Foundation Design of the Incheon Bridge

Sung-Min Cho

Principal Research Engineer, Korea Expressway Corporation

50-5, Sancheok-ri, Dongtan-myeon, Hwaseong-si, Gyeonggi-do, South Korea, chosmin@ex.co.kr

E-mail: 조성민 chosmin@ex.co.kr

ABSTRACT: The Incheon Bridge, the longest bridge of Korea which was opened to the traffic in 2009, is an integration of several special featured bridges and the major part of the bridge consists of cable-stayed spans to cross the Yellow Sea. All the foundations consist of drilled shafts, large diameter bored pile foundations which were penetrated into the bedrock under the seabed. A single pile-bent type foundation system was selected as well as the pile-cap type foundations. New design scheme according to the LRFD (load & resistance factor design) specification was implemented for the project. The estimation of bearing capacity and settlement of rock socketted drilled shafts was carried out based on the understanding of the site condition, the ground properties and pile load test results. The results of the load tests were thoroughly analyzed by a number of experts to determine the resistance factor, giving a unique opportunity to improve the current LRFD concept in Korea. Geotextile tubes to block seawater were made to construct the foundation at the foreshore site whose tidal difference between ebb and flow was so large. Rip-raps which were designed by physical modeling and analysis are spread around the pile to prevent the scouring of the foundation. Circular dolphin structures made of the flat sheet piled wall and in-filled aggregates surround the piers near the navigation channel to protect the bridge against the collision with aberrant vessels. The structural design of the dolphin as a ship impact protection system was performed with numerical analyses of which constitutional model was verified by the physical model experiment using the geo-centrifugal testing equipment.

Keywords: Foundation, Pile, Drilled shaft, Incheon Bridge, Geotextile Tube, Ship-Impact Protection

1. INTRODUCTION

The Incheon Bridge is an 18.4 km long sea-crossing bridge connecting the Incheon International Airport with the expressway networks around the Seoul metropolitan area (Figure 1). This bridge is an integration of several special featured bridges and the major part of the bridge consists of cable-stayed spans. This marine cable-stayed bridge has a main span of 800 m wide to cross the vessel navigation channel in and out of the Incheon Port.

Although superstructures of the bridge are multifarious, all the foundations for the bridge consist of large-diameter drilled shafts, a kind of the cast-in-place bored pile. Drilled shaft pile foundations were penetrated into the bedrock to support the colossal superstructures. The bearing capacity and deformational characteristics of the foundations were verified through the static load tests using 8 full-scale pilot piles and 2 working piles. A single pile-bent type foundation system was selected as well as the pile-cap type foundations. Horizontal load tests for the pile were also performed to find the lateral stability of the foundation.

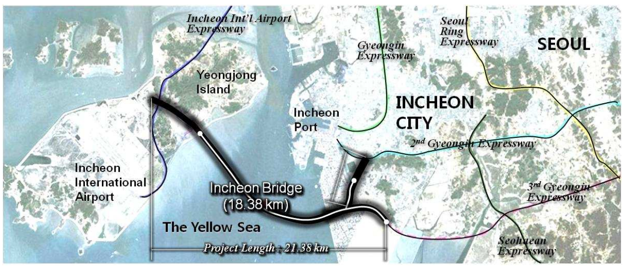


Figure 1: Location of the Incheon Bridge

Rip-raps which were designed by physical modeling and analysis are spread around the pile to prevent the scouring of the foundation. Geotextile tubes as a cofferdam to block seawater were made to construct the foundation at the foreshore site whose tidal difference between ebb and flow was so large. Circular-cell type dolphins surround the piers near the navigation channel to protect the bridge against the collision with aberrant vessels. Each dolphin structure consists of the flat sheet piled wall and in-filled aggregates to absorb the collision impact. The structural design was performed with numerical analyses of which constitutional model was verified by the physical model experiment using the geo-centrifugal testing equipment.

Geotechnical analysis and designs of the foundation for this epoch-making bridge including the auxiliary structures are introduced in this paper.

2. DESIGN CRITERIA

There are two main philosophies of civil engineering design: Working or Allowable Stress Design (WSD, ASD) and probabilistic-based limit states design. Ultimate strength design or load factor design locates somewhere between these two methodologies. LRFD, a relatively new development in civil engineering design, especially in geotechnical engineering, is a reliability-based limit states design methodology. The most conventional and general method for geotechnical designs is the ASD In ASD or WSD, the stresses in members under service loads are compared to an allowable stress divided by a factor of safety. The factors of safety used in ASD are based on past experience and engineering judgment, not specific consideration of the uncertainties involved in design. There is only one factor to account for all the uncertainties that may be encountered in loads and resistance.

New design scheme of bridge by LRFD was implemented for the Incheon Bridge project. Incheon Bridge is the first project of the AASHTO LRFD applications to both the superstructures and the substructures of a bridge in Korea. Basic concept of the LRFD can be described in equation (1) as follows:

$$\Sigma r_i Q_i \leq \Sigma \phi_i R_i \tag{1}$$

In the equation, Q_i is nominal load and R_i is nominal resistance, r_i is load factor and ϕ_i is resistance factor.

Nominal resistance R_i is corresponding with the allowable stress evaluated in ASD. In the LRFD philosophy by this equation, design loads are increased and design resistances are reduced by multiplying two respective factors; namely, r_i (> 1.0) and ϕ_i (< 1.0). Foundations are proportioned so that the factored resistances are larger than the factored loads. In ASD, it needs only to decide the global factor of safety. For example, the safety factor for a bridge foundation typically would be 3.0 in design stage without load tests. In LRFD, it is essential to find the values of r_i and ϕ_i. The key improvements offered by LRFD over the traditional working stress design (WSD) are the ability to provide a more consistent level of reliability between different designs and the possibility of accounting for load and resistance uncertainties separately (Foye et al., 2006). Limit states design approach provides a consistent design manner between structural engineers and geotechnical engineers and it also possible to make a rational and consistent framework for design and risk management of design uncertainties. However, from the early 1980s when the limit states design for the foundation was first introduced into North American engineering practice, this kind of design concept has not been well accepted by geotechnical engineers. Moreover, it is not easy to define the value of resistance factor ϕ_i which can be influenced by geotechnical characteristics of the ground, efforts of quality control, skill of engineers, and various factors during design and constructions. Loads and load combination according to the Korean design codes as well as the AASHTO design specifications were additionally applied into the design to consider the domestic conditions.

At the beginning of the project, many kinds of the test and investigation were carried out to find out the design parameters accounting for the detail design. Especially, a number of full-scale static pile load tests were conducted for both the offshore sites and the onshore areas not only to determine site specific resistance factors but also to remove any excessive margin of the stability came from the conventional ASD practices. Literature reviews and researches to grasp the changes in the AASHTO specifications were also zealously undertaken. And several countermeasures were introduced to guarantee the reliability of the first LRFD design in Korea. Additional subsurface explorations were widely done to verify the design and high-level quality controls were applied into the field site during the construction to secure the reliability of the design. Comparison with results by the ASD scheme was also fulfilled at the preliminary design stage.

3. SITE CHARACTERISTICS

Geotechnical investigations including in-situ tests and laboratory tests were performed along the bridge alignment to find the engineering characteristics of the ground. Seismic survey using the Bubble Pulser, the Chirp, and the Sparker systems on the sea provided the subsurface sedimentary structure information. The comparison between the analysis of seismic data, the borehole data, and the laboratory testing data enabled to classify the subsurface layer stratigraphically and to get accurate properties of the ground. Differential GPS positioning techniques replaced conventional methods in performing area surveys. The survey vessel and barges were controlled by monitoring system with DGPS and Navigation Program. Predetermined boreholes drilled as NX-size (diameter 76 mm) and self-elevating-platform barges with legs installed with hydraulic pressure jack were used to get over the inclement weather conditions and tidal differences in the offshore site. Steel casing pipes were installed to the weathered rock layer to protect the bored hole and boring was continued to the depth more than 3 times of the pile diameter from the pile tip. During the test boring, standard penetration test (SPT) was performed at 1.5 m intervals in soil layers and weathered rock layer. In alluvial deposits, undisturbed samples were obtained in clay layer using a thin wall tube. Rock core sample were recovered by using a double, or triple core barrel with a diamond bit. Various kinds of in-situ tests for the soil and rock were carried out to find the engineering characteristics and laboratory tests were also conducted with obtained specimen. Major methods of the site explorations are summarized in Table 1.

The basement of the site consists of Precambrian metamorphic rocks, Jurassic igneous and sedimentary rocks, and Cretaceous volcanic rocks. So, major types of the rock are Jurassic biotite granite, gabbro, granodiorite, and K-feldspar granite. The basement rock was later intruded by aplite, pegmatite, intermediate and acidic dykes. Biotite granite, which consists of most of the basement rock, shows fine to coarse grain size and equigranular and/or foliated texture. Regional lineaments were investigated by comparing Landsat image (30m) with geologic maps. Lineaments were found as trend of faults that were formed during Jurassic and Cretaceous in age. The subsurface ground was stratified with marine deposits (clay, silt and sand; thickness 15-30 m), residual soil (sand; thickness is up to 20 m), weathered rock (RQD is 0 to 20 %; thickness is up to 20 m), soft rock (bedrock) and hard rock layers from the surface. There was a reclaimed layer locally at some working area. Figure 2 shows a ground profile at the main span section of the bridge route. Depth of the sea is about 20 m under the cable-stayed bridge (CSB). Index properties of the soil layer are summarized in Table 2. SPT results of the cable stayed bridge site versus the depth are plotted in Figure 3(a). Difference between each layer from the soft deposit to weathered rock can be recognized. Undrained shear strengths of the clayey soils obtained from the triaxial compression test with UU conditions, C_{uu} are shown in Figure 3(b). C_{uu} increases as depth increases.

Marine deposit layers at the top of the seabed had been formed by erosion, transportation and sedimentation and tidal currents that result in complex depositional patterns. The residual soil layer was formed by weathering of rocks. The weathered rock layer preserves the original macroscopic features. The boundary with residual soil layer was defined by N values (50 blows/ 15 cm) of the SPT with additional considerations from physical observations. The soft rock layer is relatively fresh and has higher strength than the weathered rock. The boundary with weathered rock was decided by TCR (total core recovery), RQD (rock quality designation), field strength, weathering status and fracture conditions.

