced symmetrically by six 12 mm diameter bars with covering of 7.5 cm. This pile, 24.6 m long was situated in a soil profile of clay layers. The concrete had a cylinder strength f_c' of 303 ksc. The steel had a modulus of elasticity of $2.1(10)^6$ ksc and a yield strength of 3000 ksc. The working load on this pile was 100 tons. The head of this pile was observed to have horizontal movement of 5.0 cm due to horizontal soil movements resulting from excavation.

Checking for integrity of this pile was done accordingly by means of shock/seismic test, which revealed the following conclusions:

i. Test No.1: This pile had a minor crack at depth about 2.50 - 3.00 m and a weakness in the concrete occuring at depth 12.0 m. This pile was suggested to retest.

ii. Retest: This pile was tested after the head had been cut down to the level 1.0 m below the head and was found to have an intrusion/necking occuring at 10.0 m depth. This intrusion/necking is equivalent to 15 % of the cross sectional area of the pile diameter. There was no crack along the pile shaft.

In the analysis, the following assumptions are made:

1. The value of E_p of $9.29(10)^5$ t/m^2 is used. This value is obtained from dividing the actual E_pI_p by the idealized I_p, in which the actual E_p and actual I_p and the idealized I_p are $2.63(10)^6$ t/m^2, $6.36(10)^{-3}$ m^4 and $1.80(10)^{-2}$ m^4, respectively.

2. The constant value of E_s for each clay layer obtained from previously suggested correlations is used.

An idealized finite element model of the pile and soil and their parameters and the horizontal soil movement profile are shown in Figure 4.20.

The objectives are to investigate the adequacy of the pile capacity at the following stages:

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(a) At application of horizontal soil movements

At this stage, the horizontal soil movement profile along the soft soil layer is applied incrementally in terms of a set of specified displacements on the boundaries in the model. Solutions for pile movement and curvature distributions are shown in Figure 4.21 and 4.22, respectively. The predicted maximum curvatures plotted against the predicted horizontal movements of the pile head is shown in Figure 4.23.

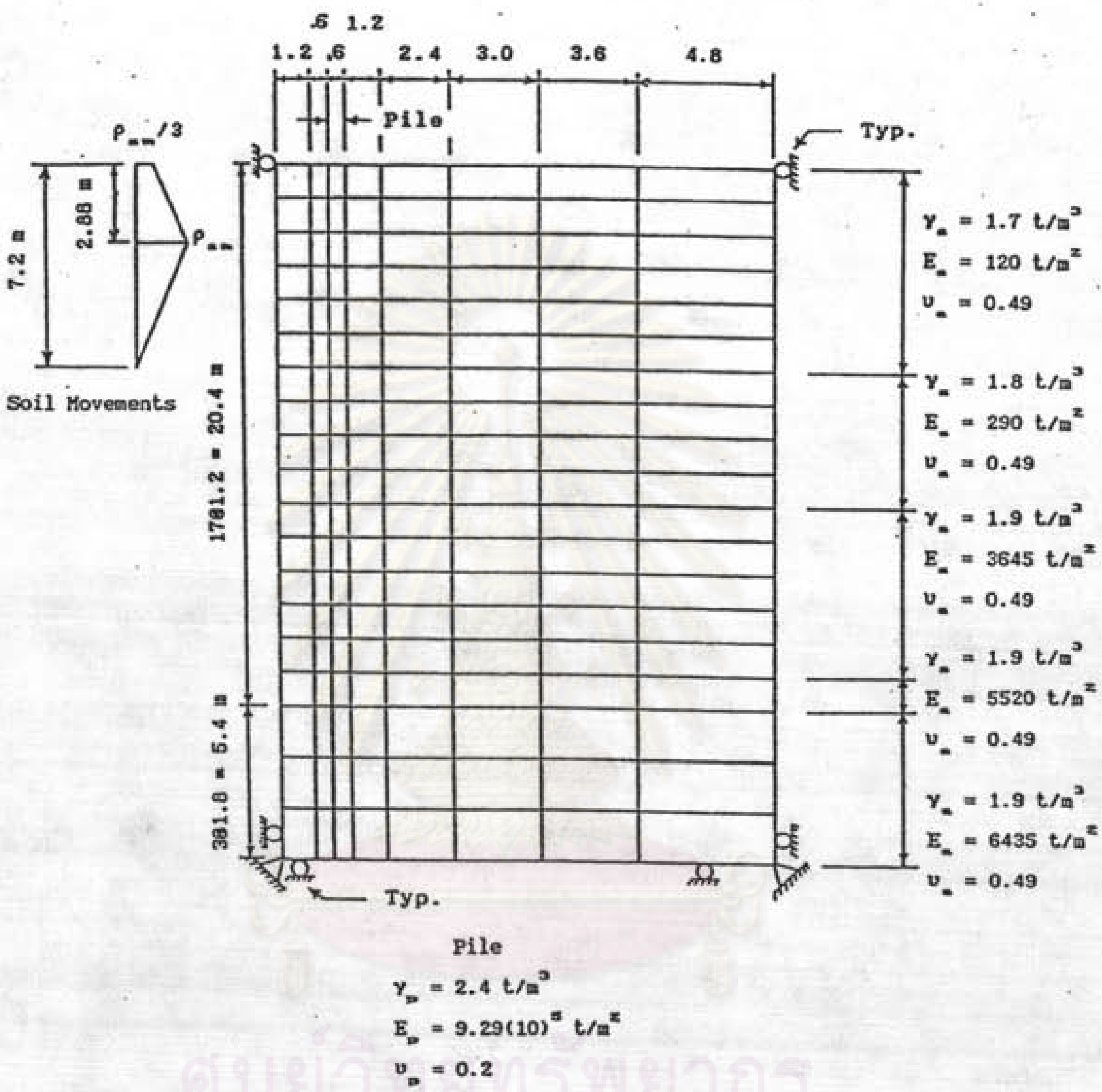

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FIGURE 4.20 Idealized Finite Element Model for Practical Problem

