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執行期間： 年 月 日[illegible]

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本成果報告包括以下應繳交之附件：[illegible]

□赴國外出差或研習心得報告[illegible]

□赴大陸地區出差或研習心得[illegible]

□出席國際學術會議心得報告[illegible]

□國際合作研究計畫國外研究報[illegible]

執行單位：[illegible]

中華民國 年 月 日

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# 行政院國家科學委員會專題研究計畫成果報告

## 層狀土層混合模式分析法及其應用（二）

## Hybrid Modeling for Layered Soil-Structure Interaction and Its Applications

計畫編號：NSC 89-2211-E-002-056
執行期限：88 年 8 月 1 日至 89 年 7 月 31 日
主持人：陳正興　　國立臺灣大學土木工程學研究所

一、中文摘要

對於一個動態之土壤~結構互制作用問題，混合模式法（Hybrid modeling）乃以有限元素法模擬近域土壤及結構之多變性，而以連體力學法模擬遠域土壤之波動特性，為一非常經濟有效之分析方法。本研究擬利用混合模式分析電腦程式，分析探討高速鐵路行車所引起之基礎振動及各項震波阻隔措施之效益等問題。本研究有關理論推導及程式撰寫之部份已於去年完成，今年計畫內容則為各種應用情況之分析及探討，包含基礎阻抗問題之分析以及將其應用於高速鐵路行車所引起之振動問題分析與利用溝槽、地下牆等隔震措施之效益問題分析。

**關鍵詞**：土壤~結構互制、地盤振動反應、混合模式、隔離振動

Abstract

To solve the problem of soil-structure interaction in viscoelastic layered media, the method of hybrid modeling has both the advantages of finite element approach and continuum approach. It uses the method of finite element modeling to model the geometric-complexed near-field, and uses the indirect boundary element method to model the wave motions in the layered infinite far-field soil. This study is aiming to utilize the method of hybrid modeling to analyze the associated soil-structure interaction problems, such as the problems of foundation impedance, vibration response, and the effects of vibration isolation by using excavated trenches or buried walls. It is planned to complete all works in two years. The first year will concentrate on theoretical derivations and the associated programming. The second year will concentrate on its applications, including the studies of foundation impedance problem. Furthermore, it can be used to investigate the ground vibrations induced by the High Speed Trains and to evaluate the effects of counter measures of vibration isolation by using excavated trenches and buried walls.

**Keywords**: soil-structure interaction, ground vibration response, hybrid model

## 二、Scope

To accommodate the rapid growth of north-south bound transportation in Taiwan, it is going to construct a High Speed Rail (HSR) along the West Corridor of Taiwan. The total length of the route is 345km, in which the southern section will pass through the newly developed Tainan Science-based Industry Park (TSIP). This Park is developed in 1995 to promote the high-tech industry development in south Taiwan, including the specialty zones for micro-electronics and precision machinery, semi-conductors, and agricultural biotechnology. The high-tech factories and equipments inside TSIP are very sensitive to the ground vibrations during the process of production. Therefore, it is necessary to evaluate the levels of ground vibration that maybe induced by the operation of the planned High Speed Rail.

For this purpose, many studies including field test, analytical predictions and feasible measures for noise mitigation had been conducted.[1,2] This study will focus on analytical predictions for ground vibrations that maybe induced based on the design configurations of HSR to be constructed.

The route of HSR will pass through the east-side of the TSIP with a length of 3 km, approximately. According to the basic design of the HSR Project, the maximum design speed of train is 350 km/hr and the operational speed is 300 km/hr. The train supporting structures selected are the elevated viaduct structures. The viaducts adopted are pre-stressed concrete box-girders simply supported on reinforced concrete pier-caps and single-column piers which, in turn, are supported on pile foundations. According to the drawings of HSR alignment, the span lengths and pier heights within the TSIP section are various. However, most of them will have a span length of 30m and a pier height of 8m. For simplicity in numerical modeling, constant girder spans of 30m and pier heights of 8m will be assumed in current analysis.

The pier has a rectangular cross section of dimension 2.4m×3.2m. The foundation consists of a square pilecap and 5 large diameter cast-in-place reinforced concrete piles. Each pile has a diameter of 1.8m and a length around 50m. All pile heads are rigidly connected to a massive reinforced concrete pilecap with dimension 11m×11m×2.5m. The HSR trainset considered in current analysis is an Eurotrain set, consisting of 1 power-car + 1 booster + 1 end trailer + 16 intermediate trailers + 1 end trailer +1 power-car. The entire trainset has a length of 403 meters and a total weight of 880 metric tons. The wheel-axle load is 2×17.5 metric tons at both ends of each intermediate trailer, which has a typical length of 18.7m. For the power car and boosters, the wheel-axle loads are 2×19 metric tons with an interval of 11.46m.

The TSIP site is located on the well-known Chianan Plain where is a major agricultural area in Taiwan. The alluvial deposits in this region are very deep. According to the geological and geotechnical reports available, no bed rocks had been found up to a depth of 70 meters. The deposits near the ground surface can generally be characterized as inter-bedded layers of silty clays and clayey sands with various thickness. In this region, the ground water table is usually very high, almost close to the ground surface. For low amplitude vibration analysis as will be performed herein, a linear elastic analysis will be adequate from the view point that the strain level of soils subjected to the train excitation is very low in reality. Therefore, the low-strain wave velocities of soils will be adequate for this analysis. Both the P- and S-wave velocities measured at TSIP site will be adopted in current analysis.

