

Report on

# Rehabilitation of Bridge over Hawkesbury River at Windsor

(BN 415 on MR 182)



◆ *Knowledge*

◆ *Experience*

◆ *Innovation*



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## 1. Introduction

The bridge over Hawkesbury River at Windsor (Bridge Number 415 on MR 182) is an 11 span concrete bridge with overall length 143m. The bridge carries two lane traffic and the width between kerbs is 6.1m. The original bridge with timber deck on cast iron piers was constructed in 1875. The bridge was raised by 2.4m, by extending the cast iron piers in 1897. Pier 10 in timber had been added or rebuilt at the same time. The timber deck, abutments and Pier 10 have been replaced in reinforced concrete in 1920. A footpath was added on the downstream side in 1968.

The bridge had been inspected by RTA Bridge Evaluation & Assessment Group, and also by GHD Pty Ltd. CTI Consultants Pty Ltd has carried out under water investigation of the cast iron piers.

### 1.1. Scope of Work

As requested by the Bridge Maintenance Planner Sydney, Sydney Asset Section, the following have been carried out by the Bridge Rehabilitation Projects, Bridge Section.

- Conceptual designs for the strengthening of superstructure according to the long term repair strategy as discussed in item 1.3.below.
- Cost estimation for the above.
- Capacity assessment of piers and strengthening options.

### 1.2. Details of Structure

The deck is supported on 8 reinforced concrete beams that are simply supported on the headstock and dowelled at either end (Annexure 1). The deck has been constructed in two halves with a longitudinal construction joint at the centre. The footpath is independent of the deck slab, and is supported on steel stringers.

The headstock is supported on twin cylinder cast iron piers that are braced with steel bracings at two levels. The upper level bracings are encased in reinforced concrete to form a diaphragm.

### 1.3. Present Condition of Superstructure

There are cracks in the beams with concrete spalling and corrosion of steel on the soffit of the slab, on the beams and headstocks.

RTA Evaluation & Assessment Group has assessed the "as is" condition of the bridge for Semi Trailer (42.5t) and B Double loads (Report dated Dec 2003). The load factor for beams is 1.5 (75% capacity) and rating is ST 35 or BD 51. The load factor for headstocks is 1.5 (75% capacity) and the rating is ST35 or BD 50. They have recommended that the existing bridge needs to be replaced by a new bridge.

GHD has reported extensive carbonation of concrete and corrosion of steel (Report dated Oct 2003). In the report, two repair strategies are recommended.

1. Short term repair (5 years) – Conventional patch repair and replacement with a new bridge.
2. Long term repair (25 years) – Realkalisation of beams to arrest further corrosion, patch up, and strengthening of elements if required- See 2 below.

#### 1.4. Present Condition of Substructure

CTI underwater investigation has revealed graphitisation of the cast iron piers (Report dated April 2005). This has taken place to a higher degree in the lower parts of the piers that are submerged. According to the report, the depth of layered graphitisation is 13mm, reducing the 38-40mm original thickness to 25mm. The thickness is further reduced to 12mm due to localised plug graphitisation. The extent of this local graphitisation is about 1 plug per 1 square ft area.

The infill material in the lower parts of the cylinders was found to be debris consisting of brick and gravel, while the upper portion of the cylinders were in filled with concrete.

## 2. Strengthening of Superstructure (Long Term Repair Strategy)

“As good” condition of the superstructure was checked to formulate the strengthening options to model the behaviour after being restored to the original condition.

( Assessment on “as is” condition is as described in item 1.3.)

### 2.1. Details of Analysis

The maximum bending moments, shear forces and support reactions were enveloped by analysing a grillage model in Microstran for dead and live load combinations. The traffic loads considered were 2 lane traffic with a lane reduction factor of 0.9, or single lane traffic with lane factor of 1.0, occupying different lateral positions on the carriageway to produce maximum effects on beams.

The following design parameters were adopted in the analysis.

**Material Properties:** Yield strength of steel = 230 MPa  
Compressive strength of concrete = 35 MPa

**Traffic Loads:** Beams & Headstock: T44 with DLA 40% and  
L44 including Concentrated Load, with DLA 40%  
Slab: W7  
{Design loads as per Australian Bridge Design Code- (ABDC) 1996}

### 2.2. Summary of Outcomes:

From the results it is seen that even if the superstructure elements are restored to their original condition, the main beams have a load factor less than 2.0 in flexure under the T44 and L44 loads.