Ground profile or layer can be recognized by seismic wave velocity. Figure 4 shows the result of suspension PS-logging survey using E2-2 borehole at the CSB site. The location and SPT results are depicted in Figure 2. Velocity profiles of P-wave and S-wave can be compared with the ground classifications according to the SPT results. Dynamic modulus of elasticity, E_d and dynamic modulus of shear, G_d were evaluated from seismic wave velocity data. They are also shown in Figure 4

Pile foundations of the Incheon Bridge were designed to support the vertical load by the bearing capacity mainly generated at the rock socket. Therefore the strength and deformation characteristics of the rock layer were important to the design. Soil layers over the rock strongly influenced on the lateral behavior of the pile. Figure 5 shows the relations between the deformation modulus, E_m and the Rock Mass Rating (RMR) from the pressuremeter testing and Goodman jacking at the bed rocks. It is obvious that deformation modulus is closely related with the RMR.

Table 1 Geotechnical investigation methods in the Incheon Bridge project

Types	Main methods of site investigations
In-situ tests and samplings	Test boring / Standard penetration test (SPT) with energy measurement Field vane test (FVT) / Piezo-cone penetration test (CPT_U) Pressuremeter test (PMT) / Lateral load test (LLT) / Goodman jack Borehole shear test (BST) Disturbed and undisturbed sampling for soils and rocks
Laboratory tests	Index properties / Tri-axial compression test (UU, CU, CK_0U, Cyclic) Unconfined test / Resonant column tests / Point load test Consolidation test (oedometer, Rowe cell)
Geophysical surveys	Seismic survey on the sea and the land (Tomography, Refraction, Reflection, Suspension logging) Electric resistance survey / Density logging Bore-hole television (BHTV) / Bore-hole image processing (BIPS)

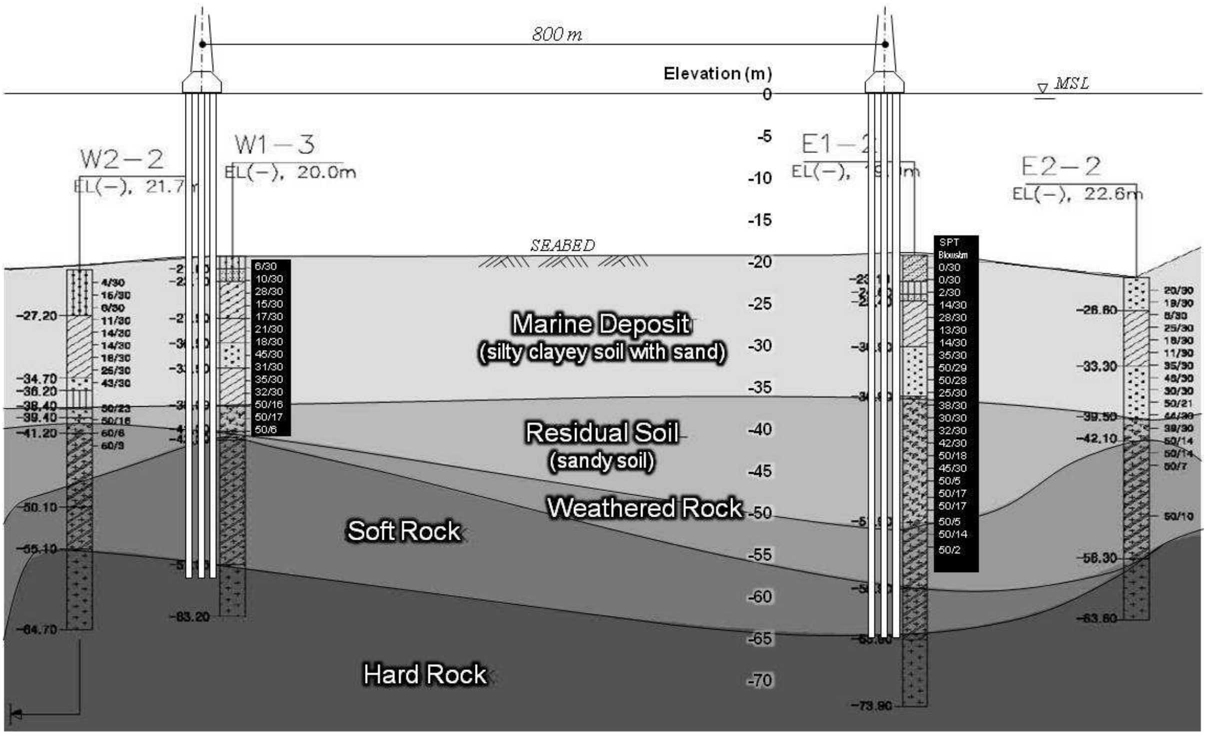
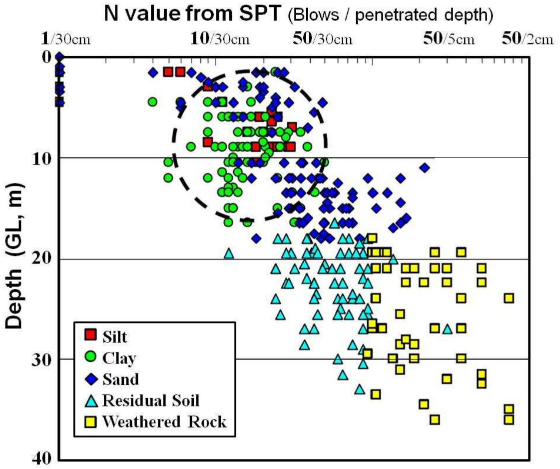


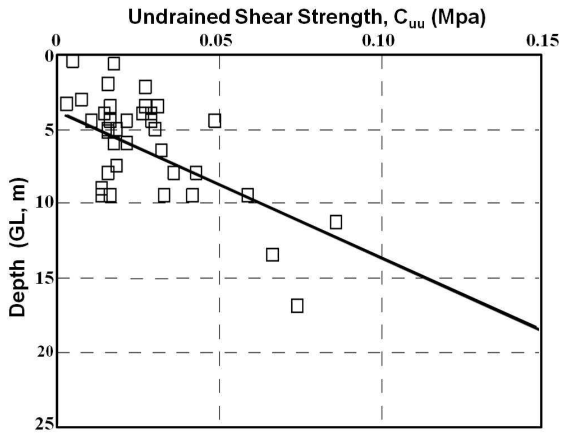
Figure 2 Ground profile with boring log at the main span of the cable-stayed bridge

Table 2 Basic index properties of the soil in the field

Items	Clayey Soil	Sandy Soil
Specific Gravity	2.67-2.79 (2.72)	2.58-2.76 (2.69)
Finer passing the #200 (%)	47-100 (91)	1-99 (45)
Atterberg limit (%) W: water contents, LL: liquid limit PL: plastic limit, PI: plastic index	W: 17-48 (33), LL: 26-75 (37) PL: 15-33 (23), PI: 2-53 (15)	W: 12-41 (27), LL: 25-56 (30) PL: 21-32 (24), PI: 1-31 (6)



(a) SPT results



(b) Undrained strength of the clay

Figure 3 Results of SPT and UU-triaxial test at the cable stayed bridge site

Marine deposit layers at the top of the seabed had been formed by erosion, transportation and sedimentation and tidal currents that result in complex depositional patterns. The residual soil layer was formed by weathering of rocks. The weathered rock layer preserves the original macroscopic features. The boundary with residual soil layer was defined by N values (50 blows/ 15 cm) of the SPT with additional considerations from physical observations. The soft rock layer is relatively fresh and has higher strength than the weathered rock. The boundary with weathered rock was decided by TCR (total core recovery), RQD (rock quality designation), field strength, weathering status and fracture conditions.

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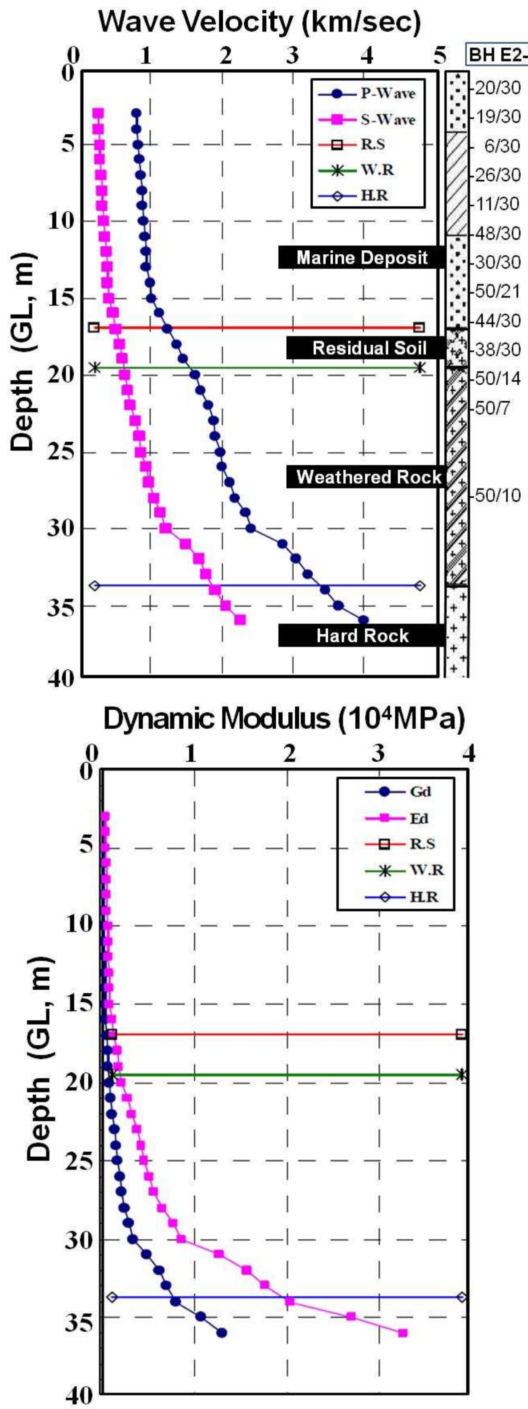


Figure 4 Seismic wave profile and dynamic modulus distribution according to ground layer (RS-Residual Soil, WR-Weathered Rock, HR-Hard Rock)

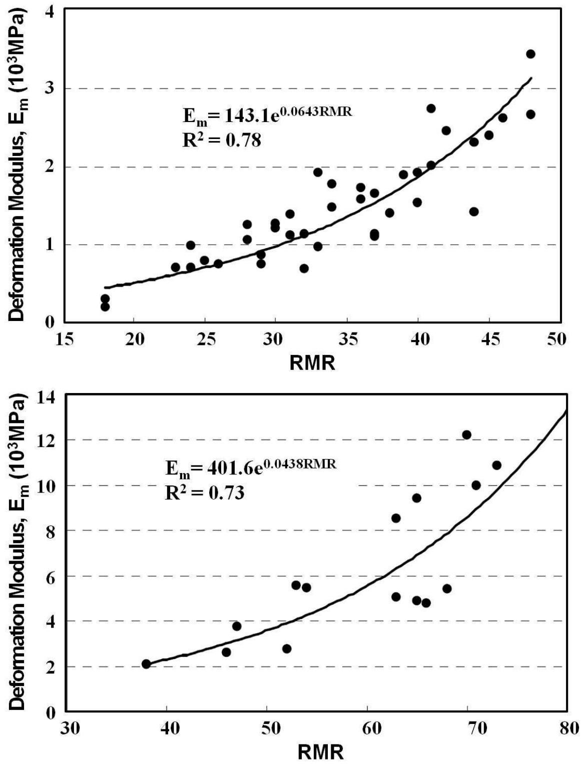


Figure 5 Relation between deformation modulus and RMR in soft rock (left) and hard rock (right)

4. DESIGN OF DRILLED SHAFT FOUNDATIONS

Incheon Bridge is supported by large-diameter drilled shafts socketted into a base rock layer. A drilled shaft is a kind of the pile foundation that is constructed by placing fluid concrete with rebar cage in a excavated hole. The main function of the drilled shaft is same as other deep foundation which transfers vertical load through weak ground to relatively strong stratum. It has been also useful to resist large lateral or uplift loads when deformation tolerances are small.

