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New Farm RiverWalk: Assessment of Flood Forces

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Abstract: *The most high profile and significant single loss of transport infrastructure during the January 2011 Brisbane floods was arguably the New Farm Floating Walkway. During the January 2011 flood events in Brisbane the floating Walkway suffered extensive damage, ultimately causing the downstream section of the structure to be washed away. The upstream section of the original walkway was subsequently removed as the piles suffered damage during the flood event. A replacement structure, the New Farm RiverWalk (NFRW) was consequently designed and commissioned and its construction is currently underway. The NFRW is unique in that it is primarily parallel to the Brisbane River. There is little guidance on the appropriate drag and lift forces needed to design such a structure. To more accurately determine these forces, both computational and physical modelling tests were carried out within the design stage of the replacement structure. The combined results of physical modelling using a 1 in 38 scale model, and a computational steady state quasi 3D model enabled flood forces to be calculated with a much greater degree of certainty.*

Keywords: *flood forces, drag, lift, physical modelling, bridge structure, walkway.*

1. INTRODUCTION

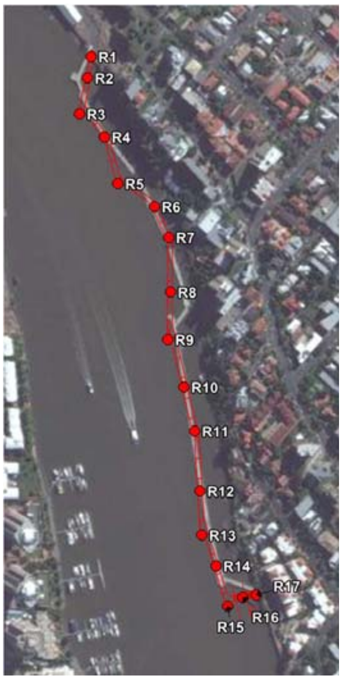
In January 2011, the City of Brisbane, Australia experienced its second-highest flood of the last 100 years, after the January 1974 flood. During this event, major flooding occurred through most of the Brisbane River catchment, severely impacting the Lockyer and Bremer River catchments where numerous flood height records were set. The flooding caused substantial loss of life in the Lockyer Valley and thousands of properties were inundated in metropolitan Brisbane, Ipswich and elsewhere.

During the January 2011 flood event, the existing floating New Farm Floating Walkway suffered extensive damage and the downstream section of the structure was ultimately washed away. The upstream section of the original walkway was subsequently removed as the piles also suffered damage during the flood event.

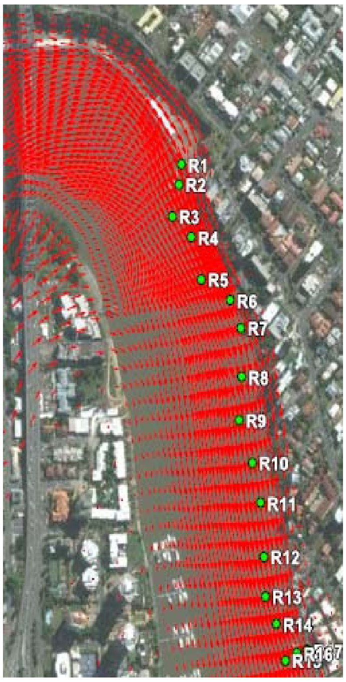
The Floating walkway was one of Brisbane City Council's (BCC) key river connections between New Farm and the City Centre, located in the Brisbane River, between the Howard Smith Wharf site and Merthyr Rd, New Farm. Figure 1 shows a locality plan of the original and proposed NFRW alignment.

The floating walkway, originally constructed in 2003, was used daily by over 3000 cyclists and pedestrians for recreational and commuting purposes. Due to the high demand for such a transport link, BCC investigated a number of replacement options. The selection process considered all stakeholders and adhered to numerous environmental and community requirements.