The live load factors are:

- Edge beams - 1.74
- Intermediate beams - 1.31
- Central beams - 1.21

Therefore they do not have adequate capacity to carry the design loads as per ABDC 1996, and need to be further strengthened.

Further, if the slab and the headstock are restored to the original condition, they have a load factor greater than 2.0 and therefore they have adequate strength to carry the design loads.

The full results of the analysis are shown in Annexure 2.

### 3. Superstructure Strengthening Options

The following strengthening options were considered for upgrading the capacity of the beams under the long term repair strategy.

- Option 1: Installation of Steel Girders.
- Option 2: External Post Tensioning of Existing Beams.
- Option 3: Fibre Reinforced Polymer (FRP) Strengthening.

#### 3.1. Strengthening Option 1: Installation of Steel Girders

##### 3.1.2. Overview:

- Steel girders (700 WB 173) are positioned centrally between the existing concrete beams to share loads with the existing beams. This involves 6 steel beams per span for all 11 spans. (Annexure 3)

##### 3.1.3. Constraints:

- Beams need to be lifted into position from down below the deck. It is recommended that the beams are transported underneath the bridge by a barge, and lifted to position. Temporary scaffolds may have to be erected in this process. It may also be necessary to drill holes through the deck for lifting, which need to be patched up later.
- The beams have to be in lengths that are shorter than the centre to centre distances between piers, in order to facilitate installation. These individual lengths have to be jointed at site.
- The steel beam splice joint layout is as shown in Annexure 4. These comprise of 2 joint types, which are full moment connections to maintain the continuity of the beams, and sliding joints to allow for thermal movements.
- In order to transfer the slab loads to the new steel beams, they need to be jacked to a predetermined force, which can be done by flat jacks on top of the new steel beams. After jacking, the space in between the soffit of the slab and the top of the beam has to be grouted.
- The available working space is limited. The clear distance between the existing concrete beams is 711mm. With the designed 700 WB173, the available clearance between the existing beams and the new beams at the flange level becomes 218mm. Therefore it may be difficult to reach some parts of the splice joints from the underside of the bridge. It may be necessary to cut openings in the deck slab, to facilitate tightening of bolts and also carry out activities such as grouting and jacking. In this case, traffic movements may have to be restricted to one lane till the deck is patched up.

#### 3.2. Strengthening Option 2: External Post Tensioning of Beams

##### 3.2.1. Overview:

- Existing concrete beams are to be strengthened by prestressing through external post tensioning.
- The designed tendon profile was rectilinear, rather than draped, to facilitate construction and jacking.

- Post tensioning tendons can be either CT stress bars or Macalloy bars (which are available in shorter lengths and be coupled to make the required length) or high tensile steel multi strand tendons.(Annexure 5)

### **3.2.2. Constraints:**

- The tendons need to be adequately protected against corrosion and other possible hazards as they are placed externally. High tensile wires are sheathed hence protected from corrosion but stress bars do not have such cover.
- In order to transfer the jacking force safely to the existing beams larger anchorage plates are required.
- The prestress should be applied with care as the steel is already corroded and the concrete is carbonated and deteriorated. This is especially important for the deck slab which may undergo excessive tensile stresses at minimum vertical loads, and at the anchorage points where the localised stresses are high.
- Stressing may have to be carried out in stages on main beams to control the differential movements between adjoining beams, and prevent slab cracking.

### **3.3. Strengthening Option 3: Fibre Reinforced Polymer (FRP) Strengthening.**

#### **3.3.1. Overview and Constraints:**

Strengthening of beams with FRP wrap was considered as one option. However, due to the following disadvantages this application was not recommended as a suitable option.

- A very good adhesion between the concrete surface and the FRP material is required. This needs to be established by a standard pull out test. Considering the extent of carbonation in the concrete, it is presumed unlikely that the existing concrete satisfies the requirement.
- Requires extensive surface preparation such as blast cleaning, and grinding and surface coating to accuracy of 0.5mm in level, which involves pollution control, which will incur high costs.
- Needs very high quality control and high competency in installation.

## **4. Assessment of Cast Iron Piers**

### **4.1. Details of Analysis**

The capacity of the cast iron pier foundations under the present condition (“as is”) was assessed under the dead and live loads, prior to formulating the strengthening options.

