



## รายงานวิจัยฉบับสมบูรณ์

โครงการ การศึกษาพฤติกรรมของท่อที่ฝังลึกในดินทรายเมื่อถูกผลัดในทิศทาง  
ด้านบนและด้านล่างโดยวิธีคำนวณทางตัวเลข

โดย สยาม ยิ้มศิริ

กรกฎาคม 2548

สัญญาเลขที่ TRG4580098

รายงานวิจัยฉบับสมบูรณ์

โครงการ การศึกษาพฤติกรรมของท่อที่ฝังลึกในดินทรายเมื่อถูกผลึกในทิศทางด้านบนและด้านล่าง  
โดยวิธีคำนวณทางตัวเลข

ผู้วิจัย

ดร. สยาม ยิ้มศิริ

ภาควิชาวิศวกรรมโยธา คณะวิศวกรรมศาสตร์ มหาวิทยาลัยบูรพา

สนับสนุนโดยสำนักงานกองทุนสนับสนุนการวิจัย

(ความเห็นในรายงานนี้เป็นของผู้วิจัย สกว. ไม่จำเป็นต้องเห็นด้วยเสมอไป)

## กิตติกรรมประกาศ

หัวหน้าโครงการวิจัยผู้รับทุนขอกราบขอบพระคุณ Dr. Kenichi Soga ที่ได้ให้ความกรุณาทำหน้าที่เป็นนักวิจัยที่ปรึกษาของโครงการวิจัยนี้ โครงการวิจัยนี้สำเร็จลุล่วงด้วยคำแนะนำและความเอาใจใส่ของท่าน อีกทั้งหัวหน้าโครงการก็ยังได้เรียนรู้สิ่งต่างๆมากมายจากท่านตลอดระยะเวลาของโครงการนี้

หัวหน้าโครงการวิจัยผู้รับทุนขอแสดงความขอบพระคุณสำหรับการสนับสนุนทั้งทางด้านเทคนิคและไม่เทคนิคของบุคคลดังต่อไปนี้เป็นพิเศษ ได้แก่ ผศ.ดร. บรรหาญ ลิลา, ผศ.ดร. สัตยชัย มิตรเอม, ผศ.ดร. จิรวัด บุญญะฐิ, และ ผศ.ดร. ปิยฉัตร ยัมศิริ

ขอขอบพระคุณท่านคณบดี ท่านหัวหน้าภาควิชาวิศวกรรมโยธา และ คณาจารย์ ของคณะวิศวกรรมศาสตร์ มหาวิทยาลัยบูรพา ที่ได้ให้การสนับสนุนในด้านต่างๆตลอดระยะเวลาของโครงการนี้

ท้ายที่สุดนี้ขอขอบพระคุณสำนักงานกองทุนสนับสนุนการวิจัยที่ได้กรุณาให้ทุนวิจัยสำหรับโครงการวิจัยนี้

## บทคัดย่อ

รหัสโครงการ: TRG4580098

ชื่อโครงการ: การศึกษาพฤติกรรมของท่อที่ฝังลึกในดินทรายเมื่อถูกผลัดในทิศทางด้านบนและด้านข้าง  
โดยวิธีคำนวณทางตัวเลข

ชื่อนักวิจัย: ดร. สยาม ยัมศิริ

Email Address: ysiam@buu.ac.th

ระยะเวลาโครงการ: สิงหาคม 2545 - มิถุนายน 2548

โครงการวิจัยนี้เป็นการศึกษาถึงผลกระทบซึ่งกันและกันระหว่างท่อและดินทราย เมื่อท่อถูกแรงกระทำให้เกิดการเคลื่อนตัวในแนวนอนและเคลื่อนตัวขึ้นในแนวดิ่ง โดยการวิเคราะห์นี้อาศัยวิธีการคำนวณทางตัวเลขที่เรียกว่า Distinct Element Method (DEM) การวิเคราะห์ได้กระทำในกรณีของดินทรายแน่นปานกลางและแน่นที่สัดส่วนระยะการฝังลึกของท่อจนถึงค่า 60 ผลการวิเคราะห์จาก DEM ได้นำไปเปรียบเทียบกับผลการวิเคราะห์จาก Finite Element Method (FEM) ที่ได้กระทำมาก่อนหน้านี้แล้ว โดยจากการเปรียบเทียบพบว่าสำหรับทรายแน่นปานกลางนั้นผลการวิเคราะห์จาก DEM และ FEM ให้ค่าแรงสูงสุดและลักษณะการเปลี่ยนแปลงการวิบัติจากแบบดินเป็นแบบลึกที่เหมือนกัน สำหรับทรายแน่นนั้นผลจาก DEM จะให้ลักษณะการเปลี่ยนแปลงการวิบัติจากแบบดินเป็นแบบลึกที่ค่อยเป็นค่อยไปกว่าผลที่ได้จาก FEM และผลจาก DEM ยังให้ค่าแรงสูงสุดที่ความลึกมากกว่าค่าจาก FEM การที่ผลการวิเคราะห์จาก DEM และ FEM มีความแตกต่างกันน่าจะมาจากลักษณะการเคลื่อนตัวของดินรอบๆท่อที่ไม่เหมือนกัน โดย DEM เป็นการวิเคราะห์ที่พิจารณาดินเป็นอนุภาคซึ่งตรงกันข้ามกับ FEM ที่พิจารณาดินเป็นวัสดุเนื้อเดียวต่อเนื่อง ผลการวิเคราะห์จาก FEM ที่ได้นี้จะเป็นการให้ข้อมูลเพิ่มเติมของพฤติกรรมของท่อที่ฝังลึกในดินทราย

คำหลัก: ผลกระทบซึ่งกันและกันระหว่างท่อและดิน, distinct element analysis, การวิบัติแบบดิน, การวิบัติแบบลึก, ทราย

## **Abstract**

Project Code: TRG4580098

Project Title: Numerical study of lateral and upward soil-pipeline interactions in sand at deep embedment conditions

Investigator: Dr. Siam Yimsiri

E-mail Address: ysiam@buu.ac.th

Project Period: August 2002 – June 2005

The soil-pipeline interactions under lateral and upward pipe movements in sand are investigated using Distinct Element Method (DEM) analysis. The simulations are performed for both medium and dense sand conditions at different embedment ratios of up to 60. The comparison of peak dimensionless forces from the DEM and earlier Finite Element Method (FEM) analyses shows that, for medium sand, both methods show similar peak dimensionless forces. For dense sand, the DEM analysis gives more gradual transition of shallow to deep failure mechanisms than the FEM analysis and the peak dimensionless forces at very deep depth are higher in the DEM analysis than in the FEM analysis. The comparison of the deformation mechanism suggests that this is due to the differences in soil movements around the pipe associated with its particulate nature. The DEM analysis provides supplementary data of the soil-pipeline interaction in sand at deep embedment condition.

Keywords: soil-pipeline interaction, distinct element analysis, shallow failure mechanism, deep failure mechanism, sand

## **Executive Summary**

An understanding of pipeline response to vertical and lateral ground movements is essential in pipeline design. Rational design under these conditions requires knowledge of soil forces resulting from the relative soil-pipeline displacement. The design formulations of the load-displacement characteristics for soil-pipeline interactions are given by ASCE's "Guideline for the Seismic Design of Oil and Gas Pipeline System (1984)". This Guideline recommends that the peak dimensionless forces and the displacement at peak forces are functions of the embedment depth of the pipe and soil friction angle, and that the non-linear relationship between the force and displacement is modeled by a rectangular hyperbola. These design values are derived from the database of pipe testing at relatively shallow embedment depths. However, there are situations where the pipe embedment ratios can be deeper than the values given in the guideline. Typical scenarios are the construction of a large embankment over an area with existing pipelines and installation of deep pipelines due to already congested underground space. If the design is based on a linear extrapolation from the ASCE data, the Guideline would overestimate the peak load exerted on the pipe and the stiffness of the soil-pipeline interaction. This overestimation would in turn results in uneconomical design.

The deep embedded pipeline problem has been investigated by Yimsiri et al. (2004). They examined various analytical solutions available for the peak forces onto a pipe and strip anchor and showed that there are large differences in the computed peak dimensionless forces for deep embedment conditions. It was concluded that a linear extrapolation from the ASCE data of shallow depth should not be employed because of the change in failure mechanism from shallow (i.e. ground surface dependent) to deep embedment conditions (i.e. ground surface independent). They performed finite element analysis to examine the transition in both medium and dense sands. Unfortunately, it is not possible to validate their findings because full-scale pipe loading experimental results or any other supplementary data are not available.

This study has been undertaken to provide supplementary data for deep embedded pipes using the Distinct Element Method (DEM). First, DEM analysis was conducted for shallow embedment depth conditions to calibrate the input parameters by comparing the results to the experimental data reported by Trautmann & O'Rourke (1983). The analysis was then

extended to deeper embedment conditions. Due to its discontinuous nature, it is considered that DEM should better simulate the soil movement close to the pipe at large pipe displacements and, hence, may yield more realistic results to the problem compared to the continuum-based finite element analysis.

For medium sand, the comparison of the peak dimensionless forces from the DEM and FEM analysis shows good match between the two. For dense sand, the DEM data show more gradual transition from shallow to deep failure mechanism and have a tendency to give a somewhat larger peak forces at very deep depths. The advantage of DEM over FEM is its ability to simulate large movement of soil around the pipe. Also, the DEM analysis can continue with unlimited pipe movement until it reaches ultimate peak force (or further), whereas the continuum based FEM often stops at some pipe displacements before the peak force can be reached because large distortion of the mesh causes numerical convergence problem. The investigation of the soil displacement patterns computed by DEM shows larger soil mass movement compared to FEM data, especially for dense sand.

The DEM results from this study confirm the findings by Yimsiri et al (2004) that the calculation of the soil-pipeline interaction at deep embedment conditions should not base on a linear extrapolation from the ASCE data of shallower embedment depths. However, the DEM results suggest that (i) there may be a possibility of underestimation of the peak dimensionless force at very deep embedment depths if the design chart proposed by Yimsiri et al. (2004) is employed and (ii) the transition from shallow to deep failure mechanism may proceed more gradually than suggested by Yimsiri et al. (2004). The results from this DEM analysis together with earlier FEM analysis will serve as a Class-A prediction of future full-scale tank tests of this problem.

# Table of Contents

	<i>Page</i>
ปกใน .....	i
กิตติกรรมประกาศ .....	ii
บทคัดย่อ .....	iii
Abstract .....	iv
Executive summary .....	v
Table of contents .....	vii
 <b>1. Introduction</b>	
1.1 General .....	1
1.2 Purposes .....	2
1.3 Layout of the report .....	3
 <b>2. Review of ASCE Guideline for pipeline design</b>	
2.1 General .....	5
2.2 Pipeline subjected to lateral movement .....	5
2.3 Pipeline subjected to upward movement .....	8
2.4 Discussion .....	9
 <b>3. Review of experimental study</b>	
3.1 General .....	10
3.2 Testing apparatus .....	10
3.3 Testing program .....	11
3.4 Properties of sand .....	13
3.5 Test results .....	14
 <b>4. Distinct element analysis</b>	
4.1 General .....	15
4.2 Distinct element method numerical analysis .....	15
4.3 Distinct element code PFC <sup>3D</sup> .....	16
4.3.1 General assumptions .....	16
4.3.2 Contact models .....	17
4.3.2.1 Linear elastic contact model .....	18
4.3.2.2 Criteria for particle sliding .....	19
4.3.3 Micromechanics parameters .....	19
4.4 DEM analysis by PFC <sup>3D</sup> .....	20
4.4.1 Numerical modeling .....	20
4.4.1.1 Tank and pipe .....	20
4.4.1.2 Sand particles .....	21
4.4.2 Numerical analysis procedures .....	24
4.4.2.1 Tank set-up .....	24
4.4.2.2 Initial condition .....	24
4.4.2.3 Pipe installation .....	24
4.4.2.4 Pipe pulling .....	25
4.5 Determination of input parameters .....	27
4.5.1 Triaxial tests .....	27



4.5.2 Pipe loading test .....	30
4.6 Determination of peak force from DEM analysis results .....	32
<b>5. DEM analyses of pipe under lateral loading</b>	
5.1 Analysis procedure and program .....	34
5.2 Input parameters .....	37
5.3 DEM analysis results of shallow embedment depths .....	38
5.4 DEM analysis results of deep embedment depths .....	44
5.5 Summary .....	50
<b>6. DEM analyses of pipe under upward loading</b>	
6.1 Analysis procedure and program .....	51
6.2 Input parameters .....	54
6.3 DEM analysis results of shallow embedment depths .....	55
6.4 DEM analysis results of deep embedment depths .....	61
6.5 Summary .....	66
<b>7. Discussion and conclusions</b>	
7.1 Selection of input parameters .....	67
7.2 Pipeline behavior under lateral loading .....	67
7.3 Pipeline behavior under upward loading .....	78
7.4 Conclusions .....	89
<b>References</b> .....	90
<b>Appendix A</b> Force-displacement curves of pipe under lateral loading .....	93
<b>Appendix B</b> Force-displacement curves of pipe under upward loading .....	106
<b>Appendix C</b> Displacement pattern under lateral loading (velocity vector) .....	116
<b>Appendix D</b> Displacement pattern under upward loading (velocity vector) .....	141
<b>Appendix E</b> Contact force pattern under lateral loading .....	159
<b>Appendix F</b> Contact force pattern under upward loading .....	176
<b>Appendix G</b> Parametric study of DEM simulation of triaxial test .....	191
<b>Appendix H</b> Parametric study of DEM simulation of lateral pipe loading .....	199
<b>Appendix I</b> Parametric study of DEM simulation of upward pipe loading .....	214
<b>Appendix J</b> Resulting publications from current research project .....	226

## Chapter 1

### Introduction

#### 1.1 General

An understanding of pipeline response to vertical and lateral ground movements is essential in pipeline design. Rational design under these conditions requires knowledge of the soil forces resulting from relative soil-pipeline displacement. These movements may arise from offshore slope failures, earthquake-induced faulting, landslide and liquefaction, urban excavation and tunneling, and excessive ground settlement. Under such circumstances, loads are induced in a pipeline by relative motion between the pipeline and surrounding soil. This occurs when the soil restricts the free motion of the pipeline or when the pipeline attempts to resist the motion of the surrounding soil. The amount of restraint or load exerted on the pipeline is a non-linear function of the relative soil-pipeline displacement. Once this relationship is established, the soil-pipeline system can be analyzed approximately by a numerical method, in which the pipe is represented by beam elements and the soil reactions are modeled by spring-slider elements in the vertical, lateral, and axial directions.

The formulations of the load-displacement characteristics for soil-pipeline interactions are given by ASCE in its “Guideline for the Seismic Design of Oil and Gas Pipeline System (1984)”. This Guideline recommends that the peak dimensionless forces and the displacement at peak forces are functions of the embedment depth of the pipe, and that the non-linear relationship between the forces and displacements is modelled by a rectangular hyperbola. These recommendations are derived from the database of research works for the pipe embedment depths of  $H_c/D \leq \sim 10$  to 20 for lateral and upward pipe movement, where  $H_c$  is the depth to the centerline of the pipe and  $D$  is the external diameter of the pipe.

There are some situations where the pipe embedment ratios can be deeper than the values recommended by ASCE. This is due to, for example, the construction of high embankment over the area with existing pipeline and the congestion of the underground structures forcing the new pipeline to go deeper (installed by pipe-jacking). Under this condition, the  $H_c/D$  ratio can be as large as 40 to 60, which is well beyond the range of the recommendations given in the ASCE Guideline. If the calculation is based on a linear extrapolation from the ASCE data of shallower embedment depths, it is expected that the

Guideline would overestimate the peak load exerted on the pipe and the stiffness of the soil-pipeline interaction. This overestimation would in turn results in uneconomical design.

This potential problem has been considered by Yimsiri et al. (2004), who performed the numerical analysis by Finite Element Method (FEM) to study the soil-pipeline behavior in sand at deep embedment condition. However, there has not been any full-scale experimental result to exactly confirm this FEM result. Therefore, this research project was undertaken to supplement additional data on the soil-pipeline interaction in sand at deep embedment depths by another approach of numerical analysis.

## 1.2 Purposes

As mentioned earlier, the ASCE Guideline is based on the research data for  $H_e/D \leq \sim 10$  to 20. Most of the published experimental studies of pipeline behavior are performed at depths smaller than the depths of interest in this study ( $H_e/D$  up to 60). To the author's knowledge, the largest  $H_e/D$  studied is 25 for lateral pipe loading (Audibert & Nyman, 1977) and 13 for upward pipe loading (Trautmann & O'Rourke, 1983). Furthermore, the critical embedment depths (the depth after which the peak dimensionless force becomes constant with embedment ratios) have been reported only for loose sand. Only Dickin & Leung (1983) reported the critical embedment depth for dense sand from their centrifuge tests. Thus, it is necessary to perform a study of pipeline behavior at deep embedment depths to gain more understanding.

In this study, the numerical analysis by Distinct Element Method (DEM) was undertaken to study the behavior of pipeline under lateral and upward loading. Firstly, the experimental results of the pipeline behavior by Trautmann & O'Rourke (1983) at shallow depth were simulated to (i) establish a simulation technique, (ii) obtain input parameters for numerical models, and (iii) examine the accuracy of the analysis. Secondly, the DEM analyses were extended to deeper embedment depths up to  $H_e/D = 60$ . Thirdly, the DEM analysis results were compared with the ASCE Guideline and the previous FEM analysis by Yimsiri et al. (2004). Finally, the discussions and conclusions were made.

The specific goals of this study are:-

1. To present the capability of the DEM analysis to simulate the pipeline-soil behavior,

2. To determine the peak forces exerted on a buried pipeline at deep embedment depths for lateral and upward pipe movements,
3. To determine the critical embedment depth, where the deep failure mechanism happens,
4. To assess the consistency of the DEM analysis results with the ASCE Guideline and the previous FEM analysis results, and
5. To investigate deformation pattern of soil particles around pipe, which may be represented more realistically by DEM analysis.

### **1.3 Layout of the report**

Following the introductory chapter, Chapter 2 briefly describes the ASCE Guideline for the design of pipeline subjected to lateral and upward movements. The discussions of the potential problems for an application of this guideline to a pipe at deep embedment depths are made.

Chapter 3 reviews the experimental study of pipeline subjected to lateral and upward loading at shallow embedment depths performed by Trautmann & O'Rourke (1983). These experimental results are used as a benchmark to determine the input parameters and to examine the accuracy of the present DEM simulation.

Chapter 4 explains the details of the DEM analysis employed in this study. The general concepts of the DEM analysis are explained. The numerical analysis procedures are described. The selection of the parameters is elucidated. The interpretation of the numerical analysis results is mentioned.

Chapter 5 presents the DEM analysis results of a pipe subjected to lateral loading. First, the DEM analysis results are presented for shallow embedment depths and are compared with the experimental results by Trautmann & O'Rourke (1983). The DEM analyses are then performed for deeper embedment depths and the results are compared with the previous published data.

Chapter 6 presents the DEM analysis results of a pipe subjected to upward loading. The structure of this chapter is similar to Chapter 5.

Chapter 7 presents the discussions and conclusions. The determination of the input parameters for the present DEM analysis is discussed. The pipeline-soil behavior under

lateral and upward loading is discussed concerning the obtained peak dimensionless forces and the soil displacement pattern with comparison to the previous FE analysis.

Appendix A presents the force-displacement curves of pipe under lateral loading with comparison with the previous experimental results and FE analysis data.

Appendix B presents the force-displacement curves of pipe under upward loading with comparison with the previous experimental results and FE analysis data.

Appendix C presents the displacement patterns of soil around pipe under lateral loading.

Appendix D presents the displacement patterns of soil around pipe under upward loading.

Appendix E presents the contact force patterns around pipe under lateral loading.

Appendix F presents the contact force patterns around pipe under upward loading.

Appendix G presents the parametric study of the effects of various parameters of the DEM analysis on the triaxial test results.

Appendix H presents the parametric study of the DEM analysis for pipe subjected to lateral loading.

Appendix I presents the parametric study of the DEM analysis for pipe subjected to upward loading.

## Chapter 2

### Review of ASCE Guideline for pipeline design

#### 2.1 General

In 1984, the Committee on the Gas and Liquid Fuel Lifelines of the ASCE Technical Council on Lifeline Earthquake Engineering published the “Guideline for the Seismic Design of Oil and Gas Pipeline Systems”. The Guideline proposed the procedures to determine the peak forces and their corresponding pipe displacements for pipeline-soil interaction at various directions and also provided the equations to describe the non-linear force-displacement relationship. These data are essential to define the spring constant in the numerical analyses of a soil-pipeline interaction problem. The design guideline for a pipeline in sand subjected to lateral and upward movements are summarized in the following sections.

#### 2.2 Pipeline in sand subjected to lateral movement

Lateral restraint of a pipeline represents the resistance (or load) offered by the surrounding soils to any lateral movement of the pipeline. Therefore, the lateral soil-pipeline interaction is similar to a vertical anchor plate moving horizontally relative to the surrounding soils. Based on the experiments performed with pipes buried in dry, uniform sand, a relationship between the force per unit length,  $F_h$ , and the lateral displacement,  $\delta_h$ , has been shown to be represented by the following hyperbolic equation (Das & Seeley, 1975; Audibert & Nymann, 1977; Trautmann & O’Rourke, 1983).

$$F_h = \frac{\delta_h}{A' + B' \delta_h} \quad \dots\dots\dots (2-1)$$

where

$$A' = 0.15 \delta_{h,peak} / F_{h,peak}$$

$$B' = 0.85 / F_{h,peak}$$

$$F_{h,peak} = \bar{\gamma} H_c N_{qh} D$$

$$\delta_{h,peak} = \begin{cases} 0.07 \text{ to } 0.10 (H_c + D/2) & \text{for loose sand} \\ 0.03 \text{ to } 0.05 (H_c + D/2) & \text{for medium sand} \\ 0.02 \text{ to } 0.03 (H_c + D/2) & \text{for dense sand} \end{cases}$$

$\bar{\gamma}$  = effective unit weight of soil

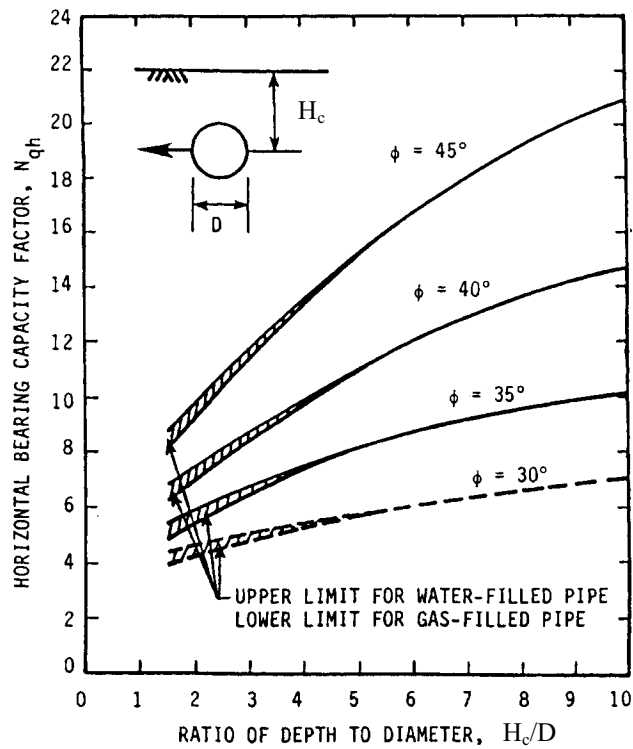
$H_c$  = depth to centerline of the pipeline

$N_{qh}$  = horizontal bearing capacity factor

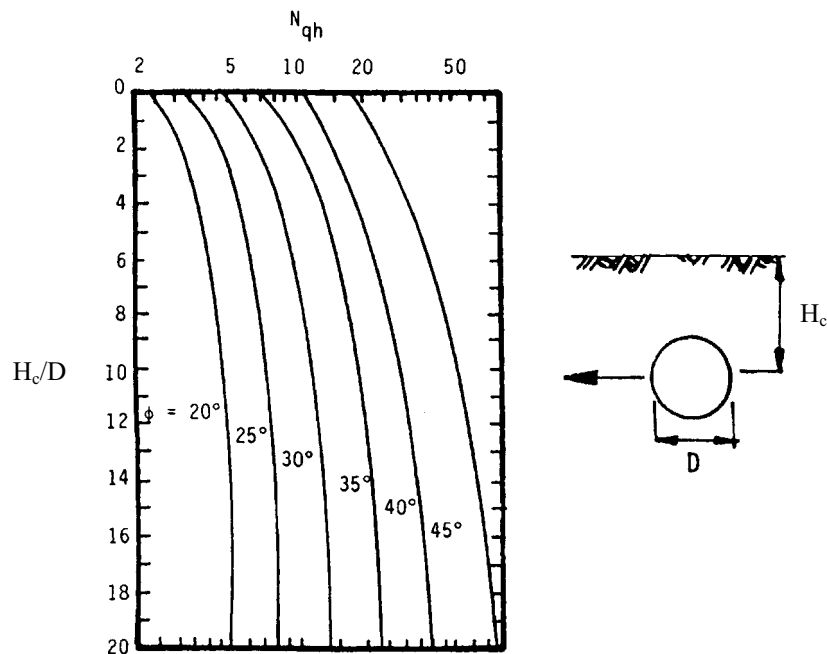
$D$  = external pipeline diameter

In the Guideline, two analytical models are proposed for determining  $N_{qh}$ . Fig. 2-1 shows  $N_{qh}$  as a function of  $H_c/D$  for soil friction angles between 30° and 45° proposed by Trautmann & O'Rourke (1983) based on the model by Oversen (1964). Fig. 2-2 shows  $N_{qh}$  as a function of  $H_c/D$  for soil friction angle between 20° and 45° based on the analytical solution of laterally loaded pile by Hansen (1961). It is noted that the values of  $N_{qh}$  recommended by Hansen are approximately 50% to 100% larger than those by Trautmann & O'Rourke for a given friction angle. This discrepancy is due to the difference in vertical restraint condition of the pipe in each model. Hansen's model assumes full vertical restraint so the interface friction mobilizes fully, whereas Oversen's model assumes that the wall is free to rise with the passive wedge of soil so the mobilized interface friction is less. Audibert & Nyman (1977) showed that the prediction from Hansen's model show good agreement with their experimental data up to  $H/D$  of 25. Trautman & O'Rourke (1983) showed that their experimental data gave good agreement with the prediction made by Oversen's model

The Guideline recommends that the values of  $N_{qh}$  be chosen after careful evaluation of the soil conditions in the field and the level of conservatism governing the design. Partially saturated, medium to fine sands will have increased shear strength relative to their dry or saturated states under conditions of short term loading. Neither Hansen's nor Oversen's models accounts for this increased strength associated with partial saturation. Thus, the Guideline suggests that Fig. 2-1 should be applicable for dry or saturated sands and gravels, and for partially saturated gravels and coarse sands and that Fig. 2-2 should be used to conservatively estimate lateral soil forces for most condition of pipeline burial in cohesionless sands and gravels.