The project objectives were to provide a river front pedestrian and cyclist connection, designed to withstand an appropriate flood event, minimise the likely structural damage to the facility due to flood events, minimise "whole of life" costs, minimise community impacts, complete construction in 2014 and deliver the entire project within a budget of $A70 million.



a) Original and Proposed Alignment



b) Velocity Vectors (WBM, 2012)

Figure 1 – Original and Proposed NFRW Alignment (In white and red respectively)

1.1. Replacement Structure

A revised alignment that closely follows the original floating alignment was ultimately chosen as the preferred and most viable option. Zigzags were included to provide views of the river, rest and cyclist slow down areas and improved pedestrian safety. The detailed alignment was designed to use standard precast concrete units which could easily be adjusted to create the zigzag alignment, cycleway curves and rest areas.

The alignment of the NFRW is constrained by the main river navigation channel, existing riverbank access paths and by the private mooring gangways of some residents who had mooring access from the previous floating walkway (refer Figure 1a).

The selected structure is an integral post-tensioned precast box girder, 1.2m in depth, constructed using a span-by-span method. The structure is continuous over multiple spans with maximum lengths of up to 250m. The deck level is set at +3.4m RL above Australian Height Datum (AHD).

2. FLOOD FORCE CONSIDERATIONS

The NFRW replacement structure is essentially a 'bridge' designed to withstand flood events up to and including the 0.05% Annual Exceedance Probability (AEP). Water levels of up to +9m AHD were predicted for the 0.05% AEP flood event, which would completely inundate the structure to a depth of 6m. Maximum flow velocities of up to 7.5m/s were predicted in the vicinity of the structure for the 0.05% AEP flood (WBM, 2012).

The Brisbane River at the NFRW location flows in a south-easterly direction and mostly along the river walk deck rather than transversally. As the NFRW is located just downstream of the 180° Kangaroo Point meander of the Brisbane River, the structure will experience flood flows with angles of attack ranging from 90º (perpendicular to NFRW) through to 0º (parallel to NFRW) and may also be subjected to complex 3D flow patterns resulting in flood flows impacting the RiverWalk deck with angles of attack of up to ±5º relative to the horizontal.

2.1. Existing Guidelines

A characteristic of NFRW is that it follows the riverbank, where available design guidance mainly covers perpendicular, or near perpendicular flow over a deck, associated with typical bridge crossings.

Design flood forces for bridge decks are specified in AS5100.2 (2004) which is based on the work of Jempson (2000). AS5100.2 provides design coefficients for standard bridge deck types (I and T girder), for flows typically perpendicular to the deck. Allowances are made in the coefficients for angled flow directions also. Jempson (2000) does contain some design coefficients (drag, lift and moment) for box girder bridge decks similar to that proposed for the NFRW, however his results do not cover the range of angles of attack required for the proposed NFRW. Consequently, drag, lift and moment design coefficients were required to calculate flood loads for the entire proposed NFRW structure, which is skewed at different angles of attack relative to the stream.

Large scale physical modelling alongside steady state quasi-3D hydrodynamic modelling was conducted to determine flow velocities, levels and force and moment coefficients required to calculate flood forces with a higher degree of certainty.

2.2. Hydraulic Modelling

Following the January 2011 flood, Brisbane City Council (BCC) built a steady state quasi 3D hydraulic model of the Brisbane River between Kangaroo Point and New Farm using TUFLOW-FV (BCC, 2012). This model was subsequently refined by WBM (WBM, 2012). Only the two more relevant flood scenarios for structural design considerations were analysed: 10,000m^3/s (overtopping condition) and 18,000m^3/s (assumed to represent 0.05% AEP flood).

The 10,000m^3/s scenario, concurrent with January 2011 flood levels, predicts water levels of 4.1 to 4.6m AHD, these levels are close to the point of submergence for the proposed deck (3.5m AHD) and was utilized to represent the structure overtopping condition. This scenario is believed to correspond to the 1% AEP event. The 18,000m^3/s scenario was assumed to represent the 0.05% AEP event. This event showed water levels ranging between 7.7m AHD and 9m AHD due to the river hydraulic gradient.

Figure 1a shows the locations along the proposed NFRW alignment where flow characteristics predicted by the hydraulic model of the Brisbane River are reported (WBM, 2012). It should be noted that these locations largely correspond to the location of proposed NFRW piers.

