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Mid Western Regional Council

Report for Redbank Creek Dam

Alternative Stabilisation Works

May 2010

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

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1. INTRODUCTION

1.1 Project Background

Redbank Creek Dam is a gravity arch dam with a maximum height of 16m from the foundation and has a crest length of 152m. The radius of curvature at the upstream face of the dam wall is 77m. The dam was constructed between 1897 and 1899, as a water supply dam for the town of Mudgee.

The dam was constructed without any contraction joints (vertically in the upstream downstream direction) and was constructed essentially in continuous lifts across the dam. Seven semi vertical cracks, transverse to the dam axis, have formed extending from the crest to the base of the dam. The average intervals of the cracks are approximately 16.5m. These cracks are believed to have formed soon after the construction and were reported as early as 1909. Wade (1909) suggested that the vertical cracks are due to both drying shrinkage (water loss from the concrete after placement) and as a result of temperature variations (cracks close and open with changes of temperatures and with the absorption or loss of moisture by the concrete). It was further reported that the cracks almost disappear in the hot periods of summer when water levels are low or during winter when water levels are high, with their maximum opening occurring during winter with low water levels.

Past studies including the PWD (1995) and GHD (2002) have demonstrated that the dam does not satisfy the current acceptable safety criteria guidelines. It was found that the dam has a deficient spillway discharge capacity and an inadequate factor of safety against sliding, even for the normal operation load case (reservoir at Full Supply Level). Despite this, the dam has stored water to the existing crest level and has passed floodwaters over the crest without failing or showing significant signs of distress. It is believed that the previous analyses have adopted conservative assumptions and have under-estimated the material strength parameters of the dam concrete and its foundation.

In 2008, DOC developed a detailed design for stabilising the dam. The design includes the following features:

- Cutting down of the crest to RL 535.11m AHD; i.e. 3.76m from the top of the existing crest at RL 538.87 m AHD;
- Provision of a drop inlet spillway to lower the full supply level to RL 532.61 m AHD;
- Installation of 21 post tensioned sub-vertical cables on the downstream face of the dam – the cables dip upstream at 60^{o} (to the horizontal) and pass from the concrete into the foundation at a distance of 0.5m from the upstream heel of the dam;
- Installation of 9 post tensioned vertical cables at the crest of the dam, contained within the body of the dam;
- Provision of foundation drains dipping upstream at approximately 60^{o} from ground level at the downstream toe of the dam;
- Provision for grouting of six vertical cracks;
- Repair of cracks and cavities;
- Provision of a waterproofing system for the upstream concrete surface; and
- Provision of a slab at the toe of the dam, as protection against foundation erosion at the toe of the dam.

Subsequently, a 1.6 m diameter outlet pipe at RL 526.21m AHD has been installed through the dam. A toe erosion protection slab was also installed in the vicinity of the outlet pipe. The outlet pipe through the dam now means that the dam is unable to store water, apart from during floods when inflow exceeds outflow through the pipe. The dam now acts as a flood retarding basin and there is no risk of a "Sunny Day" dam break event.

The existing dam reservoir is only fully filled and overtopped during severe storm events (in excess of 1:1,000 year AEP). Therefore, it was considered appropriate that the scope of the stabilisation work should be reviewed.

1.2 Scope of Study

Mid Western Regional Council engaged GHD Pty Ltd (GHD) in November 2009 to investigate the feasibility of constructing a bypass spillway on either the left or right abutment to mitigate the dam safety risk during severe flood events. GHD subsequently visited the site and it was agreed that the bypass spillway option had a number of challenges, including maintenance of access for fire fighting equipment, cost of construction and issues surrounding erosion, both in the bypass channel and where flows would return to the river.

In view of the above and in the light of the fact that the dam now acts as a flood retarding structure, GHD was requested to evaluate the works necessary to maintain the stability of the dam, particularly whether some or all of the remaining elements of the Department of Commerce design (2008) would need to be implemented.