## 四、Method of Analysis

The ground vibration induced by high speed train is a very complicated problem of train~structure~soil interaction. Current analysis is to use hybrid modeling[3,4,5] to predict the free-field ground response within the region of 400m from the centerline of the HSR route. Therefore, the analysis performed has to be able to model the effects of wave motions resulted from the passage of a 403m trainset along the HSR route, as shown in Fig. 1. In this figure, the observation points are allocated on a line perpendicular to the HSR alignment with distances equal to 100, 200, 300 and 400 meters, respectively. For every observation point, the vibrations contributed from all excitation sources generated by the train through the period of approaching to, passing over and running away with respect to the reference line of observation has to be taken into account. Therefore, the analysis performed has to be three-dimensional and the domain of analysis has to be large enough to include all effects.

To perform such a complicated analysis, method of substructuring is applied in current analysis. The whole system is divided into three substructures defined as follows:

(1) Substructure I: a trainset moving with a constant speed.

(2) Substructure II: the above-ground structure, including the viaducts, pod bearings, cap beams and piers. The model adopted in analysis will consist only a certain length of the bridge as shown in Fig. 2.

(3) Substructure III: the below-ground structure and soils. The model adopted in analysis will consist only a single pile foundation with its surrounding soils as shown in Fig. 3. The interface between Substructure II and III is selected to be at the pilecap top center.

The procedure of analysis includes the following four steps:

Step 1:

From Substructure III, calculate the dynamic impedance of a foundation and its surrounding soils by method of hybrid modeling, as well as the transfer functions between the responses at field points specified and the pilecap top center (master) node of the foundation.

Step 2:

From Substructure II, selected a certain length of the above-ground structure model, with foundation impedance added at each pier bottom. Calculate the natural frequencies and associated vibrational modes to be used for modal analysis in next step.

Step 3:

Based on Substructures I and II, method of modal superposition is applied to calculate the dynamic responses of the system for a train speed specified. To take account the effects of train~structure interaction, the train loads applied on the bridge girder are adjusted, by iteration procedures, to be consistent to the induced deformations in the underlain girder and overlying wheel sets. In this analysis, the reaction forces/moments at the master node of foundation (pier bottom) can be obtained for next step analysis.

Step 4:

Calculate the response at far-field observation points due to the excitation of a single foundation, by multiplication of the corresponding transfer functions with the reactions obtained at the master node of foundation calculated from Step 3. For a train pass over the elevated structures, the time history response at a specified field observation point can then be obtained through superposition for multiple excitations from all affected foundations, according to their locations and time shifts of a moving train.

## 四、Results

Based on the model established, the dynamic responses of the train, structure, foundation and soils can be calculated following the steps described previously. Case studies performed including:

(1) Pile foundation
(2) Caisson foundation
(3) Caisson foundation with friction isolation
(4) various span lengths on continuous foundation

All the analyses include cases for various train speeds ranged from 100 to 350 km/hr. Results of pile and caisson foundations show that the ground vibrations induced will be too large for the normal operation of TSIP and foundation isolation has no significant effects on reducing ground vibration, as shown in Fig. 4. However, the girder span length has significant effect to reducing the ground vibration. Due to the page limit, only the vertical response of item (5) is shown herein in Fig. 5. It can be seen that the 5m girder span on continuous foundation can reduce the ground vibration significantly.

## 五、Conclusions

(1). The method of analysis proposed herein can be used to evaluate the complicated train-structure-soil interaction problem. Step-wise procedure of analysis based on method of substructuring is very effective and efficient in solving the problem.

(2). For pile and caisson foundations, the predicted vibration at considerable distance from the train is still quite high.

(3). Use of 5m girder span on continuous foundation can reduce the ground vibration significantly. It is more effective than other methods such as trenching and buried walls..

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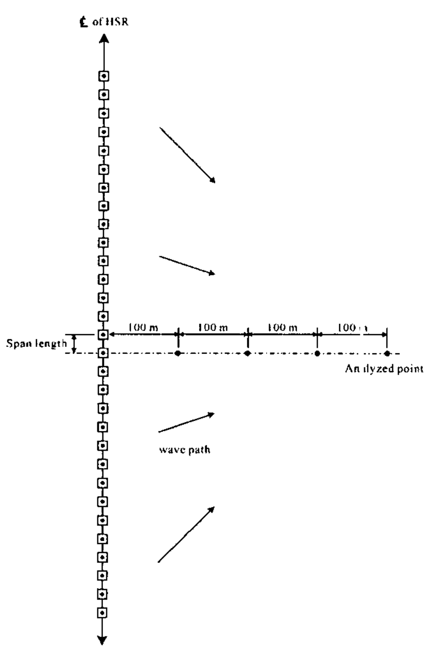


