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PREDICTION AND PERFORMANCES OF SHORT EMBEDDED CAST IN-SITU DIAPHRAGM WALL FOR DEEP EXCAVATION IN BANGKOK SUBSOIL

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ABSTRACT

This paper presents a case history of performances and predictions of a diaphragm wall which is subject to serve differently from its initial purpose for which the wall was constructed. The updated requirements associated with the additional excavation created a situation whereby the wall would become short embedded and behave like free-end supported wall. As the wall had been already constructed during the time of modification it was only possible to modify the excavation method and bracing system. Accordingly, the wall stability and possible toe movement were analyzed, to accommodate the updated requirements, by using finite element methods. Continuous inclinometer monitoring has been carried out during the construction and results are being compared and analyzed for predicted values. Performance of the wall based on comparison between the inclinometer monitoring results during different excavation stages and predicted results, are discussed. It has been found that excavation depth for first bracing layer and construction period are very important for diaphragm wall performance. Large initial movements in the wall strongly influence the wall movements in the successive excavation works. Construction practice plays a major role in deep excavation work.

KEY WORDS

Diaphragm wall, Lateral movements, Inclinometer, Settlements, Free-end, Bracing, Excavation, Critical height, Cantilever

INTRODUCTION

Economic growth in Thailand during the past decade has been forcing the construction industry to look for all feasible methods to enhance the infrastructure of the country especially for the Bangkok city. Huge number of high-rise buildings with multilevel basements are being erected within a short time possible to accommodate the booming economy. Value of urban land is increasing in many folds beyond the limits. Unlike in developed countries, the building rules and regulations of this city are only in documented form and hardly established all in practice in a short time. As a result, developers and owners enjoy relatively more freedom to modify the structural layout, even during and after construction stages, to facilitate their growing demands which are being conceptualized within a short time duration in the fast developing city, aiming to maximize the area utilization. The project reported in this paper is an example for such case.

The project site is located in the vicinity of central business area of Bangkok. The structure is a multi-storied building for a business complex. The load of the structure would be carried by bored cast-in-situ bored piles and barrette piles with toe depth of 60m below the ground level, embedded into the second sand layer of the Bangkok subsoil.

The constructed cast in-situ diaphragm wall is 0.8m thick and the toe is embedded into stiff clay at a depth 18m below the ground level. Initially the wall was designed for construction of four basements including mat foundation of 3.5m thick. The maximum excavation depth was set to be 14m and a three level of temporary bracing system was considered to construct the basements with conventional bottom up construction method.

Embedded depth below the final excavation for the mat foundation was designed to be 4m. After construction of the wall it was decided to increase the number of basements to be five for part of the building area (Tower area) and excavation depth needed to be increased by 1.8m from the originally designed depth, leaving the wall embedment depth of 2.2m only.

SUBSOIL CONDITION

The sub soil conditions at the project site is not uncommon to usual conditions observed in Bangkok comprising a 15m thick soft to medium clay layer on the top underlain by stiff clay to a depth of about 22m. Below the stiff clay is a series of alternating layers of hard silty clay, sandy clay and dense sand.

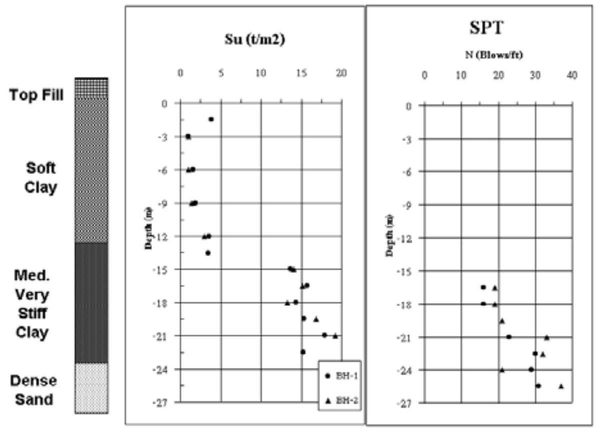

Fig. 1 Soil Profile at the Site

The soil properties used for Diaphragm wall design are summarised in the table below

Table 1 Unit weight and Undrained shear strength of soil layers.

