Foundation design methodology for the Padma Main Bridge

S.H.R. Sham, G.X. Yu & S. De Silva
AECOM Asia Company Ltd., Shatin, New Territories, Hong Kong

ABSTRACT: The foundation design of the Padma Main Bridge poses many challenges since, in addition to the large live loads it has to support, it must also be able to withstand significant forces and conditions imposed by the environment, such as wind, earthquake and very deep river bed scour, which could potentially expose free lengths of pile up to about 65m. In order to formulate a sensible design approach, the frequency of occurrence of wind speeds, various intensities of earthquakes, and depths of scour of the river bed were closely studied and probabilities of occurrence of these environmental events at the bridge site were determined. This paper describes how these various events of varying probabilities of occurrence were combined to determine a sensible combination of the various environmental events to be used as the load cases in the design of the bridge foundations. The paper then details the structural as well as geotechnical aspects of the piled foundation design.

1 INTRODUCTION

The Padma Multipurpose Bridge will be constructed in an area of poor ground conditions and severe environmental effects. With bedrock estimated to be about 8km below ground level, the foundations will be constructed on soft ground with the piles acting largely on skin friction. The effects of earthquakes and scour are considerable. Design Consultant AECOM has carried out detailed analysis of the piles using sophisticated state-of-the-art technology, to derive a foundation arrangement that is both economic and robust.

Figure 1: A view of Padma Multipurpose Bridge from the riverbank

This paper presents the development of a three-dimensional finite element structural model for each bridge module that comprises six 150 m long spans, with associated piers, pile caps and the piled foundations. The soil surrounding the piles was modelled as springs acting at regular nodes along the lengths of the pile. In the dynamic model the springs were connected through dashpots to masses that represent the free field; the tech-

nique being based on the Penzien Method. The variations in the river bed with scour were hence represented by the depth of the springs, masses and dashpots along the piles. The design strong ground motions have then been imposed on the model at the stipulated depth to simulate the earthquake events for the dynamic analyses load cases. The design forces and stresses derived from the model have then been used in the structural design of the piles.

The forces have also been imposed on a model of the foundations using the soil structure interaction program PIGLET, to determine the overall group behaviour of the piles, the settlement of the pile cap, the individual pile settlements, lateral deflections and the bending moments. The axial forces on the piles determined from PIGLET were then used to carry out the geotechnical design of the piles and to determine the pile length and the required geotechnical capacity of the piles.

Different forms of foundation have been investigated, during the design process, including raking steel tubular piles and vertical bored cast in situ piles. By consideration of cost and constructability, the optimum foundation solution has been derived.

2 SITE CONDITIONS

2.1 *Geology and ground conditions*

The location of the bridge alignment lies on the Padma River in Bangladesh between Mawa and Janjiar. Geological sections across the river have been developed (Figure 2) to present the ground conditions based on the available information from the captioned Main Bridge Stage 1 - ground investigation (GI) works and Padma Bridge Approach Road Geotechnical Investigation works. Stage 2 – ground investigation works, with sophisticated in-situ tests down to 150m below the river bed level, are currently being carried out (De Silva, Wightman & Kamruzzman, 2010).
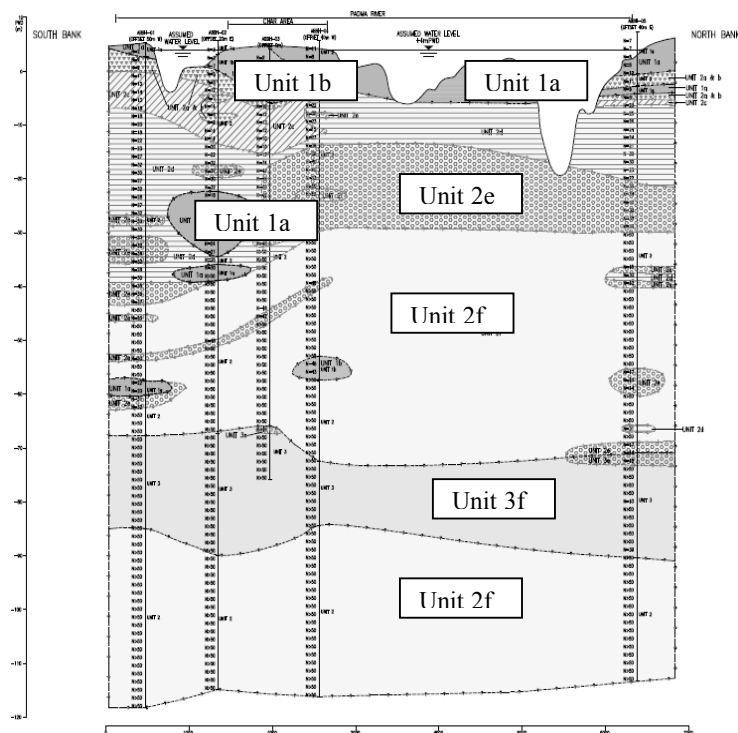

