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Tall Building Foundation Design – the 151 Story Incheon Tower

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TALL BUILDING FOUNDATION DESIGN – THE 151 STORY INCHEON TOWER

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ABSTRACT

The 151 storey super high-rise building is located in an area of reclaimed land constructed over soft marine clay in Songdo, Korea and is currently under design. This paper describes the design process in developing the foundation system of the supertall tower.The foundation design process described includes the initial stages of geotechnical site characterization using the results of investigation boreholes and geotechnical parameter selection, and a series of detailed two- and three-dimensional numerical analysis for the Tower foundation comprising 172 bored piles of varying length using finite element and boundary element methods. This paper will also provide a summary of the vertical and lateral pile load testing programs under both static and cyclic loading.

INTRODUCTION

The proposed 151 story Multi-use Incheon Tower, illustrated in Figure 1, is located in district 8 of the Songdo Incheon Free Economic Zone, and its design is currently underway. The site lies entirely within an area of reclamation underlain by up to 20m of soft to firm marine silty clay, which in turn overlies residual soil and a profile of weathered rock. The tower is composed of approximately 30 stories of office floors, 8 stories of hotel and other supporting facilities, 100 stories of residential floors, and several levels of mechanical plant. The base of the tower consists of retail, a future subway station, and several levels of parking. It is anticipated that the total area of the tower and the base for Phase 1 construction will be approximately 412,000 square meters. The structural system of the tower in the east-west direction consists of a reinforced concrete core wall system linked to the exterior mega columns with reinforced concrete or composite shear panels to maximize the effect of the structural depth of the tower. However, the lateral load resisting system of the tower in the north-south direction consists of a mega-frame structure, where the reinforced concrete core walls, for each side of the tower, are linked through multi-story structural steel trusses at 3 levels, at approximately every 30 floors. The tower superstructure is founded on a pile supported raft foundation. The 5.5 meter thick reinforced concrete raft is supported on a total of 172 bored piles, 2.5 meters in diameter, with variable lengths, extending 5 meters into soft rock for added stiffness and axial load capacity.

The foundation system is required to support the large vertical loads due to gravity and lateral loads and to restrain the horizontal displacement of the tower due to wind and seismic loading. The behavior of the foundation system influences the design of the building super structure, and potentially the lateral drift of the tower, which is highly dependent on the foundation system flexibility. Therefore, the foundation design needs to consider the interactions between the soil, foundation and super structure.

Figure 1. 151 story Incheon Tower – Architectural Rendering

In this paper, the overall foundation system design process is described, and the outcomes of the design process are presented. A summary of the full scale vertical and lateral pile load testing programs is also given.

GROUND CONDITIONS

The Incheon area has extensive sand/mud flats and near shore intertidal areas. The site lies entirely within an area of reclamation, which is likely to comprise approximately 8 meters of loose sand and sandy silt, constructed over approximately 20 meters of soft to firm marine silty clay, referred to as the Upper Marine Deposits (UMD). These deposits are underlain by approximately 2 meters of medium dense to dense silty sand, referred to as the Lower Marine Deposits (LMD), which overlie residual soil and a profile of weathered rock.

The lithological rock units present under the site comprise granite, granodiorite, gneiss (interpreted as possible roof pendant metamorphic rocks) and aplite. The rock materials within about 50 meters from the surface have been affected by weathering which has reduced their strength to a very weak rock or a soil-like material. This depth increases where the bedrock is intersected by closely spaced joints, and also sheared and crushed zones that are often related to the existence of the roof pendant sedimentary / metamorphic rocks. The geological structures at the site are complex and comprise geological boundaries, sheared and crushed seams - possibly related to faulting movements, and jointing. A diagrammatic geological model is presented in Figure 2.

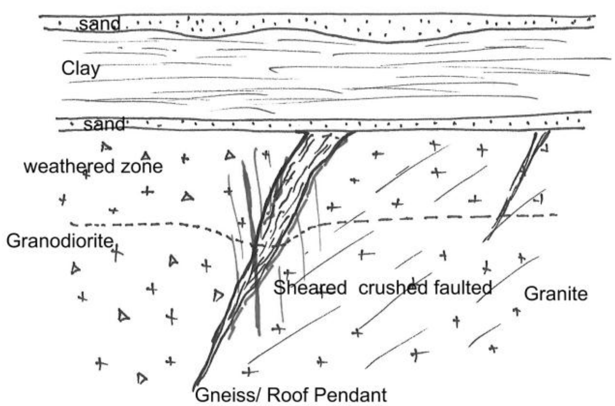

Figure 2. Diagrammatic Geological Model

From the available borehole data for the site, inferred contours were developed for the surface of the “soft rock” founding stratum within the tower foundation footprint. These are reproduced in Figure 3. It can be seen that there is a potential variation in level of the top of the soft rock (the pile founding stratum) of up to 40m across the foundation.

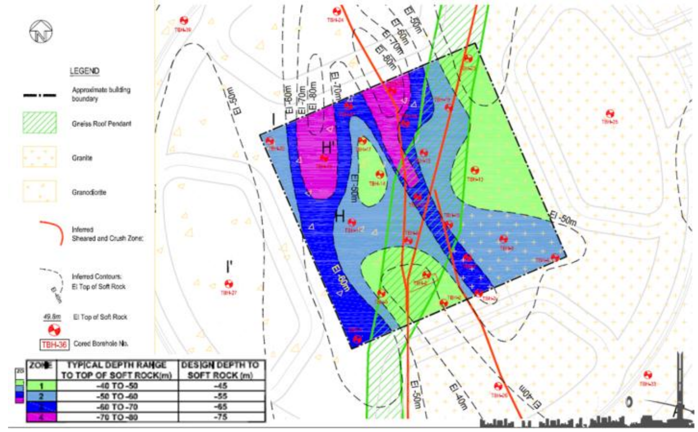

Figure 3. Inferred Contours of Top of Soft Rock

FOUNDATION DESIGN PROCEDURE

Generally, high-rise buildings on weak ground in Korea are supported on foundation systems comprising large diameter reinforced concrete bored piles socketed into rock and tied to a raft foundation. Adjacent to the Songdo 6 & 8 development site, a very large development with high-rise buildings and long span cable stayed bridges has been constructed on reclaimed land with soil conditions similar to those encountered at the 151 story Inceon tower at the Songdo site. All the high-rise building projects and the long span cable stayed bridges are founded on pile-supported rafts or pile caps. Therefore, this type of foundation was also considered to be the likely option for the tower at concept design stage, and so the design plan, including the scope of the ground investigation, was generally focused on this foundation system.