Fig. 1 Layout of observational field points

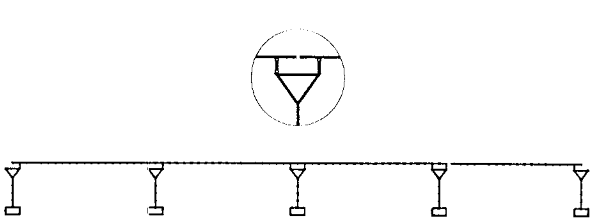


Fig. 2 Structural model

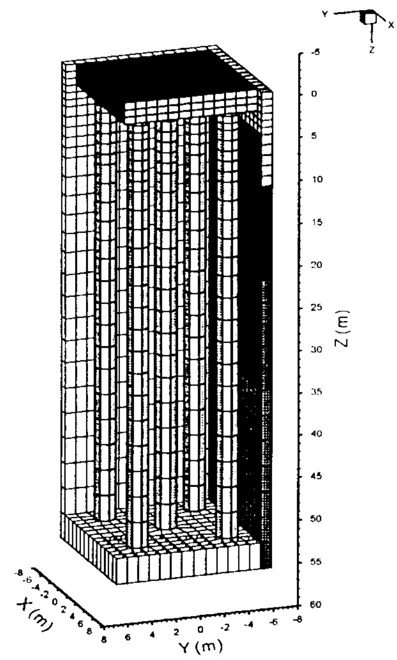


Fig. 3 Near-field model

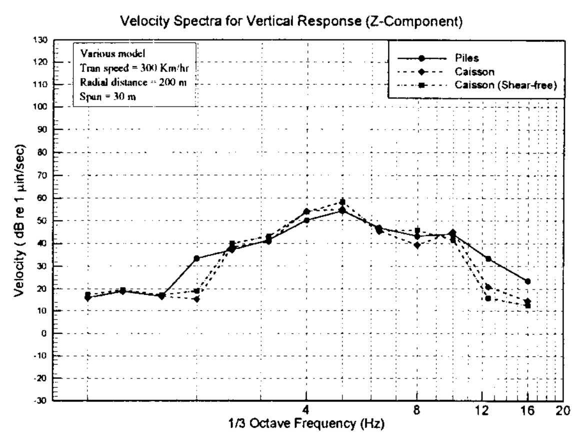


Fig. 4 Comparison various foundations

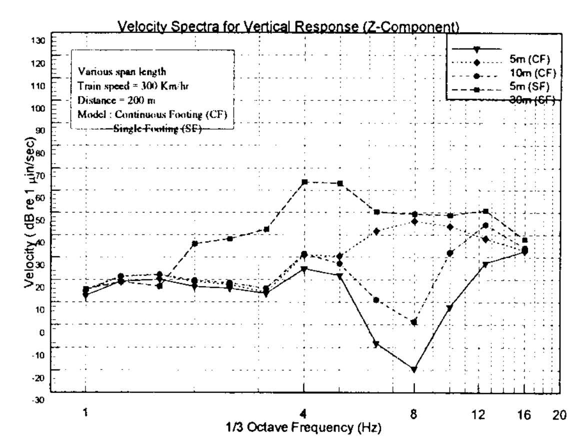


Fig. 5 Comparison of various girder span and continuous foundations

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# 出國開會報告

陳正興教授
國立台灣大學土木工程學系

第十二屆世界地震工程會議（The Twelfth World Conference on Earthquake Engineering）於 2000 年 1 月 30 日至 2 月 4 日於紐西蘭奧克蘭市之 Aotea Center 舉行，本人感謝國科會補助全部經費，得以赴紐西蘭與會，並發表論文，題目爲：Forced Vibration Tests on Pile Foundations

這次會議爲第十二次地震工程之世界會議，由紐西蘭地震工程學會及世界地震工程學會共同舉辦，共有一千六百餘人參加，爲地震工程界最大型、亦爲最重要之國際會議。這次我國共有三十餘人與會，可稱得上是有史以來參加國外舉辦之世界級會議人數最多之一次，顯示國內研究風氣之提升及國際參與之提升，大陸方面亦有四十餘人參加。由於論文數量太多，本次大會期間僅印行論文摘要集，論文全文則錄於光碟片中供查閱，專題研討論文分爲下列十個主題：

1. Earthquake Engineering in Developing Countries
2. Earthquake Engineering in Practice
3. Engineering Seismology
4. Geotechnical Engineering
5. Structural Materials, Elements and Systems
6. Lifeline Systems
7. Structural Design Criteria and Methods
8. Social and Economic Issues
9. Lesson from Recent Earthquakes
10. Other Topics

大會進行期間有數個 Parallel Sessions 同時舉行，本人主要參加大地工程之 Session，該 Session 之研討主題包括：

1. Earthquake displacement of retaining walls

2. Earthquake response of geotechnical structures

3. Earthquake response of pile foundations

4. Geotechnical materials

5. Liquefaction

6. Performance of waterfront structures during earthquakes

7. Site response

8. Slopes and embankments

9. Soil-structure interaction

本人所發表之論文爲：Forced Vibration Tests on Pile Foundations，屬於其中之第三場次。本次會議中，大部分的論文以海報形式發表，但由於本人之論文介紹一大型之現地群樁試驗結果，特經大會甄選後邀請做口頭報告。本人報告時幾乎有滿場之聽眾，並有國外學者多人發問，他們對試驗結果均相當感興趣，認爲我們做了一個很好的試驗，爲一篇好文章，本人深感欣慰，算得上是參加此次會議的最大收穫了。

由此會議中特別安排有土耳其與台灣地震之專題報導，更有很多之論文探討日本阪神大地震之震災，可看到世界各地均受到地震之摧殘，災情均大的令人可怕，深感地震工程之研究仍待加強，以造福人祉。

2 月 3 日上午舉行世界地震工程學會之理事會，選取下次會議之主辦國，結果我國以五票之差輸給加拿大，殊爲可惜，我國第一次加入此學會即提出申請主辦，能獲得十五個代表之支持，已難能可貴，可爲雖敗猶榮。

0028

# FORCED VIBRATION TESTS ON PILE FOUNDATIONS

C H CHEN[1], C S HUANG[2], C H YEH[3], M H WANG[4], J J BIAN[5], H C LIN[6], Y J LEE[7] And S H YANG[8]

## SUMMARY

of Taiwan, a series of forced vibration tests were conducted on full scale large-diameter piles to investigate their dynamic characteristics. Tests were performed on two single piles and two group piles located at Chiayi, Taiwan. For single pile tests, sinusoidal forces were applied at the top of piles in the vertical and horizontal directions, respectively. For group pile tests, sinusoidal forces were applied at various locations of the pile cap top to investigate the response functions corresponding to three translational and three rotational degrees-of-freedom of the pile cap. For each set of tests, responses in the frequency range from 1 to 20 Hz were measured. In this paper, the results of in-situ tests as well as the analytical predictions made by using the hybrid modeling were presented

## INTRODUCTION

Speed Rail (HSR) along the West Corridor of Taiwan. The total length of the route is 345km. The southern section which passes through the South-Western plain of the island will be built on elevated viaducts in order to minimize the use of lands and the potential obstacles in east-west transportation. However, this area is well known to have poor soil conditions and active seismicities. Under these circumstances, pile foundation will be the major type of foundations to be used to support the elevated structures. This section has a total length of 150km approximately. With a typical span of 30m, about 5000 piers are needed. By estimation, more than one million meters of large diameter, long piles are going to be constructed for this project. From the viewpoints of quality and quantity of piles needed, it is very worthy to perform a proto-type pilot test for the pile foundations to be used for the planned HSR project. The whole test program will include both the static and dynamic tests (Chen, 1997). Due to the page limit, only the dynamic parts will be reported herein.