Figure 2: Sections showing geological units

The pile design is based on the currently available limited GI data. The pile design will be further reviewed and refined when the stage 2 GI information is made available. The soil within the main bridge area is basically sand with a thin layer of clay/ silt on top. As presented in the section (Figure 2), the ground condition shows a thin layer of Unit 1a (fine content larger than or equal to 50 %.) with majority Unit 2 (less than 20% fine materials) underneath. Tables 1 and 2 give descriptions of the geological units. A thin layer of Unit 1b (comprises of soil with 20% to 50% fine materials) is identified between Unit 1a and Unit 2 on the Char area which lies close to the middle of Padma River but it is absent on the either bank, or even deep under the river. Approximate 20m thick Unit 3 (soil with less than 20% fine materials) stratum is identified from generally -70m PWD to -90m PWD. In general, SPT-N values increases with depth due to increase compaction. The

stratigraphy of sub-units change with depth but the south bank generally shows lower SPT-N values than the north bank at same depth. This may be due to the presence of the Unit 1a relatively deeper on this bank. More fine materials are evident on the Mawa side. Section below indicates up to 15m thickness of superficial strata Unit 1a present on the north bank (Mawa) of the River.

Table 1: The geological units identified from Stage 1 GI Programme and the Feasibility Study Investigations

Geological Unit	Criteria *		Typical Descriptions of the Materials Recovered from the Boreholes*
Unit 1a	50%≥finer than 0.06mm	Soil with fine content (Clay & Silt) larger than or equal to 50%	Fine soil dominant by clay and silt. Commonly silty CLAY, clayey SILT and slightly sandy SILT, with trace of Mica. Various colours including brownish Grey, dark grey, grey
Unit 1b	20%≥finer than 0.06mm ≥50%	Soil with 20% to 50% fine materials (Clay & Silt)	Generally coarse soil dominant by fine sand with much silt. Commonly very silty SAND. With trace of Mica. Mainly grey colour.
Unit 2	finer than 0.06mm≤50% Coarser than 2mm≤10%	Soil with less than 20% fine materials (Clay & Silt) and less than 10% materials are coarser than Coarse Sand	Coarse Soil dominant by fine and medium sand. Commonly silty SAND and slightly silty SAND. Poor graded with trace of mica. Mainly grey but randomly brownish grey.
Unit 3	finer than 0.06mm≤50% Coarser than 2mm≥10%	Soil with less than 20% fine materials (Clay & Silt) and more than 10% materials are coarser than Coarse Sand	Coarse soil dominant by fine and medium sand, with much coarse sand and gravel. Commonly slightly silty SAND and gravelly SAND. Poorly graded with race of mica. Mainly grey but randomly brownish grey.

Table 2: Classification details of different geological units

Geological Sub-unit	Typical SPT N	Classification of soil^
a & b	0 < N ≤10	Very loose to loose
c	10 < N≤17	Medium dense
d	17 < N ≤ 32	Dense
e	32 < N ≤ 50	Very dense
f	N > 50	Very dense

2.2 *Site seismicity*

Based on studies by Bangladesh University of Engineering and Technology (BUET), the maximum PGAs derived using the Abrahamson & Silva attenuation relationship are recommended as the design PGA values to be adopted at an elevation of -120 m PWD (in "bedrock") in the Padma Bridge design. These recommended values are presented in Table 3.

2.3 *River scour*

Scour is a significant consideration in the design of the foundations and consequently detailed studies have been carried out by sub consultant Northwest Hydraulics Consultants. The general scour level near the river bank for a return period has been determined at -47m PWD near the banks. In addition to the general scour, a further local scour depth of 15m is estimated at the piers for the steel raking piles or at least 21m where vertical concrete bored piles are installed. The scour levels to be considered in the pile analysis under different types of pile are stipulated in Table 4.