Soil Type	Unit Weight γ (t/m^3)	Undrained Shear Strength Su (t/m^2)
Soft Clay CH	1.6	1-2.9
Medium Clay CH	1.93	3.5
Stiff Clay CH	2.0	13.7-17.95

SITE CONDITION

The project site is located in the corner of a main road and a street of the central business district in Bangkok. The planned building is facing both the main road and street. The building plan was set out in accordance with building code having a setback of 6m from an adjacent property boundaries. Behind it are two story buildings located more than 10m away. Deep excavation is ideal in such site condition. Figure 2 shows the layout plan of the building.

DIAPHRAGM WALL DESIGN

Originally diaphragm wall was designed with a 3 level temporary bracing to allow a maximum excavation depth of 14m below the existing ground for construction of mat foundation (see Fig. 3). Computer programs of finite element methods (WALLAP and CRISP) and finite difference method (FLAC) were used to simulate the staged excavation and predict the wall behaviour and performance. In the computer modeling, undrained strength of soil with total stress parameters derived from laboratory and field test results were used. 0.8m thick Diaphragm wall with toe depth of 18m was found to be adequate enough for the planned basement construction in such soil condition.

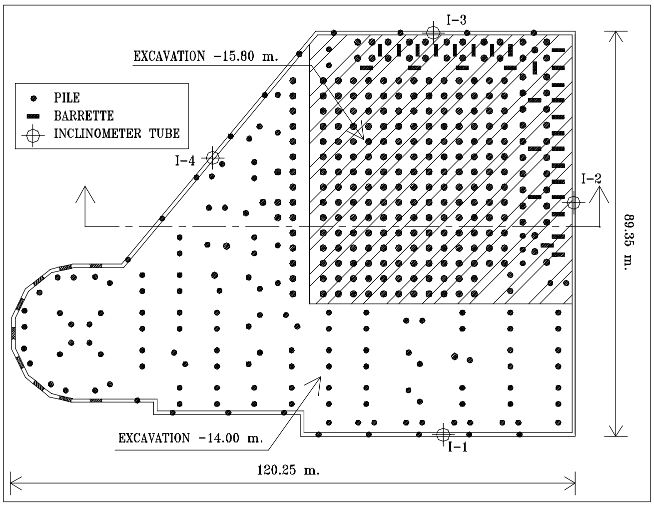

Fig. 2 Layout Plan of Diaphragm Wall & Piles

Estimated maximum bending moment of 90 t-m/m on the excavation side and 56 t-m/m on soil side of the wall. The maximum deflection of the wall predicted was about 35-37mm with some degrees of toe movement up to 11mm.

Bored piles of 1.20m in diameter were in-cooperated within and below the toe of the wall to support the column load and dead load of the wall. Foundation piles (dia. 1.20m), including bored piles and barrette piles (2.70mx0.80m) were founded at the depth of 60m in dense sand layer. Four inclinometer tubes I-1 to I-4 (20m to 25m in length) were installed in the wall panels with bored pile legs to observe the behaviour and performance of the wall.

Diaphragm wall and foundation piles were constructed in parallel. After completion of diaphragm wall construction, but construction of bored piles were still in progress, overall design of the building was revised to suit the requirements of the property market. Number of basement floors was increased from 4 to 5 in the main tower area. As a result, the diaphragm wall constructed was deemed to support for the maximum excavation depth of 15.8m.

Firstly diaphragm wall design and construction stages were then reviewed. Since the wall has been constructed, excavation sequence and bracing system can only be modified so that the modified deeper excavation can be done without impairing the structure and stability of the wall.