**Figure 2-1** Horizontal bearing capacity factor for sand as a function of depth to diameter ratio of buried pipeline (after Trautmann & O'Rourke, 1983)



**Figure 2-2** Horizontal bearing capacity factors as a function of depth to diameter ratio for pipelines buried in sand (after Hansen, 1961)



### 2.3 Pipeline in sand subjected to upward movement

Upward restraint of a pipeline represents the resistance offered by the surrounding soils to any upward movement of the pipeline. Therefore, the upward soil-pipeline interaction is also similar to a horizontal anchor plate subjected to upward movement. Based on the tests performed with pipes buried in dry, uniform sand, a relationship between the force per unit length,  $F_v$ , and the vertical upward displacement,  $\delta_v$ , is represented by the following hyperbolic relationship (Trautmann & O'Rourke, 1983).

$$F_v = \frac{\delta_v}{A'' + B'' \delta_v} \quad \dots\dots\dots (2-2)$$

where

$$A'' = 0.07 \delta_{v,peak} / F_{v,peak}$$

$$B'' = 0.93 / F_{v,peak}$$

$$F_{v,peak} = \bar{\gamma} H_c N_{qv} D$$

$$\delta_{v,peak} = 0.01 \text{ to } 0.015 H_c \text{ for dense to loose sand}$$

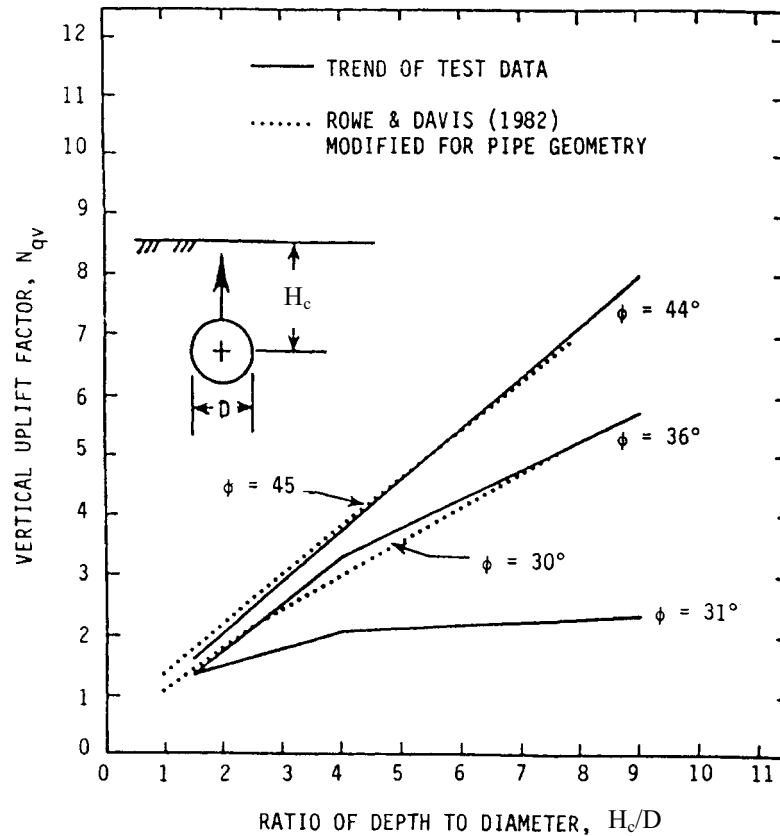
$$\bar{\gamma} = \text{effective unit weight of soil}$$

$$H_c = \text{depth to centerline of the pipeline}$$

$$N_{qv} = \text{vertical uplift factor}$$

$$D = \text{external diameter of pipe}$$

Fig. 2-3 summarizes the values of  $N_{qv}$  for vertically displaced pipeline in dry or saturated sand as a function of  $H_c/D$  and soil friction angle. The plots shown by the solid lines are based on the test results reported by Trautmann & O'Rourke (1983). The plots shown by the dotted lines were derived from the finite element analysis reported by Rowe & Davis (1982) for a horizontal anchor plate lifted under the conditions of no soil dilatancy and no interface friction. In Fig. 2-3, the Rowe & Davis' solutions have been modified to account for the cylindrical shape of the pipe by subtracting the weight of the soil occupying the upper half of the pipeline from the predicted forces. The vertical uplift factors shown in Fig. 2-3 account for the weight of a gas pipeline. For liquid pipelines, the weight of the contents should be added to the force determined from the figure to give the total resistance to uplift.



**Figure 2-3** Vertical uplift factor for sand as a function of depth to diameter ratio of buried pipelines (after Trautmann & O'Rourke, 1983)

## 2.4 Discussion

The force-displacement relationships of lateral and upward pipe movements are assumed to be a rectangular hyperbola. The peak dimensionless forces,  $N_{qh}$  and  $N_{qv}$ , and the corresponding pipe displacements at the peak forces are influenced by the burial depth. The constants  $N_{qh}$  and  $N_{qv}$  increase with  $H_c/D$ . At deeper embedment depths, it is expected that this Guideline would overestimate the peak force and soil stiffness if the calculation is based on a linear extrapolation from the data of shallower embedment depth. This is due to the fact that the failure mechanism at deeper embedment depths is different from that at shallow embedment depths as experimentally shown by several researchers (e.g. Audibert & Nyman, 1977; Akinmusuru, 1978; Dickin & Leung, 1985). At the depths beyond the critical embedment depth, the peak dimensionless force should become constant independent of the embedment ratio. The critical embedment depth has been reported only for loose sand. There is no available data for medium and dense sands.

## **Chapter 3**

### **Review of experimental study**

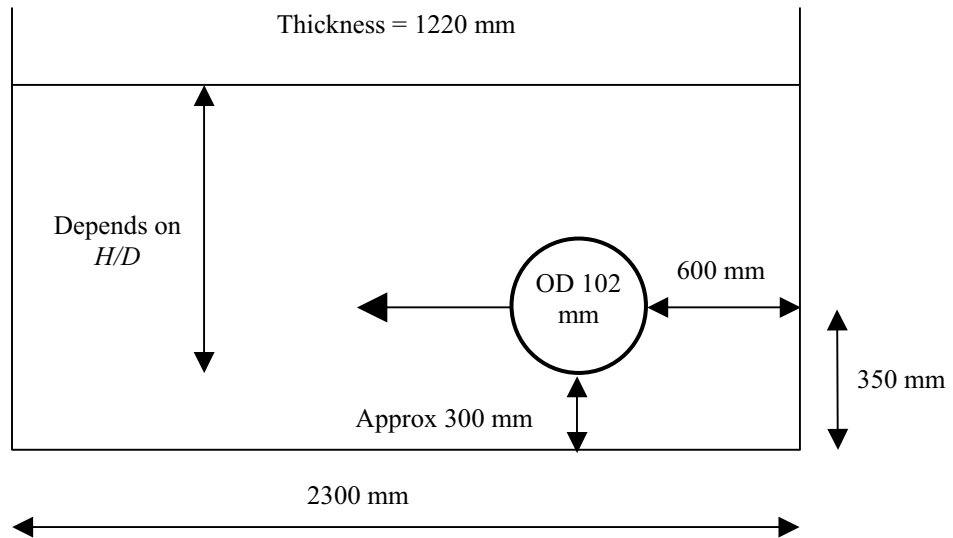
#### **3.1 General**

The experimental results of the pipeline behavior subjected to lateral and upward loadings by Trautmann & O'Rourke (1983) are used as benchmarks to determine the input parameters for the present DEM models and to examine the capability of the DEM analysis technique. Their results of  $\phi 102$  mm pipe in medium and dense sands are of concern in this research because the sand placed around a pipeline is often in the state of medium to dense conditions. In the following, their experimental study is summarized.

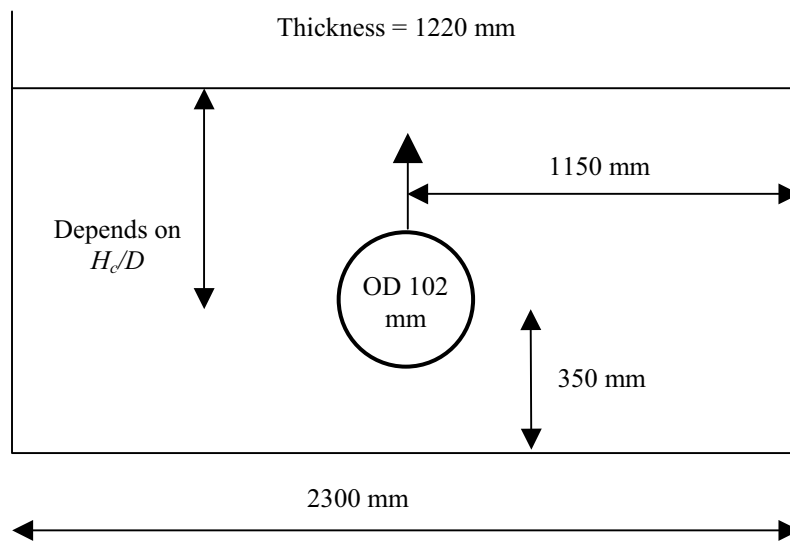
#### **3.2 Testing apparatus**

The pipe loading tests were performed in a test compartment of  $1.2 \text{ m } (W) \times 2.3 \text{ m } (L) \times 1.2 \text{ m } (D)$ . The steel pipe had outside diameters of 102 mm, wall thicknesses of 6.4 mm, and length of 1.20 m. The schematic diagrams of the test set ups are shown in Figs. 3-1 and 3-2. The value of  $H$  is the depth to the base of the pipe and  $H_c$  is the depth to the centerline of the pipe.

To ensure uniformity of test conditions, sand was placed in the test compartment with a mechanical spreading hopper suspended from an overhead service crane. The crane also transported an electric vibrator, which was used to compact the sand to obtain medium or dense deposits.



**Figure 3-1** Experimental set-up of lateral pipe loading



**Figure 3-2** Experimental set-up of upward pipe loading

### 3.3 Testing programme

The testing programme was designed to investigate the effects of embedment depth and density of sand on the pipeline behavior subjected to lateral and upward loadings. The testing

programmes are shown in Tables 3-1 and 3-2 for lateral and upward pipe loadings, respectively.

**Table 3-1** Summary of experiments of lateral loading of pipe in sand  
(Trautmann & O'Rourke, 1983)

Test No.	Pipe Diameter (mm)	Density (kN/m <sup>3</sup> )	<i>H/D</i>
45	102	16.4	2.0
26	102	16.4	4.0
27	102	16.4	4.0
29	102	16.4	4.0
30	102	16.4	4.0
46	102	16.4	6.0
51	102	16.4	8.5
48	102	16.4	11.5
49	102	16.4	11.5
22	102	17.7	2.0
23	102	17.7	4.0
24	102	17.7	6.0
25	102	17.7	8.5
32	102	17.7	11.5

**Table 3-2** Summary of experiments of upward loading of pipe in sand  
(Trautmann & O'Rourke, 1983)

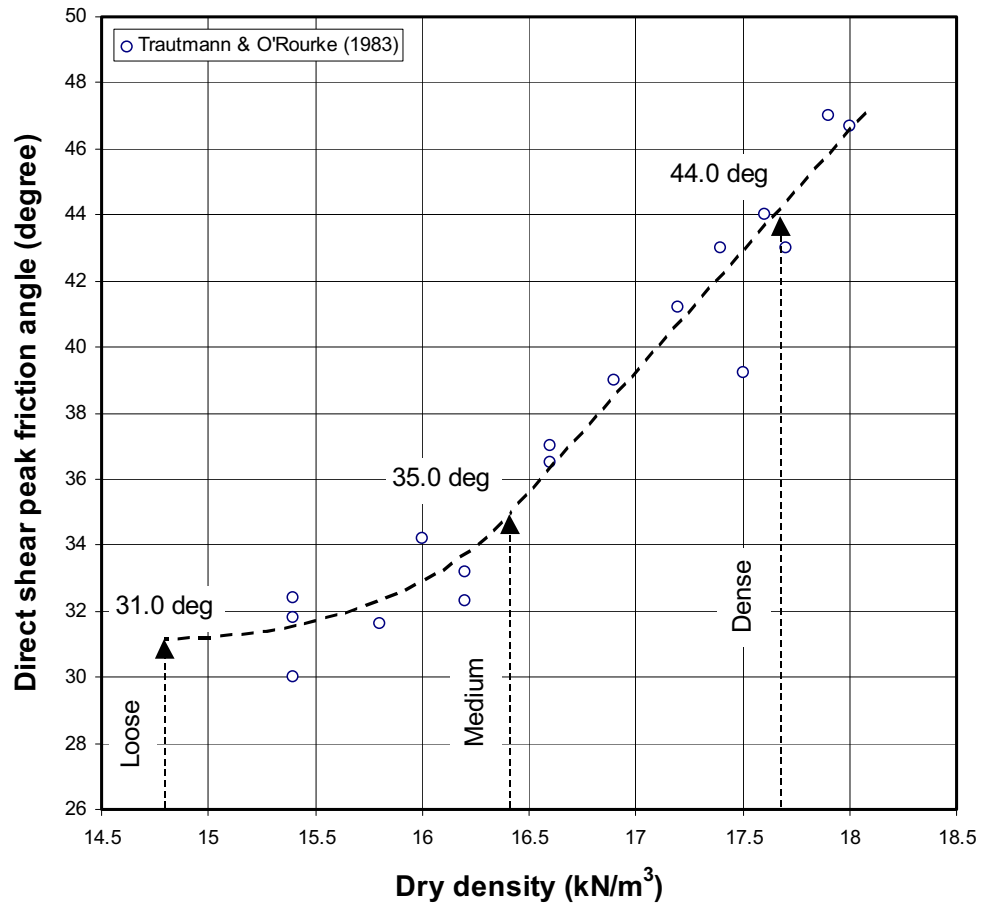
Test No.	Pipe Diameter (mm)	Density (kN/m <sup>3</sup> )	<i>H<sub>c</sub>/D</i>
44	102	16.4	1.5
43	102	16.4	4.0
42	102	16.4	8.0
41	102	16.4	13.0
38	102	17.7	1.5
37	102	17.7	4.0
40	102	17.7	8.0
39	102	17.7	13.0
50	102	17.7	4.0

### 3.4 Properties of tested sand

Cornell filter sand was used for all the tests. This soil is a clean, subangular, fluvioglacial sand, having a coefficient of uniformity  $C_u$  of 2.6, and an effective grain size  $D_{10}$  of 0.2 mm. It contains 20% quartz, 40% limestone fragments, 20% siltstone fragments, and 10% quartzite fragments in addition to trace quantities of other constituents. Its specific gravity is 2.74 and the minimum and maximum densities, measured in accordance with ASTM standard, are 15.5 and 18.3 kN/m<sup>3</sup>, respectively. Uncompacted sand deposits made with the sand spreader, however, had a density of 14.8 kN/m<sup>3</sup>, which was lower than the value determined by the ASTM standard procedure. In this study, therefore, the density of 14.8 kN/m<sup>3</sup> obtained with the sand spreader was adopted as the minimum density. The three densities used in the test series were 14.8, 16.4, and 17.7 kN/m<sup>3</sup>, which corresponded to 0, 45, and 80% relative density, respectively. According to Lambe & Whitman (1979), their states can be described as “very loose”, “medium”, and “dense”. However, only medium and dense sands are of interest in this research.

The angle of shearing resistance for the sand was measured in a direct shear test device at normal stresses ranging from 2.5-20 kN/m<sup>2</sup>, which includes the range of stress encountered during the actual tests. It was shown that the angle of shearing resistance was essentially constant with respect to normal stress over the range of 2.5-20 kN/m<sup>2</sup>. Fig. 3-3 shows the relationship between friction angle and initial sand density for a normal stress of 5 kN/m<sup>2</sup>. For the test densities of 14.8, 16.4, and 17.7 kN/m<sup>3</sup>, the corresponding peak friction angles are 31°, 35°, and 44°, respectively.

The results of the torsional simple shear tests and triaxial tests under drained condition on Cornell filter sand were presented by Turner & Kulhawy (1987). The sand specimens were prepared by air pluviation and tamping was employed to achieve the desired density. The data from the triaxial tests were used for the determination of the input soil parameters. The triaxial test results are presented in Sections 4.5.1.



**Figure 3-3** Relationship between peak friction angle from direct shear tests and dry density

### 3.5 Test results

The experimental results by Trautmann & O'Rourke (1983) are presented in terms of force-displacement relationship in comparison with the present DEM analysis results in Appendix A and B.

## **Chapter 4**

### **Distinct element analysis**

#### **4.1 General**

The Distinct Element Method (DEM) was employed as a numerical analysis approach in this study. In the following, the concepts of the DEM analysis are briefly explained. The numerical simulation by the distinct element code PFC<sup>3D</sup> is described. The determination of the input parameters for DEM modelling is explained. The interpretation of the numerical analysis results is mentioned.

#### **4.2 Distinct element method numerical analysis**

The Distinct Element Method (DEM) simulates the mechanical behavior of a system comprised of a collection of arbitrary shaped particles which displace independently from one another and interact only at contacts. The analysis allows finite displacements and rotations of discrete bodies, including complete detachment, and recognizes new contacts automatically as the calculation progresses. The method was introduced by Cundall (1971) for analysis of rock mechanics problems and then applied to soils by Cundall & Strack (1979).

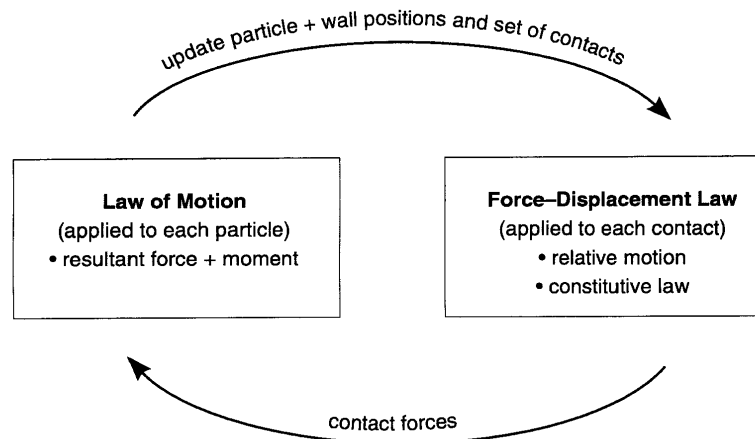
In the DEM analysis, the interaction of particles is viewed as a transient problem with the state of equilibrium developing whenever the internal forces balance. The equilibrium of contact forces and displacements of a stressed assembly of particles are found through a series of calculations tracing the movements of the individual particles. These movements are the results of propagation through the medium of disturbance originating at the boundaries. This is a dynamic process in which the speed of propagation depends on the physical properties of the discrete system.

The dynamic behavior is represented numerically by a timestepping algorithm, in which it is assumed that the velocities and accelerations are constant within each timestep. The solution scheme is identical to that used by the explicit finite-difference method for continuum analysis. The timestep is chosen to be very small so that, during a single timestep, disturbance cannot propagate from any particle further than its immediate neighbors. Hence, at all times, the forces acting on any particles are determined exclusively by its interaction with the particles with which it is in contact. The critical timestep can be calculated from the



speed at which the disturbance can propagate. The speed is a function of the physical properties of the discrete system (e.g. mass and moment of inertia of the particles, and contact stiffnesses). The use of an explicit, as opposed to an implicit, numerical scheme makes it possible to simulate the non-linear interaction of a large number of particles without excessive memory requirements or the need for an iteration procedure.

The calculation cycles performed in DEM analysis repeatedly alternate between the application of Newton's second law of motion to the particles and a force-displacement law at the contacts as well as the constant updating of the particles and walls position as shown in Fig. 4-1. Newton's second law is used to determine the motion of each particle arising from the contact and body forces acting upon it, while the force-displacement law is used to update the contact forces arising from the relative motion at each contact. The important characteristic of the DEM is an allowance of finite displacements and rotations of discrete bodies, including complete detachment and ability to recognise new contact automatically as the calculation progresses. Thus, the efficient contact detection and revision algorithm is very important for DEM simulation. Further details of the available algorithm can be found in e.g. Cundall (1988a) and Lian et al. (1994).



**Figure 4-1** Calculation cycle in DEM analysis (Itasca, 1999)

### 4.3 Distinct element code PFC<sup>3D</sup>

#### 4.3.1 General assumptions

In this study, DEM numerical analysis is performed by the distinct element code PFC<sup>3D</sup>, which is commercially available by Itasca Consulting Group. The code PFC<sup>3D</sup> models the

movement and interaction of the stressed assemblies of rigid spherical particles. It can be viewed as a simplified implementation of the DEM because of the restriction to rigid spherical particles: the general DEM can handle deformable polygonal-shaped particles. The code PFC<sup>3D</sup> performs calculation with the following assumptions;

- Particles are treated as rigid bodies.
- Contacts occur over a vanishing small area (i.e. at a point).
- Behavior at the contacts uses a soft-contact approach, wherein the rigid particles are allowed to overlap to another at contact points.
- The magnitude of the overlap is related to the contact force via the force-displacement law, and all overlaps are small in relation to particle sizes.
- All particles are spherical.

Hereafter, the spherical particle and the rigid boundary wall acting on the particle in PFC<sup>3D</sup> model are sometimes simply referred to as “ball” and “wall”, respectively.

By default, PFC<sup>3D</sup> operates in “static” mode – that is, internal damping is applied that causes the system of particles to reach equilibrium in a minimum number of cycles. If the system is physically unstable, then the resulting motion will be damped but not eliminated. The type of damping embodied in PFC<sup>3D</sup> acts to suppress acceleration motion rather than velocity. In this way, steady motion (e.g. particles in steady flow down a chute) is not prohibited. For situations where accelerations and inertial forces are known to be important, the damping should be reduced to a value that is more reasonable physically. In the present analysis, the static mode was employed. More information concerning the theoretical background of the DEM analysis by PFC<sup>3D</sup> can be found in the PFC<sup>3D</sup> manual (Itasca, 1999).

#### **4.3.2 Contact models**

DEM analysis requires knowledge of a force-displacement law at particle contacts. Particles interact with other particles and boundary walls through the forces that develop at their contacts. The term “contact” is used here to denote this interaction and each contact corresponds to a physical touching. In PFC<sup>3D</sup>, every contact involves two entities (either ball-to-ball or ball-to-wall) and occurs at a single point through which the contact force acts.

A contact model describes the physical behavior occurring at a contact. It is common to use an elastic-perfectly plastic relationship between the contact force and relative contact displacement. The normal and tangential contact stiffnesses relate the normal and tangential components of contact forces and relative contact displacements. In PFC<sup>3D</sup>, the normal components are related by a secant stiffness, while the tangential components are related by a tangent stiffness. Thus, the total normal contact force can be computed at any time if the total normal contact displacement is known, but only the increment of tangential contact force can be computed for a given increment of tangential contact displacement. This means that if the normal contact stiffness is altered during the course of a simulation, there will be an immediate effect upon the entire assembly, but if the tangential contact stiffness is altered during the course of a simulation, it will only affect the new increments of tangential contact force. This fact makes it possible for the PFC<sup>3D</sup> code to handle arbitrary placement of balls and changes in ball radius after a simulation has begun.

#### 4.3.2.1 Linear elastic contact model

The linear elastic contact model is employed in this study. In the linear elastic contact model, contact forces and relative contact displacements are linearly related by constant contact stiffnesses, which are functions of the intrinsic contact stiffnesses of the two contacting entities. The linear elastic contact model is defined by the normal contact stiffness  $K_N$  and tangential contact stiffness  $k_T$  (force/displacement) of the two contacting entities (ball-to-ball or ball-to-wall).

$K_N$  and  $k_T$  relate contact force and relative contact displacement in the normal and tangential directions via Eqs. (4-1) and (4-2). The normal contact stiffness is a secant stiffness since it relates the total normal contact force  $f_N$  to the total normal contact displacement  $d_N$ , whereas the tangential contact stiffness is a tangent stiffness since it relates the increment of tangential contact force  $\Delta f_T$  to the increment of tangential contact displacement  $\Delta d_T$ .

$$f_N = K_N d_N \quad \dots\dots\dots (4-1)$$

$$\Delta f_T = k_T \Delta d_T \quad \dots\dots\dots (4-2)$$

The contact stiffnesses, when two particles interact, are computed assuming that the stiffnesses of the two contacting entities act in series. The secant normal contact stiffness and tangent tangential contact stiffness are, therefore, defined by;

$$K_N = \frac{K_N^A K_N^B}{K_N^A + K_N^B} \dots\dots\dots (4-3)$$

$$k_T = \frac{k_T^A k_T^B}{k_T^A + k_T^B} \dots\dots\dots (4-4)$$

where the superscripts *A* and *B* denote the entities in contact (ball-to-ball or ball-to-wall).

#### 4.3.2.2 Criteria for particle sliding

Particle sliding occurs when the tangential contact force reaches its maximum allowable value which is taken to be the minimum coefficient of inter-particle friction angle of the two contacting entities multiplied by the magnitude of the normal contact force. Every contact is checked for particle sliding by calculating the maximum allowable tangential contact force;

$$f_{T,max} = \tan \phi_{\mu,min} f_N \dots\dots\dots (4-5)$$

where  $f_{T,max}$  is the maximum allowable tangential contact force,  $\phi_{\mu,min}$  is the minimum inter-particle friction angle of the two contacting entities, and  $f_N$  is the normal contact force.

When the particle sliding occurs, the magnitude of the tangential contact force is scaled down to remain at its maximum allowable value.

#### 4.3.3 Micromechanics parameters

During the analysis, several micromechanics quantities can be obtained over spherical volumes (referred to as a “measurement sphere”) positioned at user-selected locations in the model. These quantities include coordination number, porosity, sliding fraction, stress, and strain rate. However, the only micromechanics parameter relevant to this study is the porosity and void ratio.

Porosity *n* is defined as the ratio of the total void volume within the measurement sphere to the measurement sphere volume;

$$n = \frac{V_{void}}{V_{sphere}} = \frac{V_{sphere} - V_{ball}}{V_{sphere}} = 1 - \frac{V_{ball}}{V_{sphere}} \quad \dots\dots\dots (4-6)$$

where  $V_{sphere}$  and  $V_{void}$  are the volumes of the measurement sphere and the void, and  $V_{ball}$  is the volume of the measurement sphere occupied by the balls.  $V_{ball}$  is computed by;

$$V_{ball} = \sum_{N_p} V_p - V_{overlap} \quad \dots\dots\dots (4-7)$$

where  $N_p$  is the number of particles that intersect the measurement sphere,  $V_p$  is the volume of particle  $p$  contained within the measurement sphere, and  $V_{overlap}$  is the volume of particle overlap contained within the measurement sphere. Note that both partial volumes of particles that intersect the measurement sphere and particle overlaps arising from normal contact forces are accounted for in this computation.

Once porosity is obtained, the void ratio  $e$  can be computed by;

$$e = \frac{n}{1 - n} \quad \dots\dots\dots (4-8)$$

## 4.4 DEM analysis by PFC<sup>3D</sup>

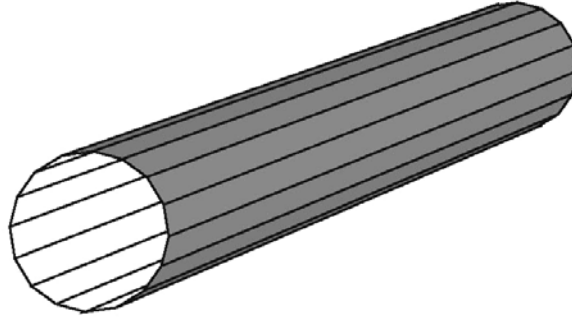
### 4.4.1 Numerical modeling

The experimental set-ups described in Chapter 3 were modeled by PFC<sup>3D</sup> for the DEM analysis. The modeling strategies are described as follow.

#### 4.4.1.1 Tank and pipe

The tank and pipe were modeled by series of planar wall. The dimensions of the tank were the same as the actual tank for the cases of  $H/D < 30$  and  $H_c/D < 30$ , whereas the width was doubled for the cases of  $H/D \geq 30$  and  $H_c/D \geq 30$ . The pipe cannot be modeled as a cylindrical shape due to the fact that the wall in PFC<sup>3D</sup> has to be always in plane. Therefore, the circular cross-section of the pipe was approximately modeled by a 16-side polygon (see Fig. 4-2). The tank wall was assumed to be smooth; the tank model has a normal contact stiffness equal to that of the particles but has zero tangential contact stiffness and zero surface friction. The pipe has identical contact stiffness in both normal and tangential directions and equal to that of the particles. The pipe has its surface friction angle equal to half of the inter-

particle friction angle of sand. The parametric study (Appendix H and I) shows that the change in the pipe surface friction angle between  $\phi_{pipe} = \phi_{\mu}/2$  and  $\phi_{pipe} = \phi_{\mu}$  has negligible effects on the pipe loading results.

