INSTRUMENTATION FOR DEEP EXCAVATION MONITORING - A CASE STUDY

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RESULTS OF ANALYSIS

Center of Gravity and Eccentricity

The position of the center of gravity of the original foundation and machines was only 30.7 cm below the foundation surface. Besides, there was also excessive eccentricity existing. In general, the c.g. of machine combined with the foundation and supporting piles should be located as close as possible on a vertical line through the centroid of the pile group and the eccentricity of the combined masses should not be greater than 5% (TOMLINSON, 1977). For the original foundation, the eccentricity was found to be 9% which is almost double of the allowable eccentricity. The c.g. of foundation and machine was eccentric on the side of the compressor which was the main cause for the larger vibrating amplitudes on this end.

Vibrating Amplitudes

Table 2 tabulates the results of analysis for the original foundation under vibration. The amplitude in point A (see Fig. 3) which had the largest vertical amplitude measured was chosen for comparison. It can be seen that the computed amplitudes gives the same order as the measured amplitudes. The slightly higher computed values might indicate the effect of neglecting damping, footing embeddment and restraint of floor slab.

TABLE 2 COMPARISON OF COMPUTED AND MEASURED AMPLITUDES BEFORE IMPROVEMENT

Direction	Measured Amplitudes 2A, mil		Computed Amplitudes 2A, at P_t A, mil
	at Machine	at P_t A	
Vertical	9.5	7.9 - 8.0	8.9
Horizontal	4.5	1.05	1.0
Axial	12.2	1.85	3.7

METHODS OF IMPROVEMENT

As seen in the analysis results, the main cause of the vibrating problem was the insufficient rigidity of the pile group. To increase the rigidity of a pile group, either one of the following two methods can be applied:

(1) Increase the number of piles in the pile group and also enlarge the cap,

(2) Use cement or chemical grouting to increase the supporting strength of the pile group.

Judging from the soil profile, it seems that grouting can only be applied to the medium silty sand layer which is at a depth from 7.5 m to 20 m below ground surface. Grouting at this depth could increase the bearing capacity of the pile group. However, the soils at an upper depth are fine-grained silty clay or clayey silt which are not suitable for ordinary grouting. For this reason, the horizontal and axial vibrating amplitude of the foundation could hardly be improved since the lateral elastic resistance of the pile group is contributed mainly from the soils at the upper depth. It is extremely difficult to analyse and predict the behavior of the pile group after grouting. Besides, the effectiveness of grouting depends primarily upon the technique used. The available information provided was not sufficient for selecting a suitable type of grouting technique. Therefore it was proposed to increase the number of piles by adding 16 piles of 40 cm diameter and 15 m long around, and enlarge the pile cap to 7 m x 11.4 m. However, due to the limitation of headroom in the plant, buried cable lines, and other considerations, the foundation was reinforced by adding 32 cast-in place piles of 40 cm in diameter and 10 m long. The arrangement of piles and the enlarged cap is shown in Fig. 5. The thickness of the enlarged portion was 1.6 m and 0.7 m respectively at the compressor side and the motor side.

The vibration amplitudes of the foundation after improvement were measured. In general, the vibration amplitudes of the foundation were reduced to about half of its original value. Largest vertical amplitudes were still detected at the side near point A. The computed amplitudes of the improved foundation are compared in Table 3 with the maximum amplitudes at point A. The computed amplitudes are less than the maximum measured amplitude indicating the complicated behavior of the foundation after underpinning. It is possible that the effectiveness of some of the supplementary piles installed was not fully developed, or the transmission of vibrating motion from the origin foundation to the enlarged portion was not as perfect as if the foundation were in a single block. However, the vibrating amplitudes in the majority part of the foundation had been reduced to less than the allowable value. The performances of the foundations after underpinning have been greatly improved.

TABLE 3 COMPARISON OF COMPUTED AND MEASURED AMPLITUDES AFTER IMPROVEMENT AT POINT A

Direction	Measured Amplitudes 2A, mil	Computed Amplitudes 2A, mil
Vertical	3.9 - 4.8	1.0
Horizontal	0.5 - 0.8	0.14
Axial	0.6 - 1.7	0.4

CONCLUSION

The compressor foundations after improvement have given satisfactory performance. Similar approach of analysis was recently adopted for another compressor foundation of identical type in the first stage, second phase expansion of the China Steel Mill.

ACKNOWLEDGEMENT

The contribution of CTCI Engineering Corporation in planning of the design and layouts of the enlarged footing is gratefully acknowledged.

REFERENCES

1. BARKAN, D.D. (1962), Dynamics of Bases and Foundations, McGraw-Hill Company, New York.

2. HARRIS, C.M. and CREDE, C.E. (1976), (Ed.), Shock and Vibration Handbook, 2nd Ed., McGraw Hill International Book Company.

3. SINGH, J.P., DONOVAN, N.C. and JOBSIS, A.C. (1977), Design of Machine Foundations on Piles, Journ. Geotech. Engn. Div., ASCE, Vol. 103, No. GT8, pp. 863-877.

4. TOMLINSON, M.J. (1977), Pile Design nad Construction Practice, A Viewpoint Publication, London

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INSTRUMENTATION FOR DEEP EXCAVATION MONITORING—A CASE REPORT

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SUMMARY Due to the variable nature of soil behavior in situ, many soil engineering design have to use greatly simplified soil condition and to employ idealized assumptions. It has long been recognized that field performance records play a vital role in the improvement of the state-of-the-art of soil engineering practice. Use of field instrumentation for monitoring soil engineering construction has become more and more popular in developed countries. The contribution of instrumentation to construction safety cannot be overemphasized.

In Taiwan, deep excavation for building construction becomes a common practice in the last few years. However, either due to improper design and lack of sufficient knowledge of soil behavior, many problems, even to the extent of complete failure, have occurred during excavation work. This paper describes one of the first project in Taiwan using field instrumentation for monitoring deep excavation of a tall building structure. Instrumentations installed at the site include inclinometers, piezometers, heave points and settlement points. Field monitoring data in relation to the construction progress are analyzed and discussed.