Maximum axial loads and bending moments on the piers were calculated by analysing the bridge for vertical and horizontal loads with a Microstran model. At the end supports of the beams, the shear capacity of the dowel connections together with the concrete to concrete frictional resistance were assessed, and found adequate to transfer the longitudinal braking force across to other spans. As such, it is reasonable to assume that the braking load is shared by all spans. Therefore the bridge was modelled as a 2D frame comprising of all spans, piers and abutments, with moment released beams to model simply supported ends.

The soil resistance on cast iron piers was modelled as linear spring supports. In the absence of more accurate data, the soil was assumed to be soft clay, in order to produce more conservative results.

The lengths of piers were measured from the original drawings, since the actual contract levels were not available.

The following design parameters were considered in the analysis.

**Traffic Loads:** Semi Trailer ST 42.5t with DLA 30%  
Longitudinal braking load 10% of L44 load =190 kN per lane (NAASRA 1976)  
(Current legal loads)

T44 with DLA 30%  
Longitudinal braking load = 400 kN per lane  
(Design loads as per ABDC 1996)

**Material properties:** Ultimate tensile strength of cast iron = 170 MPa  
(as recommended in the CTI investigation report)  
Linear spring stiffness of soil 5000 kPa/m

## 4.2 Summary of Outcomes

The critical section for the capacity assessment is the lower part of the cylinders where the greatest section loss has taken place and also where there is no contribution from the infill material.

The critical section is therefore an annular ring with the outer diameter 1016mm (40") and 25 mm thick, after allowing for the general graphitisation. A further reduction of 8 voids, 30mm diameter and 13mm thick per section was made to cater for local graphitisation.

The load factor for the cast iron piers was marginally above 2.0 under the current legal loads. However, the load factor was less than 2.0 under the ABDC loads.

## 4.3. Recommendations for the Piers

### 4.3.1. Monitoring

Although the substructure was found marginally adequate to carry the current legal loads, risk factors such as:

- lack of accurate foundation data in carrying out the assessment of substructure
- possible further section loss due to progressive graphitisation
- lack of warning before failure due to the brittle nature of cast iron

make it essential that the bridge be closely monitored for any foundation movements.

### 4.3.2. Strengthening of the Piers

As stated in 4.2 above, the capacity of the cast iron piers is inadequate for the ABDC design loads. (The design loads considered for the analysis are the T44 vehicle load and the 400 kN/lane braking load as per ABDC 1996.) Therefore, the piers need to be strengthened to upgrade the capacity, if the long term repair strategy is followed.

The strengthening can be done by concrete jacketing, and this will involve erection of coffer dams during construction and pollution control. However, before carrying out any conceptual designs for the substructure, it is necessary to carry out detailed foundation investigation to establish soil data and contract levels of piers.

CTI has recommended installation of impressed current cathodic protection designed to AS 2832.3 to protect further graphitisation, if the capacity is found adequate. However, this alone is considered not applicable due to the present condition of structural inadequacy.

**6.0 Cost Estimates for Long Term Repair Strategy**

**6.1. Cost Estimate for Superstructure Strengthening Option1**

Installation of new steel girders - \$ 1,910,000  
(see Annexure 6)

**6.2. Cost Estimate for Superstructure Strengthening Option 2**

External post tensioning of beams -\$1,500,000  
(see Annexure 7)

**6.3. Cost Estimate for Repairs & Strengthening to Cast Iron Piers**

As discussed in 5.2, due to the lack of more foundation data, it is not possible to carry out conceptual designs for pier strengthening at this stage. Therefore it is not possible to produce a cost estimate.

However, an approximate cost estimate based on similar types of work would be \$3,600,000 which may be used as a guide.

**6.4. Cost Estimate for Re-Alkalisation and Other Repairs  
(As Per GHD Report)**

Deck joints, slab repairs, deck drainage & reseal, repair beam ends	\$ 394,650
Realkalisation	\$1,576,600
Repairs to diaphragm walls	\$ 126,000
Repairs to steel bracings	\$ 33,500
Miscellaneous items	\$ 31,000
<b>Total</b>	<b>\$2,161,750</b>

**6.5. Summary of Cost Estimates for Long Term Repair Strategy**

**6.5. 1.Total Cost Estimate for Strengthening Option 1**

Installation of steel girders	= \$ 1,910,000
Strengthening to cast iron piers	= \$ 3,600,000
Re-alkalisation and other repairs (GHD report)	= \$ 2,161,750
<b>Total Cost</b>	<b>= \$ 7,671,750</b>