The foundation design process adopted for the tower comprised the following three main stages: Stage 1 – Concept Design; Stage 2 – Detailed Design, and Stage 3 – Post Design (testing and monitoring). These three stages are briefly described in the following sections.

CONCEPT DESIGN

The aim of the Concept Design was to firstly establish the foundation system and to evaluate the approximate foundation behavior, based on a simplified ground model developed from the available geotechnical data. From this stage of the design, the following foundation design details were provided to the tower structural designers for preliminary design purposes:

- Pile capacities (geotechnical & structural) for a range of pile diameters.
- Horizontal and vertical pile stiffness values (single pile & group) for a range of pile diameters.

Using this information, the structural designers commenced the preliminary structural design process by including the different raft and pile layouts in the 3-dimensional finite element structural analysis model, in order to account for the effects of soil/structure interaction The foundation system development included the following:

- Development of pile layout options for various pile diameters.
- Preliminary selection of raft size (plan dimensions and thickness).
- Preliminary evaluation of building performance, under gravity and lateral load effects.
- Assessment of the pile group efficiency.
- Assessment of the foundation stiffness and its impact on the overall behavior of the tower.
- Assessment of the superstructure stiffening effects on the load distribution among the piles.

Based on the above, several foundation layout options were developed for further assessment and refinement at the detailed design stage.

DETAILED DESIGN

The three main components to be considered in the detailed design stage of the tower foundation system are shown in Figure 4 and are discussed in the following sections.

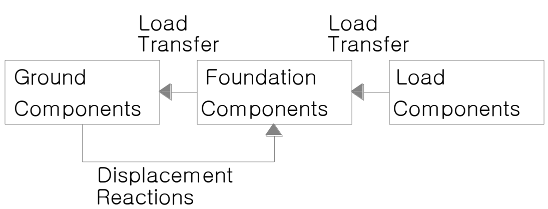

Figure 4. Main Components of Foundation Analysis

LOAD COMPONENTS

The building loads can be classified according to their source or loading characteristics with direction. Figure 5 depicts the tower raft foundation configuration, core wall, and mega column layout at the tower raft level.

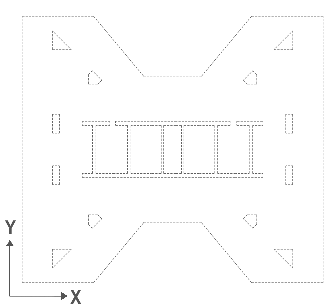

Figure 5. Tower Basement Floor Plan

The typical loads of the tower are summarized as follows:

- Vertical Load, P_z (Dead Load +Live Load) = 6622MN
- Lateral Load, P_x (Wind Load) = 146MN, P_y(Wind Load) = 112MN
- Lateral Load, P_x (Seismic) = 105MN, P_y(Seismic) = 105MN
- Overturning Moment, M_x(Wind Load) = 12578MNm, M_y(Wind Load) = 21173MNm
- Torsional Moment, M_z(Wind Load) = 1957MNm.

The load combinations provided by the structural designers were adopted for the geotechnical design of the foundation system. Comprehensive seismic analyses were performed for the tower and the foundation system, including response spectrum and time history analyses, for both frequent and extreme seismic events. However, wind load still controlled the overall tower design, and characteristically for super high-rise buildings, the wind load is a critical load case for both the building foundation and the superstructure. The wind load combinations of P_x, P_y and M_z are dependent on the wind direction, wind speed and the building shape, and can be determined from analysis or wind tunnel tests. Some 24 wind loading combinations were provided by the structural designer in the following format:

$$AP_x + BP_y + CM_z \qquad (1)$$

where A, B and C are factors applied to the various load components. Some examples of these factors are shown in Table 1.

Table 1. Examples of Wind Load Combination

Load Case	***A***	***B***	***C***
4	+100%	-45%	-70%
7	-90%	-60%	+40%
11	+45%	-100%	+30%
20	+70%	-40%	-100%

In addition to the wind and seismic loading described above, detailed site specific seismic hazard studies were performed that included the effects of near and far earthquakes, including the potential for liquefaction of the reclaimed soil. The tower foundation system is to be located below the reclaimed soil and the tower superstructure will be separated from the podium structure to reduce interaction between the podium structure and the tower structure. In addition, most of the podium structure is located above the water table to avoid the possible effects of liquefaction. While the seismic and wind engineering management approaches are very critical in determining the foundation and structural design concepts, they are not the focus of this paper. Attention will focus on the design and behavioral characteristics of the piles, including strength and stability under combined axial load/bending moments/shear forces, and the effects of the soft clay on their resistance to lateral loads from extreme wind and seismic events.

FOUNDATION COMPONENTS

The raft size and thickness was originally assessed by the structural designers based on the loading conditions, the pile layouts and the structural demands on the raft foundation to transfer the loads to the piles in the most effective manner and with due consideration given to the presence of deep elevator pits and other architectural requirements.

