

Design And Construction of the Sutong Bridge Foundations

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The Sutong Bridge across the lower Yangtze River in China will have, when completed, the longest span (1,088 m) and the highest towers (306 m) of any cable stayed span in the world. The foundations for this record setting span will also be record setting. They are located in water depths exceeding 30 m with maximum flows exceeding 3 m/sec. The soil at the site of the main pylons consists of layers of silty sands and silty clays extending to bed rock at 270 m below river elevation.

This paper describes the innovative methods used by the team of foundation designers and constructors to support this record setting bridge at this very challenging site. This paper covers three specific topics related to the foundations of the Sutong Bridge:

- Design and construction of the 131 drilled shafts (2.8/2.5 m diameter and 114/117 m long) under each of the two main pylons, including the post-grouting of all drilled shaft tips.
- Design and construction of the scour protection for the two main pylons.
- Construction methods used to construct the waterline pile caps (113.8 m by 48.1 m by 13.3 m deep) under each pylon.

Introduction

Detailed design of the Sutong Bridge was performed by The Design Group of Sutong Project. Construction management of the entire project is being performed by the Jiangsu Provincial Sutong Bridge Construction Commanding Department (CCD). COWI A/S and Ben C. Gerwick, Inc. served as special consultants to CCD during design and construction of the bridge.

The width of the Yangtze River at the bridge site is approximately 6 km and the total length of the Sutong Bridge is approximately 8 km. The site conditions for the main towers are extremely challenging. The northern pylon, Pier 4, is located in about 30 m water depth and the southern pylon, Pier 5, is located in about 16 m water depth. The river is subject to both high fresh water run off volumes and tidal effects, creating currents exceeding 3.0 m/sec in the extreme conditions.

Maximum potential wave heights at the site exceed 3.5 m. The river is alluvial and subject to rapid changes in bottom contours due to high erosion and deposition rates. The river bed at the northern pylon consists of sandy materials and at the southern pylon, the bed material is mainly silty loam and silty clay. These site characteristics create a condition where the river bottom will immediately respond to the introduction of any structure such as a bridge



[Fig.1] Sutong Bridge – 1,088 m Main Span

pier or pylon. Hydraulic model studies for the bridge, performed by the Nanjing Hydraulic Research Institute, predicted up to 29 m of scour (100 year return period) at the south pylon with a caisson foundation and up to 24 m of scour (300 year return period) at the same location for a large diameter pile foundation solution (Jensen, 2004).

Bedrock is located at approximately 240 m below the river bottom. The soils in the upper 240 m consists of layered sediments of fine sand, coarse sand, silty sands and gravels with occasional layers of clay.

The river is the main waterway to the entire Yangtze Basin with heavy barge traffic and up to 50,000-t container ships in the main navigation channel.

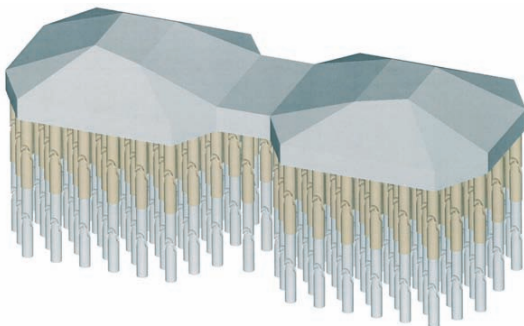


[Fig. 2] Bridge Location

Foundation Design

The foundation for each A-shaped pylon consists of 131 drilled shafts, 2.8/2.5 m in diameter. See Fig. 4 for layout of the pile cap and drilled shafts at each pylon. The drilled shafts are capped by a 13 m deep dumbbell-shaped pile cap with plan dimensions of 113.8 m by 48.1 m.

The bottom of the cap tremie seal is positioned at Elev. -10.0 m, approximately 12.0 m below mean sea level. The drilled shaft casings are 2.8 m diameter with a wall thickness of 25 mm. See Fig. 5. The permanent casings extend from Elev. -7.0 to Elev. -53. The drilled shafts beyond the casing tip are 2.5 m diameter and extend to a design tip elevation of -124 at Pier 4 and -121 at Pier 5. Post grouting of the drill shaft tips was performed to increase the total ultimate capacity.



[Fig. 4] Pylon Foundation



[Fig. 5] 2.8 m Dia. Drill Casing



[Fig. 3] Sutong Bridge

The ultimate load capacity of the drilled shafts was confirmed by four offshore load tests, two at Pier 5, the south pylon, and two at the approach piers. The two tests at Pier 5 confirmed an ultimate capacity of 92 MN (20,700 kips). Testing was performed using the Osterberg Cell Method. Test pile SZ5 was tested twice, before and after tip grouting, to give an indication of the increased capacity obtained through tip grouting. The tests indicated that the bearing capacity of the drilled shaft was increased by 20% or

15 MN (3,375 kips) by the tip grouting. The load deformation curve after grouting showed a much more rigid behavior than before grouting. This result demonstrated that the tip grouting had a positive effect on not only the tip but also on the side

friction on the lower portion of the pile.

Drilled Shaft Design

Drilled shaft design methods have traditionally relied on mobilizing skin friction along the shaft length to resist service axial loads. End bearing, if not discounted, is usually significantly reduced and mainly employed to satisfy extreme load conditions or safety factor requirements. This is mainly due to the concept of strain incompatibility since ultimate end bearing is mobilized at a shaft displacement two or three orders of magnitudes larger than the displacement required to mobilize ultimate skin friction. This is especially true for larger

diameter shafts. Moreover, the displacement needed to mobilize significant end bearing is likely to be larger than estimated due to drilling-induced soil disturbance at the tip of the shaft and debris remaining after cleanup. As a result, developments in drilled shaft construction technology have been mainly focused on increasing shaft diameter or shaft length in order to increase shaft axial capacity.

An effective alternate technique that can be used to increase shaft axial capacity is post-grouting of drilled shaft tips. This technique, although introduced four decades ago, has not been widely used in the US despite its significant potential for cost saving and improvement of quality control of drilled shaft construction. This technique works by effectively preloading and densifying the soil and any remaining debris under the tip of the shaft by pressure grout delivered by a system of pipes pre-attached to the reinforcement cage of the shaft. As a result, larger end bearing capacity can be mobilized at the tolerable displacement limit, thus increasing overall shaft capacity without having to increase its length or diameter.