Table 3: Seismic design parameters

Return Period (Years)	Recommended Horizontal PGA in cm/s^2 (Abraham and Silva 2008) at -120m PWD	Recommended Horizontal PGA in terms of " g " at -120m PWD	Recommended Horizontal PGA in terms of " g " at river-bed/ground level
2	4	0.004	0.008
10	11	0.011	0.022
50	32	0.032	0.064
100	51	0.051	0.102
200	80	0.080	0.160
475	141	0.141	0.282
1000	230	0.230	0.460

Table 4: Scour levels to be used in the design

Pile Type	Design Period	Scour Level at Pier Location Close to the Bank	Scour Level at Pier Location Mid River
Raking Steel	1 in 100 yrs	-62m PWD	-62m PWD
Tubular Pile	1 in 500 yrs	-70m PWD	-70m PWD
Vertical	1 in 100 yrs	-68m PWD	-68m PWD
Concrete Bored Pile	1 in 500 yrs	-75m PWD	-75m PWD

The soil material above the scour level as specified is considered not to contribute to the positive skin friction of the pile capacity. An extreme scour level, the check flood scour, at -70m PWD close to the bank and at -55m PWD at mid river is also taken into account in the pile design.

3 PILE FOUNDATION OPTIONS

3.1 *Pile arrangements*

For the main bridge piled foundation, there are totally two types of options. The first one will be the steel raking piles, the second one will be vertical concrete bored piles. Both of them have been analyzed. The no. of pile being analyzed are as follows:
- 6 Steel Raking Bored Piles (for the seismic isolated bridge deck) – Figure 3.
- 8 Steel Raking Bored Piles (for the bridge deck without seismic isolation) – Figure 4.
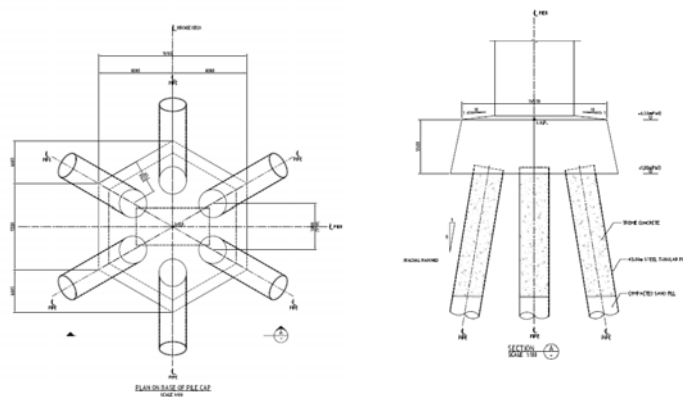- 12 Vertical Concrete Bored Piles (without seismic isolation) – Figure 5.

Figure 3: Foundation arrangement for six raking steel piles (includes seismic isolation of deck)
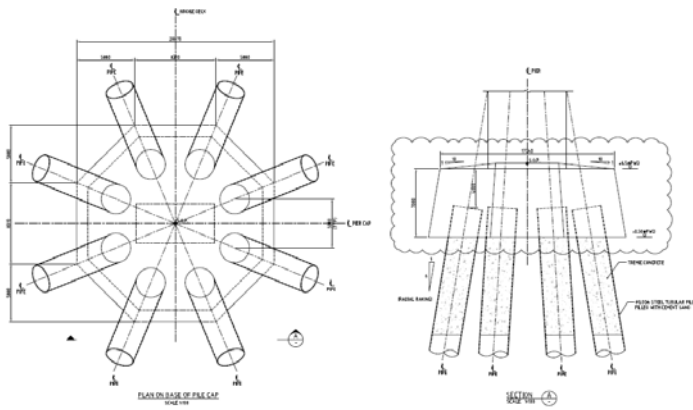
Figure 4: Foundation arrangement for eight raking steel piles (no seismic isolation system)
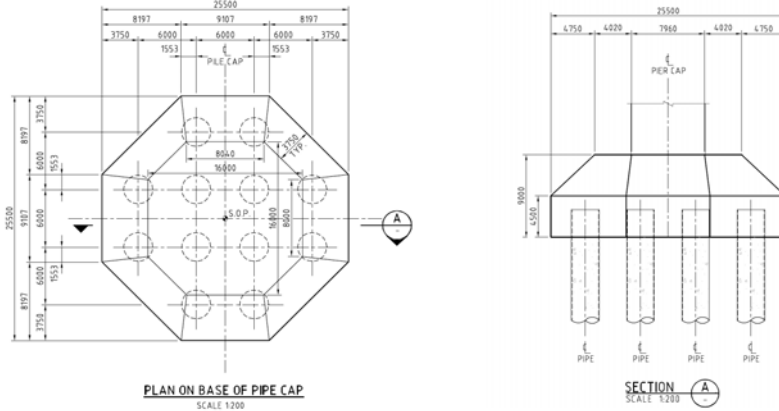

Figure 5: Foundation arrangement for twelve vertical concrete bored piles (no seismic isolation system)

3.2 *Geotechnical pile design*

The forces have also been imposed on a model of the foundations using the soil structure interaction program PIGLET, to determine the overall group behaviour of the piles, the settlement of the pile cap, the individual pile settlements, lateral deflections and the bending moments. The axial forces on the piles determined from PIGLET were then used to carry out the geotechnical design of the piles and to determine the pile length and the required geotechnical capacity of the piles.