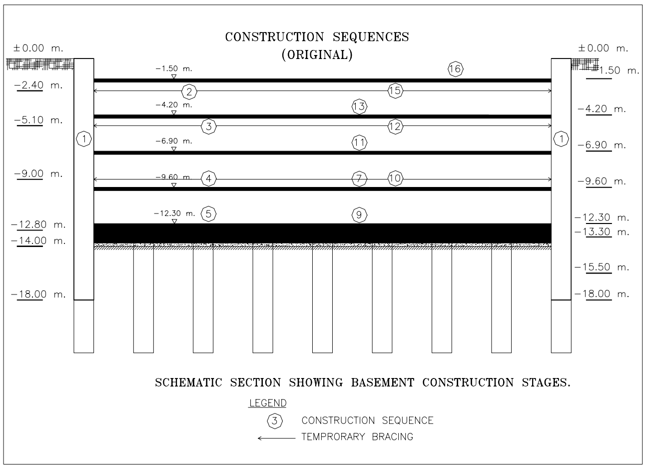

Fig. 3 The Original Construction Sequence with three temporary bracing levels

MODIFIED CONSTRUCTION SEQUENCE

Finite element computer modeling was again carried out for the desired excavation depth. The model analysis showed that 4 level temporary bracing was necessary with strut pre-loading. It also indicated the diaphragm wall in free-end condition with some degrees of toe movement in a range of 23mm. The maximum predicted wall movement at the top was 77mm.

To prevent progressive toe movement with time, followings were recommended for excavation works.

(i) frequent monitoring of diaphragm wall movements when excavation close to the final excavation depth.

(ii) casting a reinforced lean concrete slab with a thickness of 20cm and 6m wide in front of the wall just after reaching the excavation depth of -13.8m.

(iii) excavation to -15.8m must be carried out leaving a 12m wide soil berm with lean mixed concrete slab on top of the berm all around the wall.

Apart from four inclinometer tubes installed in the wall, a total of 16 sets of settlement observation stations were installed around the wall perimeter. Each station comprised two to three settlement points located at 3m intervals and 3m away from the wall.

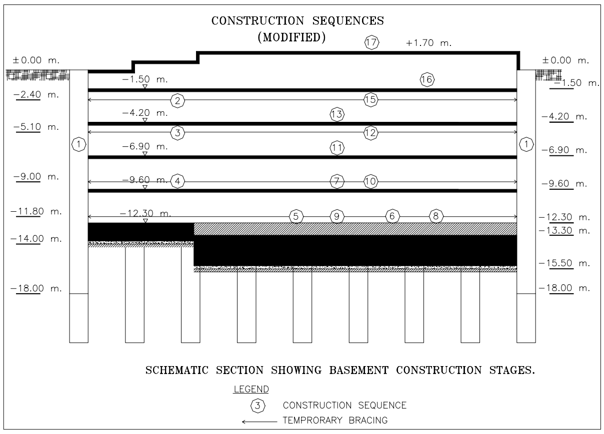

Fig. 4 The Modified Construction Sequence

The risk of over excavation was carefully assessed not only for diaphragm wall but also for the existing buildings near by the site. Review of the site condition indicated that the existing buildings surrounding the project site were more than 10m away from the excavation zone and they were found subject to no risk.

BRACING SYSTEM

The bracing system included continuous waling beams along the diaphragm wall and longitudinal and transverse struts. The spacing of strut was generally 6.4m to 6.8m. Table no. 2 shows the type of wide flange structural steels used for the bracing.

Table 2 Structural steel sections used for temporary bracing

Bracing Layer	Strut	Wale
1st	2W300x300x94kg/m	2-W300x300x94kg/m
2nd	2W350x350x137kg/m	2W350x350x137kg/m
3rd	2W350x350x137kg/m	2W350x350x137kg/m
4th	W350x350x137kg/m	2W350x350x137kg/m

LATERAL MOVEMENTS OF THE WALL

Monitoring of wall movements was carried out by reading the inclinometer tubes profiles installed in the wall at every major construction stages. The wall constructed was designed for initial excavation of -2.5m to install the first bracing layer. However the excavation has been done to about -3.0m to -3.2m at once and a movement of 40mm was observed. Time delay during construction of capping beam and over excavation deeper than the critical height ($H_{cr} = 2C/\gamma$) was unsupported around three months resulted an increase in wall movements to 55mm -70mm. Tension cracks were developed in the soil of active side about 2-3m away from the wall. After installation of struts and pre-loading them, the lateral movements of the wall ceased off. An increment of 2mm-4mm was recorded with further excavation stages (See Fig. 5). Struts from first to fourth levels were pre-loaded 40 t/m, 75 t/m, 75 t/m and 40 t/m respectively, 20%-25% of design load of bracing system, prior to further excavation. The pre-loading of the strut was very effective to reduce lateral movement of the wall and ensuring good intact between wall and bracing system.