The size and number of piles, and their layout, were developed from a series of trial analyses undertaken collaboratively by the geotechnical and structural designers. The pile layout and raft foundation thickness were optimized to allow for even load distribution between the piles, to minimize the overall and differential settlements, and to minimize the shear and bending moments in the raft. The founding depth of each pile within the group was assessed by the geotechnical designer, considering both the pile performance and capacity. The preferred raft and pile layout was selected from the various options developed during the concept design stage, and comprised a 5.5 meter thick raft, founded at a level of EL-8.7m, supported on a total of 172 reinforced concrete bored piles 2.5 meters in diameter founding a minimum of 2 pile diameters into the soft rock, or below EL-50m, whichever was deepest. The final selected pile layout is presented in Figure 6. In locations where the piles are expected to be in the vicinity of sheared/crushed rock zones, the piles will be founded at a rock level below the sheared zones whenever possible, in order to bridge the weak soft layers of soil and to "stitch" the different layers to allow for transfer of the loads into the rock in an efficient manner to achieve a satisfactory performance of the overall foundation system.

GROUND COMPONENTS

A detailed interpretation of the geological and geotechnical conditions based on the available comprehensive ground investigation (Halla 2008) was undertaken in order to:

- Assess anticipated ground conditions for the tower
- Develop geotechnical properties and characteristics for the various strata
- Develop geotechnical design parameters.

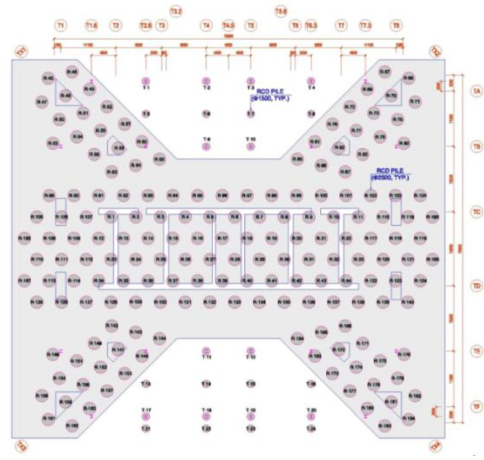

Figure 6. Pile Layout Plan

The footprint of the tower was divided into eight zones which were considered to be representative of the variation of ground conditions and geotechnical models were developed for each zone. Appropriate geotechnical parameters were selected for the various strata based on the available field and laboratory test data, together with experience of similar soils on adjacent sites. One of the critical design issues for the tower foundation was the performance of the soft UMD under lateral and vertical loading, hence careful consideration was given to the selection of parameters for this stratum. Typical parameters adopted for foundation design are presented in Table 2.

Table 2. Typical Geotechnical Design Parameters

Stratum	***E_v*(MPa)**	***E_h*(MPa)**	***f_s*(kPa)**	***f_b*(MPa)**
UMD	7 - 15	5 -11	29 - 48	-
LMD	30	21	50	-
Weathered Soil	60	42	75	-
Weathered Rock	200	140	500	5
Soft Rock (above EL-50m)	300	210	750	12
Soft Rock (below EL-50m)	1700	1190	750	12

E_v = Vertical Modulus $\quad$ f_s = Ultimate shaft friction
E_h = Horizontal Modulus $\quad$ f_b = Ultimate end bearing

MAIN DESIGN PROCESS

Once the three components of loading, foundation layout and ground conditions were reasonably well defined, the foundation design could be undertaken. The key issues that needed to be addressed in the foundation design were as follows:

1. Ultimate capacity and global stability of the foundation system under vertical, lateral, and overturning moment load combinations.
2. The influence of the cyclic nature of wind and earthquakes on foundation capacity and movements.
3. Overall foundation settlements
4. Differential settlements, both within the tower footprint, and between high-rise and low-rise areas.
5. Possible effects of externally-imposed ground movements on the foundation system, for example, movements arising from ongoing consolidation settlement of the UMD.
6. Earthquake effects, including the response of the structure-foundation system to earthquake excitation, and the possibility of liquefaction in the soil surrounding and/or supporting the foundation.
7. Dynamic response of the structure-foundation system to wind-induced and seismic forces.
8. Impact of the foundation stiffness on overall foundation rotation under wind and seismic dynamic/cyclic loadings, which has direct impact on the overall drift of the supertall and slender towers.
9. Structural design of the foundation system; including the load-sharing among the various components of the system (i.e. the piles and the supporting raft), and the distribution of loads within the piles. For this, and most other components of design, it is essential that there be close cooperation and interaction between the geotechnical designers and the structural designers.

POST DESIGN STUDIES

During the main design stage, the pile design is generally based on numerical analyses and previous experience in similar conditions at adjacent sites. Pile load test data is invaluable in confirming design assumptions and finessing the foundation design. When the piles are instrumented, detailed information can be derived on the distributions of shaft friction and soil stiffness at various depths along the pile shaft. Therefore, a comprehensive vertical, lateral and cyclic pile load testing program was developed and executed for the tower foundation piles. In addition, monitoring of the piles and foundation raft behavior during construction of the superstructure is planned to be carried out in order to assess overall behavior of the foundation and compare with predicted performance, as well as providing valuable information to the structural designer regarding the anticipated final behavior of the superstructure itself.

The objectives of the pile load tests are shown in Table 3 below and can be summarized as follows:

- To assess and confirm the constructability and integrity of the piles using the proposed construction techniques (reverse circulation drilled piling techniques).
- To allow comparison of measured pile performance with design expectations and refinement of the geotechnical parameters adopted in design (e.g. ultimate skin friction and end bearing values, pile foundation stiffness, and the effect of dynamic loading on the axial and lateral pile stiffness).
- To assess possible variability of pile performance in relation to variations in ground conditions across the foundation footprint.

ASPECTS OF THE DETAILED DESIGN STAGE

The challenge for the tower foundation design was to simulate the group interaction effects of the large pile group under vertical and lateral loading (including negative skin friction due to the consolidating soft UMD) so as to optimize the pile group design and provide accurate input parameters to the structural designer. In order to assess the performance of the piled raft foundation, a suite of foundation analyses were undertaken using both commercially available software and Coffey Geotechnics' in-house developed programs, as summarized in Table 4.