2.2.1. Flow Velocities

Results showed that the direction of flow within the vicinity of the NFRW is predominantly parallel to the axis of the deck, however a secondary transversal recirculation was observed across the river. It is believed that this recirculation is caused by the 180° river bend located immediately upstream of the NFRW (BCC, 2012).

Water velocities reported for the 10,000m^3/s and 18,000m^3/s event are listed in Table 1. The maximum flood flow velocities predicted vary along the length of the structure but are typically 6 to 7m/s for the 10,000m^3/s and 18,000m^3/s events. These velocities are approximately double what had previously been considered for the original floating walkway design. They increase the design flood force of the proposed replacement structure fourfold. Modelling results show that maximum velocities occur at the water surface, which for the 10,000m^3/s event approximately correspond to deck level. Note that maximum point flow velocities were interpolated to obtain velocities over individual spans.

Modelling results for the 18,000m^3/s event (0.05% AEP) flood indicated a transversal angle of attack (β) of $\leq \pm 5°$ (refer Figure 2). Similarly, flow vectors for the 10,000m^3/s event (point of submergence) showed an angle of attack (β) of $\leq \pm 2°$. These transversal flows are believed to be caused by turbulent flows across the main river channel (BCC, 2012). From several cross sections reported in BCC (2012), it is apparent that the angle of attack changes and that the direction of the transverse flow reverses along the length of the proposed NFRW.

For the purposes of structural design for the NFRW it was assumed that, the angle of attack for flow transverse to the river channel is ±5° and the angle of attack in the vicinity of the structure is ±2°.

Table 1 Predicted Flow Velocities at NFRW (WBM, 2012)

ID	Maximum V	Average V	Max at structure RL	Max vertical V	Flow direction °N	Bed level mAHD	Surface level mAHD	Angle of attack °	
R1	5.6	5.3	n/a	0.01	147	-0.90	3.6	0.1	
R2	4.4	4.0	n/a	0.02	157	-1.75	3.6	0.3	
R3	4.7	3.9	n/a	0.28	153	-21.90	3.5	3.4	
R4	6.1	5.3	n/a	0.11	153	-12.80	3.5	1.0	
R5	5.3	4.5	n/a	0.09	153	-16.20	3.5	1.0	
R6	3.4	3.2	n/a	0.03	153	-6.30	3.6	0.5	
R7	2.7	2.5	n/a	0.03	159	-4.80	3.5	0.6	
R8	4.5	4.1	n/a	0.02	167	-11.85	3.6	0.3	
R9	4.7	4.2	n/a	0.01	171	-11.75	3.6	0.1	
R10	4.7	4.3	n/a	0.02	172	-10.95	3.6	0.2	
R11	4.6	3.6	n/a	0.04	172	-10.30	3.5	0.5	
R12	4.7	4.4	n/a	0.02	171	-10.40	3.6	0.2	
R13	4.9	4.3	n/a	0.1	169	-12.25	3.5	1.2	
R14	4.8	4.4	n/a	0.05	168	-10.55	3.6	0.6	
R15	4.6	3.8	n/a	0.14	164	-9.35	3.5	1.7	
R16	3.8	3.3	n/a	0.09	163	-7.60	3.6	1.4	
R17	1.7	1.6	n/a	0.02	163	-2.70	3.5	0.7	
max	6.1	5.3			172		3.6	3.4	
min	1.7	1.6			147		3.5	0.1	
average	4.4	3.9			162		3.6	0.8	
std dev	1.0	0.9			8		0.1	0.8	
					angle of attack at 95% confidence			2.2	degrees