INTRODUCTION

Partly due to governmental regulatory requirement and partly due to the desire of maximum utilization of ground space, all buildings in excess of five stories in Taiwan incorporate at least one level of basement. For many tall buildings, particularly in the Taipei City area, 3 or even 4 levels of basement are not uncommon. It has been reported that about 80% of tall buildings in Taiwan adopts compensated or floating foundation. For these structures, deep excavation becomes an essential part of the construction program. Due to the rapid development of construction projects in the past few years and lack of strict control on geotechnical evaluation, numerous cases of damages to adjacent structures caused by improper or inadequate safety measures during excavation have been reported. In 1976 within a period of one year, governmental agencies were directly involved in arbitration of 198 cases of damage compensations due to improper construction. Among which majority of the cases involved excavation problems (MOH and OU, 1979). Although field instrumentation for construction safety monitoring has been used extensively in developed countries for many years, the use of such system for building excavation control received attention in Taiwan only very recently. This paper presents a case study carried out on the construction of a tall building in Taipei.

DESCRIPTION OF THE CONSTRUCTION

The building concerned is the China Airlines Building located in the eastern part of Taipei City. The building consists of 13 stories above ground surface and two levels of basement. As shown in Fig. 1, the main tower block occupies only about fifty-five percent of the site whilst the basement structure extends to the entire area which is about 3,300 sq.m. Compensated mat foundation was utilized to support the structure. The superstructure was constructed with one-way prestressed concrete slab in combination with central reinforced concrete core. The substructure consists of two levels of basement and was constructed with conventional reinforced concrete. Due to difference in loading conditions, construction of the substructure was made in two stages. The first stage involved construction of the substructure immediately below the tower block and part of the superstructure. According to the design plan, when the superstructure reached the level of 7 stories, construction of the surrounding substructure was to be commenced. The mat foundations of the two substructures were connected with hinge connection. For the purpose of reducing differential settlement of the two parts of the structure, the tower block was preloaded by filling the basement with water.

For the substructure excavations, two rows of prepakt piles, 40 cm in diameter, were used as the retaining structure.

Due to the unconventional construction scheme and the difficult subsoil conditions, the developer and constructor, BES Engineering Corporation, engaged Moh and Associates to review the foundation and structural design, and to take the responsibility of instrumentation installation and moniroting for excavation safety control.

SITE AND SUBSOIL CONDITIONS

The site of the China Airlines Building is located in the eastern business district of Taipei City. The site is bounded by a 40 m wide major roadway Nanking East Road on the south and the 20 m wide Lung Chiang Street on the west. On the other side of the Lung Chiang Street, there are several multi-storey buildings. Besides the stability of the excavation itself, other major concerns of the excavation work are effect of excavation and dewatering on the settlement of nearby buildings and the road surfaces.

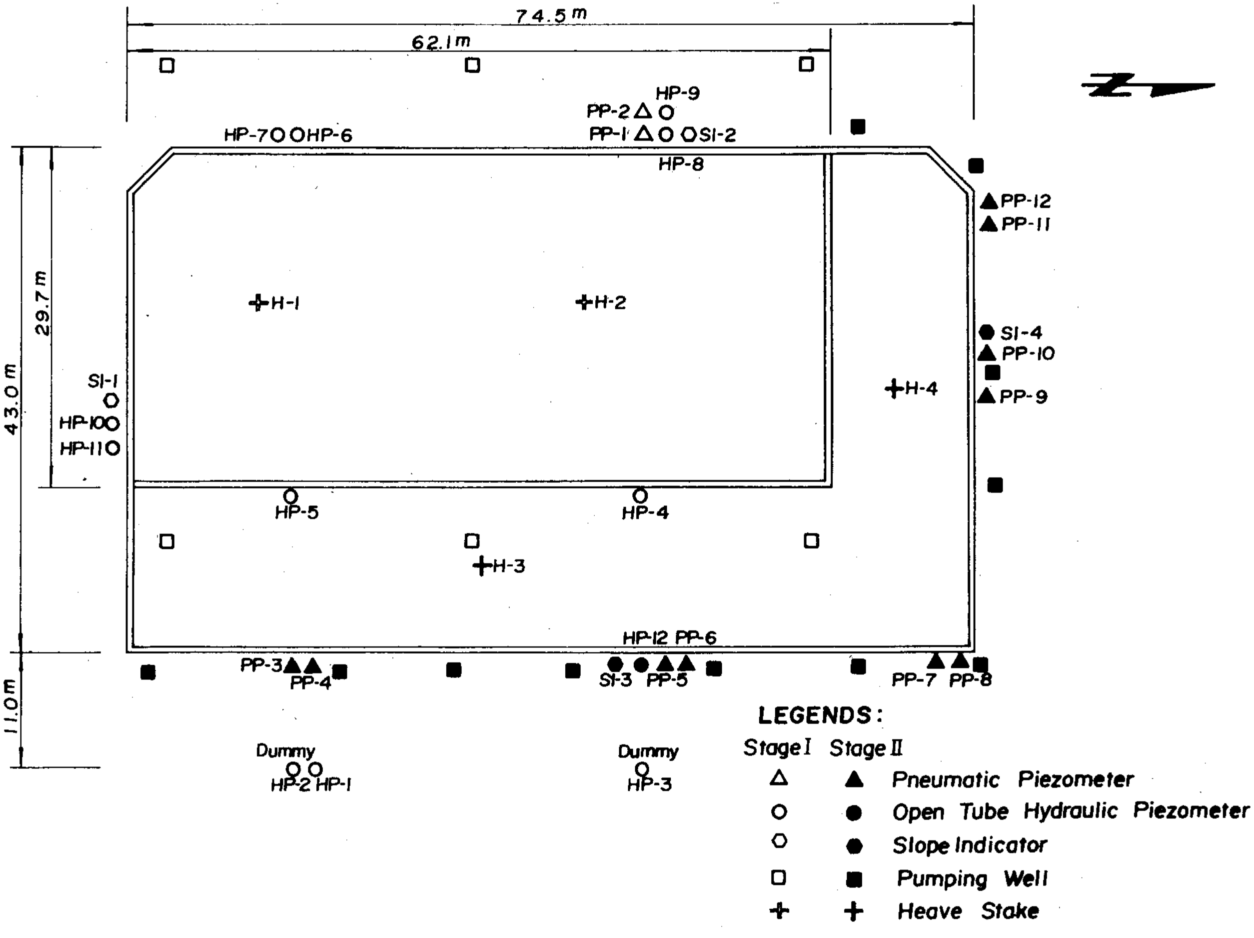


Fig. 1 Plan View of Building Layout and Location of Instrumentations

The subsoil conditions at the site are relatively uniform. Subsoil profile can be described as consisting of a thin layer, about 3 m thick, of soft, medium plastic clay underlain by a 6 m thick layer of fine to medium sand containing some gravels. This sand layer is loose at the top and becomes medium dense at deeper depths. Since the groundwater table was found to exist in this stratum and the excavation work would be carried out mainly in this layer, possibility of seepage and running sand was of concern. Underlying the sand is a 16 m thick layer of soft and compressible silty clay. The natural water content of the clay is very close to its liquid limit. Beneath the soft clay are alternative layers of medium dense to dense sand silty clay and clay with medium consistency. A typical soil profile with average design parameters is shown in Fig. 2.