To investigate the dynamic behavior of soil-pile interaction, a lot of tests have been conducted, including small scale tests in the laboratory and full scale tests in the field ( Novak and Grigg, 1976; Hakulinen, 1991; Janes et al., 1991; El-Marsafawi et al., 1992; and Imamura et al., 1996). The pile tests conducted for the planned High Speed Rail will be the proto-type large diameter piles including the bored piles and the pre-cast concrete piles (designated as PC piles, thereafter

## LAYOUT OF PILES

and 13 PC piles of diameter 0.8m. The spacing between piles (center to center) is three times of the pile diameter. All piles are of a length of 34m. To utilize the constructed piles for multiple tests, single pile tests (dynamic tests and vertical load tests) were conducted first. Afterwards, two pile caps were then constructed for group-pile tests. One group consists of 6 bored piles connected by a massive concrete cap of dimension

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5 *National Center for Research on Earthquake Engineering, Taipei, Taiwan*
6 *National Center for Research on Earthquake Engineering, Taipei, Taiwan*
7 *Depart. of Civil Engrg., National I-Lan Institute of Agriculture and Engrg., Ilan, Taiwan*
8 *Department of Civil Engineering, National Taiwan University, Taipei, Taiwan*

12m×8.5m×2m, another group consists o:` 12 PC piles connected by a massive concrete cap of dimension 9m×8m×2m. Two groups are 12m apart.

The bored piles were constructed by two different methods. The B2, B10 and B13 piles were constructed by using the temporarily full-casing method, in which the whole depth of the excavated pile hole was protected by continuously rotating steel casing during the excavation stage of the pile construction. The others were constructed by conventional slurry method of reverse-circulation type. The designed compressive strength of concrete of the bored piles is 2.8MPa. The main reinforcements used were 78-, 52- and 26-D32mm rebars in the upper, middle and lower parts, respectively of the pile. The PC piles used in this test are hollow-typed precast pre-stressed concrete piles made by method of centrifugal spinning. The thickness of the pile tube is 120mm. The compressive strength of concrete is 8MPa. The pile section has 38-9mm• high strength steel wires and the effective pre-stress transferred to concrete is equal to 800kPa. The spiral reinforcement has a nominal diameter of D5.5 (diameter in mm), with a typical pitch of 100mm at the middle portion of the pile length. At both ends, the pitches are reduced to 75mm and 50mm within a range of 1000mm and 800mm, respectively. In addition, the top 2 meters of the pile head are strengthened with longitudinal bars of 19-D19 and hoop reinforcement of D9@500mm. Each precast segment has a length of 17m. During the constructions, the pile was driven by DELMEG D100-13 driving machine with a hammer weight of 100kN. To form a complete pile of length 34m, two segments were connected by site welding. After the complete pile was driven into the ground, the inner hole of the pile was refilled with non-shrinkage concrete with longitudinal reinforcements of 8-D22mm and hoop reinforcements of D10mm@75mm. The compressive strength of the in-filled concrete is 2.1MPa.

Forced vibration tests were performed on two single piles and two group piles. The single piles tested consist of a pre-cast concrete pile (P6) and a bored pile (B10). For group pile tests, both the bored pile group and the PC pile group along with their caps are used for forced vibration tests

## GEOLOGICAL CONDITIONS

the ground surface are generally inter-bedded layers of silty clays and clayey sands. By summering the explorations conducted at site, the soil profile from the ground surface down to the depth of 40m is shown in Fig. 2. It consists of yellow clays with some organic materials (0m~3m), gray silty sands (3m~8m), soft clays (8m~12m), medium dense sands (12m~21m), clay layer inter-bedded with thin layers of fine sands (20m~32m), and medium to dense sands (32m~40m). At deeper depths, the deposits are still inter-bedded layers of clay and sand distributed down to a very deep depth. The ground water level is very high at the site, just about 1m below the ground surface. The shear wave velocities measured from the cross-hole seismic surveys are summarized as shown in Fig. 2

## LAYOUT OF TEST

tests. Both shakers have a maximum eccentricity ratio of 10:1. The larger shaker (MK-12.8-4600) has a weight of 7.54kN and a maximum eccentric moment of 520N-m, which can be run to a maximum frequency of 10 Hz.