Diameter of the drilled shaft for the Incheon Bridge was up to 3.0 m. Piles were designed to support any loads and dead weight that transferred to the foundation. For the offshore bridges around the main span and overland bridges, a multi-column foundation system with pile cap was applied. Single drilled shaft-column system was designed for other offshore bridges far from the main span and bridges on the mud flat area. The pile length was up to 76 m from the pile cap to the tip and it was determined from geotechnical analysis and penetrated into the rock.

A number of pile load tests (PLT) were conducted for all sections not only to determine site specific load resistance factors in the LRFD implemented sections but also to remove any conservativeness in the ASD implemented sections. At on-the-sea parts, over 30,000 tons, more than three times the designed load was added to a pile in the world's largest scale test. It is equivalent to 77 Jumbo-747 planes carrying a full load of passengers, packages and fuel pressing a pile. We also conducted lateral load tests on reduced scale as well as full-scale piles with due consideration of possible loading situations in which the actual piles, having substantial lengths above seabed with pile-bent, are expected to subject to large dynamic hydraulic pressure and lateral loads during the bridge's life time.

4.1 Pile Load Tests

A full scale pile load tests using eight pilot piles were carried out in order to establish criteria for bearing capacity evaluation. Bi-directional load tests including the Osterberg cell test were conducted in order to verify the actual bearing capacity of the drilled shafts. Thanks to the sacrificial loading cell at the end of the pile, upward loading against skin friction and downward loading against end bearing during the cell's working in two directions separates the each resistance parameters. Schematic concept of the bi-directional load test is depicted comparing with the conventional testing method in Figure 6. Strain gauges attached along the pile shaft give a information of skin friction as well as the load-transfer behavior. Figure 6 also shows an example of strain gauge instrumentation of a pilot pile which penetrated into the hard rock with casing pipe to the upper part of the weathered rock.

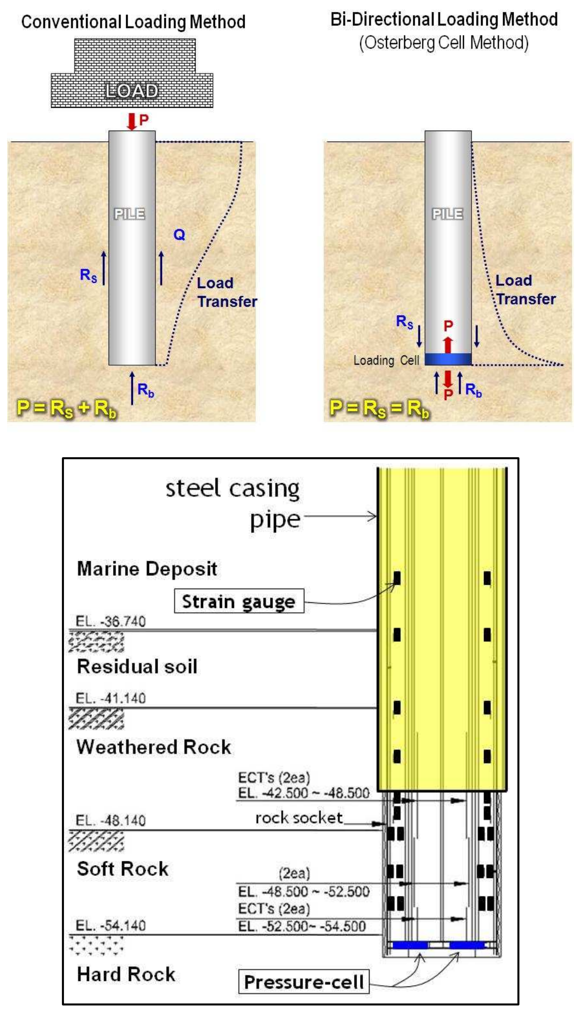


Figure 6 Schematic diagram of the pile load tests (left) and an example of instrumentation for the bi-directional test in the Incheon Bridge site (right)

Three pilot piles with multi-level loading cell system were experimented to verify the side resistance varying according to the depth and the ground type. All the testing results were thoroughly analyzed by a number of experts to determine the resistance factor, giving a unique opportunity to improve the current LRFD concept (Cho et al, 2009B; Kim et al., 2007). During construction an additional proof load test was conducted to confirm the design assumptions in addition to a series of lateral load tests on three reduced-scale piles and nine as-constructed piles at the site to confirm the lateral load carrying capacity. For the bridge section constructed by FSLM, the load-transfer characteristics were additionally examined by instruments installed at a pile with pile-bent to confirm whether or not the load from the supper structure is transferred to the foundation.

The examples of the load-displacement curves at top and bottom of the pressure cell installed at the tip of a pile are drawn in Figure 7. Upward compressive load up to 137 MN and downward load up to 142 MN were applied at the end of this pile. The estimation of bearing capacity and settlement of rock socketted drilled shafts was carried out based on the understanding of the site condition, the ground properties and pile load test results. Load-transfer analysis using the readings from the strain gauge attached along the pile was performed. An example of the strain gauge installations are shown in Figure 6. Pile settlements predicted by both elastic modulus and deformation modulus which were obtained from PMT were compared the measured value. It was found the settlement estimation using the elastic modulus gave a conservative result. Figure 8 shows skin friction and end bearing of the testing drilled shaft of Figure 7.

Very large scale pile load test by the bi-directional scheme in Incheon Bridge project was inevitable and it was also the only testing method considering the level of the load and troubled offshore conditions on the sea. Although the tests were very successful, it was not easy to cast away the doubts of the limitations of the bi-directional method. Some researchers still insist that this kind of test may give a different result comparing with conventional top-down type static loading test, while there are also many researches which prove that the results of the O-cell test were similar as that of the conventional static test.

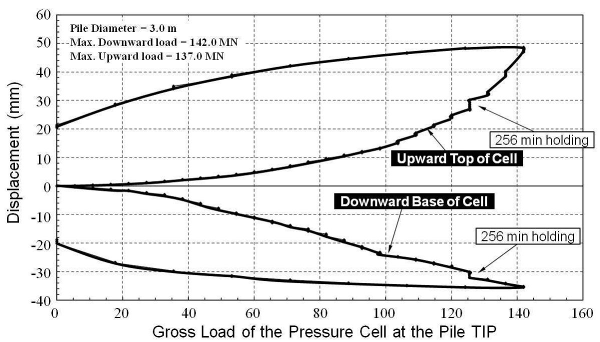


Figure 7 Example of bi-directional pile load test result for a pilot pile (diameter=3.0m)

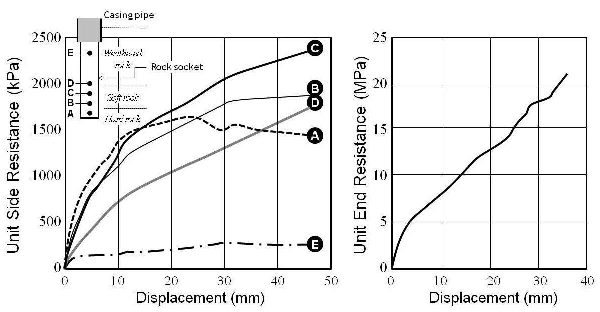


Figure 8 Pile resistances at the rock socket of the testing drilled shaft in Figure 7

From a recent research in Korea, which compares the top-down loading tests and bi-directional loading tests according to the coupled load-transfer analysis, bi-directional loading test underestimated the settlement of the pile. The main reasons of this result might be due to 1) the separation between the skin friction and the base resistance, and 2) the upward direction of the side resistance along the shaft. The settlement at the pile tip as the loading along the pile shaft may not be considered in the bi-directional test. This can cause an overestimation of the bearing capacity. However, that research also concluded that the underestimation of the settlement was not severe in the pile which penetrated into very stiff layer like as rocks. Several uncertainties of the test were taken into considerations and the testing results were applied into the design conservatively as stated later.

Details on the lateral load test, vertical proof tests, and load-transfer tests under the actual bridge load are skipped over in this paper.

4.2 Bearing Capacity Evaluations

FHWA method, Davisson (1972) method and DeBeer's log-log method (DeBeer, 1970) were reviewed to decide the ultimate resistance. Settlement corresponding to the ultimate resistance was more than 4.2 % of the pile diameter by Davisson method and it was less than 0.5 % of the diameter by DeBeer method. It was concluded that Davisson method may overestimate the settlement, on the contrary, the method by DeBeer's theory may underestimate the settlement. FHWA defines the ultimate resistance as the value at the settlement of 5 % diameter. In the Incheon bridge project, 1 % of pile diameter criterion was used to determine the resistance of the pile to enhance the safety of the mega structure. It means that the unit ultimate resistance of a pile corresponded to the displacement equal to 1% of the pile diameter.

When the rock is brittle in side, the residual shear strength of the socket can be assumed to be zero (AASHTO, 2004). According the FHWA (1999) researches, in some brittle rocks, the side shear may develop fully at a small value of displacement and then decrease with further displacement. If the rock is ductile in shear, both shaft and base resistance can be added directly. From the analysis on the testing results, when the drilled shaft is embedded into the weathered rock and soft rock of the Incheon site, shearing force can be mobilized even with large relative displacements since the bedrock has a lot of discontinuity such as joints and partial cracks. And both skin friction and end resistance can be added to calculate the total settlement of the rock socket. Pile load tests proved that the layer of weathered rock and soft rock is ductile in shear. In figure 9, skin friction of these layers keeps on growing despite the increase of the settlement. So, for cases where the pile shaft displacement exceeded 10 mm, the frictional resistance was also considered in addition to the end bearing capacity for the bearing capacity estimation. However, ductile characteristics of the hard rock were not confirmed through the tests. So, only the end bearing of the pile was considered for hard rock layer.