Drilled Shaft Axial Capacity

The maximum demand axial shaft load was determined to be 44.0 MN for the pylon foundation. With an adopted design safety factor of 2.0, the design axial load capacity of the pylon shaft should therefore be at least 88.0 MN.

The Chinese codes which applied to this project determine the ultimate axial capacity of the drilled shaft as the minimum of: 1) the load at which the shaft settlement is 80 mm, 2) the load at which creep is 0.2 mm per hour at the end of 24 hr load application, and 3) the load at which there is a dramatic and sudden change in the load versus displacement curve.

The soils at the pylon site consist mainly of firm to stiff CL clay extending to elevation -45 m followed by layers of medium to very dense fine to coarse sands and silty sands with occasional loam layers. Bedrock is located at approximately 240 m below riverbed. Based on the soil conditions, and the estimated skin friction of a 2.5 m diameter shaft, the design team decided that a shaft tip elevation of -124 m at the northern pylon and -121 m at the southern pylon would be sufficient if a significant percentage of end bearing could be relied on. To achieve this while meeting the settlement and creep criteria, and to minimize the detrimental impact of

drilling-induced soil disturbance and remaining debris at the bottom of the drilled hole, post-grouting of the shaft tips was selected as the most economical solution. Also, a 2.8 m diameter permanent steel casing was selected to extend to an average elevation of -53 m to maintain hole stability and to increase lateral stiffness of the foundation in the upper clay layers.

Drilled Shaft Construction

Due to the high river currents, all drilled shaft construction was performed from a steel platform constructed over the top of the pier site. In addition, an upstream mooring platform (13 m by 44 m) and a downstream batch plant platform (39 m by 44 m) were constructed immediately adjacent to the main platform. See Fig. 6.



[Fig. 6] Batch Plant Platform

The top elevation of the drill platform was +7 m, approximately 3 m above high water. The main platform was used as both a template to drive the 131 drill shaft casings and to provide a work deck for the drill units. Casings at the northern pylon were driven to grade with a vibratory hammer and at the southern pylon a diesel hammer was used. See Fig. 7. After installation



[Fig. 7] Casing Installation Hammer

of each drilled shaft casing, bracing was added to tie each casing into the work deck, and thereby adding rigidity to the entire work deck.

Drilling was performed with 8 rotary-drill units positioned on the top of the work deck. Drill bits varied depending on the formations encountered. See Fig. 8 and Fig. 9. A bentonite slurry with a minimum positive head of 3 m was used to maintain drill hole stability. Both drilling and concreting operation were conducted simultaneously on the platforms. A minimum concrete strength of 5 MPa (725 psi) was required in an adjacent drilled shaft before drilling was allowed. The post tip grouting operation was also performed concurrently with these operations. However, the grouting operations maintained a minimum distance of 50 m from drilling and concrete placement operations in order to avoid hole-instability problems from elevated pore water pressure created by the grouting operation.



[Fig. 8] 2.5 m Dia. Drill Bit



[Fig. 9] Drill Bit Exiting Hole

The reinforcing cages were fabricated in four sections and coupled together with threaded mechanical connectors on the work deck over the top of the casings prior to lowering them into the drill hole. See Fig. 10. Concrete was placed with a tremie pipe centered in the drill hole.



[Fig. 10] Reinforcing Cage Installation

Concrete was supplied by a batch plant with a capacity of 100 cubic meters per hour, positioned on the downstream platform. Cement and aggregates were delivered to the platform by barges moored directly to the downstream platform.

Post grouting of the drilled shaft tips was performed with 4 loop-shaped pipes pre-attached to the reinforcing cage. The bottom of each loop turned at the bottom of the cage and extended into the interior of the drill shaft approximately 50 cm. Grout exited the pipes through 6 holes, 8 mm in diameter, drilled in the underside of each loop. A one-way valve was created by encasing the loop in a bicycle tire. To ensure that the system was not plugged during the concrete placement operation, clean water was pumped through the system under pressure to confirm open access to the surrounding tip area. Post grouting was performed with neat cement grout.

Post-Grouting of Drilled Shaft Tips

Post-grouting of drilled shaft tips is usually conducted using two techniques; the flat jack, or the sleeve-port (also called tube-a-manchette). In the first technique, grout is delivered by tubes attached to the rebar cage to a steel plate with a rubber membrane underneath at the tip of the shaft. In the second technique, which was used in this project, grout is delivered to the tip through loop-shaped pipes which are pre-attached to the rebar cage. For this project, six tubes were used. The bottom of each pipe, at the tip of the shaft, has a U-shape and extends approximately 50 cm into the interior of the shaft, as shown in Fig. 11. Grout was discharged through eight holes 8 mm in diameter in the underside of each U-shaped pipe, which was encased by a bicycle tire to act as a tight fitting rubber sleeve creating a one-way valve. The

advantage of this system is that it allows clean water to be pumped under pressure to ensure that the system is not plugged during concrete placement operations, and confirms that an open access to the shaft tip area is maintained.



[Fig. 11] U-shaped Grouting Pipes at the Shaft Tip

The major issues in post-grouting of shaft tips, other than the design of the grout delivery system, are to determine the grout pressure and grout quantity. The work of Mullins et al (2006) shows that the level of grout pressure is the most important factor affecting the gain in end bearing and the stiffness in its load-displacement relationship. Another secondary factor is the time period of application of grout pressure. In this project, grouting operations have been controlled by both grout quantity and grout pressure. The project criteria require the grout pressure to reach the targeted level for at least five minutes, and the grout quantity to be at least 80% of the design value. Obviously, since grout pressure acting on the shaft tip is resisted by the shaft skin friction, the maximum grout pressure, and therefore the maximum achievable enhancement in end bearing, is governed by the ultimate skin friction resistance. This also means that the process of post-grouting of shaft tips will cause an upward movement in the shaft as the soil-shaft interface is strained. Therefore, field measurements of the uplift movement of the top of the shaft when related to the applied grout pressure can provide valuable information to verify the axial capacity of production shafts. For the Sutong Bridge, the skin friction of the pylon shafts was estimated as 64 MN; therefore, for a 2.5 m diameter shaft, the estimated maximum grout pressure that can be applied at the shaft tip was 13 MPa plus the buoyant

weight of the shaft. Practically, a lower grout pressure was used since the maximum pressure that can be applied in the field was 7 MPa.