A number of assessments were made to evaluate whether the stresses in the dam would fall within acceptable limits by reducing the crest of the dam without installing post-tensioned anchorage. Reducing the crest did not result in acceptable stresses, while at the same time maintaining a sufficient capacity within the dam to retard storms up to the 1:100 AEP.

In view of the above, the scope of work was amended on 28th April 2010 and confirmed via email to encompass the following:

- Undertake a finite element stress analysis on the dam with water at the existing Full Supply Level (FSL) to estimate the stresses that the dam has successfully withstood; and
- Undertake a finite element stress analysis for the 3.76m crest cut down option, as designed by the Department of Commerce but without the installation of vertical post tensioned anchors, to determine the stresses for the 1:100,000 AEP and PMP floods and compare these to the stresses to which the dam has historically been subjected, in order to assess the potential safety margin.

2. FLOOD ANALYSIS

2.1 General

DoC (2008) proposed various stabilisation works, including lowering the crest level to RL 535.11m. Figure 1 and Figure 2 present the dam storage capacity curve and the spillway discharge rating curve respectively for the reduced dam crest level. The spillway discharge capacity was determined using the following equation:

$$Q = C_d L H^{1.5}$$

where:

Q = spillway discharge capacity (m^3/s)
C_d = discharge coefficient = 1.8
L = length of the spillway (the base length of the spillway is 120m);
H = Water head above the spillway sill (m)

Updated flood level estimates based various storm durations (up to 6 hours) for selected AEP storm events are presented in Figure 3 to Figure 6 below. These are also summarised in Table 1. The flood level estimates were based on the assumption that the initial reservoir storage level commences at the invert level of the outlet pipe (i.e. RL 526.21m) with a flood k_c value of 2.43. The annual exceedance probability of the water level in the reservoir is plotted in Figure 7.