a) 10,000 m^3/s

ID	Maximum V	Average V	Max at structure RL	Max vertical V	Flow direction °N	Bed level mAHD	Surface level mAHD	Angle of attack °	
R1	6.4	6.0	6.3	0.04	157	-0.90	9.0	0.4	
R2	5.5	5.4	5.5	0.03	162	-1.75	8.6	0.3	
R3	5.5	5.1	4.5	0.28	154	-21.90	8.3	2.9	
R4	7.0	6.4	6.9	0.12	153	-12.80	8.3	1.0	
R5	6.3	5.8	5.9	0.12	154	-16.20	8.2	1.1	
R6	5.4	4.8	5.3	0.13	154	-6.30	8.3	1.4	
R7	4.3	4.1	4.2	0.04	158	-4.80	8.2	0.5	
R8	5.7	5.2	5.6	0.04	167	-11.85	8.2	0.4	
R9	5.9	5.4	5.8	0.02	171	-11.75	8.0	0.2	
R10	6.1	5.6	6.0	0.05	172	-10.95	7.9	0.5	
R11	6.0	5.1	5.9	0.06	172	-10.30	7.9	0.6	
R12	6.1	5.7	6.1	0.04	171	-10.40	7.9	0.4	
R13	6.4	5.6	6.3	0.23	169	-12.25	7.9	2.1	
R14	6.4	5.8	6.3	0.02	168	-10.55	7.9	0.2	
R15	6.1	5.0	6.0	0.14	165	-9.35	7.7	1.3	
R16	5.5	4.6	5.4	0.11	163	-7.60	7.7	1.1	
R17	3.7	3.2	3.5	0.03	163	-2.70	7.7	0.5	
max	7.0	6.4			172		9.0	2.9	
min	3.7	3.2			153		7.7	0.2	
average	5.8	5.2			163		8.1	0.9	
std dev	0.8	0.8			7		0.3	0.7	
					angle of attack at 95% confidence			2.1	degrees

b) 18,000 m^3/s

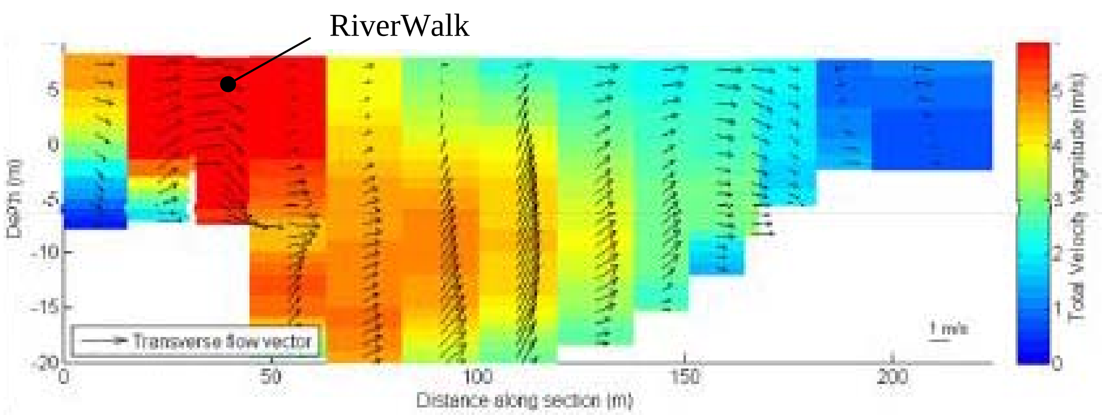


Figure 2 – Brisbane River Transversal Velocity Vectors at NFRW Location, 18,000 m^3/s (Arup, 2013)

2.3. Physical Modelling

The physical modelling was carried at the University of Auckland in a flume with a 1/38 scale of the deck and with flow velocities and directions representative of those along the proposed deck. Only the 10,000m^3/s and 18,000m^3/s flood events were modelled. As a consequence of the lack of definitive design guidance regarding drag, lift and moment coefficients, physical modelling was conducted to determine the input factors required to determine design flood forces. The conducted tests are listed in Table 2. Both the 10,000m^3/s and 18,000m^3/s flood events were modelled in a large flume with a 1/38 scale model of the deck based upon a Froude similitude. Figure 3 shows the NFRW purpose built physical model.

The flow velocities, water depth, submergence, skew angles and attack angles were chosen as being representative of locations and conditions along the NFRW structure. Some additional test cases at skews of 0º and 90º were selected to replicate existing data. Flow velocities were measured using a pitot tube, flow depths were measured directly through the glass walls of the flume, while forces acting on the model were measured using 3D load cells sensitive to 0.5N. Melville et al. (2012) and Arup (2013) document the experimental modelling tests and results in detail. Two distinct models were used in the flume, one with a solid balustrade (as shown in Figure 3 below) and a second one without a balustrade to replicate a porous balustrade with maximum aperture.

