Proposed Sand/Soil Extraction

Lot 32 DP 635271, Macarthur Road, Spring Farm

Flood Impact Assessment

For: M Collins & Sons (Contractors) Pty Limited

JUNE 27, 2011

Project Name:	Spring Farm Flood Modelling
Project Number:	30011012
Report for:	M Collins & Sons (Contractors) Pty Limited

PREPARATION, REVIEW AND AUTHORISATION

Revision #	Date	Prepared by	Reviewed by	Approved for Issue by
A	20.06.11	Tak Nose / Ben Patterson	Chris Kuczera	
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SMEC COMPANY DETAILS

SMEC Australia
74 Hunter Street, Newcastle, NSW 2300

Tel: **(02) 4925 9626**

Fax: **(02) 9925 5566**

Email: **ben.patterson@smec.com**

www.smec.com

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1 INTRODUCTION

This report is a flood impact assessment in support of an application to extend sand/soil extraction and subsequent rehabilitation of land comprising part of Lot 32 DP 635271and Lot 22 DP 833317 Macarthur Road, Elderslie.

M. Collins and Sons currently operate a major extractive industry on the adjoining land to the study site, Lot 22 DP 833317, Macarthur Road, Spring Farm. A comprehensive sand and soil extraction and product blending activity is conducted on the property to the immediate south of the subject land; namely Lot 1 DP 587631. Approval of the existing operation for the extraction of 850,000 cubic metres of sand and soil was granted on 13 October 1988. The relevant approval was issued by the Department of Planning, pursuant to a prevailing Section 101 Direction of the day and has been the subject of several subsequent amendments.

Extraction has proceeded incrementally and will be completed within approximately five years. Rehabilitation is also occurring sequentially. The rehabilitation is focused upon returning the broader floodplain areas to turf production or other similar agricultural land uses. The riverbank is being revegetated with riparian vegetation and the adjoining anabranch is also being embellished with additional native vegetation plantings.

A proposal to provide a further ecological linkage between the anabranch and the river is currently also being investigated.

M. Collins and Sons are exploring prospects of economically expanding supply opportunities from known/established resource bases.

The Elderslie Sand and Soil Deposits have long been acknowledged as a regionally significant resource, which have been conserved for extraction purposes and relevant post extractive land uses, in a rehabilitated landscape.

As described in the project preliminary environmental assessment, it is proposed to incrementally extract approximately 400,000 cubic metres of sand/soil material by open pit methods and to progressively reinstate a final landform which blends with the approved post extractive landform for the adjoining extraction area to the south. Refer **Section 2** for a more detailed description of the proposed development.

It is proposed to seek an amendment to the consent issued by the Department of Planning in respect of the adjoining extraction activity (Lot 22 DP 833317), pursuant to Section 75W of the Environmental Planning and Assessment Act, 1979 (as amended) and the Environmental Planning and Assessment Regulation, 2000.

The Director General of the Department of Planning has issued Director General's requirements pursuant to this application, and these Requirements list the following under "key issues":

> ***Soil & Water*** *– including*
>
> *- a detailed description of the proposed water management system;*
>
> *- a detailed assessment (including modelling) of the potential surface and groundwater impacts of the project, paying particular attention to the potential impacts on the Nepean River and its tributary; and*
>
> *- <u>an assessment of potential flood impacts</u>;*

This report focuses specifically on the potential *flood impacts* of the proposed development and supplements specialist reports by others which are looking at the potential soil and water impacts, in order to address the Director General's Requirements.

The site is subject to flood inundation in events as frequently as the 2 to 5 yr Average Recurrence Interval (ARI) from the Nepean River. A discussion of existing flooding behaviour is outlined in **Section 3**.

Previous modelling undertaken for the '*Upper Nepean Floodplain Management Study & Plan*' (SMEC, April 2001) used a MIKE-11 model that was based on the modelling developed for the '*Upper Nepean River Flood Study*' (*Lyall Macoun Consulting, 1995*). The previous modelling studies were utilised to establish a TUFLOW two-dimensional flood model of the subject site and immediate surround in order to define the flood behaviour at the subject site, and to allow for the comparison of before and after development scenarios in order to document any potential flood impacts and relevant management measures.

Of particular importance are the potential impacts of the proposed extraction on changes in velocity distribution on the floodplain, which can in turn alter the effects of scour and erosion that occurs during flood conditions.

In this respect, the design surface for the development has been developed by Lean & Hayward to remain well clear of the Nepean River bank itself (*allowing a full 40m setback from the top of bank of the Nepean River*). In addition, discussions were held between the designers (*Lean & Hayward*) and Endeavour Energy regarding the need to retain the existing high voltage power line across the northern boundary of Lot 22 DP 833317, and that the maximum grades along this alignment need to be able to allow a rigid truck vehicle access between power poles, such that Endeavour Energy can access its power poles for maintenance purposes.

The potential impact of changes in velocity distribution as a result of the proposed extraction are important in terms of the maintaining the integrity of the Camden Bypass Bridge Viaduct. In this respect, this report has particularly focussed on the changes in velocity in a range of flood events, including the 20 yr, 100yr, 2000yr ARI and PMF events. The focus of the analysis however has been on two particular events, being the 100 year and 2000 year ARI events, in accordance with the requirements of the AUSTROADS AP-23/94 Waterway Design — A Guide to the Hydraulic Design of Bridges.

In this respect, the TUFLOW two-dimensional hydro-dynamic hydraulic model has been used to estimate the potential for additional scour at the subject site, in particular focussing on critical infrastructure such as the Camden Bypass Bridge Viaduct. The model results have been used to provide a comparison of existing and proposed case velocity distributions for the 100 year and 2000 year ARI events. These results are outlined in **Section 4**.

Section 5 summarises the results of the estimation of velocity changes and looks at the potential for any erosion effects on the downstream Camden Bypass Bridge Viaduct, and makes conclusions and recommendations.

2 DESCRIPTION OF THE PROPOSED DEVELOPMENT

2.1 The Site

As described above, the subject land comprises part of Lot 32 of DP 635271 and part of Lot 22 DP 635271, with a site area of some 7.0 hectares and is accessed from Macarthur Road. It largely comprises a Nepean River alluvial floodplain landscape. **Figure 1** shows the existing site features and the proposed extraction area.

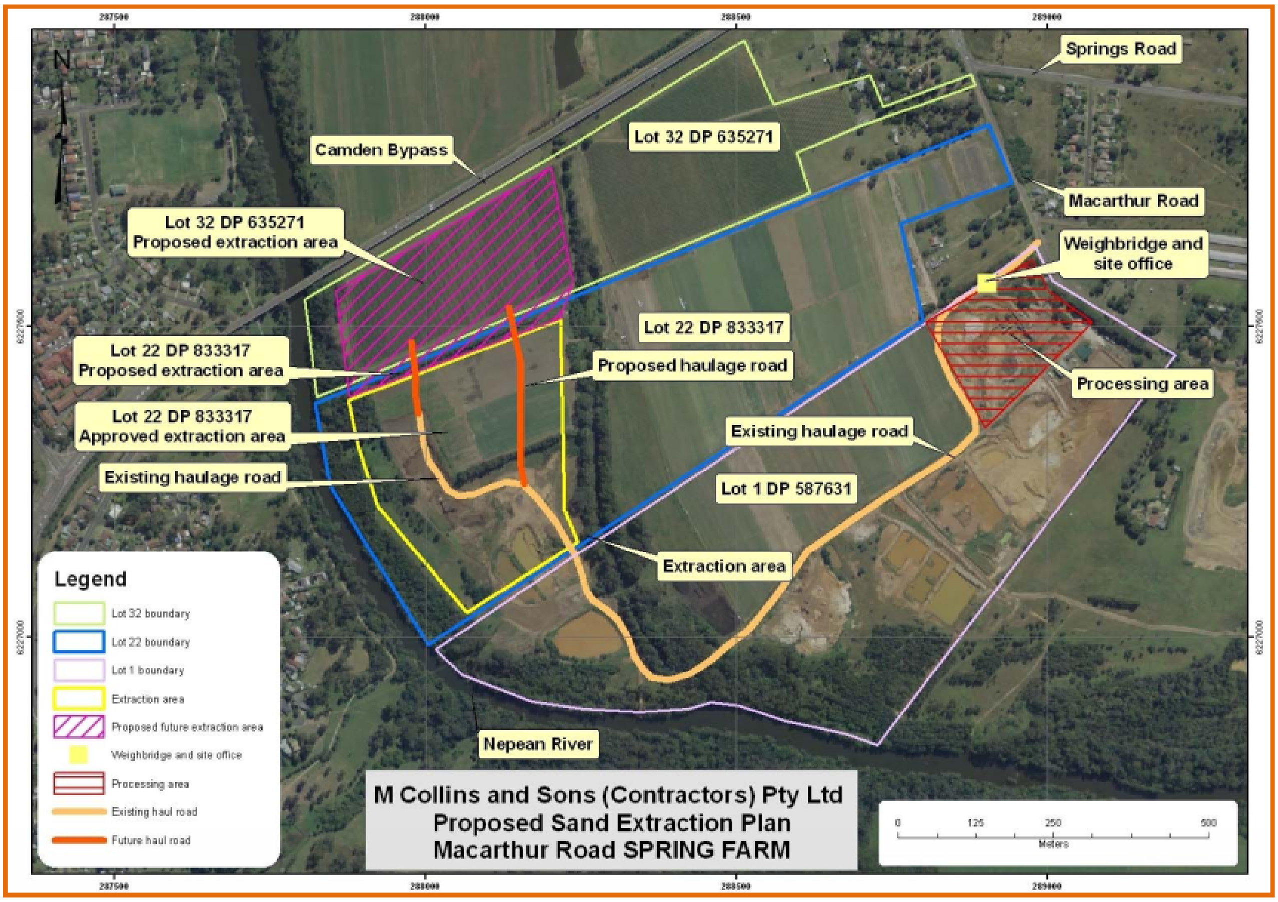


Figure 1: Site layout showing existing site features, and proposed sand/soil extraction area

The western boundary comprises the Nepean River channel, fringing rural residential development fronting Macarthur Road forms the eastern boundary. The elevated Camden Bypass Bridge Viaduct is located to the immediate north of the site and the M. Collins and Sons current extraction activity adjoins to the south.

An anabranch of the Nepean River bisects the allotment running north-south about midblock and extends into adjoining lands, with an ultimate confluence with the Nepean River further to the south of the subject site. This anabranch forms the eastern extent of proposed sand/soil extraction as part of this application within Lot 32 DP 635271, as shown in the Figure above.

As described above, the western boundary comprises the Nepean River channel, which is steep and heavily vegetated. Refer **Photos 1 to 4** below.

Photo 1 – western boundary of subject site looking north showing heavily vegetated riparian vegetation associated with the Nepean River channel, and grassed open floodplain. The elevated Camden Bypass Bridge Viaduct is shown to the north in the Photograph.

Photo 2 – view from south-western corner of subject site looking north towards the elevated Camden Bypass Bridge Viaduct. Note grassed open floodplain.

Photo 3 – view looking north towards the elevated Camden Bypass Bridge Viaduct over the Nepean River. Note height of bridge deck above river.

Photo 4 – view looking north towards the elevated Camden Bypass Bridge Viaduct over the Nepean River. Fence boundary in the foreground. Area between the site boundary and the bridge is characterised by grassed floodplain area.

2.2 Proposed Development

As described in the project preliminary environmental assessment, it is proposed to incrementally extract approximately 400,000 cubic metres of sand/soil material by open pit methods and to progressively reinstate a final landform which blends with the approved post extractive landform for the adjoining extraction area to the south. **Photo 5** shows an example of the existing extraction operation at the adjoining site.

The area to be extracted and the conceptual final landform are shown in **Figures 2 and 3.** The area extends from the anabranch in the east to the high Nepean River riverbank to the west. Importantly, it is not proposed to remove vegetation or material from the river frontage, with a minimum 40 metre setback being observed. **Figure 3** and **Figure 4** show the area where the proposed extraction will be performed, with the relative difference in the ground levels being approximately 6 to 7 m.

For the purposes of modelling of the sand/soil material extraction, we have conservatively adopted a more sudden change in the topography – i.e. we have modelled the effect of opening up the proposed extraction pit in a non-progressive manner – i.e. it has been assumed that the entire pit has been excavated, and that it has been excavated over the existing topography, as shown in **Figure 3**, rather than modelling a much smoother finished landform, as shown in **Figure 4**.

Photo 5 – Photograph shows the existing sand extraction operations using large front end loaders, particle size screens and articulated highway trucks for removing finished product. Note height of temporary batters around sand extraction typically 6 to 7 metres.

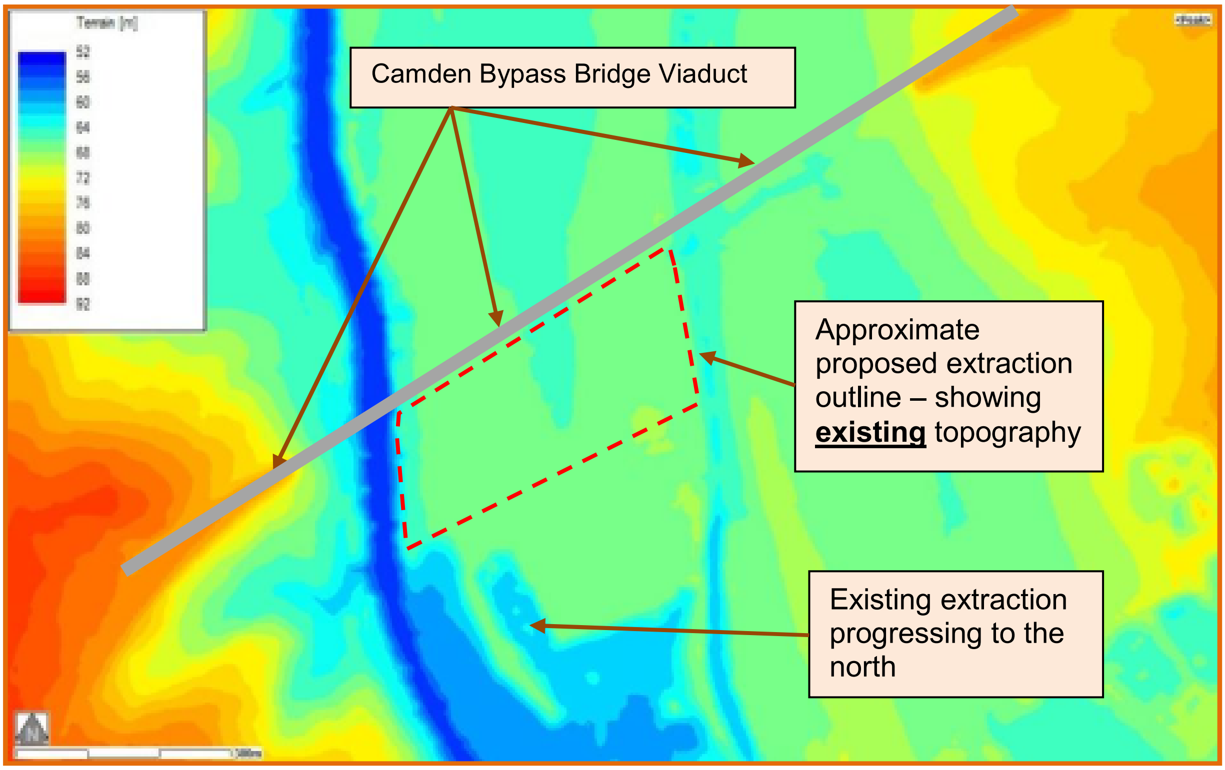


Figure 2: Existing Site Topography

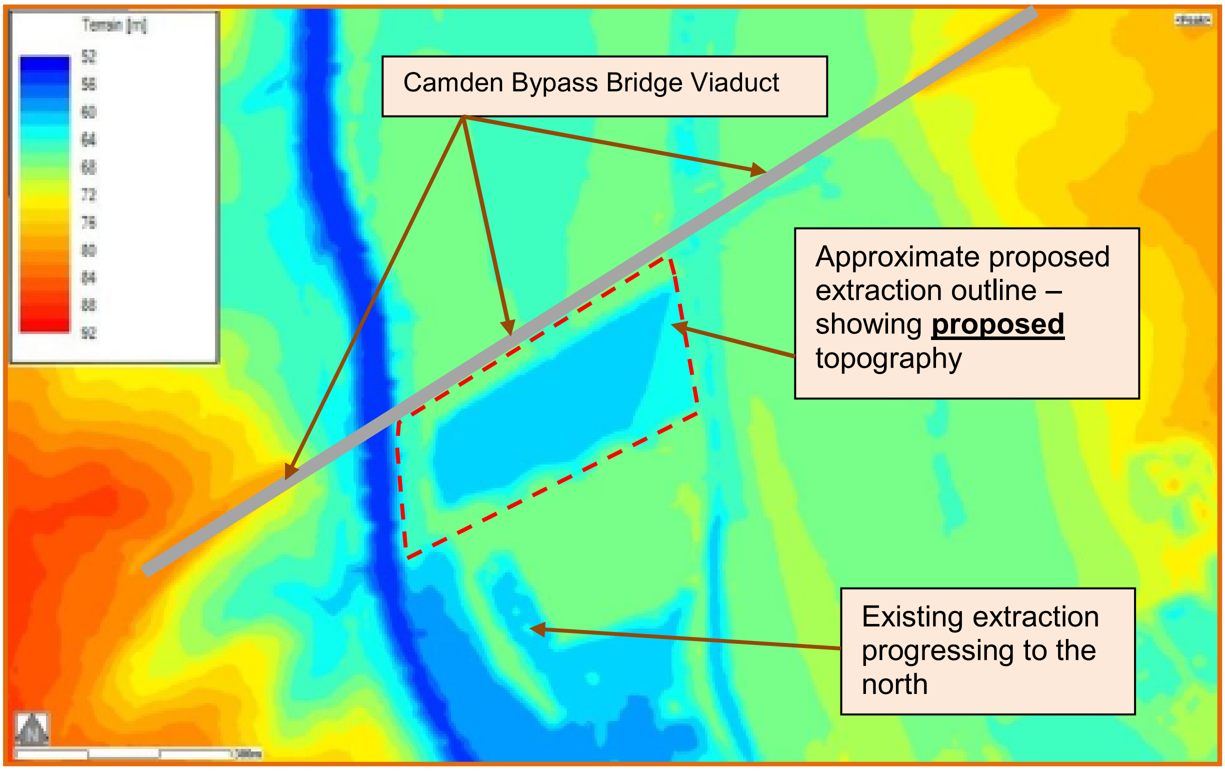


Figure 3: Proposed Site Topography (used in Modelling)

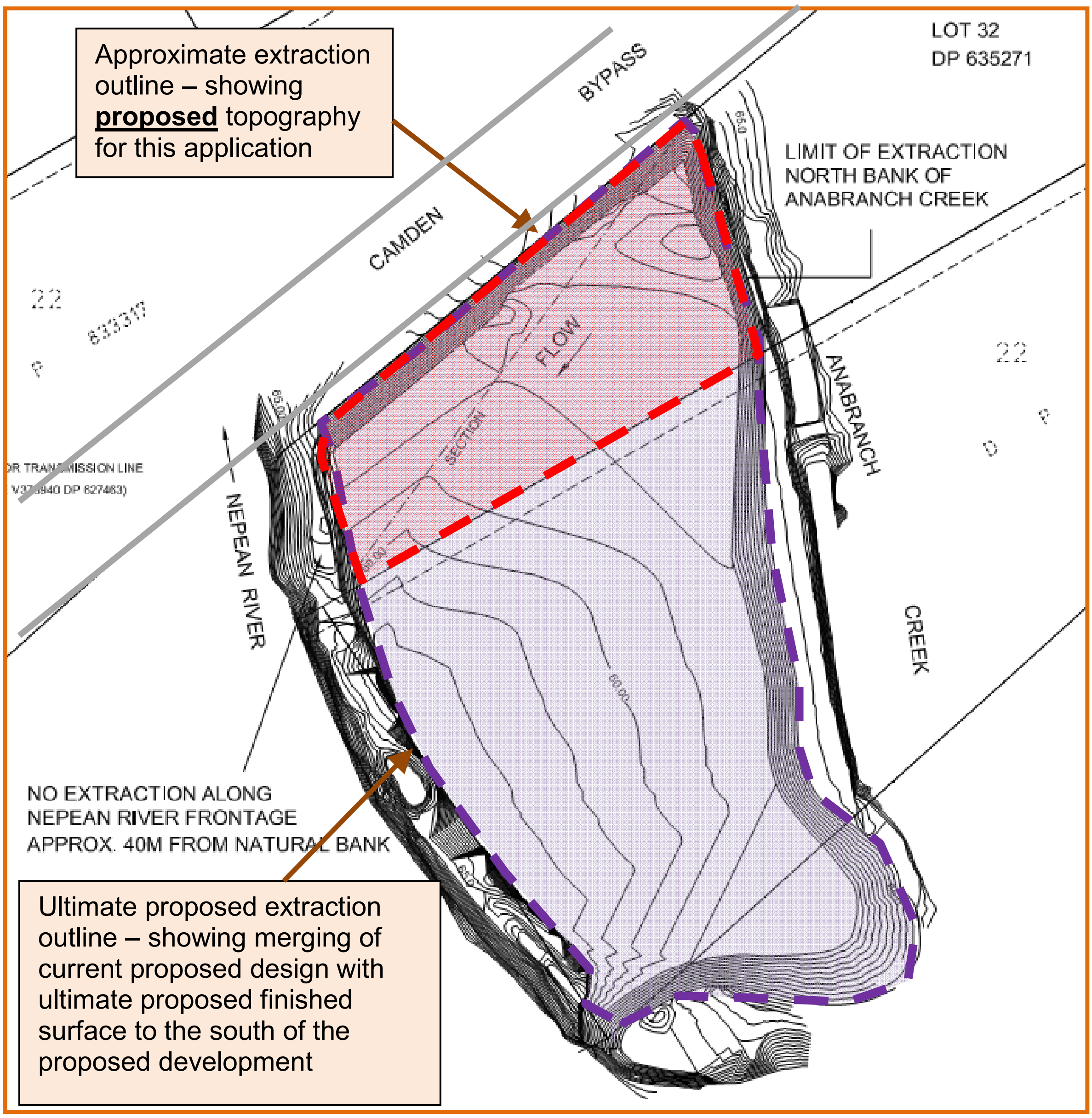


Figure 4: Ultimate finished landform, once merged with existing sand/soil extraction to the south of the proposed extraction area (not adopted in the flood impact modeling, as it is being rehabilitated).

In addition, **Figure 5** below indicates the finished landform design for the proposed development site. Note that the design specifically allows for gaps between successive High Voltage power poles that span the southern boundary of the proposed extraction site. These gaps are to allow flood water to flow back to the south on the receding limb of the flood hydrograph, once the main flood peak has occurred. The mounds that support the electricity poles will be revegetated with natural grasses. This will ensure the potential for scour erosion within the gaps is effectively managed. Note that extraction area will initially fill with floodwater from backwater flooding from the Nepean River backing up from the south, under conditions when the Nepean River is on the rising limb of the flood hydrograph, as described above. Under this scenario, velocities will be low (*due to the nature of backwater flooding having a near horizontal hydraulic profile*) and the extraction area will fill right up to the level of the existing ground surface on the western, northern and eastern boundaries. Then as the floodwaters build (*e.g. in events exceeding a 2 to 5 year ARI event*) the entire extraction site will be inundated, and the flow behaviour across the subject site will begin to be dominated by the main overbank flow in the Nepean River. **Section 4** outlines the typical flood levels and velocities across the subject site during the 20, 100, 2000 yr ARI and Probable Maximum Flood events.