Overall Stability of Tower Foundation

When considering the overall stability of a piled raft foundation system under vertical, lateral and overturning moment loadings, conventional "text book" methods are generally not applicable or feasible. Therefore an assessment of the overall stability of the tower foundation was undertaken using Coffey's in-house computer program CLAP, which computes the distributions of axial and lateral deflections, rotations and axial and lateral loads and moments, at the top of a group of piles, subjected to a combination of vertical loads, lateral loads, moments, and torsion. The ultimate load combinations were applied in the analysis and the ultimate capacities of the piles were reduced by a geotechnical reduction factor of 0.65 (adapted from guidelines given in Australian Piling Code AS2159-1995). The contribution of the raft to the overall stability of the foundation was ignored and overall stability was satisfied if the foundation system did not collapse under these conditions. For the proposed foundation system comprising 172-2.5 meter diameter bored piles, the limit state requirements for overall stability of the tower foundation were satisfied for the six critical wind and seismic loading cases analyzed.

Table 3. Summary of Pile Load Tests

Test Type	Purpose	Loading Method	Monitoring Items
● **Vertical** (4 No. test piles)	● Estimation of the end bearing and shaft friction capacities within weathered/soft rock. ● Evaluation of the vertical pile stiffness ● Check of pile response and stiffness to due to static and dynamic/repetitive/cyclic loading such as wind and seismic loads	● Bi-directional load cells (O-cells) embedded at two locations in pile (1 in upper shaft and 1 close to pile toe)	● Pile movement of shaft and toe ● Stress, strain along piles. ● Pile stiffness under repetitive/cyclic loading due to wind and seismic loads
● **Horizontal** (1 No. test & 1 No. reaction pile)	● Evaluation of the lateral pile stiffness ● Lateral deformation characteristics of UMD around pile head ● Check of pile response and stiffens due to static and dynamic/repetitive/cyclic to loading such as wind and seismic load	● Loading of the test pile against a reaction pile (static & dynamic loading)	● Lateral load and displacement ● Pile deflections along the shaft ● Pile stiffness under cyclic/repetitive loading.

Table 4. Software Programs Employed for Foundation Design

Computer Program	Purpose of Analysis
PLAXIS 2D Foundation (axisymmetric analysis)	Preliminary assessment of overall settlement of tower foundation
PLAXIS 3D Foundation	Assessment of foundation under vertical and lateral loading
DEFPIG (University of Sydney)	Assessment of foundation under lateral loading
CLAP (Coffey Geotechnics)	Assessment of foundation under vertical, lateral, bending, and torsional loading
GARP (Coffey Geotechnics and University of Sydney)	Assessment of foundation under vertical and moment loading
ERCAP(Coffey Geotechnics)	Assessment of podium piles under lateral loading
ERLS (Coffey Geotechnics)	Assessment of ground behavior to seismic loading

<u>Tower Foundation Settlement</u>

An assessment of the Tower foundation settlement has been undertaken using the computer the Geotechnical Analysis of Rafts with Piles (GARP) program developed by Sydney University in conjunction with Coffey. GARP employs the boundary element method to calculate interactions between pairs of piles and between a pile and the raft and finite element analysis of raft behavior. GARP can take into account different pile types across the foundation assigning individual stiffness values and geotechnical capacities to each pile and has been successfully used by Coffey on numerous tall tower projects (Badelow et al, 2006); (Poulos & Davids, 2005).

The settlement of a pile group is always greater than the settlement of a corresponding single pile, as a result of the overlapping of the individual zones of influence of the piles in the group. One of the inputs therefore required by GARP is the pile group interaction factors (α) for a range of pile spacings. Appropriate interaction factors were assessed using Coffey's in-house program CLAP, adopting the following assumptions:

- Varying geotechnical models present across the site (8 models).
- Varying pile lengths (ranging from about 41m to 71m).
- A rigid boundary at the top of the Hard Rock at EL-86.5m.
- The interaction effects are negligible at a distance of 15 pile diameters from each pile.
- The elastic modulus between the piles is three times greater than that near the piles, due to smaller strain levels existing between the piles.

Using a simplified boundary element approach, CLAP computes the single pile flexibility values and the two-pile interaction factors for each pile type specified. When calculating the pile flexibilities, the analysis allows for non-linear pile-soil behavior by limiting the axial and lateral pile-soil pressures to the ultimate values specified by the user. Interaction factors are computed using a purely elastic analysis. The interaction effects of one pile on another pile are based on the elastic flexibility of the influencing pile, with non-linearity only being introduced for the effect of the influenced pile on itself.

Six load combinations were considered in the analysis and a summary of the assessed maximum and minimum settlement values together with the angular rotation of the foundation raft is presented in Table 5.

The maximum predicted settlement for all cases occured within the heavily loaded core area, with the maximum value occurring as a result of DL + LL loading combination. The largest angular rotation of 1:570 occurred under Wind Load Combination 11, and was considered to be within the range generally acceptable for tall structures. It should also be noted that the analyses undertaken did not consider the stiffness of the superstructure, which is likely to be a conservative assumption, as the superstructure will provide additional stiffness to the foundation system and thus reduce the differential settlement. In addition, this analysis did not take into account additional stiffness due to the dynamic nature of wind and seismic loads, which can be significant.

Table 5. Summary of Predicted Vertical Settlement due to combined gravity and wind loads

Load Case	**Wind Load Combination**	**Settlement (mm)**		**Maximum Angular Rotation of the Raft**
		Max.	**Min.**	
DL + LL	-	67	28	1:790
0.75(DL + LL + WL)	1	52	18	1:730
0.75(DL + LL + WL)	4	52	18	1:730
0.75(DL + LL + WL)	7	53	18	1:740
0.75(DL + LL + WL)	11	55	19	1:570
0.75(DL + LL + WL)	15	54	19	1:570
0.75(DL + LL + WL)	20	52	20	1:870