Calculation of the moment and curvature (M-Ψ) relationship for the pile section reveals that

At cracking: $M_{cr} = 7.57\ t/m^2$ and $\Psi_{cr} = 4.41(10)^{-4}$ rad/m;

At ultimate: $M_u = 5.75\ t/m^2$ and $\Psi_u = 6.38(10)^{-2}$ rad/m.

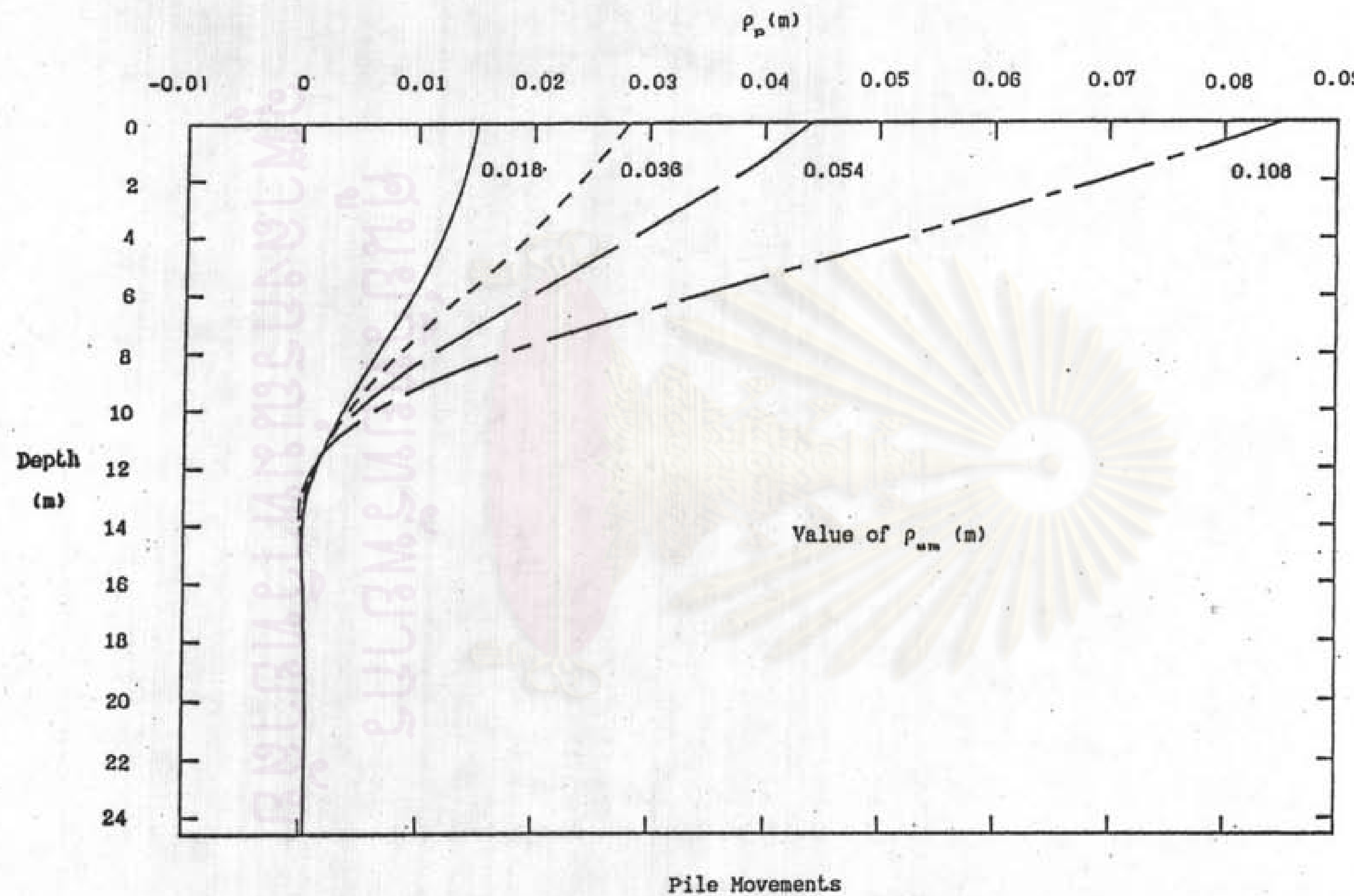

FIGURE 4.21 Pile Movements Resulting from Horizontal Soil Movements

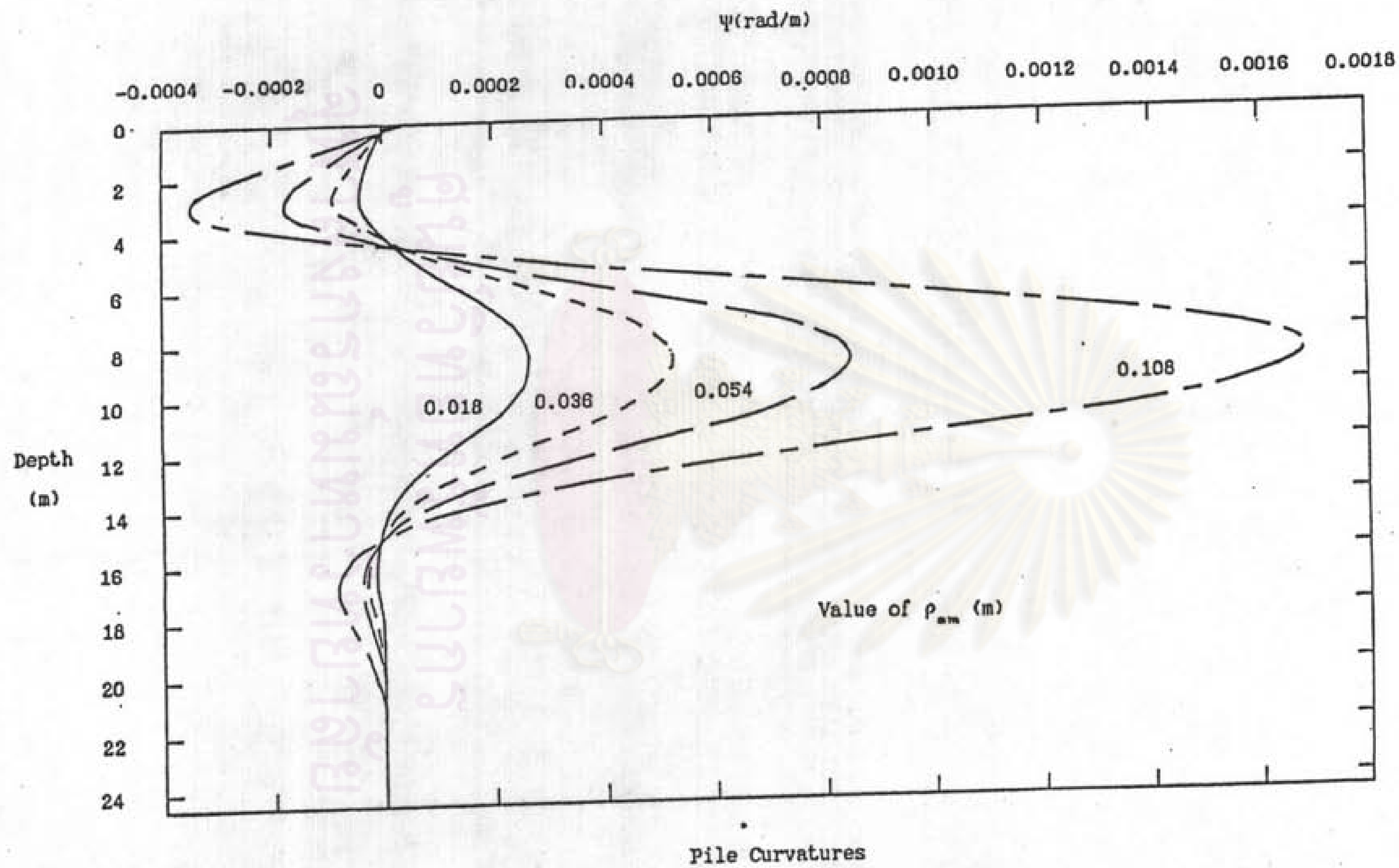

FIGURE 4.22 Pile Curvatures Resulting from Horizontal Soil Movements

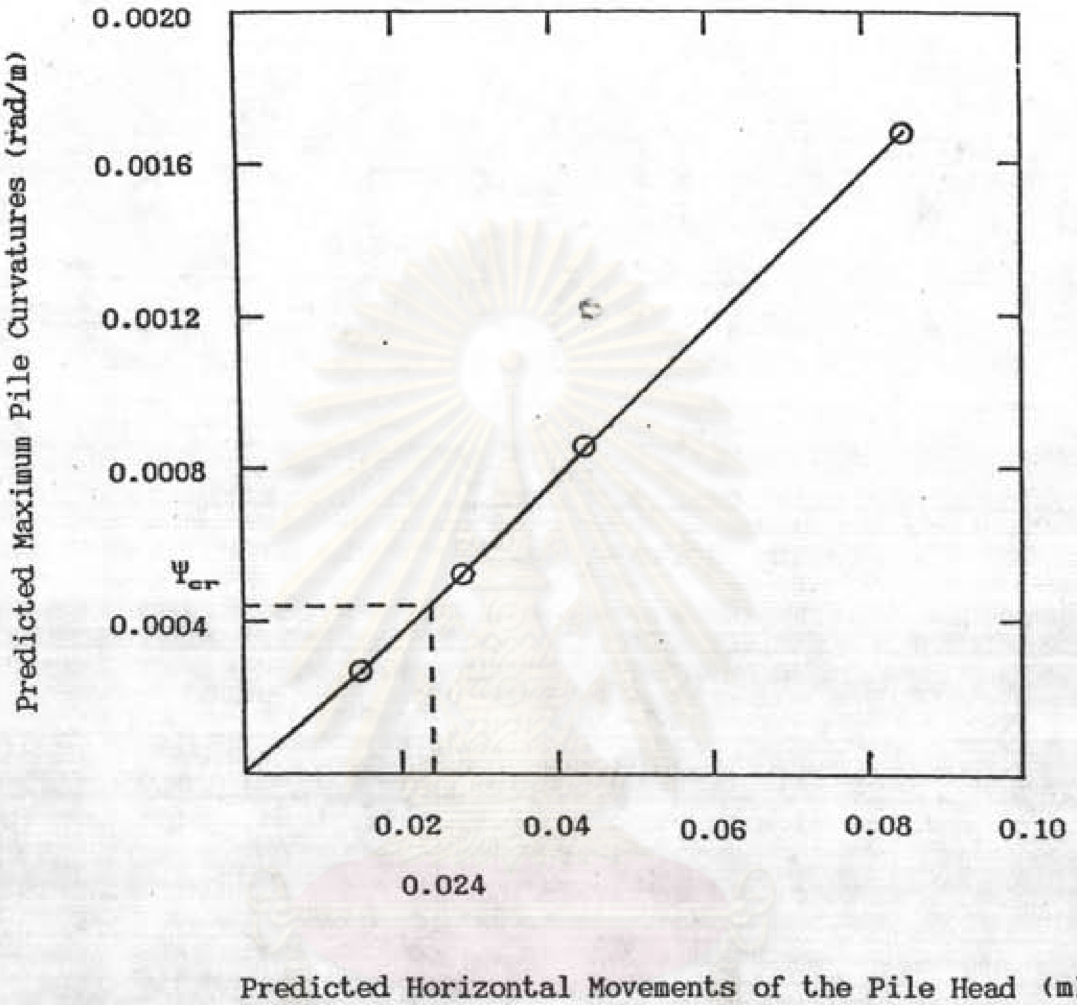

FIGURE 4.23 Predicted Maximum Pile Curvatures VS. Predicted Horizontal Movements of the Pile Head

(b) At application of vertical pressure on the pile head

In order to obtain the predicted horizontal movement of the pile head of 5.0 cm as that measured at site, the horizontal soil movement profile is applied for the value of ρ_{am} of 