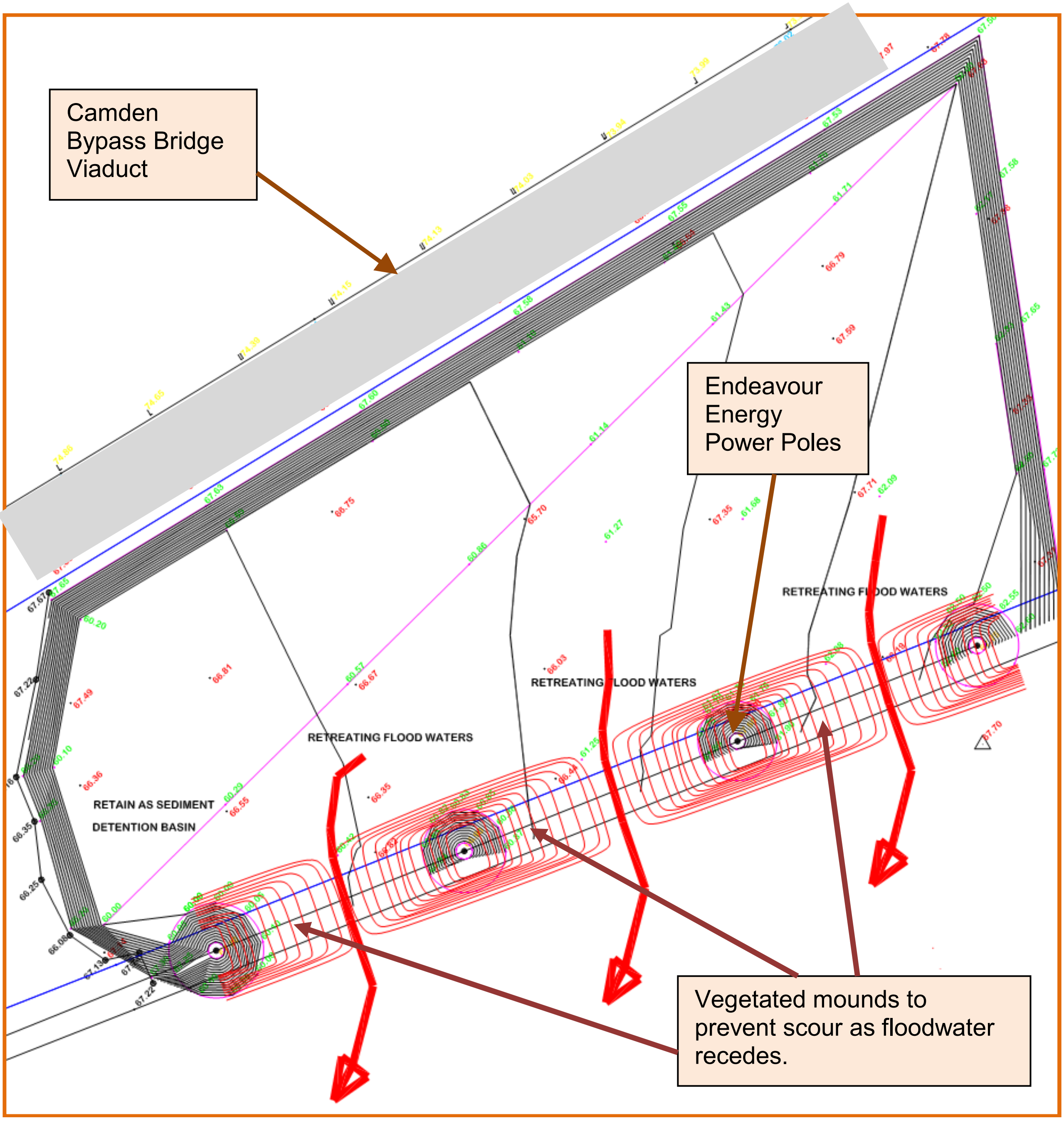


Figure 5: Finished landform for this development, showing the existing power poles on the southern side operated by Endeavour Energy. Discussions with Endeavour Energy were undertaken by Lean & Hayward during the finished surface design to ascertain their batter requirements. Gaps were left in the design to allow flood water to recede to the south into the existing extraction area, and then ultimately back to the Nepean River on the falling limb of any flood.

3 PREVIOUS MODEL ESTABLISHMENT AND FLOODING BEHAVIOUR

3.1 Previous Studies

Previous mapping of Nepean River flooding was undertaken by SMEC Australia Pty Ltd for the *'Upper Nepean Floodplain Management Study & Plan'* which was completed in April 2001.

Modelling for the SMEC 2001 study was based on MIKE-11 modelling for the *'Upper Nepean River Flood Study'* which was undertaken in 1995 by Lyall Macoun Consulting Engineers (LMCE) on behalf of the then NSW Department of Land and Water Conservation (DLWC). This model extended from Menangle Bridge downstream to the confluence with the Warragamba River. The model also included several of the tributaries that drain to the Nepean River, however the cross sections within the tributaries were coarsely spaced, and were included in the model mostly to take account of the storage effects of backwater flooding. Floods assessed in the study were the 20 and 100 year ARI events and the Probable Maximum Flood (PMF).

Additional modelling was undertaken by both Lyall Macoun Consulting and SMEC Australia Pty Ltd in 1996 and 1999, mostly associated with the more detailed modelling of tributaries.

A series of flood maps at a 1:1000 scale were prepared for the flood affected areas of Camden and surrounds based on the results of the 1995 Flood Study and updated to incorporate the results of the Tributary Flood Studies (LMCE 1999).

The SMEC April 2001 Floodplain Management Study and Plan produced flood maps at a scale of 1:10,000 for the Nepean River and tributaries between Menangle Weir and the confluence with Bringelly Creek for the 20 and 100 year ARI flood events and the PMF. These maps were used to derive calibration marks for the current round of modelling for this study.

3.2 Flood Behaviour

The Upper Nepean River catchment that was addressed in the 1995 Flood Study has an area of about 1,800 km^2 and comprises the Nepean River and its tributaries upstream of the confluence with the Warragamba River. At Cowpasture Bridge in Camden (*the bridge 2.5km downstream of the Camden Bypass Bridge Viaduct*), the main flood reference location, the catchment area is some 1,380 km^2.

The Nepean River together with its major tributary streams, the Avon and Cordeaux rivers, rises in the Illawarra Range west of Wollongong at elevations of around 450m. After draining heavily timbered country in its headwaters, the Nepean River is joined on its left bank by the Bargo River and on its right bank by the Cataract River a further 15km downstream. The Nepean River then flows in a generally northwest direction past Camden until it is joined on its left bank by the Warragamba River downstream of Wallacia.

There are four major water storages in the Upper Nepean Catchment – Cataract, Cordeaux, Avon and Nepean Dams. Although these dams control some 50% of the catchment to Camden, detailed hydrologic studies indicate that the storages have minimal mitigating impact on the majority of floods at Camden.

The river's course is marked by a series of steep gorges with clearly defined floodplains in the flatter areas. The largest of these floodplains occurs in the immediate vicinity of Camden. The Nepean River runs along the northern and eastern boundaries of Camden and is joined by Matahil, Sickles, Narellan and Navigation Creeks near the town.

Camden is the major urban centre located on the Upper Nepean River. The satellite suburbs of Elderslie and Narellan are located generally north and east of the main town and the village of Cobbitty is located on a ridge to the north. To the west of the main town, the principal areas of development are the "rural residential" areas of Ellis Lane and Grasmere.

Under flood conditions, the Nepean River overflows its banks and commences to inundate the low lying floodplain around Camden during floods in excess with an Average Recurrence Interval (ARI) of two (2) years. This corresponds to a flood height of 8.5m (64.2m AHD) on the flood warning gauge at Cowpasture Bridge.

The areas along the minor tributaries are affected by backwater flooding from the Nepean River. The tributaries themselves have relatively small catchments and therefore generally do not contribute significantly to flooding in the main river. During major flooding, Camden is surrounded by floodwaters.

Table 1 lists historical floods where the flood level exceeds 12.0m at the Cowpasture Bridge gauge (RL 67.7m, AHD). A detailed Annual list of peak flood heights was prepared for the Upper Nepean River Flood Study (DLWC 1995 – Volume 2) and is reproduced below. It should be noted that discharges are estimates only, calculated using gauge heights at the Cowpasture Bridge gauge.

Date	Peak Gauge Height (m)	Discharge (m^3/s)	Date	Peak Gauge Height (m)	Discharge (m^3/s)
1860	14.19	4860	1898	15.21	7170
1864	15.40	7400	1904	13.80	3870
1867	14.01	4390	1912	12.20	2540
1868	12.50	2720	1916	13.60	3700
1870	14.50	5320	1949	12.61	2780
1871	14.00	4280	1950	12.46	2660
1873	**16.53**	**10,060**	1956	12.42	2660
1875	13.90	4050	1961	12.51	2720
1877	14.50	5320	1964	14.08	4500
1879	14.20	4860	1975	12.70	3030
1889	13.80	3870	*1978*	*13.45*	*3580*
1890	13.70	3760	1988	12.80	3050

Table 1 - Major Historical Floods for the Upper Nepean River at Cowpasture Bridge (Camden)

It should be noted that the largest flood on record, in 1873 had a peak discharge estimate of some 10,060 m^3/s. The estimated 100 year ARI event has a peak discharge estimate of some 7,900 m^3/s. Thus the 1873 flood on the Upper Nepean River was estimated to be well in excess of the 100 year ARI event.

Also note for the 1978 event, which was used in the 1995 Flood Study as a calibration event, as the gauge actually recorded a stage hydrograph for this event.

Site Specific Flood Behaviour

As described above the Nepean River channel itself has a capacity of approximately a 2 to 5 year ARI event. Above these events, flow begins to push out onto the Nepean River Floodplain, of which the subject site forms a part.

As the floodwaters build under conditions when the Nepean River is on the rising limb of the flood hydrograph, the extraction area will initially fill with floodwater from backwater flooding from the Nepean River backing up from the south. Under this scenario, velocities across the subject site will be low (*due to the nature of backwater flooding having a near-horizontal hydraulic profile*) and the extraction area will fill right up to the level of the existing ground surface on the western, northern and eastern boundaries. Then as the floodwaters build further (*e.g. in events exceeding a 2 to 5 year ARI event*), the entire extraction site will be inundated, and the flow behaviour across the subject site will begin to be dominated by the main overbank flow in the Nepean River. **Section 4** below outlines the typical flood levels and velocities across the subject site during the 20, 100, 2000 yr ARI and Probable Maximum Flood (PMF) events.

Once the flood peak has been reached, velocities across the subject site will drop off, and once the river levels recede, the extraction area will drain back towards the river under limited hydraulic grade (note the variation in water level within the river is relatively slow, such that even on a falling limb of a main river flood hydrograph, localised velocities associated with the extraction area emptying (*termed "draw-down"*) are not expected to dominate in terms of maximum peak velocities across the subject site. **Section 4** shows that the peak velocities experienced across the subject site generally correspond with the peak discharges in the river floodplain area.

Potential for collapse of the natural river bank in the aftermath of a flood that overtops the high bank.

The modelling confirms that the retained natural river bank will be overtopped during the 20, 100, 2000 year and PMF events.

The water table in the adjacent river bank will also rise in the natural river bank during the major floods events. In the aftermath of the flood, the water level in the river will recede however the water level in the river bank may remain elevated for an extended period of time following the flood.

In order to assist with the stabilisation of the river bank, the existing vegetation on the river bank will be retained and improved with the additional plantings of native vegetation within the buffer zone provided (40m wide) on the river side of the proposed extraction.

4 HYDRAULIC MODELLING

4.1 TUFLOW Model Establishment

The hydraulic flood modelling for this flood impact assessment was undertaken using a TUFLOW (two-dimensional hydro-dynamic) hydraulic model. The hydraulic model was developed to assess the flood impact due to the proposed development, particularly near the Camden Bypass Bridge Viaduct and the surrounding properties. The TUFLOW model is a two dimensional finite difference grid and performs hydraulic calculations of unsteady flow that vary with time.

A two dimensional hydro-dynamic model was adopted for this project because the inclusion of the proposed sand/soil extraction in the surface topography is likely to set up complex two-dimension flow re-distribution during the peak of a major flood event, and this type of flow re-distribution is likely to be defined more reliably using a two dimensional flood model rather than the more simple one-dimensional model, such as the original MIKE-11 model used for the Flood Study (1955, Lyall Macoun Consulting) and Floodplain Management Study (2001, SMEC).

TUFLOW was selected as it a fairly robust two-dimensional model, that is well accepted by Council and Department of Environment and Heritage (ex DECCW) officers.

Rather than model the entire Nepean River system outlined in the Flood Study, it was only necessary to generate a “near field” flood model that extended approximately 2km upstream and downstream of the subject site in order to give a platform with which to examine the relative impacts of the proposal on surrounding existing properties and infrastructure.

As outlined in **Section 1**, of particular concern raised by the Department of Planning’s Director General’s requirements, was the effect of scour on the existing Camden Bypass Bridge Viaduct. For this reason, the model domain was extended to an area upstream and downstream of the subject site (note: over two (2) kilometres upstream and downstream).

Once the TUFLOW model domain was established, it was necessary to calibrate the model to the original MIKE-11 model results for a range of events, to ensure the model was consistent with previous flood modelling.

The TUFLOW model has been calibrated for the 20yr ARI, 100 yr ARI, and Probable Maximum Flood (PMF) events based on the SMEC MIKE 11 maximum water surface levels derived from the SMEC *‘Upper Nepean Floodplain Management Study & Plan’* completed in April 2001. Design hydrographs were generated from gauged hydrograph shapes outlined from the Flood Study for the 1978 Flood event. The hydrograph shape was interpreted using peak discharge estimates for the various design storms required to generate design hydrographs for the 20 year, 100 year and PMF events.

As such, the TUFLOW model developed for this study is considered suitable for assessing the flood impact at the proposed sand/soil extraction site.

4.2 TUFLOW Model Domain and Geometry

The model domain for this study extends approximately 2.6 km upstream and 2 km downstream along the Nepean River from the proposed development site, respectively.

The model utilises an 8 m grid cell and is based on the Digital Elevation Model (DEM) interpolated at a 10 m resolution, developed primarily from topographic LiDAR survey data purchased for this project. Bathymetry for the waterway (i.e. the Nepean River) was sourced from previous MIKE 11 model cross sections (*i.e. typically approximately 400m spacing along the river*) and incorporated into the DEM.

The Digital Elevation Model (DEM) for the TUFLOW model domain is displayed in **Figure** 6.

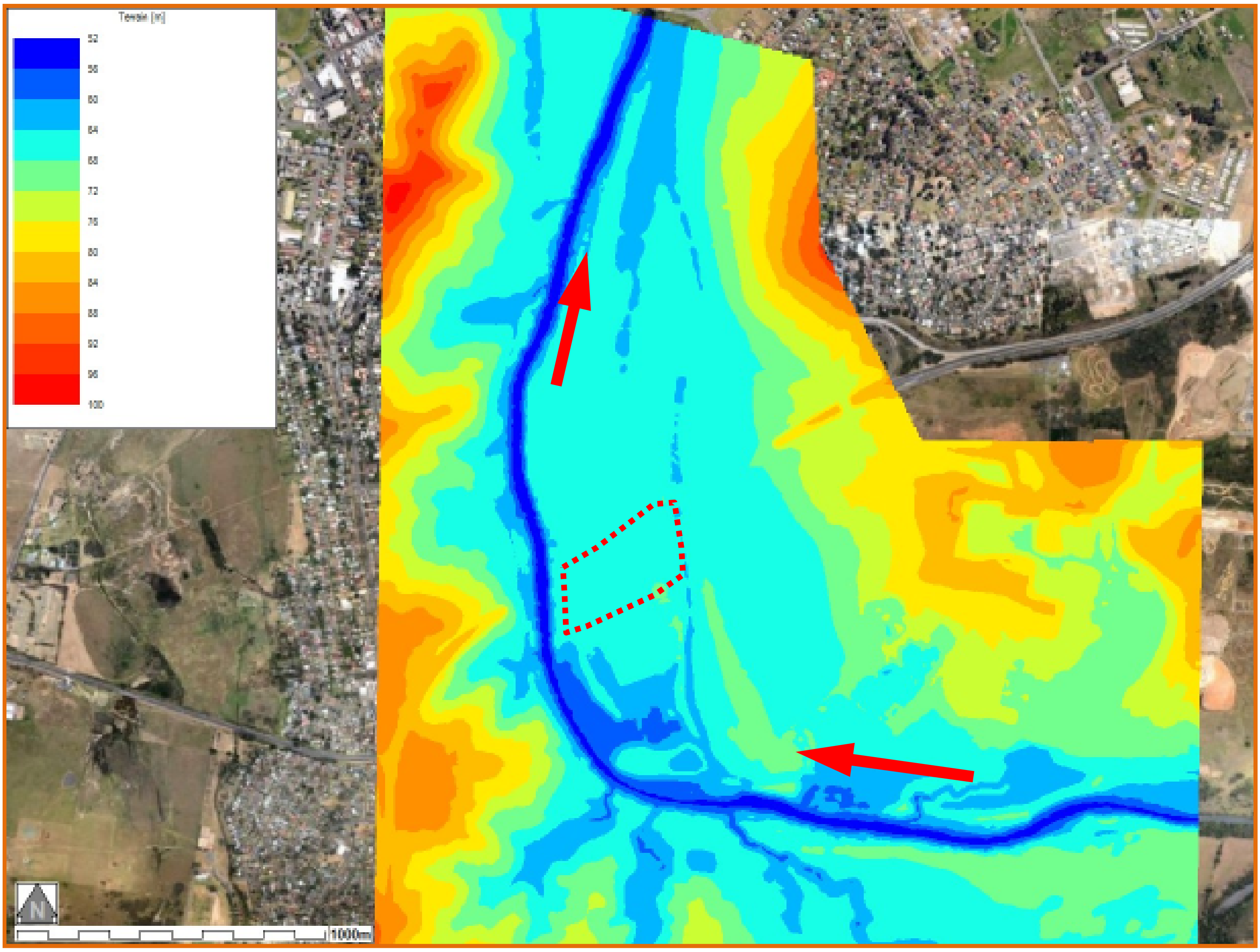


Figure 6: TUFLOW model domain Digital Elevation Model (DEM)

Surface roughness of the model domain was mapped from examination of high resolution aerial photography imagery, and based on our site walkover. The roughness map and adopted Manning's roughness values are presented in **Figure 7**.

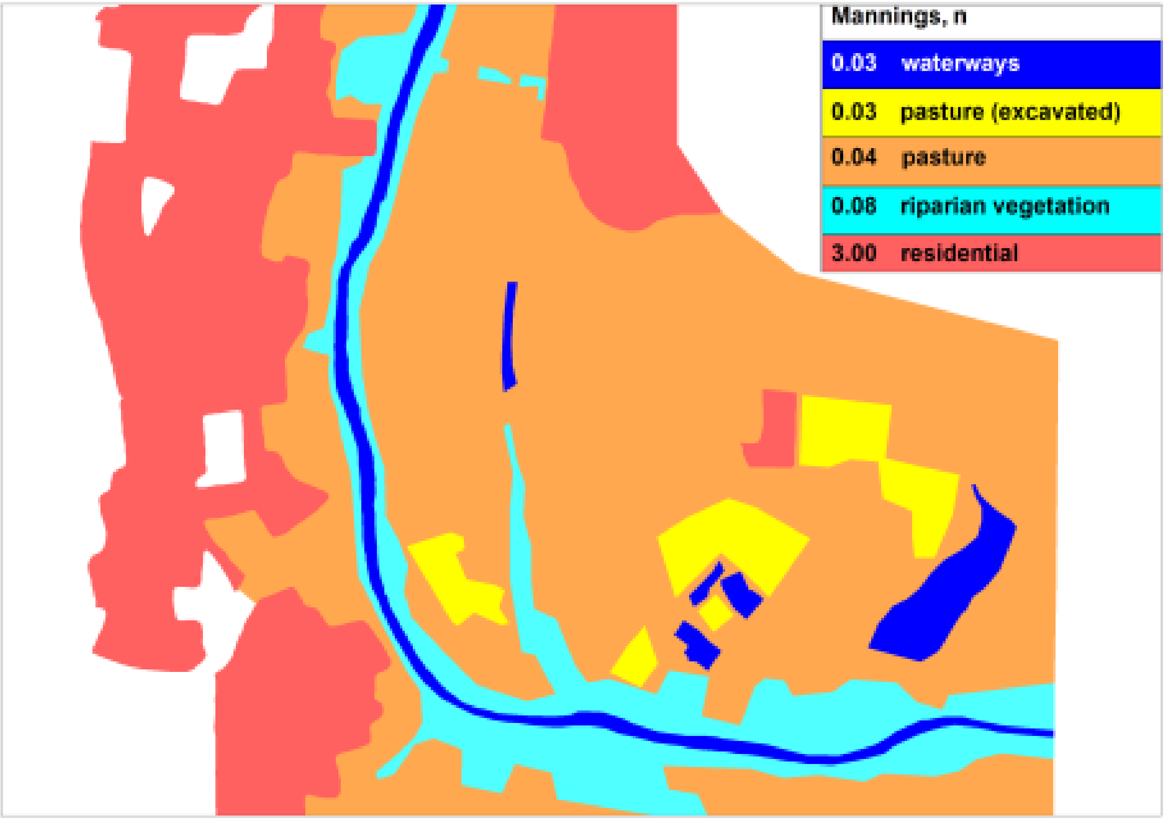


Figure 7 – Adopted Manning's roughness values for the TUFLOW model domain

HEC-RAS modelling was used to derive the head loss coefficients ("K" values) adopted in TUFLOW for flow at the Camden Bypass Bridge Viaduct. The derived "K" values were 0.2 and 0.47 for the flow below and above the bridge deck respectively. The HEC-RAS model was also used (refer **Section 5** below) to determine the potential bridge scour potential for the Camden Bypass Bridge Viaduct as a result of velocity changes brought about by the proposed development

4.3 Model Boundary Conditions

4.3.1 Upstream Boundary Conditions

Hydraulic model inflows for the upstream boundary located approximately 2.6 km upstream from the proposed development site were defined using discharge hydrographs. The 20 year, 100 year, 2000 year and PMF recurrence flood discharge hydrographs were pro-rated based on the March 1978 flood discharge hydrographs. The discharge hydrographs applied to the upstream boundary condition are summarised in **Table 2** and shown in **Figure 8**.