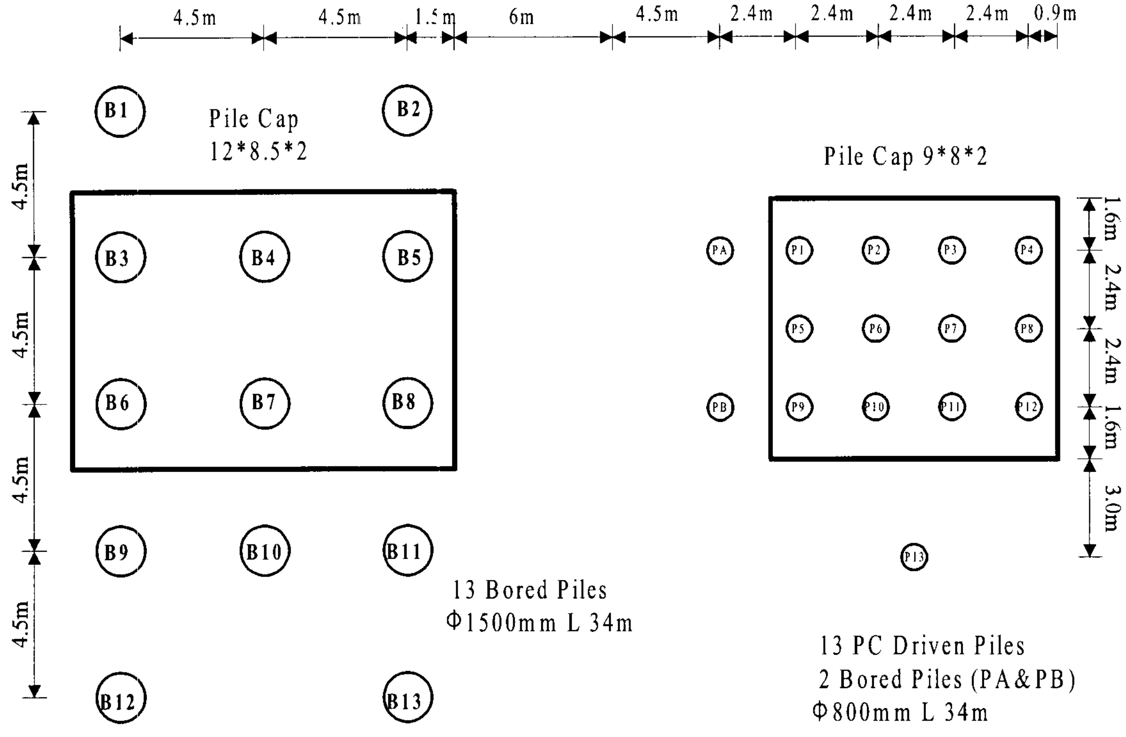



**Fig. 1 Layout of pilot pile test**

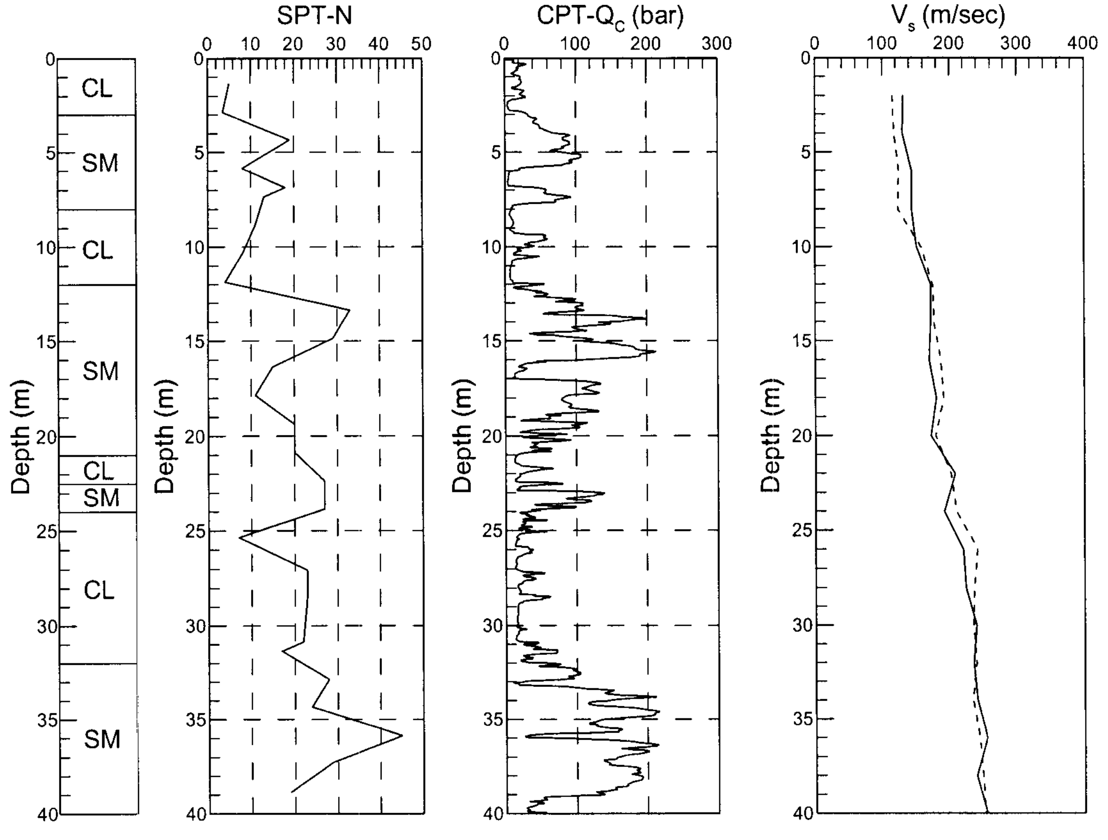



**Fig. 2 Geological profile of test site**

The smaller one (MK-12-460, ANCO) has a weight 1.67kN and a maximum eccentric moment of 52N-m, which can be run to a maximum frequency of 20 Hz. For both shakers, the maximum force can be generated is 44.5kN.

For single pile tests, the arrangement is shown in Fig. 3. The shaker was mounted on top of a 4cm thick steel plate that was prefixed on top of the pile head. In each test, the shaker was arranged to generate sinusoidal force in the horizontal and vertical directions, respectively. For all frequencies tested, the eccentric mass was adjusted to keep the applied horizontal force below 20kN, to keep the piles vibrate in the linear ranges.

For each set of the group pile tests, a total of six test runs were performed so that the frequency response functions corresponding to all six degrees of freedom of the rigid pile cap can be calculated. The six test runs are denoted as *CX*, *CY*, *CV*, *TX*, *TV*, and *RV*, as shown in Fig. 4. The first letter represents the location of the shaker while the second letter indicates the direction of force generated.