**6.5.2. Total Cost Estimate for Strengthening Option 2**

External post tensioning of Beams	= \$ 1,500,000
Strengthening to cast iron piers	= \$ 3,600,000
Re-alkalisation and other repairs (GHD report)	= \$ 2,161,750
<b>Total Cost</b>	<b>= \$ 7,261,750</b>

## **7. Recommendations**

The cost of rehabilitating the bridge under the long term repair strategy is high and considered not cost effective, as the bridge, even after the rehabilitation, would only be capable of taking the design loads as per ABDC 1996 and not the design loads as per the current code AS 5100-2004. Therefore it is recommended that this bridge be replaced with a new bridge within 5 years.

Further, since the bridge is under distress at present, the bridge must be closely monitored for propagation of cracks, further concrete deterioration, adverse movements and excessive deflections of elements, till this is replaced with a new bridge.

It is recommended that the actual stresses in the bridge beams be monitored by strain gauges to establish the actual live load factors under the real traffic loads. In addition the deck levels must be monitored quarterly for any displacements.

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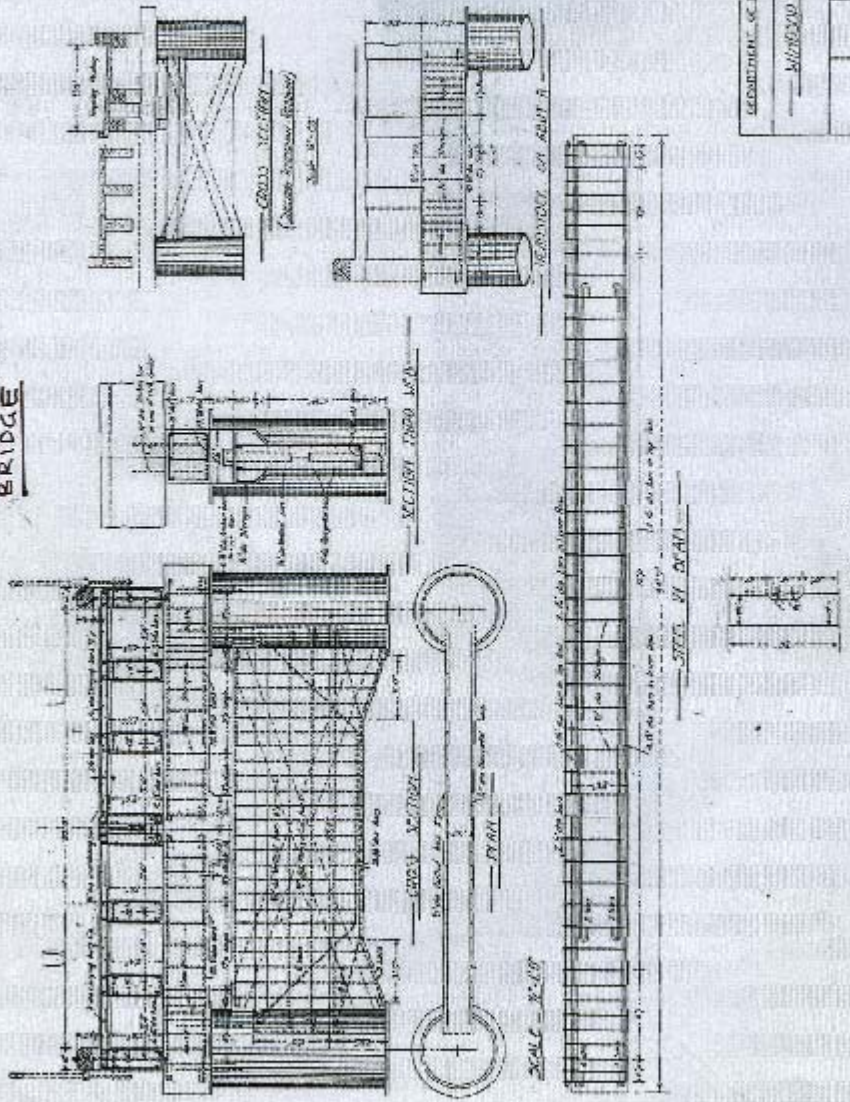
***Craig Gibbons***