Design case	Water depth	Flow velocity [m/s]		Skew angle in plan [°]	Angle of rotation to horizontal [°]
		model	prototype		
Q2000 Submerged Structure below water surface		0.91	5.6	0	0
				5	0
				15	0
					-5
					+5
				30	0
					-5
					+5
				40	0
				90	0
				15	+2
					-2
				30	+2
					-2
		1.12	6.9	40	0
Q100 Overtopping	Structure at water surface level	0.71	4.4	0	0
				5	0
				15	0
				30	0
				40	0
				90	0

Table 2 Test Programme (Melville et al., 2012)

Figure 3 - NFRW Physical Model

2.3.1. Experimental Results and Observations

Figure 4 compares experimental coefficients obtained for 18,000m^3/s event with normalized values based on coefficients reported in A5100.2 (2004). In general, obtained experimental drag, lift and moment coefficients for flows perpendicular to the structure were similar to those reported. However, significant differences in lift forces were observed for experimental oblique flows (Arup, 2013).

At the structure, the 10,000m^3/s event flows created significant down lift whilst the 18,000m^3/s flow produced uplift due to aerofoil effects in the water flow (refer to Figure 4b). BCC's flood study (BCC, 2012) also showed a small vertical angle of attack (≈2°) between the direction of the water flow and the structure, which further increases the lift coefficient values by approximately 50%.

Experimental results also showed that the planar angle of flow attack had a negligible effect on the drag, lift, moment and skin friction coefficients. A further observation from the physical modelling was that the effect of the parapet on drag was more severe than expected and appeared to be due to wave drag during overtopping.

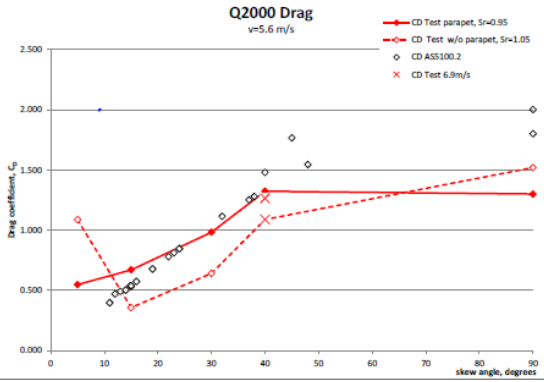
2.4. Design Considerations to Counteract Water Forces

On account of the results returned from the physical modelling, significant lift forces had to be designed for. As uplift from the 18,000 m^3/s flow can create sagging at pier locations, reinforcement was added at every pier/deck soffit junction to resist it. The effect of buoyancy on the voided box girder during a flood event was minimised by the inclusion of discreet air and drainage outlets in the soffit and webs of the girder. The uplift forces from the 18,000 m^3/s flow are a reverse of the 10,000m^3/s flow event where the forces are as a result of negative (down) lift.

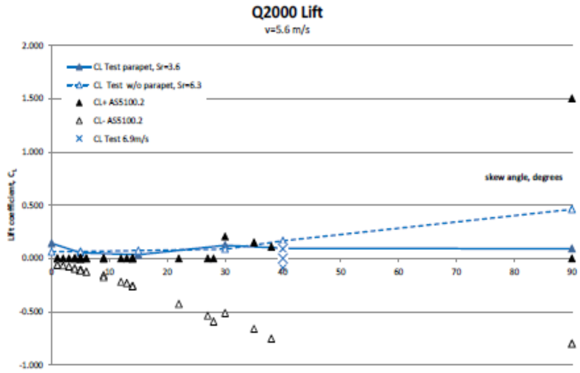
Due to the large, variable lift forces and overturning moments at each span of the structure, the use of bearings was found to be impractical to counteract the flood loads and various degrees of reinforcement were applied from unit to unit along the proposed structure.

