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QUEENSFERRY CROSSING, UK – SCOUR PROTECTION DESIGN

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Abstract

The Queensferry Crossing is the United Kingdom's tallest bridge and the world's longest three-tower, cable-stayed bridge. On completion it will link Edinburgh with the county of Fife. Ramboll's Marine and Energy Infrastructure team has led the design of scour protection for the main crossing foundations. Analysis of scour potential has been conducted on all nine foundations and, for foundations which require scour protection, a scour protection design has been prepared. Predictions at detailed design were undertaken using a number of methods including hydrodynamic numerical modelling to assess flow velocities in the estuary in the vicinity of the foundation structures. Scour protection design was undertaken in accordance with the recommendations of May et al (2002) and took into account the combined effect of waves, currents, local flow effects, bed shear stress amplification and propeller wash. The scour protection design was verified using a physical model built and tested at Aalborg University. This paper describes the scour prediction and design process including the use of numerical and physical modelling and monitoring to develop the proposed solution.

Key words: scour, physical modelling, scour protection, bridges

1. Introduction

Ramboll is the lead consultant in the Design Joint Venture for the Queensferry Crossing. The other members of the group include Sweco and Leonhardt Andra und Partners. The Contractor is Forth Crossing Bridge Constructors (FCBC), a consortium of Hochtief, Dragados, American Bridge and Morrison Construction.

Figure 1: Queensferry crossing under construction

The bridge, which is presently under construction, will have a total length of 2.67km. The southern landfall of the bridge is located at Echline, west of South Queensferry with the northern landfall of the bridge located at St Margaret's Hope, west of North Queensferry. The main bridge consists of two spans supported on three towers, the North, South and Central Towers. Eight viaduct piers support the bridge deck from the Southern Abutment to the back span of the South Tower and two viaduct piers will carry the bridge from the back span of the North Tower to the Northern Abutment. It is due to be completed in May 2017.

Ramboll's Marine and Energy Infrastructure team has led the design of scour protection for the main crossing foundations including the analysis of scour potential and scour protection design. This paper

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describes the design process.

2. Foundations

The foundations for the structure comprise a combination of circular caisson foundations founded on either bedrock or underwater concrete and spread footings constructed within permanent sheet pile cofferdams, also founded on either bedrock or underwater concrete plugs. Reinforced concrete bases were constructed in the plugged and dewatered caissons, likewise for the pier starter segments which sat upon the bases. The method of construction varied between different locations dependent on the depth of foundation below the water level or ground level. In each case, the excavated formation was founded in rock that was extensively inspected by suitably qualified geologists from Ramboll either directly or via underwater CCTV to confirm that the excavated formation met design requirements and was clear of silt and other soil/debris. As largely gravity solutions, the foundations rely to an extent on the superficial soils for resistance against sliding, overturning and/or bearing capacity. Therefore, where scour is predicted, if it occurs, the stability of the foundation may be compromised.

3. Ground Conditions

The stratigraphy at each of the foundations was characterised in geotechnical design reports. Generally, this comprised Alluvium, and Fluvioglacial deposits followed by Glacial Till. Beneath these soils, rock, historically known as 'oil shales', are found. This rock comprises layers of strong but fractured sandstone, siltstone, mudstone, and limestone.

The scour analysis was primarily concerned with the erosion of the material above rock. Following dredging for construction access, it was found that, in most cases, a layer of cohesive Alluvium would be exposed and remain so on the bed of the Firth. Therefore, it was important to identify a method of scour prediction that was appropriate to these ground conditions. The table below summarizes the key characteristics of the Alluvium most vulnerable to scour.

Table 1: Characteristic Alluvium Parameters

Parameter	Value
Characteristic D_{50} (top layer)	0.03mm – 1.64mm
Characteristic D_{50} (second layer)	1.35mm – 14.28mm
Residual Angle of Shearing Resistance	6.48^{o} – 9.66^{o}

4. Hydrodynamic Conditions

4.1 *Water Levels*

Tides in the Forth Estuary are semi-diurnal. The mean spring tidal range used in the design is given in the table below.

Table 2: Design water levels

Water Level	Ordnance Datum (m)
Highest Astronomical Tide (HAT)	3.41
Mean High Water Springs (MHWS)	2.85
Mean Water Level (MWL)	0.35
Mean Low Water Springs (MLWS)	-2.15
Lowest Astronomical Tide (LAT)	-3.99

Scour assessments were undertaken assuming Mean Water Level as peak tidal flows generally occur about half way between high and low tide and this was considered the most realistic and reasonable assumption. The sensitivity of the predicted scour depths to moderate changes in water level was checked and was found to be insignificant.