In addition to the upper limit governed by ultimate skin friction, the grout pressure should exceed the hydrostatic pressure at the shaft tip. The project criteria adopted the following method to determine the minimum operating pump pressure:

$$P_g = P_w + \zeta \sum_i \gamma_i L_i \quad (1)$$

where P_g and P_w are the pump and hydrostatic pressures at the shaft tip level, respectively, and γ_i and L_i are the effective unit weight and thickness of each layer (i) above the shaft tip, respectively. ζ is an empirical coefficient for grout resistance, which is a function of the type of soil material at the shaft tip. For sands, ζ ranges from 1.5 to 3.0. Therefore, the estimated minimum operating grout pump pressure in this project was 3 MPa.

Based on the estimated upper and lower grout pressure limits and the required gain in end bearing to meet the design safety factor, the design team decided to use a grout pressure of 5 MPa, which was subject to verification by field tests. Although not used during the design phase of this project, one can estimate the gain in end bearing as a function of the applied grout pressure and shaft settlement using the recent work of Mullins et al (2006), which suggests the following equation:

$$TCM = 0.713(GPI)(\%D)^{0.364} + \frac{\%D}{0.4(\%D)+3.0} \quad (2)$$

where %D is the shaft settlement as a percentage of its diameter D, TCM (tip capacity multiplier) is the ratio between the end bearing at a %D settlement to the end bearing at a settlement equal to 5% shaft diameter. GPI (grout pressure index) is the ratio of the applied grout pressure to the ungrouted end bearing at a settlement of 5%D. The ungrouted end bearing at a settlement of 5%D was estimated as 3.5 MPa for the pylon shafts. Therefore, for an applied grout pressure of 5 MPa, i.e. GPI of 1.43, the estimated TCM from this approach for 1%D, 3%D, and 5%D settlement is 1.3, 2.2, and 2.8, which correspond to an allowable end bearing capacity of 4.6, 7.7, and 9.8 MPa, respectively. Therefore, if this approach was used during the design phase of the project it would also indicate that a 5 MPa grout pressure would be sufficient to obtain at least 25 MN in end

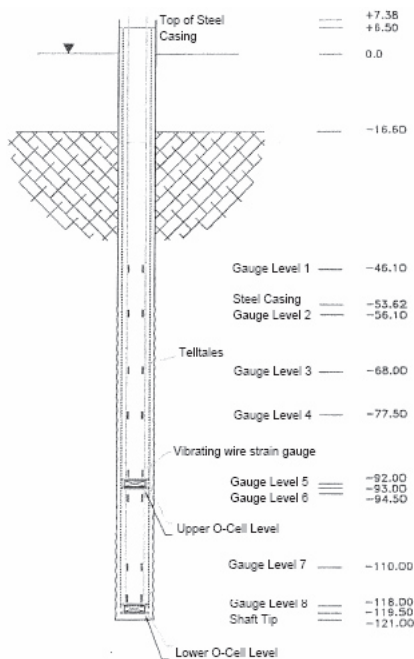
bearing while meeting the project settlement requirement of 80 mm. The design end bearing of 25 MPa was the end bearing targeted to obtain an ultimate axial shaft capacity with safety factor of 2.0.

The design grout quantity was estimated as 100 kN based on the porosity and grout penetration ratio of the soil at the shaft tip. Consideration was also given to the grout going upward around the pile shaft.

O-Cell Tests with Post-Grouting of Shaft Tips

To measure and verify the skin friction and end bearing capacities of the design shafts, several on-shore and offshore shafts were tested. The results presented herein are from an O-cell test on an offshore shaft constructed at the southern pylon site which was loaded before and after grouting.

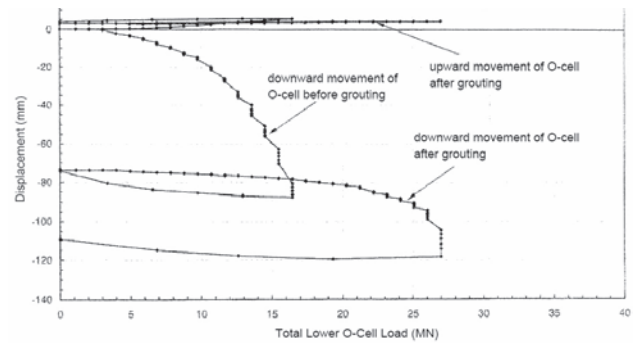
The 2.5 m diameter test shaft had a tip elevation of -121 m with a 2.8 m diameter 25 mm thick permanent steel casing with a tip elevation of -53 m, as shown in Fig. 12. Six U-shaped grout pipes were attached to the shaft reinforcement cage as shown in Fig. 11. O-cells were placed at two levels. The upper level was 28 m above the shaft tip with two 870 mm diameter O-cells and a total nominal ultimate load of 55 MN, while the lower level was 1.5 m above the shaft tip with two 660 mm diameter O-cells and a total nominal ultimate load of 32 MN. Four LVWDTs (Linear Vibrating Wire Displacement Transducers) were installed at each O-Cell level. Eight (8) levels of vibrating wire strain gauges



[Fig. 12] Test Shaft Layout and Instrumentation

and four telltales were used as shown in Fig. 12. The test was conducted by LOADTEST Singapore office. The soil profile at the test site consists of layers of gray silty CL clay down to elevation - 53.5 m overlying layers of fine to coarse sands that extend well below the shaft tip elevation.

The test was conducted in two phases; before and after grouting of the shaft tip. The first phase consisted of a one-stage load test. In this phase the lower O-cells were pressurized in 17 loading increments, each 0.9 to 1.0 MN and lasting 30 minutes, while the upper O-cells were kept closed. As shown in Fig. 13 and summarized in Table 1, at the end of the 17th increment, the total lower O-cells load was about 16.5 MN with a total expansion of 93 mm, mostly from end bearing settlement, which is larger than the 80 mm limit required by the project design criteria. At the end of the first phase tests, the shaft was unloaded in 5 increments.