INSTRUMENTATION

Five types of instrumentations were installed. They include inclinometers, piezometers, strut strain gauges, settlement points and heave stakes. The locations of the various instrumentations except the strain gauges are shown in Fig. 1. Table I lists the numbers and types of instrumentation used and the frequency of observations.

TABLE I SUMMARY OF INSTRUMENTATIONS

<table>
<tr><th rowspan="2" colspan="2">Item</th><th colspan="2">Number Installed</th><th rowspan="2">Type</th><th rowspan="2">Frequency of Observation</th></tr>
<tr><th>Stage I</th><th>Stage II</th></tr>
<tr><td colspan="2">Heave Stake</td><td>2</td><td>2</td><td>MAA</td><td>Before and after each step of excavation; Sometime once a day.</td></tr>
<tr><td rowspan="2">Settlement Points</td><td>Pavement & Other Buildings</td><td>38</td><td>32</td><td>MAA</td><td>Every 10 days.</td></tr>
<tr><td>Mat Foundation</td><td>23</td><td>Same as Stage I</td><td>MAA</td><td>Normally once a week; more frequent when needed.</td></tr>
<tr><td rowspan="2">Piezometers</td><td>Pneumatic</td><td>2</td><td>10</td><td>Soil Instruments Ltd.</td><td>Once every 2-3 days.</td></tr>
<tr><td>Hydraulic</td><td>11</td><td>1</td><td>MAA</td><td>Once or twice a day.</td></tr>
<tr><td colspan="2">Strut Strain Gauges</td><td>36</td><td>24</td><td>Geonor</td><td>Once or twice a day.</td></tr>
<tr><td colspan="2">Inclinometers</td><td>2</td><td>2</td><td>SINCO</td><td>Before and after each step of excavation, preload. Normally once a week.</td></tr>
</table>

DEPTH m	LOG	SOIL DESCRIPTION	N VALUE	γ_t t/m3	c	$\bar{c}$	ϕ	$\bar{\phi}$	m_v cm2/kg	C_v cm2/sec	q_u t/m2
					t/m2		DEG.				
2.0 3.0		Clay	4	2.0							
9.0		Fine to Coarse Sand with Gravels.	6	1.99		0		33.6			
28.0		Silty Clay	5	1.93	4.0	2.3	11.3	22.8	15.3×10^{-3} (8.4×10^{-3})*	1.9×10^{-3} (9.1×10^{-3})*	3.8
32.0		Silty Sand	8	2.0							
45.0		Sandy or Silty Clay	10	1.91	6.2	8.0	20	25	7.5×10^{-3} (3.1×10^{-3})*	7.5×10^{-3}	
54.0		Hard Clay	22	2.0					4.5×10^{-3} (2.7×10^{-3})*	22.0×10^{-3}	

* DATA IN PARENTHESES FOR RECOMPRESSION

Fig. 2 Typical Subsoil Profile and Design Parameters

RESULTS AND DISCUSSION

Heave Measurements

Measurements of heave of subsoils during excavation were made with surveying instrument by measuring the elevation of the heave stakes. For the Stage I work, the depth of excavation was 9.75 m, two heave stakes were buried at a depth of 13.3 m. The depth of excavation for the Stage II work varied from 8.3 m in the surrounding basement area and 9.2 m in the sewage tank area. Heave stakes were placed at 11 m depth.

From Fig. 3, which presents the results of heave measurements, it can be seen that heaving of subsoils reached the maximum values shortly after completion of the excavation work. The measured values are fairly close to those values estimated on the basis of laboratory test results. However, the estimated rates of heaving appeared to be more rapid than the actual values.

Settlement Measurements

Settlement of the main building structure

As described earlier, construction of the project was divided into two stages for the main building structure or the tower block, and the surrounding basement structure. The mat foundations of the two structure were connected with hinge connection. In order to limit the differential settlement of the hinge connection within the allowable value of 5.6 cm set by the structural design requirement, the following construction sequence was specified:

(a) To maintain the excavated area dry and to increase the effective stress in the subsoils below the foundation level, the groundwater level in the excavation area be lowered to 1 m below the excavated level.

(b) After completion of the two levels of basement, preload the structure by filling the tanks in the mat foundation with water. The water shall be pumped out after completion of the 12th floor.

(c) After completion of the 7th floor of the main structure, excavation for the surrounding basement shall commence.

(d) Graded coarse material shall be placed above the mat foundation outside the main structure.