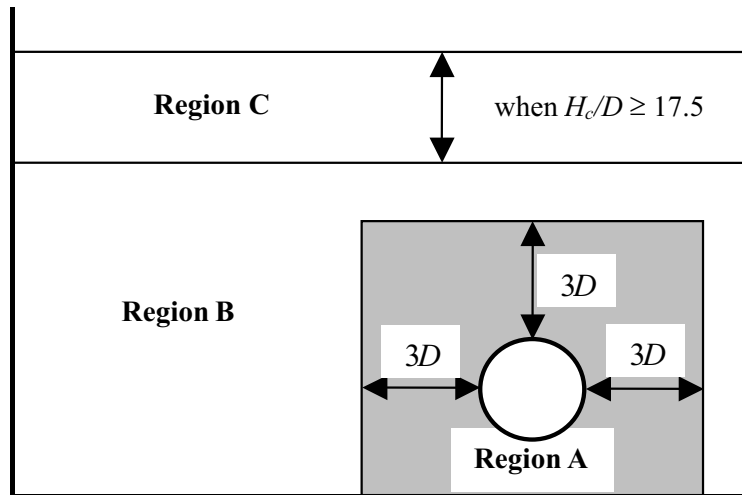


**Figure 4-2** DEM model of pipe

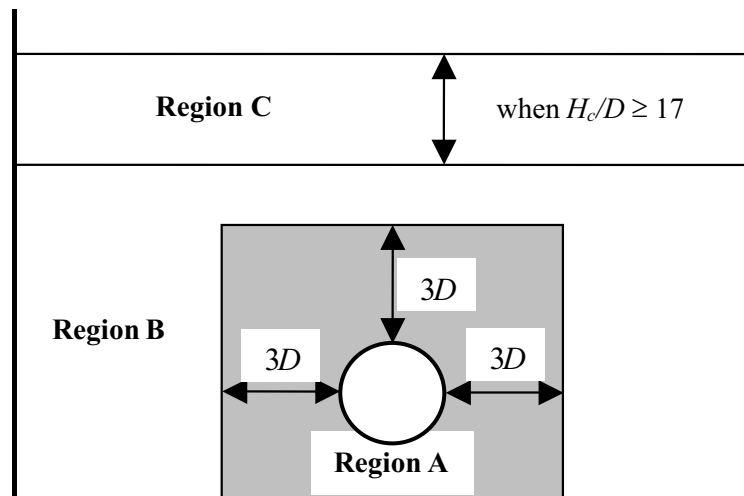
#### 4.4.1.2 Sand particles

Sand was modeled as a collection of spherical particles with its size distribution follows normal distribution. Due to computational limitation, it was not possible to model using the actual particle size ( $D_{10} = 0.2$  mm and  $D_{60} = 0.52$  mm) because the number of balls reaches several hundreds thousand. Therefore, the sand particles were modeled by using larger sizes than actual sand with varying sizes in various regions of the model. For the cases of  $H/D < 30$  and  $H_c/D < 30$ , the numerical models are shown in Fig. 4-3 which are identical to the size of experimental tank test by Trautmann & O'Rourke (1983). At the region near the pipe (Region A), the particles are smaller with  $r_{average} = 12.5$  mm and standard deviation = 2.5 mm (25 times larger than actual sand). Further away (Region B), the particles are larger with  $r_{average} = 25.0$  mm and standard deviation = 5.0 mm (50 times larger). When  $H/D \geq 17.5$  (for lateral pipe loading) or  $H_c/D \geq 17$  (for upward pipe loading), there is Region C with the particle size of  $r_{average} = 37.5$  mm and standard deviation = 7.5 mm (75 times larger). For the cases of  $H/D \geq 30$  and  $H_c/D \geq 30$ , the numerical models are shown in Fig. 4-4 which have their width doubled to reduce boundary interference. At Region D, the particles have  $r_{average} = 25.0$  mm and standard deviation = 5.0 mm (50 times larger than actual sand). Further away (Region E), the particles are larger with  $r_{average} = 37.5$  mm and standard deviation = 7.5 mm (75 times larger). This allowed the number of particles to be less than 180,000 for deepest case. The effects of the extent of the fine particles on the numerical analysis results are also

investigated in the parametric study (Appendix H and I). The results show that the extent of fine particles of three time of the diameter of the pipe give acceptable results. The normal and tangential contact stiffnesses of the particles are assumed to be equal ( $K_N = k_T$ ).

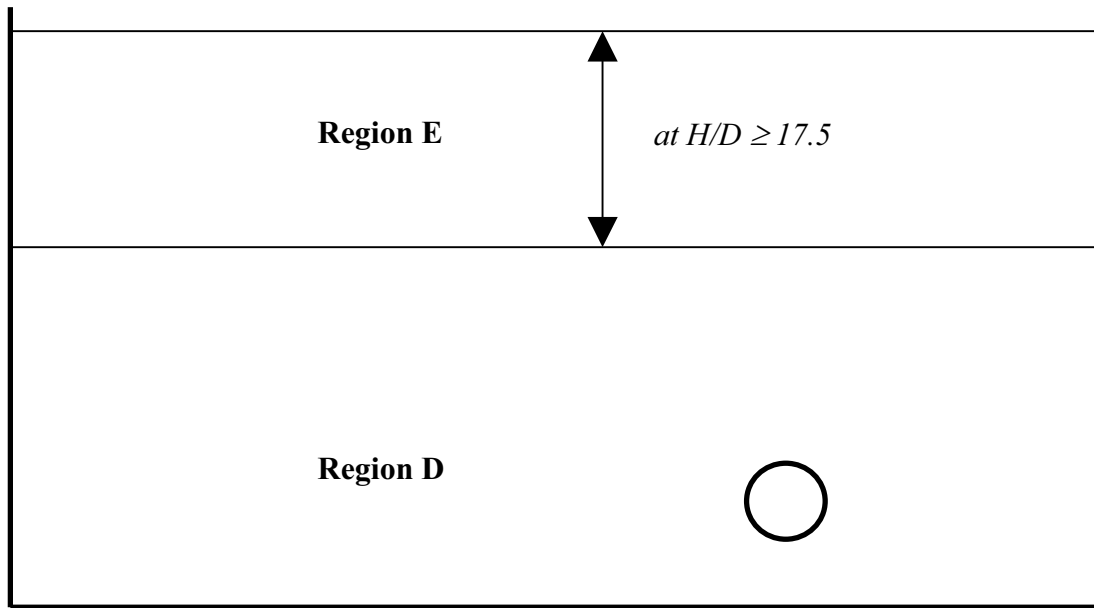


(a) Lateral pipe loading

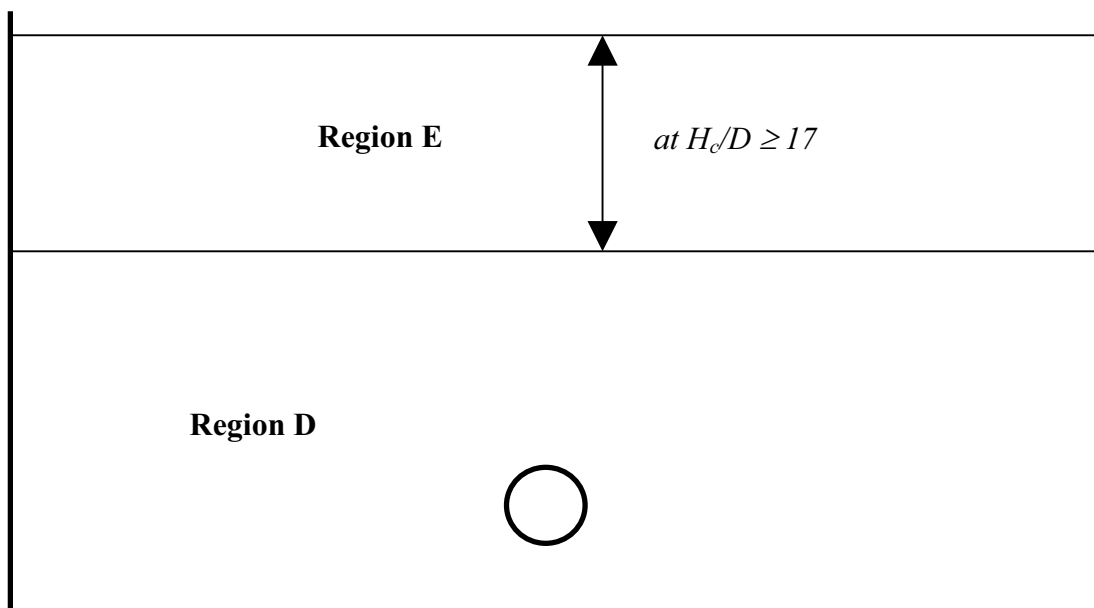


(b) Upward pipe loading

**Figure 4-3** Various regions of the DEM model ( $H/D < 30$  and  $H_c/D < 30$ )



(a) Lateral pipe loading



(b) Upward pipe loading

**Figure 4-4** Various regions of the DEM model ( $H/D \geq 30$  and  $H_c/D \geq 30$ )



## 4.4.2 Numerical analysis procedures

The DEM analysis procedures consist of 4 main steps which are explained as follow.

### 4.4.2.1 Tank set-up

Test tank was numerically prepared by filling the predetermined number of reduce-sized particles into the required cubical space before radius expansion of the spheres to a specified value to obtain predetermined void ratio of the specimen. The specimen preparation method by this radius-expansion has some advantages over the wall-moving method in which the full-sized particles are filled into the cubical space and the positions of the boundary walls are adjusted until the required void ratio is achieved. The radius-expansion method results in more isotropic and uniform specimen, needs less time to reach equilibrium, and preserves the boundary geometry (Itasca, 1999).

### 4.4.2.2 Initial condition

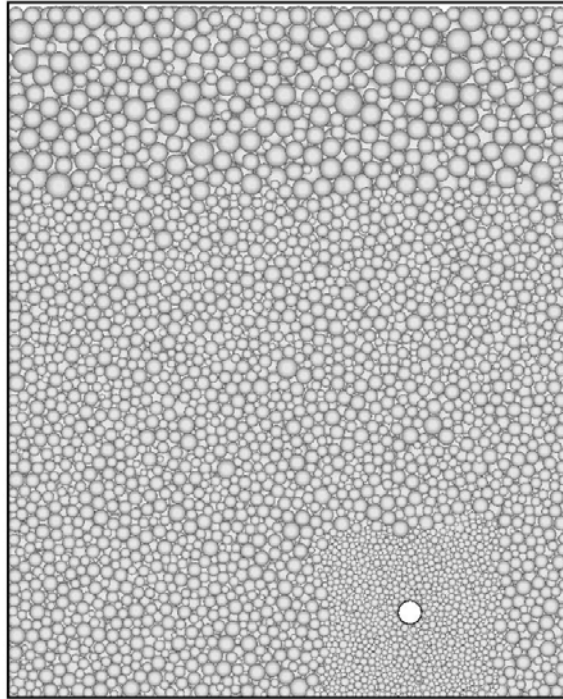
After set-up, the generated balls were subjected to gravitational force to reach the geostatic condition. During this stage, there were some movement of balls and change in void ratio of the specimen. The initial void ratios of the specimens were adjusted (by trial & error) in order to obtain the required void ratios at the end of the initial condition step. The change of the embedment depth ( $H$  or  $H_c$ ) during this stage was also taken into account (by trial & error).

### 4.4.2.3 Pipe installation

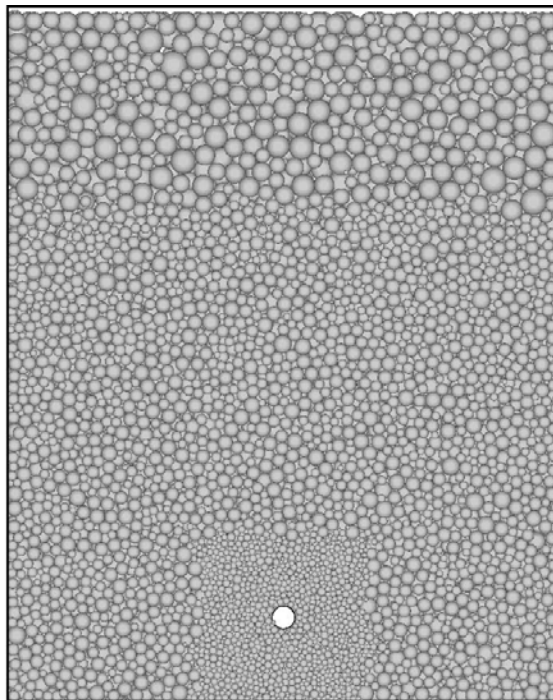
The pipe was placed into the model by deleting the balls which were located at the desired location before the pipe was installed. The model was then allowed to reach equilibrium again by allowing the pipe to slightly adjust its location by keeping the conditions of  $\Sigma F_x = 0$  and  $\Sigma F_y = 0$ . The examples of the DEM model after pipe installation are shown in Fig. 4-5.

#### **4.4.2.4 Pipe pulling**

The pipe was pulled laterally or upwardly by imposing lateral or upward displacement to the pipe. In this manner, the rotation of the pipe was not allowed in the DEM simulation, which is consistent with the experimental set-up and in-situ condition. As reported by Trautmann & O'Rourke (1983) for lateral pipe pulling, the magnitude of the measured torque was negligible with respect to the strength of the pipe in all cases. This means that, in the in-situ condition, the nearby sections of the pipe will restrict the rotation of the laterally loaded section.



(a) *Lateral pipe loading (medium sand,  $H/D = 25$ )*



(b) *Upward pipe loading (medium sand,  $H_c/D = 25$ )*

**Figure 4-5** Examples of DEM model

#### 4.5 Determination of input parameters

The input parameters for DEM modelling are listed in Table 4-1. These parameters are very difficult to quantify by laboratory tests because the parameters are of micro-scale. Therefore, most parameters are determined from trial&error by calibrating the numerical results with the experimental data of (i) triaxial test results by Turner & Kulhawy (1987) and (ii) pipe loading test results at shallow depth by Trautmann & O'Rourke (1983). In the following, the determination of the input parameters are described.

**Table 4-1** Input parameters for DEM analysis

Parameters	Values
Normal contact stiffness of particle, $k_{N,sand}$	From Eqs. (4-10) and (4-11)
Tangent contact stiffness of particle, $k_{T,sand}$	$k_{T,sand} = k_{N,sand}$
Normal contact stiffness of pipe, $k_{N,pipe}$	$k_{N,pipe} = k_{N,sand}$
Tangent contact stiffness of pipe, $k_{T,pipe}$	$k_{T,pipe} = k_{N,pipe}$
Normal contact stiffness of wall, $k_{N,wall}$	$k_{N,wall} = k_{N,sand}$
Tangent contact stiffness of wall, $k_{T,wall}$	$k_{T,wall} = 0$
Inter-particle friction angle, $\phi_\mu$	$\tan \phi_\mu = 0.5, 1.0, 3.0$
Pipe surface friction angle, $\phi_{pipe}$	$\phi_{pipe} = \phi_\mu/2$
Tank wall surface friction angle, $\phi_{wall}$	$\phi_{wall} = 0$
Density of particle, $\rho$ (kg/m <sup>3</sup> )	2740
Radius of particle, $r$	Varies in Regions A, B, C, D, E

##### 4.5.1 Triaxial tests

The triaxial test data on Cornell filter sand was simulated by DEM analysis. The selected experimental details are given in Table 4-2. Table 4-3 presents the detail of DEM simulation. Fig. 4-6 shows that the DEM analysis can simulate the stress-strain relationship from triaxial tests by using a high value of inter-particle friction angle. At low values of the inter-particle friction angle, the match cannot be achieved and the value of  $\tan \phi_\mu = 3.0$  is the lowest value that can still yield good match. The requirement of high inter-particle friction angle for DEM simulation is due to the use of spherical particles, which allows excessive particle rolling (e.g. Thomas & Bray, 1999). Fig. 4-7 shows the dependency of the contact stiffness on the confining stress. The relationship can be derived as shown in Eq. (4-9). The

obtained power value is within the normal value for soil, which is approximately 0.5 (Hardin & Black, 1966). This is due to the fact that the DEM analysis matches both stiffness and strength from triaxial test. It has been shown that the triaxial stiffness (Young's modulus) has a relationship with confining stress in the same manner as the contact stiffness does (Chang & Liao, 1994).

$$k \text{ (N/m)} = 204340 p_o'^{0.41} \quad (p_o' \text{ in kPa}) \quad \dots\dots\dots (4-9)$$

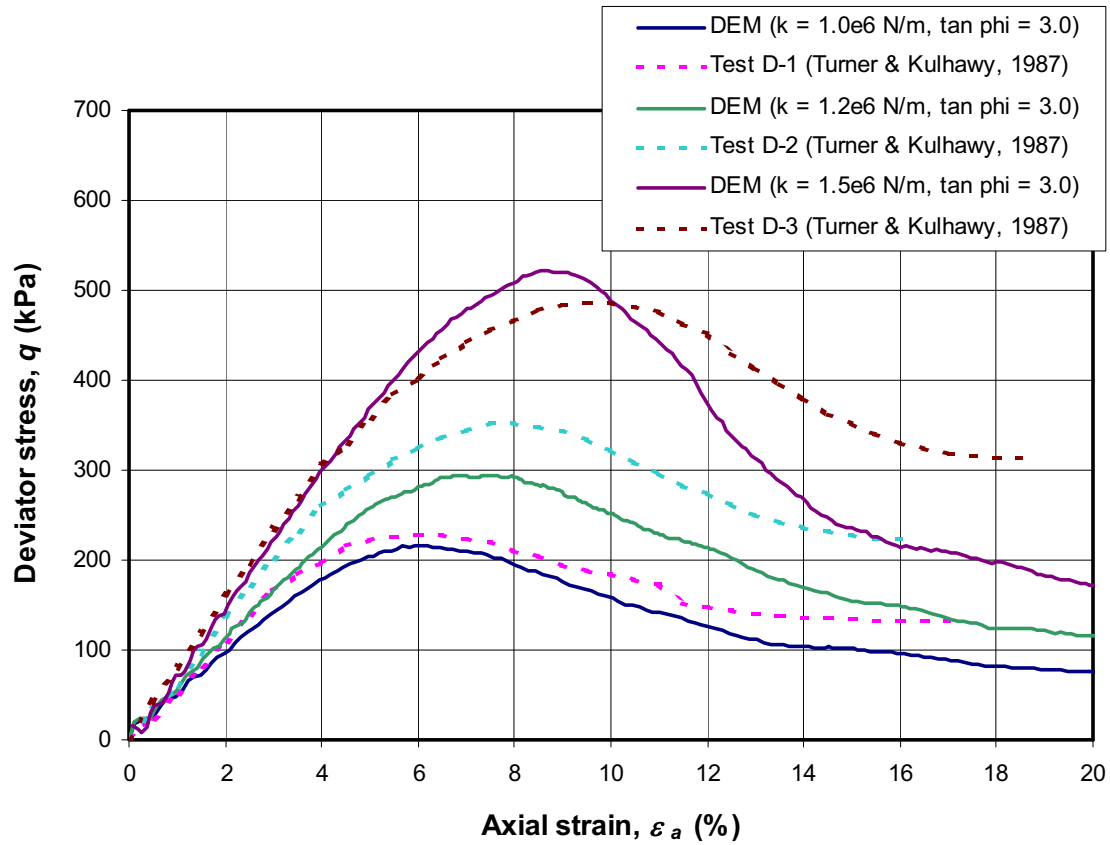
**Table 4-2** Triaxial test programme (after Turner & Kulhawy, 1987)

Test No.	Test type	$\gamma_{dry,o}^*$ (kN/m <sup>3</sup> ) (after consoln.)	$e_o^*$ (after consoln.)	$p_o'$ (kPa)
D-1	CIDC	18.66	0.517	40
D-2	CIDC	17.79	0.540	69
D-3	CIDC	18.05	0.517	98

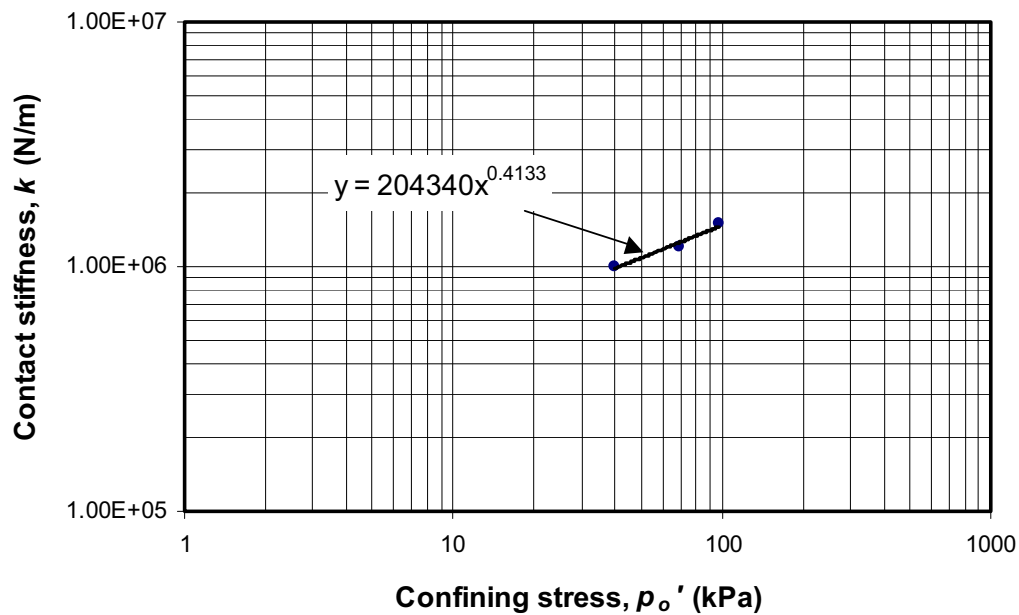
\* estimated from the relevant data because they were not exactly stated in Turner & Kulhawy (1987)

**Table 4-3** DEM analysis detail

Test No.	$e_o$ (after consoln.)	$p_o'$ (kPa)	$k_{N,sand} = k_{T,sand}$ (N/m)
D-1	0.515	40	$1.0 \times 10^6$
D-2	0.517	69	$1.2 \times 10^6$
D-3	0.497	98	$1.5 \times 10^6$



**Figure 4-6** Calibration of DEM results against triaxial test data



**Figure 4-7** Relationship between contact stiffness and confining stress from triaxial simulation

### 4.5.2 Pipe loading test

It was found that the contact stiffness from Eq. (4-9) could not give a good match when used in the pipe loading simulation. Therefore, the contact stiffness was, instead, derived by fitting the peak forces from the DEM analysis results with the pipe loading data at shallow depths by using  $\tan \phi_\mu = 0.5, 1.0, 3.0$ . It was, at first, intended to use  $\tan \phi_\mu = 3.0$ , which was consistent with the value found from the triaxial simulation. However, it was found that using lower value of the inter-particle friction angles could still achieve good match of the peak forces. The force-displacement curves from the case of lowest  $\tan \phi_\mu$  ( $\tan \phi_\mu = 0.5$ ) yield stiffest results due to the largest contact stiffness used, and vice versa. In lateral and upward pipe loading, the case of  $\tan \phi_\mu = 0.5$  shows good match with the experimental results at shallow depth ( $H/D = 2, 4$ , and  $8.5$ ); however, when the embedment depth increases, the case of  $\tan \phi_\mu = 3.0$  give better match. The resulting force-displacement relationships are presented in Appendix A and B. The obtained relationships between the contact stiffness and the vertical effective stress are shown in Fig. 4-8 and the following relationships are proposed.

#### Lateral pipe loading

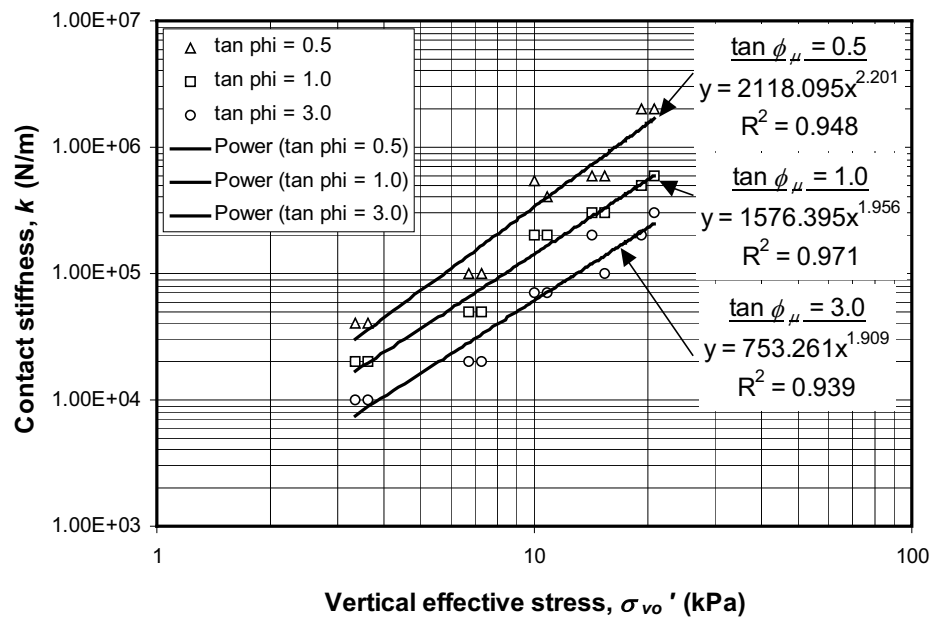
$$\left. \begin{aligned} k \text{ (N/m)} &= 2118.095 \sigma_c'^{2.201} && \text{for } \tan \phi_\mu = 0.5 \\ k \text{ (N/m)} &= 1576.395 \sigma_c'^{1.956} && \text{for } \tan \phi_\mu = 1.0 \\ k \text{ (N/m)} &= 753.261.859 \sigma_c'^{1.909} && \text{for } \tan \phi_\mu = 3.0 \end{aligned} \right\} \dots\dots\dots(4-10)$$

$\sigma_c'$  (kPa) = vertical effective stress at bottom of pipe

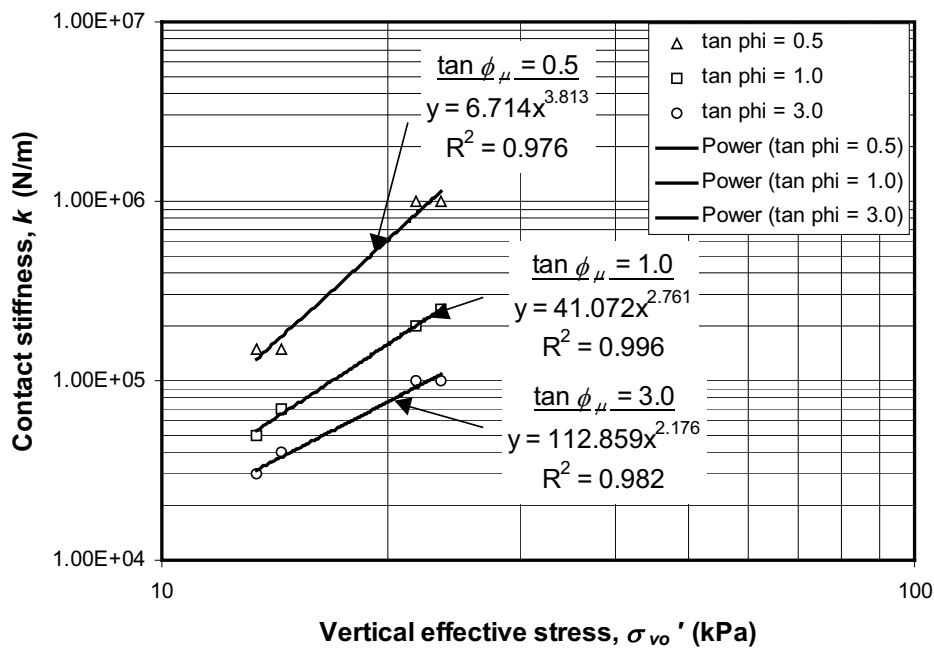
#### Upward pipe loading

$$\left. \begin{aligned} k \text{ (N/m)} &= 6.714 \sigma_c'^{3.813} && \text{for } \tan \phi_\mu = 0.5 \\ k \text{ (N/m)} &= 41.072 \sigma_c'^{2.761} && \text{for } \tan \phi_\mu = 1.0 \\ k \text{ (N/m)} &= 112.859 \sigma_c'^{2.176} && \text{for } \tan \phi_\mu = 3.0 \end{aligned} \right\} \dots\dots\dots (4-11)$$

$\sigma_c'$  (kPa) = vertical effective stress at center of pipe



(a) Lateral pipe loading



(b) Upward pipe loading

**Figure 4-8** Relationship between contact stiffness and confining stress from pipe loading simulation



It is noted that various combinations of the contact stiffness and  $\tan \phi_\mu$  can yield similar peak forces. With larger  $\tan \phi_\mu$ , the required contact stiffness is lower. It is interesting to find that the contact stiffness affects the strength (peak force) of pipe loading problem, which is not the case for triaxial problem where the contact stiffness affects only modulus, not strength (see Appendix G). This may be due to the more complex mode of shearing in pipe loading problem. It is also noted that the obtained power value is greater than 0.5 which is the normal value for soil (Hardin & Black, 1966); this may be also due to the complexity of mode of loading. The peak force values employed for fitting is governed by complex pipeline-soil interaction at relatively large strain; however, the contact stiffness is the behavior at very small strain

#### 4.6 Determination of peak force from DEM analysis results

The peak forces from the DEM analysis results were determined by the following criteria:-

- (a) The peak force was defined as a largest force of the force-displacement curve or as a force which reaches the constant value with increasing pipe displacement.
- (b) When the condition of peak or constant load was not achieved because the analysis could not continue far enough to reach this condition, a rectangular hyperbola was used to fit the data as suggested by Trautmann & O'Rourke (1983).