		Adopted River Discharge Hydrographs for upstream Boundary Conditions (m^3 / s)			
Time (hr)	1978	20 yr	100 yr	2000 yr	PMF
0	0	0	0	0	0
6	100	150	300	460	500
12	100	150	300	460	1,000
18	100	150	300	460	1,000
24	250	400	600	919	1,000
30	1,000	1,500	2,000	3,065	6,300
36	2,300	3,500	6,000	9,194	14,000
42	3,300	4,650	7,700	11,800	16,500
48	3,650	4,900	7,900	12,106	17,950
54	3,500	4,800	7,600	11,646	17,200
60	3,000	4,100	6,800	10,420	15,000
66	2,800	3,700	5,800	8,888	14,000
72	2,100	2,400	4,800	7,356	10,000
78	1,500	1,800	3,300	5,057	7,000
84	1,000	1,300	2,000	3,065	5,000
90	800	1,100	1,500	1,800	2,000

Table 2 – Adopted River Discharge Hydrographs used for Upstream Boundary Conditions

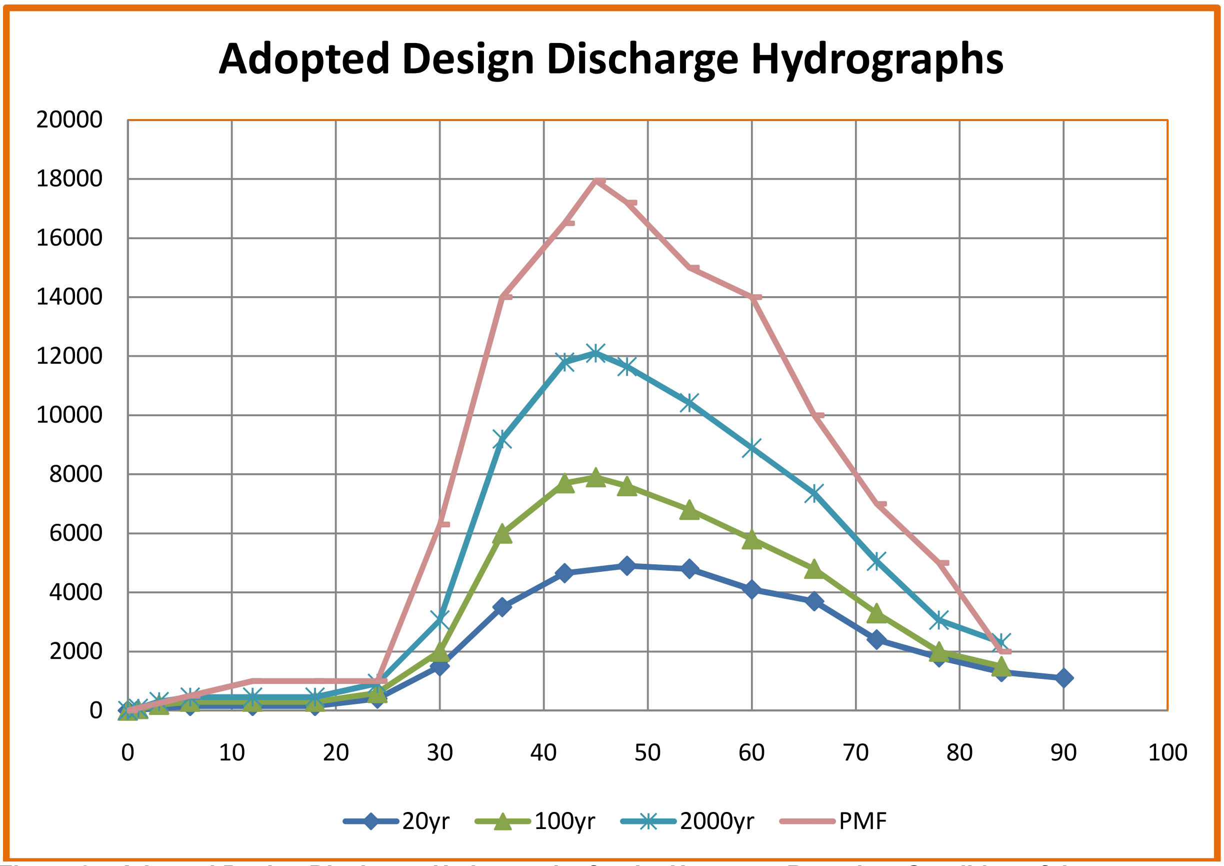


Figure 8 – Adopted Design Discharge Hydrographs for the Upstream Boundary Condition of the TUFLOW model

4.3.2 Downstream Boundary Conditions

Stage hydrographs were applied to the downstream boundary located approximately 2 km from the proposed development site. In a similar manner to the upstream boundary condition, the 20 year, 100 year and PMF recurrence flood stage hydrographs were extrapolated based on the March 1978 flood stage hydrographs. The extrapolated data was calibrated such that the downstream tailwater levels were within 0.5% of the previously calibrated MIKE 11 model (refer **Section 4.4** for details on model calibration).

The stage hydrographs applied to the downstream boundary conditions are summarised in **Table 3** and shown in **Figure 9.**

		Stage Hydrographs adopted for Downstream Boundary Condition (m AHD)			
Time (hr)	**1978**	**20 yr**	**100 yr**	**2000 yr**	**PMF**
0	53.5	53.5	53.5	54.2	53.5
6	58.0	59.0	64.0	64.8	63.5
12	58.0	59.0	64.0	64.8	66.0
18	58.0	59.0	64.0	64.8	66.0
24	62.0	63.0	65.0	65.8	66.0
30	65.0	65.5	66.0	66.9	70.0
36	67.0	68.5	70.0	70.9	71.7
42	69.0	69.4	70.5	71.5	72.4
48	69.0	70.0	71.1	72.1	73.7
54	69.0	69.8	70.8	71.7	73.6
60	68.3	69.2	70.7	71.7	72.4
66	68.3	69.1	70.0	70.9	72.6
72	67.5	67.2	69.1	70.0	71.6
78	66.3	66.0	67.9	68.7	70.6
84	65.0	65.5	66.3	67.2	70.0
90	64.5	65.2	65.7	66.1	66.5

Table 3 - Adopted River Stage Hydrographs used for Downstream Boundary Conditions

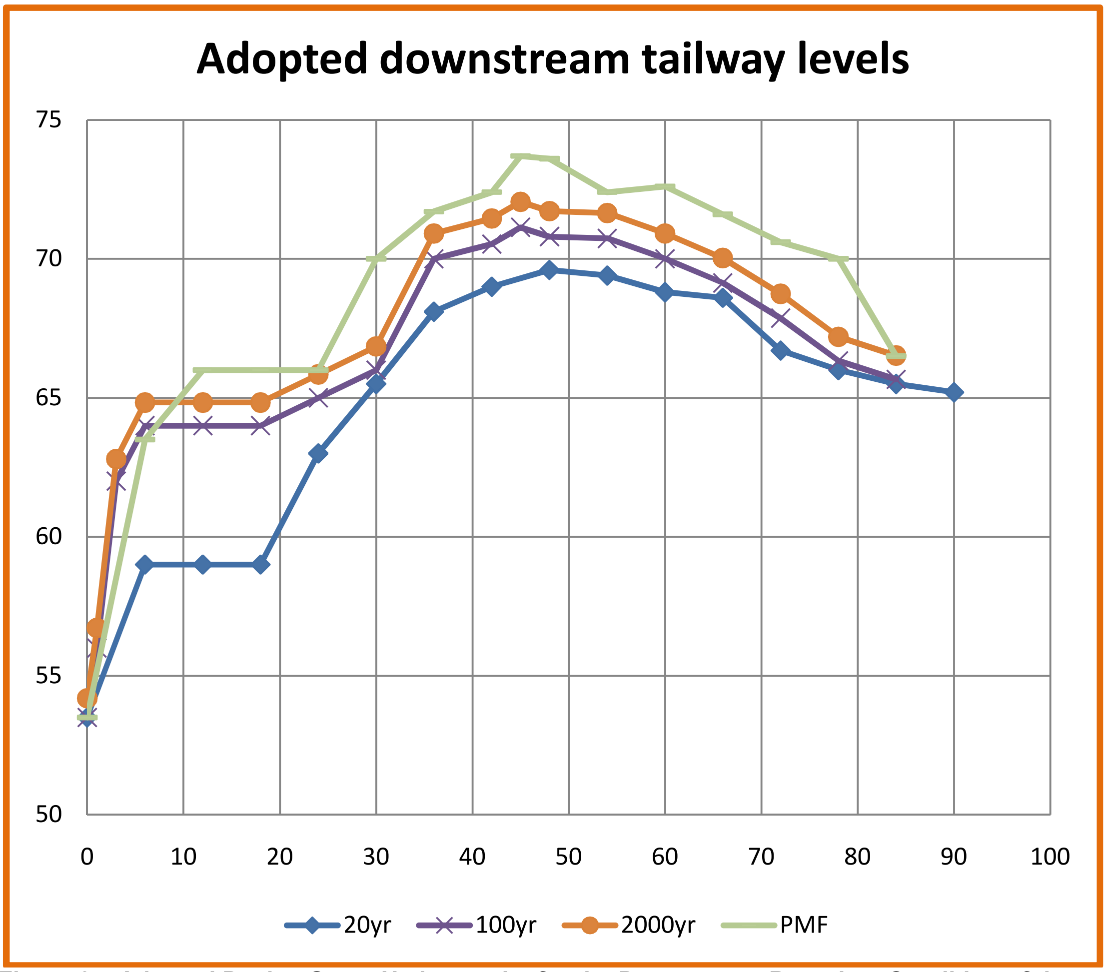


Figure 9 – Adopted Design Stage Hydrographs for the Downstream Boundary Condition of the TUFLOW model

4.4 Model Calibration

Model calibration is a process of setting physically realistic values for model parameters such that the model reproduces observed values to the required level of accuracy. The process provides confidence in the hydraulic model results and is essential to accurately assess the flooding impact due to the proposed sand extraction works.

Due to limited availability of recorded discharge and stage hydrographs within the model domain, the developed model was calibrated to the previous Flood Study MIKE-11 flood model levels for a range of return intervals (20 year, 100 year and PMF flood levels reported from the 2001 SMEC Floodplain Management Study).

The calibration process included a series of minor modifications to the surface roughness, grid schematisation and to the resolution of the finite difference grids until a good agreement with the MIKE-11 model was achieved.

The model calibration points corresponding with flood levels reported at MIKE-11 model cross section are presented in **Figure 10**.

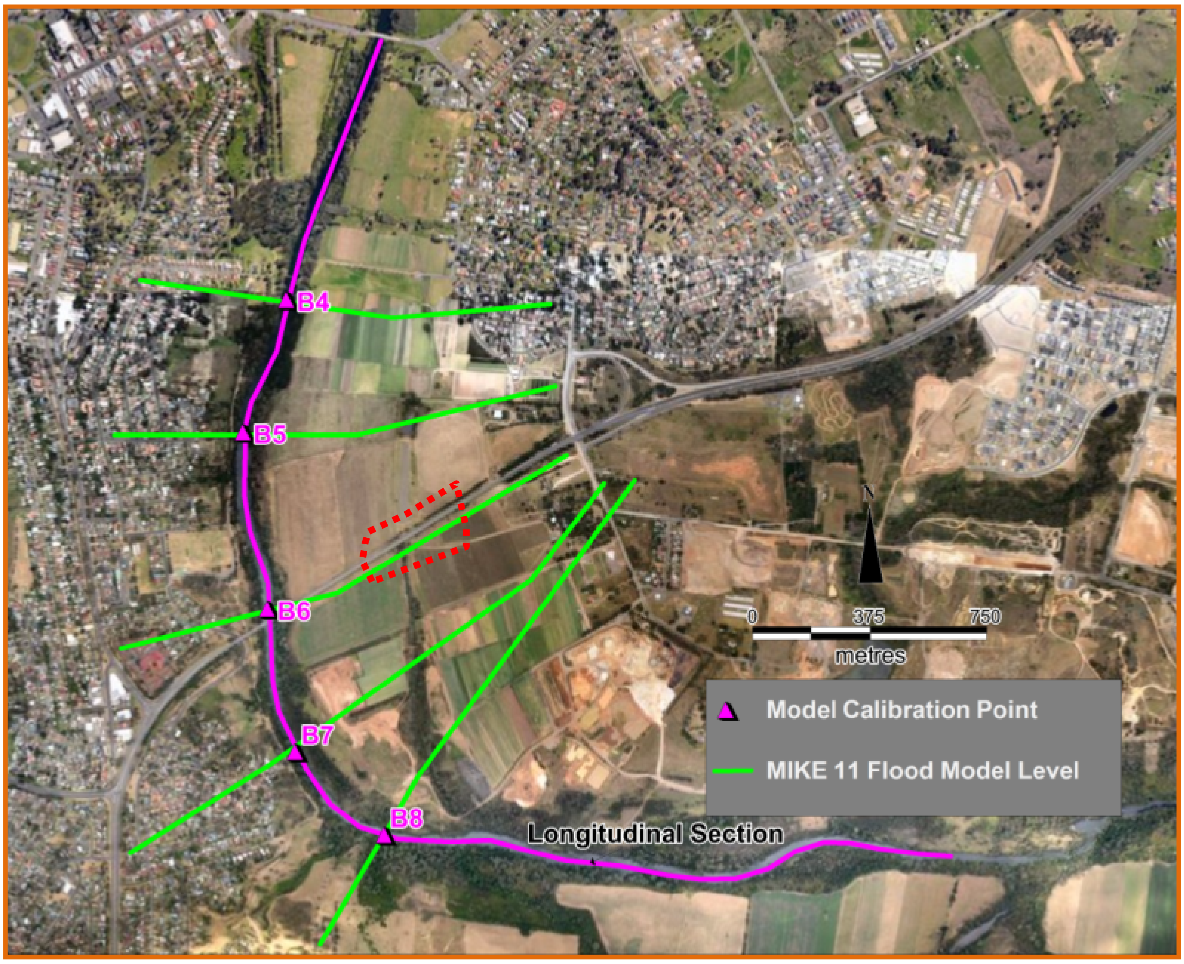


Figure 10 – TUFLOW model calibration points compared to MIKE-11 model Cross Section Locations (after 2001 SMEC Floodplain Management Study)

It can be seen from **Table 4** and **Figure 11** that the model produced good overall agreement with the MIKE-11 data in the study area. The errors in the modelled flood levels are within acceptable levels (*typically ±0.1 to 0.2m*) when compared to the predicted MIKE-11 flood model levels; thereby demonstrating that the model is capable of accurately simulating flood impact assessment due to the proposed sand/soil extraction operations.

	20 year event flood levels (m, AHD)			100 year event flood levels			PMF flood levels		
Calibration Point	MIKE-11 (m, AHD)	TUFLOW Modelled (m, AHD)	Diff.	MIKE-11 (m, AHD)	TUFLOW Modelled (m, AHD)	Diff.	MIKE-11 (m, AHD)	TUFLOW Modelled (m, AHD)	Diff.
B4	70.1	70.3	-0.2	71.8	72.0	-0.2	75.8	75.5	0.3
B5	70.3	70.4	-0.1	72.0	72.2	-0.2	75.9	76.0	-0.1
B6	70.4	70.7	-0.3	72.2	72.5	-0.3	76.0	76.2	-0.2
B7	70.7	70.7	0.0	72.4	72.5	-0.1	76.1	76.3	-0.2
B8	70.9	70.8	0.1	72.6	72.6	0.0	76.3	76.4	-0.1

Table 4 – Differences between Modeled TUFLOW Flood levels and original MIKE-11 Model

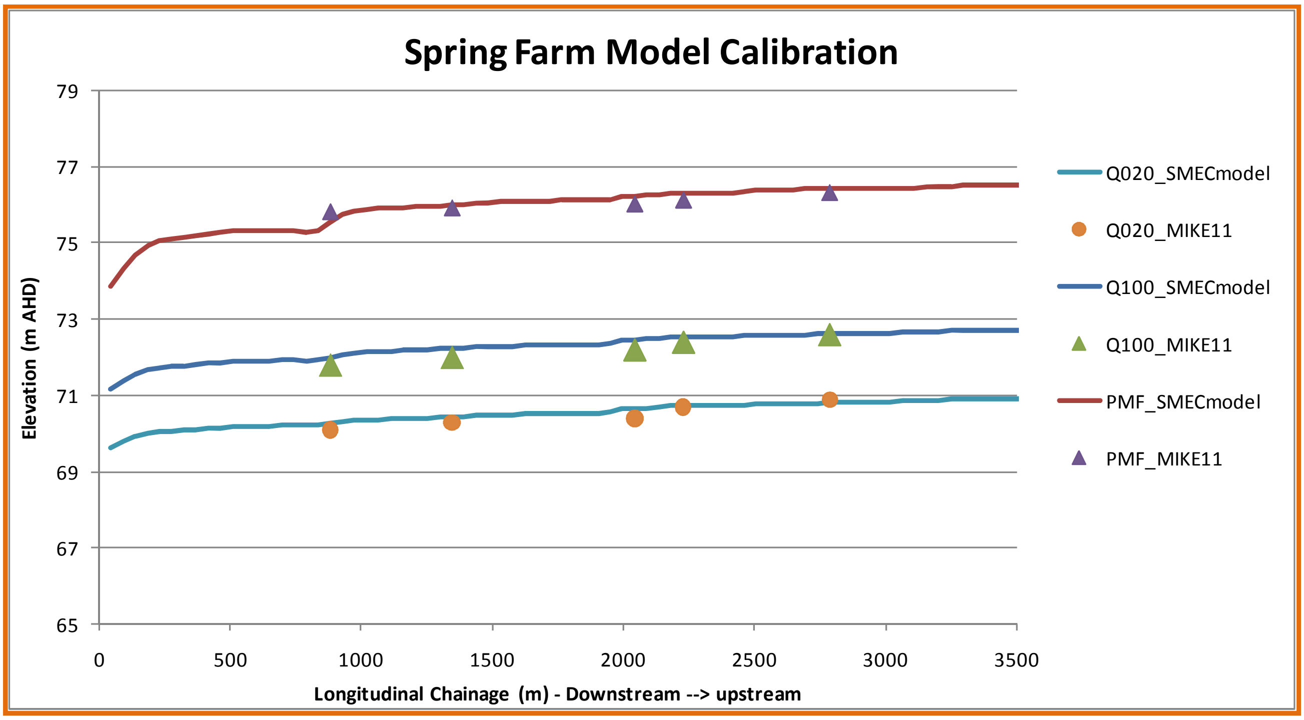


Figure 11 – TUFLOW Model Calibration against previous MIKE-11 Cross Section Locations (after 2001 SMEC Floodplain Management Study)

Calibration scenarios outlining existing conditions peak flood extent, flood contours and velocity distribution are shown in **Figure 12** (20 yr ARI), **Figure 13** (100yr ARI) and **Figure 14** (PMF) respectively.

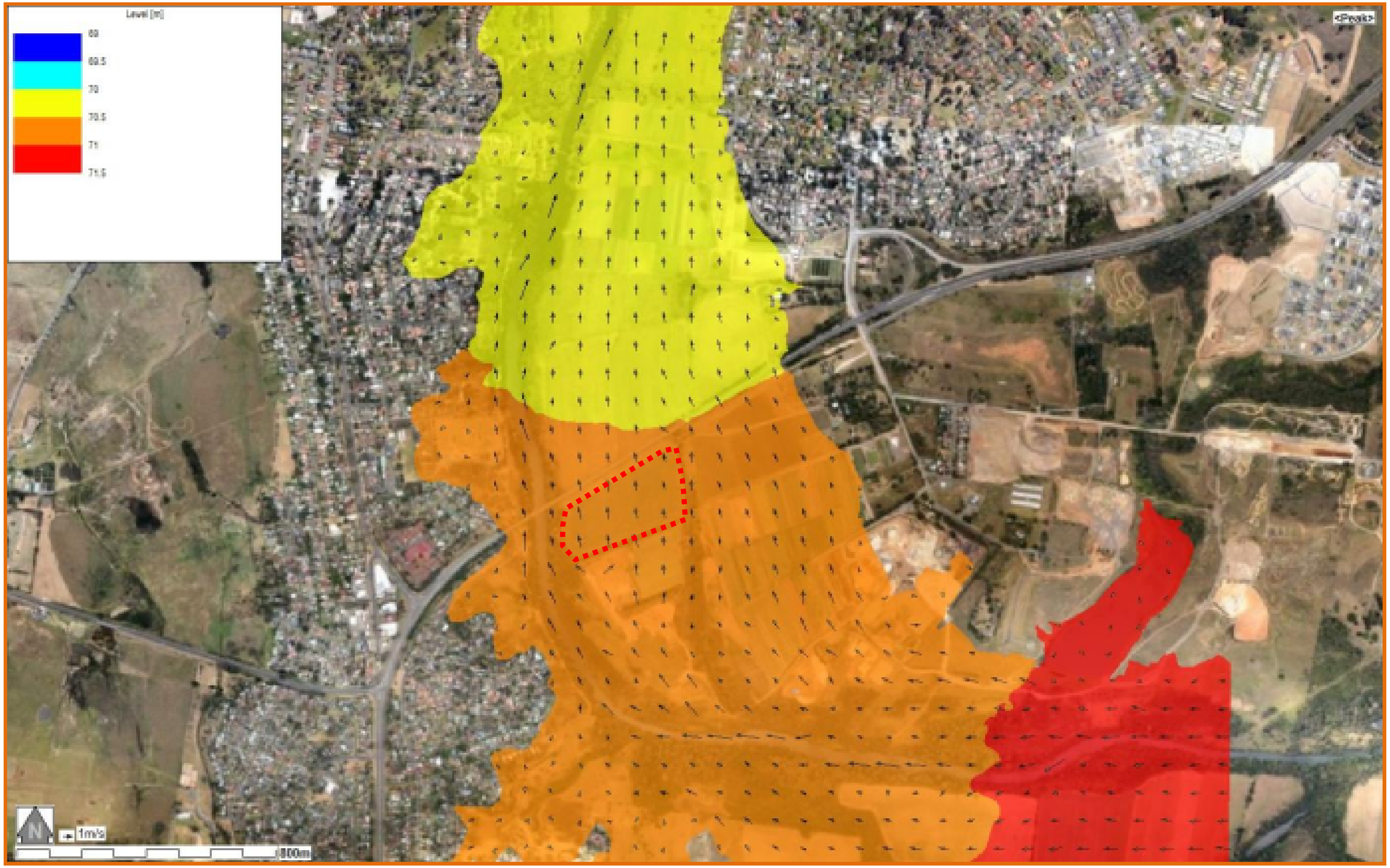


Figure 12 – Estimated existing conditions peak flood extent, flood contours and velocity distribution for 20yr ARI event.

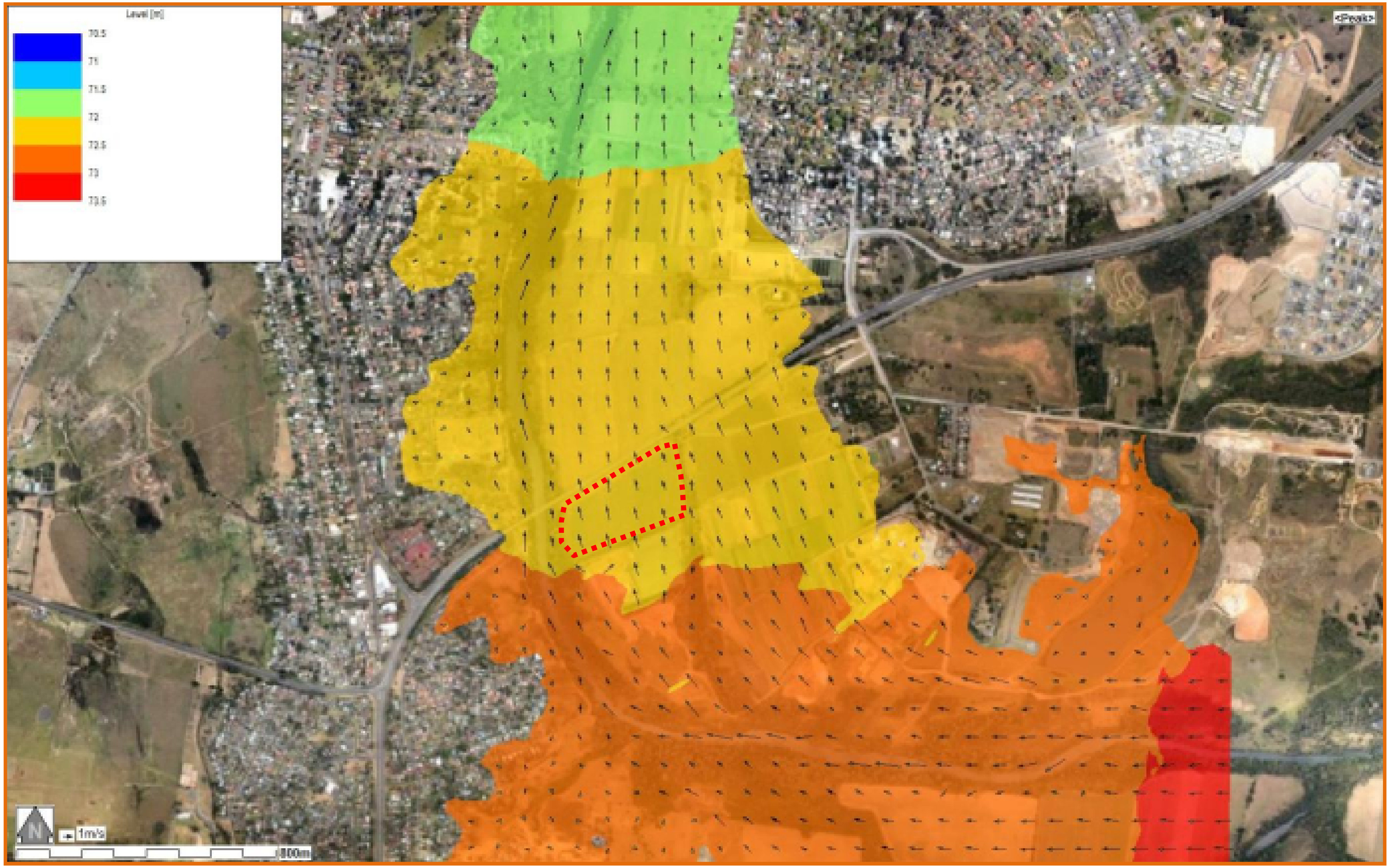


Figure 13 – Estimated existing conditions peak flood extent, flood contours and velocity distribution for 100yr ARI event.