7.08 cm. Then the vertical pressure on the pile head representing the axial load of 100, 200 and 500 tons are applied. Solutions for pile movement and curvature distributions are shown in Figure 4.24 and 4.25, respectively.

Figure 4.26 shows the calculated interaction diagram for the pile section and the predicted maximum moments developed in the sway pile having a head movement of 5.0 cm for various axial loads. These moments are calculated from multiplying the predicted maximum curvatures by the pile stiffness E_pI_p.

It can be concluded from Figure 4.21 and 4.22 that the horizontal soil movement profile largely affects the pile movements and curvatures between the head and twice the depth of that profile. The points of maximum and large curvatures are at 9 m and 3 m below the head, respectively. Since this pile has very low reinforcement ratio of 0.0024, failure is govern by cracking moment, which is greater than the ultimate moment. Referring to Figure 4.23, the predicted horizontal movement of the pile head corresponding to the calculated curvature at cracking is 2.4 cm. This very low value can be predicted to be the limiting horizontal movement of the pile head for this pile. It can be predicted that failure of this pile has occurred since it was observed to have a horizontal movement of the head of 5.0 cm.

It can also be concluded from Figures 4.24, 4.25 and 4.26 that the axial load hardly affects the pile movement and curvature. At application of soil movements, Figure 4.25 indicates that the pile has cracking at depth of between 6.0 and 12.5 m, which disagrees with shock/seismic test. However, the maximum curvature occuring at depth