Using the procedure described by Konder (1963), the force-displacement data for each test were plotted on the transformed axes of  $\delta/F$  and  $\delta$ , where  $\delta$  is the pipe displacement and  $F$  is the force exerted on the pipe. A linear regression was then fitted to the data to obtain the slope,  $b$ , and the  $\delta/F$ -axis intercept,  $a$ . The force-displacement can then be approximated as;

$$F = \frac{\delta}{a + b\delta} \quad \dots\dots\dots (4-12)$$

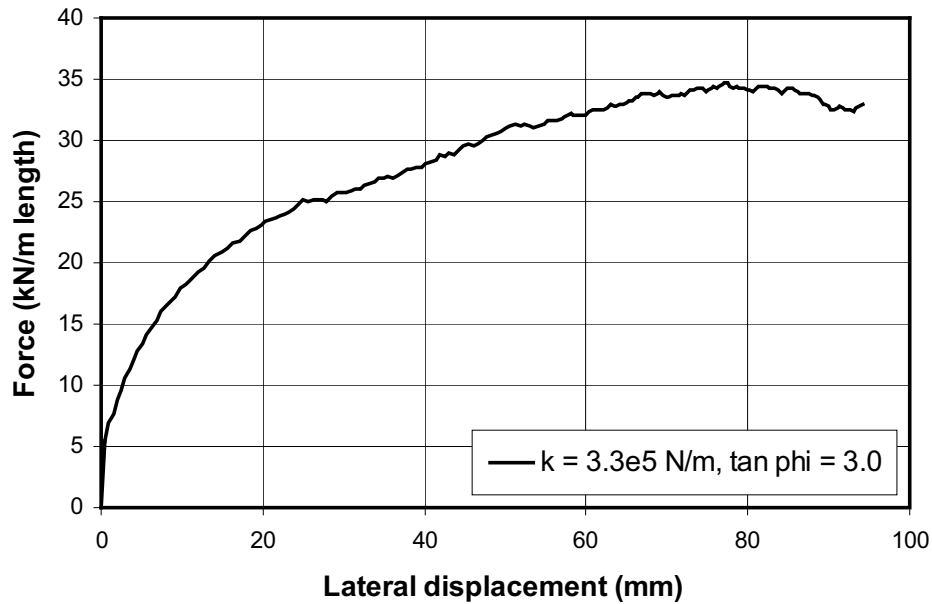
where  $1/a = \lim_{\delta \rightarrow 0} (\partial F / \partial \delta) = \text{initial stiffness of force-displacement curve}$

$$1/b = \lim_{\delta \rightarrow \infty} (F) = \text{peak force}$$

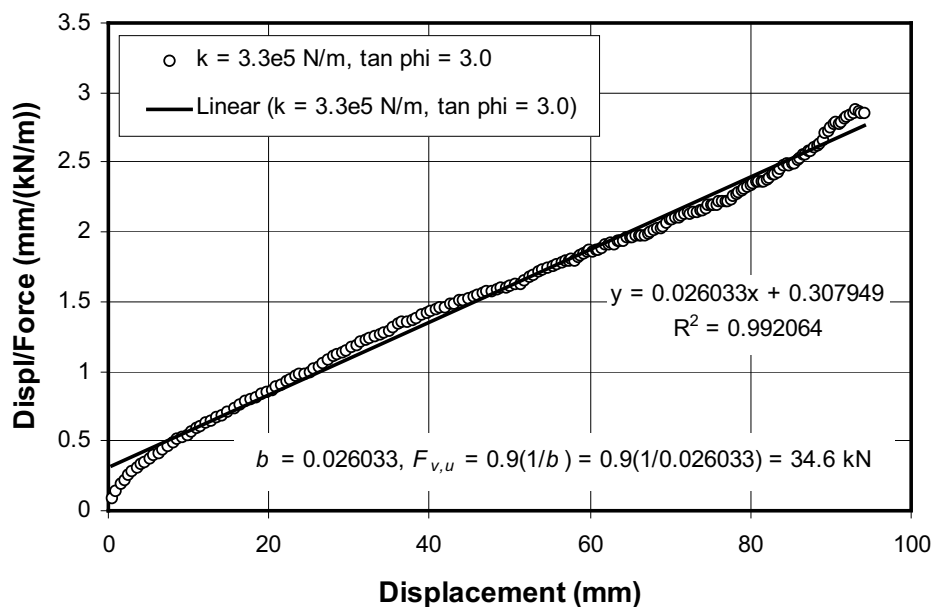
However, as described by Duncan (1980), the hyperbolic model commonly overestimates peak force values. In the instances when an peak force could be determined or

reasonably estimated, the hyperbola was found to overpredict the actual ultimate force by an average of 11%. This trend can be characterised by a failure ratio  $R_f$  of 0.9. This correction value was then applied to all results, i.e.  $F_{peak} = 0.9(1/b)$ .

An example of the hyperbolic fitting is shown in Fig. 4-9.



(a) Force-displacement relationship



(b)  $\delta_v/F_v \sim \delta_v$  relationship

**Figure 4-9** Peak force determination by hyperbolic fitting for lateral pipe loading, medium sand,  $H/D = 14.5$

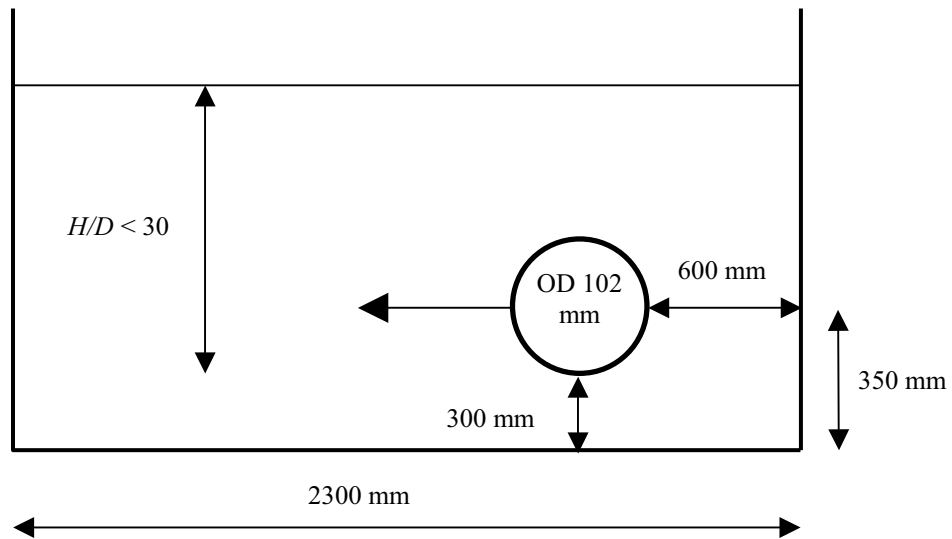
## **Chapter 5**

### **DEM analyses of pipe under lateral loading**

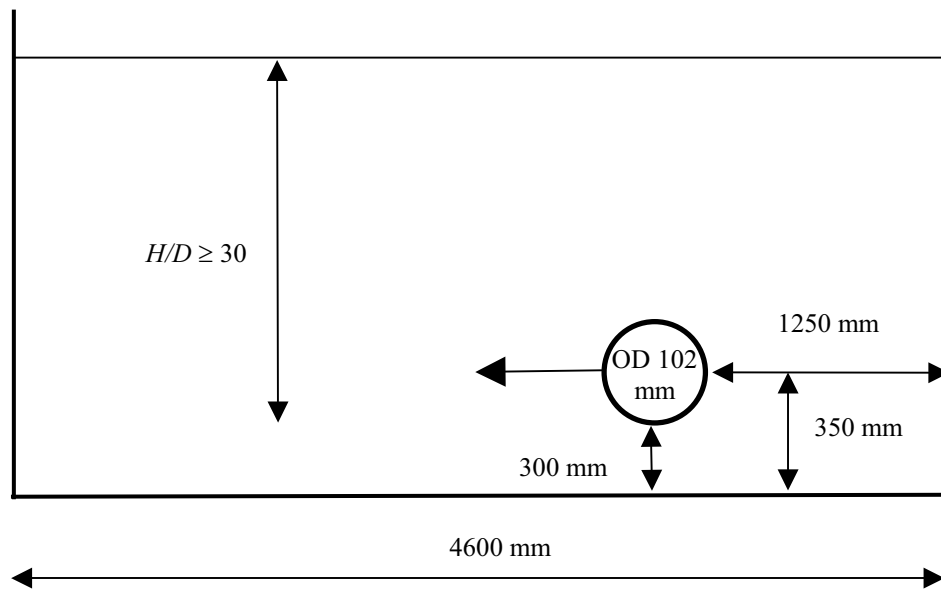
#### **5.1 Analysis procedure and program**

The geometry of the numerical model was based on the experimental study by Trautmann & O'Rourke (1983). For all cases of medium sand and the cases of dense sand with  $H/D < 30$ , all dimensions of the numerical model are the same as those of the experimental set-up except that the embedment depth was adjusted to the required value (Fig. 5-1a). For dense sand with  $H/D \geq 30$ , the width of the model was doubled to reduce the effect of boundary interference. The analyses were performed for medium and dense sands.

The tank and pipe were modelled by series of planar wall. Sand was modelled as a collection of spherical particles and the size distribution follows normal distribution. The tank wall was assumed to be smooth; the tank model has a normal contact stiffness equal to that of the particles but has zero tangential contact stiffness and zero surface friction. The pipe has identical contact stiffness in both normal and tangential directions and equal to that of the particles. The pipe has its surface friction angle equal to half of the inter-particle friction angle of sand. The pipe was pulled laterally by imposing lateral displacement to the pipe. In this manner, the rotation of the pipe was not allowed in the DEM simulation, which is consistent with the experimental set-up and in-situ condition. As reported by Trautmann & O'Rourke (1983), the magnitude of the measured torque was negligible with respect to the strength of the pipe in all cases. This means that, in the in-situ condition, the nearby sections of the pipe will restrict the rotation of the laterally loaded section. The DEM analysis program is shown in Tables 5-1 and 5-2.



(a) for all cases of medium sand and the cases of dense sand with  $H/D < 30$



(b) for dense sand with  $H/D \geq 30$

**Figure 5-1** Geometry of the DEM analyses of lateral pipe loading

**Table 5-1** Summary of DEM analysis program and their input parameters: medium sand

Test No.	$H/D$	Case of $\tan \phi_\mu = 0.5$		Case of $\tan \phi_\mu = 1.0$		Case of $\tan \phi_\mu = 3.0$	
		$\tan \phi_\mu$	$k_{N,sand}$ (N/m)	$\tan \phi_\mu$	$k_{N,sand}$ (N/m)	$\tan \phi_\mu$	$k_{N,sand}$ (N/m)
LMHD2	2	0.5	$4.0 \times 10^4$	1.0	$2.0 \times 10^4$	3.0	$1.0 \times 10^4$
LMHD4	4	0.5	$1.0 \times 10^5$	1.0	$5.0 \times 10^4$	3.0	$2.0 \times 10^4$
LMHD6	6	0.5	$5.5 \times 10^5$	1.0	$2.0 \times 10^5$	3.0	$7.0 \times 10^4$
LMHD85	8.5	0.5	$6.0 \times 10^5$	1.0	$3.0 \times 10^5$	3.0	$2.0 \times 10^5$
LMHD115	11.5	0.5	$2.0 \times 10^6$	1.0	$5.0 \times 10^5$	3.0	$2.0 \times 10^5$
LMHD145	14.5	0.5	$2.4 \times 10^6$	1.0	$8.1 \times 10^5$	3.0	$3.3 \times 10^5$
LMHD175	17.5	0.5	$3.6 \times 10^6$	1.0	$1.2 \times 10^6$	3.0	$4.8 \times 10^5$
LMHD22	22	0.5	$5.9 \times 10^6$	1.0	$1.8 \times 10^6$	3.0	$7.4 \times 10^5$
LMHD25	25	0.5	$7.8 \times 10^6$	1.0	$2.3 \times 10^6$	3.0	$9.4 \times 10^5$
LMHD30	30	0.5	$1.2 \times 10^7$	1.0	$3.3 \times 10^6$	3.0	$1.3 \times 10^6$
LMHD40	40	0.5	$2.2 \times 10^7$	1.0	$5.9 \times 10^6$	3.0	$2.3 \times 10^6$
LMHD60	60	0.5	$5.4 \times 10^7$	1.0	$1.3 \times 10^7$	3.0	$5.0 \times 10^6$

Note: 1.  $k_{N,sand} = k_{T,sand} = k_{N,pipe} = k_{T,pipe} = k_{N,wall}$

2.  $k_{T,wall} = 0$

3.  $\phi_{pipe} = \phi_\mu/2$

4.  $\phi_{wall} = 0$

**Table 5-2** Summary of DEM analysis program and their input parameters: dense sand

Test No.	$H/D$	Case of $\tan \phi_\mu = 0.5$		Case of $\tan \phi_\mu = 1.0$		Case of $\tan \phi_\mu = 3.0$	
		$\tan \phi_\mu$	$k_{N,sand}$ (N/m)	$\tan \phi_\mu$	$k_{N,sand}$ (N/m)	$\tan \phi_\mu$	$k_{N,sand}$ (N/m)
LDHD2	2	0.5	$4.0 \times 10^4$	1.0	$2.0 \times 10^4$	3.0	$1.0 \times 10^4$
LDHD4	4	0.5	$1.0 \times 10^5$	1.0	$5.0 \times 10^4$	3.0	$2.0 \times 10^4$
LDHD6	6	0.5	$4.0 \times 10^5$	1.0	$2.0 \times 10^5$	3.0	$7.0 \times 10^4$
LDHD85	8.5	0.5	$6.0 \times 10^5$	1.0	$3.0 \times 10^5$	3.0	$1.0 \times 10^5$
LDHD115	11.5	0.5	$2.0 \times 10^6$	1.0	$6.0 \times 10^5$	3.0	$3.0 \times 10^5$
LDHD145	14.5	0.5	$2.8 \times 10^6$	1.0	$9.4 \times 10^5$	3.0	$3.8 \times 10^5$
LDHD175	17.5	0.5	$4.2 \times 10^6$	1.0	$1.4 \times 10^6$	3.0	$5.5 \times 10^5$
LDHD22	22	0.5	$7.0 \times 10^6$	1.0	$2.1 \times 10^6$	3.0	$8.5 \times 10^5$
LDHD25	25	0.5	$9.3 \times 10^6$	1.0	$2.7 \times 10^6$	3.0	$1.1 \times 10^6$
LDHD30*	30	0.5	$1.4 \times 10^7$	1.0	$3.9 \times 10^6$	3.0	$1.5 \times 10^6$
LDHD40*	40	0.5	$2.6 \times 10^7$	1.0	$6.8 \times 10^6$	3.0	$2.7 \times 10^6$
LDHD60*	60	0.5	$6.4 \times 10^7$	1.0	$1.5 \times 10^7$	3.0	$5.8 \times 10^6$

Note: 1.  $k_{N,sand} = k_{T,sand} = k_{N,pipe} = k_{T,pipe} = k_{N,wall}$

2.  $k_{T,wall} = 0$

3.  $\phi_{pipe} = \phi_\mu/2$

4.  $\phi_{wall} = 0$

5. \* bigger tank

## 5.2 Input parameters

The input parameters for DEM models were determined as described in Section 4.5. The input parameters for all DEM analyses are presented in Tables 5-1 and 5-2. For the cases of  $2 \leq H/D \leq 11.5$ , the values of  $k_{N,sand}$  were derived from fitting peak forces from the DEM analysis with the results of pipe loading experiment (as described in Section 4.5.2). For the

cases of  $14.5 \leq H/D \leq 60$ , the values of  $k_{N,sand}$  were derived from the obtained relationships from shallow depths as follow.

$$\left. \begin{aligned} k \text{ (N/m)} &= 2118.095 \sigma_c'^{2.201} && \text{for } \tan \phi_\mu = 0.5 \\ k \text{ (N/m)} &= 1576.395 \sigma_c'^{1.956} && \text{for } \tan \phi_\mu = 1.0 \\ k \text{ (N/m)} &= 753.261.859 \sigma_c'^{1.909} && \text{for } \tan \phi_\mu = 3.0 \end{aligned} \right\} \dots\dots\dots (5-1)$$

$\sigma_c'$  (kPa) = vertical effective stress at bottom of pipe

### 5.3 DEM analysis results of shallow embedment depths

In this section, the DEM results of shallow embedment depths ( $H/D \leq 11.5$ ) are presented and compared with the experimental results by Trautmann & O'Rourke (1983) to examine the accuracy of the DEM analysis. The results in terms of the hyperbolic force-displacement relationship, the peak forces, and the corresponding pipe displacement are summarized in Tables 5-3 and 5-4.

Table 5-3 DEM analysis results: medium sand

H/D	$\tan \phi_\mu = 0.5$			$\tan \phi_\mu = 1.0$			$\tan \phi_\mu = 3.0$		
	Hyperbolic fit	$F_{h,peak}$ (kN)	$\delta_{h,peak}$ (mm)	Hyperbolic fit	$F_{h,peak}$ (kN)	$\delta_{h,peak}$ (mm)	Hyperbolic fit	$F_{h,peak}$ (kN)	$\delta_{h,peak}$ (mm)
2	—	2.2	28.0	—	2.2	40.0	—	2.2	46.0
4	—	5.1	33.3	—	5.8	50.6	—	5.6	66.8
6	—	10.5	64.2	—	12.0	69.6	—	11.4	63.7
8.5	$F_h = \frac{\delta_h}{0.0630\delta_h + 0.3813}$	14.3	55.0	$F_h = \frac{\delta_h}{0.0538\delta_h + 0.3347}$	16.7	55.0	$F_h = \frac{\delta_h}{0.0475\delta_h + 0.4190}$	19.0	81.7
11.5	$F_h = \frac{\delta_h}{0.0414\delta_h + 0.2302}$	21.7	49.2	$F_h = \frac{\delta_h}{0.0365\delta_h + 0.2143}$	24.7	53.8	$F_h = \frac{\delta_h}{0.0358\delta_h + 0.3516}$	25.1	87.0
14.5	$F_h = \frac{\delta_h}{0.0372\delta_h + 0.2751}$	24.2	66.7	$F_h = \frac{\delta_h}{0.0282\delta_h + 0.2530}$	31.9	80.4	$F_h = \frac{\delta_h}{0.0260\delta_h + 0.3079}$	34.6	106.1
17.5	$F_h = \frac{\delta_h}{0.0337\delta_h + 0.1726}$	26.7	46.0	$F_h = \frac{\delta_h}{0.0210\delta_h + 0.2133}$	42.9	92.3	$F_h = \frac{\delta_h}{0.0196\delta_h + 0.3286}$	45.9	150.3



22	$F_h = \frac{\delta_h}{0.0281\delta_h + 0.1956}$	32.0	62.1	$F_h = \frac{\delta_h}{0.0169\delta_h + 0.1339}$	53.2	70.6	$F_h = \frac{\delta_h}{0.0162\delta_h + 0.2185}$	55.4	118.1
25	$F_h = \frac{\delta_h}{0.0294\delta_h + 0.0955}$	30.6	29.1	$F_h = \frac{\delta_h}{0.0205\delta_h + 0.1018}$	44.0	45.7	$F_h = \frac{\delta_h}{0.0147\delta_h + 0.1748}$	61.3	108.4
30	$F_h = \frac{\delta_h}{0.0226\delta_h + 0.1235}$	39.8	48.9	—	86.7	38.7	$F_h = \frac{\delta_h}{0.0114\delta_h + 0.1882}$	78.7	114.1
40	$F_h = \frac{\delta_h}{0.0256\delta_h + 0.0612}$	35.1	21.2	$F_h = \frac{\delta_h}{0.0158\delta_h + 0.0808}$	57.0	46.3	$F_h = \frac{\delta_h}{0.0103\delta_h + 0.1164}$	87.0	97.5
60	$F_h = \frac{\delta_h}{0.0209\delta_h + 0.0326}$	43.0	13.8	$F_h = \frac{\delta_h}{0.0118\delta_h + 0.0450}$	76.1	33.6	$F_h = \frac{\delta_h}{0.0092\delta_h + 0.0544}$	97.7	52.5

Note:  $F_h$  is in kN/m length.

$\delta_h$  is in mm.

Table 5-4 DEM analysis results: dense sand

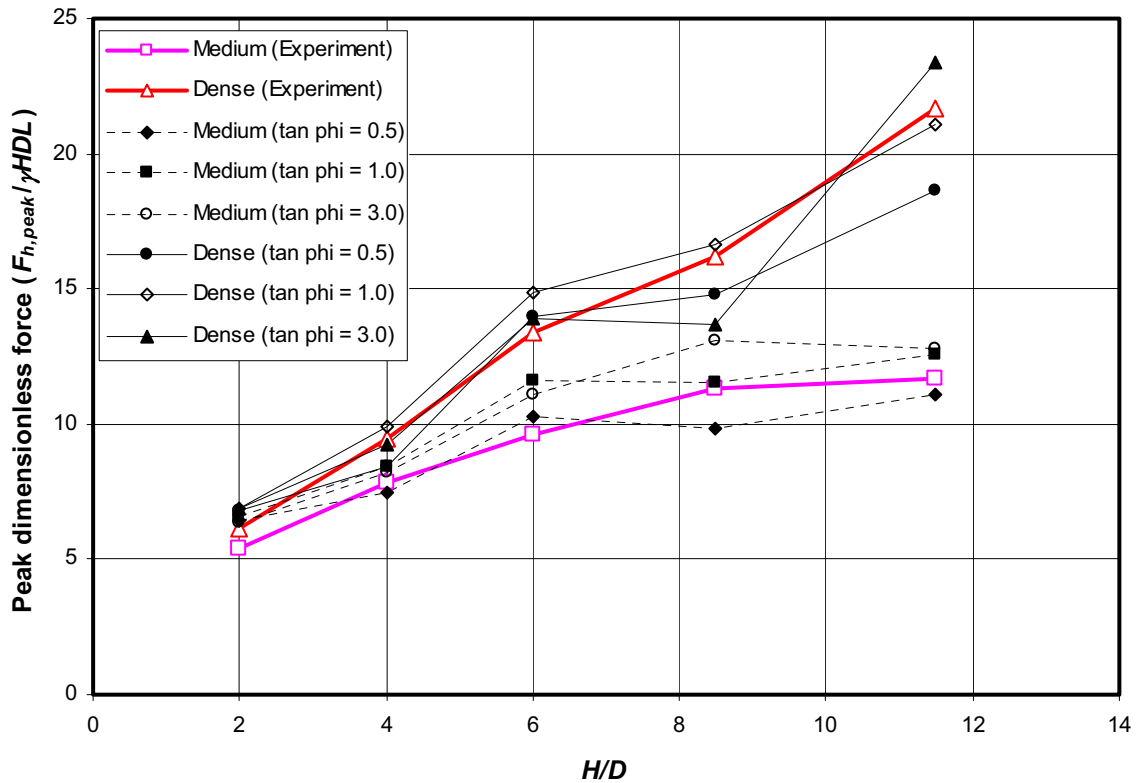
H/D	$\tan \phi_\mu = 0.5$			$\tan \phi_\mu = 1.0$			$\tan \phi_\mu = 3.0$		
	Hyperbolic fit	$F_{h,peak}$ (kN)	$\delta_{h,peak}$ (mm)	Hyperbolic fit	$F_{h,peak}$ (kN)	$\delta_{h,peak}$ (mm)	Hyperbolic fit	$F_{h,peak}$ (kN)	$\delta_{h,peak}$ (mm)
2	—	2.5	10.7	—	2.5	35.9	—	2.5	55.4
4	—	6.2	21.5	—	7.3	35.8	—	6.8	79.0
6	—	13.6	15.7	—	17.0	42.2	—	15.3	79.5
8.5	—	23.2	39.0	—	26.2	64.0	—	21.5	93.6
11.5	—	39.5	29.2	—	44.7	40.1	—	49.7	68.4
14.5	—	49.0	9.6	—	59.7	45.2	—	63.1	75.7
17.5	—	64.3	9.5	$F_h = \frac{\delta_h}{0.0121\delta_h + 0.0569}$	74.1	40.8	$F_h = \frac{\delta_h}{0.0105\delta_h + 0.1565}$	85.3	127.9
22	—	110.9	8.3	—	107.7	35.7	—	104.8	55.6
25	—	122.5	7.8	—	117.7	27.4	—	126.9	59.7

30	—	170.6	22.5	—	167.6	42.5	$F_h = \frac{\delta_h}{0.0056\delta_h + 0.1239}$	161.3	199.8
40	—	300.7	8.0	$F_h = \frac{\delta_h}{0.0038\delta_h + 0.0285}$	234.8	67.0	$F_h = \frac{\delta_h}{0.0032\delta_h + 0.0727}$	278.6	202.6
60	—	852.9	10.0	$F_h = \frac{\delta_h}{0.0016\delta_h + 0.0077}$	417.8	63.5	$F_h = \frac{\delta_h}{0.0020\delta_h + 0.0390}$	449.6	175.4

Note:  $F_h$  is in kN/m length.  
 $\delta_h$  is in mm.

The comparison of the force-displacement curves from the DEM analyses with the experimental results is shown in Appendix A. The peak forces from DEM results were forced to match the experimental results by adjusting the values of contact stiffness by trial & error. For medium sand, the force-displacement curves from DEM match the experimental results well. However, for dense sand, the force-displacement curves from DEM show little difference from the experimental results and the difference is more for the case of  $\tan \phi_\mu = 3.0$ .