Figure 1 Redbank Creek Dam Storage Capacity Curve

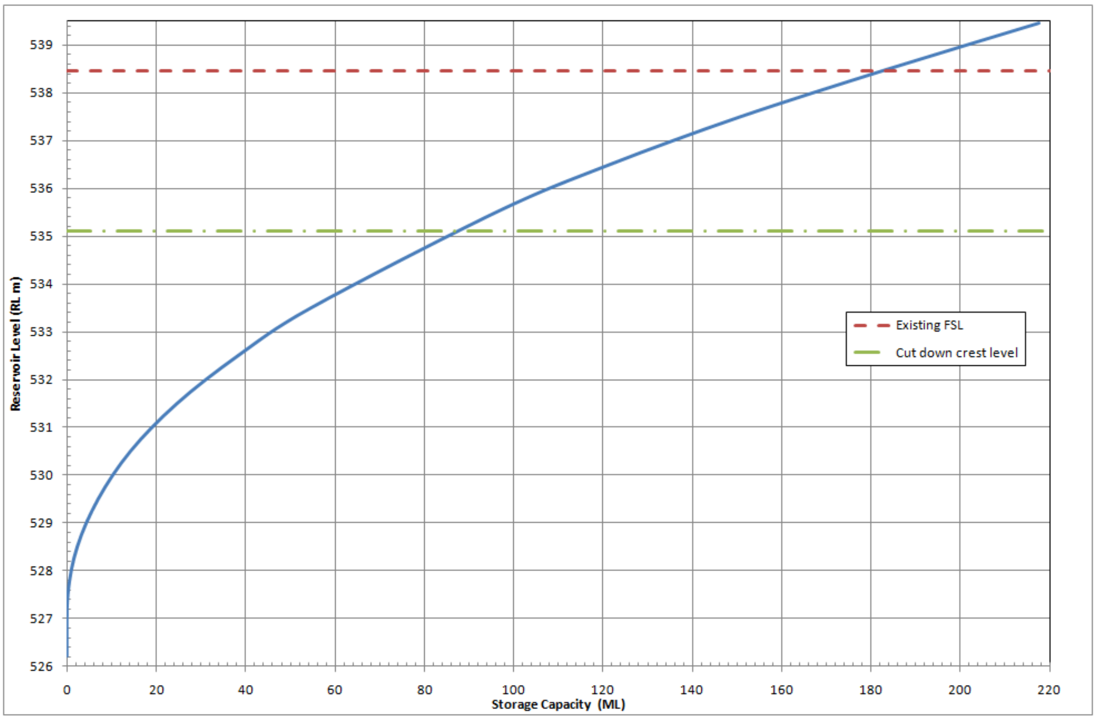
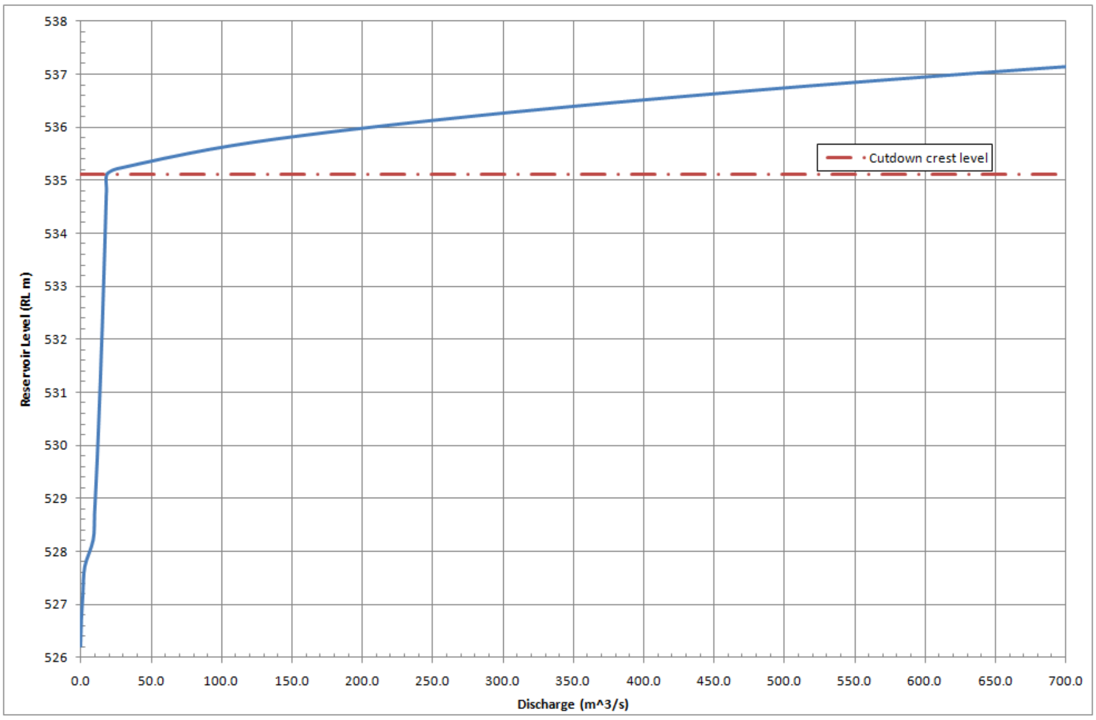


Figure 2 Redbank Creek Dam Spillway Discharge Rating Curve



It should be noted that the longer storm duration may result in extended reservoir impoundment and crest overtopping durations. However, during the occurrence of such a rare event, the dam safety emergency plan should have been activated and residents downstream should have been notified and evacuated.

Table 1 Summary of Flood Level Estimates for k_c = 2.43

Flood AEP	Minimum Peak Water Level	Maximum Peak Water Level	Reservoir Impounding Duration	Range of Crest Overtopping Duration
1E-02	RL533.39m	RL534.45m	3 to 8 hours	Nil
1E-03	RL535.29m	RL535.34m	5 to 8.5 hours	1 to 2.5 hours
1E-05	RL535.80m	RL535.89m	6 to 10 hours	2 to 6 hours
1E-07 (PMP event)	RL536.05m	RL536.31m	6 to 10 hours	2 to 8 hours

It should be noted that the maximum water level for the PMP remains approximately 2.1 m below the original dam FSL, i.e. the water loadings in the dam should remain well below those already successfully resisted.