4.2 *Waves and Currents*

Scour predictions and design were carried out using combined wave and current conditions at all foundations using 200 year return period wave and 200 year return period current data. The data was defined in the contract and originally produced as part of the Environmental Impact Assessment by Jacobs-Arup (Jacobs et al, 2009) by extrapolating from 50 year return period data produced using a 3D hydrodynamic model in the MIKE3 Flexible Mesh (FM) finite volume modelling package by DHI. The extrapolation was undertaken using relationships in Offshore Technology Report OTO 2001/010 (HSE, 2001). This data has been verified for the detailed design and construction stage by Ramboll's numerical modelling study, as described below. This data is included in Tables 3 and 4 below. Refer to Figure 2 for the locations of the foundations.

Table 3: 200 year return period wave parameters

Location	Water Depth at MWL (m)	Significant Wave Height, H_s (H_{m0}) (m)	Peak Period, T_p (sec)	Direction from (°N)
Pier N1	3.55	1.54	3.85	261
North Tower	7.65	1.76	4.08	270
South Tower	21.10	1.78	4.63	74
Central Tower		1.86	4.12	285
Pier S1	4.55	1.42	4.44	54
Pier S2	2.95	1.28	3.48	307
Pier S3	2.65	1.25	3.50	312
Pier S4	2.15	1.21	3.61	317
Pier S5	1.66	0.96	3.87	328

Table 4: 200 year return period current parameters

Location	Surface Layer Velocity (m/s)	Mid-depth Velocity (m/s)	Bottom layer Velocity (m/s)	Surface Layer Velocity (m/s)	Mid-depth Velocity (m/s)	Bottom layer Velocity (m/s)	Depth averaged current speed (m/s)
Pier N1	-0.15	0.38	-0.15	0.38	-0.14	0.38	0.41
North Tower	-0.27	0.53	-0.27	0.52	-0.24	0.51	0.58
Central Tower	-2.45	-0.30	-2.06	-0.39	-1.97	-0.68	2.17
South Tower	-1.00	0.03	-0.98	0.03	-0.93	0.03	0.97
Pier S1	-0.86	-0.31	-0.85	-0.30	-0.82	-0.30	0.90
Pier S2	-0.49	-0.19	-0.49	-0.18	-0.48	-0.18	0.51
Pier S3	-0.47	-0.25	-0.47	-0.25	-0.47	-0.25	0.53
Pier S4	-0.47	-0.25	-0.47	-0.25	-0.47	-0.25	0.53
Pier S5	-0.14	0.11	-0.14	0.11	-0.14	0.11	0.18

Note: Positive U is towards E, Positive V is towards N

4.3 *Hydrodynamic Model Study*

The numerical model study prepared for the detailed design used the MIKE21 Flow Model FM software package. The numerical model study describes the flow patterns for:

- The baseline conditions, prior to construction
- The construction phase, with temporary access channels dredged for marine plant
- The final foundation structures

A MIKE 3D hydrodynamic model was already established at the Environmental Impact Assessment Stage by Jacobs-Arup. This study built on the results of that model.

The bathymetric data used in the Ramboll model was taken from a geophysical survey conducted in October 2011, supplemented by data used in the Jacobs-Arup model (Jacobs et al, 2009) where there was no data from 2011. In the areas not covered by the surveys the bathymetric levels are based on digitised levels from United Kingdom Hydrographic Office Admiralty charts. The data presented in the charts were more uncertain than the survey data. However, the levels were found to be reasonably close to the levels from the survey data.

The baseline bathymetry was used for the baseline and operational phase simulations. A revised bathymetry was used for the temporary construction phase, based on construction drawings received from site. The boundary data for the modelling was based on a predicted time series of tidal elevations. Because the freshwater input was negligible, the river inflow was been omitted from the model setup.

Calibration and validation of the numerical model were based on a comparison with predicted tidal data time series in the model area. Measurements of water level and current were also included in the calibration and validation. Measured and predicted data from the Jacobs-Arup study were used for comparison. Therefore, the 50 year return period events were used in order to allow for direct comparison. A summary of the results of the model study is given below.

4.3.1 *Baseline Condition (50-yr return period)*

The modelled currents for the baseline scenario were overall fairly close to the design currents and it was evaluated that the range of the modelled currents and design currents with a 50 years return period were comparable. Therefore, it was assumed that the effect of the construction works seen in the model were representative of the prototype.