DL = Dead Load, LL = Live Load, WL = Wind Load

An independent assessment of the tower foundation settlement under (DL + LL) loading condition was carried out using the 3-dimensional finite element program PLAXIS 3D Foundation developed by PLAXIS NL. The analysis assumed uniform ground conditions across the Tower foundation with the top of Soft Rock at EL-50m. All of the 172 piles were modeled with a toe depth of EL-55m and the top of the Hard Rock is assumed to be at EL-79m. The calculated maximum settlement of the tower foundation under the (DL + LL) loading condition was 68mm, occurring within the heavily loaded core area. This value compared very well with the value of 67mm assessed using GARP for the same location and under the same loading conditions. A differential settlement of about 19mm was calculated using PLAXIS 3D between the centre and perimeter of the tower foundation. This differential settlement was about 50% less than the value assessed using GARP (36mm). In the GARP analysis, the variation in ground conditions across the tower footprint and associated variations in individual pile lengths were modeled. Differences in the analysis methods and assumptions adopted therein could also contribute to the difference in the magnitude of the predicted differential settlement. Neither analysis model accounted fully for the stiffening effects of the tower superstructure during construction and under permanent and completed conditions.

Foundation Settlement

Critical input parameters for the 3-Dimensional Finite Element structural numerical analysis were the bored pile head stiffness values for the piled foundation. The assessment of these parameters is discussed in the following sections.

Assessment of Vertical Pile Behavior

The vertical pile head stiffness values for each of the 172 foundation piles under serviceability loading conditions (DL + LL) were assessed using the computer programs CLAP and GARP. CLAP was used to assess the geotechnical capacities, interaction factors and stiffness values for each pile type under serviceability loading for input into the group assessment. CLAP computes the distributions of axial and lateral deflections, rotations and axial and lateral loads and moments, at the top of a group of piles, subjected to a combination of vertical loads, lateral loads, moments, and torsion. GARP was used to assess the group foundation behavior of the Tower.

The computed individual pile vertical stiffness values ranged from about 600 MN/m near the centre of the foundation system to about 1300 MN/m near the corners. The analysis was non-linear, and therefore the higher stiffness values for the outer piles degraded more rapidly under loading than the central piles. The concentration of loads on outer piles within a group is a real phenomenon that has been measured in the field. Therefore, it was considered that foundation behavior could be simulated more realistically by using the individual pile stiffness values, rather than an average value for all piles within the group. Lower and upper bound estimates of pile stiffness values were provided to the structural engineers to include in their analysis, in order to capture the upper and lower bound behavior of the raft foundation and the potential impact on the tower superstructure.

Assessment of Lateral Pile Behavior

One of the critical design issues for the tower foundation is the performance of the pile group under lateral loading. Therefore, several numerical analysis programs were used in order to validate the predictions of lateral behavior obtained. The numerical modeling packages used in the analyses were:

- 3D finite element computer program PLAXIS 3D Foundation;
- Computer program DEFPIG developed by Sydney University in conjunction with Coffey;
- Coffey's in-house computer program CLAP.
- 3_D finite Element Structural Analysis Programs (MIDAS SET, ETABSs, SAFE) that included the effect of soil structure interaction.

PLAXIS 3D provided an assessment of the overall lateral stiffness of the foundation. The programs DEFPIG and CLAP were used to assess the lateral stiffness provided by the pile group assuming that the raft is not in contact with the underlying soil and a separate calculation was carried out to assess the lateral stiffness of the raft and basement. Table 6 presents the computed lateral stiffness for the piled mat foundation obtained from the analyses.

Assessment of Pile Group Rotational Stiffness

An assessment of the rotational spring stiffness values at selected pile locations within the foundation was undertaken using Coffey's in-house computer program CLAP. To assess the rotational spring constant at each pile location, the average dead load, horizontal load (x and y direction) and moment (about the x, y and z axes) were applied to each pile head. The passive resistance of the soil surrounding the raft, and the friction between the soil and the raft, were not included in the analysis as it was assessed that the base friction of the raft and the passive resistance of the soil on the raft would be relatively small when compared to lateral resistance of the piles. Table 7 presents a summary of the assessed rotational spring stiffness values obtained from the analysis for four piles considered to represent the range of values for different piles within the pile foundation.

Table 6 Summary of Lateral Stiffness of Pile Group and Raft

Horizontal Load (MN)	**Pile Group Disp. (mm)**	**Lateral Pile Stiffness (MN/m)**	**Lateral Raft Stiffness (MN/m)**	**Total Lateral Stiffness (MN/m)**
149	17	8760	198	8958
115	14	8210	225	8435

Table 7. Rotational Spring Constants Including Horizontal Loads Applied at the Pile Heads

Pile		**Pile Head Angular Rotation (rad.)**	**Pile Head Rotational Spring Stiffness (MN.m/rad)**
3	Maximum	0.094	2680
	Minimum	0.036	1380
27	Maximum	0.144	1750
	Minimum	0.056	903
70	Maximum	0.126	2000
	Minimum	0.049	1030
78	Maximum	0.187	1350
	Minimum	0.073	700

The overall torsional stiffness of the piled mat was assessed using the computer program PLAXIS 3D Foundation. A schematic of the PLAXIS model analyzed is given in Figure 7. The overall torsional stiffness of the piled mat estimated using PLAXIS was 10,750,000 MNm/radian, which is approximately equivalent to 16mm displacement at the edge of the raft for the applied torsional moment of 1956MN-m applied at the centre of the raft.

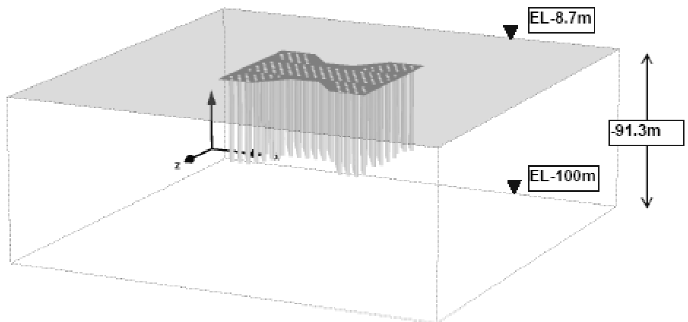

Figure 7. Schematic of PLAXIS 3D Model

Cyclic Loading due to Wind Action

Wind loading for the tower structure was quite severe, and therefore in order to assess the effect of low frequency cyclic wind loading, an assessment based on a method suggested by Poulos and Davids (2005) was undertaken. The method suggests that adequate foundation performance under cyclic loading should be achieved provided the following criterion is met:

$$\eta R_{gs}^{*} \geq S_{c}^{*} \qquad (2)$$

where: R_{gs}^{*}= design geotechnical shaft capacity

$_{c}^{*}$= half amplitude of cyclic axial wind-induced load

η= a factor assessed from geotechnical laboratory testing.