***Manager  
Bridge Rehabilitation Projects***

415  
GREEN

MS  
CEN-200-42

# DETAILS OF EXISTING BRIDGE



DEPARTMENT OF HIGHWAYS  
WINDSOR BRIDGE

REDUCTION RATIO  
**21.2x**

10182 492800 104 SHEET 2 OF 193

Annexure 1

**Live Load Factors in Flexure and Shear**

**Project:** Bridge over Hawkesbury River at Windsor (B/N 415)  
**Components:** Superstructure Beams, Slab & Headstock

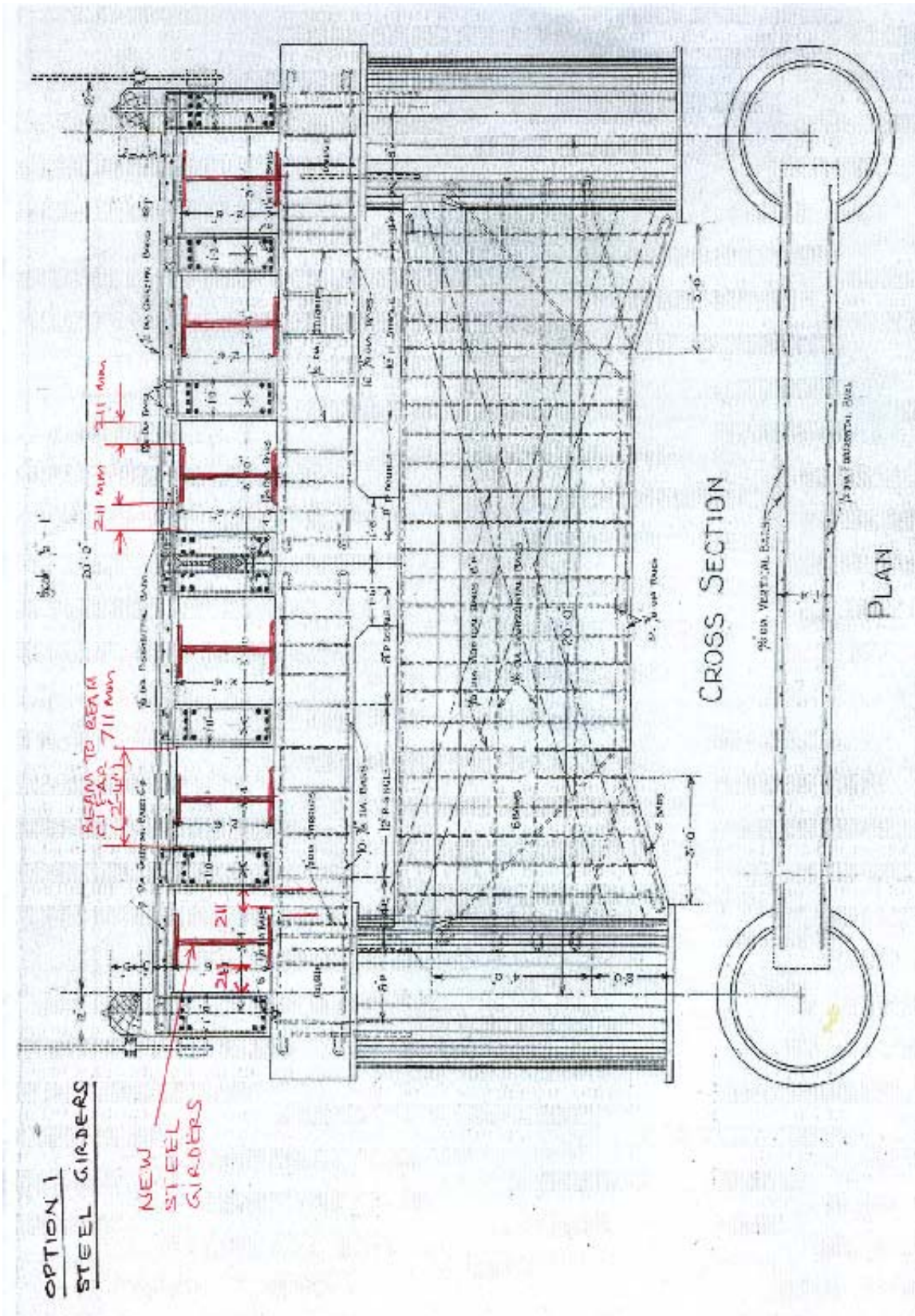
**Design loading:** T44, L44 with DLA 40% (ABDC-1996) for beams, W 7 with NAASRA (1976) distribution for slab  
**Design traffic lanes:** 2 lanes 3.1m wide [available width between kerbs, (20 ft )=6.096m ]  
**Condition of beams:** "As new" condition of beams, after beams are restored to the original condition