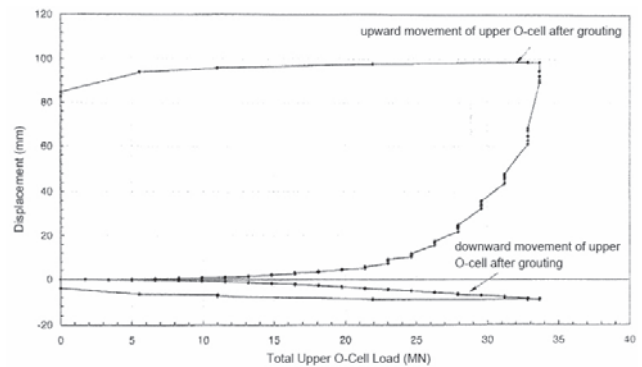


[Fig. 13] Load-displacement Curves Before and After Post-grouting of Shaft Tip from Stage I

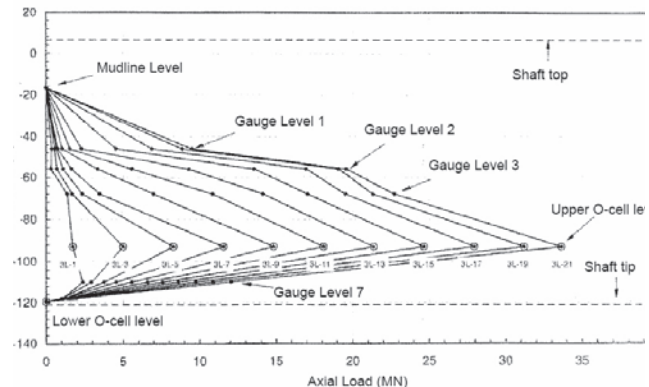
The second phase of the test was conducted 5 days after grouting of the shaft tip. The grouting process was conducted in three cycles to help achieve a uniform treatment of the soil at the shaft tip. In each cycle, the grout pressure was increased in equal increments to the design level, while the grout quantity was distributed equally in the straight grout pipes. In the first cycle 50% of the neat cement grout quantity was extruded, followed by pressure washing the grout pipes with clear water. After at least 1.5 hours of waiting, 30% of the grout quantity was extruded after which the grout pipes were pressure washed again with water. After at least 3.5 hours of waiting, the third cycle was completed by extruding the remaining 20% of the grout quantity. In the first and second cycles, there was more emphasis on controlling the grout quantity, while in the third cycle more emphasis was put on controlling the grout pressure.

Two main stages of load tests were conducted in the second phase. In the first stage the lower level O-cells were pressurized in 28 loading increments, each 0.9 to 1.0 MN, while the upper level O-cells were kept closed to assess the improvement in end bearing after grouting. As shown in Fig. 13, after the final loading increment, an end bearing load of 27 MN was achieved with 44 mm tip settlement (1.8%D), while for a 1%D settlement, the measured end bearing capacity after shaft tip grouting was 5.3 MPa, which agrees well with the value predicted by the Mullins et al (2006) approach. This level of gain in end bearing was satisfactory and showed that the process of post-grouting of shaft tips can be reliably depended on to obtain the required design axial load capacity of the shafts while meeting the project settlement limit and eliminating the risk associated with drilling-induced soil disturbance and remaining debris at the tip of the shaft.

To measure the skin friction response along the shaft after grouting, a second stage of loading was conducted as part of the second phase of the test. This time, the upper level O-cells were pressurized in 1.6 to 1.7 MN load increments while the lower level O-cells were unlocked. The test was stopped when the upper shaft segment ultimate skin friction was reached after moving 106 mm upward, but before reaching the ultimate skin friction of the lower segment, as shown in Fig. 14. From this test, it can also be noticed that the lower 28 m long shaft segment close to grouted tip has a much stiffer skin friction load-displacement response than the upper 77 m segment. This is also evident from the load distribution curves based on strain gauge measurements shown in Fig. 15.



[Fig. 14] Load-displacement Curves after Post-grouting of Shaft Tip from Stage 2



[Fig. 15] Shaft Load Distribution Curve from Stage 2 Using Strain Gauge Data

Scour Protection

The conceptual scour design for the two main piers was performed by COWI A/S, Denmark. The detail design was performed by Jiangsu Provincial Communication, Planning & Design Institute. Hydraulic studies and surveys were performed by Nanjing Hydraulic Research Institute.

The hydraulic design parameters for the scour protection were a combination of the current, water level and in some cases, waves acting at the same time. See Fig. 16.

[Table 1] A Summary of O-cell Tests Procedure and Results Before and After Grouting

Stage	Loading Level		Upper Level O-Cells			Lower Level O-Cells		
			Max Load (MN)	O-cell Hydraulic System	Total Expansion (mm)	Max Load (MN)	O-cell Hydraulic System	Total Expansion (mm)
Before Grouting	1L-1 to 1L-17		0	Closed	-1.7	16.5	Pressure	+93.0
After Grouting	1	2L-1 to 2L-28	0	Closed	-1.4	27.0	Pressure	+118.5
	2	3L-1 to 3L-21	33.7	Pressure	+106.0	0	Free	+113.7



[Fig. 16] 3 m/sec Currents at Pylon

The pile group width perpendicular to the river current is 48 m and the length is about 112 m. The hydraulic model tests showed the extension of the scour around the structure to be essentially equal in all four directions, approximately 60 m.

The ideas presented for scour protection were developed based on COWI's experience in combination with their understanding of the very difficult conditions in Yangtze River with deep water, high currents and high sediment transport.

The major problem associated with the scour protection was its construction. The scour protection in itself had to be made in a way that it was not too difficult to construct, and also, that it would prevent scour during construction. It was assessed that if the bridge piers were made without prior scour protection, the development of scour would be so rapid that it would be difficult to construct the scour protection later on and the bed level would have eroded to such a low level, that the advantage of the existing bed levels would have disappeared.

Therefore, the scour protection as presented in Fig. A-7 in the Appendix was designed in such a way as to allow for construction of piles through the center of the temporary scour protection and then later on the final scour protection could be introduced.

In addition, due to the very high flow velocities and high sediment transport, the adopted scour protection scheme would need to be relatively simple and robust and not require very accurate dredging levels before placing of the material in the scour protection.