4.3.2 *Construction Phase (Dredged Channel)*

Generally the current speeds were smaller in the construction phase than in the baseline condition, with increasing effect due to dredging towards the shoreline. At Piers S1 and S2 the reduction is 15%-25% while at S3 and S4 the reduction is 20%-50% of the maximum baseline current speed. At pier N1 the reduction is 15%-40%. At the Towers a smaller current speed occurs only at the North Tower with a reduction of approximately 10%.

The only pier where the simulations showed increasing maximum current speed was in a 150 x 150 m area around the structure at pier S5, where spots with relatively high current speed were seen. It seemed that the dredged channel created local current conditions with vortices and high current speed because of the difference between the relatively deep channel and the surrounding areas in the corner behind Port Edgar Marina, that were dry during ebb. However, the scour prediction using Erodibility Index analysis indicated that even with higher current speeds of 0.8m/s the alluvium at bed level is able to withstand erosion. Therefore, it is unlikely that these temporary current speed increases will cause scour outside of the dredged channel. The area around S5 was monitored as part of a wider environmental study to identify any significant changes to bathymetry and trigger further action if required.

4.3.3 *Operational Phase with Backfilling*

Generally the predicted current speed was marginally smaller in the operational phase with backfilling than in the baseline condition. At the Central Tower and at the South Tower some decrease in the maximum current speed compared to baseline was seen. The reduction in the maximum current speed compared to Baseline is around 5% for the piers S1 to S5 and N1, approximately 10% at North and South Towers and approximately 20% at Central Tower.

4.3.4 *Operational Phase without Backfilling*

Generally the predicted current speed was at the same magnitude as in the case with all the dredged areas backfilled. The current speed was smaller in the operational phase without backfilling than in the baseline condition. However, at S5 there are spots were the current speed increases, as was seen in the construction phase runs. No backfilling of the dredged channel was implemented in the works. Therefore, these results are the most relevant to the design. The result of the model tests, showing the differences to the baseline case is given below.

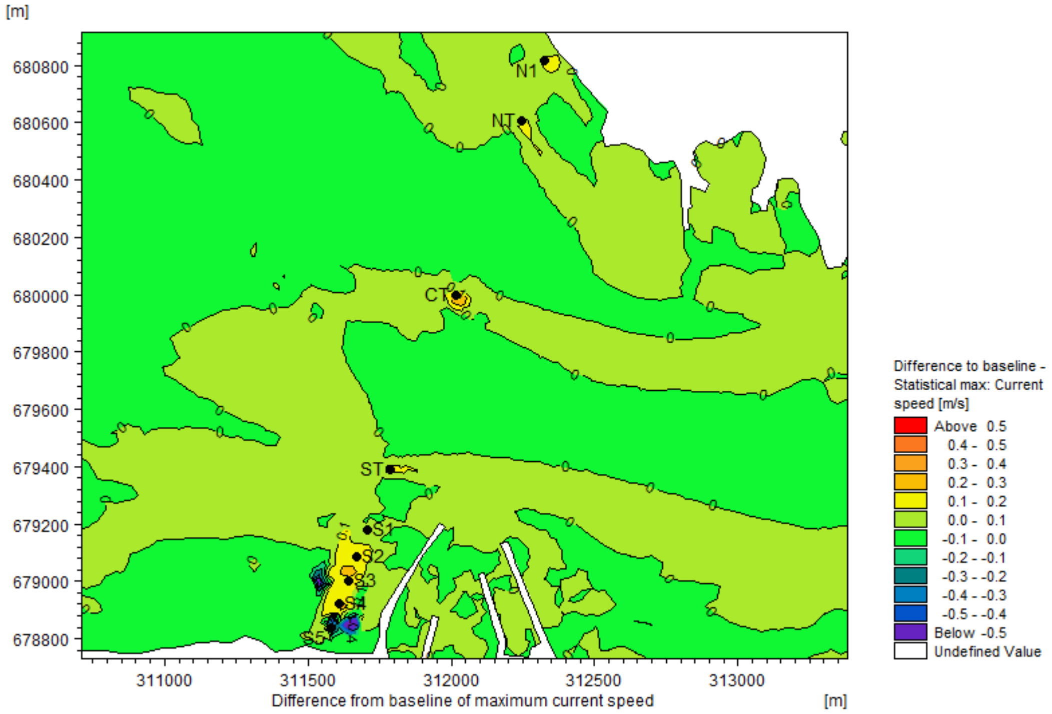

Figure 2: Results of model tests showing differences to baseline for scenario 3 (with wind from the West)

4.3.5 *Conclusion*

The model simulations of the proposed temporary and permanent works, suggested that no significant hydrodynamic impacts, specifically changes in current speed, would be caused by the temporary works and permanent works. The current speeds were reduced in some areas, as a likely result of the deeper water depth due to dredging, but this was not considered to be detrimental to the project. The conclusions arising from these results were assumed to also be valid for the 1 in 200 year return period events.