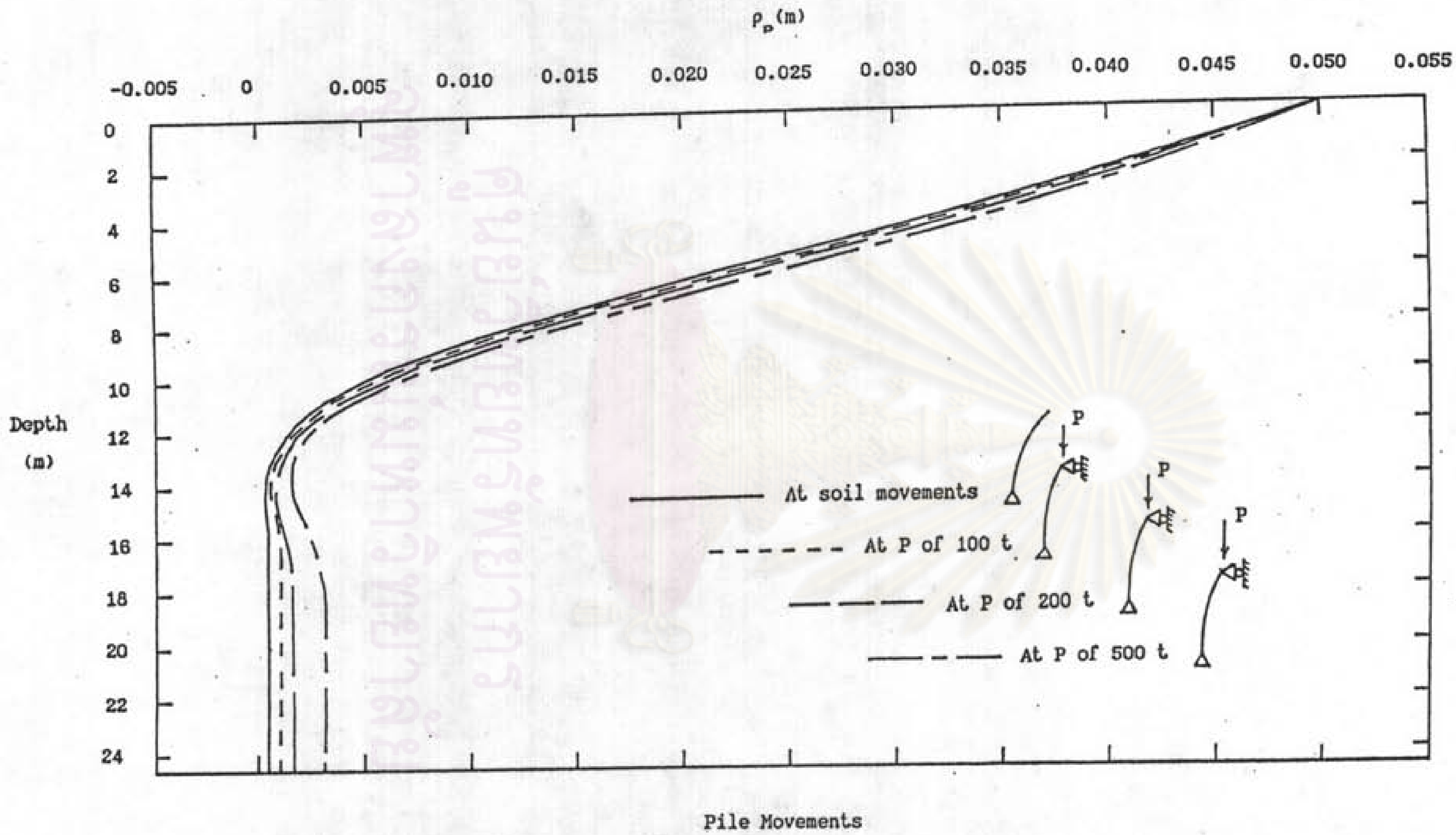

FIGURE 4.24 Pile Movements Resulting from Axial Loads

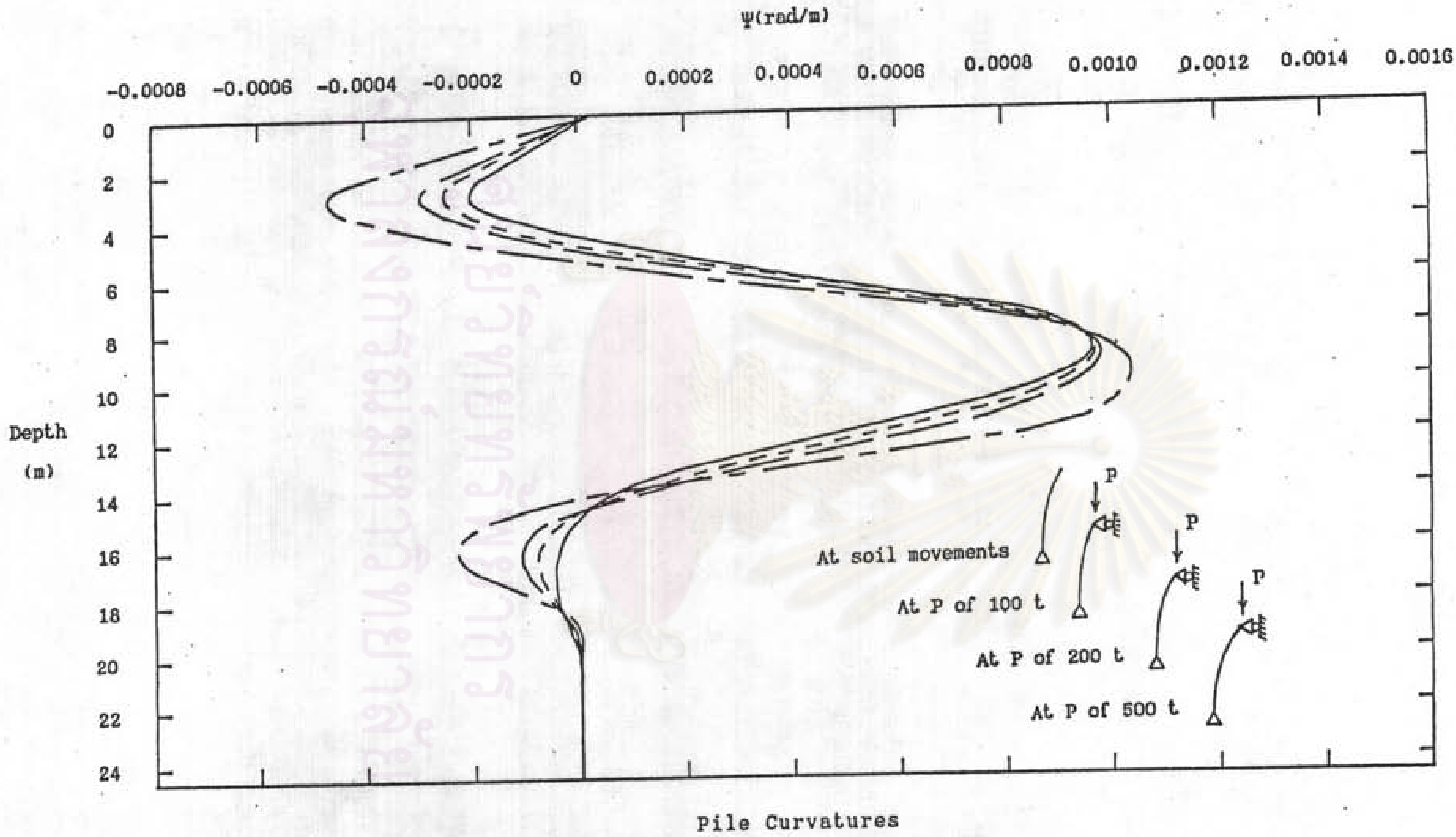

FIGURE 4.25 Pile Curvatures Resulting from Axial Loads

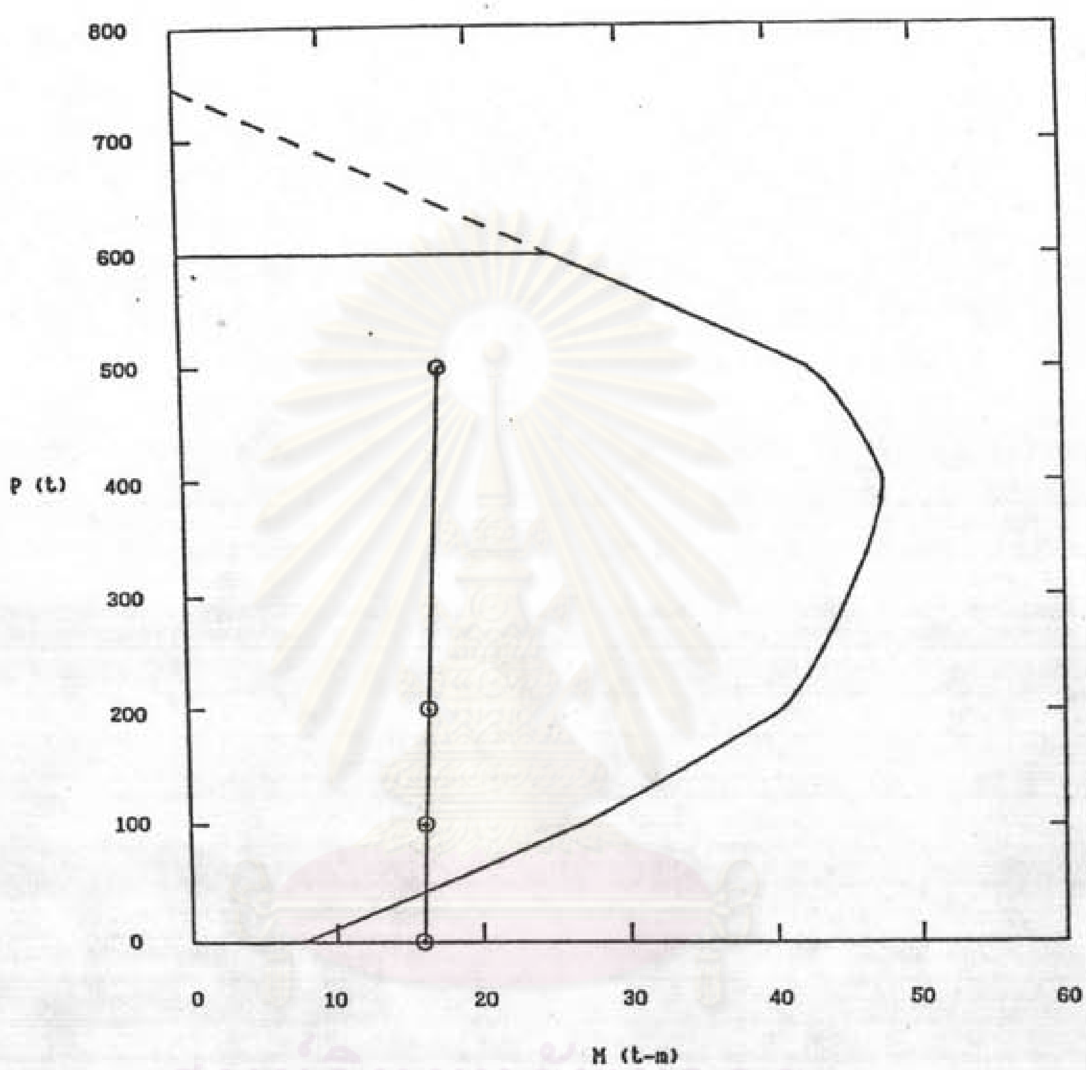

FIGURE 4.26 Calculated Interaction Diagram and Predicted Moment-Axial Compression for the Pile Section

about 9.0 m agrees with that test. Figure 4.26 indicates that capacity of the pile is not critical under axial loads, but under horizontal soil movements.