Skin friction of the weathered rock has been very difficult to estimate due to its kaleidoscopic variations. In the Incheon Bridge site, weathered rocks were classified into 3 groups according to the SPT results. Penetrated depth of the split spoon sampler when the blow number reaches the 50 was the criterion of the classifications. A meaningful difference in side resistances of the weathered rock can be observed in Figure 10. The lower boundary of the load testing results was proposed as a design guide. Figure 10 also presents the relation between the end resistances between the TCR. This kind of correlations using available field data used to give useful tools to compensate for the limitations of tests and investigations. Table 4 is a guideline to evaluate the end resistance of the drilled shaft on the soft rock

Table 3 Ultimate resistance estimations and corresponding settlement by two methods

	Test pile	A	B	C	D
Davisson Method	Ultimate Resistance (MN)	-	190	460	235
	Settlement at the ultimate resistance (mm)	-	100	255	128
	Settlement/Pile-diameter (%)	-	4.2	10.4	5.2
DeBeer Method	Ultimate Resistance (MN)	160	42	95	66
	Settlement at the ultimate resistance (mm)	15	2.5	5.5	6
	Settlement/Pile-diameter (%)	0.5	0.1	0.2	0.2

layer and it is also an example to utilize the TCR, point load strength (P_L), elastic modulus (E_m), and unconfined strength (q_u). Resistance factor to calculate the end resistance of the soft rock was set to 0.6. Table 5 summarizes the resistance factors for the rock embedded drilled shafts of the Incheon Bridge (Cho et al., 2006; Kim et al., 2006). Resistant factor for the extreme event state was decided as 0.95 considering the possible limitations of the bi-directional tests. Combination of resistance factors of Table 5 and ultimate resistance, i.e. end-bearing shown in Table 4 in compliance with the equation (1) made an actual design of the axially loaded pile foundation.

Based on these guidelines of the bearing capacity evaluation, the embedment length to the ground including the bedrock of each drilled shaft was calculated. The bearing capacity of piles of a pylon for the cable stayed span is compared with the load in Figure 11. 24 piles were installed to each pylon of cable stayed bridge.

4.3 Analysis for Connecting with the Pile Cap

Piles of a bridge pier were connected with a column through a pile cap (footing) except for pile-bent system (single drilled shaft column). Behavior of the pile foundation can be different according to the connection method between piles and the pile cap. This difference causes a change of the design method. Connection methods between pile heads and the pile cap are divided into two groups; rigid connections and hinge connections. Most design codes have specified to use rigid connection method for the highway bridge. In the rigid connection method, maximum bending moment of a pile occurs at the pile head and this helps the pile to prevent the excessive displacement (Table 6). Rigid methods are also good to improve the seismic performance. Bending moment and axial force at the pile head of the pile cap located in the ground are shown in Figure 12 as an example.

However, some of current Korean specifications prescribe that conservative results through investigations for both the fixed-head condition and the free-head condition should be reflected in the design. This statement may induce an over-estimated design for the bridge which has very good quality structures with casing covered drilled shafts and the PC-house contained pile cap. Because the assumptions of free-head conditions (hinge connections) are unreal for the elevated pile cap system with multiple piles of the long span sea-crossing bridges. On the other hand, elastic displacement method to evaluate the pile reactions under the pile cap is not suitable for this type of bridges due to impractical assumptions. External forces in the global structure system of the pile groups and pile cap under or on the water are described in Figure 13. Bending moments of the pile along the shaft were compared with each other according to the pile head connection methods for the elevated pile cap structure. Figure 14 presented the results of bending moment estimation for the conditions of Figure 13. In this figure, capacity ratio more than 1.0 indicates the unstable member.

Full modeling techniques which analyze the superstructure and the substructure simultaneously were performed. Loads and stress state of the very large diameter drilled shaft and the pile cap for Incheon Bridge were investigated through the full modeling for rigid connection conditions.

4.4 Constructions of the Drilled Shaft

The drilled shafts were constructed by a number of methods with due consideration of the site and ground conditions, such as the RCD method, Beneto method, rotating method, earth drill method, etc. During the construction of drilled shafts, steel casing pipes were first installed in the decomposed rock layer using a vibratory hammer with concurrent excavation of the materials inside of the casings. A rather strict control of the steel casing installation was adopted by using limits of 1% in verticality with 75 mm of deviation in the actual drawing. Toe locations of the drill shafts were determined based on the rock quality inspection procedure set by the owner. For a selected inspection pile for each pier, the committee on rock quality inspection directly inspected the rock quality. A special care was exercised for foundations with a pile-bent system as they require strict quality control. After excavation, excavated soils and rock fragments were removed and a rebar cage was installed after which underwater concrete was poured using a tremie pipe to complete a shaft. A cross-hole sonic logging (CSL) was conducted seven days after the shaft completion to check the pile integrity. CSL was conducted for all shafts supporting the V-shaped pylon and those with the pile-bent in the main navigation channel of Incheon Port. For those approach-bridge sections and the vidaduct section with pile caps, the CSL was conducted on selected piles. The pile caps were constructed after installing a PC house using a floating crane. In addition to the drilled shafts, a number of steel pipe piles with 2.4 m and 1.8 m in diameter were installed for the overhead crane and the large block erections for the side span of cable stayed bridge, jacket platforms, trestle, ship impact protection dolphins. For the steel pipe piles, dynamic load tests were performed to estimate the ultimate bearing capacity.

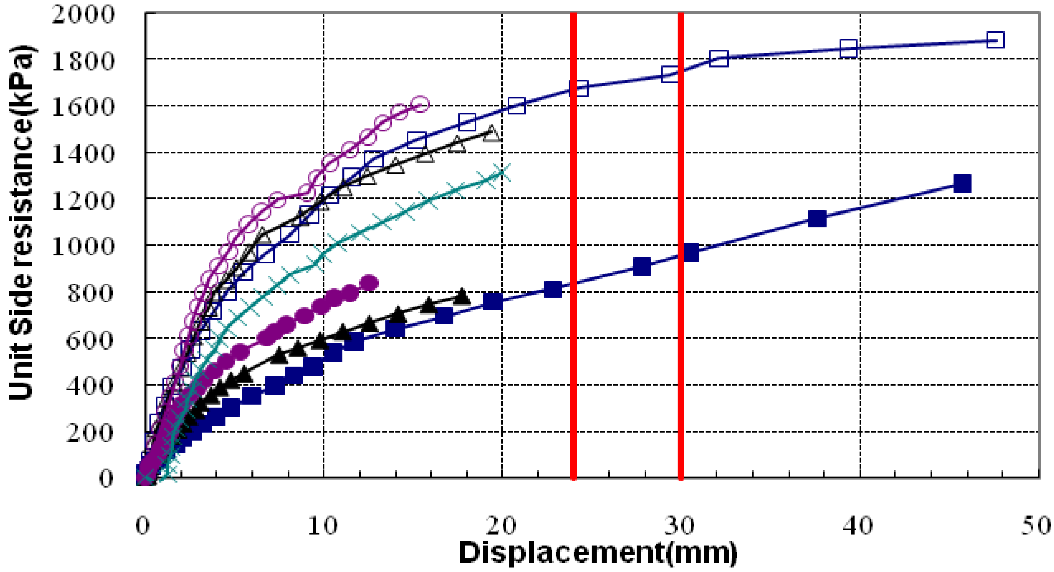


Figure 9 Skin friction behavior of the socket in the weathered rock and soft rock layers

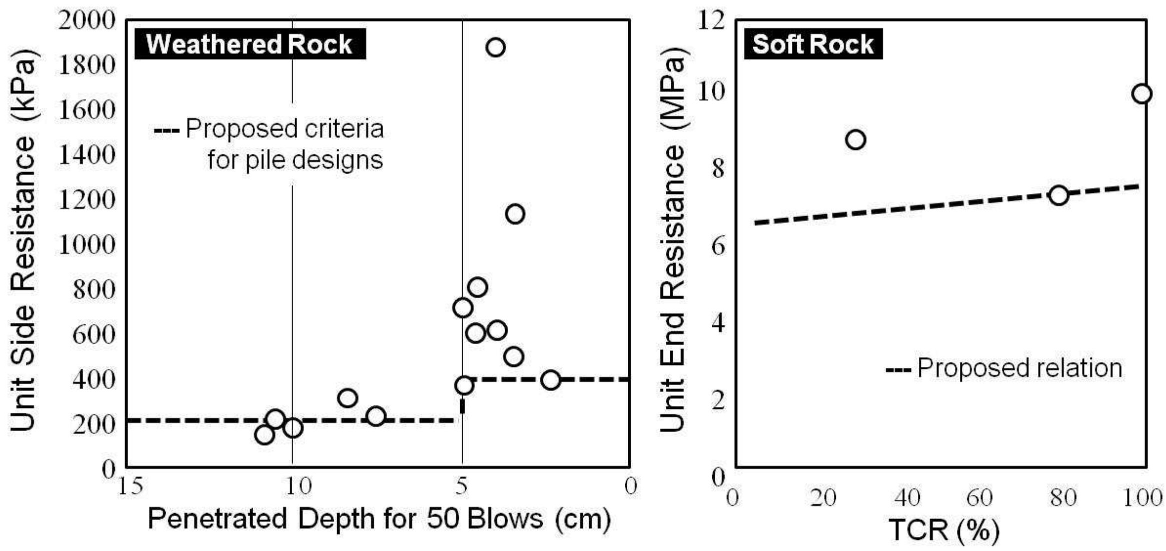


Figure 10 Skin friction criteria of the weathered rock according to the SPT results (left) and the relation between the end resistance of the soft rock with the TCR (right)

Table 4: End bearing resistance evaluation table for the pile socketted in the soft rock stratum

q_u (MPa)	P_L (MPa)	E_m (MPa)	TCR (%)	Ultimate base resistance (MPa)	ϕ
$q_u > 12$	$P_L > 20$	-	-	7	0.6
$q_u < 12$	$P_L > 20$ or No data	-	-	$[700\text{-}13\times(12\text{-}q_u)]/100$	0.6
$q_u > 12$ or No data	$P_L < 20$	-	-	$[700\text{-}11.33\times(20\text{-}q_u)]/100$	0.6
No data	No data	$E_m < 2100$	-	$[700\text{-}0.22\times(2100\text{-}E_m)]/100$	0.6
No data	No data	No data	TCR > 80	7	0.6
			TCR < 80	$[700 - 0.98 \times (80\text{-TCR})]/100$	0.6