Figure 3 Reservoir level estimates for 1 in 100 AEP storm duration, ranging from 1 hour to 6 hours

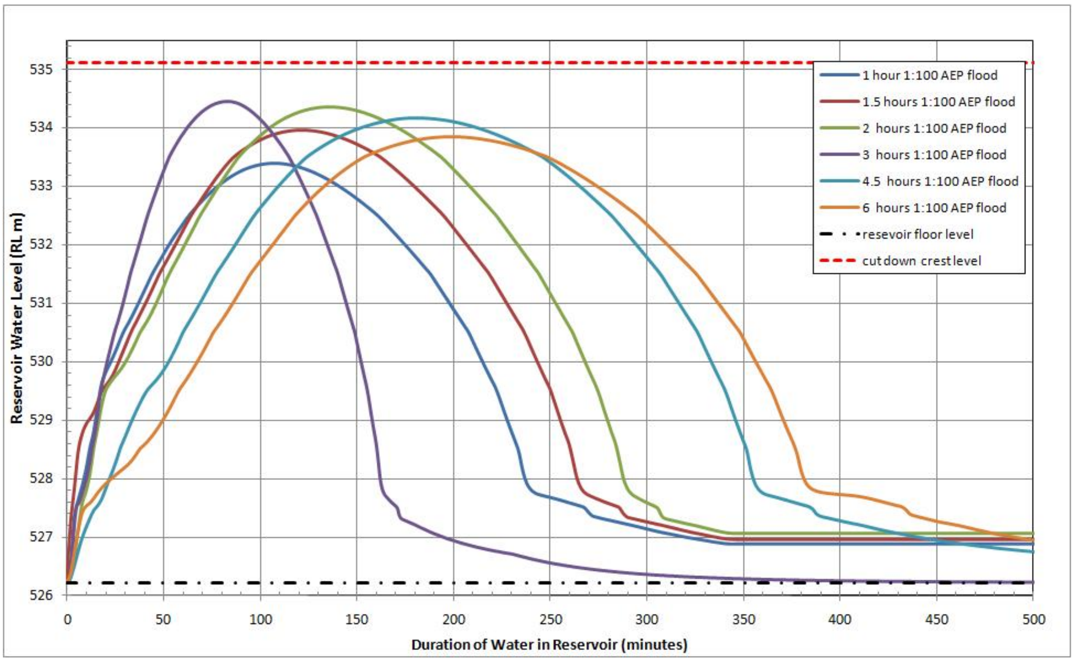


Figure 4 Reservoir level estimates for 1 in 1,000 AEP storm duration, ranging from 1 hour to 6 hours