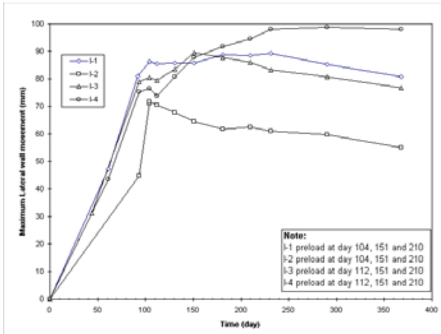

Fig. 5 Maximum lateral movements at the top of wall vs. Construction Time.

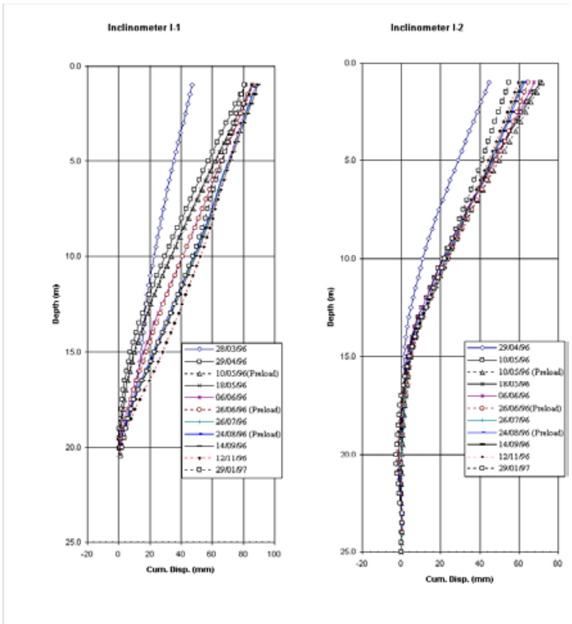

Fig. 6A. Lateral Movements of the wall shown by Inclinometer Tubes I-1 and I-2.

The pattern of movements of the wall shown by the inclinometer readings is similar to that of cantilever condition. Because of large movements were induced Initially, braced mode was not developed. After reaching the final excavation depth the observed movements of the wall at the toe level were in a range of 1.2mm to 16mm while the predicted toe movement was about 23mm. Predicted and observed wall movements is tabulated as below;

Table 3 Predicted and observed wall movements

Location on the Wall	Predicted for 14.0m Excavation (mm)	Predicted for 15.8m Excavation (mm)	Observed on Site (mm)
Top	35-37	77	55-98
Toe	11	23	1.2-16

The maximum lateral movements were found in I-4. At the final excavation stage, movements of the wall increased considerably below the first bracing level down to toe of the wall. It was noted that in I-4 area the longitudinal and transverse struts of the bracing system become diagonal to the wall sections due to the geometry of wall layout. The stiffness of the bracing in this particular zone is considered to be much less than that of others.

The wall's fixity was found to be at depth 20m-22.5m from inclinometer readings when the final excavation reached.

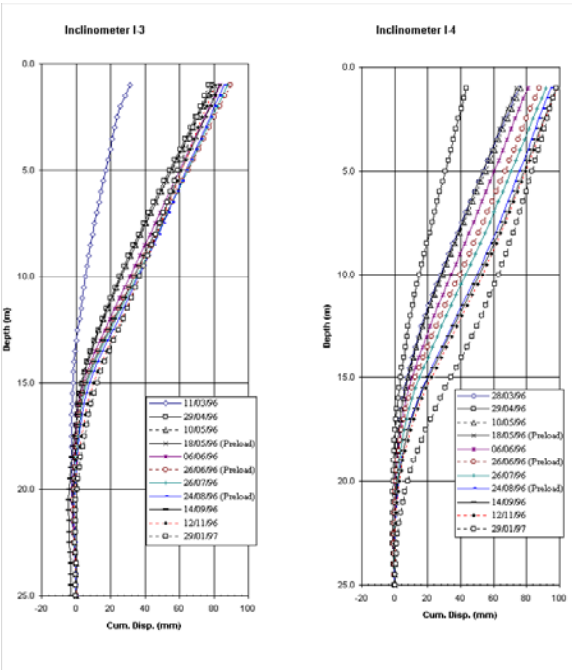

Fig. 6B Lateral movements of the wall shown by Inclinometer Tubes I-3 and I-4.