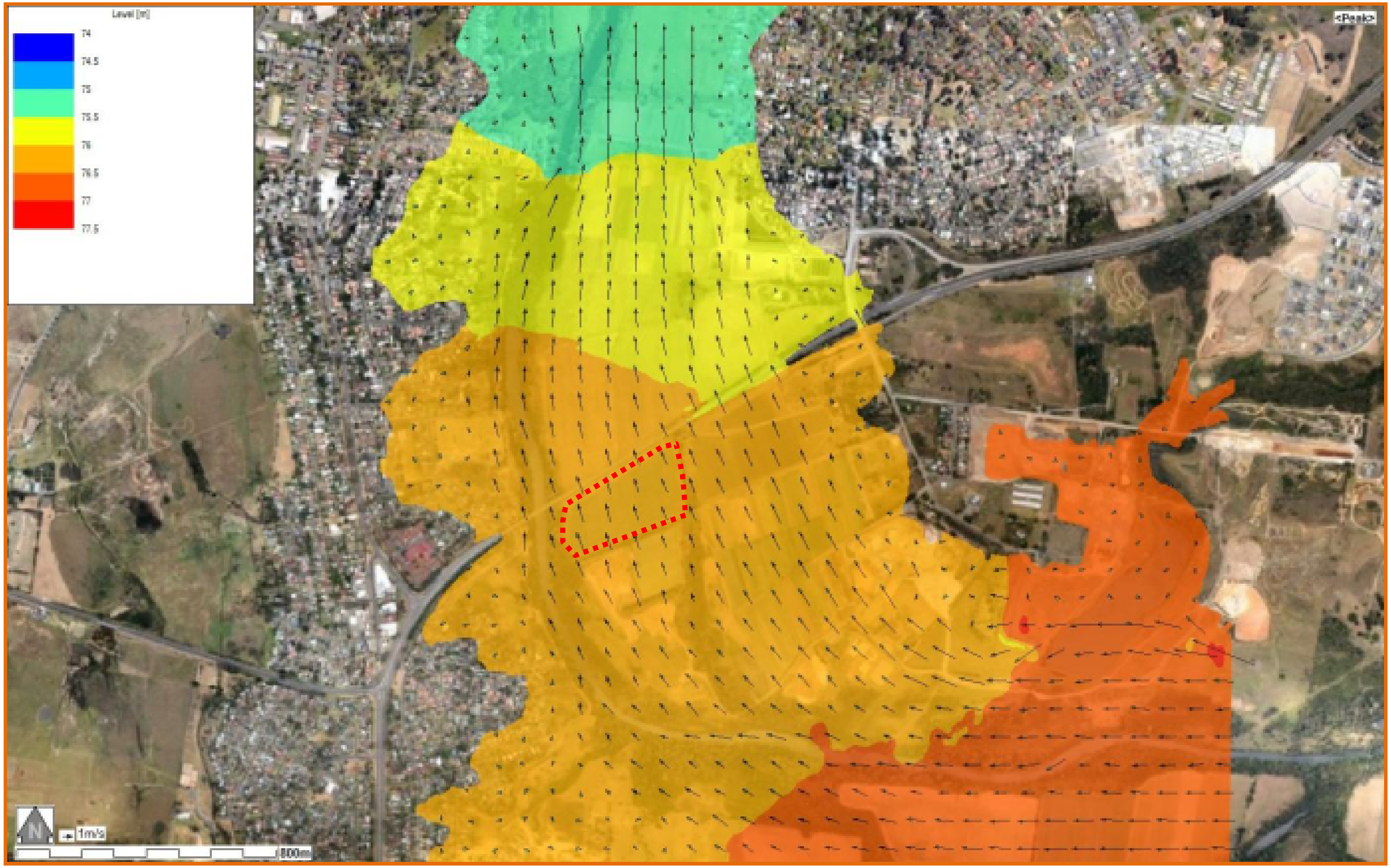


Figure 14 – Estimated existing conditions peak flood extent, flood contours and velocity distribution for Probable Maximum Flood (PMF) event.

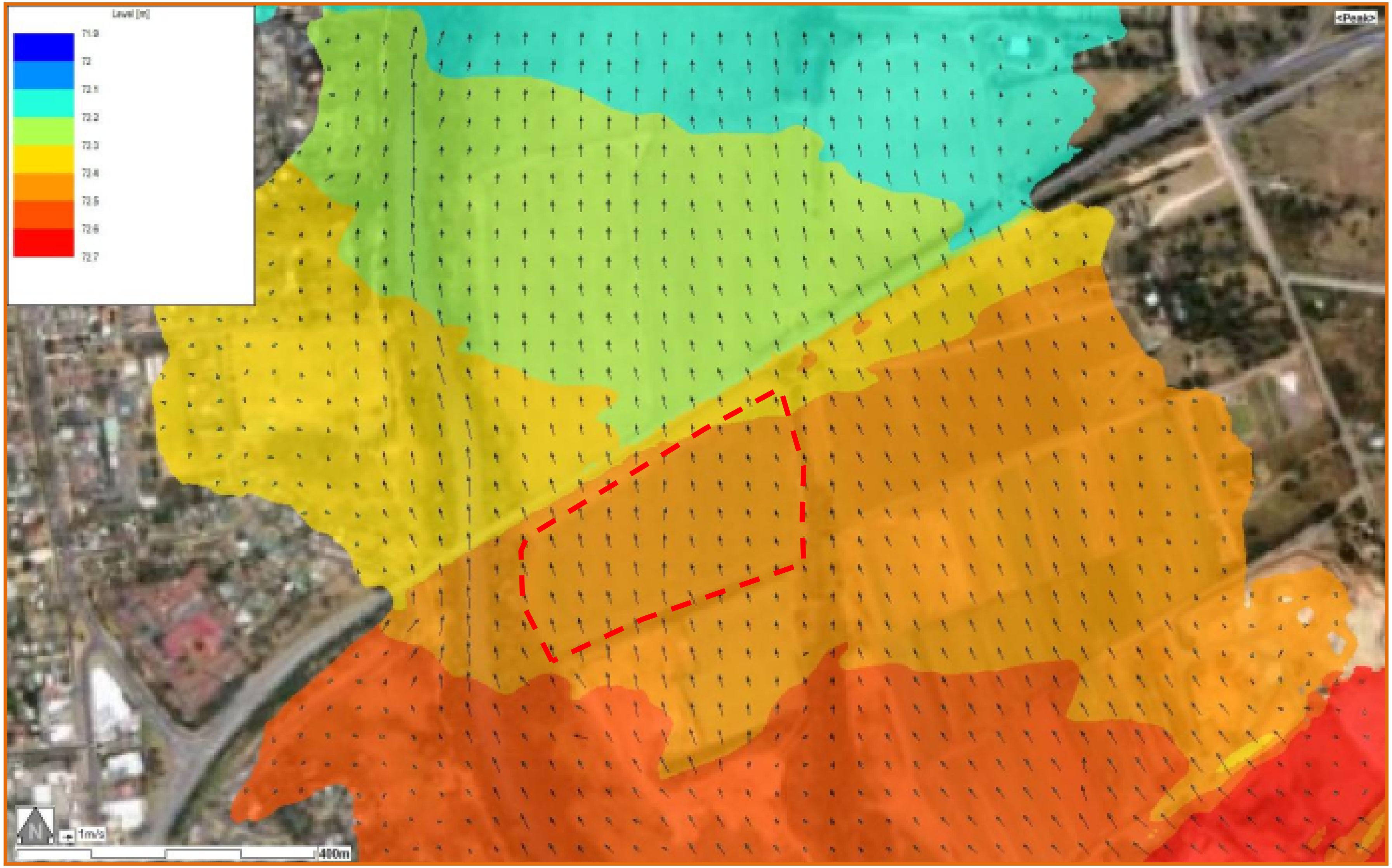


Figure 15 – Estimated existing conditions peak flood extent, flood contours and velocity distribution for 100yr ARI event – Zoomed in extent showing site details

4.5 Design Scenario Simulations

The hydraulics at the study site is reasonably complex due to the presence of the Nepean Bypass Bridge, existing sand and soil extraction operations in the adjoining land and the anabranch immediately east of the proposed site.

In order to assess the impact of the proposed development on flooding, it is typically appropriate to assess the potential impacts up to the 100 year ARI flood. In addition, we have also examined the impact on peak velocity changes potentially affecting the Camden Bypass Bridge Viaduct during the 2000 year ARI event, in accordance with the guidelines outlined in "*AUSTROADS AP-23/94 Waterway Design — A Guide to the Hydraulic Design of Bridges*".

As such, design scenario simulations for 100 year ARI and 2000 year ARI were primarily used for this study.

Note that event in a 20 yr ARI event the right hand bank of the Nepean River is subject to significant overtopping, although overbank velocities are not significant, compared to the channel velocities. During much larger flood events such as the PMF, the overbank velocities on the right hand bank approach the velocities within the main river channel.

Figure 15 above shows a detailed map outlining the existing conditions peak flood extent, flood contours and velocity distribution are for the 100yr ARI across the subject site. Note that the 100 yr ARI flood levels across the subject site are estimated to vary between approximately RL 72.4 m, AHD to 72.3 m, AHD.

Modified Geometry

For the design scenario simulations, the model geometry was modified to reflect the proposed development. The design geometry data for the proposed sand and soil mining operations, was received via email from David Morrison (Lean & Hayward) to Ben Patterson (SMEC) on 12 May, and was incorporated in the DEM to reflect the proposed development.

The comparison of existing and design DEM is provided in **Figure 2** and **Figure 3**.

4.6 Flood Impact Modelling Results

A more detailed examination has been undertaken of the velocity distribution across the subject site for a range of design storms.

The following **Figure 16** to **Figure 19** outline the results of velocities difference plots in and around the subject site for the 20, 100, 2000yr ARI and PMF events. This sequence of Figures shows that by far the worst case for relative increase in velocity is for the 20yr ARI event, and as the flood events progressively get larger, the relative change in velocity reduces substantially, until such time as extreme events (*where the site is inundated by some metres of water at the peak of the flood event*), the maximum change in velocity is negligible. This is an expected result as for larger events, when the peak water level is well above the surface features that are changing (*e.g. excavation in this instance*), the effect of the higher water levels is that the changed surface topography features are "drowned out" by the larger flood events, and therefore no significant changes in velocity are observed.

However, while the greatest change in velocity is observed in the 20 year ARI event, the absolute value of the velocities, as shown in **Figure 12** above is quite low (*i.e. velocities in the range 1.0 to 1.5 m/s*). That is to say, that while the velocities do increase in a 20 yr ARI event, they are not particularly high to start with, and probably within an acceptable range for the existing native and introduced grass species within the overbank area to withstand. Having said this, it would be important that all temporary batters are protected with erosion controls (*such as matting and spray seed hydro-mulching*) to ensure that in the event of a flood, these construction batters are not subject to undue erosion.

In conclusion, the sequence of **Figure 16** to **Figure 19** below indicates that while the 20 yr ARI event shows the greatest increase in velocity, compared to existing conditions, it is believed that the absolute velocities would not be sufficient to cause significant erosion of the exiting overbank regions. Also, during the largest PMF flood event, the velocity changes are insignificant when compared to existing conditions, and therefore this event has not been considered further. It is expected at the downstream boundary of the study area there would not be any potential for change to the natural river bank stability as a result of the proposed sand/soil extraction.

Therefore, the remainder of the focus of the flood impact assessment has been to look at the velocity differences across and around the site in the 100 yr ARI and 2000 yr ARI events. These events have been chosen because they are specified to be looked at by the AUSTROADS AP-23/94 Waterway Design — A Guide to the Hydraulic Design of Bridges.

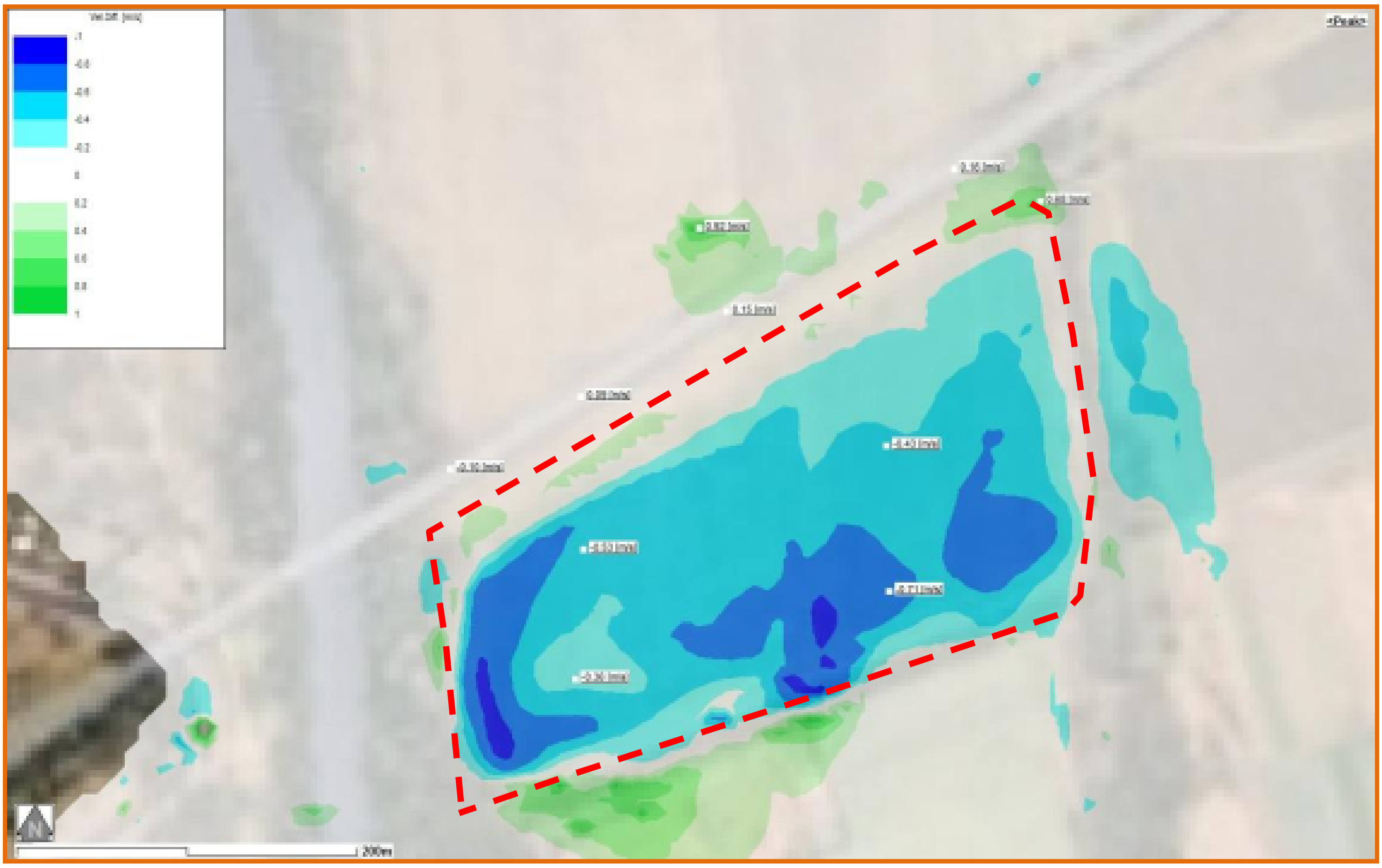


Figure 16 - 20 yr ARI Velocity Difference Plot

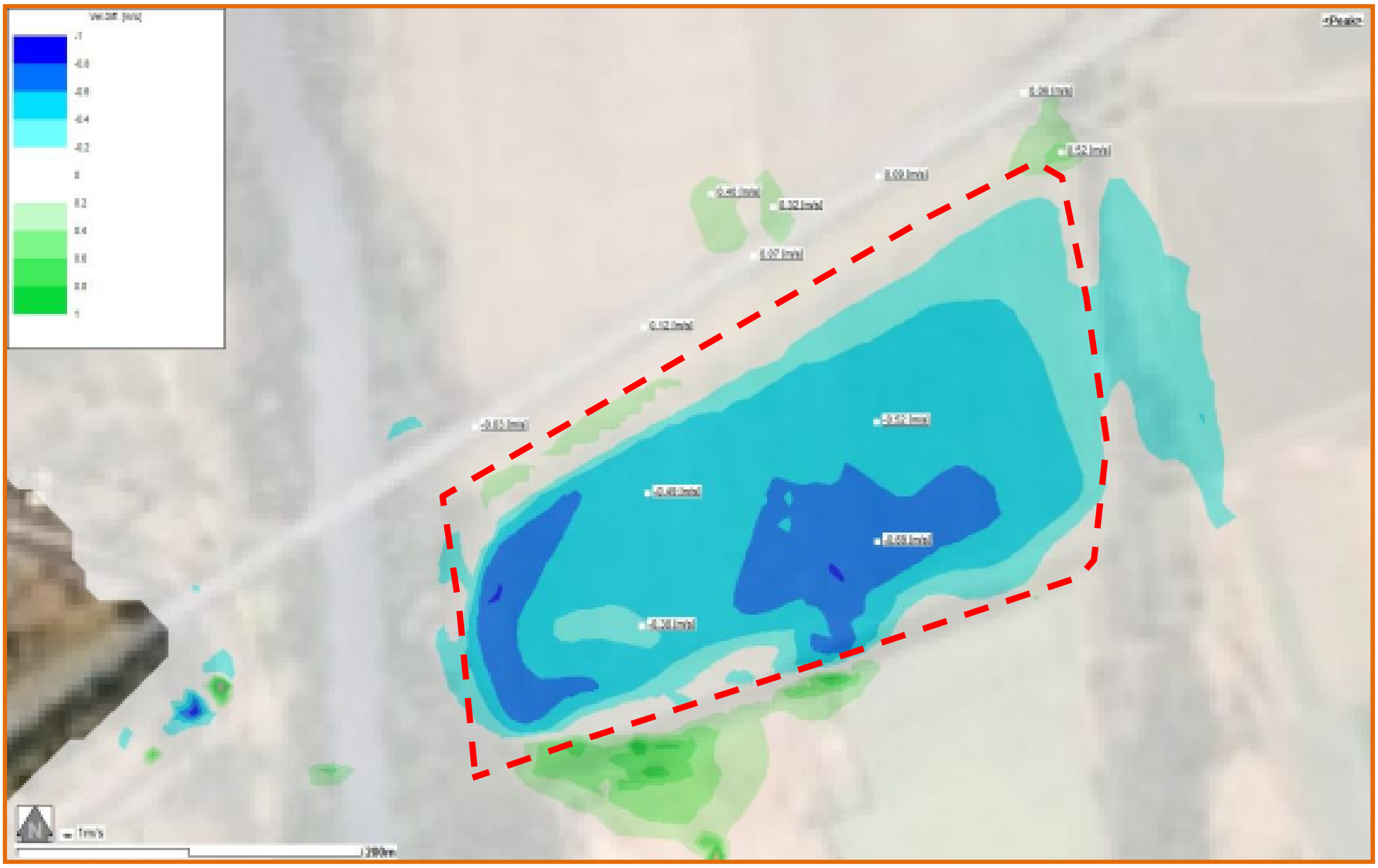


Figure 17 - 100yr ARI Velocity Difference Plot

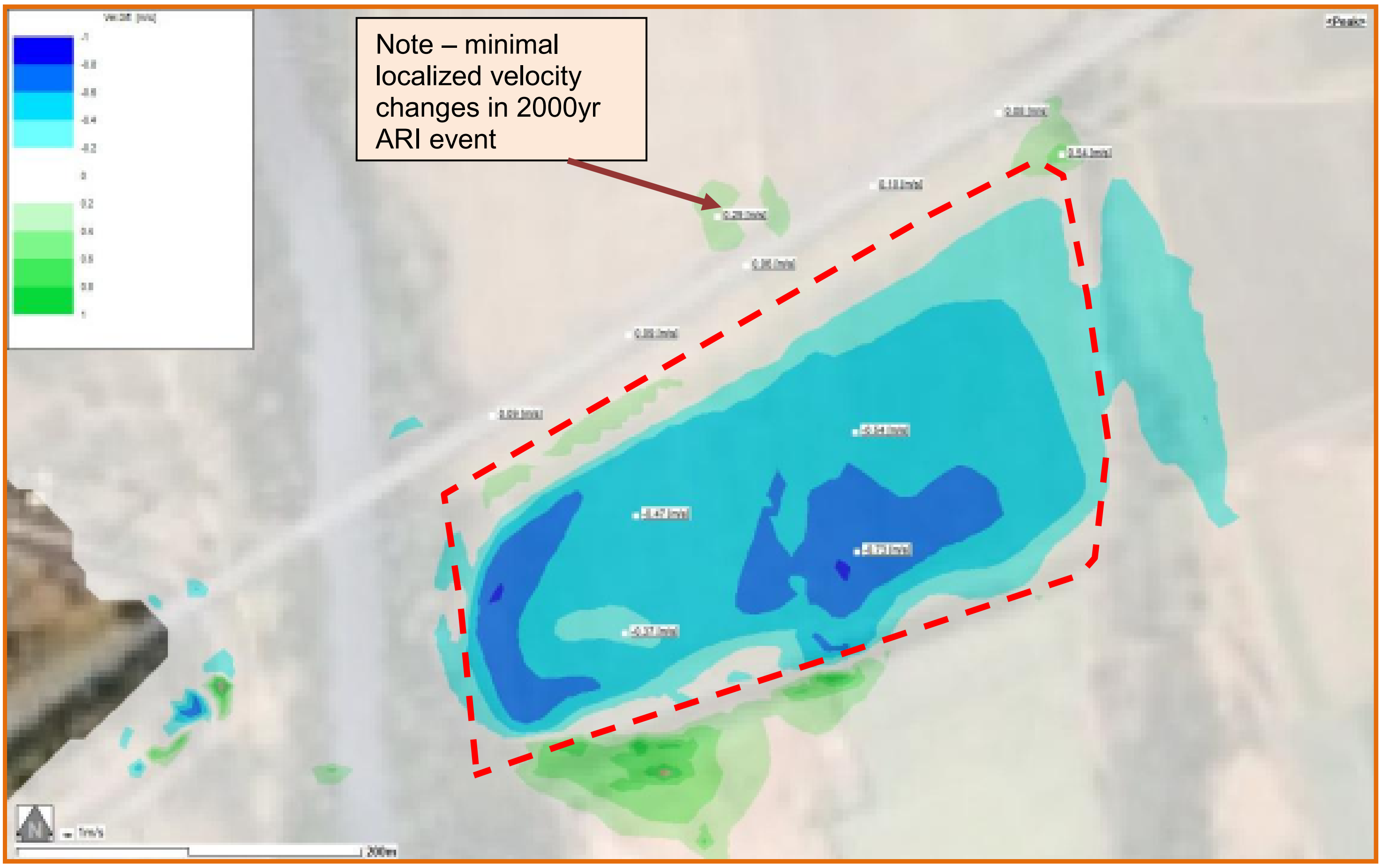


Figure 18 - 2000 yr ARI Velocity Difference Plot

Figure 19 - PMF Velocity Difference Plot

Figure 20 to **Figure 22** below show the velocity distribution across the subject site in more detail for the 100yr ARI event. As can be seen the model is estimating velocities in the range of 0.5 to 1.5 m/s across the site for existing conditions in the 100 year ARI event. Localised higher velocities (*i.e. slightly higher than 1.5 m/s*) can be seen around the Camden Bypass Bridge Viaduct piers, also under existing conditions, just downstream of the northern site boundary, as noted in **Figure 12**. This behaviour is typical of localised velocity increases around bridge piers due to eddy vortices. In addition, some slightly higher velocities can be seen around the existing spoil mound just to the south-west of the subject site. This is not a concern as the topography in this area is being locally modified by the existing extraction operations, and this spoil mound will not be present once the finished landform is complete.