Stability of the excavation and change of subsoil bearing capacity due to consolidation of the subsoils were constantly evaluated. During construction, due to problems of demolishing of houses situated on the site, the Stage II excavation was delayed until after the completion of the 11th floor of the main structure. It was found that there might be stability problem as well as bearing capacity problem when the Stage II excavation reached the depth of 6.7 m below the ground surface, the constructor was advised to pump out the water stored in the foundation. It was further

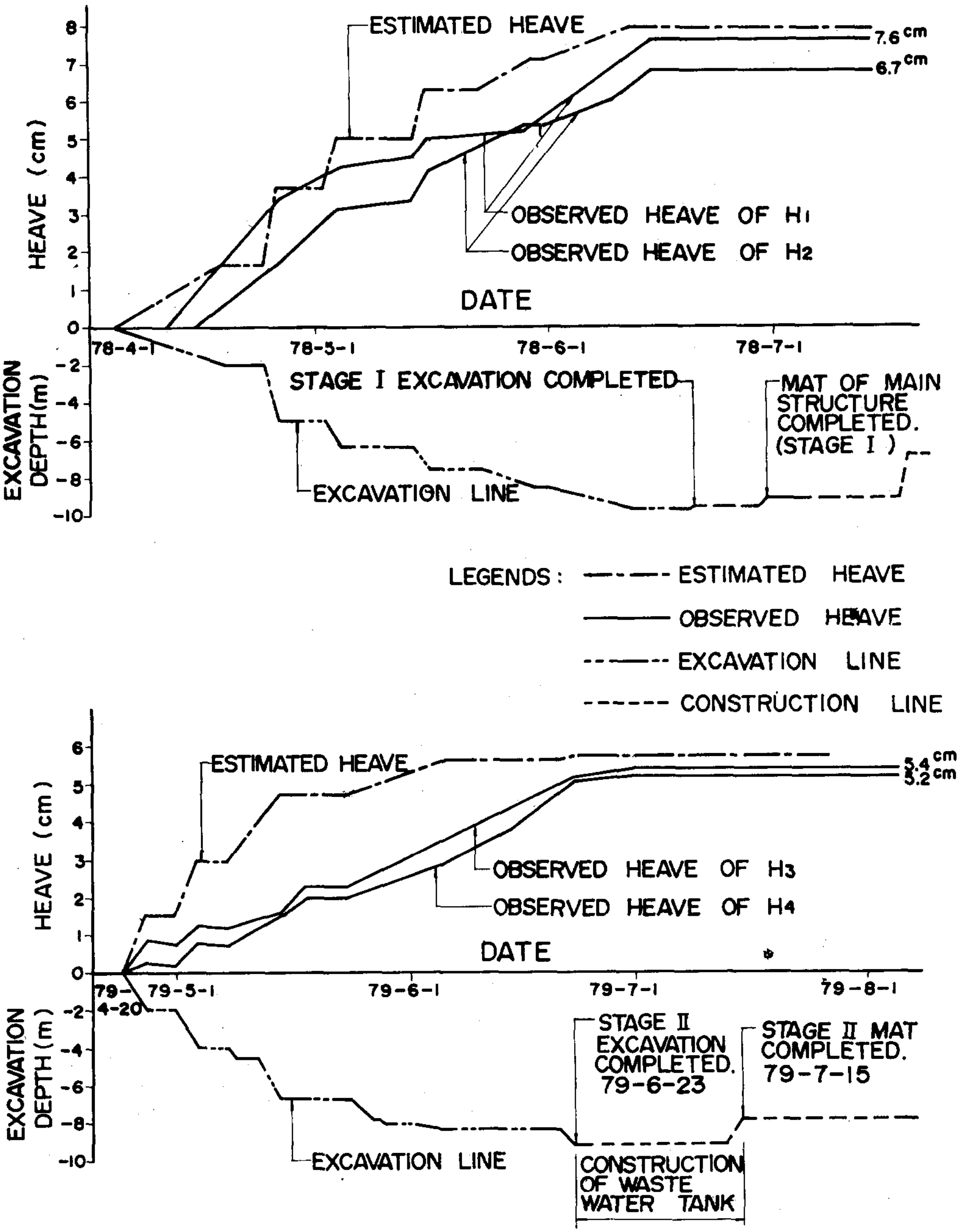


Fig. 3 Results of Heave Measurements

decided that the construction of the 13th floor be delayed until the completion of the Stage II mat plus gravel backfill and the roof floor was poured 7 days after the Stage II B1 floor was completed. These measures were taken to avoid excessive unbalanced loading acting on the subsoil and the possibility of extra shear force exerting on the connecting hinge.

According to results of settlement observation of the mat foundation of the main structure, the structure has a slight tilting towards the east and north, i.e. towards the Stage II excavation area. By adopting the construction sequence described above, differential settlement and tendency of tilting were greatly reduced. Figure 4 shows the settlement contours of the main mat foundation after the Stage I construction and after completion of the entire structure. Figure 5 shows the settlement and loading-time curve at the mid-point of main mat. The figure shows that values of settlement computed on the basis of one-dimensional consolidation of the underlying soil strata are generally larger than the observed values. This discrepancy is similar to results reported by many other investigators. Estimations were also made on the residual settlements five years after completion of the construction, Table II. The estimated differential settlement at the connecting hinge is 3 cm which is smaller than the allowable 5.6 cm used in the structural design.

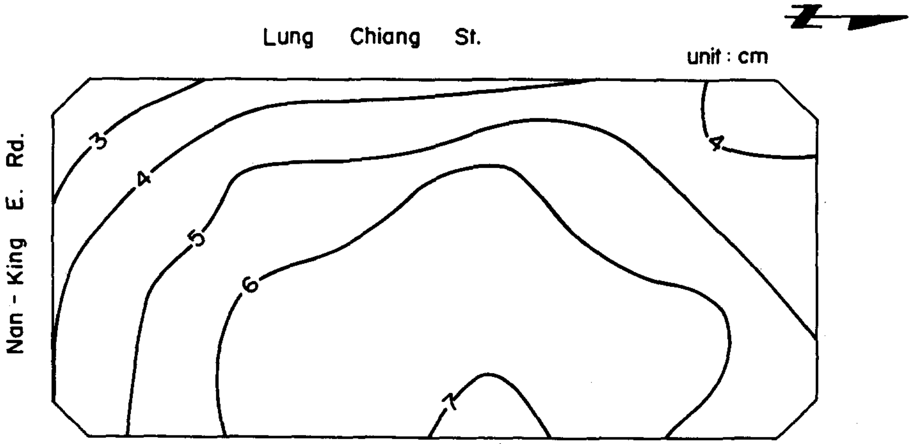
TABLE II ESTIMATED RESIDUAL SETTLEMENT 5 YEARS AFTER COMPLETION OF CONSTRUCTION

Settlement Point	Settlement at End of Construction, cm		Residual Settlement, cm	
	Estimated	Observed	Estimated	Estimated from Obs. Value
A	17.3	9.5	8.1	4.4
B	12.3	8.0	4.7	3.1
C	9.1	5.7	2.4	1.5