Table 5: Summary of the Geotechnical Pile Design

	Design 1 (6 Piles with seismic Isolation)	Design 2 (12 Piles without seismic Isolation)	Design 3 (bored piles without seismic Isolation)
Type of Piles	Steel Raking	Steel Raking	Vertical Concrete
No. of Piles	6	8	12
Diameter (m)	3	3	3
Scour Level(100-year)	-62mPWD	-62mPWD	-68mPWD
Pile Founding Level (mPWD)	-114mPWD	-96mPWD	-121mPWD
Critical Load (MN)	81.2	64.7	67.4
Shaft Resistance (MN)	58.2	38.8	40.6
End Bearing (MN)	83.7	83.7	84.8
Geotechnical Capacity (MN)	83.7	68.3	67.8
Ratio Geotechnical Capacity/Pile Design Load	1.03	1.05	1.01

3.3 *Structural design of piles*

A 3-dimensional non-linear time history dynamic analysis has been performed for the structural design of the foundations to determine the impact on the structure of seismic action. For the plan alignment of the main bridge, the subtended angle is less than 18 degree (the radius is 3000m and one module of the Main Bridge is 900m), the structure may be modelled as a straight line in plan.

Where foundation and/or column connection forces have been determined from plastic hinging of the columns, the resulting force effects have been determined without consideration of combined load cases. The seismic analysis has considered soil structure interaction using an appropriate range of soil parameters reflecting the actual site conditions. Since there is no active fault within 5 km from the construction site, when the bending moments in the piers due to permanent loads are small, the vertical component of seismic action does not need to be considered (BS EN 1998-2, 4.1.7).

In the nonlinear dynamic analysis, the bending moment and curvature relationship can be used to describe the nonlinear behaviour of the plastic hinge. Each column shall have a minimum lateral flexural capacity (based on expected material properties) to resist a lateral force of $0.1 \times$ Pdl , where Pdl is the tributary dead load applied at the center of gravity of the superstructure. For large amplitude deformations P-δ effects have been considered. Plastic hinges have been detailed in accordance with AASHTO LRFD Clauses 5.10.11.4 and 5.10.12.
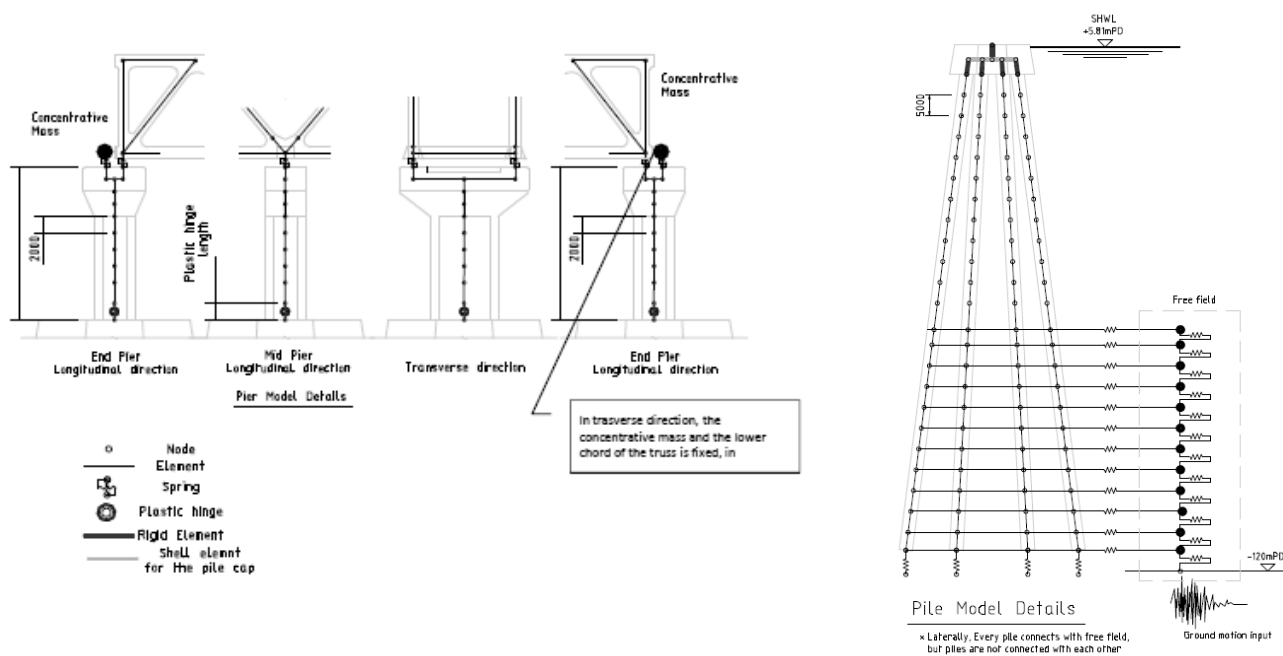