Table 5 Resistance factor for the axially loaded drilled shafts which are embedded to rocks in the Incheon Bridge project

Limit state	Layer		Resistance factor	
			Skin friction	End bearing
Strength Limit State	Weathered rock	50/15cm ≤ N < 50/10cm	0.50	-
		50/10cm ≤ N < 50/5cm	0.60	0.50
		50/5cm ≤ N	0.75	
	Soft rock		0.70	0.60
	Hard rock		0.65	0.50
Extreme event	All layers		0.95	

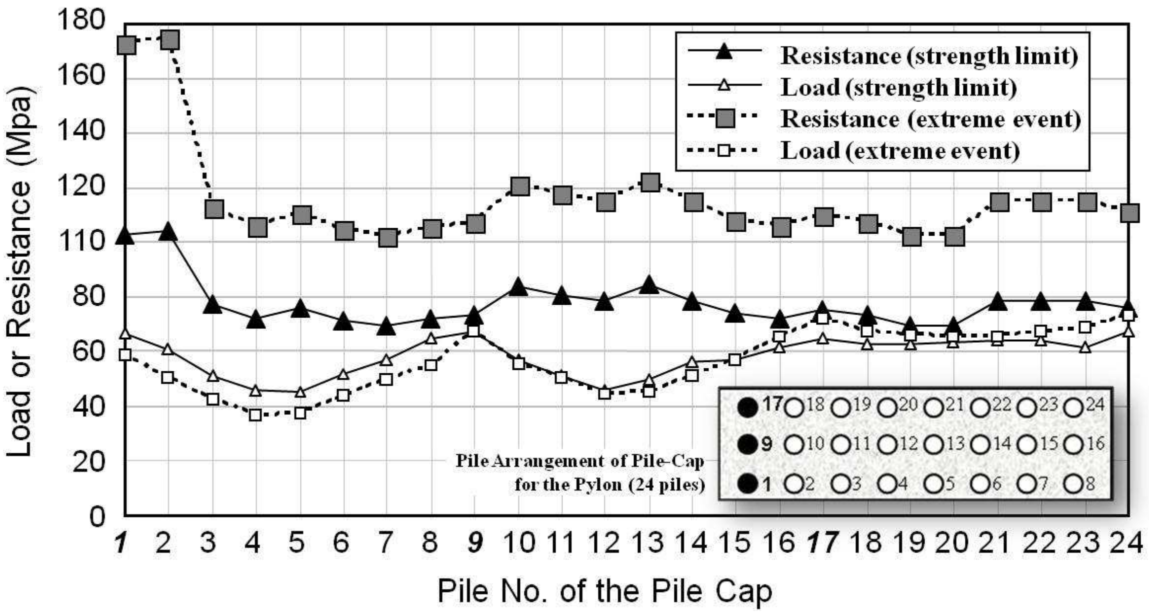


Figure 11 Comparisons of the bearing capacity with the vertical load for all 24 drilled shafts of a pylon of the CSB in the Incheon Bridge

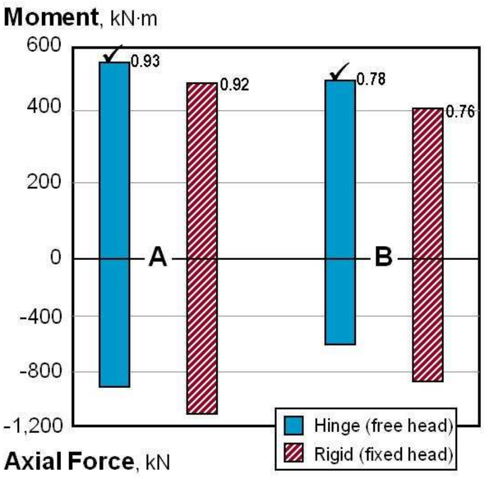


Figure 12: Moment and axial force at pile head in the pile cap which is located in the ground

Table 6: Comparison of pile head connections

Connections	Rigid (Fixed head)	Hinge (Free head)
Max. bending moment location	Head of pile	Middle of pile (ground)
Lateral displacement	Small (relatively)	Large (Reltively)
Degree of Indeterminacy	High	Low

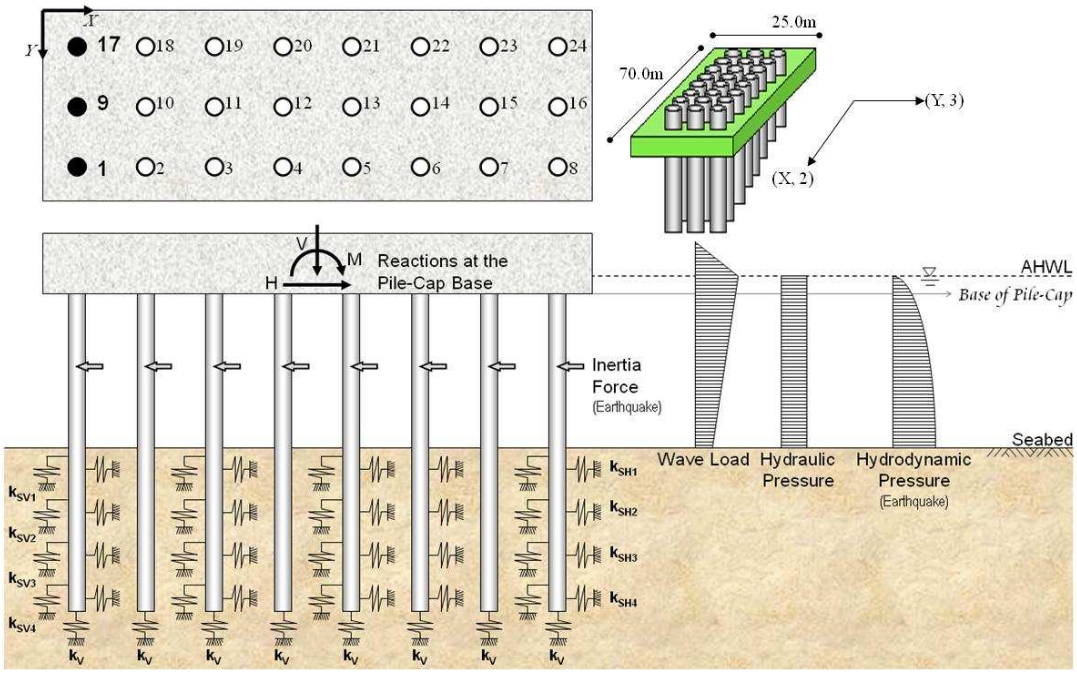


Figure 13: External forces of the multi-pile system which has a protruding pile cap from a seabed

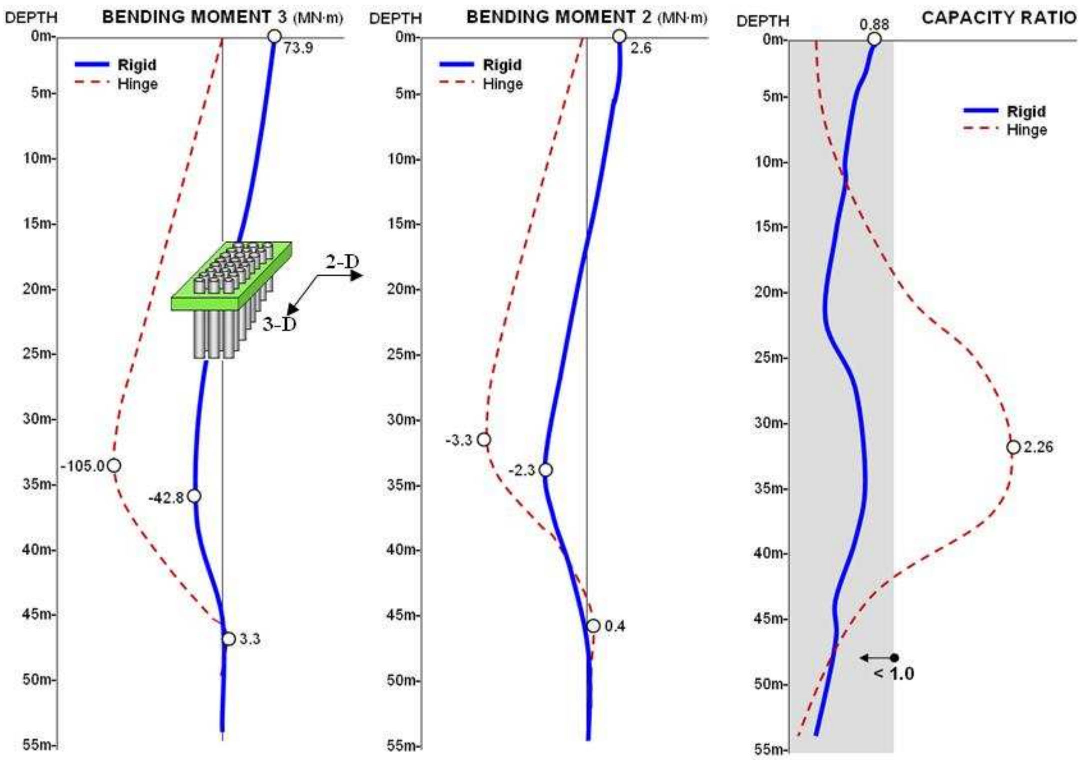


Figure 14: Comparison of the bending moment of a pile according to the pile head connections

5. AUXILLARY WORKS FOR THE FOUNDATIONS

5.1 Scour Protection of the Foundations

A foundation of the bridge shall be embedded below an estimated scour depth, or has an optimal countermeasure against scour. The appropriate measures against the scour can be classified into two types. One is the increasing the resistance of bed materials. The other is the method that weakens the factors which induce scour. The riprap is the most common countermeasure thanks to simple method of constructions and low cost. Scour protections with the ripraps were planned for piers of the cable stayed span and approach bridge span. All the relevant piers concerned have a group of drilled shafts and the pile cap. Effects of dolphins for the ship impact protection were also considered. Oceanographic investigations, numerical modeling, scour rate test, physical modeling with fixed bed and movable bed, and field monitoring were performed to design the scour protection around the pier. Design flow for the scour protection is shown in Figure 15.