It is obvious that there is no reduction of axial load capacity from that predicted for a straight pile under concentric loading if failure of this pile will not occur prior to application of axial loads. For a bored pile, which normally has very low reinforcement, under horizontal soil movements, adequate reinforcement from the pile head to twice the depth of soft soil layer is suggested herein.

It is interesting to note that the limiting horizontal movement of the pile head is largely dependent on the pile material. Therefore, it is advantageous to use a more flexible pile for a site where horizontal soil movements are likely to occur.

Comparisons with Field Measurements and Poulos Method

It is somewhat difficult to carry out a comparison between observed and predicted pile behaviour in a soil under horizontal soil movements because only a few published measurements with sufficient data are available (16). However, to verify the accuracy of the proposed method, measured pile movements conducted in 1969 by Leussink and Wenz which is described in Poulos's paper (16) and those predicted by Poulos method (16) are considered herein.

A test pile was built up of four channel sections to form a box 0.85 m wide, 30 m long and installed in a soil profile of sand and organic clay and peat underlying sand at 20 m depth. The pile head was hinged to an almost rigid support. A rectangular test embankment of ore was constructed to a maximum height of about 6 m and measurements taken of the horizontal soil movements at various locations beneath the embankment and of the test pile situated adjacent to the embankment. It was found that the soil movements were sufficiently large (maximum of about 80 cm) to cause failure of the test pile. Soil movements taken just prior to failure enabled a comparison to be made between the predicted and measured displacement profile of the pile.

In the analysis, the following assumptions made by Poulos are also made herein:

1. The pile is pinned at the head and restrained from movement, while the tip is effectively pinned and restrained from movement at a depth of about 5 m in the sand beneath the organic clay.

2. The constant E_s with depth of 350 t/m^2 is used.

3. The E_p of $1.62(10)^6$ t/m^2 is used. This value is obtained from dividing the actual E_pI_p by the idealized I_p, in which the actual E_p and actual I_p and the idealized I_p are $2.1(10)^7$ t/m^2, $3.94(10)^{-3}$ m^4 and $5.12(10)^{-2}$ m^4, respectively.

An idealized finite element model of the pile and soil and their parameters and the horizontal soil movement profile are shown in Figure 4.27.

The resulting comparison is shown in Figure 4.28, which reveals a remarkable degree of agreement between measured pile movement profile and that predicted by the proposed method. The