5. Scour Prediction

At the time of the detailed design of the scour protection, primarily between 2010 and 2012, the principal methods for the estimation of scour at bridges adopted in the UK were based on theoretical and empirical methods. In general these methods did not take account of specific soil properties; having been derived from experimentation with uniform, granular soils. Predicted scour depths were primarily governed by pile diameter and current speed.

The preferred method adopted within the Manual on Scour at Bridges and Other Hydraulic Structures (May et al, 2002) is the modified Breusers et al (1977) method. This method does not use information on the cohesive nature of the soil or soil strength and is for granular soils only.

Typically, scour depths achieved in cohesive soils tend to be less than those generated in granular materials; either because of time-scale or because of material strength and cohesion. Therefore scour predictions using methods for non-cohesive materials are likely to be over conservative when applied to cohesive soil conditions. That said, it is noted that once erosion does start in cohesive materials, the process of erosion, in clumps of material, could still lead to large scour holes forming quickly.

The geotechnical investigations showed that the soil structure in the Forth Estuary varies spatially as well as with depth; however, the bed material at the location of the foundations was consistently identified as Alluvium, a mainly cohesive material. Granular soils were identified in only two samples of the existing bed material. The granular soils were found at a depth which is to be dredged, consequently only cohesive Alluvium would be present at the location of the foundations. Therefore, scour predictions were made using the Erodibility Index method (Annandale, 2006) which took account of the predominantly cohesive nature of the bed material.

The Erodibility Index method of Annandale (2006) is one of the recognised methods for assessing scour competence in rock and is recommended in the US Federal Highway Administration HEC18 manual and is now also recommended in Kirby, et al (2015). The method uses the properties of the bed material to produce an erodibility index, K. This index is a measure of the ability of the bed material to resist erosion. The erodibility index is compared with the supplied stream power, which is a function of wave and current action. If the stream power exceeds the erosion threshold, then scouring will occur.

The influence of both the wave and current conditions were accounted for in the shear stress and bed velocity inputs of the stream power calculation. The wave related bed shear stress was combined with the current shear stress using the method of Soulsby as described in Whitehouse (1998). This gives the maximum bed shear stress due to the interaction of waves and currents, and accounts for non-linearities which occur in wave-current behaviour. Where current velocity and wave velocity were combined, this was done by linearly summing the current depth averaged velocity with the amplitude of bottom orbital wave velocity. Three different wave and current combinations were used to calculate available stream power at each foundation in order to gauge the sensitivity of the results. These were:

1. Mean combined wave and current shear stress, T_{c+w} & current only velocity, U_c
2. Mean combined wave and current shear stress, T_{c+w} & combined wave and current velocity, U_{c+w}
3. Maximum combined wave and current shear stress, T_{maxc+w} and current only velocity, U_c

All analyses considered the condition after dredge, prior to any backfill or placement of scour protection. The analysis predicted that without scour protection, significant scour may occur at four foundations. At other foundations zero scour was predicted. The three different combinations assessed showed variation in results. Combinations 2 & 3 gave the most conservative results, while combination 1 predicted scour at fewer locations. The maximum predicted scour depths are typically governed by the maximum scour depth achievable under the HEC-18 method.

Scour protection was recommended for the four foundations where scour was predicted.

6. Scour Protection Design

Scour protection is to be installed at six of the bridge foundations for the following reasons:

- To prevent exposure of vulnerable geological strata, where scour is predicted.
- To prevent lowering of the sea bed adjacent to the bridge pier, and to reduce the risk of ship collision.
- For structural stability and geotechnical design, where scour is predicted to compromise this.
- For aesthetic reasons, at two foundations in the intertidal zone, even though scour was not predicted.

Scour protection was sized using the recommendations outlined in May et al (2002) and the D_{n50} of the design grading was determined using the average of three separate methods recommended in that document: Escarameia and May (1992), Pilarczyk (1990), and Maynord (1995). These methods are also recommended in Kirby et al (2015). Averaging of the results of these methods was a contractual requirement.