Empirical formula assuming unconsolidated and non-cohesive sandy deposits was used. Physical modeling was based on the non-cohesive model sediments with distorted similarity. Numerical modeling was performed prior to the physical experiments using FLOW 3D model which complies with full 3-D Navier-Stokes equation. Distribution of the scour depth around the pylon is depicted in Figure 16 with the physical model.

As the riprap is embanked on the seabed, Isbash's formula was used to calculate the stable weight of the riprap for scour protection under the flow. Proposed diameters of the stable stone for the piers around the SIP dolphins are 50-60 (cm) and the gradation of stones in the riprap was suggested. To secure the stability of drilled shafts and dolphins, the lateral extent and the thickness of ripraps were decided by the numerical modeling and the physical experiments. Because the calculated armor stone sizes of all the piers were smaller than the riprap size, additional execution of armor stone would not be necessary.

For inland sections of the Incheon Bridge, either trestles or cofferdams were used to block the water to the foundation working site. Geotextile tubes were one of alternatives of the cofferdam. Geotextile tube is a kind of geotextile containers and it is filled with grain materials by hydraulic pumping. Hydration and cementation of the volume after the filling make the tube have the stability and help it resist external loads as a retaining structure in and out of the water. Sand, dredged soil or, sludge has been commonly used as a filling material. Civil engineers have used increasingly in recent years geotextile tubes filled with sand for the retention and erosion protection of dredged material in the sea and the river. Geotextile tube method was introduced into Korea in the late 1990s. Submersed breakwaters to protect the waterfront against erosions were the first applications of this method in 2001. Temporary road made of geotextile tubes was used for bridge constructions on the river in the suburbs of Seoul in 2003 (Cho et al., 2008). Major advantages of the geotextile tube methods include its low cost, fast construction speed, and environmentally friendliness. For Incheon Bridge construction site, a water-sand mixture of 80:20 by weight was used for backfill with sand forms installed between the top and bottom tubes to increase the frictional resistance (Figure 18). Height of the road ridge was determined to prevent the overtopping of the wave according to the Korean design specification for the port. And internal crest of the road embankment was designed to exceed the highest water level. Tubes were stacked up to 4 layers. Diameters of the tube were up to 5.0 m.

Sand was selected as filler materials. Table 7 and Figure 19 show the preliminary test results to determine the filler material. Sand had the advantage of the reducing the filling time and the stabilization of the shape. Figure 19 shows the tube height variation according to the elapsed time and the materials after the filling started (Univ. of Incheon, 2007).

Field instrumentation was conducted for earth pressure, pore pressure, and deformation to ensure the stability of the tubes during construction (Figure 20). Figure 20 presented the earth pressure changes of the tube during the 2nd layer tube injection in the underwater condition. Earth pressure in the 1st layer tube (road-side) was 0.12 kg/cm^2 before the beginning of the injection into the 2nd layer tube and it increased with elapsed time of the injection. Porewater pressure was excluded in this value. Earth pressure in the 1st layer tube after 6 hours inflations of the 2nd layer tube was 0.2 kg/cm^2 and it increased twofold at the end of the filling (0.4 kg/cm^2). Earth pressure beneath the sea-side tube of the 1st layer also changed from 0.22 kg/cm^2 to 0.51 kg/cm^2. 0.78 kg/cm^2 of the pressure was measured beneath the 2nd layer tube after the injection into the tube and Figure 20 indicates that this pressure was distributed to the 1st layer tubes.

6. FOUNDATION PROTECTIONS AGAINST SHIP COLLISION

A bridge across the waterway can be merely an obstacle from the viewpoint of ship navigations. So, in waterways where ship collision is anticipated, bridges shall be designed to resist ship impact forces, and/or, adequately protected by a kind of ship impact protection (SIP) systems including dolphins, berms, islands, fenders, or other sacrificial devices (AASHTO, 1997; 2007).

The curved shape of the Incheon Bridge route shown in Figure 1 was inevitable to accept the government request to locate the bridge 3 km away to the south from the Incheon Port for the safety of vessels passing under the bridge. The competent maritime authority also demanded a protective facility around the piers to resist against collision of very large vessel (up to 100,000 DWT) with the 10 knots speed. For the SIP design, additional several specifications were adopted. They were AASHTO's Guide Specification and Commentary for Vessel Collision Design of Highway Bridges, US Army Corps of Engineers' Manual: Design of Sheet Pile Cellular Structures, British Standard: 6349 Maritime Structures, and Korean Design Criteria for the Port and Fishery Harbor.

6.1 Type of the Ship Impact Protection System

Dolphin type SIP which consists of steel sheet piled wall and infilling aggregates were selected for this project. Large diameter circular dolphins were made at 44 locations of the both side of the main span around the piers of the cable-stayed bridge portion. Figure 22 shows the alignment of dolphins. Diameter of the dolphin is up to 25 m.

This dolphin is a kind of circular cell structure filled with aggregates. It has been used for harbor facilities and cofferdam constructions. Cross-section and materials of the SIP structure for the Incheon Bridge are described in Figure 22. The dolphin as SIP for the Incheon Bridge is the circular sheet pile structure filled with crushed rock and closed at the top with a robust concrete cap. Schematic deformation characteristics of the dolphin structure which was penetrated into the seabed ground are depicted in Figure 23. SIPs are sacrificial structures that will be partly or fully destroyed in the event of a severe impact. The stopping capability of the dolphins involves huge deformations, non-linear soil behavior and dynamic soil-structure interaction.

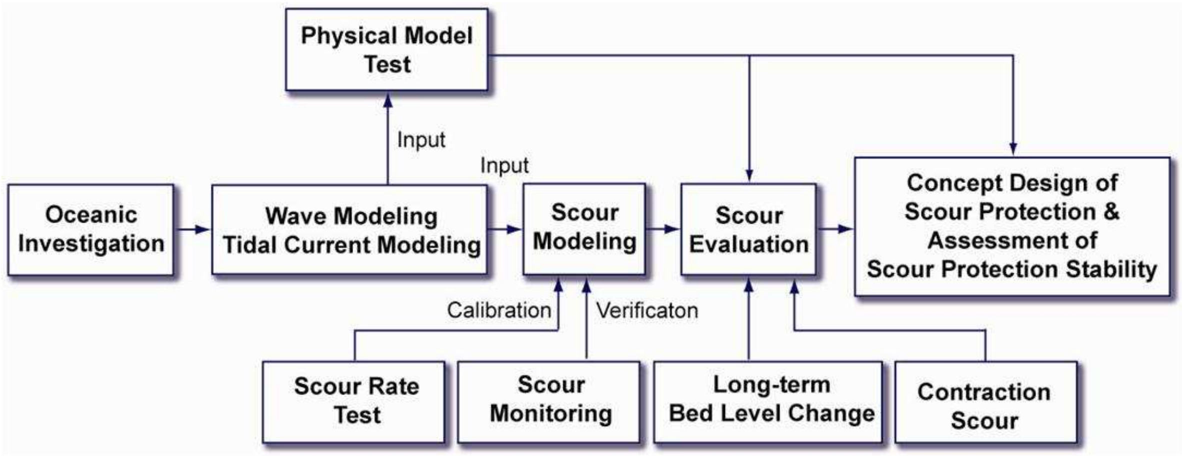


Figure 15: Flow chart of the scour protection designs in the Incheon Bridge

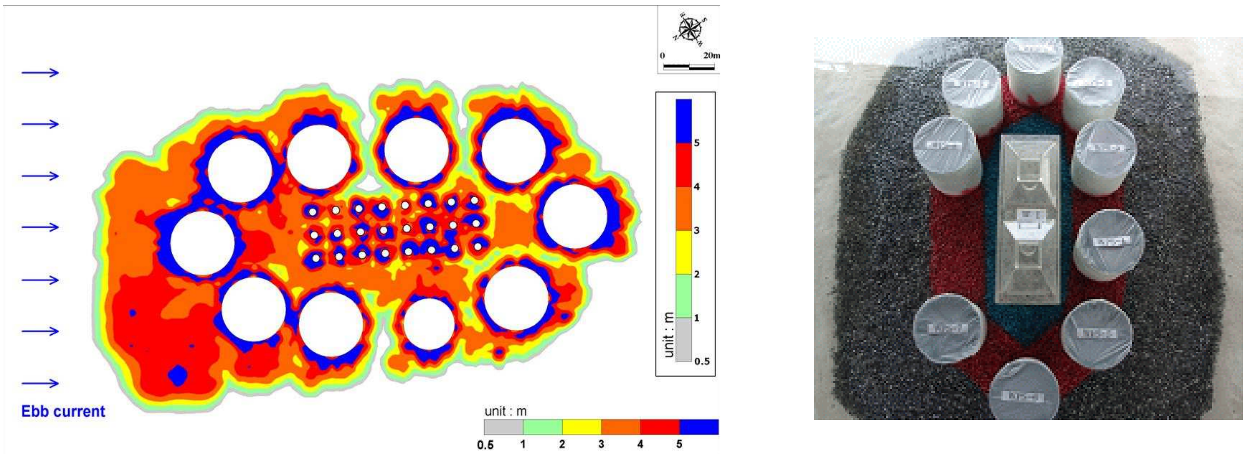


Figure 16: Scouting depth around the pylon of the CSB

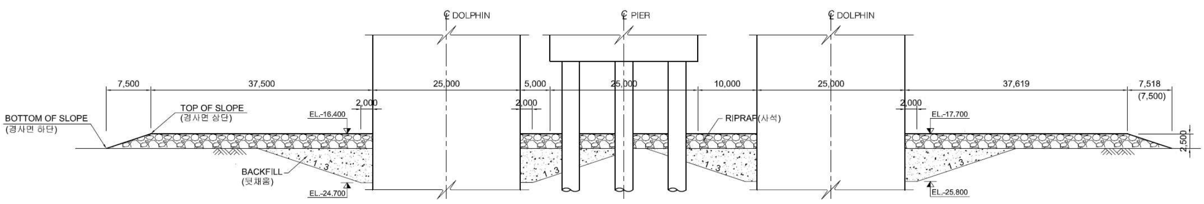


Figure 17: Cross-section of the riprap protection around the pier and dolphins

5.2 Geotextile Tube Cofferdams

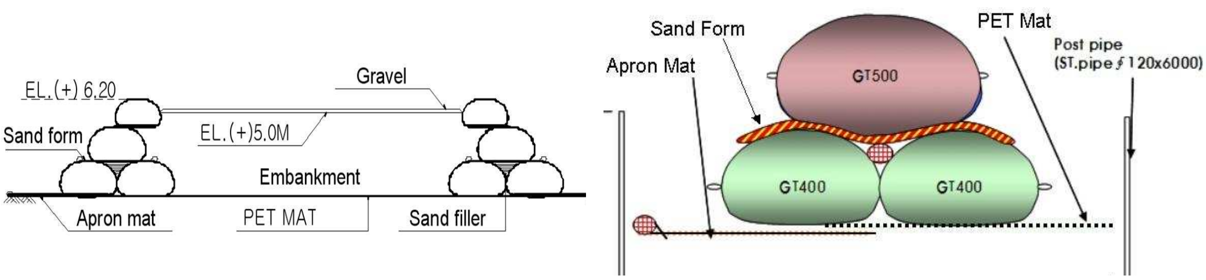


Figure 18: Geotextile-tube dikes to make temporary dry site for foundation works

Considering failure mechanism, stability assessment was performed for the strength limit state and service limit state. Circular wall of the dolphin cell was made of straight web type steel sheet pile (thickness 12.7 mm, length 500 mm). Gravels and crushed stones were decided as in-filling material. Diameter of the filling material was up to 200 mm. Large scale tri-axial compression test for the crushed stone was carried out. The friction angle of the crushed stone as a filling material was reduced to 38° considering the possibility of contracting behavior as the impact (Cho, 2009).