Figure 23 to **Figure 25** show the velocity distribution across the subject site in more detail for the 2000yr ARI event. As can be seen the model is estimating velocities also in the range of 0.5 to 1.5 m/s across the subject site for existing conditions in the 2000 year ARI event. Localised higher velocities (*i.e. slightly higher than 1.5 m/s*) can be seen around the Camden Bypass Bridge Viaduct piers just downstream of the northern site boundary, as noted in **Figure 12**. As discussed above, this behaviour is typical of localised velocity increases around bridge piers. As with the 100 yr ARI plot, some slightly higher velocities can be seen around the existing spoil mound just to the south-west of the subject site. As discussed above, this does not pose as this is a temporary site feature.

Generally it can be seen from these more detailed Figures, as outlined above that while the overall velocities are slightly higher for the 2000yr ARI event, they are not substantially higher than the 100yr ARI event, and importantly, the velocity differential between existing and proposed cases reduces with the 2000yr ARI flood event, when compared to the 100yr ARI event.

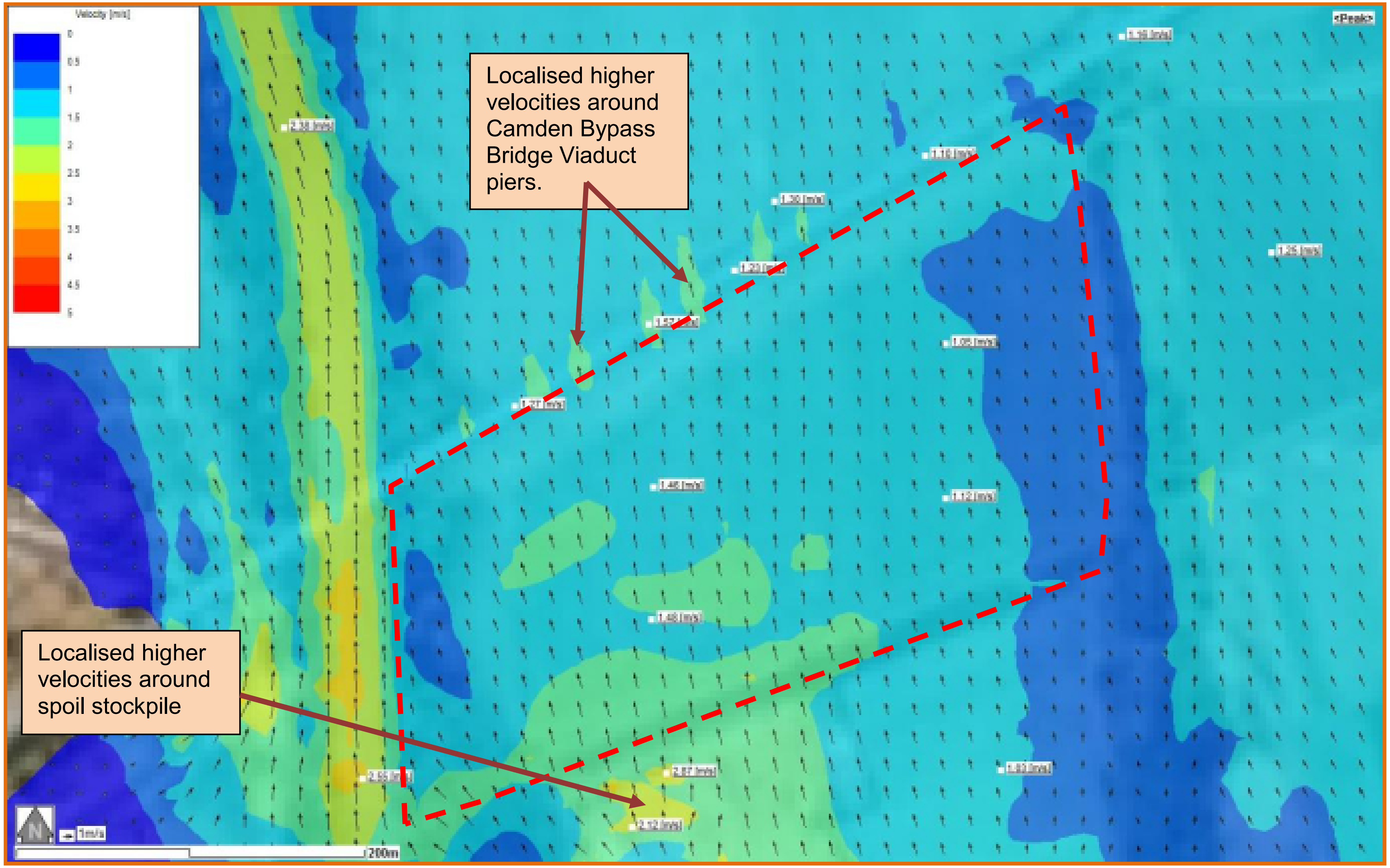


Figure 20 – Velocity Distribution across subject site for EXISTING CONDITIONS – 100yr ARI

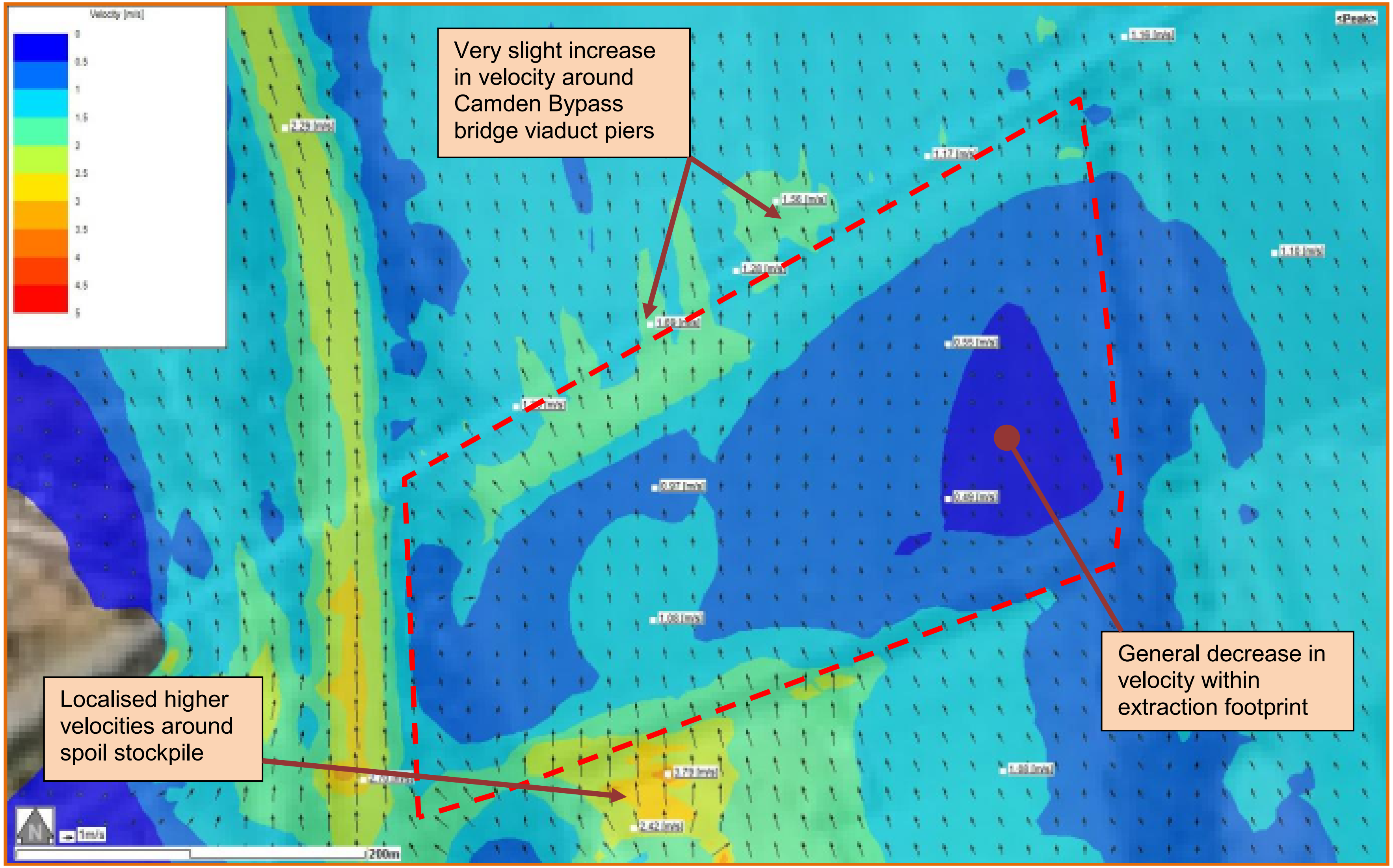


Figure 21 – Velocity Distribution across subject site for PROPOSED CONDITIONS – 100yr ARI

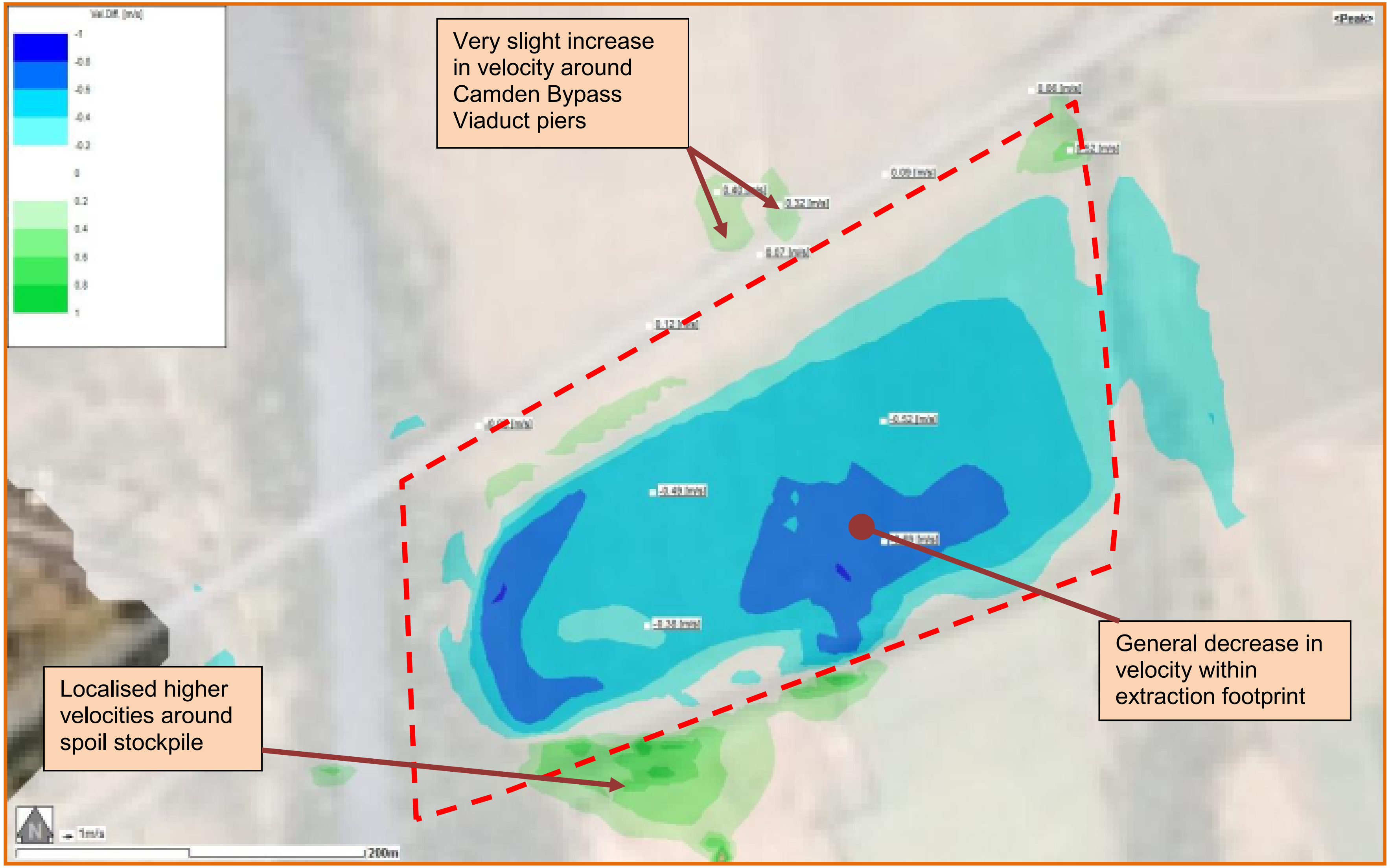


Figure 22 - 100yr ARI Velocity Difference Plot

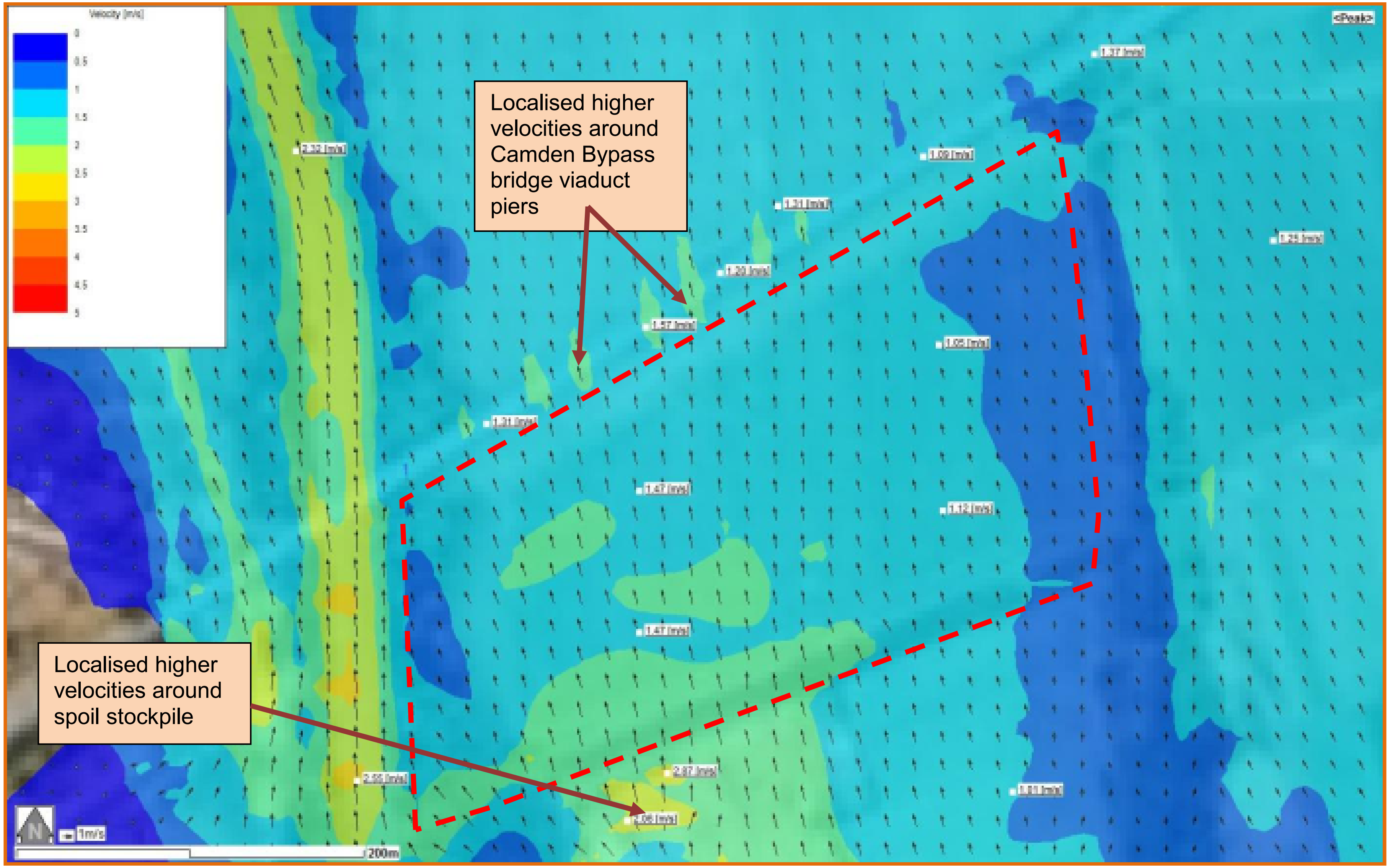


Figure 23 - Velocity Distribution across subject site for EXISTING CONDITIONS – 2000yr ARI

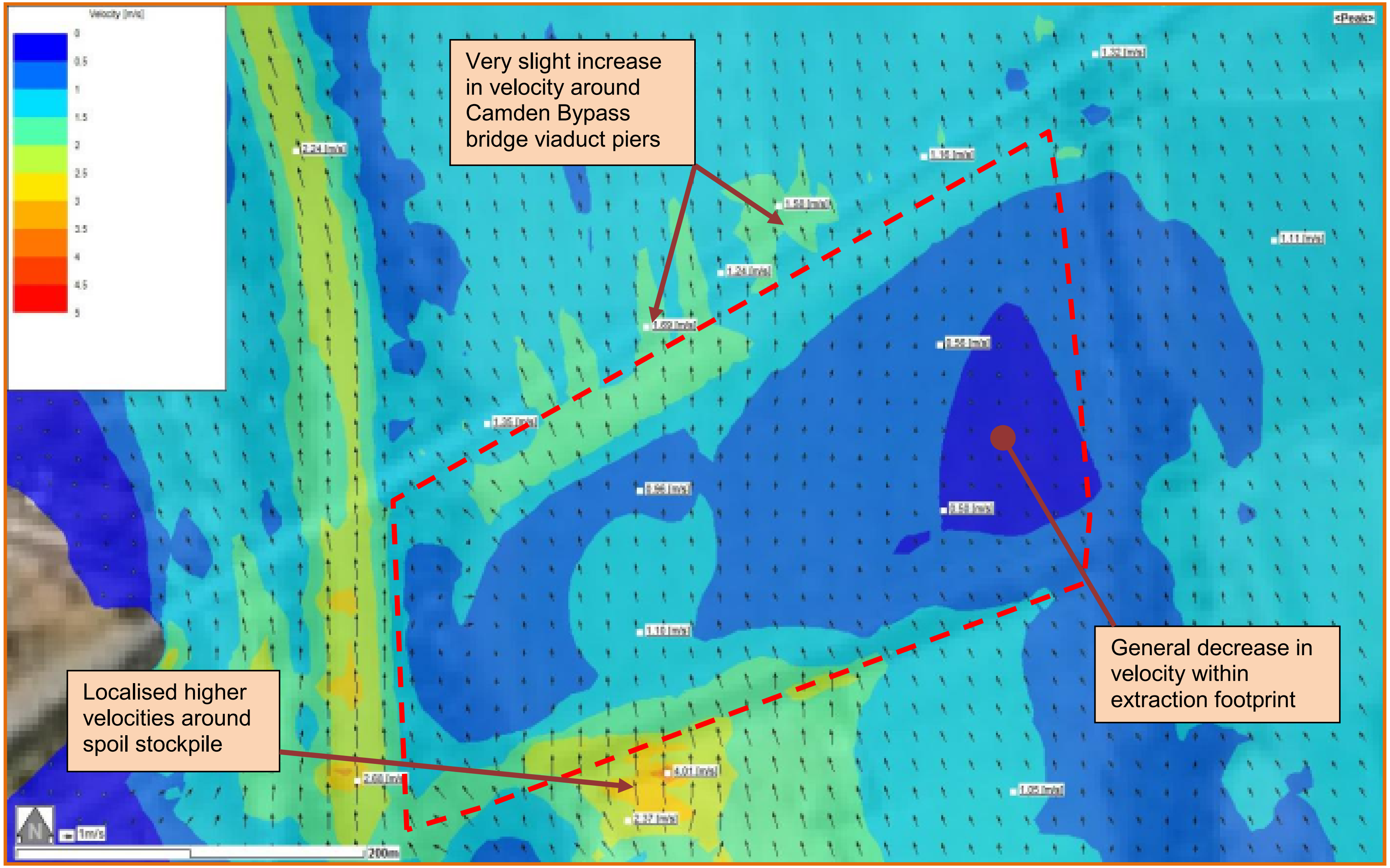


Figure 24 - Velocity Distribution across subject site for PROPOSED CONDITIONS – 2000yr ARI

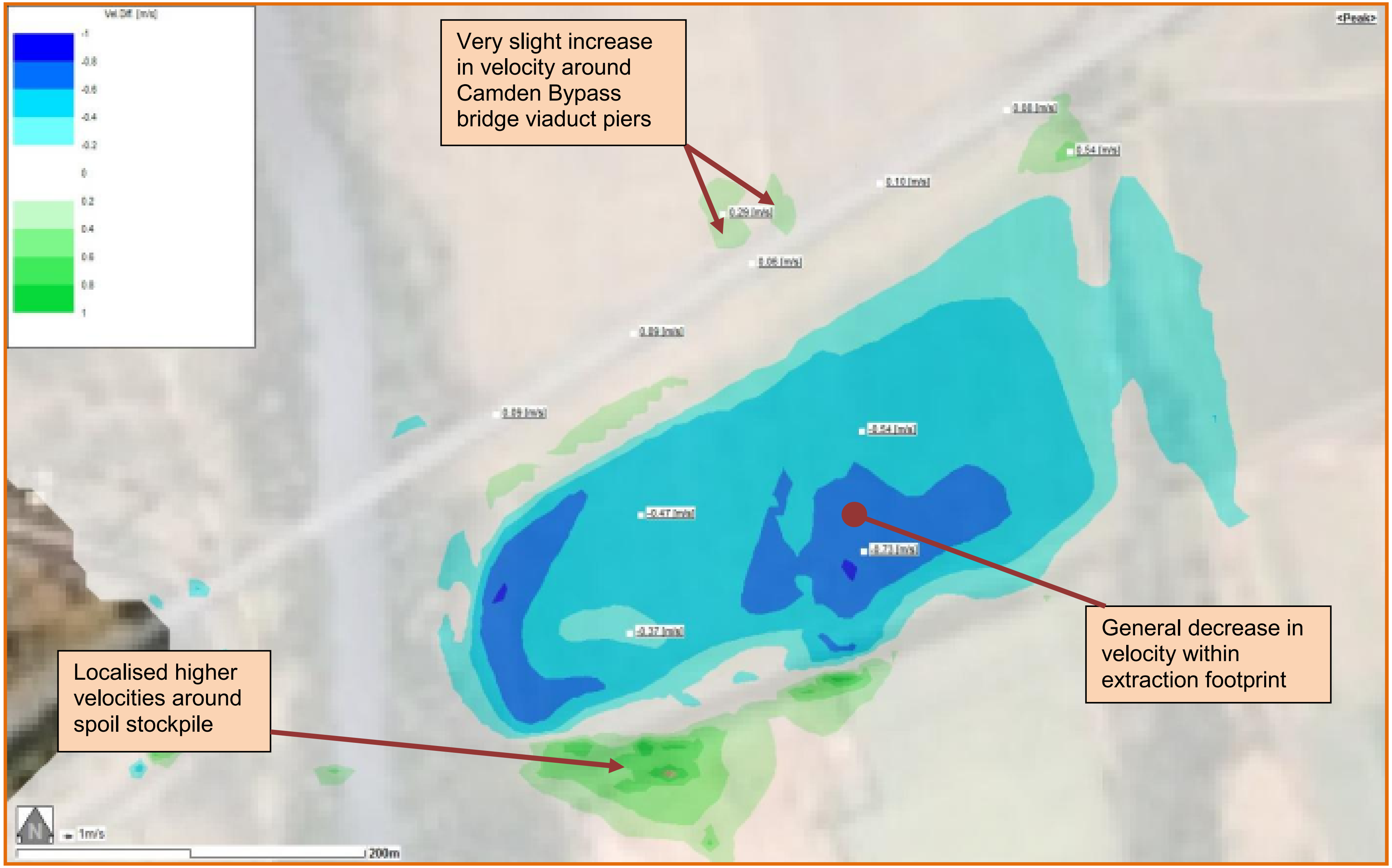


Figure 25 - 2000yr ARI Velocity Difference Plot

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Bridge Design Information

A review of the Camden Bypass Bridge Viaduct design drawings (**Appendix A**) – including the bore logs at the bridge piers (Department of Main Road for State Highway No. 2 – Bridge over the Nepean River dated August 1970 (*Drawing No. 2B2712 – Sheet 2 of 37 Sheets* - as *provided by the RTA*) have been used to understand the nature of the underlying material. Geotechnical data derived from the design plans indicates varying sands and sandy clays overlying shale bedrock at depth of between 15.6 to 21.5m below natural surface in the vicinity of the subject site.