Provided the criterion is met, there is a reduced likelihood that full shaft friction will be mobilized, reducing the risk of degradation of shaft capacity due to cyclic loading. The factor η was selected to be 0.5, based on experience with similar projects. To assess the half amplitude of cyclic axial wind induced load, the difference in pile load between the following load cases was computed.

- CASE A: 0.75(DL + LL)
- CASE B: 0.75(DL + LL + WL_x + WLy)

where: DL = Dead Load; LL = Live Load

WL_x = Vertical Load resulting from *x*-Component of Wind

WL_y = Vertical Load resulting from *y*-Component of Wind

The difference in axial load between the two load cases is assessed to be the half-amplitude of the cyclic load (S_c^{*}). Table 8 below summarizes the results of the cyclic loading assessment and Figure 8 shows the assessed factor for each pile within the foundation system. The assessment indicates that degradation of shaft capacity due to cyclic loading in unlikely to occur.

Table 8. Summary of Cyclic Loading Assessment

Quantity	Value
Maximum Half Amplitude Cyclic Axial Wind Load S_c* (MN)	29.2
Maximum Ratio $\eta = S_c^{*}/R_{gs}^{*}$	0.43
Cyclic Loading Criterion Satisfied?	Yes

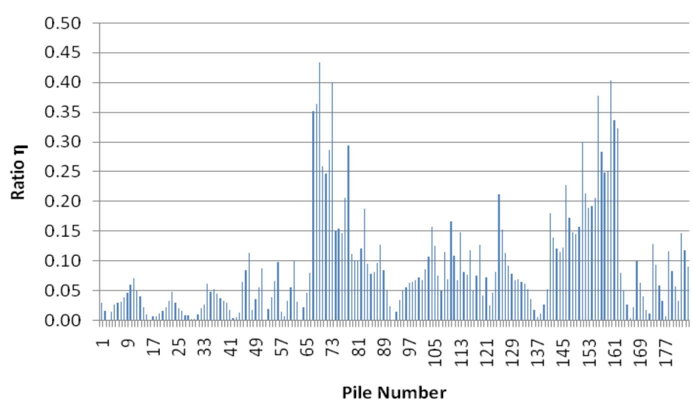

Figure 8. Results of Cyclic Loading Analysis

SEISMIC DESIGN

Earthquake Hazard

The first stage in assessing seismic response of the Incheon 151 Tower site was to undertake a seismic risk assessment and to obtain information on the general area around the site, based on historical and geological information. A desktop study was compiled by the Seismology Research Centre in Melbourne (associated with Monash University), which provided a review of earthquakes and earthquake hazard in the Incheon area. The desktop study defined hazard in terms of the ground motion recurrence at Incheon considering both nearby (within 100km) earthquakes in Korea, and the large distant (500 to 1000km) earthquakes along the very active tectonic plate boundary south and east of Japan. The total hazard was computed considering all of these earthquake sources. It was apparent that the nearby and distant source zones were quite distinct, and each was treated individually for the purpose of producing representative time histories for design purposes. The seismic return period adopted for the assessment was 2475 years, based on the Korea Building Code 2008. The peak ground acceleration was assessed to be 0.1g for local earthquake events and 0.024g for distant events.

Site Response Analysis

An assessment was undertaken of the seismic response spectra via an acceleration-time history analysis for the Incheon 151 Tower site. The assessment considered three surface levels – EL+2.3m, EL-2.5m, and EL-8.7m; and base level – EL-50m, to model bedrock excitation at the top of Soft Rock. The earthquakes were selected by the Seismology Research Centre and Coffey's in-house computer program ERLS (Earthquake Response of Layered Soils) was used to evaluate the response of the horizontally layered soil profile to the ground excitation resulting from the earthquakes at the base of the profile.

Some of the findings from the analyses were as follows:

- The amplification of acceleration from bedrock to surface was within the range of 0.8 to 2.4.
- There were pronounced peaks in the response spectra at natural periods of about 0.2s to 1s, which coincided approximately with the natural period of the ground profile.

Using the approach developed by Tabesh and Poulos (2001), the average inertial force on each pile was estimated from the computed maximum surface accelerations, and the maximum ground movements were also computed and applied to a typical pile to simulate the kinematic ground movement effects on the pile. The program ERCAP was used for these analyses. The maximum bending moment was found to be well within the structural capacity of the piles.

Liquefaction Assessment

A preliminary evaluation of liquefaction potential of the Incheon 151 Tower and Podium areas was carried out via conventional methods based on SPT values. This assessment was based on assumed parameters for the reclamation fill material, which were not available at the time of design. A conservative SPT value of 4 was therefore assigned to the fill.

The liquefaction potential at the Tower location was assessed to be low, but the reclamation fill at the adjacent podium location was assessed to be potentially liquefiable. It is decided that additional reinforcement could be incorporated in the upper section of the podium piles in order to carry the additional lateral loads resulting from possible liquefaction of the reclamation fill. This option was deemed preferable to undertaking additional ground treatment measures in the fill, as the lateral load imparted by the low-rise Podium structure to the supporting piles was assessed to be relatively small.