With the orientation of the deck being close to the direction of the water flow it is unlikely that small debris would be trapped by the structure and balustrade. Nonetheless, to limit the debris forces that can be applied to the structure, the base fixing bolts of the proposed balustrade is designed to collapse through a controlled failure system to limit the load that debris laden flood flows can apply to the structure. Also, a balustrade with maximum aperture was chosen to minimise drag forces.

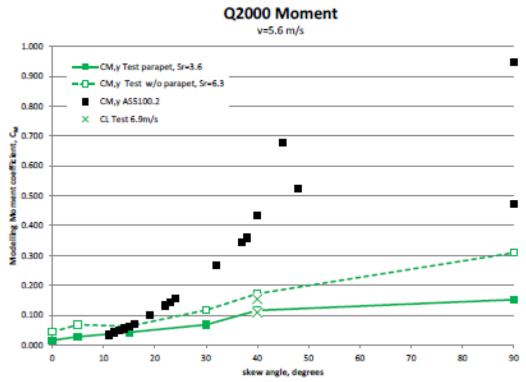
Large-scale debris (log, container, pontoon, etc.) and ship impact was also considered in the design, as per design criteria requirements, with a maximum pier design force of 500kN (equivalent to a 40 Tonnes vessel/pontoon hitting the piles at a velocity of 3 m/s and a maximum deck design force of 1300kN (equivalent to a 200 Tonnes vessel/pontoon hitting the deck at a velocity of 4m/s) to consider accidental vessel impact.



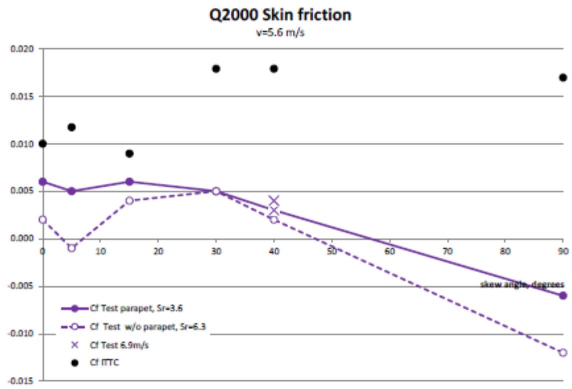
a) Drag Coefficient



b) Lift Coefficient



c) Moment Coefficient



d) Skin Friction Coefficient

Figure 4 – NFRW Experimental versus Reported Coefficients, 18,000 m^3/s

2.5. Dynamic Effects

The dynamic loads that might act on a structure similar to the proposed NFRW are mainly small scale unsteady velocities due to turbulence and vortex shedding with approximate periods of 0.1 to 1 s (BS 6349-1, 2000). The piers and opening span are the only structural elements with periods on the range of small scale unsteady velocities and vortex shedding (0.1 to 1s). The natural period of the main structure is outside the range of response (≈3s).

The magnitude of unsteady velocities caused by turbulence cannot be determined from the steady state hydraulic modelling conducted and was not measured during the conducted physical modelling.

It is believed that turbulence can cause short term fluctuations in the water velocity of approximately ±10% (WBM, 2012). Also, mean velocity fluctuations during conducted physical modelling were estimated to be between ±5% and ±20%.

To date, there are limited field observations of the effect of turbulence velocities on pontoons, bridges and other riverine structures. Brown and Chanson (2013), conducted velocity measurements during the January 2011 flood event in an inundated street adjacent to the Brisbane River left bank, located some 4 km upstream of the NFRW location (Gardens Point Road). They sampled turbulent velocity data at a relatively high frequency (50 Hz) using an acoustic Doppler velocimeter (ADV). Flow velocities recorded during their field measurements ranged between 1 and 2.5m/s.

Their analysis of the measured ADV signals showed large fluctuations around the mean values but highlighted that the main contributor was the slow fluctuation turbulent intensity. Their measured high frequency (small scale) turbulence properties were close to values measured in the laboratory.

Their results suggest that turbulence intensity during the January 2011 at the measured location was mainly comprised of slow fluctuation turbulence and in a lesser way by the velocity fluctuations induced due to the small scale turbulent processes of concern for the NFRW. Although, it is believed that these results cannot be directly extrapolated to locations with completely differing flow and geometrical conditions (such as the NFRW location where velocities of about 6 m/s flowing parallel to the river walk deck will be experienced), they provide confidence that the velocity fluctuations considered to calculate dynamic loads for the NFRW (± 10%) are adequate.