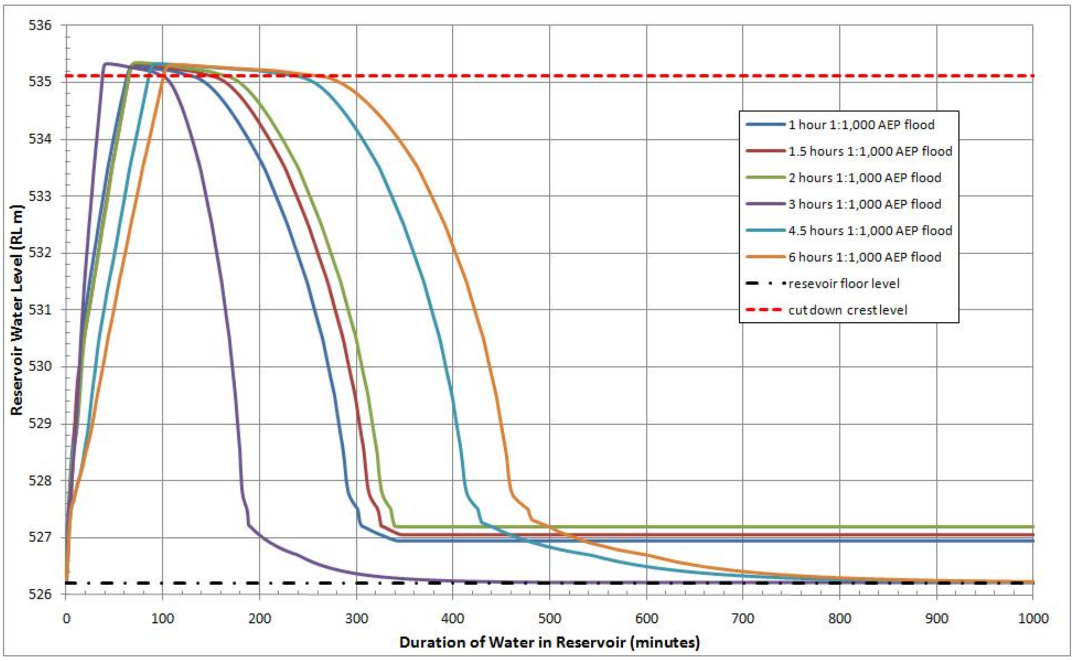


Figure 5 Reservoir level estimates for 1 in 100,000 AEP storm duration, ranging from 1 hour to 6 hours

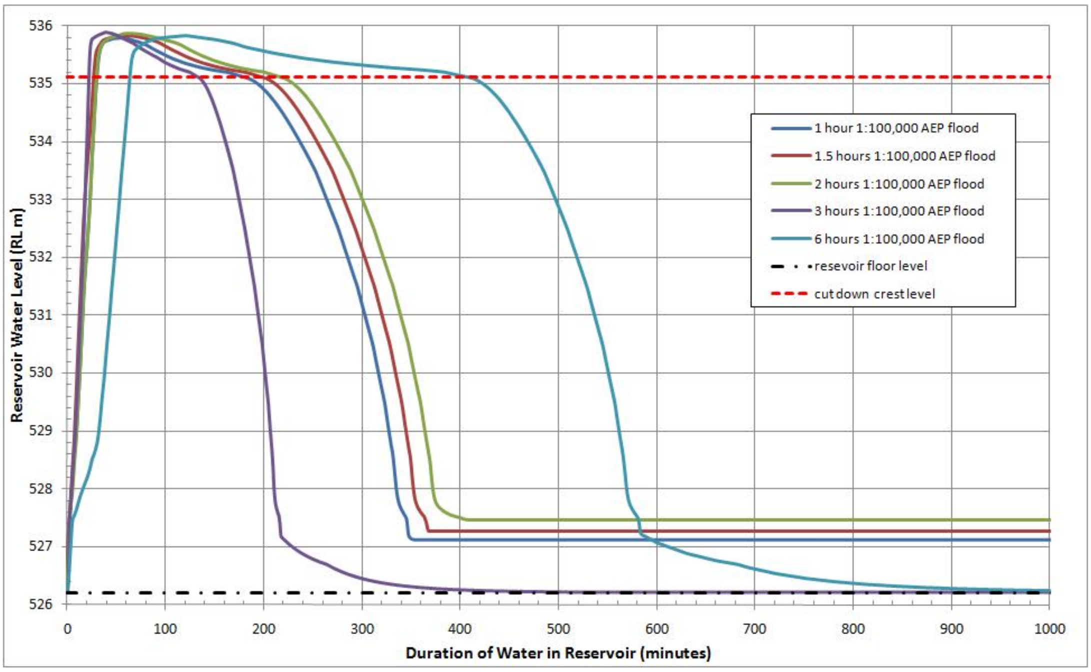


Figure 6 Reservoir level estimates for PMP storm duration, ranging from 1 hour to 6 hours