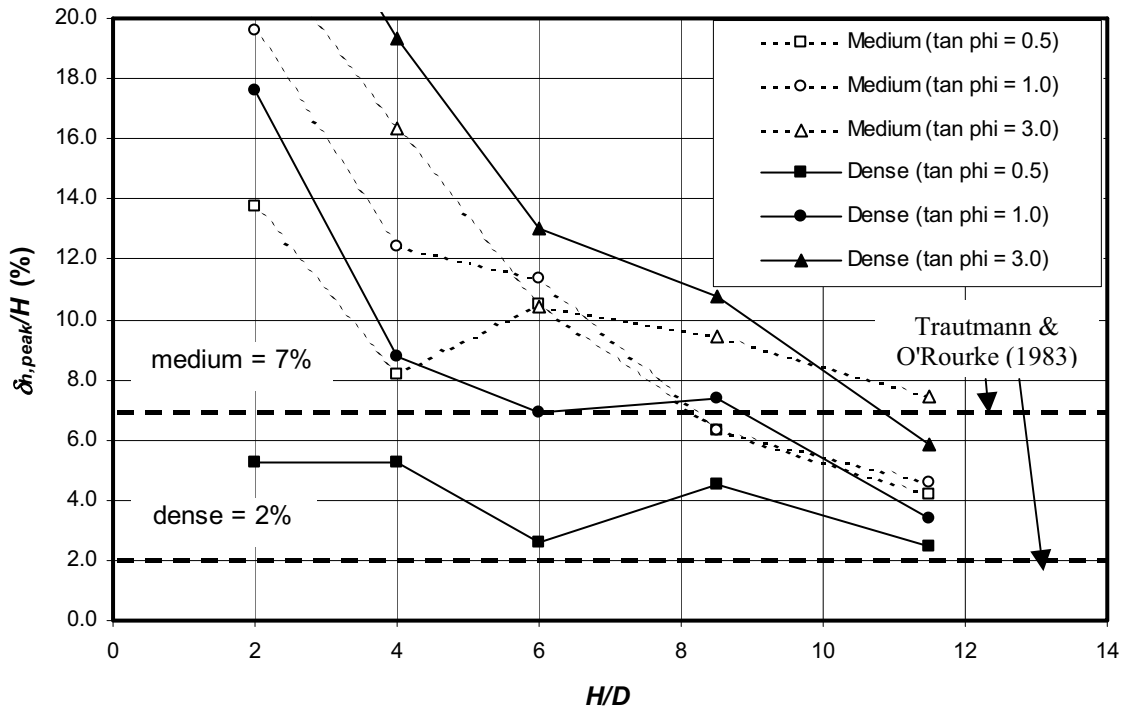
Fig. 5-2 presents the peak dimensionless forces plotted against  $H/D$ . The results show that the DEM analysis can simulate the problem of soil-pipeline interaction reasonably well.



**Figure 5-2** Plots of peak dimensionless force against  $H/D$

Fig 5-3 shows the normalized peak displacements (pipe displacement at peak force normalized by embedment depth) at different  $H/D$ s. The data found in the experiments by Trautmann & O'Rourke (1983) are also included in the figure. At  $2 \leq H/D \leq 6$ , the dimensionless peak displacements from the DEM analysis are larger than those from the experiment, whereas, at  $H/D = 8.5$  and  $11.5$ , these two results become more consistent. The

dimensionless peak displacement shows the trend to decrease with  $H/D$ , which is opposite to the experimental results reported by Dickin & Leung (1983).



**Figure 5-3** Plots of dimensionless peak pipe displacement against  $H/D$

#### 5.4 DEM analysis results of deep embedment depth

The results of DEM analysis for deep embedment depths in terms of the hyperbolic force-displacement relationships, the peak forces, and the corresponding pipe displacements at peak forces are summarized in Tables 5-3 and 5-4.

For medium sand, the force-displacement curves from DEM analysis follow the same trend as those from FE analysis for all embedment depths. For dense sand, the force-displacement curves from DEM analysis show some difference from those of FE analysis, especially at deep depth. The results from the case of  $\tan \phi_\mu = 0.5$  (highest contact stiffness used) show stiffest behavior, whereas the results from the case of  $\tan \phi_\mu = 3.0$  (lowest contact stiffness used) show softest behavior, which is more consistent with the FE results.

Fig. 5-4 presents the peak dimensionless forces plotted against  $H/D$  of up to 60. For medium sand, the peak dimensionless forces from the cases of  $\tan \phi_\mu = 1.0$  and 3.0 yield similar values with the FE results for the whole range of the  $H/D$  of interest ( $H/D = 2$  to 60),

whereas the results from the case of  $\tan \phi_\mu = 0.5$  show smaller values. The peak dimensionless forces increase approximately linearly with  $H/D$  at shallow depths and reach their maximum values at some depths after which the maximum dimensionless forces become approximately constant with  $H/D$ . The depth when this transition occurs is termed “critical embedment depth” (e.g. Dickin & Leung, 1985) and it indicates a deep failure mechanism. The DEM results show that the critical embedment depth for medium sand is 12 with the peak dimensionless force of 15, which is consistent with the results obtained from FE analysis. For dense sand, the DEM results for the cases of  $\tan \phi_\mu = 1.0, 3.0$  yield closest results with the FE analysis with somewhat larger peak dimensionless forces at very deep embedment depth ( $H/D \geq 40$ ) and more gradual transition from shallow to deep failure mechanism. However, the results from the case of  $\tan \phi_\mu = 0.5$  (highest contact stiffness used) show the largest overestimation and does not show any transition from shallow to deep failure mechanism.

Fig 5-5 shows the dimensionless peak displacement plotted against  $H/D$ . The recommended values from ASCE Guideline are also included in the figure. The dimensionless peak displacement decreases with depth. The stiffest behavior (smallest dimensionless peak displacement) is from  $\tan \phi_\mu = 0.5$  which is consistent with the case of largest contact stiffness. The dimensionless peak displacements from the DEM analyses are quite different from those recommended by the ASCE Guideline. At shallow depth, the dimensionless peak displacements from DEM are larger than the ASCE Guideline; however, their values decrease with embedment depth and become smaller than the ASCE Guideline at large depth.

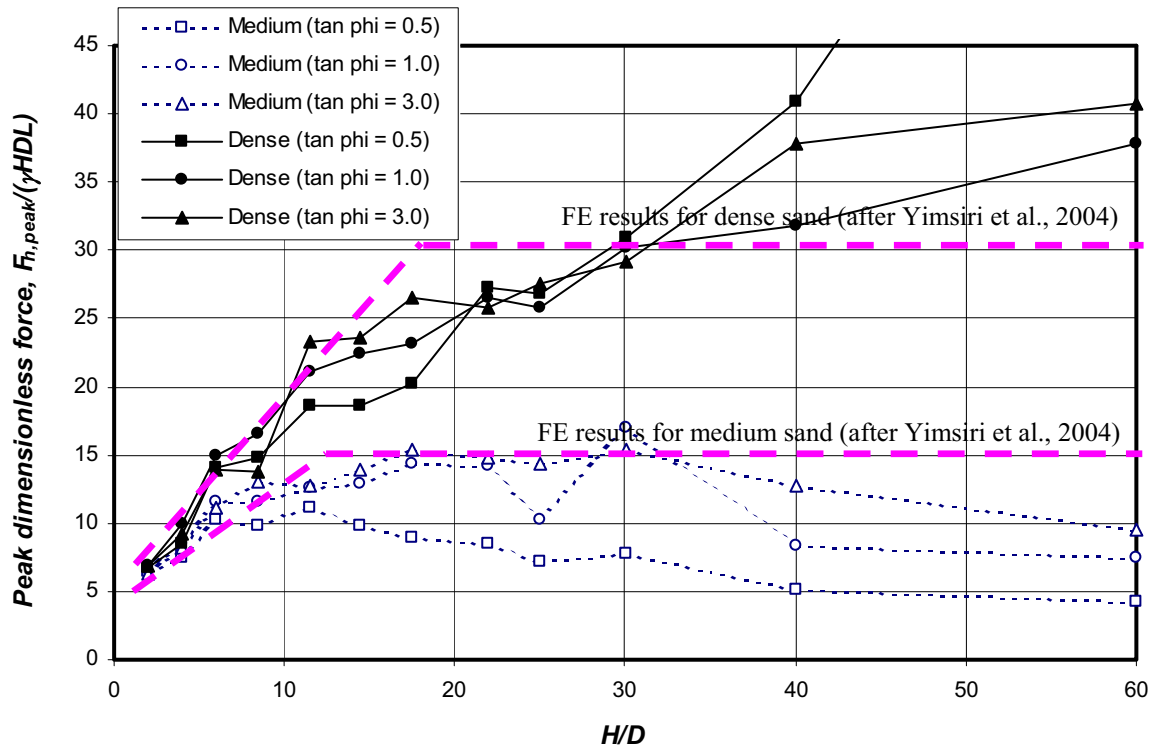


Figure 5-4 Relationship between peak dimensionless force and  $H/D$

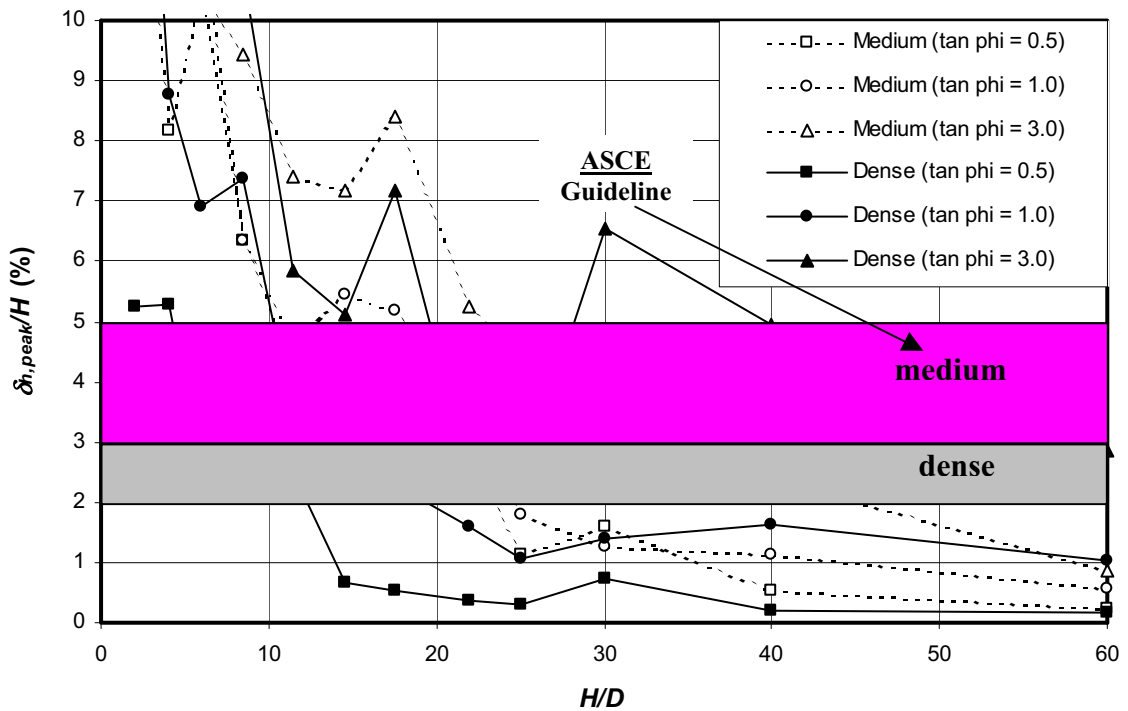


Figure 5-5 Relationship between dimensionless ultimate displacement and  $H/D$

The normalized hyperbolic force-displacement relationship of pipe under lateral loading is proposed by ASCE as shown in Eq. (5-2). This is compared with the DEM analysis results in Figs. 5-6 and 5-7 for medium and dense sands, respectively.

$$\frac{F_h}{F_{h,peak}} = \frac{\delta_h / \delta_{h,peak}}{0.15 + 0.85\delta_h / \delta_{h,peak}} \quad \dots\dots\dots (5-2)$$

For medium sand, the relationship recommended by ASCE reasonably matches the DEM analysis results for the case of  $\tan \phi_\mu = 3.0$ , whereas it forms a lower boundary for the cases of  $\tan \phi_\mu = 0.5, 1.0$ . For dense sand, the relationship recommended by ASCE reasonably matches the DEM analysis results for the cases of  $\tan \phi_\mu = 1.0, 3.0$ ; however, this relationship cannot yield good match with the case of  $\tan \phi_\mu = 0.5$  due to the large scatter of the DEM results themselves.



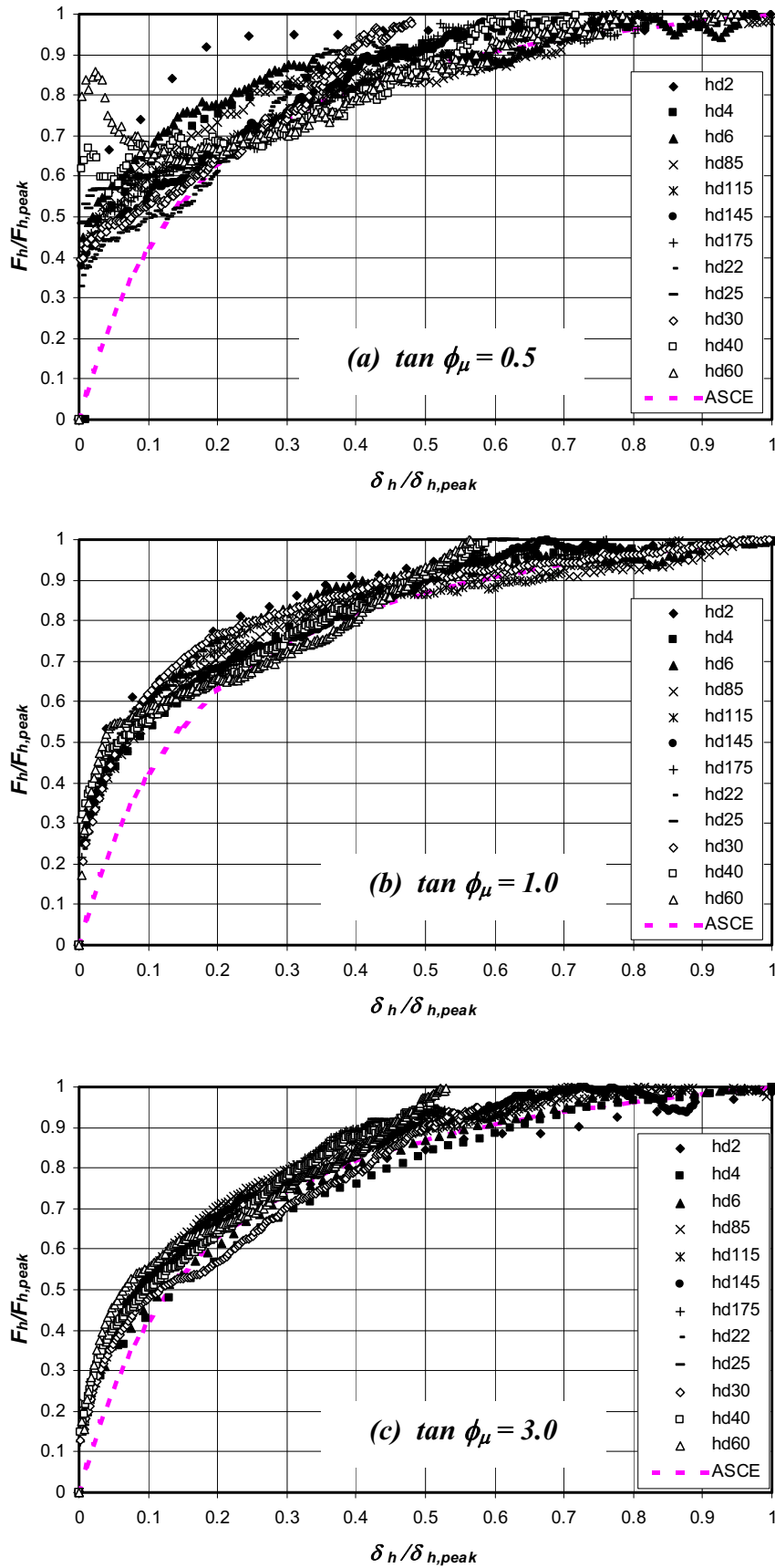


Figure 5-6 Normalized force-displacement relationship: medium sand

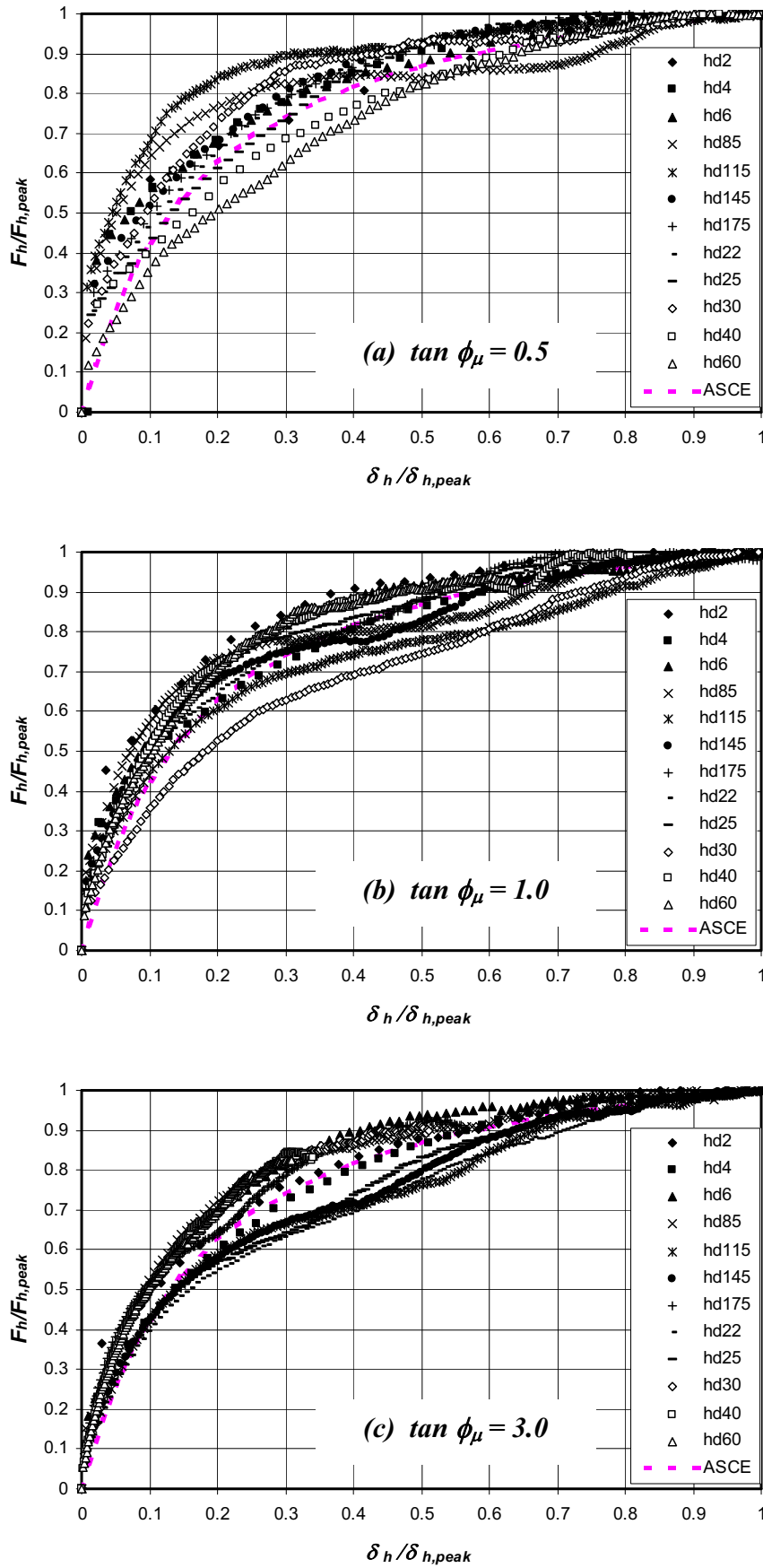


Figure 5-7 Normalized force-displacement relationship: dense sand

## 5.5 Summary

The DEM analysis is used to simulate the pipeline-soil interaction under lateral loading at deep embedment condition by using the input parameters from the calibration of the analysis with the experimental results at shallow depth. At shallow depth, the force-displacement relationships from the DEM analysis show that the results for medium sand match the experimental data well for all cases of  $\tan \phi_\mu$ , whereas the results for dense sand match the experimental data well only for the cases of  $\tan \phi_\mu = 0.5$  and  $1.0$  with too soft behavior for the case of  $\tan \phi_\mu = 3.0$ . At deep depth, the DEM results of medium sand in term of the peak dimensionless force and force-displacement relationship match the FE results well for all cases. The DEM results for medium sand show deep failure mechanism with the critical embedment depth of 12 and the critical peak dimensionless force of 13 which is consistent with the FE results. For dense sand, the DEM results from the cases of  $\tan \phi_\mu = 1.0, 3.0$  yield closest results with the FE analysis with gradual transition from shallow to deep failure mechanism and a tendency of overestimation of the peak dimensionless at very deep depth ( $H/D \geq 40$ ). However, the DEM results for the case of  $\tan \phi_\mu = 0.5$  show considerably overestimation and do not show any transition from shallow to deep failure mechanism.

The obtained dimensionless peak displacements from DEM do not match the recommendation by Trautmann & O'Rourke (1983) and ASCE Guideline. They show a tendency to decrease as the embedment depth increases. The obtained normalized hyperbolic force-displacement relationship from DEM analysis match the recommendation by ASCE Guideline well for the cases of  $\tan \phi_\mu = 3.0$  (medium sand) and  $\tan \phi_\mu = 1.0, 3.0$  (dense sand).

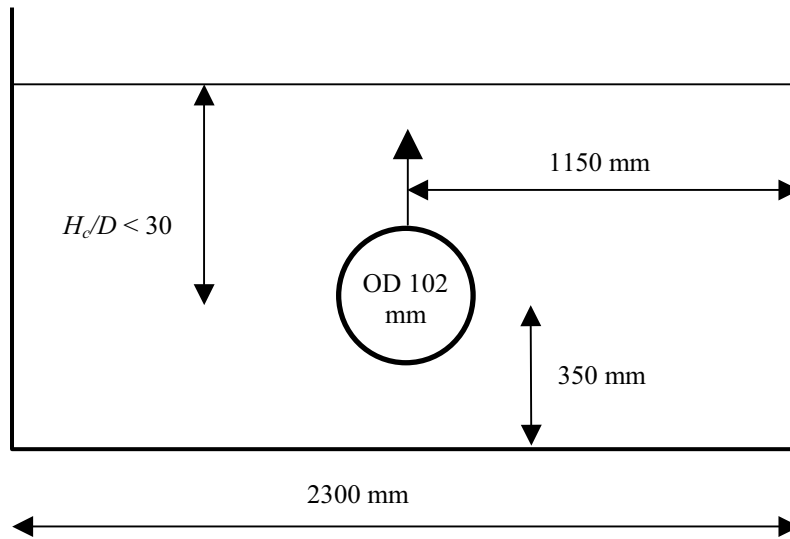
## **Chapter 6**

### **DEM analyses of pipe under upward loading**

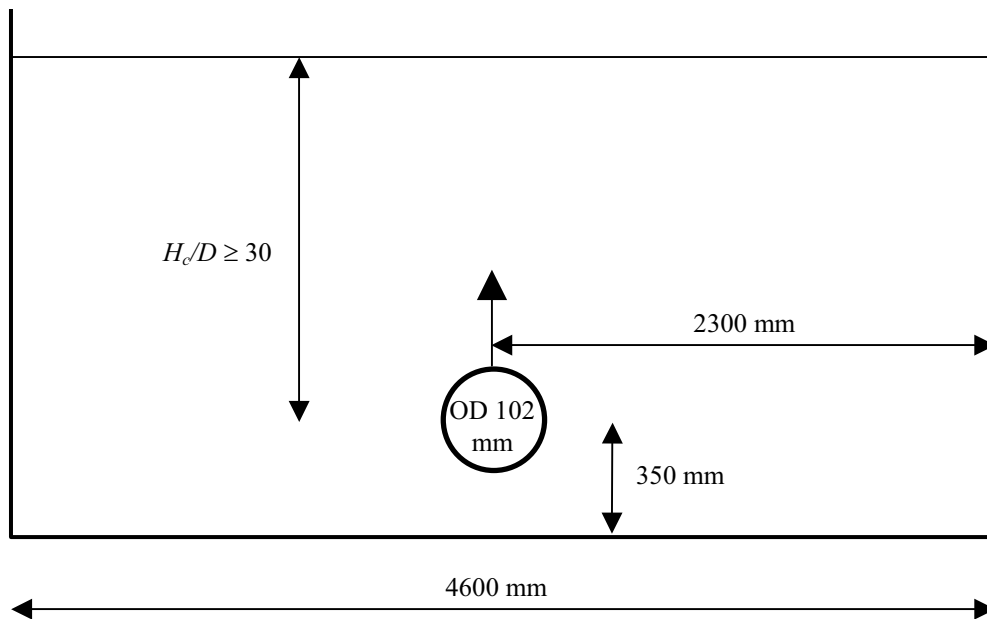
#### **6.1 Analysis procedure and program**

The geometry of the numerical model was based on the experimental study by Trautmann & O'Rourke (1983). For all cases of medium sand and the cases of dense sand with  $H_c/D < 30$ , all dimensions of the numerical model are the same as those of the experimental set-up except that the embedment depth was adjusted to the required value (Fig. 6-1a). For dense sand with  $H_c/D \geq 30$ , the width of the model was doubled to reduce the effect of boundary interference (Fig 6-1b). The analyses were performed for medium and dense sands.

The tank and pipe were modelled by series of planar wall. Sand was modelled as a collection of spherical particles and the size distribution follows normal distribution. The tank wall was assumed to be smooth; the tank model has a normal contact stiffness equal to that of the particles but has zero tangential contact stiffness and zero surface friction. The pipe has identical contact stiffness in both normal and tangential directions and equal to that of the particles. The pipe has its surface friction angle equal to half of the inter-particle friction angle of sand. The pipe was pulled vertically by imposing upward displacement to the pipe. The DEM analysis programme is shown in Tables 6-1 and 6-2.