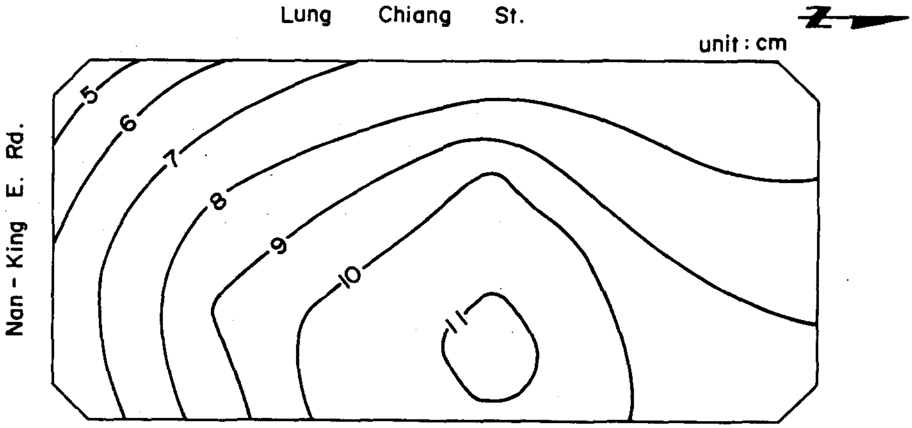
Settlement of surrounding building and streets

Settlement measurements of street pavements and buildings across the site indicate that the maximum settlement of the building across the Lung Chiang Street on the west side of the site reached 6.6 cm. The relative settlement between two adjacent settlement points was however only 1.5 cm, resulted in a differential gradient of only 1/687. No damage of the building was observed. On the other hand, cracks were found on the pavement surface of Nanking East Road which is along the south side of the site. This is partly due to groundwater and running sand seeping through voids between the prepakt piles on that side.

As shown by the soil profile, the groundwater table at the site is very high. Dewatering was required during construction of the substructure which lasted for a period of about 20 months. The maximum settlement of the



(a) Contours of Mat Settlement after Completion of 11th Floor and at Beginning of Stage II Excavation. (79-5-9)



(b) Contours of Mat Settlement after Completion of Tower Block. (80-2-3)

Fig. 4 Settlement Contours of the Main Mat Foundation

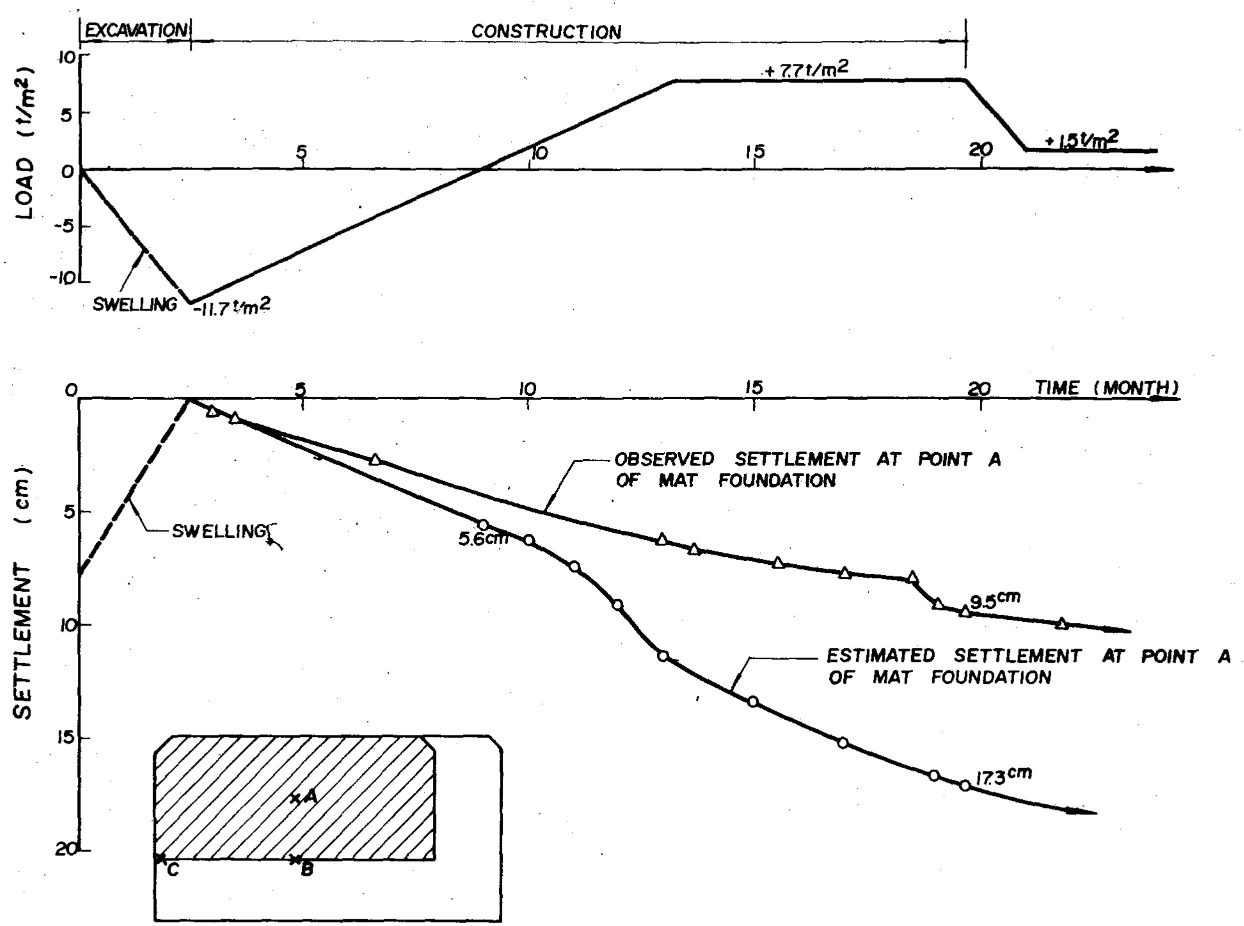


Fig. 5 Settlement and Loading-Time Curve at the Mid-Point of Main Mat

street due to the long term dewatering was about 5 cm. It was estimated that the influence of dewatering extended to a distance of about 100 m beyond the site.