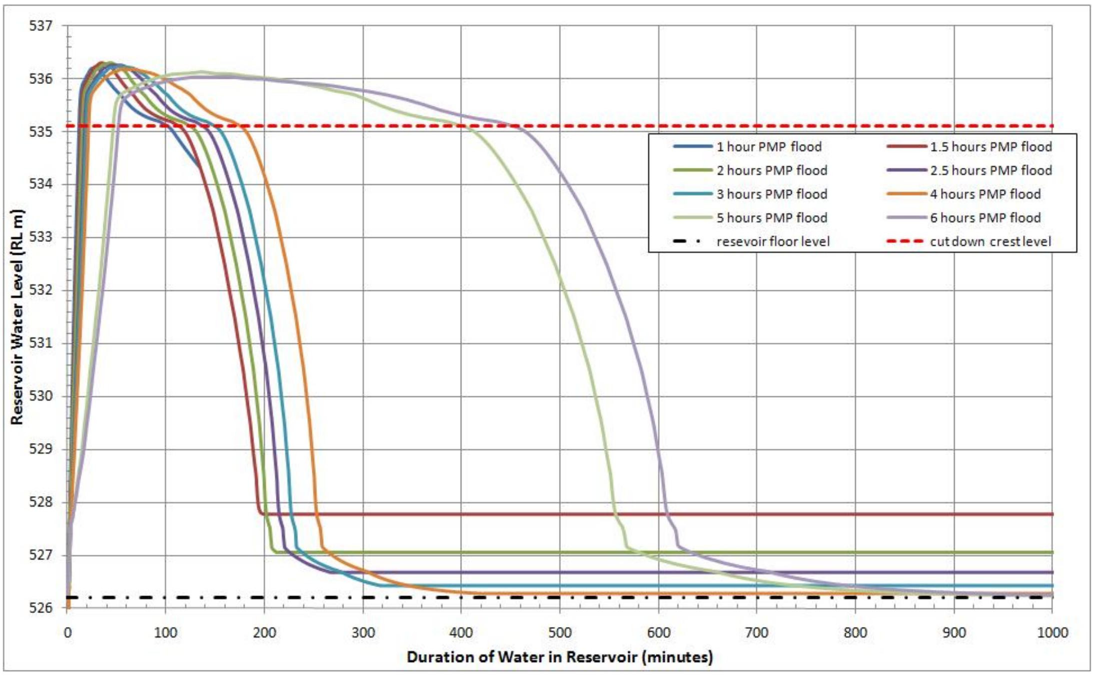
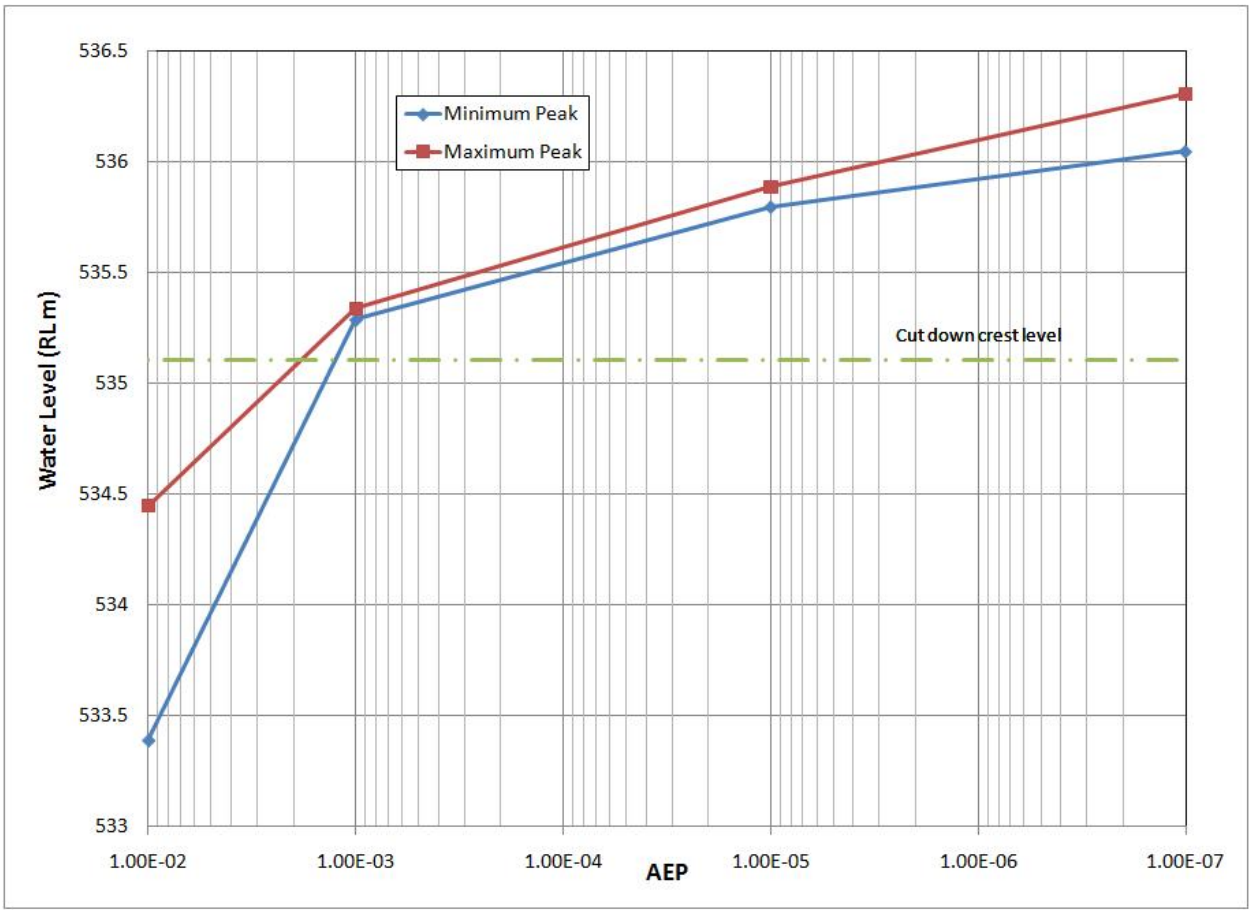