Figure 6 Modelling Structural behaviour of piers and piles

This main bridge behaves in complicated manner due to its height (120m) and the large mass of the superstructure pile cap and pile. A three dimensional non-linear time history dynamic analysis, using a modified Penzien model (see Figure 6), has been adopted for carrying out the design. This model is divided into two parts, the structure and the free field soil. The interactions between the structure and the free field are simulated by lateral spring links. Related to the free field part, in order to determine the equivalent shear modulus and effective damping ratio between each layer of the soil, free field analysis has been carried out beforehand by program SHAKE. Subsequently, a 3-dimensional dynamic analysis has been carried out using the equivalent shear modulus and effective damping as input data.
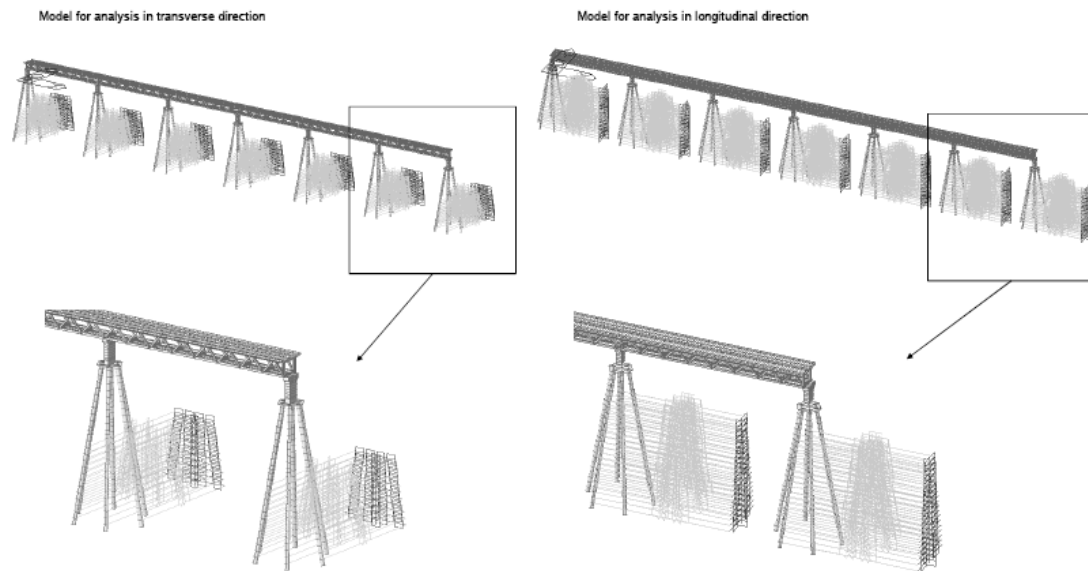
Fig. 7 Global modelling of the piers including the deck structure