3. SCOUR AT PIERS

The soil comprising the top layer of the Brisbane River bed around the proposed NFRW is generally a low plasticity very soft clay (VS) or silt, alluvial in nature. The most recent alluvial deposits include predominantly very soft silts and clays as well as loose sand. The material will most likely have been disturbed in several locations and has a thickness that ranges from 1m to 10m (Arup, 2012). An older, denser alluvial layer exists below the top layer and is considered to have been in situ for thousands of years since the sea level at Moreton Bay reached its current level (approximately 5000 years ago).

The bathymetric survey undertaken following the January 2011 flood event indicated that accretion had occurred at this site. This did not however identify the scour that occurred during the flood. Therefore, an assessment into the stability of the driven piles of the old floating walkway indicated that relatively low scour depths (0.5m to 2m) may have occurred during the flood as greater scour depths would not have enabled any piles to remain during the January 2011 flood event.

Due to the cohesive nature of the material forming the top layer of the Brisbane River bed the local scour at the NFRW piers was calculated using two different methodologies developed for non-cohesive soils (Melville and Coleman, 2000) and cohesive soils (Briaud et al., 2011).

Pier scour in cohesive materials generally progresses more slowly and is more dependent on soil properties than in non-cohesive materials. The SRICOS-EFA methodology provides the maximum potential scour for a given hydraulic condition and also accounts for time dependency, as the maximum potential scour may not be reached during one single flood event or even over the life of the structure, due to the slower progression of scour in cohesive soils. An erosion rate for low plasticity clays under a depth averaged flow velocity of 6m/s and maximum shear stress of about 150Pa was utilised in this study.

Based on present results for cohesive soils, local scour depths for the 18,000m^3/s event (with 48 hours duration) were calculated to be about 3m while scour depths for the 10,000m^3/s event were calculated to be slightly lower. Results for the scenario representing two consecutive 10,000m^3/s events (with a total duration of 96 hours) showed maximum scour depths ranging from 2.9 to 3.2m. Consequently, scour depth for the 10,000m^3/s event was assumed to be 3m. Local scour calculations for non-cohesive soils resulted in scour depths similar in range with maximum scour depths of about 3.6m for 1.5m diameter piers.

4. CONCLUSION

The location and history of the proposed NFRW created challenges with respect to the achievement of key objectives: aesthetics, durability, constructability and flood resilience. Due to the lack of current design standards for a flood resilient longitudinal walkway structure mostly parallel to the river, physical modelling and quasi-3D hydrodynamic modelling were conducted to determine flood forces and moment coefficients needed to structurally design the proposed NFRW for the two more relevant flood scenarios for structural design purposes: 10,000m^3/s (overtopping condition) and 18,000m^3/s (assumed to represent 0.05% AEP flood).

Drag, lift and moment coefficients based on obtained results for flows perpendicular to the structure were similar to those reported in the current relevant guidelines (A5100.2, 2004). However, significant lift forces were observed for experimental oblique flows. At the structure, the 10,000m^3/s event created significant down lift whilst the 18,000m^3/s event caused uplift due to aerofoil effects in the water flow.

Due to the large, variable lift forces and overturning moments at each span of the structure, the use of bearings was found to be impractical to counteract the flood loads and various degrees of reinforcement were applied from unit to unit along the proposed structure. The base of the proposed balustrade is designed to collapse through a controlled failure system to limit the load that debris laden flood flows can apply to the structure. Also, a balustrade with maximum aperture was chosen to minimise drag forces.

Local scour calculations calculated with both methodologies resulted in similar maximum scour depths of 3m to 3.6m. Scour predictions informed structural design of this complex walkway designed to withstand flood flow with various angles of attack and that will interact with complex 3D flow patterns occurring in the vicinity of the Brisbane River bank. The ultimate result from the conducted analyses, is that a new, more durable and flood resilient NFRW replacement has been successfully designed and commissioned with construction now well underway.

5. ACKNOWLEDGEMENTS

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