Reference to the geotechnical bore logs indicates that sand types vary, with inter-bedded cleaner sand zones with some clayey, (*i.e. more cohesive*) sandy clay materials, which would inherently be more resistant to scour. In addition, it is evident from the design drawings that the bridge foundations are anchored into bedrock, which is at depths of between 15 to 25m below natural surface.

A summary of the depth to bedrock at various locations in relation to the subject site are given below in **Table 5**.

Location	Estimated Depth to bedrock (m)
Note there are 5 bores across the right overbank area downstream of the subject site, Depth are shown to Bedrock as follows:	
Borehole 9 – eastern edge of site adjacent to Nepean River	16.6m
Borehole 2 – approximately 120m from eastern boundary	18.2m
Borehole 3 – approximately 240m from eastern boundary	15.6m
Borehole 4 – approximately 360m from eastern boundary	21.5m
Borehole 5 – at western boundary within the anabranch channel	18.9m
Average Depth to Bedrock	**18.2m**

Table 5 – Summary of Geotechnical Conditions at the Camden Bypass Bridge Viaduct

5.2 Findings

A high level review of the potential scour at the Camden Bypass Bridge Viaduct site has been assessed against the guidelines outlined in "*AUSTROADS AP-23/94 Waterway Design — A Guide to the Hydraulic Design of Bridges*". This was undertaken by looking at the resultant changes in velocity in the 100 year ARI and 2000yr ARI flood events as a result of the proposed development, and comparing this against the existing site conditions and geotechnical data and bridge design to determine whether there is a likelihood of scour erosion at the bridge as a result of the proposed development.

The following findings have been made:

(i) For the critical 2000yr ARI event, the resultant velocity changes on the downstream side of the site are of the order of increase of 0.1 m/s and up to 0.3 m/s over a very localized area of the floodplain on the downstream side of the site, near the bridge.

These results can be seen outlined in **Figure 25**. It is also apparent with the proposed additional vegetation being planted on the natural river bank, the impact of the sand/soil extraction will have a negligible effect on the bank's stability at the site and downstream from the site.

(ii) The net overall 2000yr ARI velocity distribution post development is estimated by TUFLOW to be of the order of between 1.2 m/s to 1.7 m/s on the downstream side of the site in the vicinity of the existing bridge as outlined in **Figure 24**. This is compared to an existing 2000yr ARI velocity distribution of between 1.1 m/s to 1.6m/s, as outlined in **Figure 23.** Note that the velocity increases are typically of the order of only 0.1 m/s, and that the total highest velocities do <u>not</u> exceed 2 m/s at any point near the bridge in the vicinity of the site.

(iii) It is our understanding that the alluvial sand banks on the Nepean River floodplain are relatively stable and have been accumulating over some time, (*hence the resource of sand being extracted*). As outlined in the Photo 4 (in Section 2), the existing right overbank region of the river underneath the bridge is characterised by a well established good cover of grasses and turf and therefore this area is not expected to be subject to significant scour for events where velocities do not exceed 2 m/s (CIRIA Report 116 – Figure 9 – refer **Appendix B**).

(iv) The bridge design drawings provided by the RTA also indicate that the bridge has been designed using anchor piers that are piered to bedrock, such that the bridge is likely designed to withstand the ultimate scour of the majority of overlying sand alluvial deposits.

(v) AUSTROADS AP-23/94 Waterway Design — A Guide to the Hydraulic Design of Bridges does not recommend the use of scour protection when velocities are less than 2 m/s.

Therefore, the expected impacts of the proposed development are not likely to have any significant impact on the potential for scour at the existing Camden Bypass Bridge Viaduct. However, some recommendations are made below in **Section 5.3** , to ensure that erosion does not become an issue that could affect the bridge or other adjacent infrastructure.

It should be noted that the above assessment is very conservative, given the following points:

(i) The TUFLOW modelling has assumed that the entire pit has been excavated as one large opening, rather than as a progressive excavation, with material being returned to the pit once sand extraction has taken place; and

(ii) The TUFLOW modelling estimate velocity increases during a 2000yr ARI event only over very localised area immediately to the north of the extraction area, as shown in **Figure 22** and **Figure 25**).

Therefore, it is considered that the effects of the sand extraction on scour potential affecting the stability of the Camden Bypass Bridge Viaduct is likely to be negligible.

In addition, the flood modelling has indicated that the proposal will not result in any measurable change in flood levels in the immediate locality, or elsewhere on the floodplain.

5.3 Recommendations

The following recommendations are made for the proposed development:

1. The assessment has shown that the area most prone to erosion is area to the north of the downstream northern face of the excavation pit. It is therefore recommended that the face of the northern batter be kept vegetated and protected from erosion using a combination of erosion protection blanket, and appropriate native grass vegetation.

2. In addition, the area between the top of bank of the northern construction batter and the northern boundary with the Camden Bypass Bridge Viaduct road reserve should be protected with full densely planted riparian vegetation. These measures will prevent any potential scour from undermining the existing turf vegetation mat that exists beneath the bridge viaduct, which would be critical to maintaining an erosion resistance to scour on the right overbank area of the Nepean River.

3. A monitoring programme should be established to ensure that the following areas are maintained with good vegetation cover, and to ensure that erosion is not occurring:

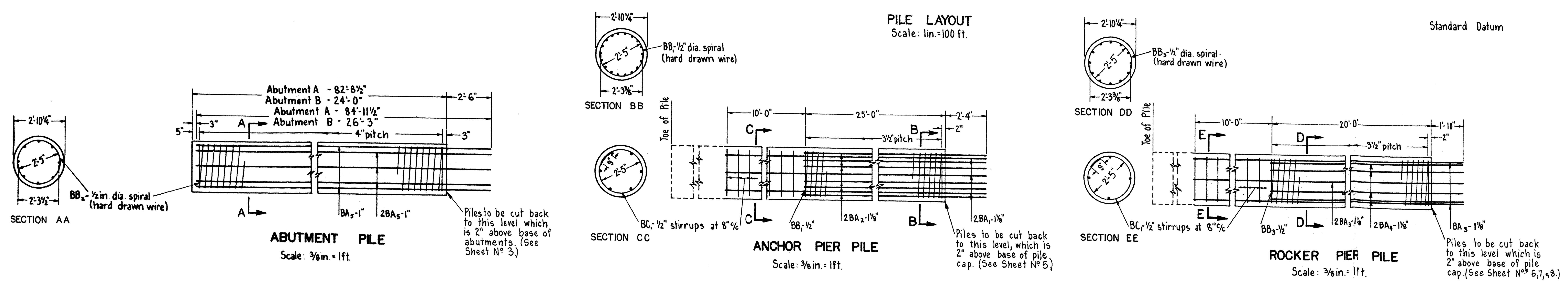
    (i) the banks of the excavation pit;

    (ii) the buffer zone between the pit and the river; and

    (iii) the buffer zone between the northern excavation batter and the road reserve boundary.

APPENDIX A – CAMDEN BYPASS BRIDGE VIADUCT DESIGN DRAWINGS

Details of Vertical Curve : Length 4000 ft.
on Pegged ℄ I.P. Ch.3530·0, R.L.235·78
Approach grades -1·27% from Sydney, +1·34% to Picton. (Bridge on sag V.C.)
26 at 130'-0" = 3380'-0"
From Sydney
Deck level R.L.257·50
Estimated level of highest recorded flood based on 1873 reading at Camden Bridge - R.L. 239·5.
Deck level R.L.248·83
Normal W.L. R.L.182·8
Deck level R.L.258·80
To Picton
R.L. 50·00
Natural surface level on ℄ R.L.
Chainage
ELEVATION
Scale : 100 ft. = 1 in.
E - denotes expansion bearing.
FR - denotes fixed bearing allowing rotation only.
Pegged ℄ bearing 231°34'35"
Nameplate
Carriageway ℄
Flood Channel
Ox Bow (normally dry)
Nepean River
B.M. 200 ft. left of Ch. 4950·00, R.L.198·41.
PLAN
Scale : 100 ft. = 1 in.
All levels to Standard Datum.
The bridge contract does not include the construction of the cast-in-place piles, the construction of the approaches, the provision of the lamps and wiring, or the bituminous pavement.
The bridge contract does include the preparation of the cast-in-place piles for the construction of the pile caps and the construction of the abutment approach slabs.
The approaches to the bridge form part of a deviation of the Hume Highway as shown on the Site Plan.
See Sheet Nº 2 for foundation bore details and pile layout plan.
38'-8"
30'-0"
Carriageway ℄
Asphaltic concrete.
Upstream
Downstream
8'-7"
9'-5"
18'-0"
CROSS SECTION
Scale: 1/8 in. = 1 ft.
Camden is 38 miles from Sydney and 8 miles from Campbelltown by road. Campbelltown is 32 miles by rail from Sydney.
SITE PLAN
Scale: 4½ miles = 1 inch.
Pegged ℄ bearing 231° 34' 35"
Carriageway ℄
NEW BRIDGE
2200' R. curve to existing Highway.
To existing Bridge
LOCALITY SKETCH
Scale: 400 ft. = 1 in.
0002 075BC2712
Approved: Engineer-in-Chief Date: 15-9-70
DEPARTMENT OF MAIN ROADS, N.S.W.
STATE HIGHWAY Nº 2 MUNICIPALITY OF CAMDEN
BRIDGE OVER NEPEAN RIVER
AT CAMDEN
PLAN AND ELEVATION
REGⁿ Nº OF PLANS 2B2712
DESIGN LOADING D.M.R. 1957
ASST. DESIGNING ENGINEER FOR BRIDGES
DESIGNING ENGINEER FOR BRIDGES
SHEET Nº 1 Nº OF SHEETS 37
CHIEF ENGINEER (BRIDGES) 15. 9. 70. DATE
MICROBOX



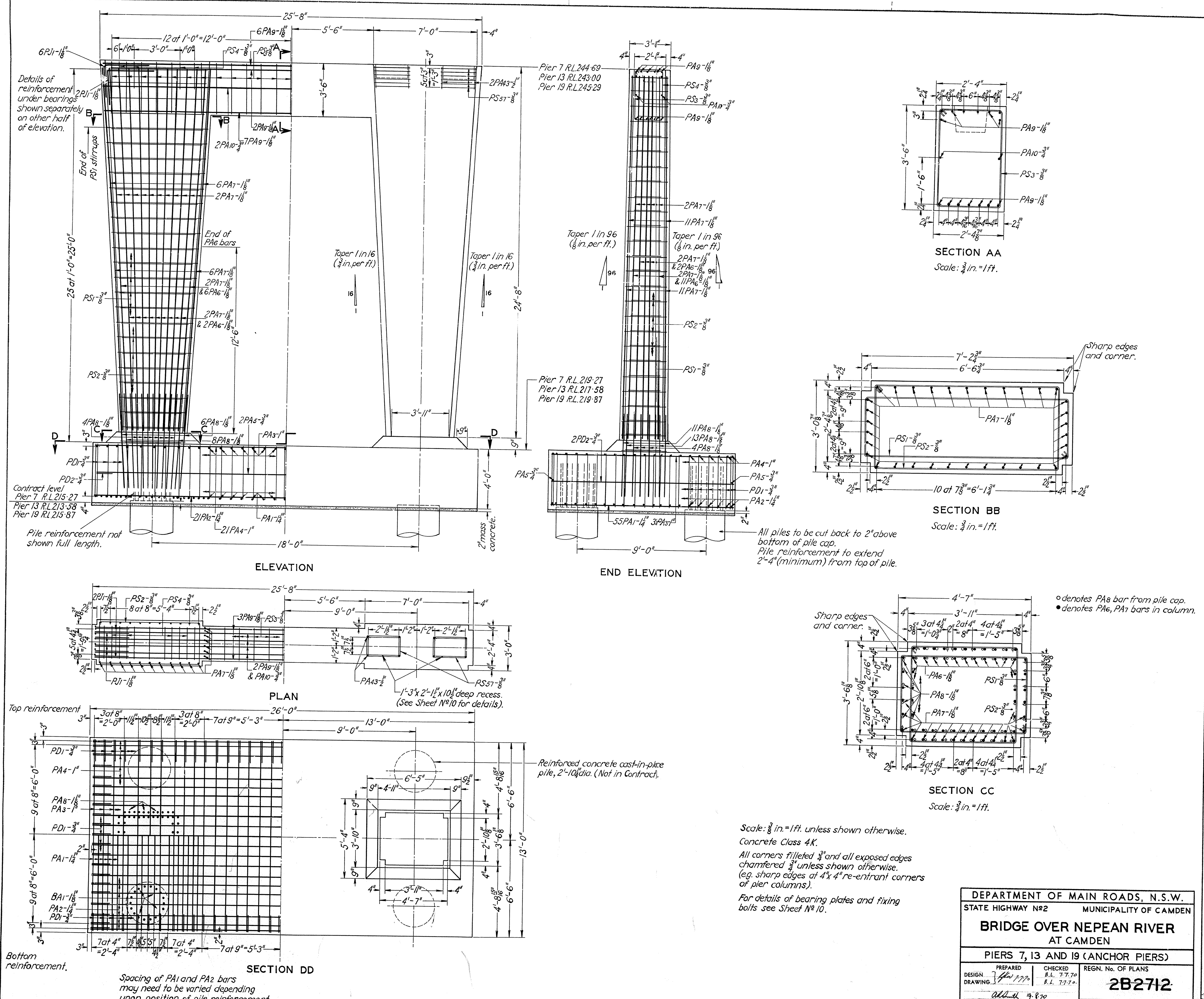
0002 075BC2712

Cast-in-place reinforced concrete piles have already been constructed. The Contract includes the cutting-back of these piles and stripping the reinforcement.

Details of the piles as constructed are given in the Specification.

DEPARTMENT OF MAIN ROADS, N.S.W.	
STATE HIGHWAY Nº 2	MUNICIPALITY OF CAMDEN
BRIDGE OVER NEPEAN RIVER AT CAMDEN	
DETAIL OF BORES AND PILE LAYOUT	
PREPARED / CHECKED DESIGN: S.L.B. 8 [signature] 11/9/69 S.L.B. & [signature] DRAWING: E. W. 1/7/70 [signature] 1/7/70 [signature] 17.8.70 ASST. DESIGNING ENGINEER FOR BRIDGES	REGN Nº OF PLANS 2B2712
[signature] 7.9.70 DESIGNING ENGINEER FOR BRIDGES	SHEET Nº 2 — Nº OF SHEETS 37

MICROBOX



Scale: 3/8 in.=1 ft. unless shown otherwise.

Concrete Class 4K.

All corners filleted 3/4" and all exposed edges chamfered 3/4" unless shown otherwise. (eg. sharp edges at 4"x 4" re-entrant corners of pier columns).

For details of bearing plates and fixing bolts see Sheet Nº 10.

DEPARTMENT OF MAIN ROADS, N.S.W.

STATE HIGHWAY Nº2 — MUNICIPALITY OF CAMDEN

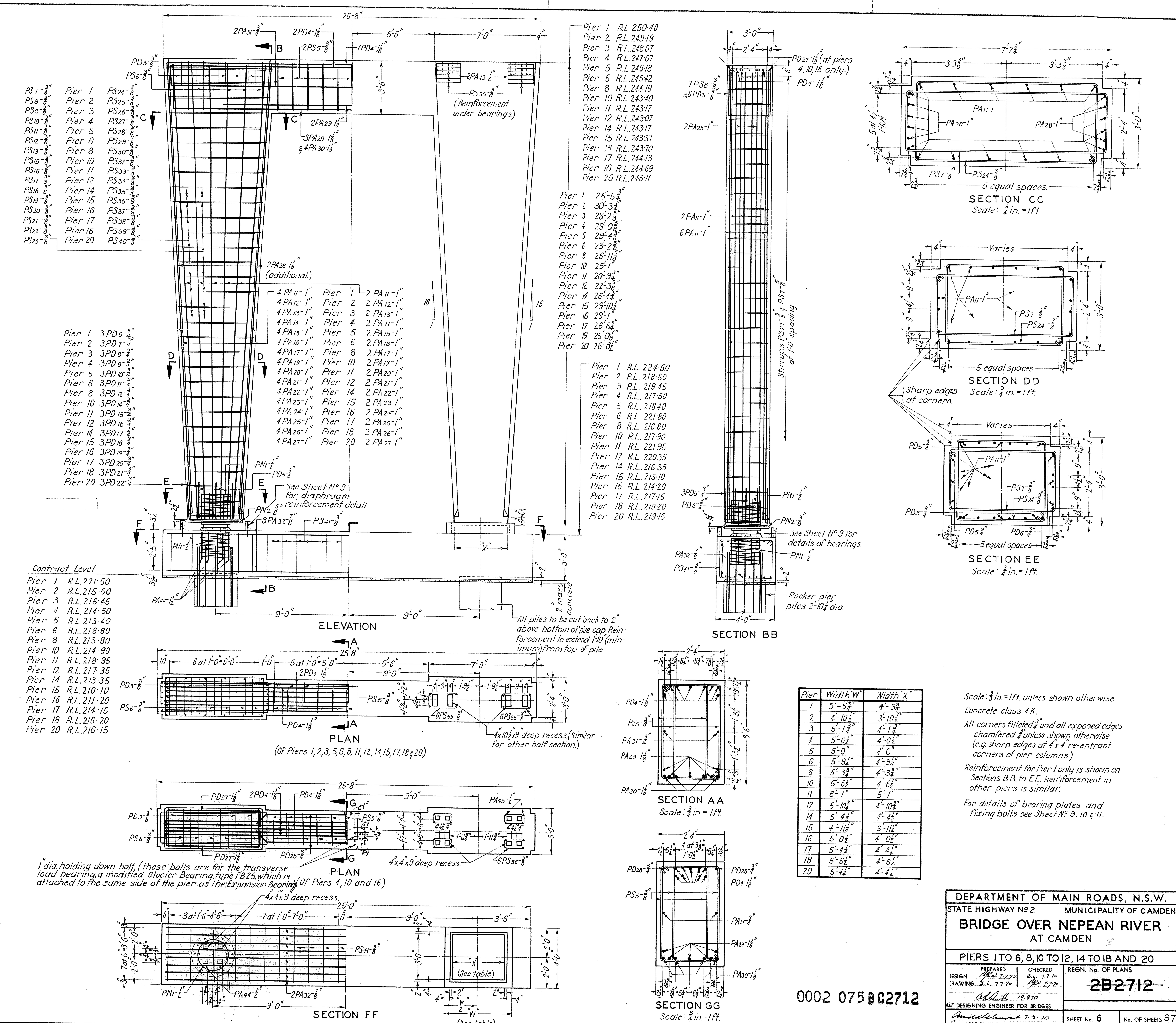
BRIDGE OVER NEPEAN RIVER

AT CAMDEN

PIERS 7, 13 AND 19 (ANCHOR PIERS)

PREPARED	CHECKED	REGN. No. OF PLANS
DESIGN / DRAWING 1.7.70	B.L. 7.7.70 / B.L. 7.7.70	2B2712
19.8.70 A/g DESIGNING ENGINEER FOR BRIDGES		
7.9.70 For ASSOCIATE BRIDGE ENGINEER	SHEET No. 5	No. OF SHEETS 37

0002 075 B02712



Pier	Width "W"	Width "X"
1	5'-5 3/4"	4'-5 3/4
2	4'-10 1/2"	3'-10 1/2"
3	5'-1 3/4"	4'-1 3/4"
4	5'-0 1/2"	4'-0 1/2"
5	5'-0"	4'-0"
6	5'-9 1/4"	4'-9 1/4"
8	5'-3 3/4"	4'-3 3/4"
10	5'-6 1/2"	4'-6 1/2"
11	6'-1"	5'-1"
12	5'-10 3/4"	4'-10 3/4"
14	5'-4 1/2"	4'-4 1/2"
15	4'-11 1/4"	3'-11 1/4"
16	5'-0 1/2"	4'-0 1/2"
17	5'-4 1/4"	4'-4 1/4"
18	5'-6 1/2"	4'-6 1/2"
20	5'-4 1/4"	4'-4 1/4"

Scale: 3/8 in. = 1 ft. unless shown otherwise.

Concrete class 4K.

All corners filleted 3/4" and all exposed edges chamfered 3/4" unless shown otherwise (e.g. sharp edges at 4"x4" re-entrant corners of pier columns.)

Reinforcement for Pier 1 only is shown on Sections B.B. to E.E. Reinforcement in other piers is similar.

For details of bearing plates and fixing bolts see Sheet No 9, 10 & 11.

0002 075 B 02712

DEPARTMENT OF MAIN ROADS, N.S.W.