In the dynamic model the springs were connected through dashpots to masses that represent the free field; the technique being based on the modified Penzien Method. The variations in the river bed with scour were hence represented by the depth of the springs, masses and dashpots along the piles. The design strong ground motions have then been imposed on the model at the stipulated depth to simulate the earthquake events for the dynamic analyses load cases (refer to Figure 8 for the seismic ground motions).
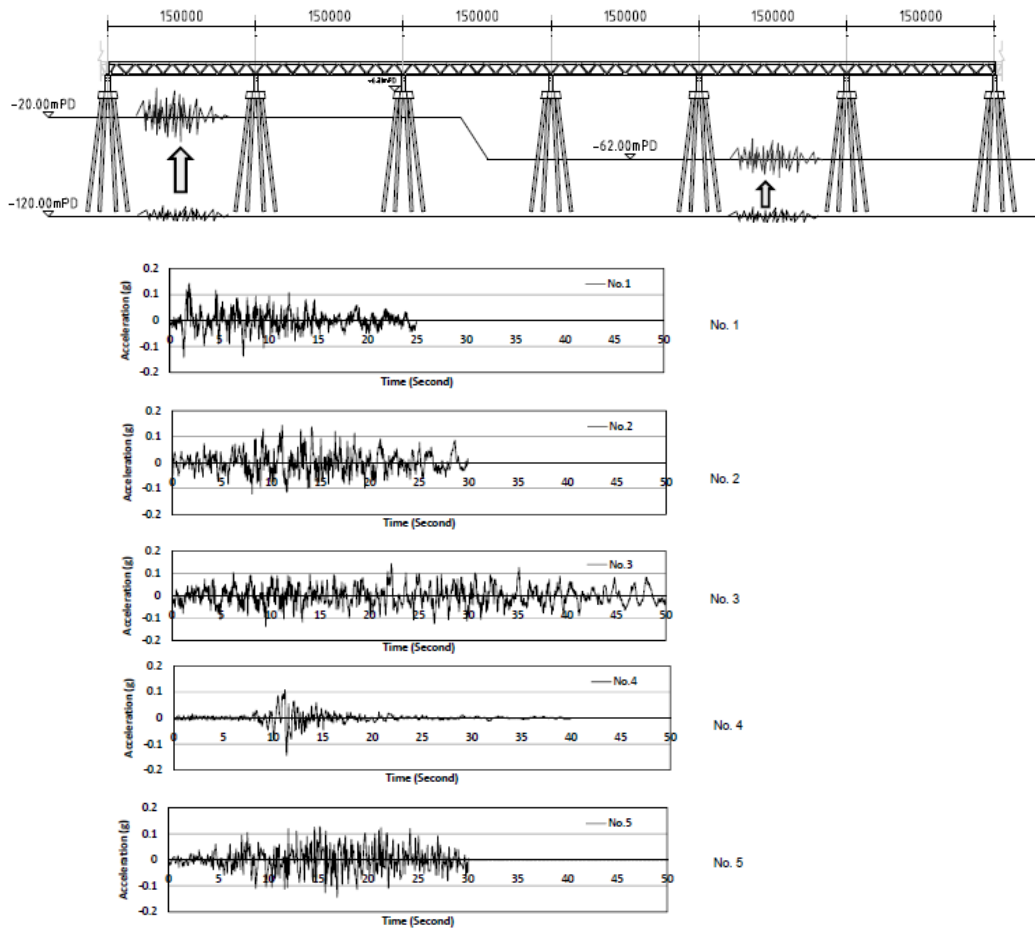