Member Details	Shear					Flexure						
	Vi kN	Vd kN	V*=(1.2 Vd+2.0Vi) kN	As Rectangular beam		MI kNm	Md kNm	M*=(1.2Md +2.0MI) kNm	As Rectangular beam		As Flanged beam	
				phi.Vu	LLF				phi.Mu	LLF	phi.Mu	LLF
Edge Beams B1, B8	67.7	78.4	229.48	385	4.30	256	234	792.80	561	1.09	725	1.74
Intermediate Beams B2, B3, B6 & B7	92.3	79.0	270.00	385	3.14	325	257	958.40	561	0.78	734	1.31
Central Beams B4 & B5	93.0	49.1	244.92	323	2.84	221	165	640.00	355	0.71	478	1.27
Headstock	114.0	66.2	307.44	1351	11.15	97.7	55.3	261.76	387.3	3.28	NA	NA
Slab			7.8 v high		>2	11.32	0.3	23.00	25	2.18	NA	NA

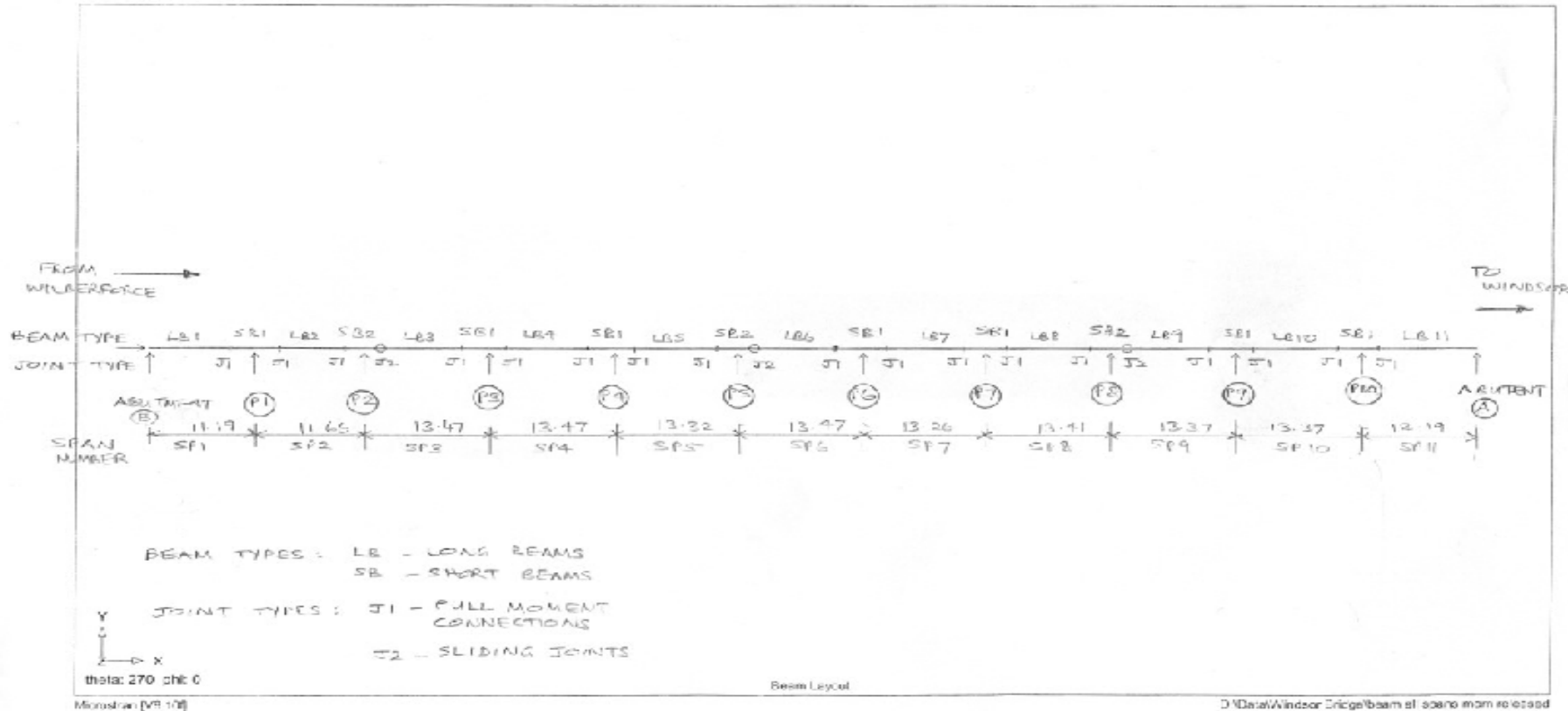
Note:

1. Moment LLF=(phi. Mu-1.2\*Md)/MI
2. Shear LLF=(phi Vu-1.2\*Vd)/Vi
3. Shear links for all super structure beams B1, B2, B3.....B8: Y12 @ 300 c/c
4. Slab main reinforcement: Y12 @100 c/c
5. Yield stress of steel, fy=230 Mpa
6. Comp. Stress of concrete, f'c=35 Mpa

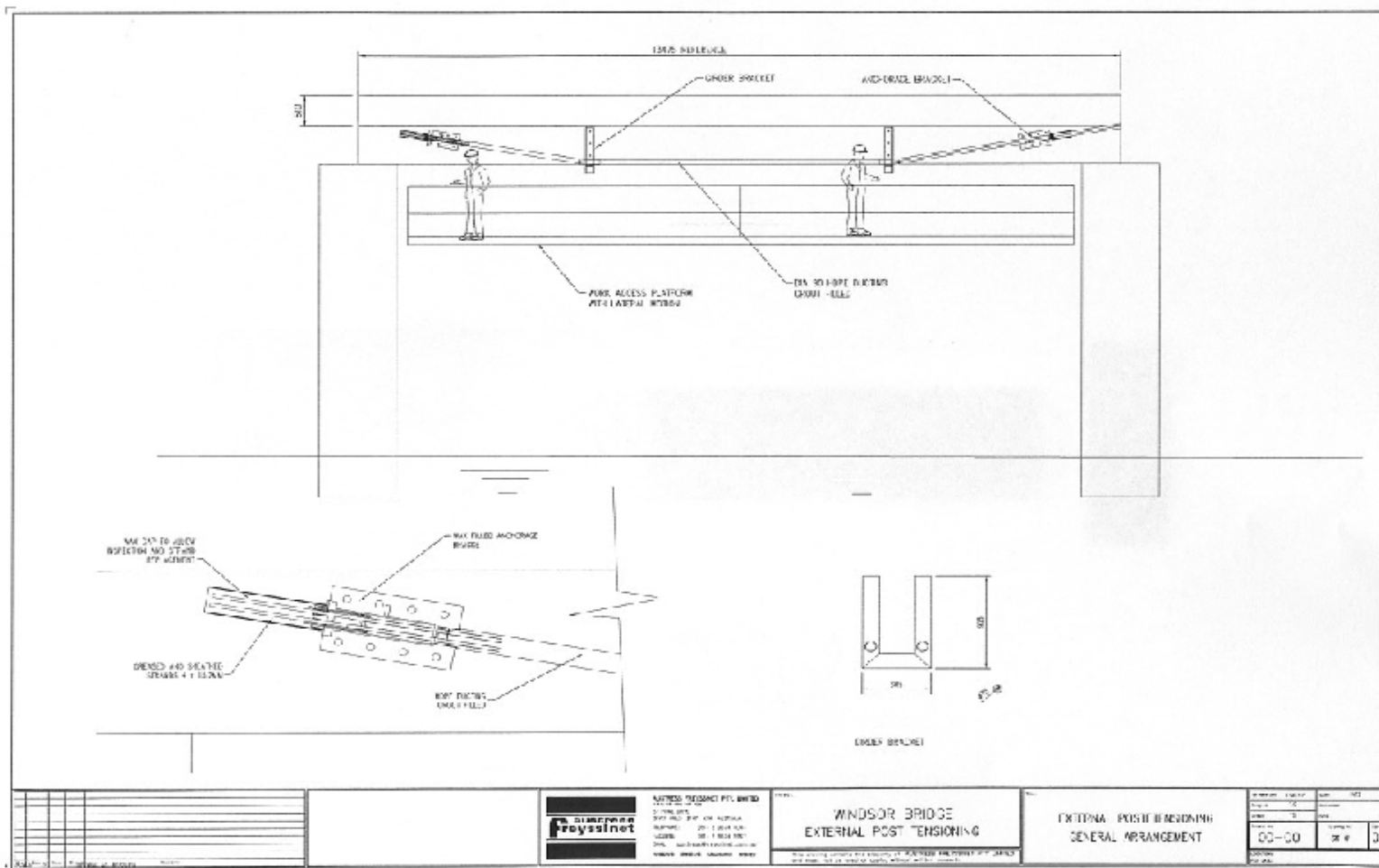
Annexure 2-Live Load Factors for Beams



Annexure 3-Strengthening Option1 Beam Layout



Annexure 4-Strengthening Option1 Beam Types and Joint Layout



Annexure 5-Strengthening Option 2-General Layout

## Cost Estimation for Strengthening Option1

### Strengthening of Bridge over Hawkesbury River at Windsor

#### Option 1-Installation of Steel Girders

Item	Description	Quantity	Unit	Rate	Amount excluding contingencies	Contingencies %	Amount including contingencies
1	Supply and delivery of 700WB 173 Grade 300 steel beams	148	t	2405	\$ 355,940.00	50%	\$ 533,910.00
	<b>Installation of steel girders including transporting from motorised barge equipped with 2 forklifts or 3 Ton HIAB, installation of site joints, jacking up and grouting between the girders and the deck soffit</b>						
2	Barge hire Including operating crew	75	day	1580	\$ 118,500.00	50%	\$ 177,750.00
3	Hoisting equipment hire-3 ton HIAB including operator	75	day	450	\$ 33,750.00	50%	\$ 50,625.00
4	Establishment/de-establishment of barge	2	item	800	\$ 1,600.00	50%	\$ 2,400.00
5	Mobilisation/de-mobilisation of hoisting equipment	2	item	300	\$ 600.00	50%	\$ 900.00
6	Supply and erection of temporary scaffolds for 3 spans at a time and dismantling and re-erection for all subsequent spans	1	item	200,000	\$ 200,000.00	50%	\$ 300,000.00
7	Labour charges for 6 men crew for placing beams on temporary scaffolds, positioning of beams on headstock and installing connections 8.5 hrs per day@\$50/hr plus 35% overheads	65	day	3450	\$ 224,250.00	50%	\$ 336,375.00