From the settlement observation data, it was found that the maximum ground settlements behind the wall were approximately 34% to 50% of the lateral wall movements, except for the area near the I-4 where the ground settlements equal the lateral wall movements.

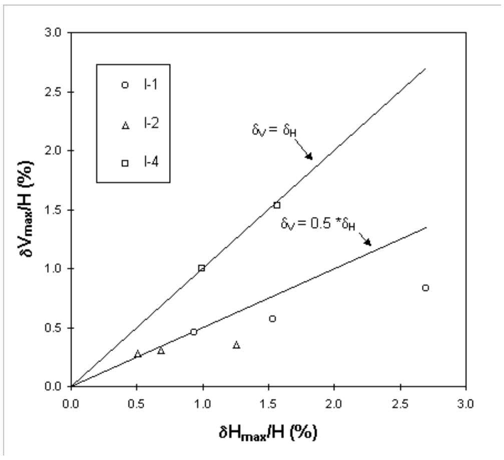

Fig. 7 Relationship between maximum ground settlements and maximum lateral wall movements

ANALYSIS ON PERFORMANCE OF THE WALL

Based on the deformation of the wall measured from the inclinometer readings, the bending moments were computed from the well known equation to check structural performance of the wall

$$M = E_c . I_{ef} . [dS/dy] \qquad (1)$$

where E_c = Modulus of Elasticity of concrete
I_{ef} = Effective moment of inertia of the wall section
S = slope given by inclinometer reading
y = elevation of the slope given

Generally the bending moments computed were within the design moments and the performance of the wall is found to be satisfactory. However a direct comparison between bending moment envelopes from computer simulation and bending moments derived from inclinometer readings shows some differences, especially at the strut levels and below the depth of 12m. These differences can be due to the following;

1. Stiffness of medium to stiff clay layer used based on the field and laboratory tests for simulation is much lower than the actual in-situ soil stiffness. In simulation for the wall design, influence on soil stiffness below the final excavation level by bored piles was not considered.

2. Efficiency and stiffness of bracing system, including pre-loading forces. In the original design calculation, much lower pre-loading forces were used.

3. large initial movements during the wall in cantilever condition.

4. Inclinometer tubes (20-25m in length) were installed in diaphragm wall panels with bored pile legs. The bored pile

legs provide the fixity of the wall and increase the bending moments at a location near the bottom of the wall.

5. Moment redistribution in the wall is considered not fully activated.

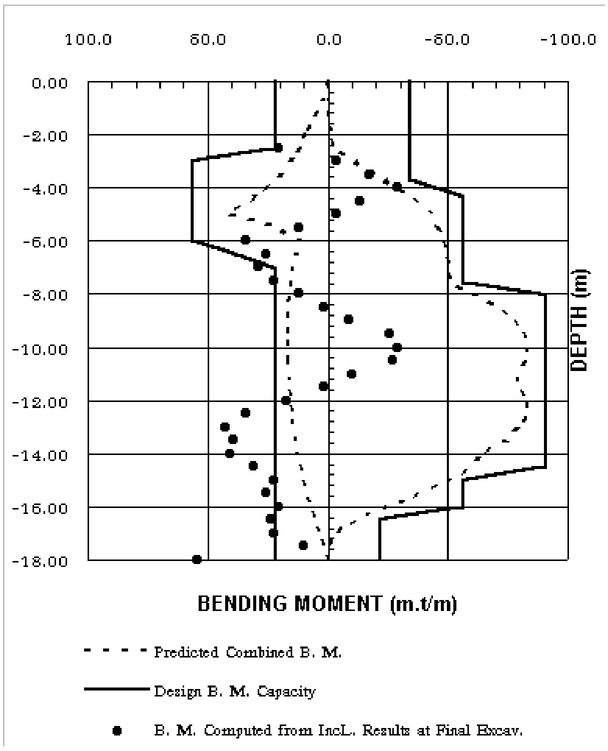

Fig 8 Design envelope of bending moments from computer modeling and bending moments computed from inclinometer readings at final excavation