Where piers sit in the intertidal zone, this method does not apply during the initial stages of a flooding tide. Therefore, the scour protection was also treated as a revetment with the effects of breaking waves considered in the stability of the armour stone.

6.1 *Design method to May et al (2002)*

Each of the three formulas is dominated by different parameters, generating different sizes of armour stone from the same input conditions. Table 5 below identifies the sensitivity and dominating factors of each of the three methods.

Table 5: Review of each method contributing to scour protection sizing

Escarameia and May (1992) $$d_{n50} = C_1 \frac{U_b^2}{2g(s-1)} \quad (1)$$

Description of method	Sensitivity analysis in relation to this project	Weighting in final $D_{n5}0$ sizing
Calculates D_{n50} based on a coefficient of turbulence intensity caused by the obstruction in the flow of the river, i.e the bridge pier.	Directly linked to the depth averaged velocity 0.1m/s increase relates to approximately 40mm increase in D_{n50} size.	Produces the largest D_{n50} values of the three methods.

Pilarczyk (1998) $$d_{n50} = \frac{\mu}{(s-1)} \frac{0.035}{\Psi_{CR}} \frac{K_T K_Y}{K_S} \frac{U^2}{2g} \quad (2)$$

Description of method	Sensitivity analysis in relation to this project	Weighting in final $D_{n5}0$ sizing
Calculates D_{n50} based on the stability of armour stone within an inclined revetment.	Not dominated by any one factor, small changes in depth averaged velocity and angle of revetment change D_{n50} marginally. Method more applicable to scour of revetments.	Produces a D_{n50} value in between that of the other two methods.

Maynord (1995) $$d_{s30} = S_f C_s C_v C_f y_0 \left(\sqrt{\frac{1}{s-1}} \frac{U}{\sqrt{K_1 g y_o}} \right)^{2.5} \quad (3)$$

Description of method	Sensitivity analysis in relation to this project	Weighting in final $D_{n5}0$ sizing
Calculates a $D_{n5}0$ (scaled up from a D_{s30}) based on the angle of the scour protection to the horizontal. i.e the angle of the revetment. Again mainly applicable to scour of rock revetments.	Dominated by the value of ε, (angle of the bank to the horizontal), all other coefficients remain the same for each analysis as criteria are very general. 0-10deg increase in bank angle relates to an increase in D_{n50} of 20mm whilst a bank angle of 45deg translates to a D_{n50} of 110mm.	Produces the lowest D_{n50} values of all the methods.

The inclusion of the Maynord D_{n50} values in the average grading reduced the scour protection D_{n50} design values to a size smaller than either the Escarameia & May or Pilarczyk D_{n50} values. Therefore, the validity of the averaged D_{n50} design values was checked using the entrainment function, or Shields (1936) parameter method, and the Erodibility Index method of Annandale (2006). This is described below.

The design was conducted in accordance with contractual requirements to average the results of the assessments using the equations above. Taking the Maynord (1995) D_{n50} values into account, it was considered possible that the results may be less conservative. Therefore, the results of the methods above were compared against two other scour protection analysis methods for verification: the Shields(1936) Parameter and Annandale (2006) Erodibility Index methods.

The results of this comparison, which indicated that the D_{n50} values generated using the May et al (2002) recommended methods were higher than those generated using the comparative methods, provided more confidence that the material would be stable under the design wave and current conditions.

6.2 *Other design parameters*

6.2.1 *Armour thickness*

The thickness of armour stone was specified at 2.5xD_{n50} as also recommended in May et al (2002).

6.2.2 *Filter layers*

The filter material layer was designed in accordance with CIRIA (2007) to fulfil the general requirements of stability, segregation and internal stability at the armour/filter interface. Since the bed material was largely cohesive, the filter rule for the interface between the bed and the backfill was considered less significant. Where there is no backfill, the filter material particles may embed into the top of the cohesive alluvium during initial placement. However, it is expected that the in-situ density and finer pore size of the alluvium will prevent significant downward migration of the uncompacted larger grain size/filter material. Where the filter material was to be placed above backfill material, the filter rules, as described above, would be satisfied.