In the tests for group piles, the steady state responses of pile caps were measured by 12 servo-type velocity meters denoted as $x_i$, $y_i$, and $z_i$ (i=1,2, 3, and 4), as shown in Fig. 4. For the tests of single piles, the responses were measured in the same way except that the sensors were mounted at four corners of the steel plate. The resolution of the sensors used is 0.001cm/sec. The sampling rate is set to 50 times of the force frequency, and the total number of sampling points per record is 1,000. All tests were conducted with a frequency increment of 0.2 Hz in the frequency range from 1 to 10 Hz and a frequency increment of 0.4 Hz for the force frequencies between 10 Hz and 20 H

## TEST RESULTS AND DISCUSSIONS

in the group pile tests, one can compute the responses of interest by manipulating the measured data from all sensors. For example, the horizontal motion of the pile cap in *X*-direction is determined by taking the average of the measured data from sensors $x_i$ (i=1,2, 3, and 4) (see Fig. 4). Then the amplitude is determined by matching a sinusoidal function to the data in the least square sense. For the details of data processing, one should refer to Huang et al.(1997).

### Single Pile Test

Figure 5 shows the vertical responses of B10 bored pile subjected to a vertical vibrating force, while Figs. 6 and 7 show the horizontal and rotational pile head responses, respectively, of this pile subjected to a horizontal vibrating force. It should be noted that the top of the pile is about 1m above the ground surface and the head condition is free in rotation during the tests. The measured responses are designated with square symbols in Figs. 5 to 7. Based on the test results obtained, it can be seen that all three components are not significantly changed with the exciting frequencies, especially the phase angles. Neglecting the small variations with respect to the exciting frequency, it is evident that a simplified frequency-independent impedance function can be chosen for engineering applications, which will be more simple and convenient in practice.

### Pile Group Test

Figures 8 and 9 show the horizontal responses of both pile caps in the *X* and *Y* directions obtained from the tests *CX* and *CY*, respectively. The response magnitudes in the *X* and *Y* directions are very close. The response of the PC pile group has a clear peak at 5.5 Hz. The vertical responses of both pile caps obtained from the *CV* tests are shown in Fig. 10. In this figure, the responses recorded at smaller frequencies are fluctuated due to the effects of noises, which will affect the responses significantly at very small excitation forces. The responses are quite stable as the frequency of test is increased. Besides, it should be noted that the discontinuities appeared on the response curves obtained are actually resulted from changes of the applied forces, i.e. change of shaker used or change of eccentricity of shaker. Under these circumstances, different magnitudes of input force were applied at different test runs. Small variations of response are unavoidable in a test. Generally speaking, the results obtained are quite consistent in all frequencies interested. From the results shown in Figs. 8 to 10, one can find that the group of 6 bored piles shows much stiffer than the group of 12 PC piles. The horizontal responses have shown peak values at frequency 5.5 Hz, while the vertical responses of both pile groups do not have clear peaks in the frequency range tested.