3-Dimensional finite element analysis

Independent 3-D Finite Element Analysis Models (FEAM) using the general analysis programs (MIDAS, ETABS, SAFE) were also performed to include the soil structure interaction and the stiffening effects of the superstructure. The analyses also included the construction sequence of the tower and allowed for more realistic load redistribution between the piles because of the significant stiffness of the superstructure.
The structural model allowed for the inclusion of the foundation rotation due to the pile flexibility on the overall drift and the dynamic characteristics of the tower, and the inclusion of different pile stiffnesses under dynamic/cyclic wind and seismic forces. The piles in the Midas analysis program were represented by springs with variable stiffness to simulate the pile stiffness computed from the geotechnical analyses. This type of analysis can be performed with several pile stiffnesses to study the impact on the overall foundation behavior and on the raft and key structural elements.

An optimum pile layout and a balance between axial pile stiffness and raft bending stiffness was reached, resulting in a reduction in raft foundation thickness from 5.5 meters to 4.5 meters.

The soil structure interaction model developed herein by Samsung will be used as a basis for correlating the actual foundation system behavior to that predicted for the tower during construction and for the permanent building conditions. An extensive monitoring program has been developed for the foundation system of the tower that will allow for measurement of the actual load distribution in some piles, the foundation settlement under the tower raft and across the site, and the strains in the raft. These data collected during construction will provide immediate feedback on the foundation stiffness, which in turn can be used for calibrating the overall structural analysis model and on the overall structural behavior during construction and under permanent building conditions.

SUMMARY OF PILE LOAD TESTS

A total of five pile load tests were undertaken, four on vertically loaded piles via the Osterberg cell(O-cell) procedure, and one on a laterally loaded pile jacked against one of the vertically loaded test piles. For the vertical pile test, two levels of O-cells were installed in each pile, one at the pile tip and another at between the weathered rock layer and the soft rock layer.

The cell movement and pile head movement were measured by LVWDTs in each of four locations, and the pile strains were recorded by the strain gauges attached to the vertical steel bars. The monitoring system is shown schematically in Figure 9.

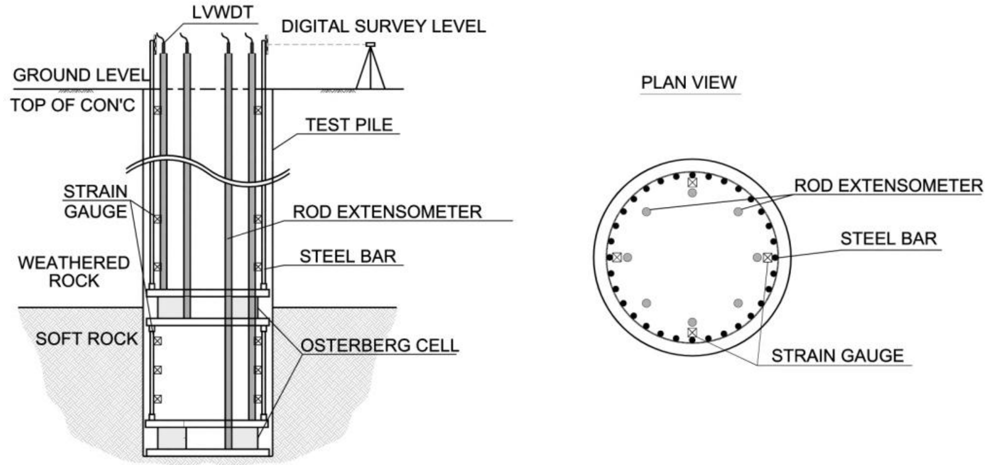

Figure 9. Schematic of Monitoring for Vertical Pile Load Test

The double cell test system was planned to obtain more accurate and detailed data for the main bearing layer, and so the typical test was performed in two stages as shown in Figure 10. Stage 1 was focused on the friction capacity of weathered rock and the movement of soft rock socket and pile shaft in weathered rock layer, while stage 2 focused on the friction and end bearing capacities of the soft rock, with the upper O-cell open to separate the soft rock socket from the remaining upper pile section.

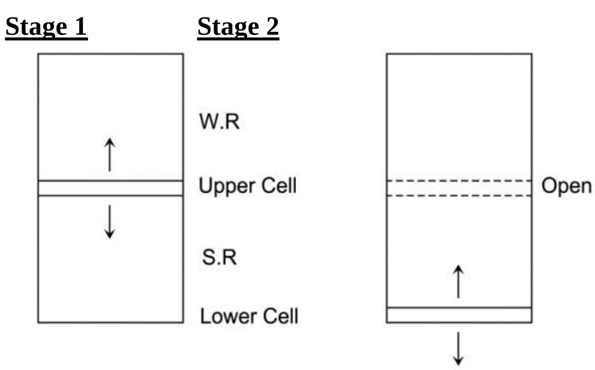

Figure10. Typical Procedure of O-Cell Test

The vertical test piles were loaded up to a maximum one way load of 150MN in about 30 incremental stages, in accordance with ASTM recommended procedures. The dynamic loading-unloading test was carried out at the design loading ranges by applying 20 load cycles to obtain the dynamic characteristics of the pile rock socket.

A borehole investigation was carried out at each test pile location to confirm the ground conditions and confirm the pile length and soft rock socket depth of 5-6m before piling work commenced, and also to properly match the test results to the actual ground strata. The pile tests were undertaken in mid 2010 and a summary of the vertical pile test results is shown in Table 9, which is based on the pile test analysis performed by the Load Test Corporation.

Test Pile 3 (TP3) results are not shown herein due to construction defects identified in the pile; thus, these test results were ignored in obtaining the average results. While the overall performance of the test piles exceeded expectations, Test Pile 3 highlighted the fact that the steep variability in rock conditions within a short distance could affect the overall pile quality of the pile and may require careful assessment, during construction, of the pile excavation and the quality of the rock at all levels. The pile testing program also demonstrated that the foundation system could still be optimized, given the higher than anticipated shaft and base resistances that were obtained in the other four pile tests.