Groundwater Level and Piezometric Pressure

The present site is located in the Taipei Basin. It has been reported (for example, MOH and OU, 1979) that due to deep well pumping, the groundwater pressure distribution in the Taipei Basin is not in static condition. Observation of the groundwater pressure variation is an important item for subsurface construction. A total of 24 piezometers was installed at the site. Pneumatic piezometers were used for their fast response and hydraulic piezometers were adopted for economy.

The original design of the dewatering system utilized pumping wells, about 12 m deep, the locations of the wells are shown in Fig. 1. However, due to obstructions of some of the wells, additional vertical and inclined wells points were installed. Additional problems were encountered during the dewatering process due to seepage of water or sand between the prepakt piles around the excavation. This problem is caused partly by poor workmanship of the prepakt piles and partly due to the ineffectiveness of grouting operation between the prepakt piles. Data obtained from the piezometer readings such as illustrated in Fig. 6 were found extremely useful in controlling pumping rate and degree of lowering of the groundwater level.

Strut Loads

The internal bracing systems for the excavations consisted of 3 levels of steel H-beams. The sizes of the H-beams were selected according to the earth pressure distribution diagram shown in Fig. 7. All struts were prestressed to about 15 to 20% of the design load after installation.

Thirty sets of vibrating wire strain gauges were installed at various locations of the H-beam bracing for the purpose of monitoring variation of the strut loads with excavation. At each location, gauges in pair were mounted on each side of the web of the beam for checking possible bending of the beam. The average strut loads and their standard deviations measured at various stages of excavation and construction are shown in Fig. 8. Data indicate that all the measured loads were less than the design allowable loads. This was particularly true for the first level S1 struts. On the basis of the measured average loads, an earth pressure distribution diagram was constructed as shown in Fig. 7. It appears that for the subsoil conditions at the present site, the use of TERZAGHI and PECK's (1967) method to estimate earth pressure distribution gives reasonably accurate result.

Lateral Displacement of Retaining Structure

One of the major problem during deep excavation is lateral displacement of the earth retaining structure. Lateral displacement would not only affect the stability of the excavation but would also affect safety of surrounding areas. Excessive lateral displacement will cause settlement of nearby buildings and roadways. For the present project, two inclinometer wells were installed immediately outside the prepakt piles for each of the