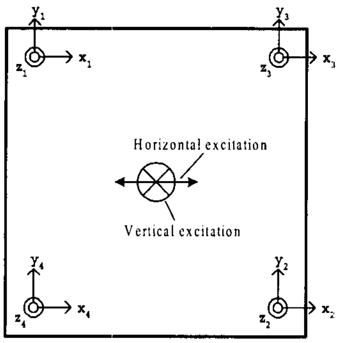



**Fig. 3 Layout of single pile test**

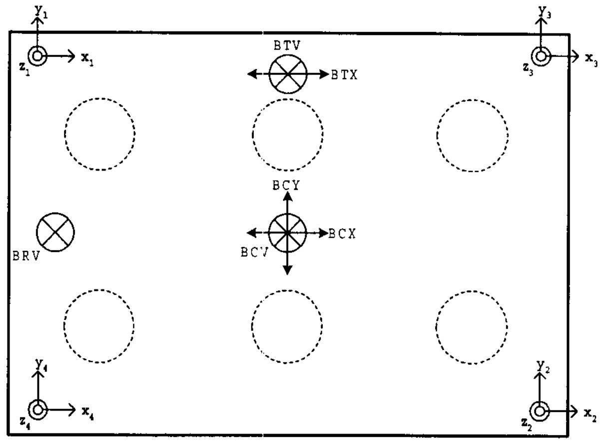



**Fig. 4 Layout of group pile test**

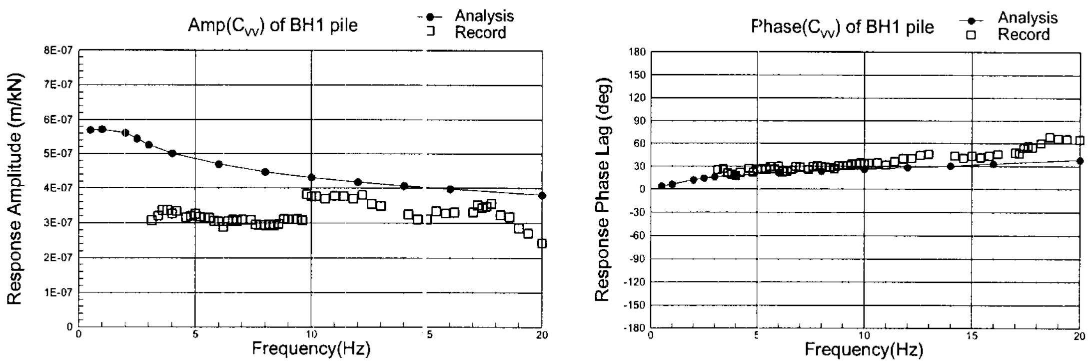



**Fig. 5 Vertical response of the bored pile due to vertical excitations**

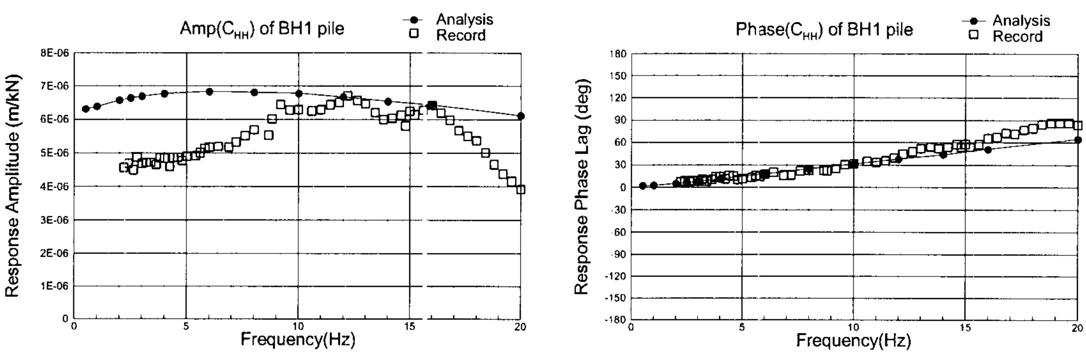



**Fig. 6 Horizontal response of the bored pile due to horizontal excitations**

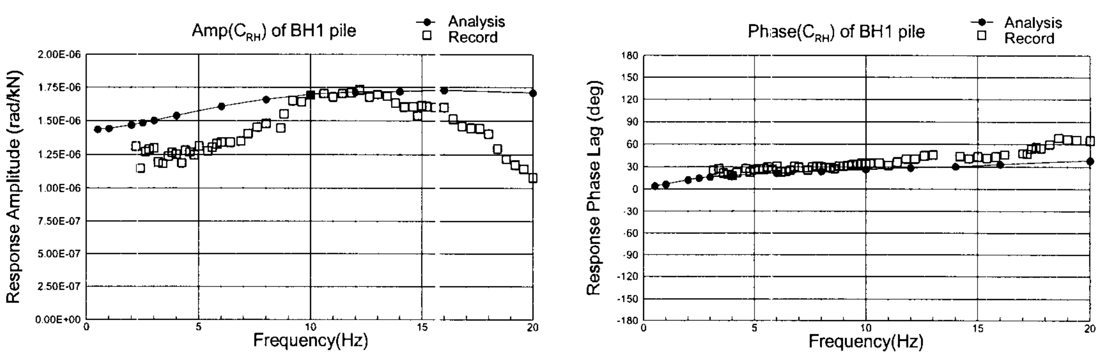



**Fig. 7 Rotational response of the bored pile due to horizontal excitations**

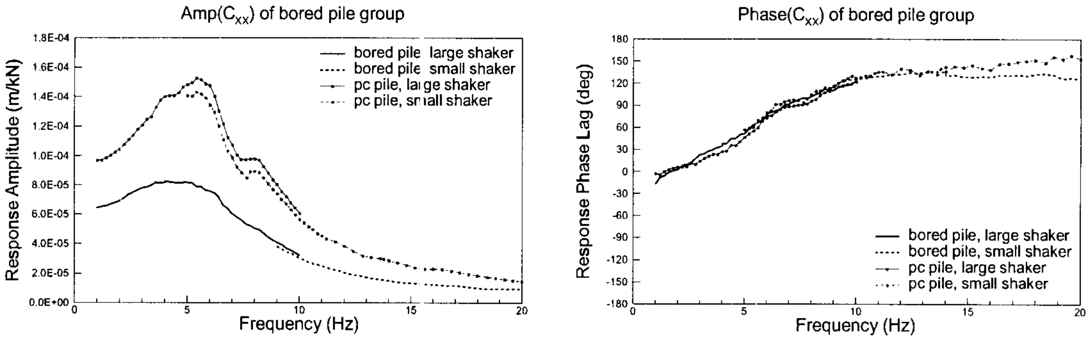



**Fig. 8 Translations in X-direction of both pile groups in CX test**

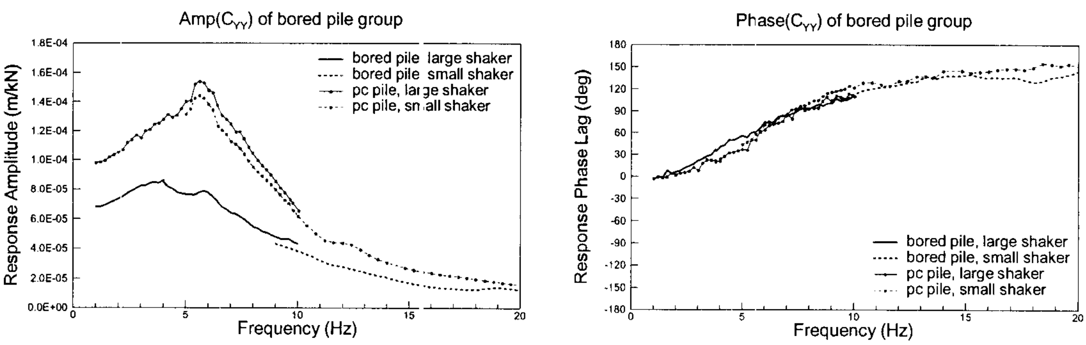



**Fig. 9 Translations in Y-direction of both pile groups in CY test**

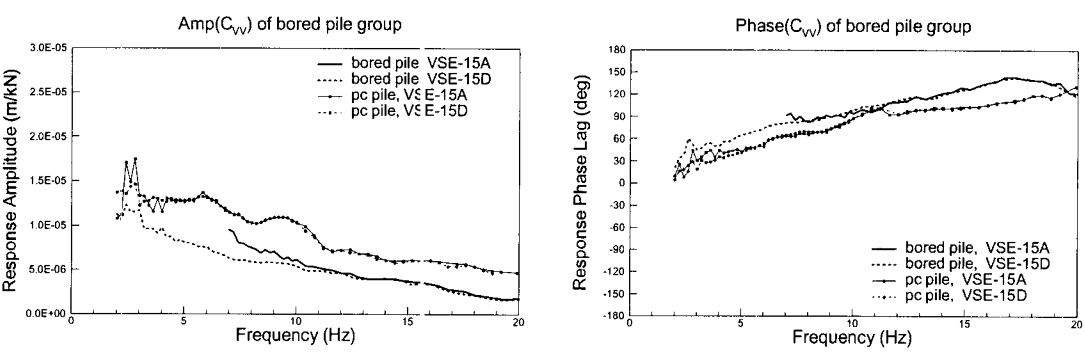



**Fig. 10 Vertical responses of both pile groups in CV te**

## ANALYTICAL MODELING

To model the soil-structure interaction response of single piles, the method of hybrid modeling has been shown to be very effective (Gupta et al., 1982 and Chen, 1988). Basically, the hybrid modelling uses method of substructure by partitioning the entire soil-pile system into a near-field (NF) and a far-field (FF), as shown in Fig. 11. To effectively model the pile, the NF/FF interface can be chosen to be a slender cylinder which cuts through the soil region around the pile. The near-field, consisting of the pile and a finite portion of surrounding soils, can be modelled appropriately using axisymmetrical finite elements. The far-field is a semi-infinite layered half-space with a surface cavity. The dynamic impedance of the far-field can be formulated by using the indirect boundary element method (Lee, 1992), and can thus be represented by a complex-valued boundary impedance matrix. Imposing the conditions of force equilibrium and displacement compatibility along the interface between the near- and far-fields, the impedance of the whole system can then be obtained by assembling the impedance matrices of the near- and far-fields in the frequency domain. By giving the externally applied forcing functions at the pile head, the dynamic responses of the pile can then be solved through a rather small analysis.

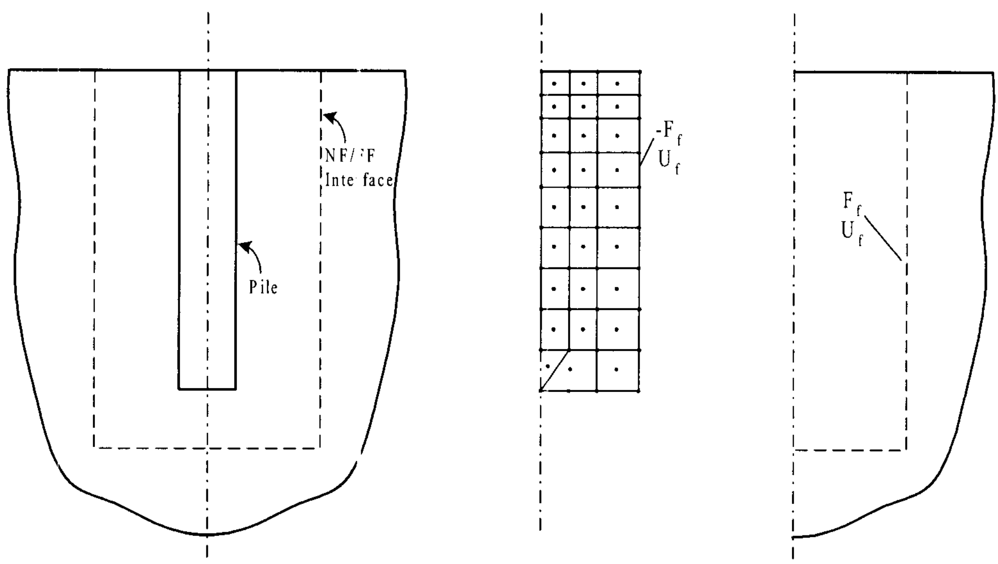