6.2.3 *Extent of scour protection*

The area of scour protection was derived based on guidance given in May et al (2002), this generally states that the area of protection should be two times the diameter of the object that is causing the scour. In most cases this was the bridge pier and the scour protection area was relatively small. However, in some instances the object causing scour was the larger foundation caisson and hence the area of scour protection is considerably bigger. Also, in some situations, where the top of the caisson was near to the surface of the scour protection, the material size was increased in order to mitigate the risk of the caisson being exposed and presenting a greater obstruction to flow and, therefore, inducing more scour. The minimum dimension of scour protection from the perimeter of the obstruction varies from 16m to 34m.

6.2.4 *Edge protection*

A method for assessing the depth of scour in the design of scour protection for sills, as given in Hoffmans and Verheij (1997), was used in this analysis. The method outlined below was employed:

- Assessment of whether a scour hole could develop at the edge of the scour protection.
- Calculation of the equilibrium scour depth of the hole.
- Assessment of the likely consequences for the main scour protection should a scour hole develop and achieve the full equilibrium scour depth.
- Suggestions of mitigation measures which may be included in the design.

The development of a scour hole at the edge of the scour protection may occur due to the turbulence generated by the change in bed roughness at the edge of the scour protection. The turbulence generated by this change in roughness is represented by the turbulence coefficient ω. The turbulence coefficient is dependent on the relative turbulence intensity, r_0, which itself is calculated based on the height of the sill (approximated as 0m for all cases), the water depth, length of bed protection and the Chézy coefficient, C. The turbulence coefficient was found to be of the order of 1.4 to 1.5 for the foundations under consideration and was applied as an enhancement factor to the mean flow velocity. In all cases, only the current flow was considered. The current flows considered were the 1:200-yr return period current flows and the peak spring tide current flows. The latter were obtained from the numerical modelling study prepared as part of the detailed design.

Applying the principle of a falling apron to the development of the scour hole, the hole is expected to develop as follows:

1. Initiation of movement of bed material and development of the scour hole as bed material is carried downstream by the flow.
2. Gradual undermining of the edge of the scour protection followed by the armour material moving down the upstream slope of the scour hole.

3. Development of an armoured upstream slope as the scour protection material moves into place in the scour hole. The length of scour protection that moves into the scour hole is estimated to be the falling apron length (2 x equilibrium scour depth due to spring tide current).

The material size and thickness required for this zone was specified on the drawings. In the event of a scour hole developing, this material would stabilise the upstream slope of the scour protection. This method met the objective of the design, which was to practically consider the risks associated with edge scour and suggest reasonable and economical mitigation measures.

7. Design Validation

A three-Dimensional (3-D) model test study was performed in a wave and current basin at Aalborg University in Froude scale 1:30 to verify the material size specified in the scour protection design. The model tests included:

- Testing of the scour protection around four different foundation geometries: N1, NT, S3 and S1, which required scour protection for geotechnical or structural stability or protection against ship collision.
- Tests at 7 different wave conditions and 8 different current conditions to match the 1 in 200 year conditions as defined above.
- Detection of damage to scour protections using visual inspection and photo overlay technique.

7.1 *Model setup in basin*

The foundations were oriented according to the incident current and wave conditions. Passive absorption was provided by beach and stone filled gabions in the sides of the basin. The scour protection was constructed in accordance with the design drawings produced using the empirical formulae above and using materials from the stock of the laboratory facility. All tests were undertaken with a horizontal seabed at the location of the models. Tolerances for the water depths in the model tests were ±2 mm. A diagram and photograph of a typical model layout are given below.