STATE HIGHWAY No 2 — MUNICIPALITY OF CAMDEN

BRIDGE OVER NEPEAN RIVER AT CAMDEN

PIERS 1 TO 6, 8, 10 TO 12, 14 TO 18 AND 20

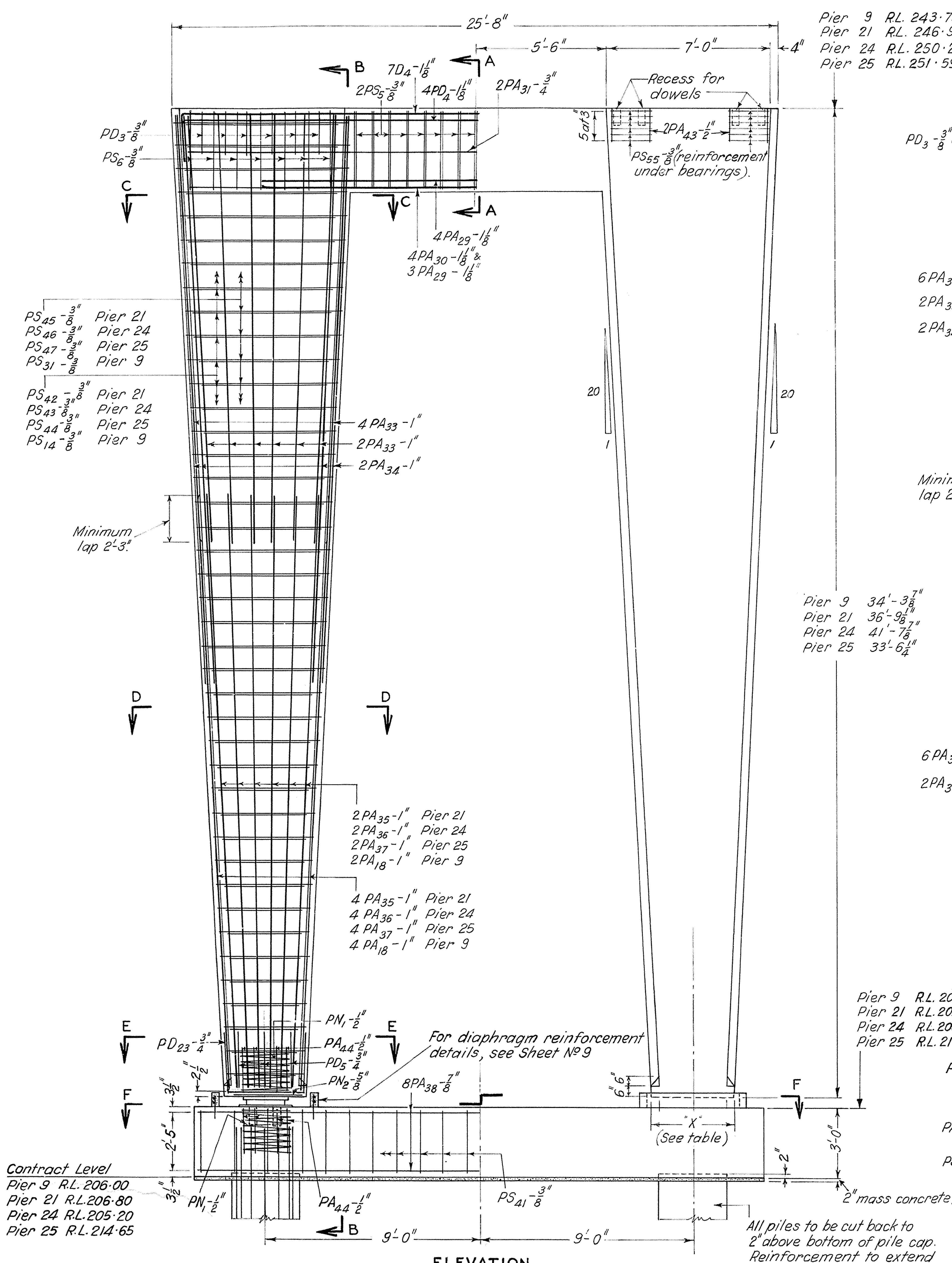
	PREPARED	CHECKED
DESIGN	7.7.70	B.L. 7.7.70
DRAWING	B.L. 7.7.70	7.7.70

19.8.70
ASST. DESIGNING ENGINEER FOR BRIDGES

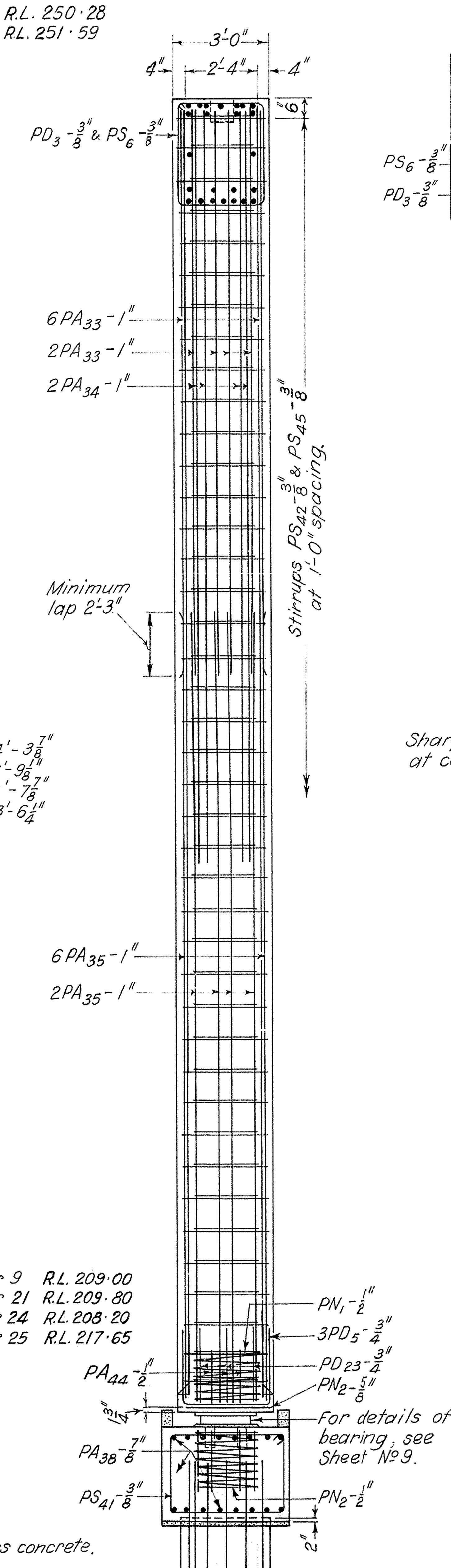
7-9-70
For ASSOCIATE BRIDGE ENGINEER

REGN. No. OF PLANS 2B2712

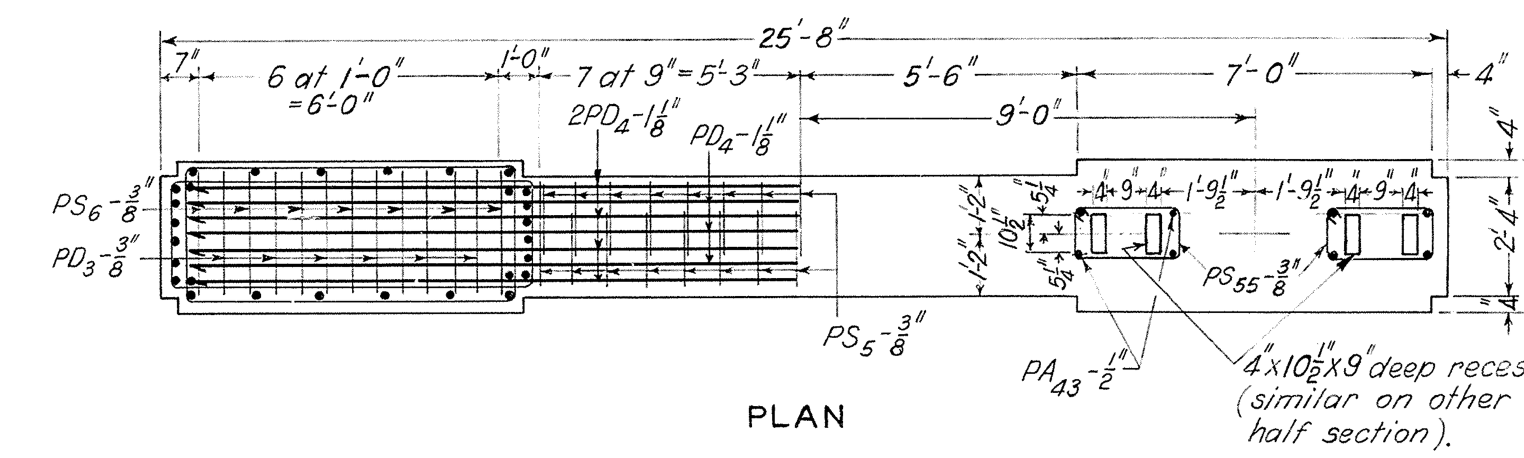
SHEET No. 6 — No. OF SHEETS 37



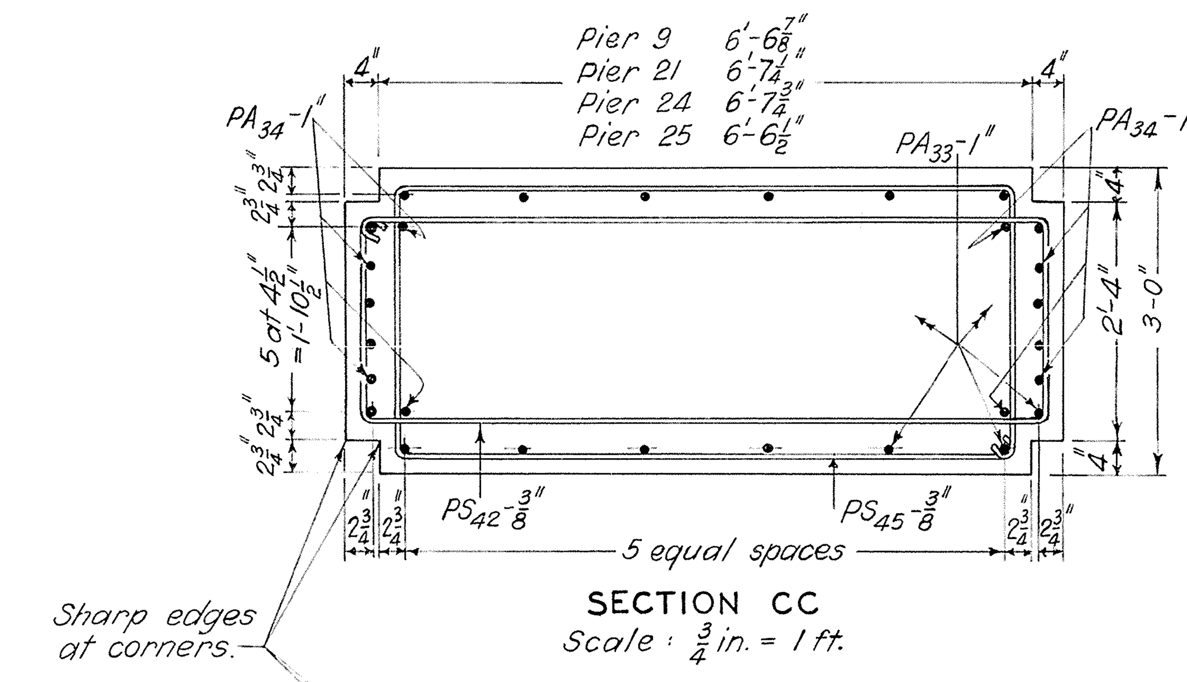
ELEVATION



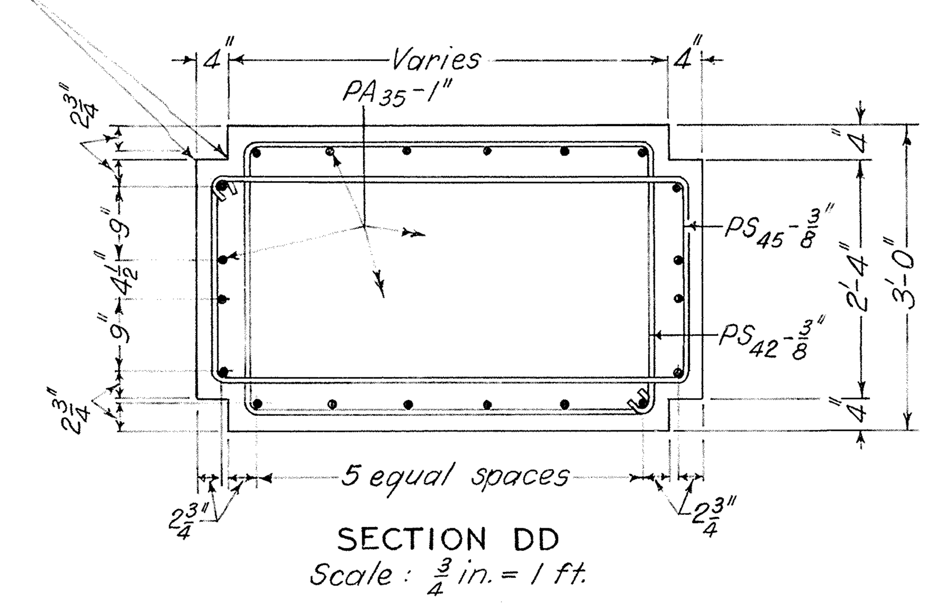
SECTION BB



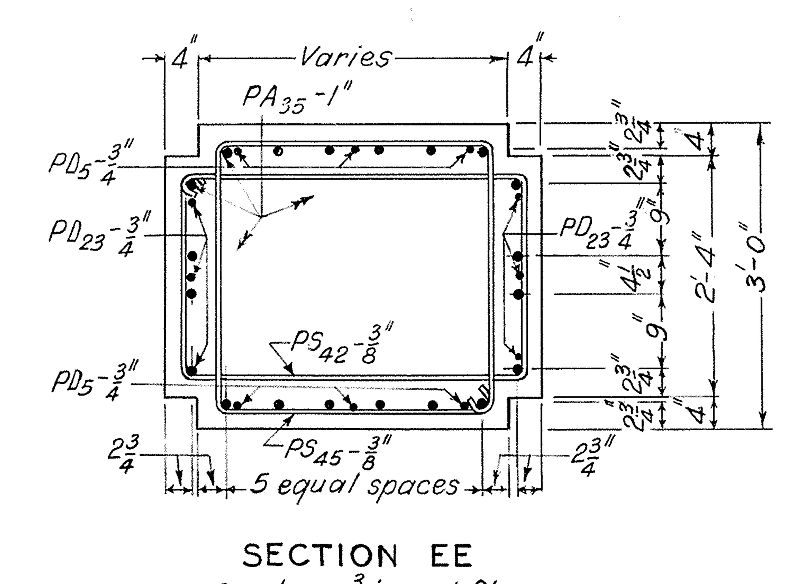
PLAN



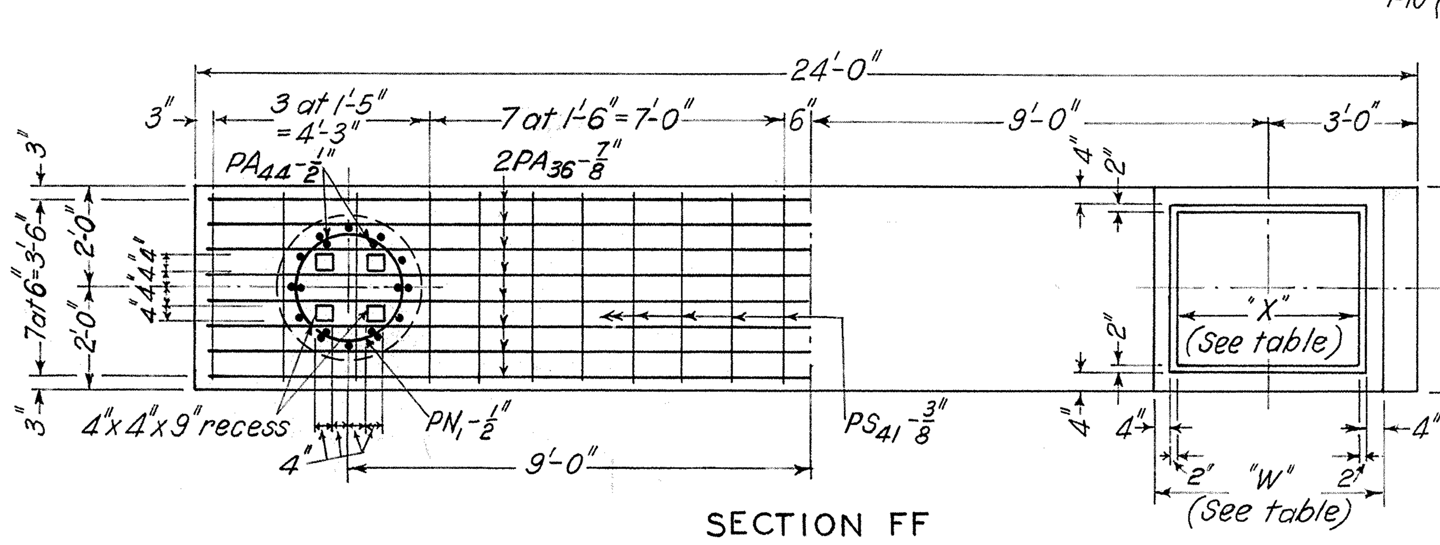
SECTION CC
Scale : ¾ in. = 1 ft.



SECTION DD
Scale : ¾ in. = 1 ft.

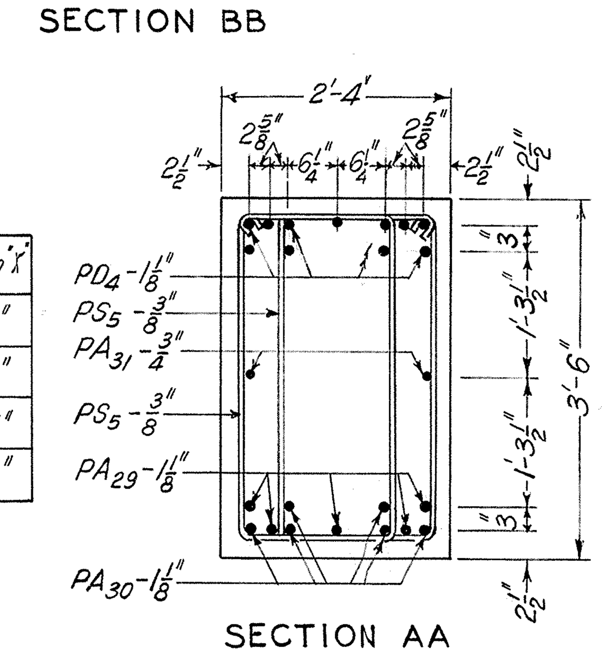


SECTION EE
Scale : ¾ in. = 1 ft.



SECTION FF

Pier	Width "W"	Width "X"
9	5'-3"	4'-3"
21	5'-0"	4'-0"
24	4'-6"	3'-6"
25	5'-4"	4'-4"



SECTION AA
Scale : ¾ in. = 1 ft.

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Scale : ⅜ in. = 1 ft. unless shown otherwise
Concrete Class 4K
All corners filleted ¾" and all exposed edges chamfered ¾" unless otherwise shown (e.g. sharp edges at 4"x4" re-entrant corners of pier columns).
Reinforcement for Pier 21 only is shown on Sections BB to EE
Reinforcement in other piers is similar.
For details of bearing plates and fixing bolts, see Sheets Nº 9 and 10

DEPARTMENT OF MAIN ROADS, N.S.W.

STATE HIGHWAY Nº 2 — MUNICIPALITY OF CAMDEN

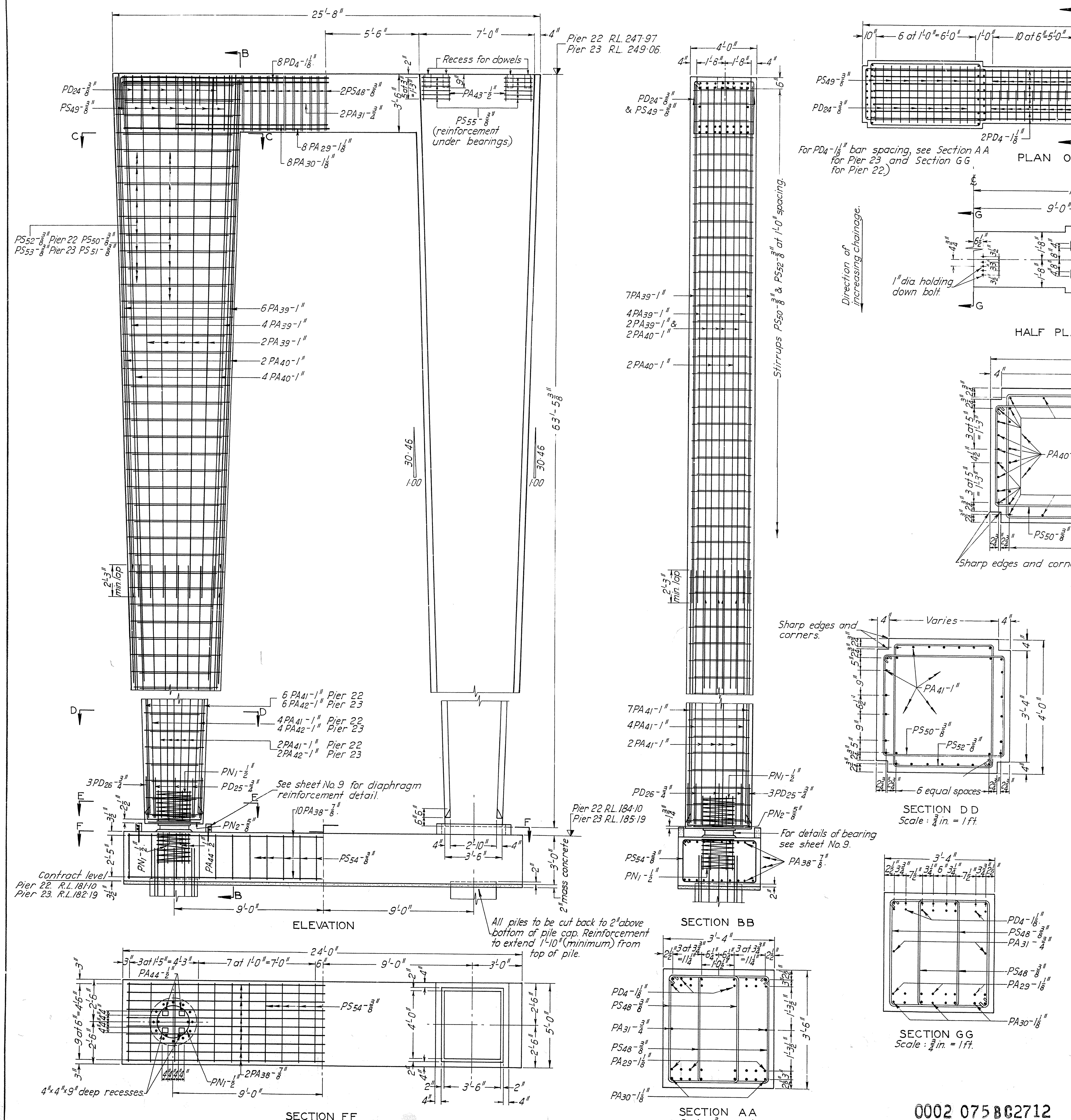
BRIDGE OVER NEPEAN RIVER
AT CAMDEN

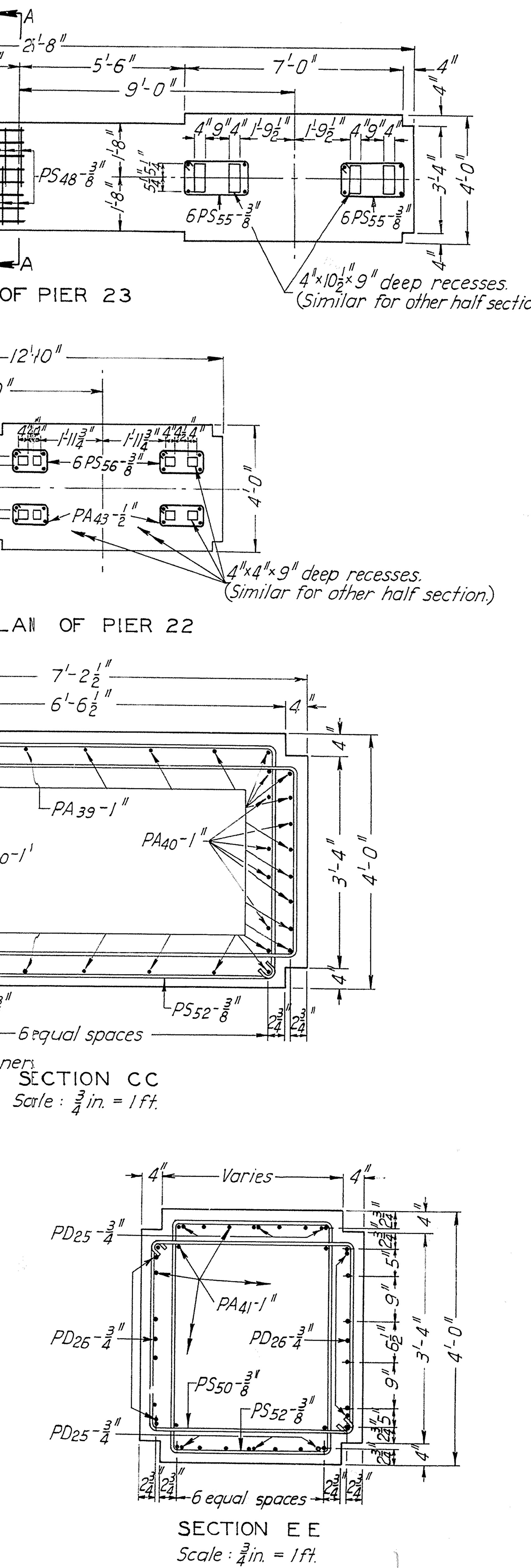
PIERS 9, 21, 24 AND 25

	PREPARED	CHECKED	REGN. No. OF PLANS
DESIGN	7.7.70	B.L. 7.7.70	2B2712
DRAWING	B.L. 7.7.70	7.7.70	

19.8.70
ASST. DESIGNING ENGINEER FOR BRIDGES
7.9.70
For ASSOCIATE BRIDGE ENGINEER

SHEET No. 7 — No. OF SHEETS 37





Scale : 3/8 in. = 1 ft. unless shown otherwise.