It was also desirable to construct the protection in smaller sections that together would constitute the total protection. The final protection should also be robust and be able to function with unavoidable inaccuracies.

With these objectives in mind, the designers refrained from the use of large prefabricated mattresses, gabions or large bamboo/willow mattresses. Such solutions could be used but would be difficult to handle and place in the very high currents prevailing at the site.

The principal ideas for the scour protection of the pylons of the Sutong Bridge included the use of three distinct areas or zones.

(1) The Central Area or Inner Zone

This zone includes the central area where the bridge piles for the main pylons and temporary structures are present.

The area extends 20 m away from the structures. In this area, the river bed would be temporarily protected by use of layers (3 nos.) of sand-filled geotextile bags. See Fig. 17.



[Fig. 17] Filling Geotextile Bags

The idea behind this concept is that by this action, the river bed will be protected but it will still be possible to bore the piles through the protection. After completion of the piling, the final protection was constructed with a filter layer of quarry-run and minimum 2 layers of armour stones (rock).

(2) Outer Area

Beyond the inner zone, the Outer Area is situated. It extends about 40 m further out from the Central Area. The scour protection consists of one layer of sand bags covered with a layer of quarry-run on top of which was placed the same type of rock armour as for the central area.

(3) The Falling Apron Area

Outside the Central and Outer Area is the Falling Apron Area. Its width varies according

to an estimate of the scour depth and the width was set at 1.5 times the actual maximum expected scour depth. The material in this area consists of quarry-run on top of which layers of quarry stones were dumped.

During construction of the Falling Apron Area, it was decided to dump a layer of sand bags at this area as well because extensive scour was occurring.

The concept of the falling apron has been used in many countries for river training structures where the scour is expected to reach to a level significantly below the level at which the structure is or can be built.

The principle is that the material in the falling apron will launch itself down the scoured slope that will thereby stabilize itself.

Table 2 shows the stone sizes of the scour protection for the North and South Pylon. The table also shows sizes of sand bags, which were exposed in the temporary protection during construction and were later on covered with stone material.

[Table 2] Stone and Sand Bag Sizes for the Scour Protection Material at the North and South Pylon

Item	Location	Density [t/m ³]	d ₅₀ [m]	
			North Pylon	South Pylon
Stones	Inner Area	2.65	0.40	0.50
Sand Bags		2.00	0.50	0.60
Stones	Outer Area	2.65	0.30	0.40
Sand Bags		2.00	0.30	0.40
Stones	Falling Apron	2.65	0.30	0.40
Stones at Slope		2.65	0.40	0.60

For practical considerations, the same size of bags were adopted all over and the same stone size for the outer area and falling apron. The actual sand bags used were of size: 1.6 m x 1.6 m x 0.6 m.

Extensive surveys were performed during construction using multi-beam echo sounder in order to control and verify the amount of materials dumped. The dumping of material was performed using a grid with 28 x 26 cells for the North Pylon and 16 x 16 cells for the South Pylon.

With respect to the armor stones, it was essential that the structural integrity be obtained. Therefore, it was crucial that 2 layers of armor stones are present in all areas. In layer thickness, this corresponds to 1.0 m for the Central Area and the Outer Area. For the inner section of the Falling Apron Area, it corresponds to 1.2 m thickness.

The scour protection is a flexible structure that will be subject to some displacement of material. Especially the Falling Apron will be moving during launching when scour occurs at its edges. Therefore a detailed monitoring program was prepared covering the entire bridge alignment.

The solution adopted, with sand bags and stone layers dumped from the water surface, was found to be the most feasible under the given difficult circumstances with water depth up to 30 m, high currents and zero visibility. The future erosion at the edges of the protection will be prevented from progressing close to the bridge piers by the use of the Falling Apron concept for the outer edge of the scour protection.

See Fig. A-6 and A-7 in the Appendix for further details of the scour protection system

Pile Cap Construction

The 13.3 m deep pile cap for each pylon is positioned at the water line with the bottom at Elev. -7.0 m and the top at + 6.3 m. The caps were constructed by first building a double-walled steel caisson in-the-wet, directly above the final location. The 1.8/2.0 m thick double wall or perimeter wall was constructed as a watertight compartment and served four basic functions. See Fig. 18.

It first served as the perimeter stiffening frame that gave the caisson its rigidity during lowering operations. Secondly, it served as the buoyancy tanks to minimize the deadweight of the caisson as it was lowered into the water and down to final grade at Elev. -10.0 m. Third, it served as a temporary cofferdam and exterior permanent form for the casting of the pile cap. And finally, the perimeter wall acted as a permanent ship-impact protection fender during the service life of the bridge. The perimeter wall was filled with concrete below Elev. -1.0 m.



[Fig. 18] 2 m Thick Caisson Perimeter Wall

The first caisson (North Pylon) when initially constructed was 118 m by 52.4 m in plan, 7 m high, and weighed approximately 3050 tonnes. See Appendix Fig. A-3. The bottom of the caisson was a steel plate stiffened by steel trusses that spanned the full width of the caisson and tied into the perimeter walls. The bottom deck of the caisson started out at approximately Elev. +6.0 and was lowered in three basic stages to Elev. -10.0. The first stage lowered the caisson approximately 5 m, at which point the caisson was floating under its own buoyancy. At the end of this stage, the lowering was stopped and the perimeter walls were increased to a height of 18 m and the lowering was completed in two stages to final grade by partial flooding of the cofferdam.

The lowering operation was performed with 16 strand jacks, DL-S418 (See Figs. 19 and 20.) supplied and operated by Dorman Long Technology, Ltd. For the layout of strand jacks see Appendix Fig. A-4.

All 16 jacks were spaced along the perimeter wall of the caisson and sat on support frames positioned over the top of the exterior drill shaft casings. Each jack had a safe working load of 418 tonnes, thus providing a safety

factor of 2.2. The entire caisson was quite stiff and relative movements of only 10 mm between adjacent jacking points created a 35% differential loading. The entire lowering operation was controlled with a Dorman Long P40 computer control system which provided communications between jacks, power-pack and control computer. The lowering operation was performed with strokes of 200 mm and a stroke range of only 5 mm to ensure stable balanced loads between jacks.