8	Jacking up beams and grout the space between steel girders and deck	1570	m	150	\$ 235,500.00	50%	\$ 353,250.00
9	Drill holes in the slab to facilitate construction and patching up	1	item	55000	\$ 55,000.00	50%	\$ 82,500.00
10	Road closures-1 lane at any instant	60	days	1000	\$ 60,000.00	20%	\$ 72,000.00
	<b>Total</b>				<b>\$ 1,285,140.00</b>		<b>\$ 1,909,710.00</b>

#### Annexure 6-Cost Estimate for Strengthening Option 1

## Cost estimation for Strengthening Option 2

### Strengthening of Bridge over Hawkesbury River at Windsor, NSW

#### External Post Tensioning of Existing Reinforced Concrete Beams

##### Details:

Overall length of the bridge 143m

Number of Spans: 11 spans, span lengths from 9.78m -13.47m

Item	Description	Qty	Unit	Rate	Amount excuding contingencies	Contingencies	Amount including Contingencies
1	Installation of Monostrands with sheathing, prestressing, jacking, grouting and capping, to 6 beams each of 11 spans (length 9.78m - 17.47m ) employing 1 crew for 22 weeks	1	item	478,000	\$ 478,000.00	50%	\$ 717,000.00
2	Fabrication of Grade 300 steel brackets, for end anchorage and intermediate saddles to all beams	1	item	100,000	\$ 100,000.00	50%	\$ 150,000.00
3	Installation of steel brackets including drilling beams, fixing with 30mm bolts epoxied into the beam ad patch up beam	1	item	120,000	\$ 120,000.00	50%	\$ 180,000.00
4	Construction of working platform supported on existing piers and removal and re-installation at each span, employing 1 crew for 22 weeks	1	item	190,000	\$ 190,000.00	50%	\$ 285,000.00
5	Cost of road closures 1 lane at a time for approximately 22 weeks	100	days	1000	\$ 100,000.00	50%	\$ 150,000.00
6	Miscellaneous expenditure	1	item	10,000	\$ 10,000.00	50%	\$ 15,000.00
	<b>Total</b>				<b>\$ 998,000.00</b>		<b>\$ 1,497,000.00</b>

## Annexure 7-Cost Estimate for Strengthening Option 2