Concrete class 4K.

All corners filleted 3/4" and all exposed edges chamfered 3/4" unless shown otherwise. e.g. sharp edges at 4"×4" re-entrant corners of pier columns.

Reinforcement for Pier 22 only is shown on Section BB to EE.
Reinforcement in Pier 23 is similar.

For details of bearing plates and fixing bolts, see Sheets No 9 and 10

DEPARTMENT OF MAIN ROADS, N.S.W.

STATE HIGHWAY No 2 — MUNICIPALITY OF CAMDEN

BRIDGE OVER NEPEAN RIVER AT CAMDEN

PIERS 22 & 23 (RIVER PIERS)

	PREPARED	CHECKED
DESIGN	7·7·70	B.L. 7·7·70
DRAWING	B.L. 7·7·70	7·7·70

DESIGNING ENGINEER FOR BRIDGES 19.8.70

For ASSOCIATE BRIDGE ENGINEER 7·9·70

REGN. No. OF PLANS ~~2B2712~~

SHEET No. 8 — No. OF SHEETS 37

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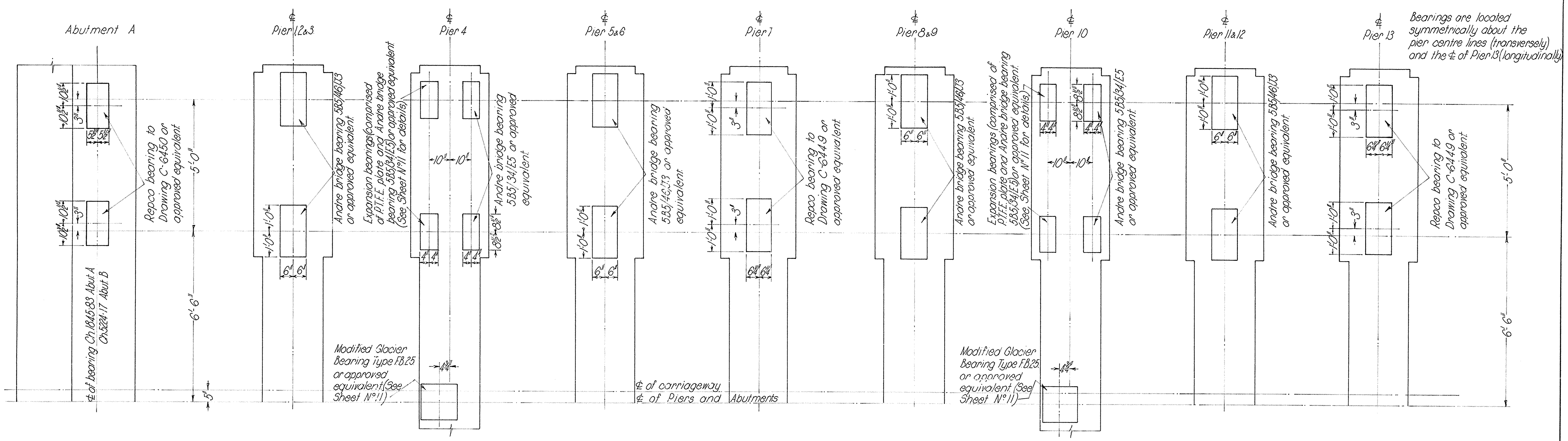
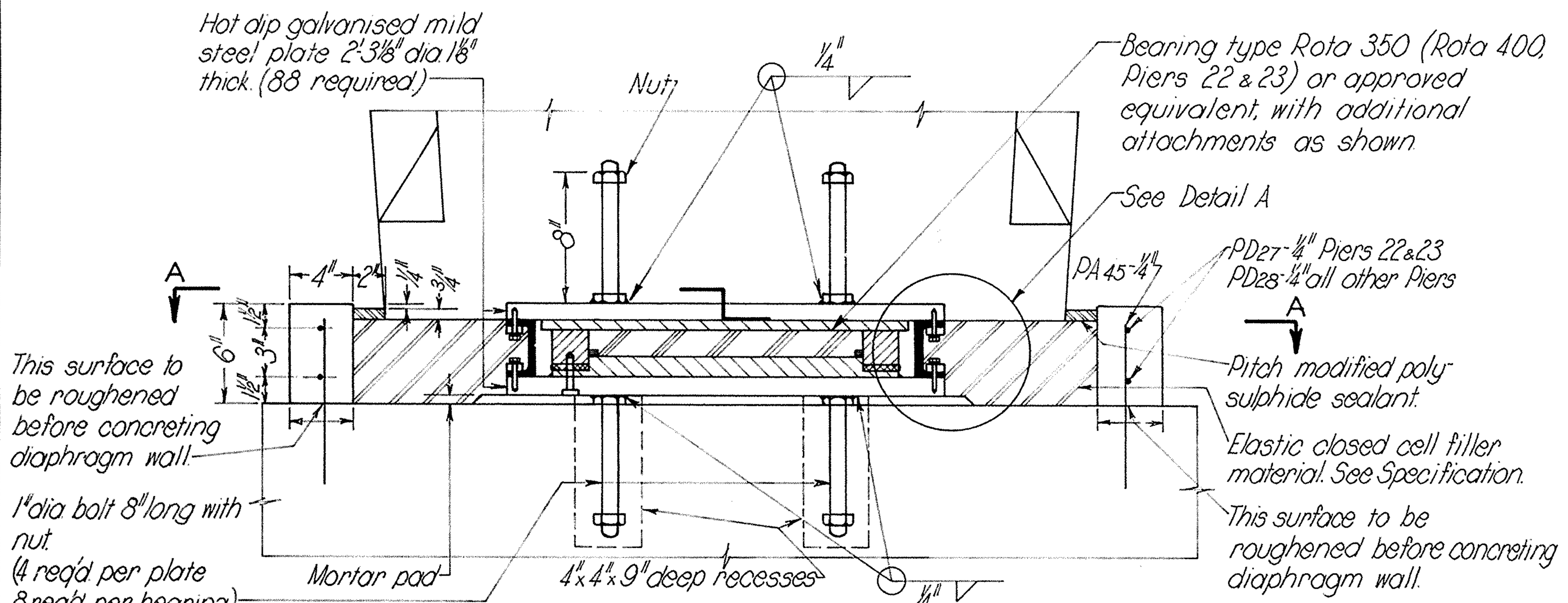
MICROBOX



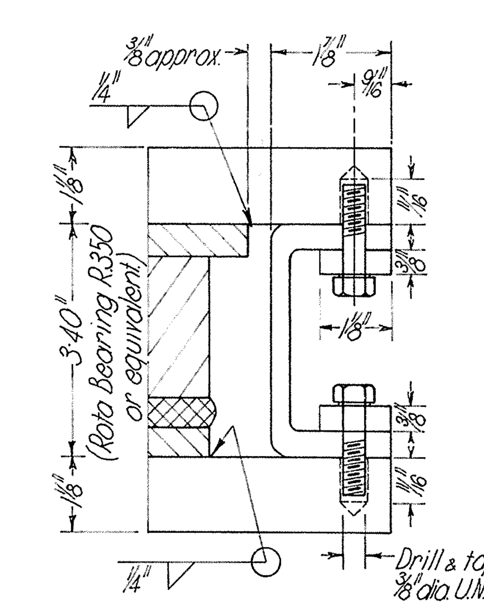
LAYOUT OF BEARINGS
AT TOP OF PIERS

Scale: 1/8 in = 1 ft.



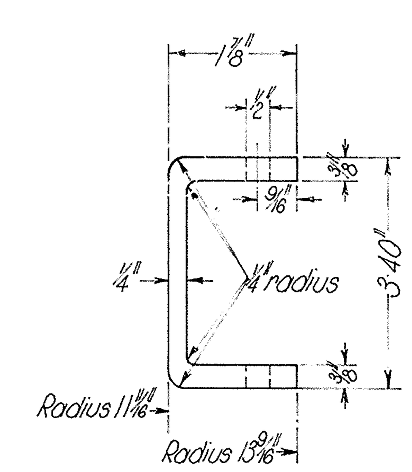
ELEVATION OF BEARING AND WATER-PROOFING

Details apply to Piers 1 to 6, 8 to 12, 14 to 18 & 20 to 25. (Total: 22 piers)



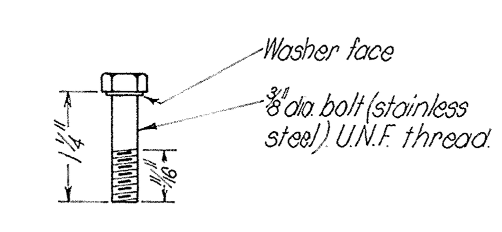
DETAIL A

Scale: 6 in = 1 ft.



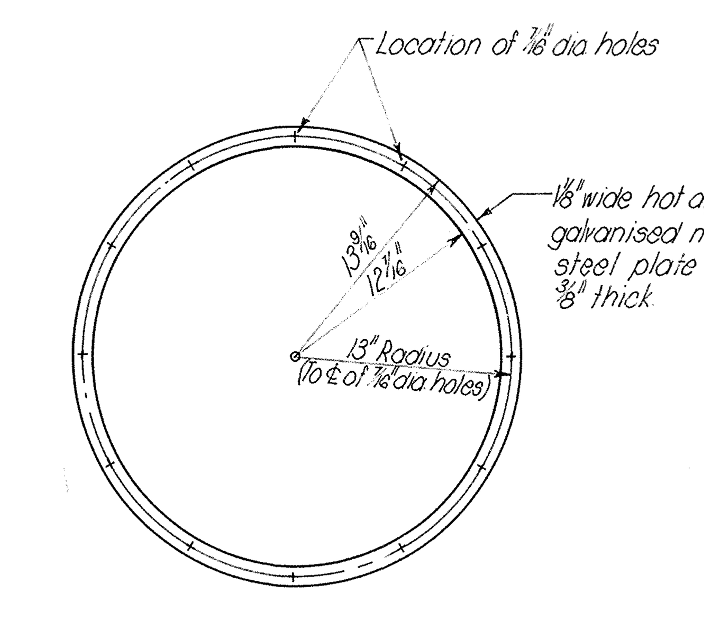
DETAIL B

Moulded Rubber Water Stop
44 required
The 1/2" dia holes at the top & at the bottom are staggered in location.
(See Section AA for details)
Scale: 6 in = 1 ft.



BOLT

(1056 required)
Scale: 6 in = 1 ft.

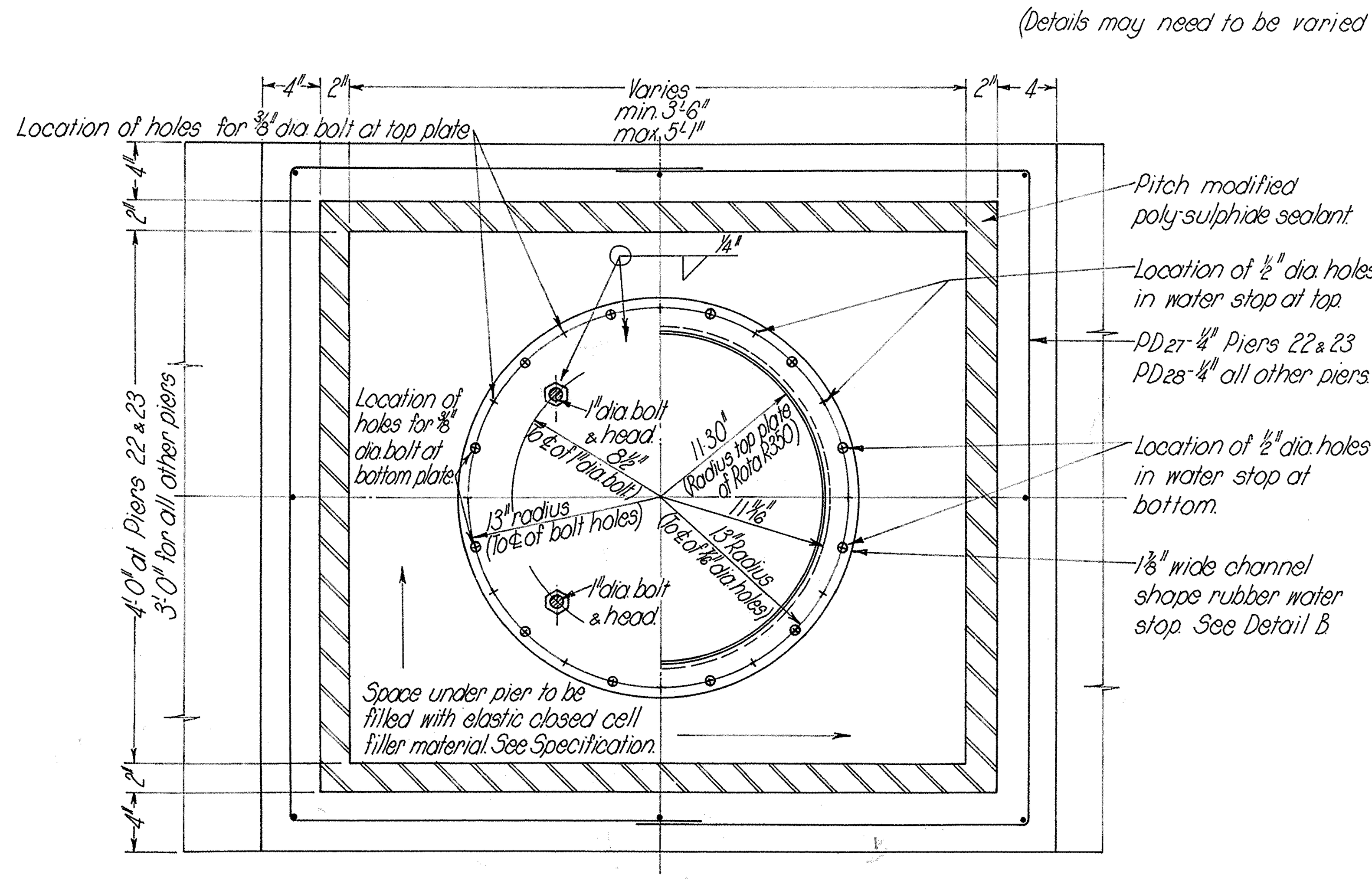


SEALING RING

88 required
Scale: 1 1/2 in = 1 ft.

LOWER ROCKER BEARING

(Details may need to be varied slightly for Piers 22 & 23)



SECTION AA

Scale: 1 1/2 in = 1 ft.

TABLE OF BEARING PERFORMANCE REQUIREMENTS

Bearing Location	Bearing Type (or approved equivalent)	Vertical Loading per Bearing (kips)		Horizontal Loading per Bearing (kips)		Rotation (Max Value) (Radian)		Movement (inches)	Number Required
		Maximum	Minimum	Longitudinal	Transverse	Longitudinal	Transverse	Longitudinal	
At base of all rocker piers except Piers 22 & 23	Rota R.350	785	450	Nil	37	0·0110	0·0040	Nil	40
At base of Piers 22 & 23	Rota R.400	960	450	Nil	40	0·0053	0·0070	Nil	4
Piers 1,2,3,5,6,8,9 etc.	Andre Bridge Bearing 5B5/46/J3	315	230	Nil	16·5	0·0124	0·0030	Nil	72
Piers 4,10,16 & 22 (fixed)	Andre Bridge Bearing 5B5/34/E5	140	90	See Note A below	8·2	0·0139	0·0030	Nil	16
Piers 4,10,16 & 22 (expansion)	Expansion Bearing (P.T.F.E. + 5B5/34/E5)	140	90	Nil	See Note B below	0·0139	0·0030	± 4½	16
Abutment	Repco bearing to Drawing C-6450	140	90	30	8·2	0·0065	0·0030	Nil	8
Piers 7,13 & 19	Repco bearing to Drawing C-6449	300	200	44	16·5	0·0087	0·0030	Nil	12
Piers 4, 10, 16 & 22 (Transverse bearing)	Modified Glacier Bearing Type FB25	Nil	Nil	Nil	33	0·0139	0·0030	± 4½	4

Note A: Horizontal loading (longitudinal) is the friction force for the Expansion Bearing.

B: Horizontal loading (transverse) of 33^k is resisted by a modified Glacier Bearing Type FB25 or approved equivalent. See Sheet Nº 11.

Unless shown otherwise all bearings are to have a protective coating of sprayed zinc or an approved zinc rich coating. (See Specification)

For details of bearings at Rocker Piers, Anchor Piers and Abutments see Sheet Nº 10.

For details of bearings at Expansion Joints see Sheet Nº 11.

Welding symbols to AS Z6 1969.

DEPARTMENT OF MAIN ROADS, N.S.W.	
STATE HIGHWAY Nº 2	MUNICIPALITY OF CAMDEN
BRIDGE OVER NEPEAN RIVER AT CAMDEN	
BEARING LAYOUT DIAGRAM & DETAIL OF LOWER ROCKER BEARING	
DESIGN — PREPARED 24/7/70, CHECKED B.L. 29/7/70; DRAWING — PREPARED B.L. 29/7/70, CHECKED 24/7/70	REGN. Nº OF PLANS 2B2712
ASST DESIGNING ENGINEER FOR BRIDGES	
For ASSOCIATE BRIDGE ENGINEER 7.9.70	SHEET Nº 9 — Nº OF SHEETS 37

D.Y.F.

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MICROBOX

APPENDIX B – Extract from CIRIA Report No. 116 Design of Reinforced Grass Waterways– Figure 9 – Recommended limiting values for erosion resistance of plain and reinforced grass

dense, tightly-knit turf established for at least two growing seasons. Poor grass cover consists of uneven tussocky grass growth with bare ground exposed or a significant proportion of non-grass weed species. Newly sown grass is likely to have poor cover for much of the first season.

4.3.2 Erosion resistance of geotextile reinforced grass

To enhance the erosion resistance of plain grass, a geotextile reinforcement system must both assist in either protecting or stabilising the surface soil particles, and improve lateral continuity between grass plants (see Section 2.2.2) Both three-dimensional mats and two-dimensional small-aperture fabrics (here defined as having an effective opening size, O_{90}, between 2 mm and 0.5 mm) are considered to fulfil these functions. Meshes only restrain, and provide lateral continuity between, grass plants. Thus by reducing the risk of localised erosion of grass plants, meshes sustain, rather than enhance, the protection provided by plain grass.

As with plain grass, erosion resistance of geotextile-reinforced grass is considered in terms of hydraulic loading parameters of velocity and duration of flow. Recommendations are shown in the velocity-duration diagram in Figure 9. Enhancement of erosion resistance over plain grass can only be achieved with good grass cover.

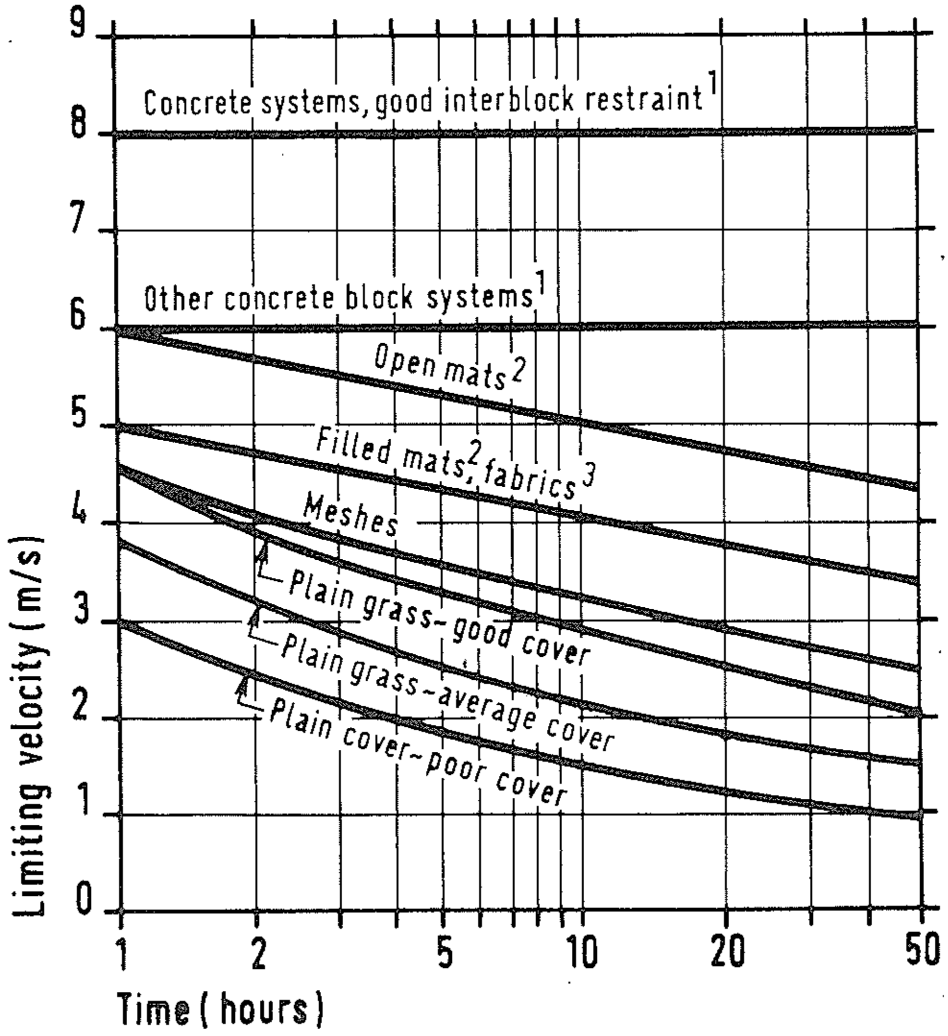

Figure 9 *Recommended limiting values for erosion resistance of plain and reinforced grass*

Notes:
1. Minimum superficial mass 135 kg/m^2, see Section 4.3.3 for other criteria.
2. Minimum nominal thickness 20 mm.
3. Installed within 20 mm of soil surface, or in conjunction with a surface mesh.
4. See Section 4.3.2 for other criteria for geotextile reinforcement.
5. These graphs should only be used for erosion resistance to unidirectional flow. Values are based on available experience and information at date of this report.
6. All reinforced grass values assume well-established, good grass cover.
7. Other criteria (such as short-term protection, ease of installation and management, susceptibility to vandalism, etc) must be considered in choice of reinforcement.