[Fig. 19] Typical Strand Jack at Pylon 5



[Fig. 20] Strand Jack Model DL-S418 & Reaction Frames Inside Caisson at Pylon 4



[Fig. 21] Pylon 5 Caisson Being Lowered

Once the 13 m high caisson reached final grade at Elev. -10.0, the caisson was locked in position and the annulus between the drilled shaft casings and the steel plate of the caisson bottom deck was sealed. A 3.0 m deep tremie concrete seal was then placed over the entire bottom area of the caisson except for the 2 m

wide exterior wall. After the tremie seal attained specified strength, the caisson was dewatered (See Fig. 22) and the rest of the pile cap was constructed in the dry. See Fig. 23.



[Fig. 22] Dewatered Cofferdam and Top of Tremie Seal

For the second caisson (South Pylon), the entire caisson was assembled full height and weighed approximately 5,800 tonnes. This caisson was lowered by a strand jacking system from Tonji University to a self-floating condition, and final grade was reached by partial flooding of the exterior perimeter walls. Once at final grade, the tremie seal was placed and the rest of the South Pylon was constructed in the dry. Both lowering operations worked well and everything went smoothly.

Conclusion

The foundation design and construction team on the Sutong Bridge have succeeded in constructing foundations for a world record setting bridge at a very challenging site on the lower Yangtze River. The design team, working in conjunction with their construction counterparts, developed innovative holistic solutions that addressed the very rigorous

requirements of the structural design while remaining fully constructible under extremely difficult conditions

Post-grouting of shaft tips was found to be an effective and economical procedure that was implemented to increase the axial capacity of the drilled shaft foundation of the Sutong Bridge in China. By preloading and compaction of the soil and any remaining debris at the shaft tips, the end bearing capacity can be significantly increased and reliably depended on. This was validated in O-cell tests conducted on shafts before and after tip grouting. Another advantage of post-grouting of shaft tips is the ability to check the axial capacity of each production shaft through measurement of grout pressure and upward movement of the top of the shaft.

Acknowledgement

The authors wish to thank the Jiangsu Provincial Sutong Bridge Construction Commanding Department for the opportunity to assist them on this very challenging bridge project.

References:

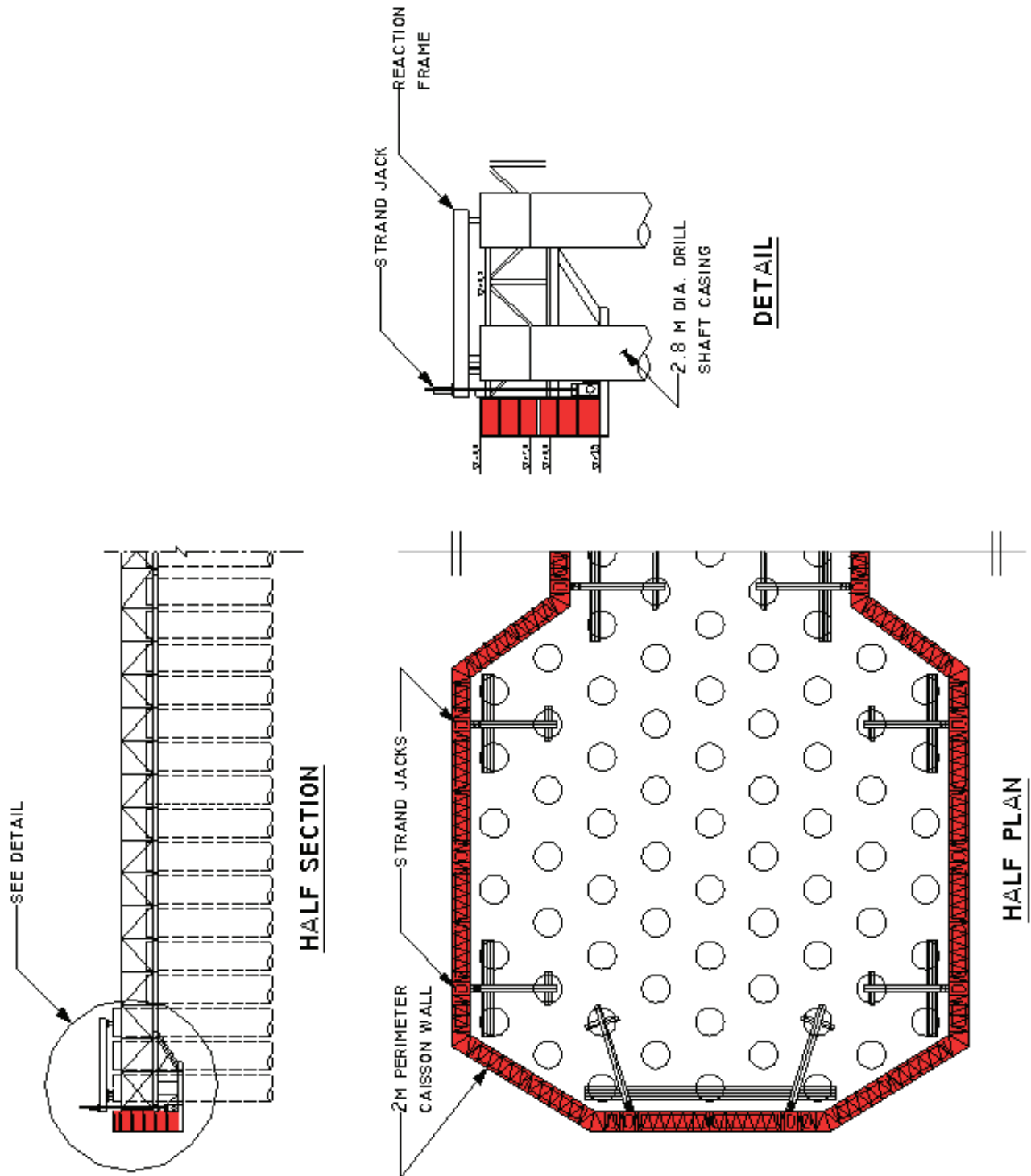
Jensen, O.J., and Truelsen, C., 2004, Scour Assessment & Conceptual Design of Scour Protection for Sutong Bridge, P.R. China.