**Single Pile Modeling**

By using the hybrid modeling to model the above-mentioned B10 bored pile tests at Chaiyi, the near-field is chosen to be a cylinder with a radius of 3m and a height of 40m. The 9-node axisymmetrical solid elements are used to model the pile and the near-field soils located in the near-field region. Element size used ranges from 0.4m near the ground surface to a size of 2m at the deeper soil layers. Properties of soil elements adopted are deduced from the shear wave velocities measured by the cross-hole seismic method as shown in Fig. 2. Damping ratios used for the pile and soil elements are of hysteretic type and set equal to 0.02 and 0.05, respectively. The far-field impedance matrix is formulated based on the same soil properties used in the near-field, by using the indirect boundary element method.

**Results of Analysis**

Based on the model constructed above, the compliances of the B10 pile due to a unit sinusoidal force applied at the pile head, just as the test conditions, are calculated and compared with the field test results. Solid lines in Figures 5 to 7 are the calculated pile head compliance of the 1.5m-diameter bored pile tested, in which $C_{vv}$ is the vertical compliance due to unit vertical excitations, $C_{hh}$ and $C_{hr}$ are the horizontal compliance and the coupled rotational response, respectively, due to unit horizontal excitations.

From comparisons, it can be seen that the analytical results agree quite well to the field test data in general. The responses of phase angle fit very well at all frequencies. However, the amplitudes calculated from analytical modeling are a little larger than the test results at all frequencies. Regarding the disturbance and uncertainties involved in a pile construction, and the complexity of the soil conditions around the pile, the results obtained from analytical modeling are quite satisfactory from the engineering point of view.




**(a) Soil-pile system (b) Near-field (c) Far-field**

**Fig.11 Hybrid modeling for the soil-pile syste**

## CONCLUSIONS

Results obtained are very valuable for both the researchers and engineers working on pile engineering. By using the hybrid modeling, results of analytical prediction for single pile are satisfactory in general; however, more studied including the constructional factors and the modeling for pile groups are indeed necessary for engineering applications.