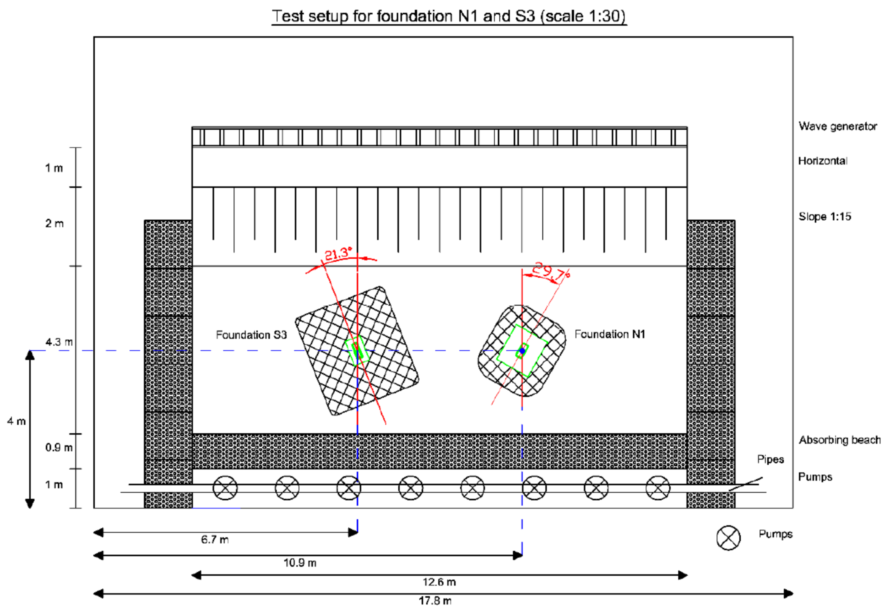

Figure 5: Typical Basin Layout

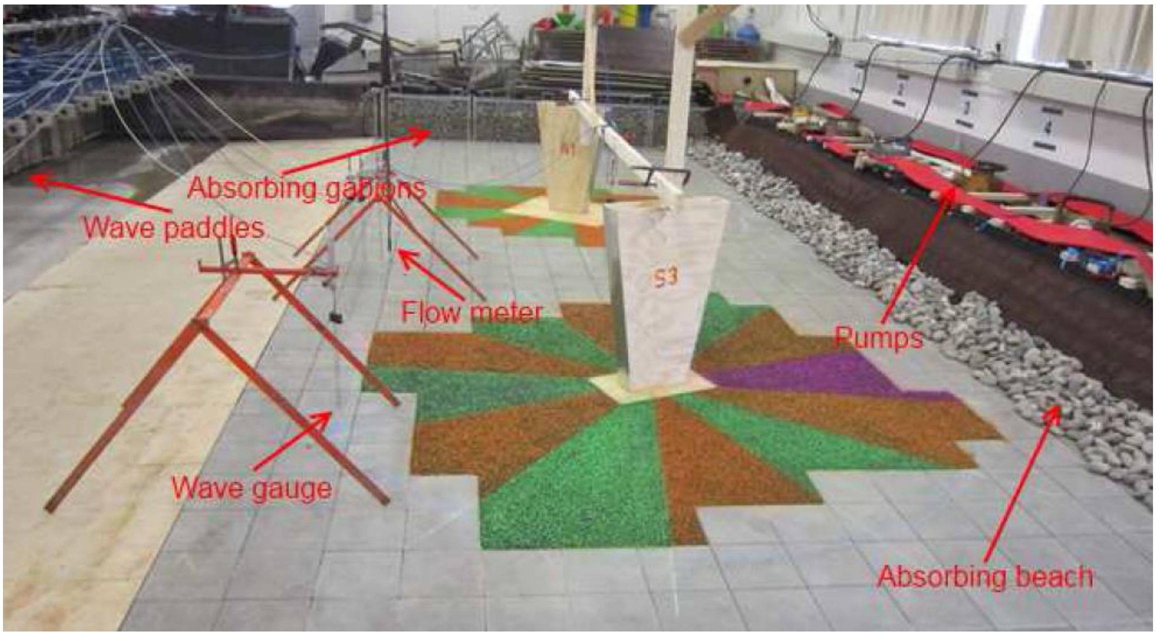

Figure 6: Photograph of typical layout used in the physical model

7.2 *Wave and current conditions – analysis and calibration*

Wave and current conditions were assumed co-aligned and the test-duration was 3 hours for each test (prototype time). The waves were unidirectional and given by a JONSWAP spectra (γ=3.3). 2-D irregular waves were generated by piston type wave paddles using the AWASYS system (http://www.hydrosoft.civil.aau.dk/AwaSys) with the so-called Filtered White Noise generation method. Sea states were calibrated without current and repeated in the tests with current.

For calibration of waves the surface elevations were measured using resistance type wave gauges. The data acquisition was performed using a National Instruments USB-6225 acquisition box and the WaveLab software package. The Wavelab software package was further used for analysing the acquired surface elevations.

Wave states and current states were calibrated separately and generation signals were stored and combined afterwards in the model tests. Incident and reflected wave spectra were separated using three wave gauges and the method by Zelt & Skjelbreia (1992). Tolerances for wave heights were kept below 3% and wave reflection in the basin was kept below 24% meaning that maximum 5% of the wave energy was reflected. Tolerances for current speeds were kept below 10%.

7.3 *Observations of scour protection performance*

The scour protection was marked in different colours to determine possible movement. Any damage was detected using photo overlay technique together with an overall stability assessment where the scour protection was carefully inspected after each test. Photos were taken at specific locations before and after each test.