Appendix:

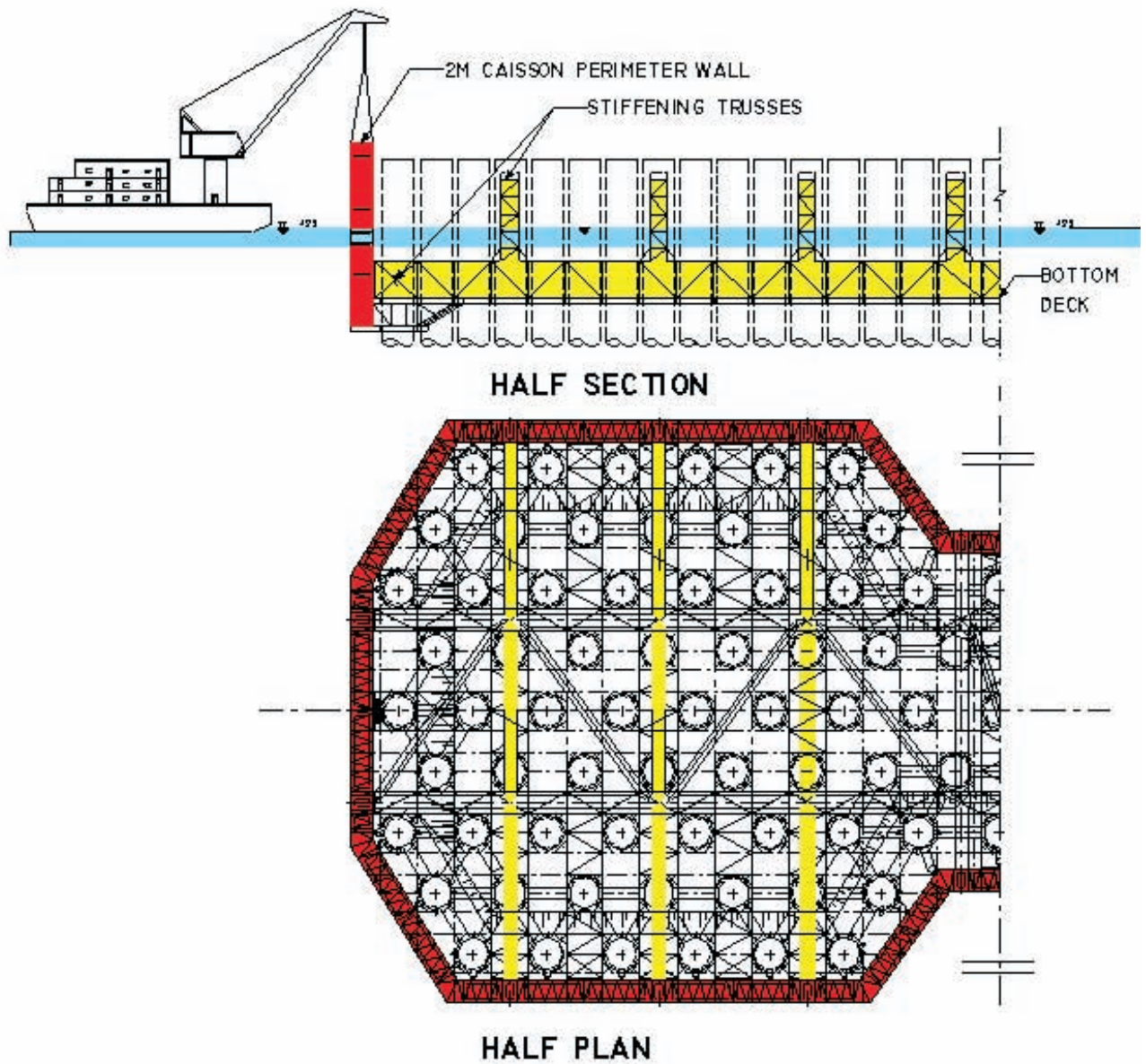
1. Fig. A-1 Main Span Side Elevation
2. Fig. A-2 Pylon Details
3. Fig. A-3 Main Pylon Caisson Details
4. Fig. A-4 Caisson Lowering System
5. Fig. A-5 Perimeter Wall Completion
6. Fig. A-6 Scour Protection Details at Main Pylons
7. Fig. A-7 Scour Protection Plan







[Fig. 23] Completion of Pile Cap in the Dry at Pylon 4



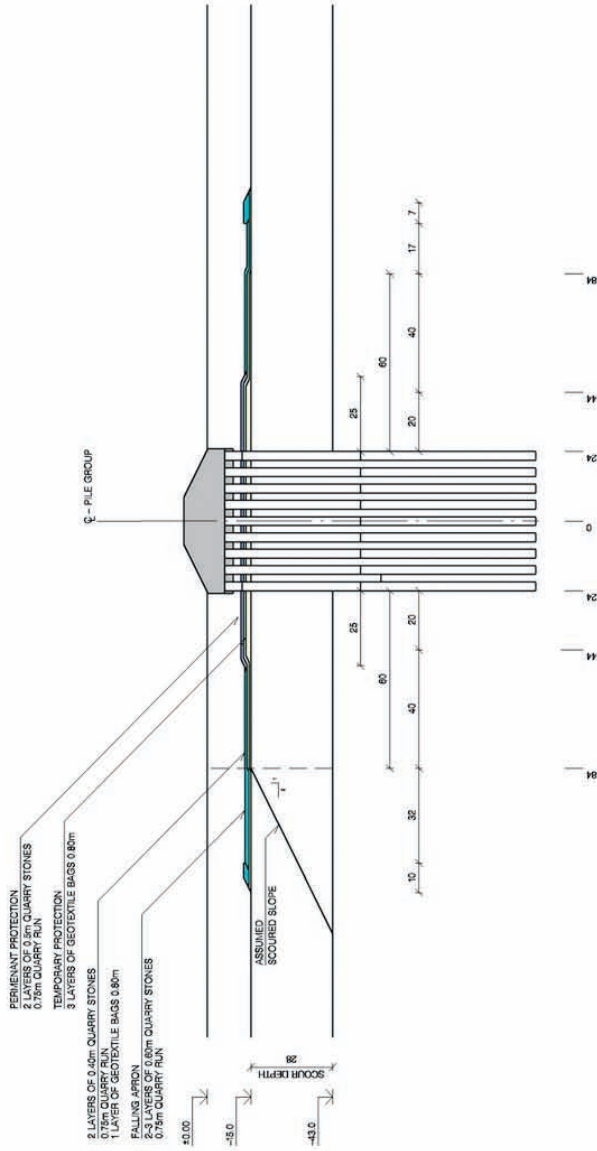
[Fig. A-4] Caisson Lowering System – Stage I