Table 7 Comparison of the filler materials

Properties	Silty Clay	Sand
Grain Size (mm)	0.003~0.03	0.075~5.0
Shape Stability	Poor	Good
Injection Time to 1.2m height	10 hours	1 hour
Convergence Time after the injection	100 hours	30 hours
Effective Height	50% of tube height	60% of tube height

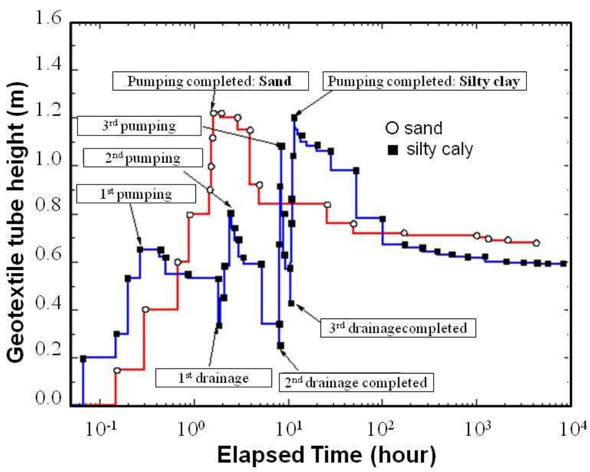


Figure 19: Variations of the tube height for 2 materials

6.2 Geotechnical Modeling and Anaysis

Numerical modeling by FEM scheme was used to find the detailed behavior of the dolphin structure. And geo-centrifugal experiments using reduced structure in the laboratory were also carried out to verify the numerical model. The centrifugal tests with simplified geometry conditions presented the insight in the relevant failure modes, the soil-structure-water interaction in the event of a real time modeled impact, and the necessary backbone data for verification of the numerical 3D modeling with actual conditions.

Thanks to this centrifuge test, the global quasi-static force response of the dolphins for direct comparison with the response predicted by 3D FEM analysis was obtained. The global response in dynamic impact scenarios for comparison with the quasi-static experimental results was also found. And it was possible to find the local force-indentation relationship for deep impact causing local indentation and even damage of the sheet piles (Kim et al., 2007). The physical model tests considered the behavior of a single circular dolphin with very simplified albeit realistic soil stratification using homogeneous layers. Thus, emphasis was placed on achieving reproducible testing conditions allowing high credibility of output and well defined conditions for comparison with the numerical 3D calculations. A fine grained Baskarp sand (grain size 0.15 mm) was used in this centrifuge model. 7 quasi-static and 11 dynamic model tests for 2 different prototype dolphins were carried out.

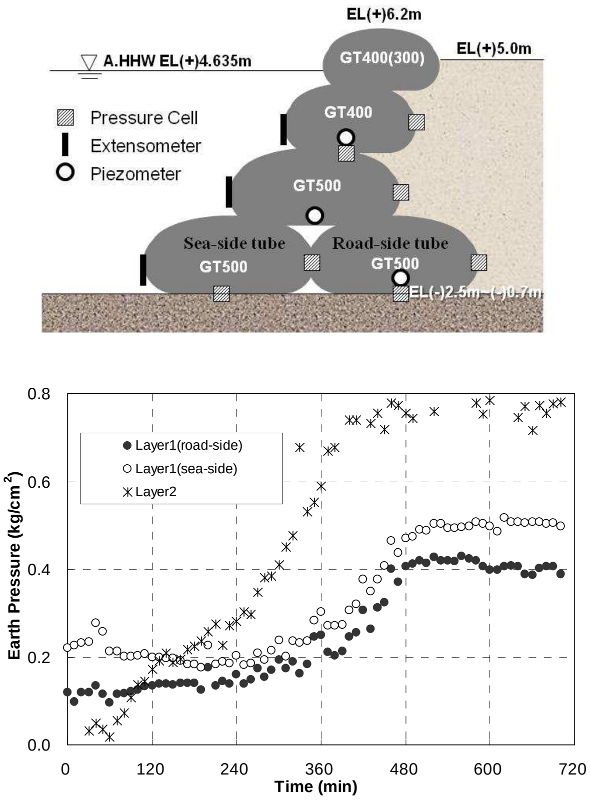


Figure 20: Instrumentation sections (left) and earth pressure measurement results during geotextile tube constructions (right)

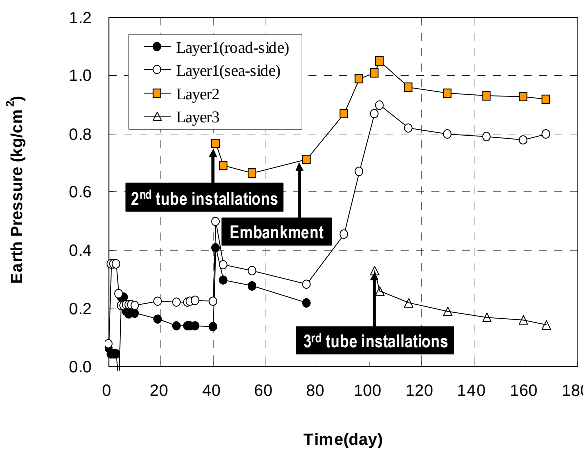


Figure 21: Long-term variations of the earth pressure beneath the tube after constructions

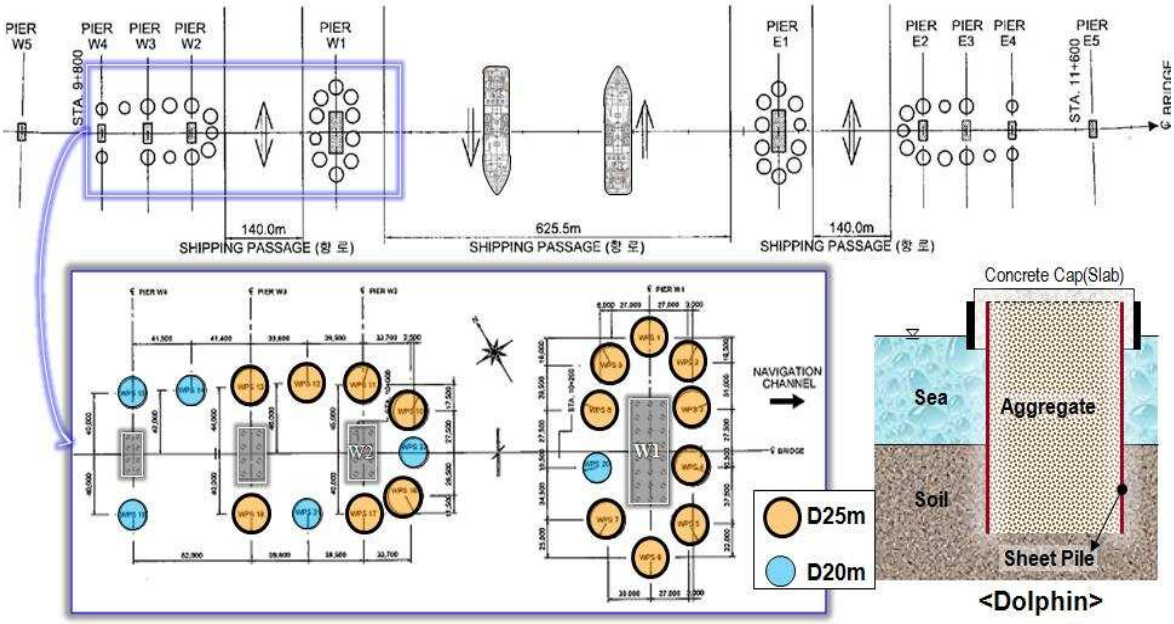


Figure 22: Layout of the dolphin as SIP

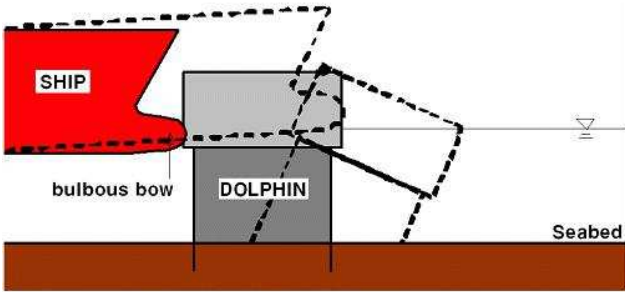
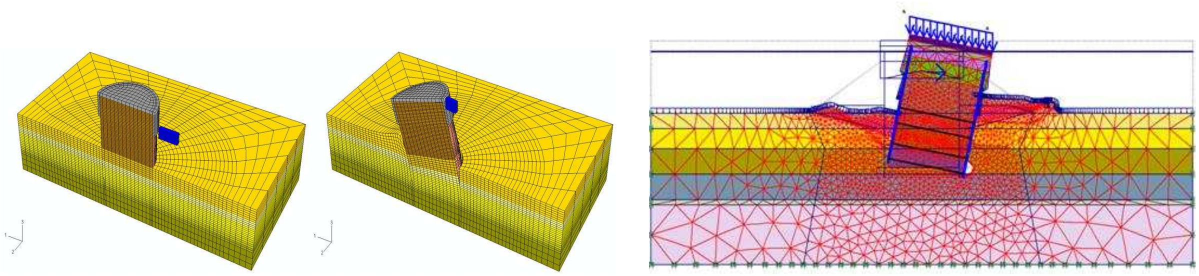


Figure 23: Dolphin's deformation due to the ship collision



(a) 3D FEM analysis

(b) 2D FEM analysis

Figure 24: (a) 3D FEA results - Displacement contour before the impact and after the impact.
(b) 2D FEA results for the impact

Designs were performed with numerical analyses of which constitutional model was verified by the physical model experiment using the geo-centrifugal testing equipment. The 3D non-linear FE models were used to analyze the structural response and energy-dissipating capability of dolphins which were deeply embedded in the seabed. In order to be able to treat the large number of design situations and soil profiles the bearing capacity calculations were also checked using 2D FEA program. It was evaluated that this would provide a conservative estimate compared with the much more cumbersome and time consuming 3D analysis. However, for the ship impact it was considered imperative to be able to quantify and qualify the behavior by 3D dynamic analyses using 3D FEA program. As a result of comparison between numerical analyses and model tests, a very convincing correspondence was observed along the entire displacement range and it seems that assessing dolphin behavior by FEM model give more conservative results than actual dynamic behavior (Figure 25).