(a) for all cases of medium sand and the cases of dense sand with  $H_c/D < 30$



(b) for dense sand with  $H_c/D \geq 30$

**Figure 6-1** Geometry of the DEM analyses of upward pipe loading

**Table 6-1** Summary of DEM analysis program and their input parameters: medium sand

Test No.	$H_c/D$	Case of $\tan \phi_\mu = 0.5$		Case of $\tan \phi_\mu = 1.0$		Case of $\tan \phi_\mu = 3.0$	
		$\tan \phi_\mu$	$k_{N,sand}$ (N/m)	$\tan \phi_\mu$	$k_{N,sand}$ (N/m)	$\tan \phi_\mu$	$k_{N,sand}$ (N/m)
UMHcD15	1.5	0.5	—	1.0	—	3.0	—
UMHcD4	4	0.5	—	1.0	—	3.0	—
UMHcD8	8	0.5	$1.5 \times 10^5$	1.0	$5.0 \times 10^4$	3.0	$3.0 \times 10^4$
UMHcD13	13	0.5	$1.0 \times 10^6$	1.0	$2.0 \times 10^5$	3.0	$1.0 \times 10^5$
UMHcD17	17	0.5	$2.4 \times 10^6$	1.0	$4.2 \times 10^5$	3.0	$1.7 \times 10^5$
UMHcD21	21	0.5	$5.3 \times 10^6$	1.0	$7.6 \times 10^5$	3.0	$2.6 \times 10^5$
UMHcD25	25	0.5	$1.0 \times 10^7$	1.0	$1.2 \times 10^6$	3.0	$3.8 \times 10^5$
UMHcD30	30	0.5	$2.1 \times 10^7$	1.0	$2.0 \times 10^6$	3.0	$5.7 \times 10^5$
UMHcD40	40	0.5	$6.1 \times 10^7$	1.0	$4.5 \times 10^6$	3.0	$1.1 \times 10^6$
UMHcD60	60	0.5	$2.9 \times 10^8$	1.0	$1.4 \times 10^7$	3.0	$2.6 \times 10^6$

Note: 1.  $k_{N,sand} = k_{T,sand} = k_{N,pipe} = k_{T,pipe} = k_{N,wall}$

2.  $k_{T,wall} = 0$

3.  $\phi_{pipe} = \phi_\mu/2$

4.  $\phi_{wall} = 0$

**Table 6-2** Summary of DEM analysis program and their input parameters: dense sand

Test No.	$H_c/D$	Case of $\tan \phi_\mu = 0.5$		Case of $\tan \phi_\mu = 1.0$		Case of $\tan \phi_\mu = 3.0$	
		$\tan \phi_\mu$	$k_{N,sand}$ (N/m)	$\tan \phi_\mu$	$k_{N,sand}$ (N/m)	$\tan \phi_\mu$	$k_{N,sand}$ (N/m)
UDHcD15	1.5	0.5	—	1.0	—	3.0	—
UDHcD4	4	0.5	$2.5 \times 10^4$	1.0	$1.5 \times 10^4$	3.0	$1.0 \times 10^4$
UDHcD8	8	0.5	$1.5 \times 10^5$	1.0	$7.0 \times 10^4$	3.0	$4.0 \times 10^4$
UDHcD13	13	0.5	$1.0 \times 10^6$	1.0	$2.5 \times 10^5$	3.0	$1.0 \times 10^5$
UDHcD17	17	0.5	$3.1 \times 10^6$	1.0	$5.2 \times 10^5$	3.0	$1.9 \times 10^5$
UDHcD21	21	0.5	$7.0 \times 10^6$	1.0	$9.4 \times 10^5$	3.0	$3.1 \times 10^5$
UDHcD25	25	0.5	$1.4 \times 10^7$	1.0	$1.5 \times 10^6$	3.0	$4.5 \times 10^5$
UDHcD30*	30	0.5	$2.7 \times 10^7$	1.0	$2.5 \times 10^6$	3.0	$6.7 \times 10^5$
UDHcD40*	40	0.5	$8.2 \times 10^7$	1.0	$5.6 \times 10^6$	3.0	$1.3 \times 10^6$
UDHcD60*	60	0.5	$3.9 \times 10^8$	1.0	$1.7 \times 10^7$	3.0	$3.0 \times 10^6$

Note: 1.  $k_{N,sand} = k_{T,sand} = k_{N,pipe} = k_{T,pipe} = k_{N,wall}$

2.  $k_{T,wall} = 0$

3.  $\phi_{pipe} = \phi_\mu/2$

4.  $\phi_{wall} = 0$

5. \* bigger tank

## 6.2 Input parameters for soil models

The input parameters for DEM models were determined as described in Section 4.5. The input parameters for all DEM analyses are presented in Tables 6-1 and 6-2. For the cases of  $1.5 \leq H_c/D \leq 13$ , the values of  $k_{N,sand}$  were derived from fitting peak forces from the DEM analysis with the pipe loading experiment (as described in Section 4.5.2). For the cases of  $17 \leq H_c/D \leq 60$ , the values of  $k_{N,sand}$  were derived from the obtained relationships from shallow depths as follow.

$$\left. \begin{aligned} k \text{ (N/m)} &= 6.714 \sigma_c'^{3.813} && \text{for } \tan \phi_\mu = 0.5 \\ k \text{ (N/m)} &= 41.072 \sigma_c'^{2.761} && \text{for } \tan \phi_\mu = 1.0 \\ k \text{ (N/m)} &= 112.859 \sigma_c'^{2.176} && \text{for } \tan \phi_\mu = 3.0 \end{aligned} \right\} \dots\dots\dots (6-1)$$

$\sigma_c'$  (kPa) = vertical effective stress at center of pipe

### 6.3 DEM analysis results of shallow embedment depths

In this section, the DEM results of shallow embedment depths ( $H_e/D \leq 13$ ) are presented and compared with the experimental results by Trautmann & O'Rourke (1983) to examine the accuracy of the DEM analysis. The results in terms of the hyperbolic force-displacement relationship, the peak forces, and the corresponding pipe displacement are summarized in Tables 6-3 and 6-4. The cases of very shallow depth ( $H_e/D = 1.5$  and 4) could not be modelled due to instability of model set-up, especially for medium sand.



Table 6-3 DEM analysis results: medium sand

$H/D$	$\tan \phi_\mu = 0.5$			$\tan \phi_\mu = 1.0$			$\tan \phi_\mu = 3.0$		
	Hyperbolic fit	$F_{v,peak}$ (kN)	$\delta_{v,peak}$ (mm)	Hyperbolic fit	$F_{v,peak}$ (kN)	$\delta_{v,peak}$ (mm)	Hyperbolic fit	$F_{v,peak}$ (kN)	$\delta_{v,peak}$ (mm)
1.5	—	—	—	—	—	—	—	—	—
4	—	—	—	—	—	—	—	—	—
8	—	8.2	21.7	—	7.1	45.1	—	7.2	47.9
13	—	18.9	62.0	—	19.3	61.0	—	18.2	69.3
17	$F_v = \frac{\delta_v}{0.0339\delta_v + 0.2019}$	26.6	54.7	$F_v = \frac{\delta_v}{0.0356\delta_v + 0.1829}$	25.3	46.6	$F_v = \frac{\delta_v}{0.0347\delta_v + 0.2562}$	26.0	68.1
21	$F_v = \frac{\delta_v}{0.0241\delta_v + 0.0948}$	37.4	35.9	$F_v = \frac{\delta_v}{0.0264\delta_v + 0.2964}$	34.1	101.3	$F_v = \frac{\delta_v}{0.0243\delta_v + 0.2874}$	37.1	108.3
25	$F_v = \frac{\delta_v}{0.0214\delta_v + 0.0449}$	42.1	19.1	$F_v = \frac{\delta_v}{0.0195\delta_v + 0.2013}$	46.2	93.8	$F_v = \frac{\delta_v}{0.0191\delta_v + 0.2597}$	47.2	124.5
30	$F_v = \frac{\delta_v}{0.0219\delta_v + 0.0679}$	41.2	28.6	$F_v = \frac{\delta_v}{0.0163\delta_v + 0.1590}$	55.1	86.0	$F_v = \frac{\delta_v}{0.0146\delta_v + 0.2098}$	61.8	132.7

40	—	—	—	$F_v = \frac{\delta_v}{0.0126\delta_v + 0.1130}$	71.2	78.2	$F_v = \frac{\delta_v}{0.0116\delta_v + 0.1596}$	77.4	120.9
60	—	—	—	$F_v = \frac{\delta_v}{0.0100\delta_v + 0.0273}$	89.6	23.5	$F_v = \frac{\delta_v}{0.0070\delta_v + 0.0811}$	128.4	102.9

Note:  $F_v$  is in kN/m length.

$\delta_v$  is in mm.

Table 6-4 DEM analysis results: dense sand

$H/D$	$\tan \phi_\mu = 0.5$			$\tan \phi_\mu = 1.0$			$\tan \phi_\mu = 3.0$		
	Hyperbolic fit	$F_{v,peak}$ (kN)	$\delta_{v,peak}$ (mm)	Hyperbolic fit	$F_{v,peak}$ (kN)	$\delta_{v,peak}$ (mm)	Hyperbolic fit	$F_{v,peak}$ (kN)	$\delta_{v,peak}$ (mm)
1.5	—	—	—	—	—	—	—	—	—
4	—	3.2	30.4	—	2.9	36.5	—	3.0	36.9
8	—	11.1	23.6	—	10.3	44.6	—	10.0	57.0
13	—	29.3	24.0	—	28.3	47.9	—	23.6	54.4
17	—	53.5	10.3	—	41.9	51.6	$F_v = \frac{\delta_v}{0.0237\delta_v + 0.2770}$	37.9	103.2
21	—	84.4	6.7	—	63.6	37.2	$F_v = \frac{\delta_v}{0.0156\delta_v + 0.1944}$	57.5	108.5
25	—	174.0	5.8	—	83.2	24.1	$F_v = \frac{\delta_v}{0.0105\delta_v + 0.1390}$	85.6	117.6
30	$F_v = \frac{\delta_v}{0.0040\delta_v + 0.0243}$	225.2	54.8	—	107.3	60.0	—	113.4	80.1

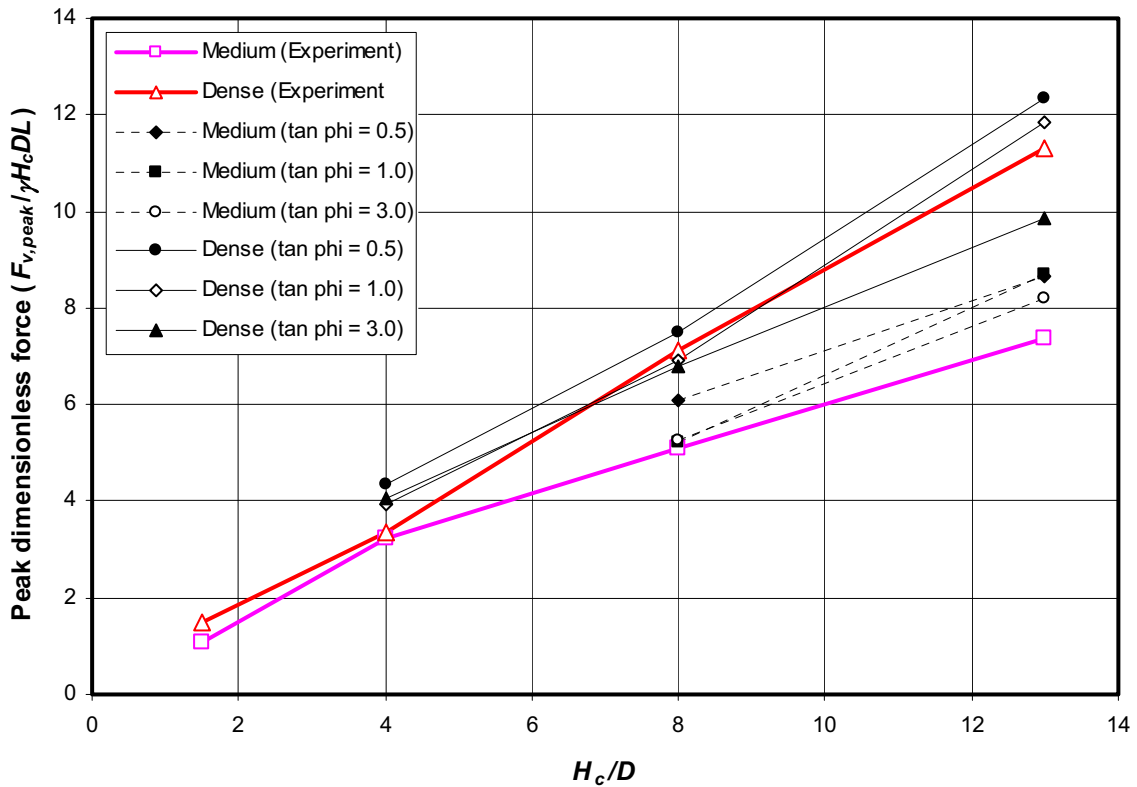
40	—	498.9	6.5	—	200.1	29.0	—	169.6	76.8
60	—	1685.2	4.1	—	330.5	22.9	—	303.3	50.8

Note:  $F_v$  is in kN/m length.

$\delta_v$  is in mm.

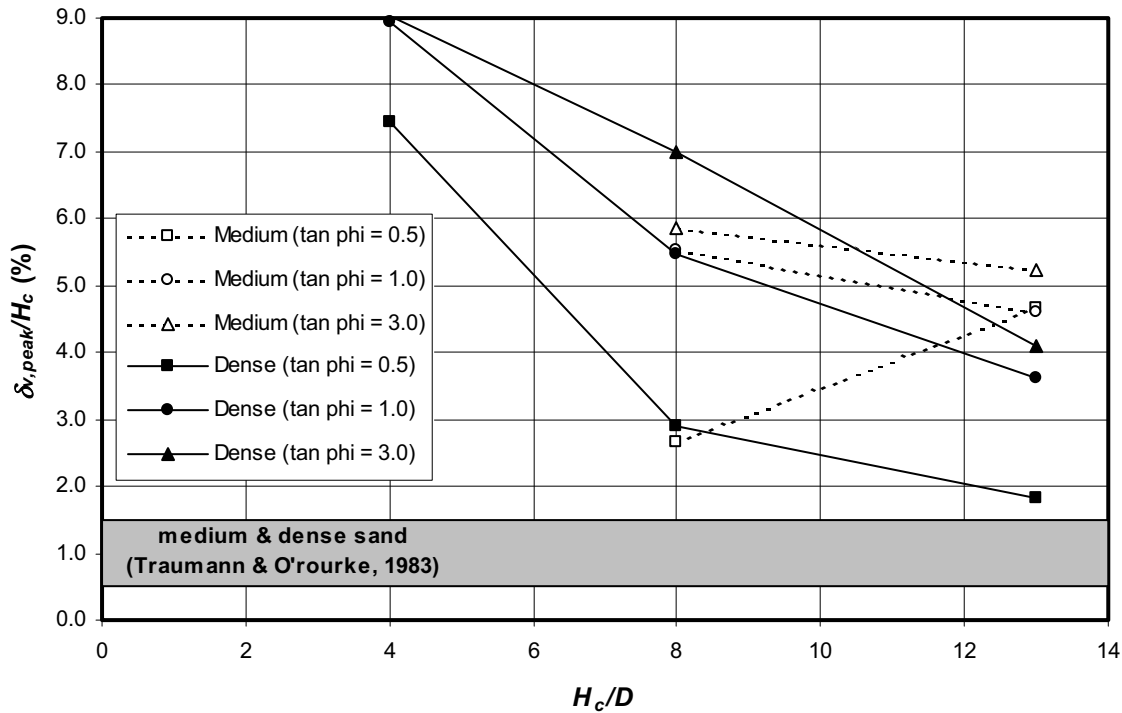
The comparison of the force-displacement curves from the DEM analyses with the experimental results is shown in Appendix B. The peak forces from DEM results were forced to match the experimental results by adjusting the values of contact stiffness by trial & error. The force-displacement curves from DEM show little difference from experimental results and the match is best for the case of  $\tan \phi_\mu = 0.5$ .

Fig. 6-2 presents the peak dimensionless forces plotted against  $H_c/D$ . The results show that the DEM analysis can simulate the upward pipe loading behavior reasonably well.



**Figure 6-2** Plots of peak dimensionless force against  $H_c/D$

Fig 6-3 shows the normalized peak displacement (pipe displacement at the peak force normalized by embedment depth) at different  $H_c/D$ s. The typical values found in the experiments by Trautmann & O'Rourke (1983) are also included in the figure. The dimensionless peak displacements from the DEM analysis are much larger than the experimental data. The dimensionless peak displacements show a tendency to decrease with  $H_c/D$ , which is opposite to the experimental results by Murrey & Geddes (1987) and Dickin (1994).



**Figure 6-3** Plots of dimensionless peak pipe displacement against  $H_c/D$

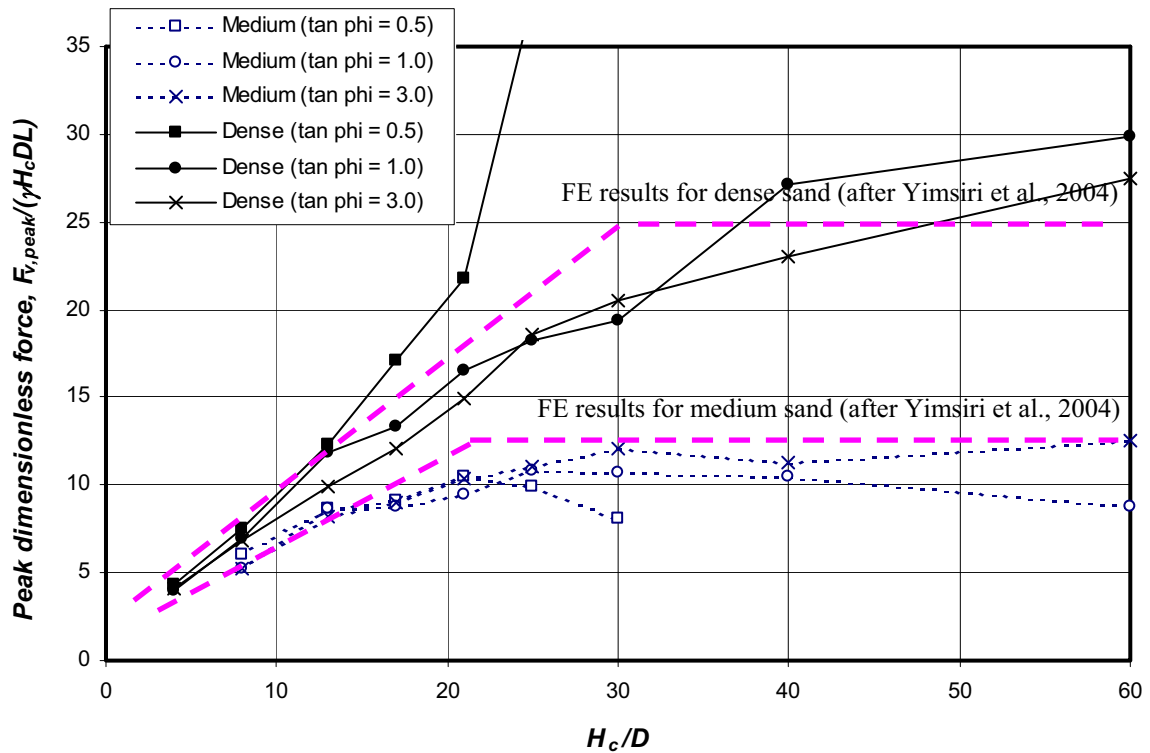
#### 6.4 DEM analysis results of deep embedment depths

The DEM analyses were performed for deep embedment depth cases for medium and dense sands. The computed hyperbolic force-displacement relationships, the peak forces, and the corresponding pipe displacements are summarized in Tables 6-3 and 6-4.

The force-displacement curves from the DEM analysis are compared with those from the previous FE analysis as shown in Appendix B. In case of medium sand, the results from DEM at all embedment depths are consistent with the FE results, except for the case of  $\tan \phi_\mu = 0.5$  at very large depth ( $H_c/D \geq 40$ ). In case of dense sand, the force-displacement curves from DEM analysis show more different trend from those of FE analysis when the embedment depth increases. The results from the case of  $\tan \phi_\mu = 0.5$  (highest contact stiffness used) show stiffest behavior, whereas the results from the case of  $\tan \phi_\mu = 3.0$  (lowest contact stiffness used) show softest behaviour which is more consistent to the FE results. When  $H_c/D \geq 17$ , the DEM results for the case of  $\tan \phi_\mu = 0.5$  become considerably different from the FE analysis by showing considerably larger peak force.

Fig. 6-4 presents the peak dimensionless force plotted against  $H_c/D$  of up to 60. For medium sand, the peak dimensionless forces from the cases of  $\tan \phi_\mu = 1.0$  and 3.0 yield

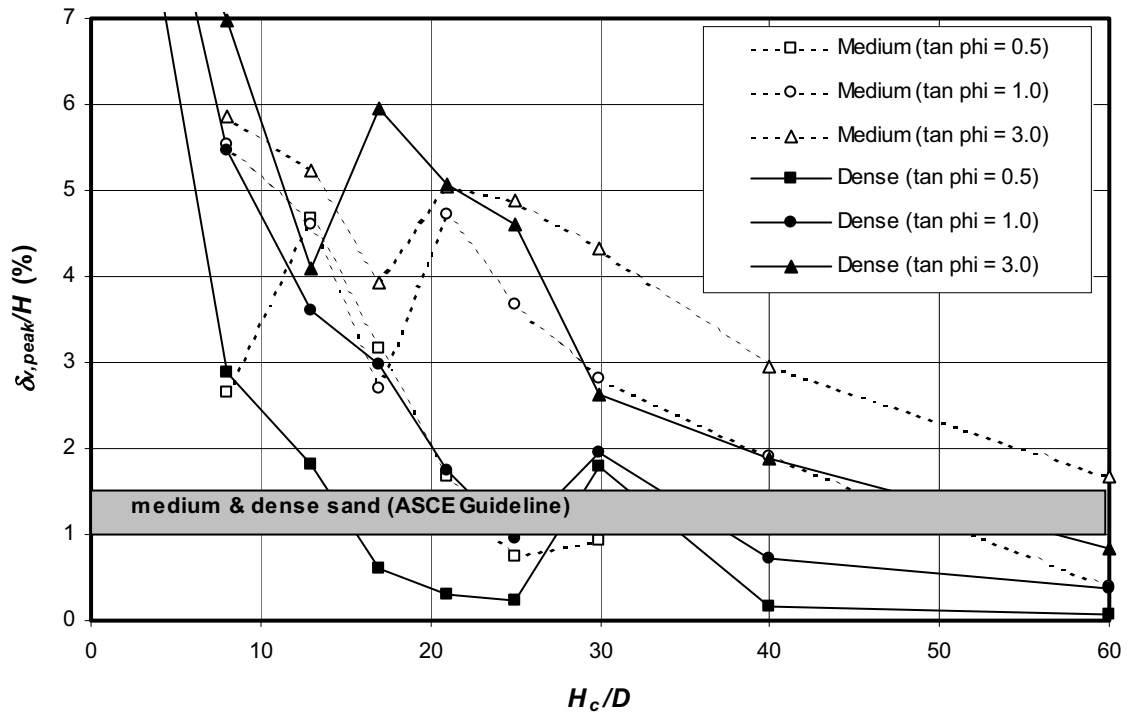
consistent results with the FE results for the whole range of the  $H_c/D$  of interest ( $H_c/D = 1.5$  to 60), whereas the results from the case of  $\tan \phi_\mu = 0.5$  show a tendency to give smaller values at deep depth. The peak dimensionless forces show deep failure mechanism by increasing approximately linearly with  $H_c/D$  at shallow depths and reaching their critical dimensionless force of 13 at the critical embedment depth of 25. This is consistent with the results from the FE analysis. For dense sand, the peak dimensionless forces from DEM are consistent with the FE analysis only for the case of  $\tan \phi_\mu = 1.0, 3.0$  which show a tendency to give larger peak dimensionless force at very deep depth ( $H_c/D \geq 60$ ) and more gradual transition from shallow to deep failure mechanism. The case of  $\tan \phi_\mu = 0.5$  shows considerably higher peak dimensionless force and does not consistent with the rest of the data.



**Figure 6-4** Relationship between peak dimensionless force and  $H_c/D$

Fig 6-5 shows the dimensionless peak displacement plotted against  $H_c/D$ . The recommended values from ASCE Guideline are also included in the figure. The dimensionless peak displacement show a tendency to decrease as the  $H_c/D$  increases. The stiffest behavior (smallest dimensionless peak displacement) is from  $\tan \phi_\mu = 0.5$  which is consistent with the case of largest contact stiffness. The dimensionless peak displacements

from the DEM analyses are quite different from those recommended by ASCE Guideline. At shallow depth, the dimensionless peak displacements from DEM are larger than the ASCE Guideline; however, their values decrease with embedment depth and become smaller than the ASCE Guideline at large depth.



**Figure 6-5** Relationship between dimensionless peak displacement against  $H_c/D$

The normalized hyperbolic relationship between force and displacement under upward loading proposed by ASCE, as shown in Eq. (6-2), is compared with the DEM analysis results in Figs. 6-6 and 6-7 for medium and dense sands, respectively.

$$\frac{F_v}{F_{v,peak}} = \frac{\delta_v / \delta_{v,peak}}{0.07 + 0.93 \delta_v / \delta_{v,peak}} \quad \dots\dots\dots (6-2)$$

The relationship recommended by ASCE reasonably matches the DEM analysis results for the case of medium sand but forms an upper boundary for the case of dense sand.