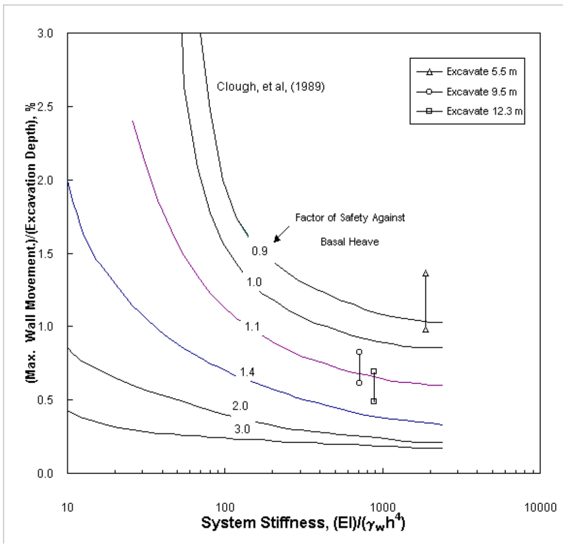

Fig. 9 Relationship amongst system stiffness, non-dimensionalized wall movement and factor of safety against basal heave .

The relationship between wall movement and system stiffness (Fig. 9) shows that because of large movements during the wall is in cantilever condition, the excavation to 5.5m with one bracing deviates from the normal trend of braced excavation.

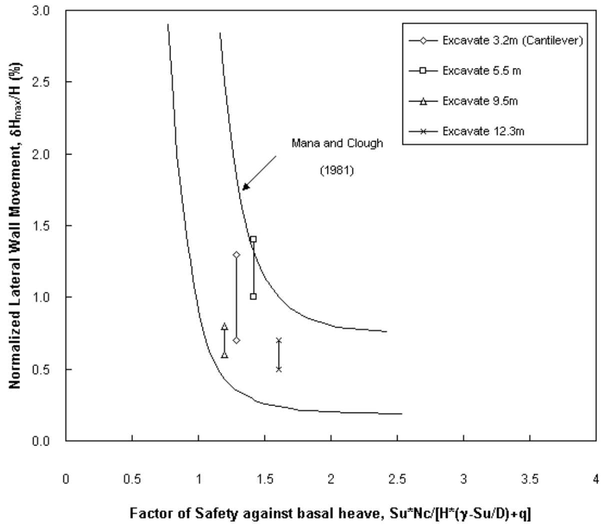

Fig. 10 Relationship between Normalized Maximum lateral movement versus factor of safety against basal heave.

The normalized lateral movements of the wall shown in Fig. 10 is within the range of 0.5-1.5% which is slightly higher than those of similar projects in Bangkok subsoil. The higher movement ratio occurred during excavation in the soft clay layer with slight over excavation and delay in first strut installation. It is observed that the first excavation depth and duration in bracing layer installation are very critical for the performance of the wall at further excavation stage.

CONCLUSION

1. Movements in first excavation stage is governing the performance of Diaphragm wall for further excavation.

2. First excavation deeper than the critical height of the soil needs to be supported immediately.

3. Apart from support system and soil properties, construction practice influenced the movement of the wall in this particular project.

4. A successful deep excavation with short embedded diaphragm wall has been achieved using a combination of prediction by computer simulations and instrumented field observations.

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REFERENCES

Balasubramaniam A.S., Bergado D.T., Chai J.C. and Sutabutr T. [1994]. "Deformation Analysis of Deep Excavations in Bangkok Subsoils" XIII Internation Conference on Soil Mechanic and Foundation Engineering , New Delhi, India. pp. 909-914.

Day P. W. [1994]. "Factors Influencing the Movement of Retaining Structures" XIII Internation Conference on Soil Mechanic and Foundation Engineering , New Delhi, India. pp. 109-114.

Gan C. H. [1997]. "Review and Analysis of Ground Movements of Braced Excavation in Bangkok Subsoil using Diaphragm Walls." M. Eng Thesis, Asian Institute of Technology. Bangkok, Thailand.

Mana I. A. and Clough G. W. [1981] "Prediction of Movements for Braced Cuts in Clay", Journal of Geotechnical Engineering Division, ASCE, Vol. 107, No. GT6, pp. 36-42.

Peck R. B. [1969] "Deep Excavation and Tunneling in Soft Ground. Proceeding of 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico. State of Art Volume, pp. 225-290.

Saxena S. K. [1975]. "Measured performance of a rigid concrete wall at the World Trade Centre." Diaphragm Walls and Anchorages. Institution of Civil Engineers, London, UK. pp. 107-112.