Figure 8 Application of seismic motions to global model

The design forces and stresses derived from the model have then been used in the structural design of the piles (refer to Figure 9 for bending moment diagrams). Each of the cases represents a different combination of scour for an individual bridge module. It can be seen that for piers with deep scour, large bending moments extend a long way down the pile, making it impossible to reduce the structural thickness over depth.
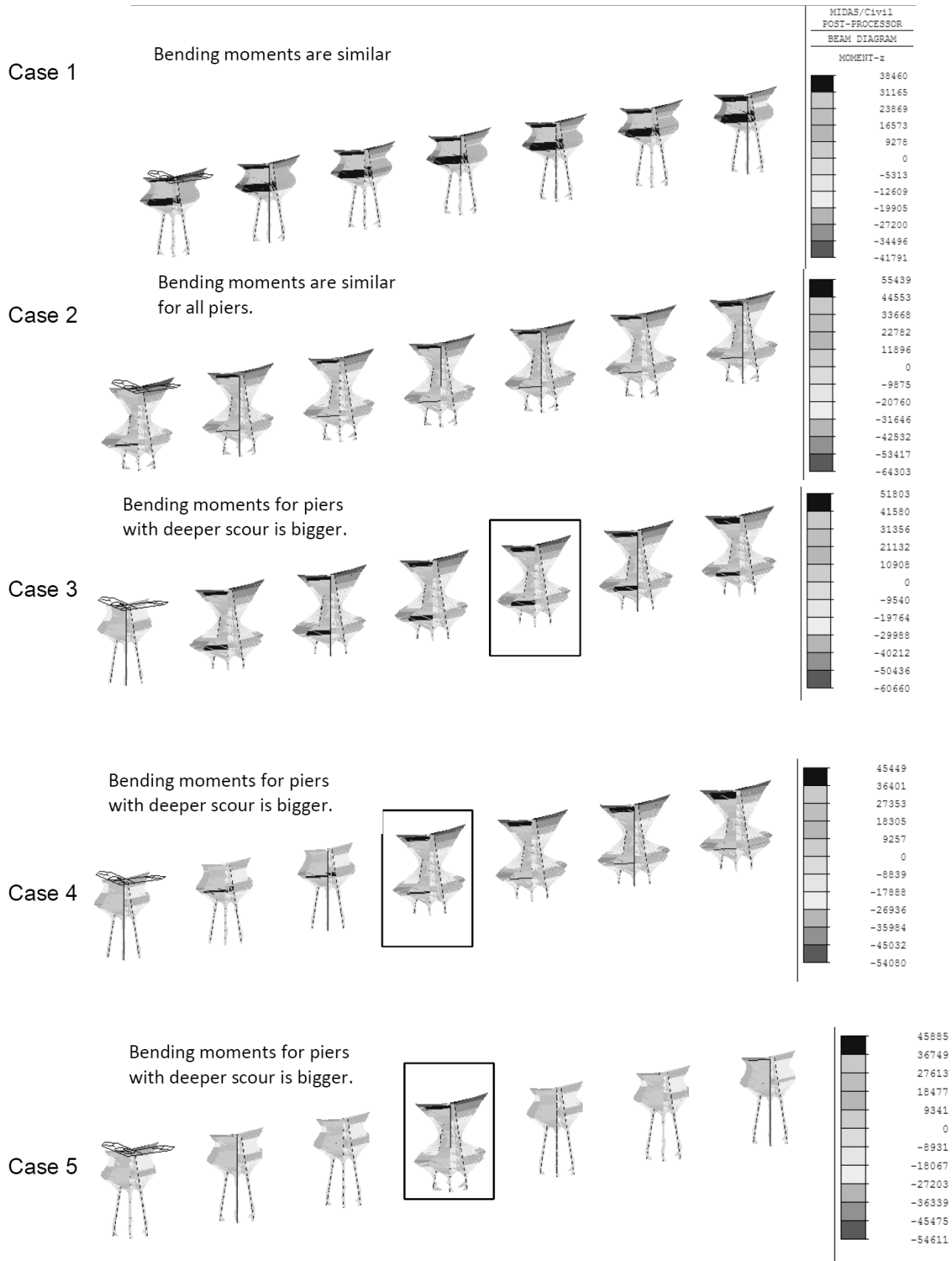

Figure 9 Results from the global model giving the design values for longitudinal bending moments in the piles

3.4 *Effect of deck isolation on the foundations*

The initial seismic design strategy was to dissipate seismic energy through plastic hinging in the columns. To achieve an efficient seismic design, all columns are engaged evenly and simultaneously by Shock Transmission Units (STUs) in the longitudinal direction and by concrete shear blocks in the transverse direction. In addition, all plastic hinge regions in the columns are to be properly detailed to ensure ductile behaviour over many cycles of earthquake excitations.

The foundations are to remain elastic and be designed for the plastic moments and shears developed in the columns above. Adopting this system requires a piled foundation of 8 steel raking piles. Each pile is 3.0 m in diameter and filled with sand for stability. The pile cap is in octagon shape with an overall dimension of 20 m x 20m x 7 m thick.

An alternative pile arrangement was investigated using 3m diameter concrete bored piles. Rather than using the rake of the piles to resist the lateral loads from seismic events and ship collision, lateral loads would be taken in bending by the piles. In order for the piles to have sufficient capacity to withstand the large lateral loads the piles would need to be heavily reinforced. With reference to Figure 8 it can be seen that the large bending moments extend a long way down the piles, to -70m PWD. In order resist this moment the temporary casing used to construct the piles would have to become permanent down to -70mPWD and act compositely with the reinforced concrete infill. There would need to be at least fifteen such piles and the piles would need to be founded at -120m PWD. The final problem with this system of foundations, was the large displacements that would be experienced by the deck from seismic effects. The displacements were determined to be excess of the criteria set for the railway. Consequently the use of concrete bored piles has not been pursued in the design.