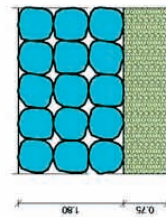
[Fig. A-5] Perimeter Wall Completion

- LEGEND:**
-  GEOTEXTILE BAGS
 -  QUARRY STONES
 -  QUARRY STONES
 -  QUARRY RUN

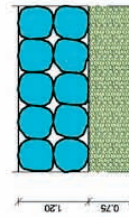
SOUTH PYLON, SECTION N - S DIRECTION
SCALE 1:500



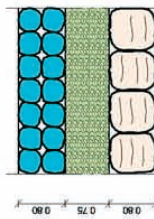
SECTION FALLING APRON, OUTER PART
SCALE 1:25



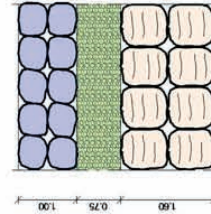
SECTION FALLING APRON, INNER PART
SCALE 1:25



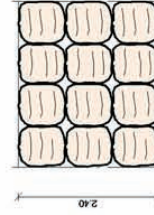
SECTION OUTER HORIZONTAL SECTION
SCALE 1:25



SECTION INNER HORIZONTAL SECTION (COMPLETED)
SCALE 1:25



SECTION CENTRAL TEMPORARY PROTECTION
SCALE 1:25



NOTE:
3 LAYERS OF BAGS ASSUMED REDUCED TO 2 LAYERS DURING PILING

**SUTONG BRIDGE
SCOUR PROTECTION
MAIN PYLONS**

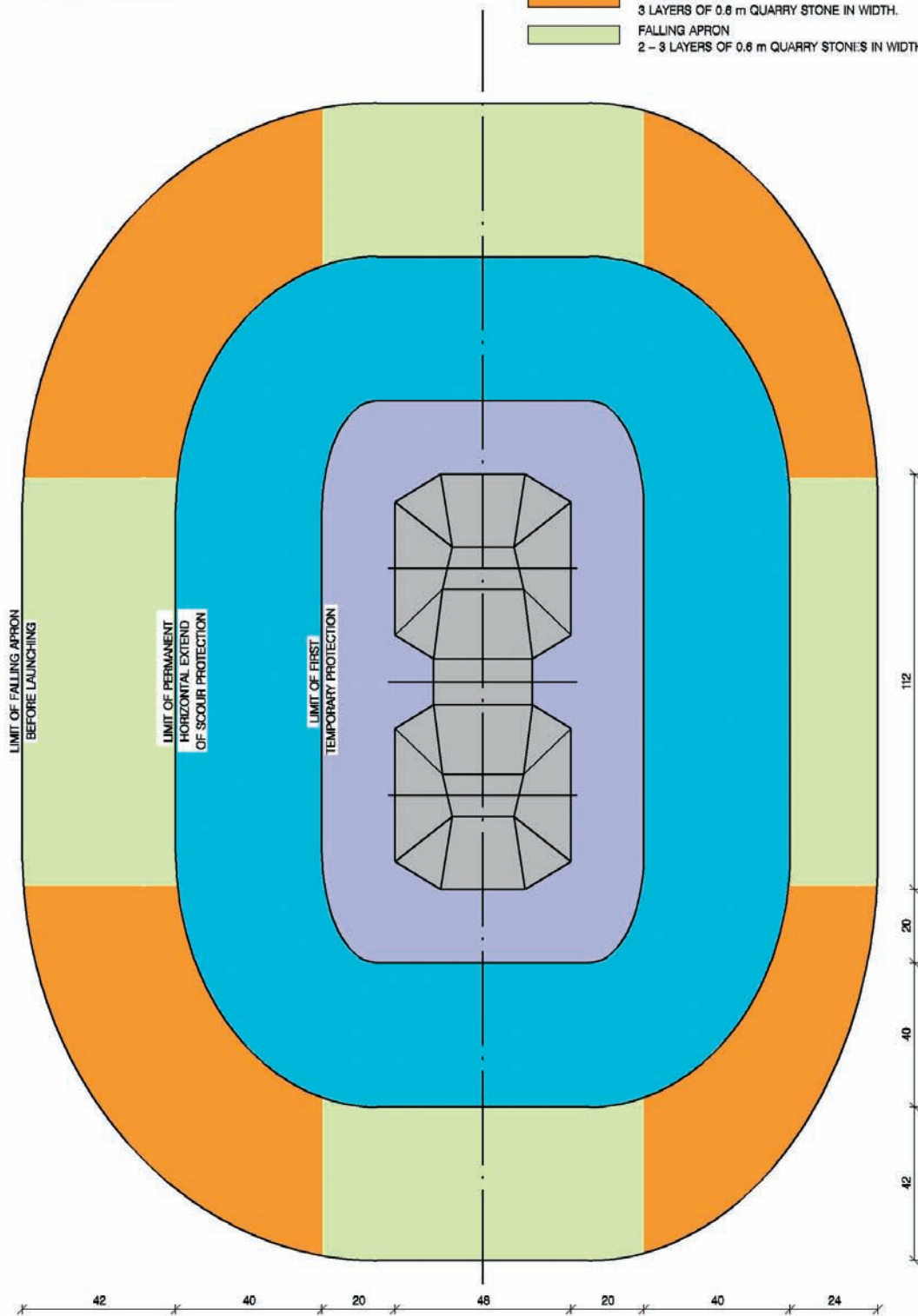


[Fig. A-6] Scour Protection Details at Main Pylons

SOUTH PYLON PLAN

LEGEND:

- FALLING APRON
3 LAYERS OF 0.6 m QUARRY STONE IN WIDTH.
- FALLING APRON
2 - 3 LAYERS OF 0.6 m QUARRY STONES IN WIDTH.



[Fig. A-7] Scour Protection Plan