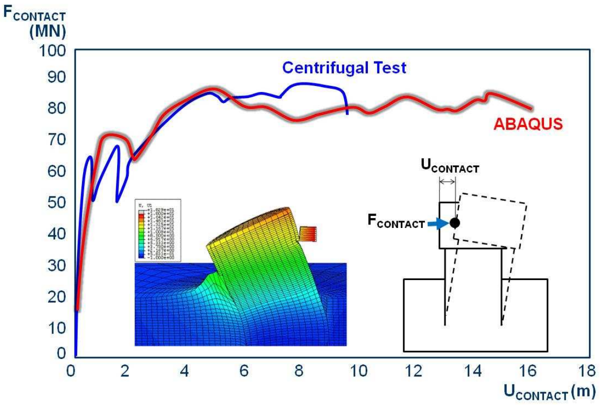


Figure 25: Comparison of the FE analysis and the centrifuge test

Dolphin structure itself was also evaluated during the ship impact according to the types of the vessel impact. Figure 26 shows the example of dolphin displacement at the impact by the ship bow and ship side respectively.

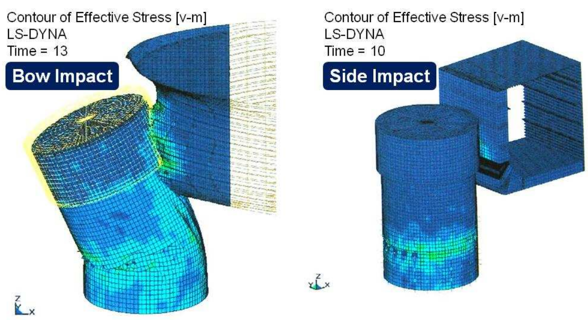


Figure 16: Displacement shape of the dolphin

6.3 Risk Analysis

Design process flow of the SIP structure is described in Figure 27. Total design was divided into 3 parts: traffic analysis, risk analysis, and structural (geotechnical) analysis. Vessel collision risk was assessed by probability based analysis with AASHTO Method-II. Annual frequency of bridge collapse (AF) was computed for each bridge component and vessel classification. The AF can be taken as multiplication value of the annual number of vessel, the probability of vessel aberrancy, the geometric probability of a collision between an aberrant vessel and a bridge pier, and the probability of bridge collapse due to a collision with an aberrant vessel. For the design of the Incheon Bridge as a critical structure, the maximum AF shall be less than 0.0001. The computed AF of the Incheon Bridge through the risk analysis for 71,370 cases of the impact scenario was less than 0.5×10^{-4} and satisfies design requirements (Lars Hauge et al., 2009; Figure 28).

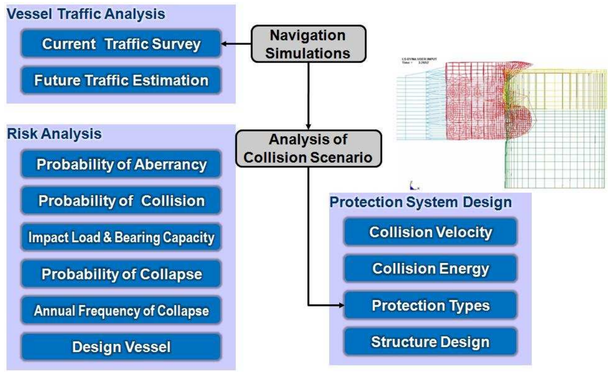


Figure 27: Design procedure of the SIP

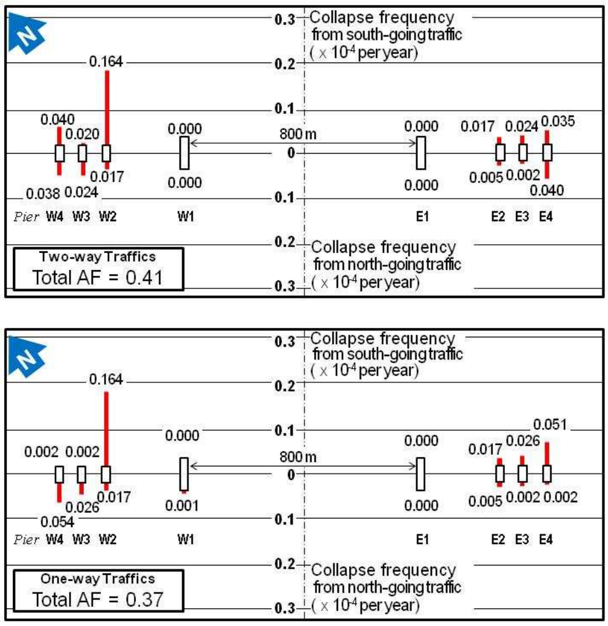


Figure 28: Calculated annual frequency of collapse after installation of the SIP dolphins

1. SUMMARY AND CONCLUSIONS

General process and major concerns of the substructure design revolved around the foundations in the Incheon Bridge project were introduced. Foundations of the bridge consist of drilled shafts, large diameter bored pile foundations which were penetrated into the bedrock under the seabed. A single pile-bent type foundation system was selected as well as the pile-cap type foundations.

New design scheme based on LRFD concept was applied for the project. Limit states method provided a consistent design manner between structural engineers and geotechnical engineers and it also contributed to establish a reasonable framework for design. Since it has not been easy to define the value of resistance factor which can be influenced by various factors, the application of LRFD into the foundation design was also a challenge to geotechnical engineers. The estimation of bearing capacity and settlement of rock socketted drilled shafts was carried out based on the understanding of the site condition, the ground properties and pile load test results. The results of the load tests were thoroughly analyzed by a number of experts to determine the resistance factor, giving a unique opportunity to improve the current LRFD concept in Korea. At the beginning of the project, many kinds of the test and investigation were carried out to find out the design parameters accounting for the detail design. Especially, full-scale static pile load tests were conducted for both the offshore sites and the onshore areas not only to determine site specific resistance factors but also to remove any excessive margin of the stability came from the conventional ASD practices.

Overall behavior of the axially loaded pile which was socketted to the bedrock layers was briefly summarized. Analysis of the pile head connections with the pile cap was presented also. Lateral behavior, displacement characteristics and structural analysis for the pile were not dealt with in this article.

Geotextile tubes to block seawater were made to construct the foundation at the foreshore site whose tidal difference between ebb and flow was so large. Rip-raps which were designed by physical modeling and analysis are spread around the pile to prevent the scouring of the foundation.

Circular dolphin structures made of the flat sheet piled wall and in-filled aggregates surround the piers near the navigation channel to protect the bridge against the collision with aberrant vessels. The structural design of the dolphin as a ship impact protection system was performed with numerical analyses of which constitutional model was verified by the physical model experiment using the geo-centrifugal testing equipment. 3D non-linear finite element models were used to analyze the structural response and energy-dissipating capability of dolphins which were deeply embedded in the seabed. The dolphin structure secures external stability and internal stability for ordinary loads such as wave and current pressure.

ACKNOWLEDGEMENTS

This paper introduces the overall achievements in foundation design of the Incheon Bridge. There are so many engineers who have contributed to complete this landmark project. Professor Myoung-Mo Kim, former president of Korean Geotechnical Society and his young geotechnical engineers of Seoul National University led a task team to establish a pile design guide for the project. Professor Eun-Chul Shin of the University of Incheon carried out the tests and field instrumentations. Engineers of SAMSUNG C&T Corporation managed to design and construct the offshore structures as a main contractor. Deltares (Geodelft) and COWI joined the SIP designs. Author expresses gratitude to their efforts and contributions.

REFERENCES

[1] AASHTO (1994), Guide Specification and Commentary for Vessel Collision Design of Highway Bridges

[2] AASHTO (2007), LRFD Bridge Design Specification, 4th Edition, American Association of State Highway and Transportation Officials, Washington DC.

[3] Cho, Sung-Min (2009), “Geotechnical Engineering Progress with the Incheon Bridge Project”, Proceedings of the International Symposium on Urban Geotechnics: Development and Future of Incheon Metropolitan City, Korean Geotechnical Society

[4] Cho, Sung-Min et al., (2009), “Incheon Bridge: Facts and the state of the art”, Proceedings of the International Commemorative Symposium for the Incheon Bridge, Incheon,Korea, pp.18-28

[5] Cho, S.M., Jeon, B.S., Park, S.I., and Yoon, H.C. (2008), "Geotextile Tube Application as the Cofferdam at the Foreshore with Large Tidal Range for Incheon Bridge Project", Proceeding of the 4th Asian Regional Conference on Geosynthetics, China

[6] Foye, K.C., Salgado, R., and Scott, B. (2006), “Resistance Factors for Use in Shallow Foundation LRFD” Journal of Geotechnical and Geoenvironmental Engineering, Volume 132, Issue 9, pp. 1208-1218

[7] Kim, J.H., Kim, Z.C., Shin, H.Y., Cho, S.M., Schaminee, P.E.L, and Gluver, H. (2007), “Centrifuge testing for the design of ship impact protection of Incheon Bridge Project”, Proceedings of the 16th International Offshore and Polar Engineering Conference, Lisbon, Portugal, pp.1613-1618.

[8] Korea Expressway Corporation (2005), Design Report for Incheon Bridge Construction Project: Section 2

[9] Lars Hauge, Henrik Gluver, Oliver Kübler, Jørgen Steenfelt, Jens-Erik Jepsen, and Erik Yding Andersen (2009), “Ship Impact Protection Design for the Incheon Bridge”, Proceedings of International Commemorative Symposium for the Incheon Bridge, Incheon, Korea, pp. 466-487

[10] University of Incheon (2007), Research report : Geotextile tube construction and Instrumentation for Incheon Bridge