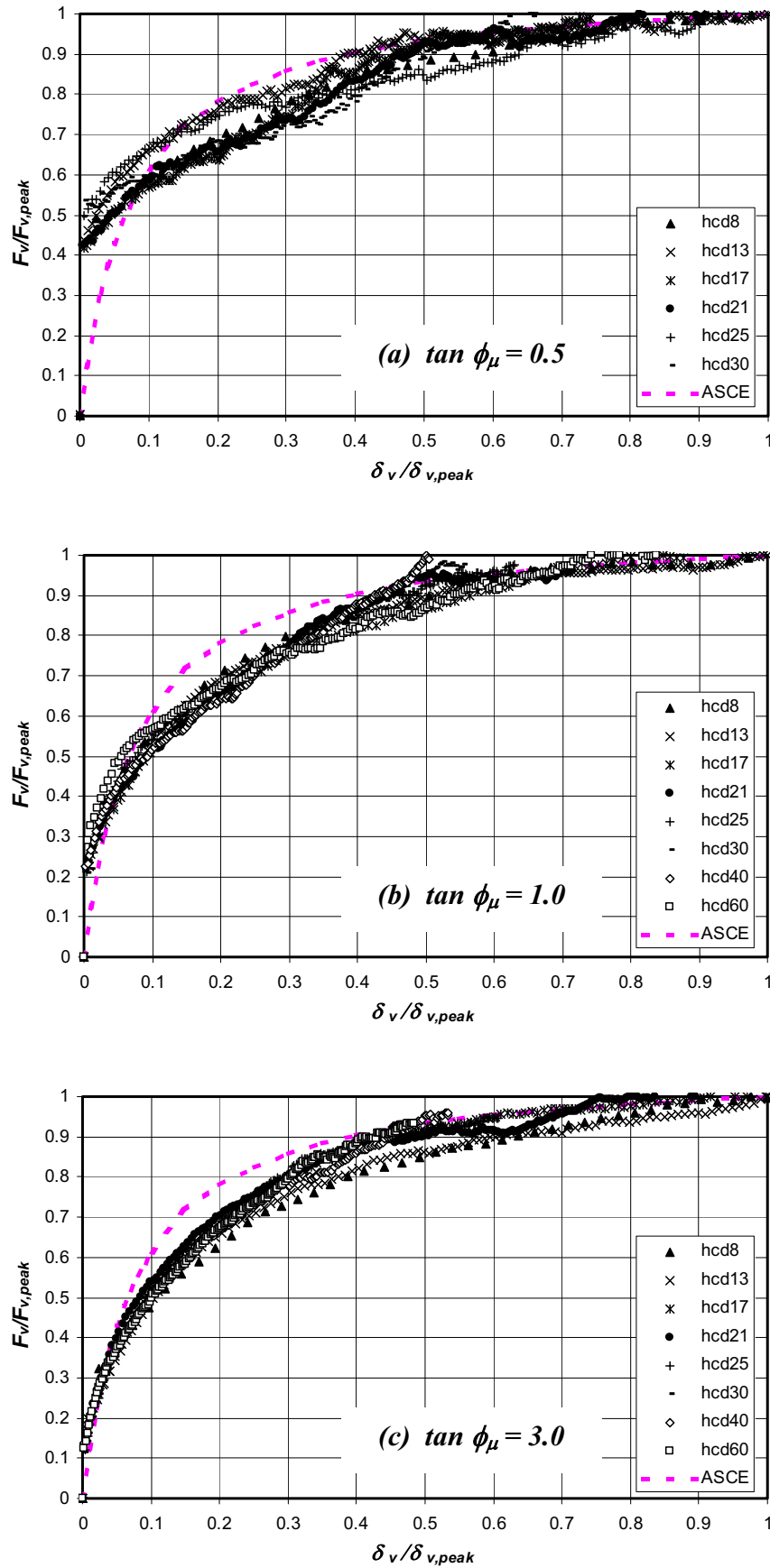


Figure 6-6 Normalized force-displacement relationship: medium sand

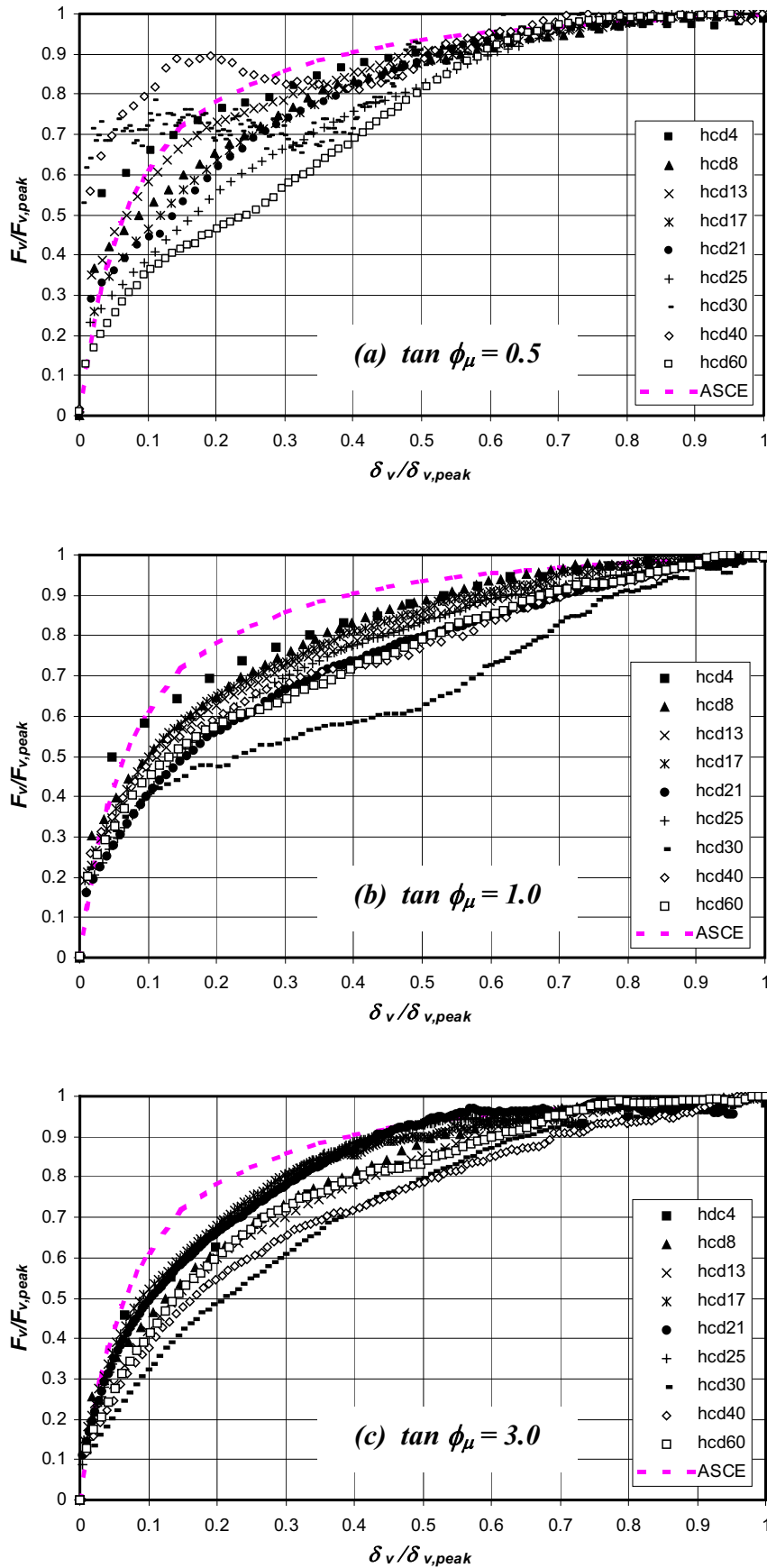


Figure 6-7 Normalized force-displacement relationship: dense sand

## 6.5 Summary

The DEM analysis is used to simulate the pipeline-soil interaction under lateral loading at deep embedment condition by using the input parameters from the calibration of the analysis with the experimental results at shallow depth. At shallow depth, the force-displacement relationships from the DEM analysis show that the results for medium sand match the experimental data well for all cases of  $\tan \phi_\mu$ , whereas the results for dense sand match the experimental data well only for the cases of  $\tan \phi_\mu = 0.5$  and  $1.0$  with too soft behavior for the case of  $\tan \phi_\mu = 3.0$ . At deep depth, the DEM results of medium sand in term of the peak dimensionless force and force-displacement relationship match the FE results well for all cases. The DEM results for medium sand show deep failure mechanism with the critical embedment depth of 25 and the critical peak dimensionless force of 13, which is consistent with the FE results. For dense sand, the DEM results from the cases of  $\tan \phi_\mu = 1.0, 3.0$  yield closest results with the FE analysis with gradual transition from shallow to deep failure mechanism and a tendency of overestimation of the peak dimensionless at very deep depth ( $H/D \geq 60$ ). However, the DEM results for the case of  $\tan \phi_\mu = 0.5$  shows considerably overestimation and do not show any transition from shallow to deep failure mechanism.

The obtained dimensionless peak displacements from DEM do not match the recommendation by Trautmann & O'Rourke (1983) and ASCE Guideline. They show a tendency to decrease as the embedment depth increases. The obtained normalized hyperbolic force-displacement relationship from DEM analysis match the recommendation by ASCE Guideline well for the cases of medium sand and form an upper boundary for the case of dense sand.

## Chapter 7

### Discussion and conclusions

#### 7.1 Selection of input parameters

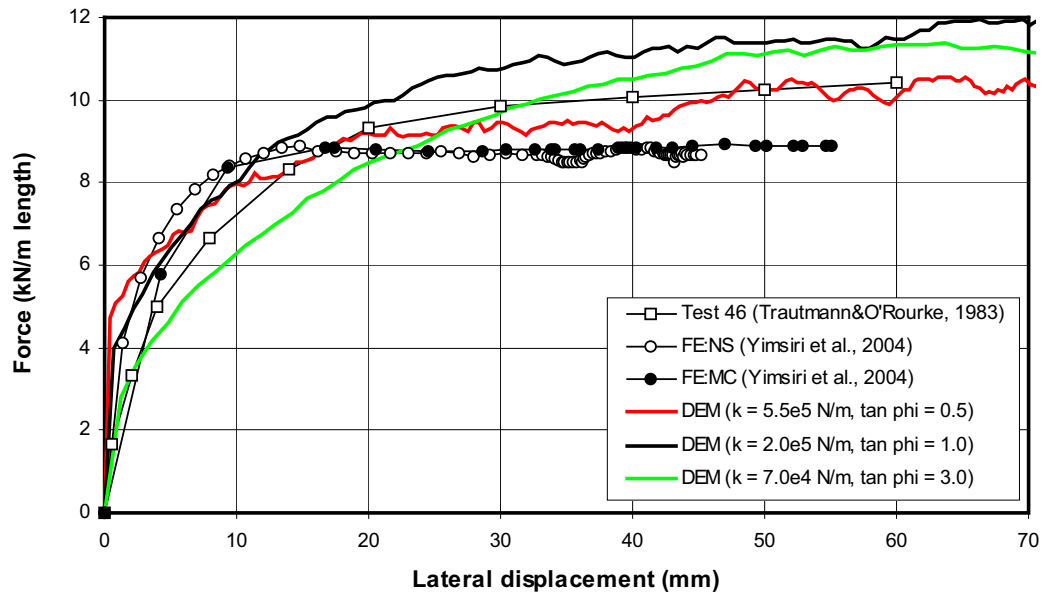
There may be some arguments about the determination of the input parameters and their validity in the present DEM analysis because some parameters may seem to be unrealistic. The use of a high value of the inter-particle friction angle is due to the use of spherical particles which allows excessive particle rolling (e.g. Thomas & Bray, 1999). The obtained power value from triaxial simulation is 0.41, which is within the normal value for soil, which is approximately 0.5 (Hardin & Black, 1966). This is due to the fact that the DEM analysis matches both stiffness and strength from triaxial test. It was shown that the triaxial stiffness (Young's modulus) has a relationship with confining stress in the same manner as the contact stiffness does (Chang & Liao, 1994). However, the obtained power value from pipe loading simulation is much greater than the normal value for soil; this may be due to the complexity of mode of shearing in pipe loading problem. The peak force values employed for fitting is governed by complex deformation at relatively large strain; however, the contact stiffness is the behavior at very small strain.

It is also noted that various combinations of the contact stiffness and  $\tan \phi_\mu$  can yield similar peak forces. With larger  $\tan \phi_\mu$ , the required contact stiffness is lower. It is interesting to find that the contact stiffness affects the strength (peak force) for this problem, which is not the case for triaxial problem where the contact stiffness affects only modulus, not strength (see Appendix G). This may also be due to the more complex mode of shearing in pipe loading problem.

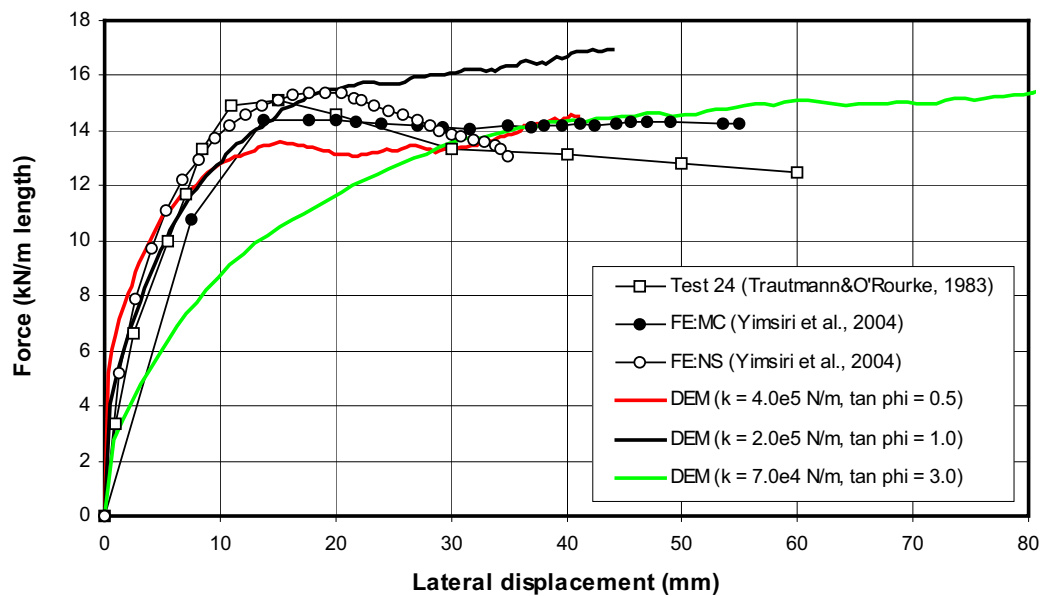
#### 7.2 Pipeline behavior under lateral loading

Fig. 7-1 shows that, at shallow depth, the force-displacement relationships from the DEM analysis for medium sand match the experimental data well for all cases of  $\tan \phi_\mu$ , whereas the results for dense sand match the experimental data well only for the cases of  $\tan \phi_\mu = 0.5$  and  $1.0$  with too soft behavior for the case of  $\tan \phi_\mu = 3.0$ . Fig. 7-2 shows that, at deep depth, the force-displacement relationships from medium sand still match the FE results well for all cases of  $\tan \phi_\mu$ , whereas the results for dense sand show that the case of  $\tan \phi_\mu =$

3.0 yield closest results with the FE analysis and a tendency of overestimation of the peak dimensionless force for the case of  $\tan \phi_\mu = 0.5$ .

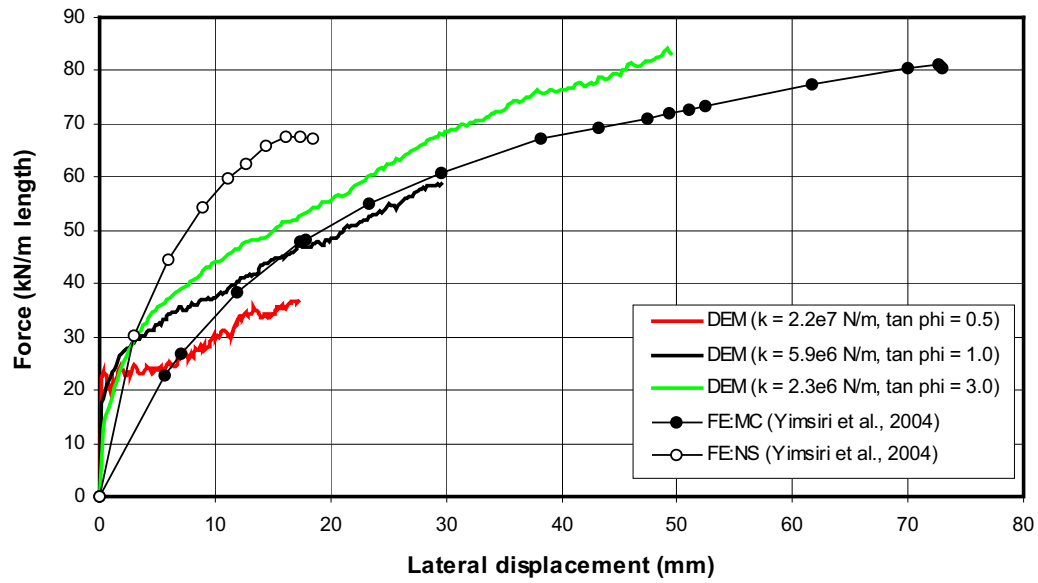
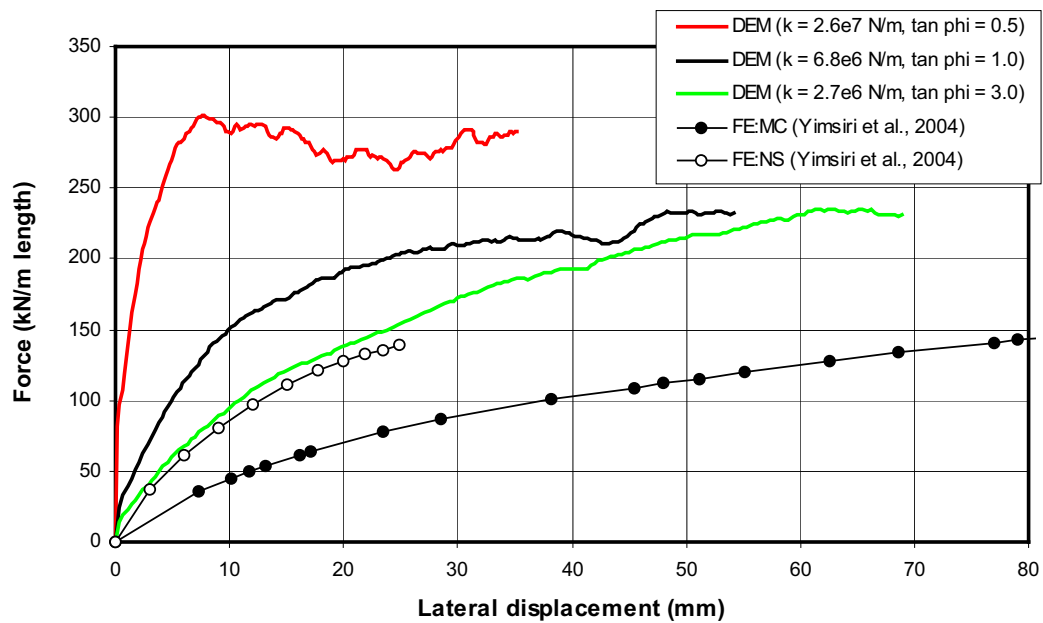


(a)  $H/D = 6$ , Medium sand



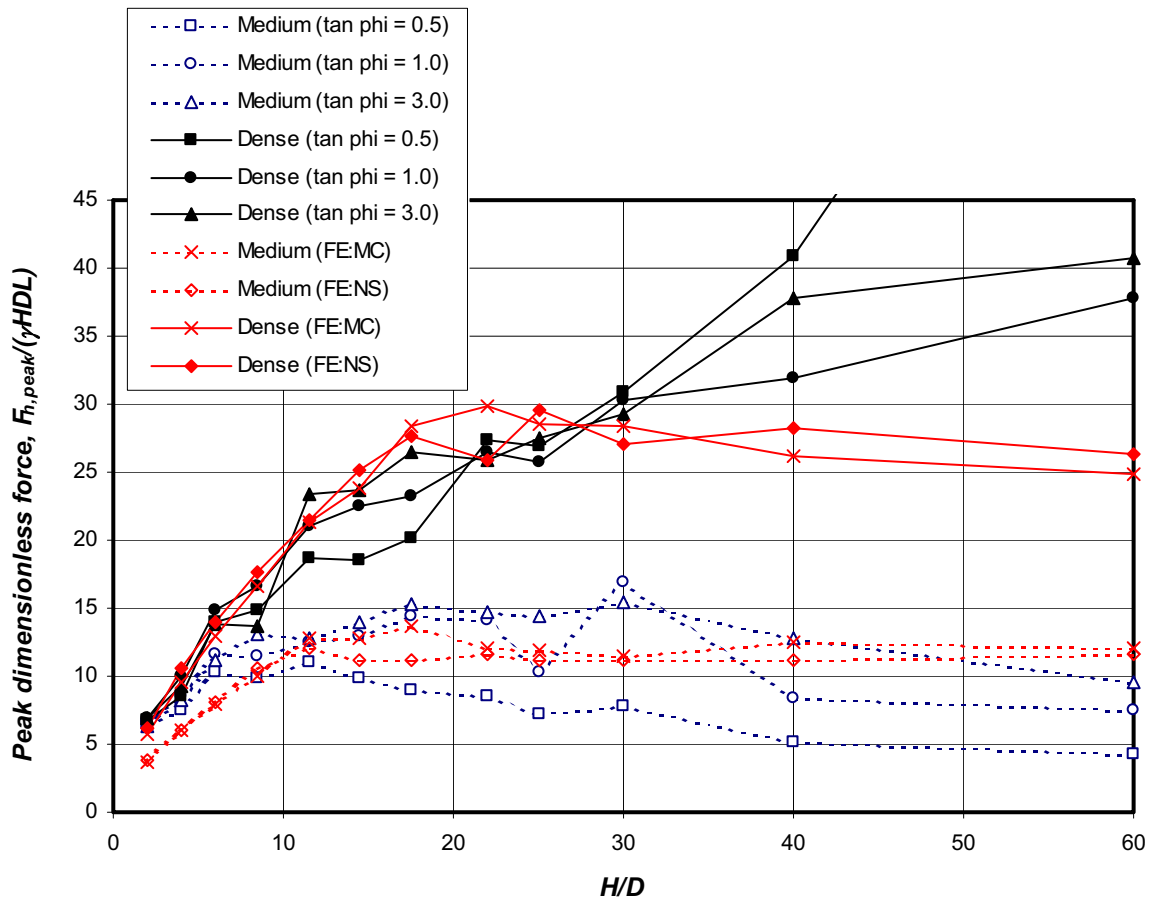
(b)  $H/D = 6$ , Dense sand

**Figure 7-1** Force-displacement relationships at shallow embedment depth of lateral pipe loading

(a)  $H/D = 40$ , Medium sand(b)  $H/D = 40$ , Dense sand

**Figure 7-2** Force-displacement relationships at deep embedment depth of lateral pipe loading

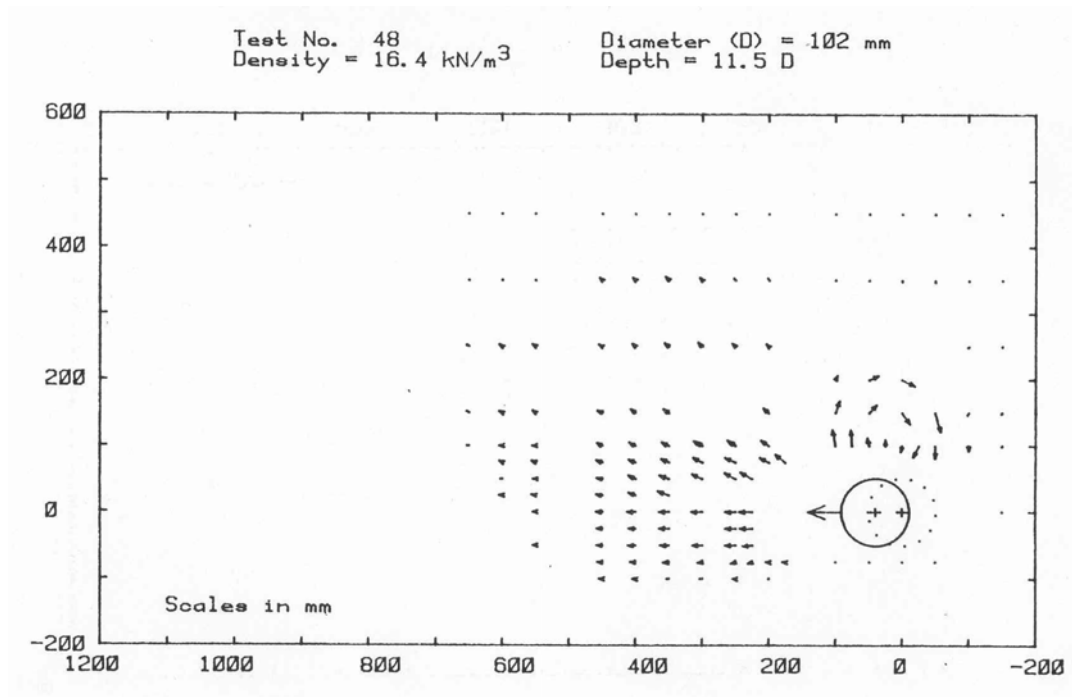
The comparison of peak dimensionless forces from the DEM and FEM analyses is shown in Fig. 7-3. For medium sand, the DEM results match the FE results well for all cases. The DEM results for medium sand show deep failure mechanism with the critical embedment depth of 12 and the critical peak dimensionless force of 13, which is consistent with the FE results. For dense sand, the DEM results from the cases of  $\tan \phi_\mu = 1.0, 3.0$  yield closest results with the FE analysis with gradual transition from shallow to deep failure mechanism and a tendency of overestimation of the peak dimensionless at very deep depth ( $H/D \geq 40$ ). However, the DEM results for the case of  $\tan \phi_\mu = 0.5$  show considerably overestimation and do not show any transition from shallow to deep failure mechanism.



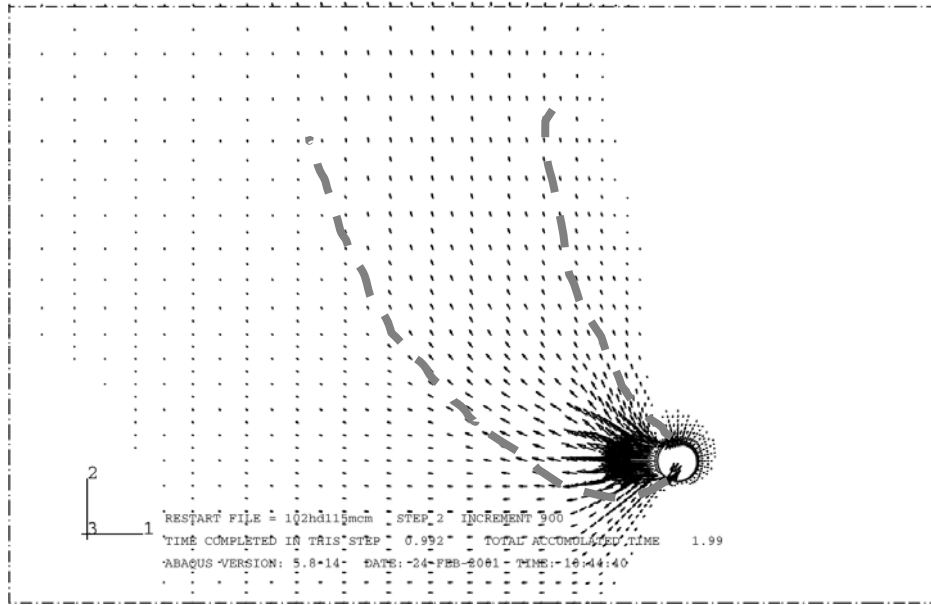
**Figure 7-3** Comparison of peak dimensionless force of lateral pipe loading from DEM and FEM

Fig. 7-4 shows that displacement pattern from the experimental tank test of medium sand at  $H/D = 11.5$ , which can be compared to those from FEM and DEM ( $\tan \phi_\mu = 3.0$ ) analyses shown in Fig. 7-5. Similarly, Fig. 7-6 shows that displacement pattern from the experimental tank test of dense sand at  $H/D = 11.5$ , which can be compared to those from FEM and DEM ( $\tan \phi_\mu = 3.0$ ) analyses shown in Fig. 7-7. It can be seen that the results from DEM better represent the displacement of sand around the pipe; however, the results from DEM involve larger soil mass compared with FEM, especially for dense sand. Fig. 7-8 shows the displacement pattern from DEM analysis at deep embedment depth. The results from medium sand show deep failure mechanism whereby the upward soil deformation does not extend to the ground surface. However, the results from dense sand still show shallow failure mechanism by which the ground surface still shows upward soil deformation. This is consistent with the peak dimensionless force data in Fig. 7-3, which shows that, at  $H/D = 30$ , the medium sand case has well reached deep failure mechanism, whereas the dense sand case is still in the transition state. The extent of soil mass involved in the deformation pattern increases as the contact stiffness used increases (or the inter-particle friction angle decreases). This makes the extent of soil deformation of dense sand at deep depth reaches the model boundary for the case of  $\tan \phi_\mu = 0.5$  (although the width of the testing compartment was already doubled to avoid or lessen the interference of boundary walls). This is consistent with the peak dimensionless force data in Fig. 7-3, which shows that the results for the case of  $\tan \phi_\mu = 0.5$  does not show deep failure mechanism due probably to this boundary effect. Fig. 7-9 presents the contact force chain pattern from DEM analysis. The results also show that the force chain involves large soil mass especially for dense sand.