Colour was added to the water and video-recording was used to observe the flow-formation around the foundations. Captions from the videos are given in the figure below. As could be expected, vortices were present close to the foundations and lee-wake vortices were present in the down-stream of the foundations. Generally, the vortices did not seem to extend outside the concrete cap of the foundations.

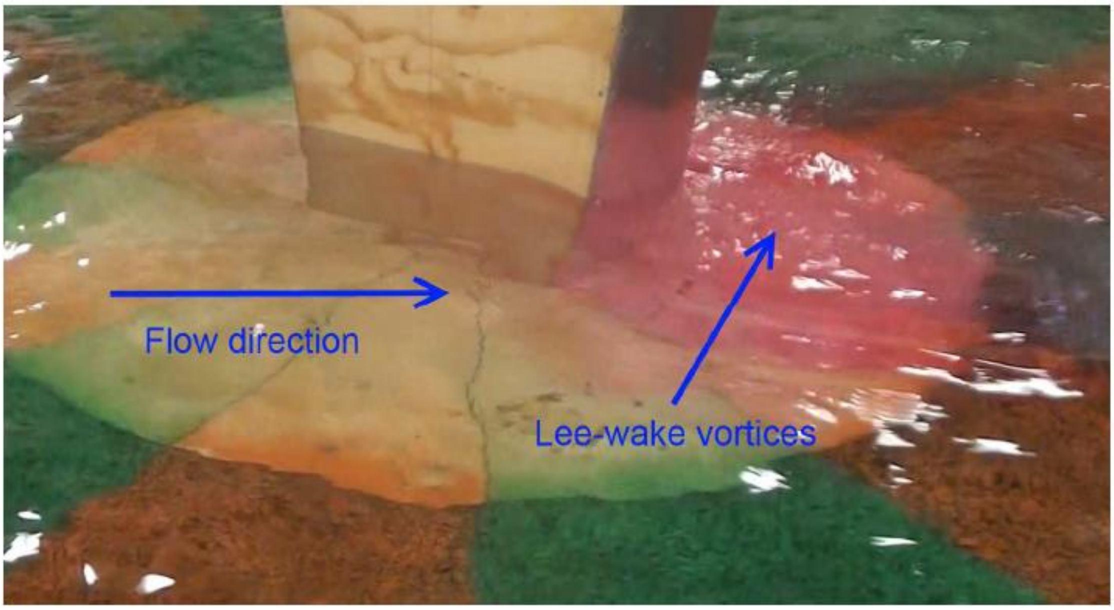

Figure 8: Illustration of lee wake vortices

No damage was observed around any of the considered foundations and with due consideration to model effects and scale effects the design for all foundations is proved to be safe (based on observations from conservative test conditions compared to the requested conditions).

8. Monitoring

Due to structural or environmental concern, in addition to the design and physical model verifications described above, bathymetric surveys will be performed on a regular basis. These will enable monitoring of the installed scour protection and surrounding areas adjacent to the protection and ensure that there are minimal changes. The monitoring includes "trigger" levels of scour/material loss in defined areas which, if exceeded, will require additional measures such as environmental assessment, scour protection, or a combination of scour protection, filter and backfill. The proposals are outlined below.

8.1 *Pre-construction monitoring*

Prior to scour protection installation:

- Bathymetric surveys on at least a biannual basis (6 month intervals) with a comparison of levels made between previous survey and current survey.
- Assessment of scour will be made should significant material loss (to affect foundation stability or environmentally sensitive areas) be witnessed.
- Areas of infill or minor loss will not be assessed or commented on further.

8.2 *Post-construction monitoring*

After scour protection installation:

- Bathymetric survey immediately after placement of protection areas.
- Bathymetric surveys on at least a biannual basis (6 month intervals) with a level comparison between previous survey and current survey continuing through defects notification.
- Continued monitoring of environmentally sensitive areas and the scour protection levels at lower

frequency (every two years) through the service life of the bridge.

- Special Inspections following significant storm/flood events.

9. Conclusion

Ramboll's Marine and Energy Infrastructure team has led the design of scour protection for the main foundations of the Queensferry Crossing. Analysis of scour potential has been conducted on all the foundations and, for those which require scour protection, a scour protection design has been prepared. This paper describes the scour prediction and design process, including the use of numerical modelling by the Ramboll Copenhagen Office and physical modelling at Aalborg University. The design will be combined with regular on site monitoring to provide a solution to protect the foundations of this very important national infrastructure.

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