Figure 7 Redbank Creek Dam Water Level Annual Exceedance Probability



Note: The above graph indicates the upper and lower envelope for flood inflows with various storm durations. It may be seen that the dam overtops at floods with a return period of about 1:950 years.

3. PRESENT CONDITION OF THE DAM

3.1 General

This section discusses the present condition of the dam.

3.2 Concrete Dam

The dam was constructed using plum aggregates, embedded in a fairly sloppy (wet) concrete mix in order to minimise contraction and shrinkage (Wade, 1909). Wade (1909) states that the maximum size of the plums were such that they could be handled by two persons. Concrete cores logged by PWD (1995) and GHD LongMac (2002) showed that the plums were of volcanic origin. From core photographs, the plums appear to be grey in colour and therefore likely to have been imported from another quarry and not the quarry between the dam and the downstream weir. The plums are typically jointed with smooth, planar surfaces containing limonite staining.

3.2.1 Compressive Strength

PWD (1995) and GHD LongMac (2002) conducted concrete coring and laboratory testing to determine the material properties of the concrete. Table 3 below gives a summary of the concrete strength properties. The concrete compressive strength ranges from approximately 15 MPa to 45 MPa and the strength of the plums range from 50 MPa to 65 MPa, indicating that the plums within the dam wall are generally stronger than the surrounding concrete. GHD (2002 & 2004) estimated the characteristic compressive strength of the concrete to be 11 MPa.

3.2.2 Tensile Strength

No laboratory uniaxial tension test results are available. It is difficult to confidently estimate the tensile strength for an existing dam constructed before quality control of construction materials became refined. It appears reasonable to follow established practices of most major dams authorities and to assign a tensile strength to the concrete in Redbank Creek Dam in line with precedent practice as applied to similar analyses.

The most common approach is to relate tensile strength to the measured compressive strength. A nominal value of 10% of compressive strength has been proposed by some dams authorities e.g. USACE (1994) and the Canadian CDSA (1999). Data from recent construction projects show a tensile strength in the range of 6% to 10% of the compressive strength. The USBR proposes a tensile strength value of 5 to 6% of the compressive strength. The recommendations appear to have been based on the intact tensile strength of concrete, with an allowance for micro fractures and other non continuous discontinuities in the concrete. Table 2 below shows the summary of tensile strength values recommended by various dams authorities.