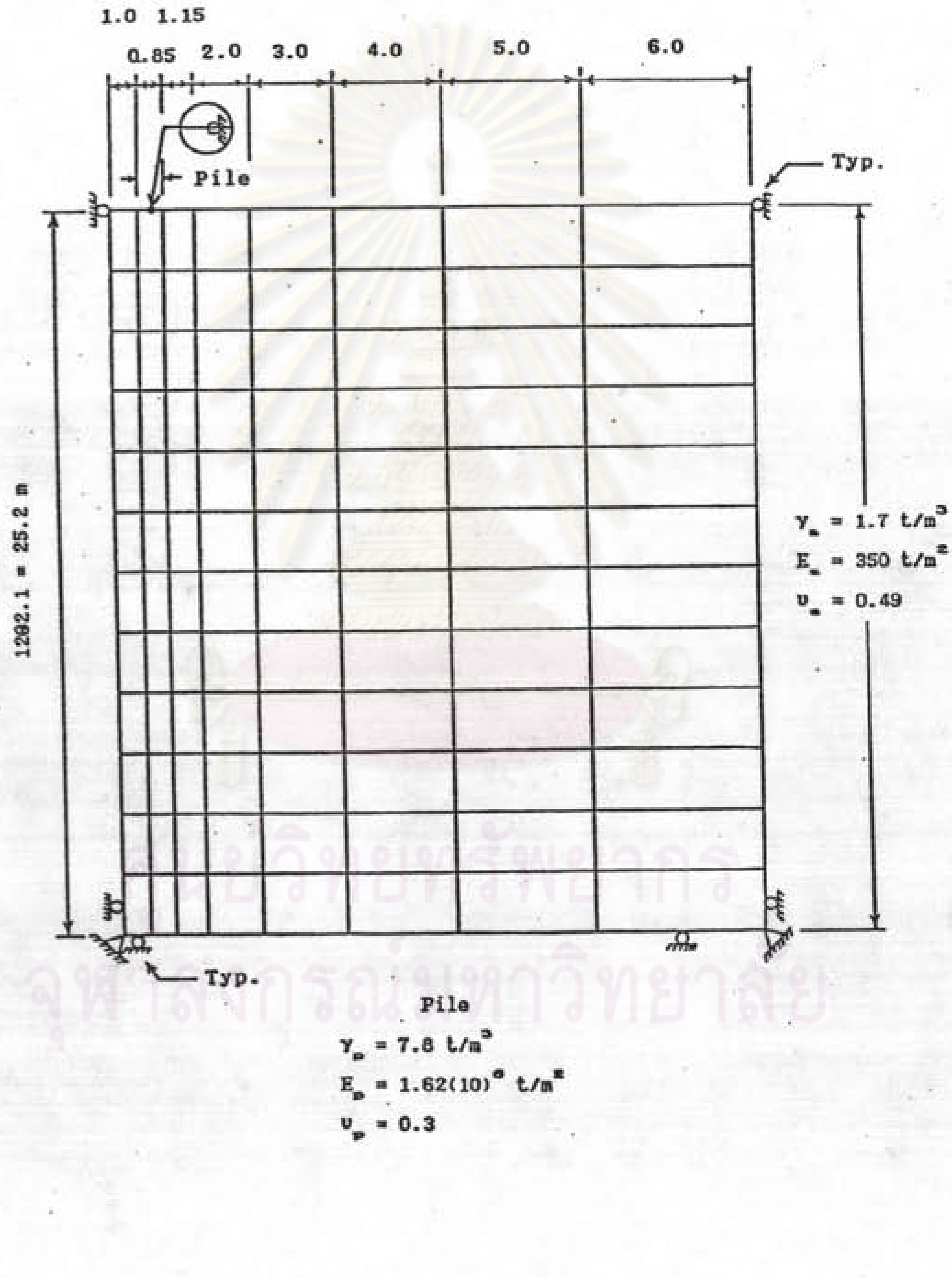

FIGURE 4.27 Idealized Finite Element Model for Field Measurements

proposed method gives greater pile movements near the head, but smaller near the tip. Some of this discrepancy may be attributed to the choice of parameters and models used in the analysis. The profiles of pile movements predicted by the proposed method and by Poulos method are similar, but those predicted by the proposed method are generally greater than those predicted by Poulos method.

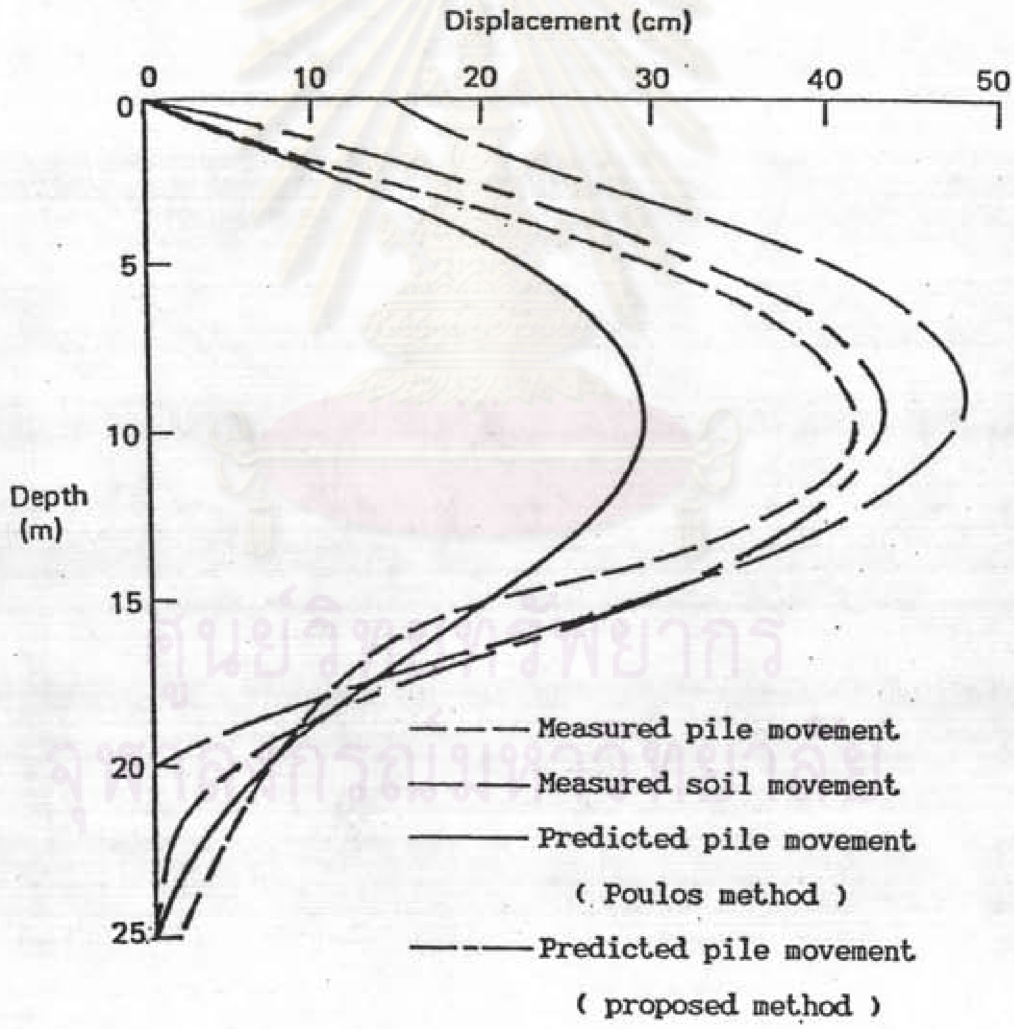

FIGURE 4.28 Comparison Between Measured and Predicted Pile Movements

It is significant to note that in the analysis the application of the horizontal soil movement profile is made at 0.5 m from the face of the pile, while in the analysis made by Poulos that application is made at the face of the pile.

Since the solutions of the analysis by the proposed method show close agreement to published data from field measurements, it can be concluded that the proposed method is a reasonably good technique for predicting the capacity of axially loaded sway piles.

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