The lateral test pile was subjected to a maximum lateral load of 2.7MN using the set-up shown in Figure 11. The dynamic load-unloading test was carried out at 900kN, 1350kN and 1800kN by applying 20 cycles to obtain the lateral dynamic performance of the pile, especially within the marine clay layer. The load-pile head displacement relationship from the lateral pile test is shown in the Figure 12. The measured lateral stiffness of the pile was greater than expected during the initial loading stage, presumably due to the repeated loading condition and also due to the overconsolidated ground conditions arising from excavation. The stiffer behavior under cyclic loading is summarized in Table 10. This stiffer pile behavior will be considered in the final structural design of the tower foundation system, as well as for re-assessing the predicted pile group movement.

Table 9. Summary of vertical pile test results (Allowable Pile Bearing Capacities)

Strata		Designed	Pile Test			
			TP1	TP2	TP4	Aver.
Soft Rock	End Bearing(MPa)	4.0	6.3	9.0	9.2	8.1
	Friction(kPa)	350	743	897	663	767
Weathered Rock	Friction(kPa)	250	357	527	178	354

Note : F.O.S = 3 is applied for end bearing from ultimate or test load.

F.O.S = 2 for shaft friction from yield loading point.

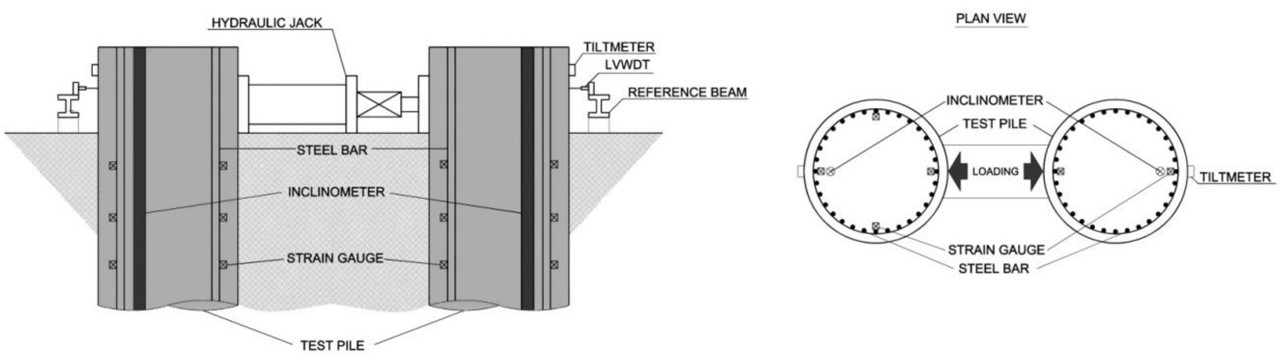

Figure 11. Schematic of Monitoring for Lateral Pile Load Test

Table 10. The Lateral Stiffness of Test Pile

Design Stiffness (MN/m)	Measured Secant Stiffness of Test Pile(MN/m)			
	Static		Dynamic	
	0~900kN	900~1,350kN	0~900kN	900~1,350kN
86~120	294	97	488	326

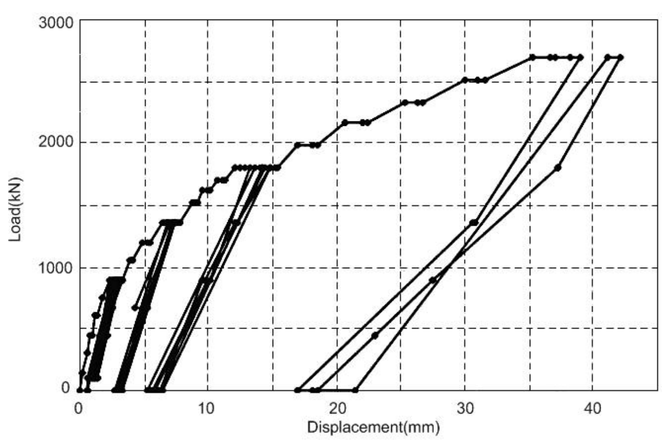

Figure12. Load vs. Displacement curve TP5

CONCLUSIONS

This paper has described the design and testing process of a pile raft foundation system for a super high rise building to be located within the reclaimed area in Songdo, Korea. The design process has involved three principal phases, namely concept design, the main design phase, and the post design/study phase, including the vertical and lateral load testing programs.

Geotechnical uncertainty is the greatest risk in any deep foundation design and construction process. Establishing an accurate knowledge of the ground conditions is essential in the development of economical foundation systems which perform to expectations.

It has been emphasized that collaboration between the geotechnical designer and the structural designer is important for the foundation design as the overall pile group behavior needs to be adequately captured in structural design and the wide range of loading conditions needs to be adequately assessed in the geotechnical design. Based on the geotechnical engineering assessment of the foundation system, a 3-dimensional finite element analysis model can be created by the structural engineers to assess to the overall behavior of supertall and slender towers by creating a 3-Dimensional FEAM to simulate soil-structure interaction, the stiffening effects of the superstructure on the foundation, and the impact of the foundation flexibility of the overall static and dynamic performance.

The use of a suite of commercially available and in-house computer programs has allowed the detailed analysis of the large group of piles to be undertaken, incorporating factors that include pile-soil-pile interaction effects, varying pile lengths, and varying ground conditions in the foundation design. An independent finite element analysis using readily available commercial programs had been used to include the effect of soil-structure interaction and to include the impact of the foundation system on the overall behavior of the tower.

The post-design process was extended in order to obtain the actual response of the ground and the piles due to various loadings. From the results of pile load tests carried out in the post-design period, the prediction of pile behavior can be refined and the pile capacities can be updated which may result in confirmation or modification of the design, which may lead to a more cost-effective design.

An extensive high quality vertical and lateral pile testing program was developed and performed for the project and it was found that the pile behavior and capacities were higher than expected, so that it would be beneficial to revise some of the more conservative assumptions made in the design. An extensive monitoring program is being developed to measure the actual behavior of the tower foundation system during and after tower construction.

Presently the tower site is fully reclaimed, the site is fenced, and enabling works are being planned.

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