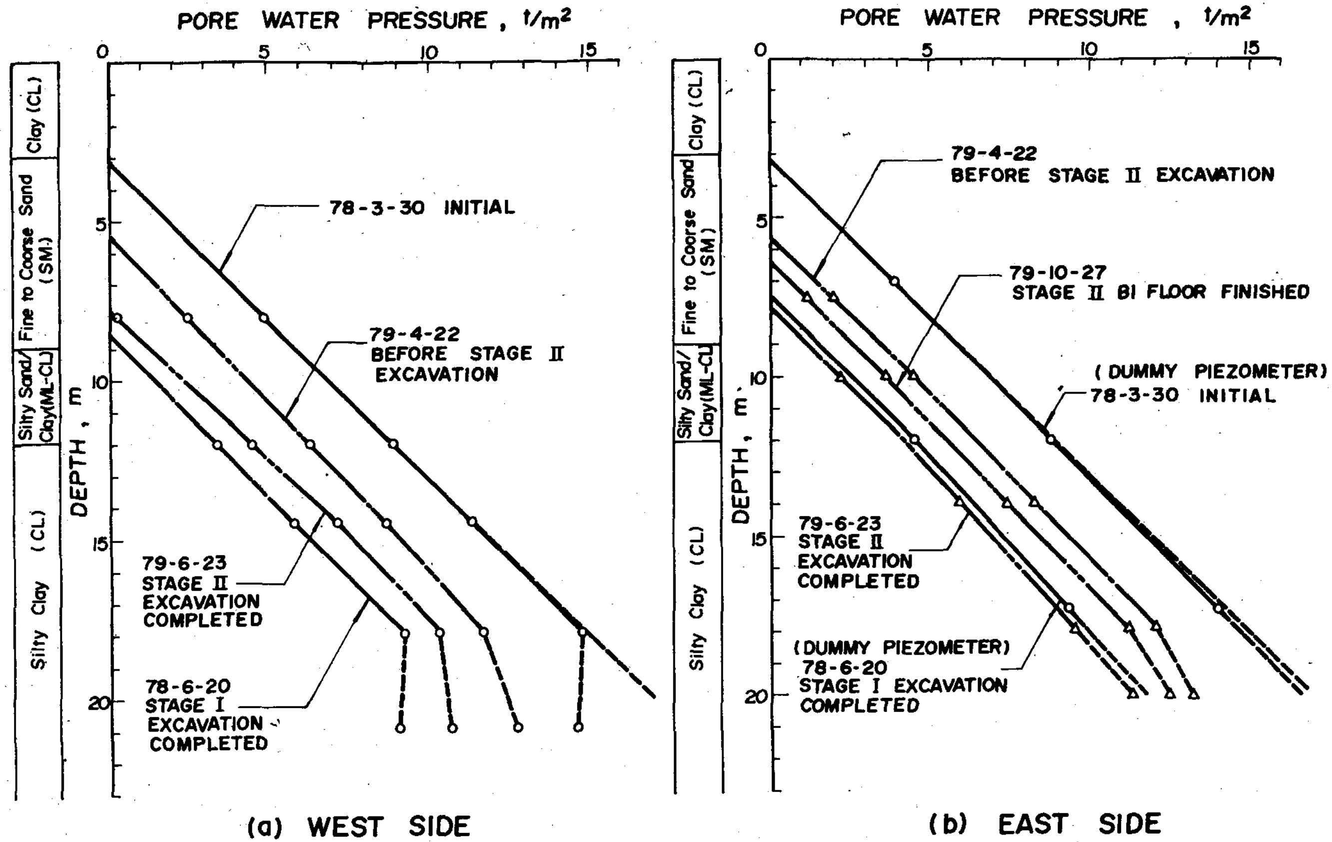


Fig. 6 Groundwater Piezometric Levels at Various Stages of Construction

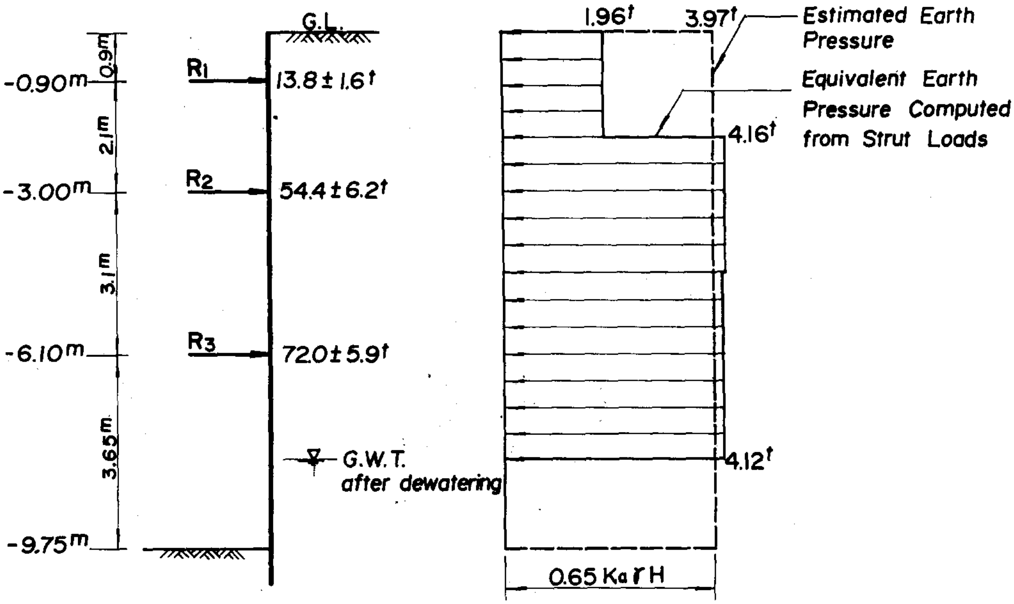


Fig. 7 Estimated and Measured Earth Pressure Distribution Diagram

two stages of excavation (Fig. 1). The lateral movements were measured by means of a SINCO inclinometer. Measurements were made prior to and after each step of excavation, prestressing the struts and after removal of the struts. The records, as illustrated in Fig. 9, indicate that the top part of the earth retaining prepakt piles moved outward during prestressing of the internal bracing, and moved inward, i.e. towards the excavation, during excavation. The inward displacement continued until and completion of the B1 level concrete floor. On 2nd September 1978, there was an earthquake of magnitude 0.06 g*. No detrimental effect was indicated by the monitoring system, probably because at that time the pouring of B1 floor has already been finished.

For the entire excavation, lateral displacements and deflections of earth retaining structure on the long sides were larger than those on the short sides. The maximum displacement occurred on the Lung Chiang Street side with a maximum distortion (ratio of deflection to length) equal to 1/807. This is much less than the allowable distortion of 1/240 for structural members subject to bending according to ACI Code (ACI, 1977). Checking the structural design of the prepakt piles, it was found that at certain locations the piles have developed plastic hinges. However, the overall safety of the retaining system was not affected due to the support

*Seismic intensity of the earthquake was within the Modified Mercalli's Scale VII.

Design Loads (tons)

Strut	Elevation m	Allowable Load
S1	-0.9	62
S2	-3.0	105
S3	-6.1	175

Design Loads (tons)