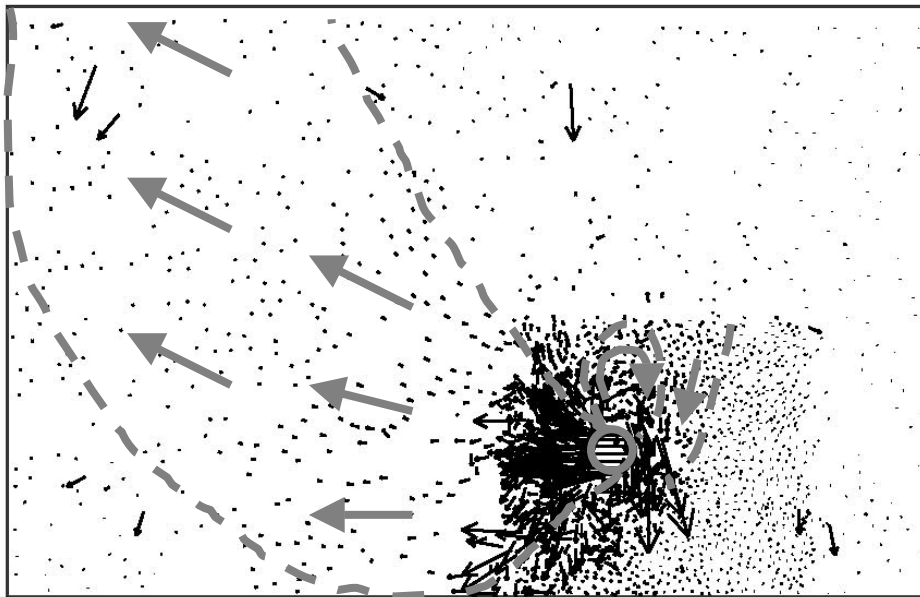




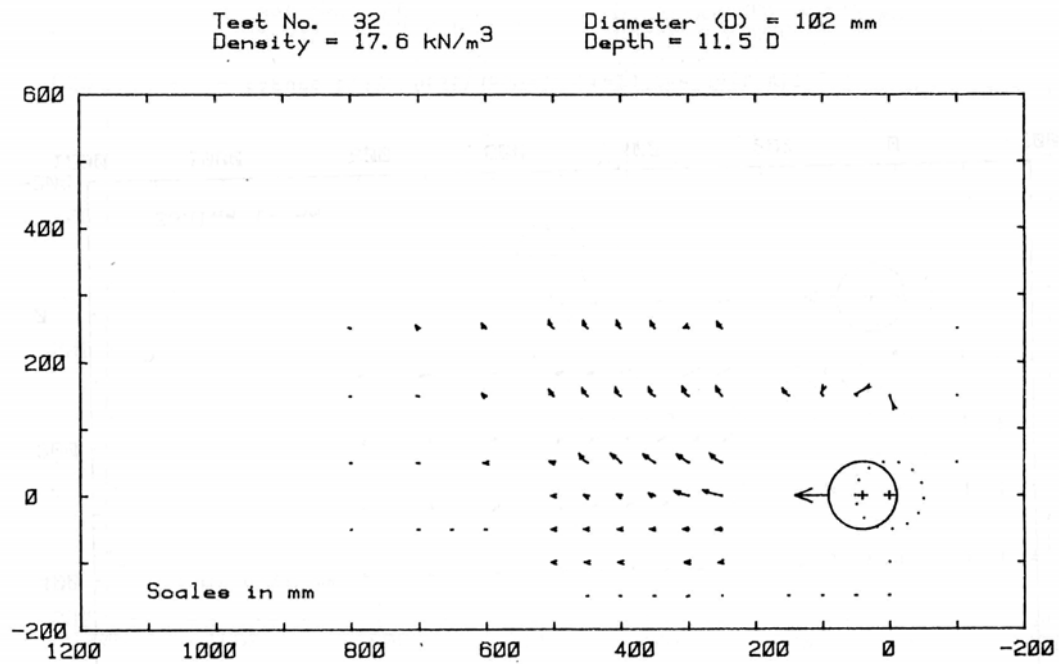
**Figure 7-4** Displacement pattern of lateral pipe loading from experimental tank test ( $H/D = 11.5$ , Medium sand) (Trautmann & O'Rourke, 1983)



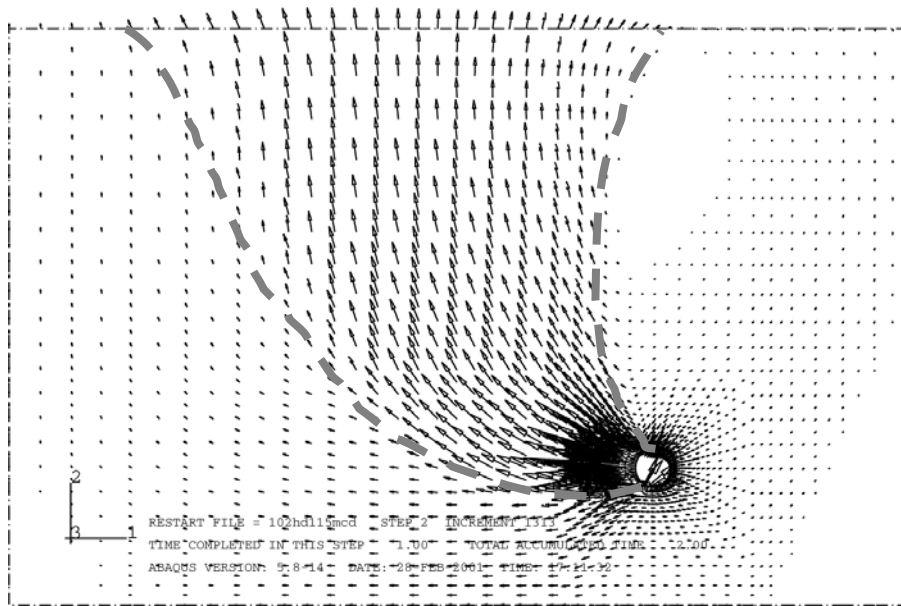
(a) FEM results

(b) DEM results,  $\tan \phi_{\mu} = 3.0$ 

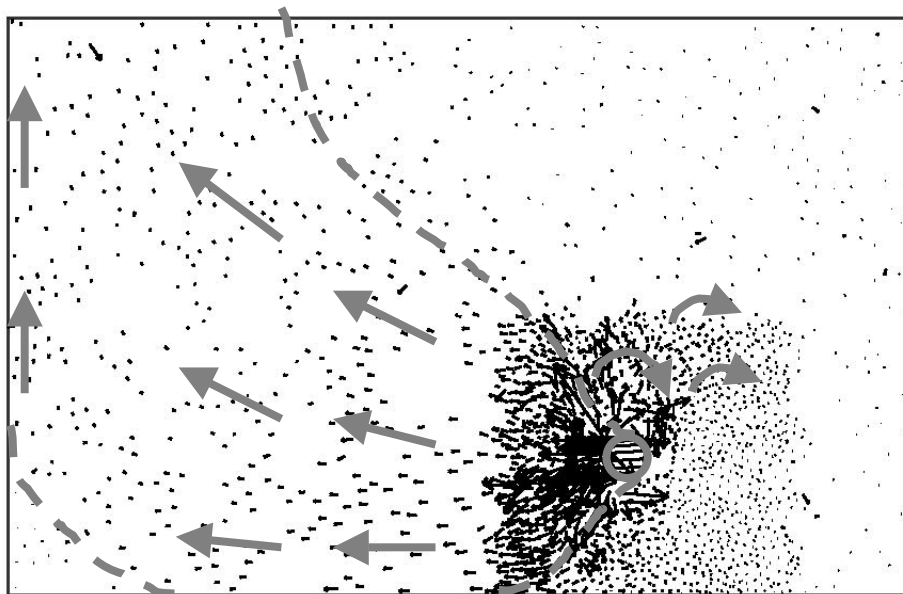
**Figure 7-5** Comparison of displacement pattern of lateral pipe loading from DEM and FEM ( $H/D = 11.5$ , Medium sand)



**Figure 7-6** Displacement pattern of lateral pipe loading from experimental tank test ( $H/D = 11.5$ , Dense sand) (Trautmann & O'Rourke, 1983)

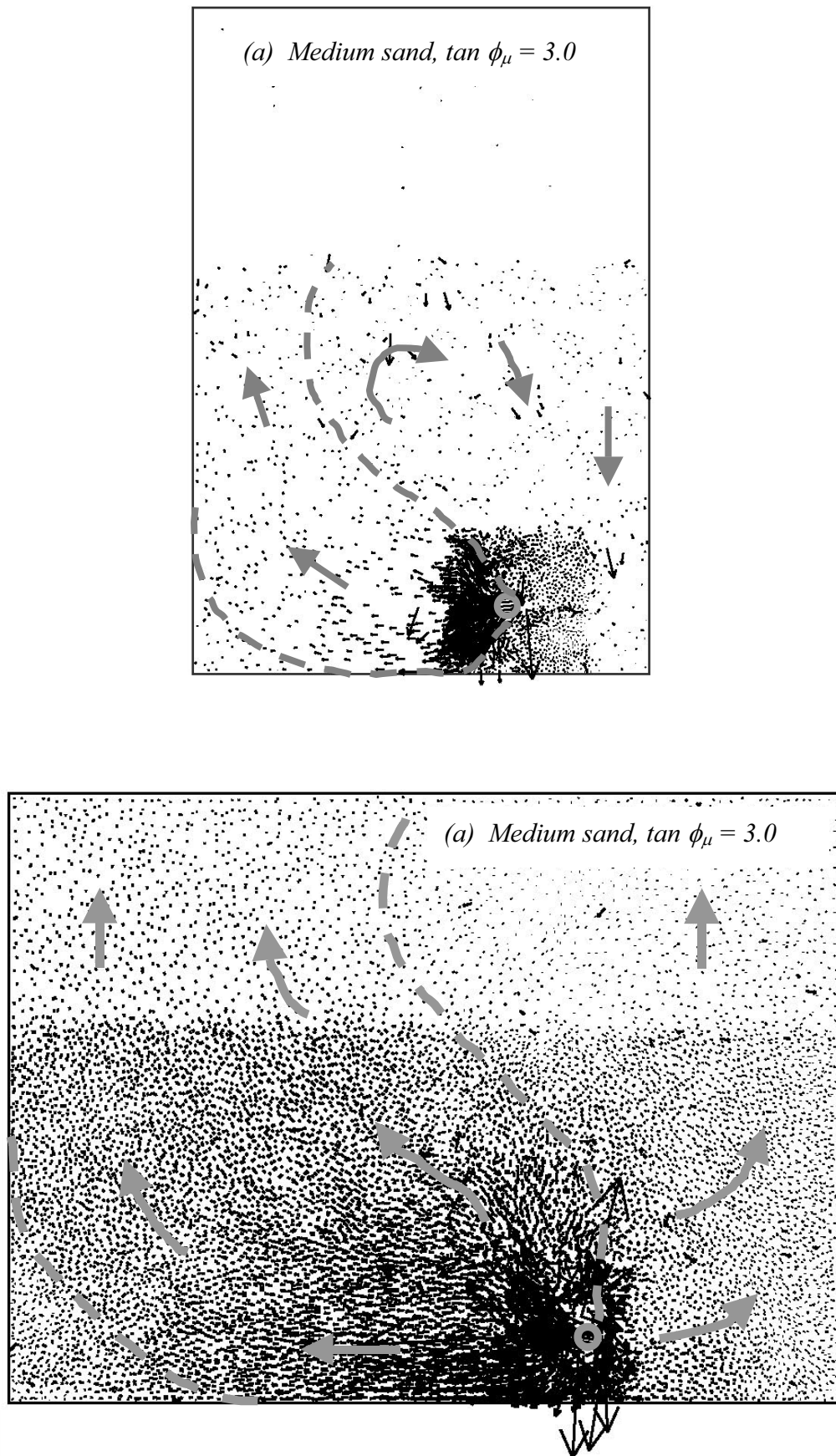


(a) FEM results

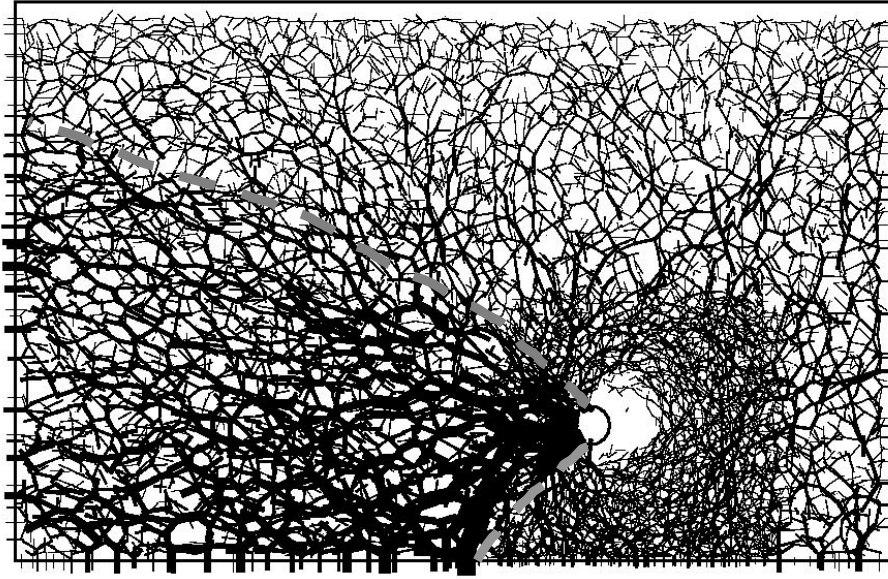


(b) DEM results,  $\tan \phi_\mu = 3.0$

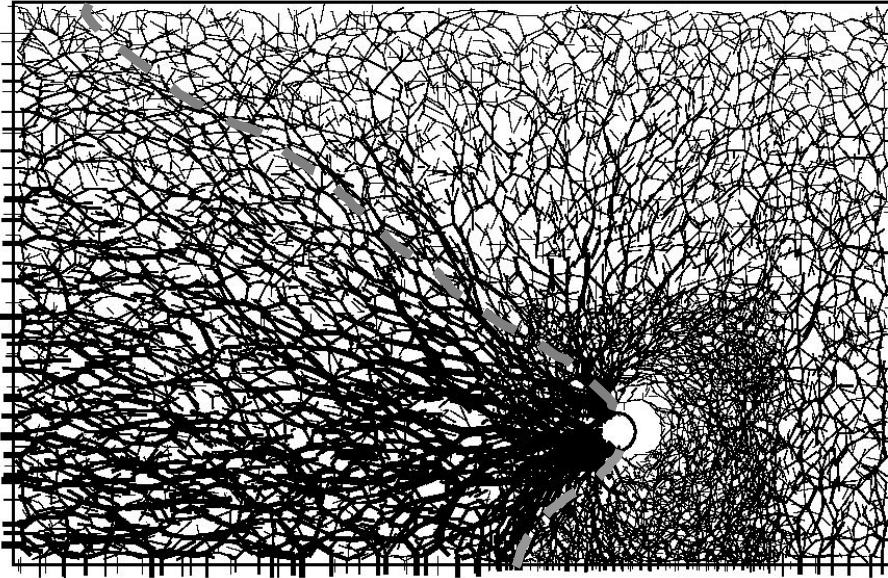
**Figure 7-7** Comparison of displacement pattern of lateral pipe loading from DEM and FEM ( $H/D = 11.5$ , Dense sand)



**Figure 7-8** Displacement pattern from DEM of lateral pipe loading ( $H/D = 30$ )



(a) Medium sand,  $\tan \phi_\mu = 3.0$

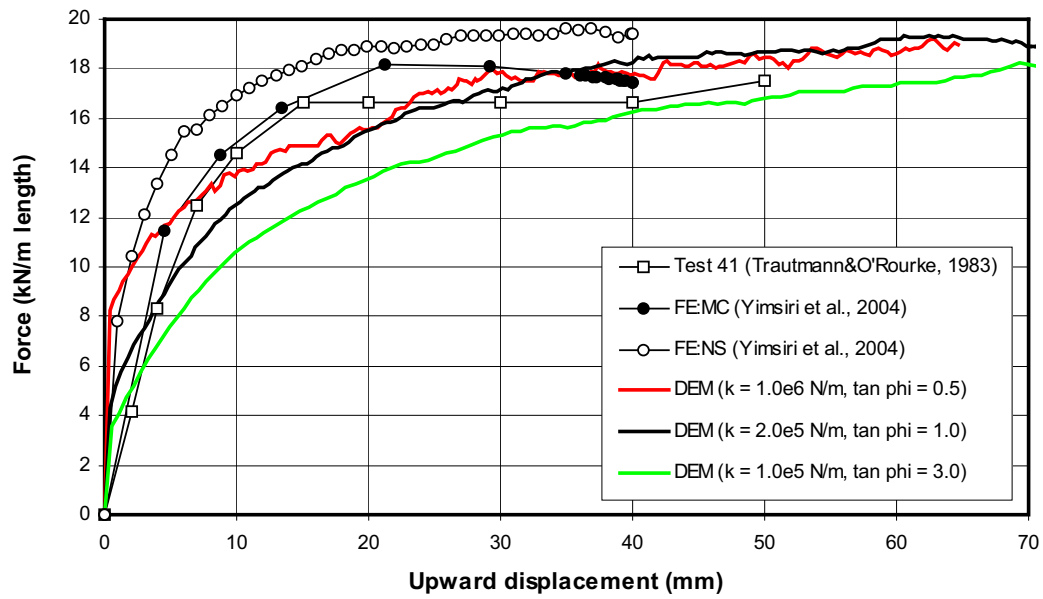
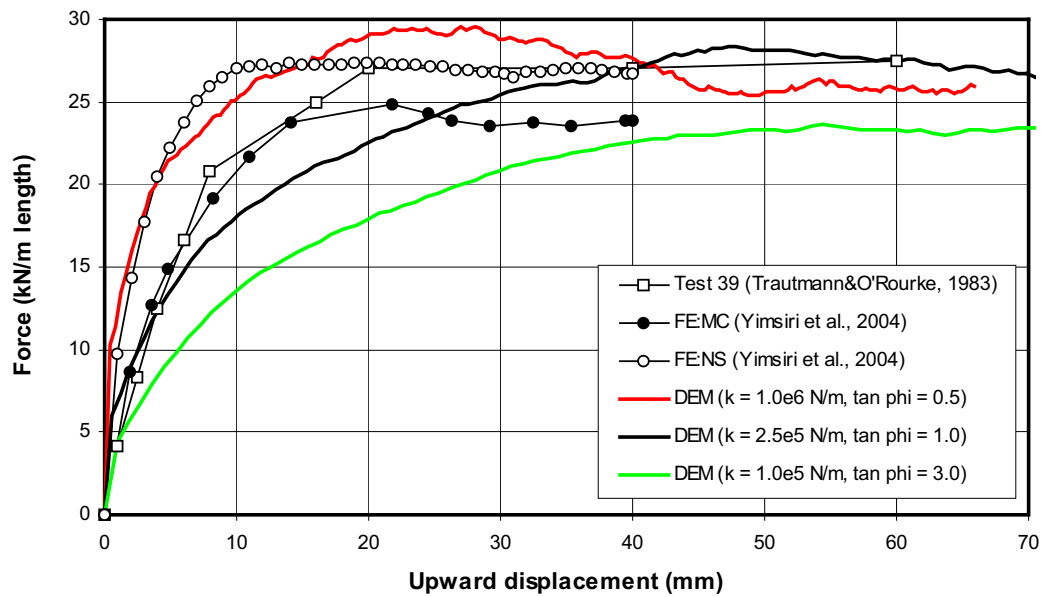


(b) Dense sand,  $\tan \phi_\mu = 3.0$

**Figure 7-9** Contact force chain pattern from DEM of lateral pipe loading ( $H/D = 11.5$ )

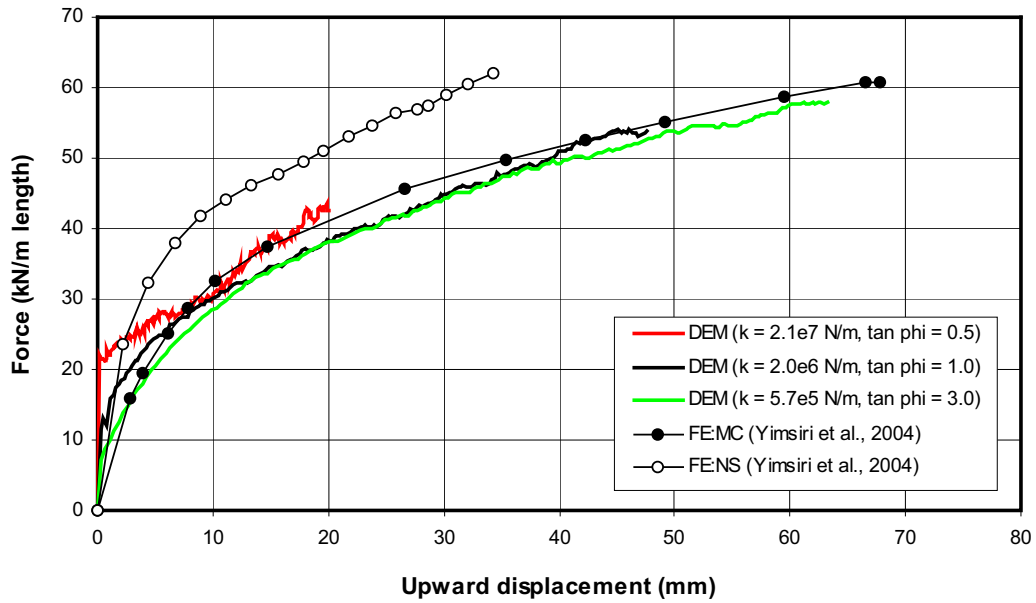
### **7.3 Pipeline behavior under upward loading**

Fig. 7-8 shows that, at shallow depth, the force-displacement relationships from the DEM analysis for medium sand match the experimental data well for all cases of  $\tan \phi_\mu$ , whereas the results for dense sand match the experimental data well only for the cases of  $\tan \phi_\mu = 0.5$  and  $1.0$  with too soft behavior for the case of  $\tan \phi_\mu = 3.0$ . Fig. 7-9 shows that, at deep depth, the force-displacement relationships from medium sand still match the FE results well for all cases of  $\tan \phi_\mu$ , whereas the results for dense sand show that the cases of  $\tan \phi_\mu = 1.0, 3.0$  yield closest results with the FE analysis and a tendency of overestimation of the peak dimensionless force for the case of  $\tan \phi_\mu = 0.5$ .

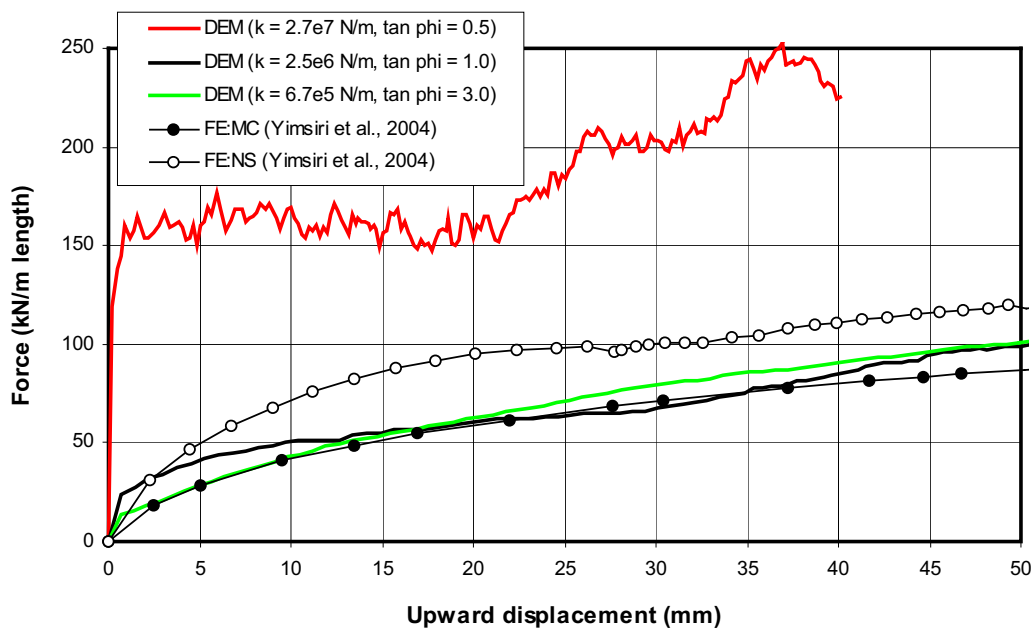
(a)  $H_c/D = 13$ , Medium sand(b)  $H_c/D = 13$ , Dense sand

**Figure 7-10** Force-displacement relationships at shallow embedment depth of upward pipe loading





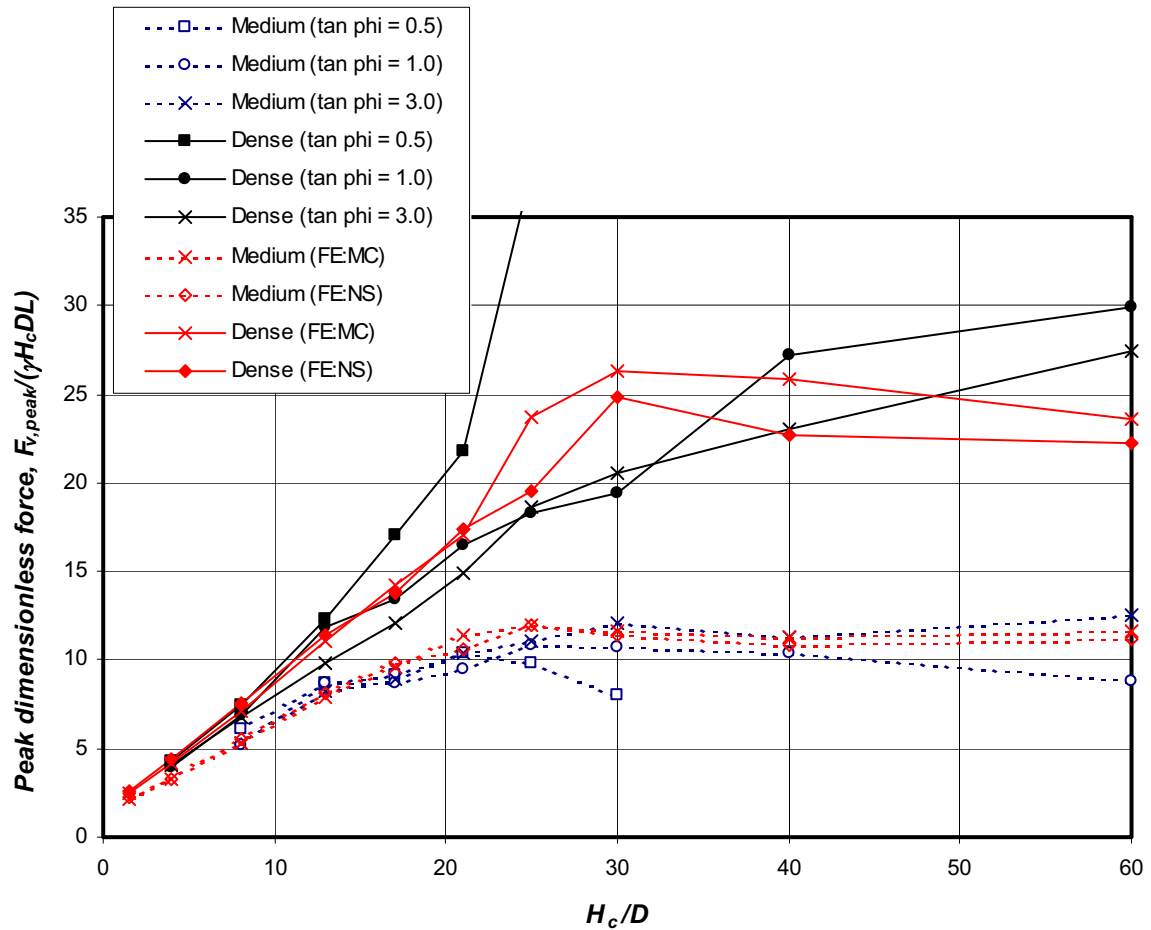
(a)  $H_c/D = 30$ , Medium sand



(b)  $H_c/D = 30$ , Dense sand

**Figure 7-11** Force-displacement relationships at deep embedment depth of upward pipe loading

The comparison of the peak dimensionless forces from the DEM and FEM analysis is shown in Fig. 7-10. For medium sand, the DEM results match the FE results well for all cases. The DEM results for medium sand show deep failure mechanism with the critical embedment depth of 25 and the critical peak dimensionless force of 13, which is consistent with the FE results. For dense sand, the DEM results from the cases of  $\tan \phi_\mu = 1.0, 3.0$  yield closest results with the FE analysis with gradual transition from shallow to deep failure mechanism and a tendency of overestimation of the peak dimensionless at very deep depth ( $H_c/D \geq 60$ ). However, the DEM results for the case of  $\tan \phi_\mu = 0.5$  show considerably overestimation and do not show any transition from shallow to deep failure mechanism.



**Figure 7-12** Comparison of peak dimensionless force of upward pipe loading from DEM and FEM