A further alternative investigated, has been the adoption of a seismic isolation system for the bridge. Isolation bearings have been used worldwide to mitigate seismic response by isolating structures from seismic input. Isolation bearings can accommodate thermal movements with minimum resistance, but will engage under seismic excitations. In this strategy, all primary structural members will remain elastic without any damage (or plastic hinging). In particular the action of friction pendulum bearings has been studied. Friction pendulum bearings use the characteristics of a pendulum to lengthen the natural period of the isolated structure so as to reduce the input of earthquake forces. The damping effect due to sliding mechanism also helps mitigating earthquake response. Since earthquake induced displacements occur primarily in the bearings, lateral loads and shaking movements transmitted to the structure are greatly reduced.

It has been found that the loads transmitted from the deck to the substructure are significantly reduced if a seismic isolation system is adopted for the bridge. If the seismic isolation system is employed the number of 3m diameter steel tubular piles can be reduced from eight, for each pier, to six. This reduction in the number of piles more than offsets the cost of the isolation system. It should also be noted that plastic hinges should not form in the substructure in a seismic event, consequently reducing the potential for major maintenance in future. Seismic isolation does not benefit the bored pile solution, as the piles are too flexible, leading to a long period and failure to activate the isolation system during a seismic event.

When considering all the above factors it has been decided that the seismic isolation system is adopted, with six 3m diameter steel tubular piles per pier.

4 DRIVABILITY OF STEEL TUBULAR PILES

As the axial forces generated on the raking piles will also be large, when combined with the vertical loads, the piles need to be embedded deep in order to be able to carry these axial loads with the required factors of safety. The lengths of the piles are expected to penetrate into the soil is of the order of 115m, depending on the riverbed level at the time of driving. Since these are raking piles, they can only be installed by driving. Therefore, a drivability analyses has been undertaken in order to check if it is feasible to drive these piles to the required depths at the proposed rake angle in the soil strata encountered at the site. The drivability assessment was carried out using the software program GRLWEAP.

The drivability analysis involved first of all conducting a static soil analysis based on the underlying soil conditions representing the Soil Resistance at Time of Driving (SRD). When the proposed pile is raked at 1 in 6, the soil stresses along the pile shaft at any increment depth in the pile is considered purely due to pile shaft resistance under fully open ended pile base conditions (cleaning and removing of soil from within the annuls of the pile) with no contribution arising from friction from the intrados of the steel tubular pile and from end bearing. This can be achieved by constantly clearing and removing the soil from within the pile annulus.

The dynamic analysis has been conducted by using the commercially available computer program called "GRLWEAP" to simulate the drivability of the pile. In order to simulate the pile driving conditions, the following parameters were required and entered in the program:

1. Static soil analysis based on API code
2. SPT N value based on β-method
3. Driving system characteristic
4. Gain/Loss factor of pile resistance as a result of soil disturbance during driving the pile

The predetermined resistance distribution for any particular depth was calculated based on unit shaft resistance and end bearing (SRD) with respect to depth is input into the program. A wave equation analysis is then performed with the gain/loss factor during pile driving into the ground is assumed to be 0.8. Since the shaft resistance and toe resistance would be reduced when the soil is being disturbed during driving.

Four cases of drivability assessment were performed each considering different pile hammer driving system to install the 3m OD steel tubular pipe pile. The pile driving systems were selected based primary on considering the factor on pile driving efficiency, stroke and energy output of the hammer system.

It was found that there are hammers available for driving the piles to the required depth. The most suitable hammer has a weight of 1130kN, a stroke of 2m and a driving energy of 2260kJ. With such a hammer the maximum driving stress on the pile is 196MPa, well within the allowable stress of 300MPa. The drivability has subsequently been confirmed with the relevant hammer manufacturer.

5 CONCLUSIONS

The difficult ground conditions and severe environmental conditions have made the foundations for the Padma Multipurpose Bridge one of the most challenging aspects of the design. Deep scour combined with earthquake loading, requires piles to be designed for large lateral loads with the piles unsupported for a length of 65m. In developing suitable design solutions, AECOM has utilised the state-of-the-art geotechnical and structural technologies, to provide the bridge with a robust foundation system that should require minimal maintenance in future.

6 ACKNOWLEDGMENTS

This paper is submitted with the permission of the Bangladesh Bridge Authority, the Government of the Bangladesh.

REFERENCES

Sham, S.H.R. & Tapley, M.J. 2010. The design of Padma Multipurpose Bridge – challenges and solutions in design of the river spans. *Proc. IABSE-JSCE Conference, Dhaka, 10-12 August 2010*
De Silva, S., Wightman, N.R. & Kamruzzaman, Md. 2010. Geotechnical ground investigation for Padma Main Bridge. *Proc. IABSE – JSCE Conference, Dhaka, 10-12 August 2010*