Strut.	Elevation m	Allowable Load
S1	-0.6	62
S2	-3.1	105
S3	-6.1	105

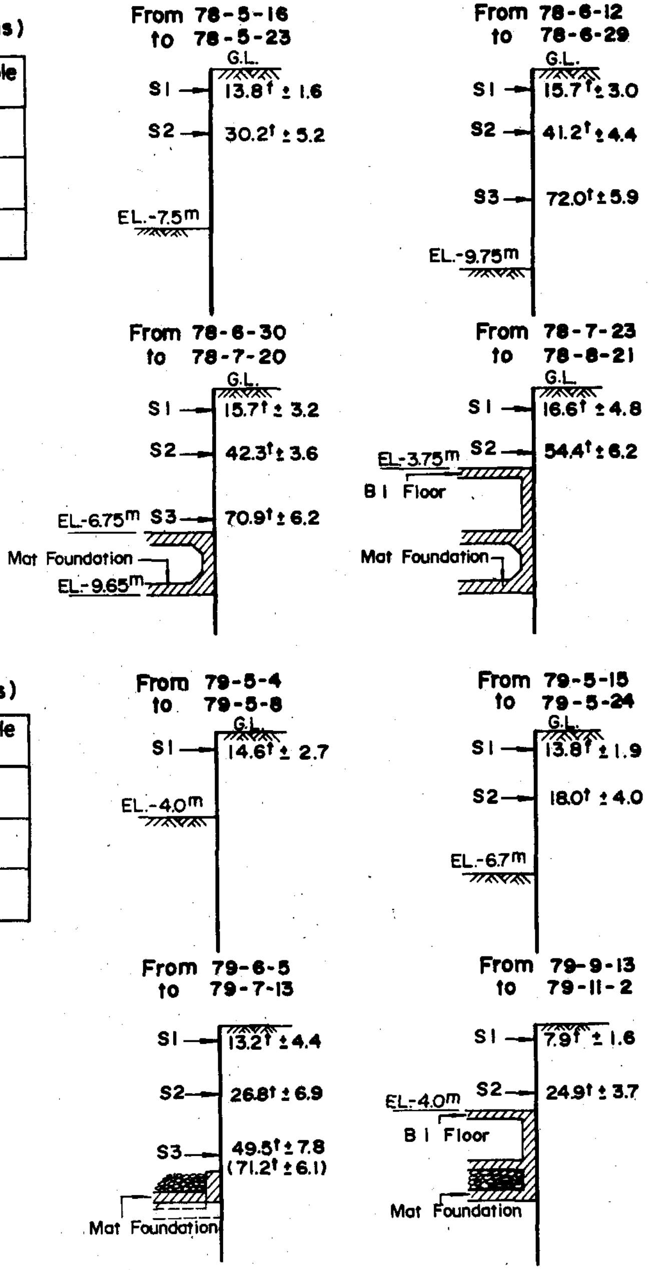


Fig. 8 Measured Strut Loads

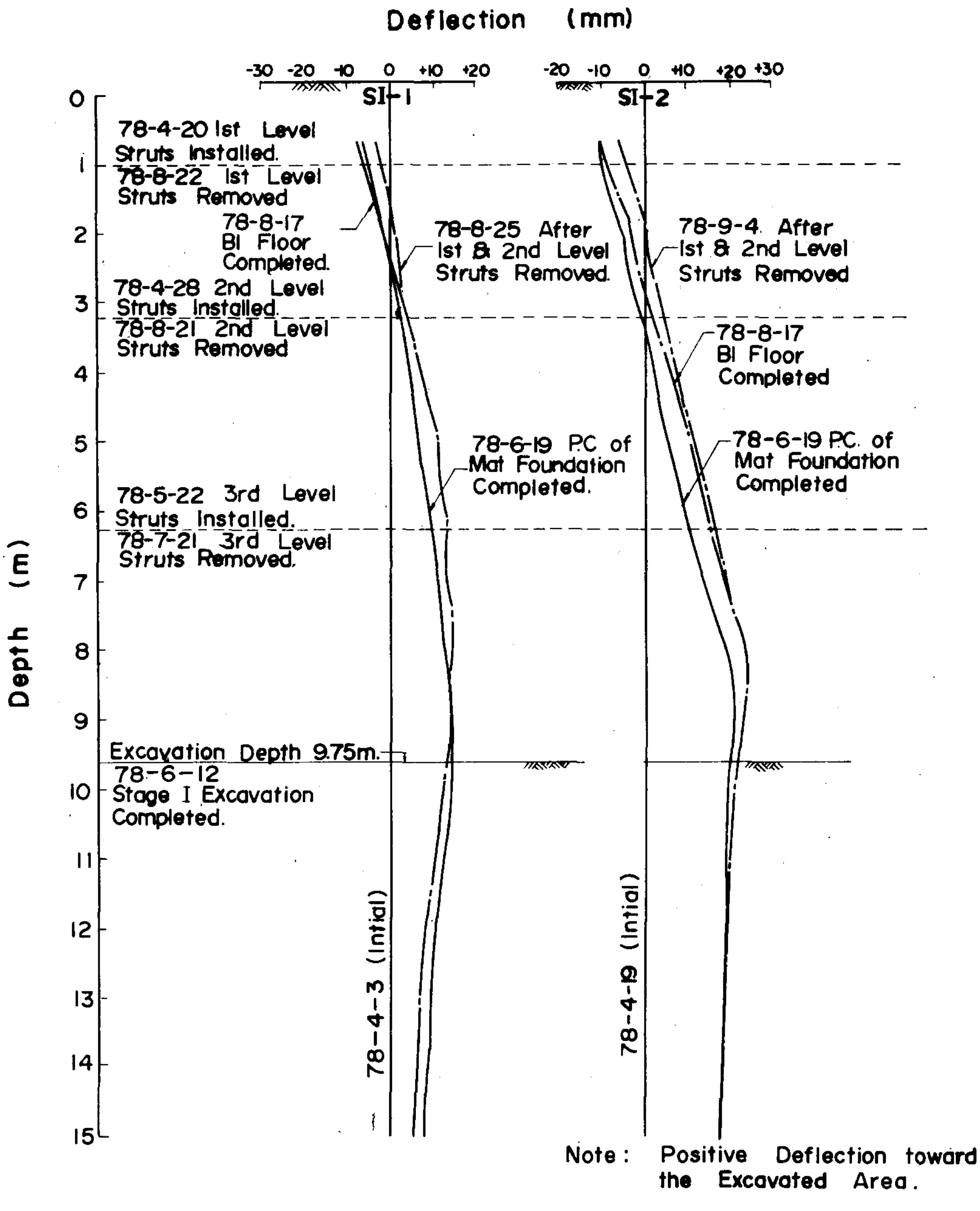


Fig. 9 Illustration of Lateral Movement of Earth-Retaining Structure

of the internal bracing system.

CONCLUSIONS

To achieve a successful construction, it is necessary to have good coordination between construction and design. This is particularly critical for deep excavation. This case report fully demonstrates the importance of proper construction management and the usefulness of instrumentation monitoring. There were several major problems encountered during the construction which were successfully resolved due to information supplied by the instrumentation. Some of the more important ones are:

(1) Seepage of groundwater and running sand through voids between prepakt piles affected dewatering operation and caused settlement of surrounding area.

(2) Loading of the main structure during construction approached the bearing capacity of the subsoil. Adjustment of construction schedule was made.

(3) During the Stage II excavation, the mat foundation of the main structure developed a tendency of slight tilting due to differential settlement as revealed by instrumentation data. Construction sequence was adjusted to alleviate this problem.

(4) The project involved a two stages construction. Hinged joints were designed to connect the mat foundation of the main structure and the mat for the surrounding basement structure. Special control of the differential settlement was exercised with the aid of instrumentation.

(5) Due to limitation of budget, it was not possible to install pressure cells to measure the in situ earth pressures acting on the retaining structure. In designing of retaining structure, active and passive earth pressures were used. It is obvious that certain amount of lateral movement of the retaining structure must occur in order to develop the active condition. Consequently, the ground surface of the surrounding area settled. This phenomenon was adequately shown by the inclinometer and settlement measurement data. It has been suggested that earth pressure at rest should be considered as the design criterion. However, the magnitudes of earth pressure used in the design have significant affect on the cost of the retaining system. In order to have better understanding of the actual earth pressure developed in relation to lateral movement of the earth retaining structure, pressure cells should be installed in conjunction with other instrumentations. Only after collection and evaluation of sufficient data, the problem of over-design or under-design can be resolved.

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REFERENCES

MOH, Z.C. and OU, C.D. (1979), Building Foundation Practice in Taiwan, Proc. Sixth Asian Reg. Conf. on S.M.F.E., Singapore, Vol. II, pp. 205-209.

TERZAGHI and PECK (1967), Lateral Supports in Open Cuts, Soil Mechanics in Engineering Practice, 2nd Ed., John Wiley and Sons, Inc., New York, pp. 394-412.

AMERICAN CONCRETE INSTITUTE (1977), Strength and Serviceability Requirements, Building Code Requirements for Reinforced Concrete (ACI 318-77), Detroit, Michigan, pp. 30-31.