Table 2 Summary of Recommended Value for Concrete Dam

Property	**Guideline and Recommended Value's** (Note 1)				
	USBR (1977)	**ANCOLD (1991)**	**USACE (1994)**	**CDSA (1999)** (Note 3)	**FERC (2002)**
Static tensile strength					
- concrete	0.05 to 0.06 f'_c	0.2 $\sqrt{f'_c}$ (Note 2)	0.1 f'_c	0.1 f'_c	0.14 $(f'_c)^{2/3}$
- lift joint	(Note 5)	0 (Note 6)	(Note 5)	0.05 f'_c	
Shear strength					
- Peak c'					
- concrete	0.1 f'_c	0.14 f'_c (Note 2)	0.1 f'_c	0.17 $\sqrt{f'_c}$	(Note 4)
- lift joint	(Note 5)	0.07 f'_c (Note 6)	(Note 5)	0.085 $\sqrt{f'_c}$	0
- Peak ϕ'					
- concrete	45^o	45^o	45^o	45^o	(Note 4)
- lift joint	(Note 5)		(Note 5)	55^o	55^o

Notes

(1) Tensile strength, compressive strength and cohesion in MPa

(2) Value applied to normal concrete with well prepared construction joints

(3) Assumes good quality concrete and lift joints

(4) Assumes pre-cracked concrete, i.e. no guidance provided

(5) Assumes intact concrete i.e. no guidance is provided for lift joint

(6) Assumes concrete of uncertain quality

In their report, "Redbank Creek Dam Stabilisation Works Design Report", Department of Commerce (DoC) (2008) adopted an apparent tensile strength of 1.1 MPa within the concrete of the dam wall. It should be noted that the corresponding true tensile strength is approximately 810 kPa. The apparent tensile strength is used to account for the linear stress strain relationship in the linear elastic finite element analysis (Raphael, 1984). The true tensile strength is thus equivalent to 8% of the compressive strength and is deemed to be realistic.

However, this estimate does not make any direct allowance for continuous un-bonded construction joints and cracks. Redbank Creek Dam has a number of cracks which are believed to pass through the dam, principally along the construction joints. A predominant horizontal crack is located approximately 3.7 m below the existing non-overspill crest level. This horizontal crack is within close proximity to a horizontal lift joint and is visible from the downstream face, although it does not appear to be noticeable on the upstream face.

The dam was likely to have been constructed roughly horizontally, with construction joints being generally horizontal across the full width of the dam. GHD LongMac (2002) found that within the dam wall, breaks in the concrete core commonly occurred at the boundary between the plum and concrete matrix. This is possibly due to the difference in stiffness of the concrete matrix and the plum. However, given that the plums are stronger than the surrounding concrete matrix, even if there is a break (discontinuity in the concrete matrix and the plum), the failure of the concrete dam in tension is governed by interlocking between the plum aggregates (or surface roughness), which in turn provides mechanical anchorage within the mass of the concrete dam wall. A large amount of energy is required to fracture through the concrete/plum interface and large dilatation has to occur before the concrete wall will fail in tension and shear. For this reason, even if the concrete is "chemically" unbonded, some residual tensile strength still exists due to the friction and interlocking between the concrete matrix/plums.

Khabbaz and Fell (1999) assessed 68 sets of data for direct tensile strength of concrete with lift joints for dams built in the USA before 1940. They found that the strength ranges from 311 kPa to 2,967 kPa, with a mean of 1,350 kPa. Similarly, EPRI (1992) evaluated the tensile strength of concrete with lift joints for 14 dams, for a total of 107 specimens, and found that the average tensile strength is approximately 1200 kPa (i.e. 80 to 90% of the monolithic concrete tensile strength) and about 60% of the samples did not fail at the lift joints.

For the reasons given above, it is not unreasonable to adopt a value for the tensile strength of the concrete joint equal to 30% of the intact concrete tensile strength (i.e. a true tensile strength of 250 kPa or an apparent tensile strength of 340 kPa). As shown in Figure 8 below, this strength represents a confidence level of approximately 90% of the EPRI (1992) test results and 99% of the Khabbaz and Fell (1999) test results.

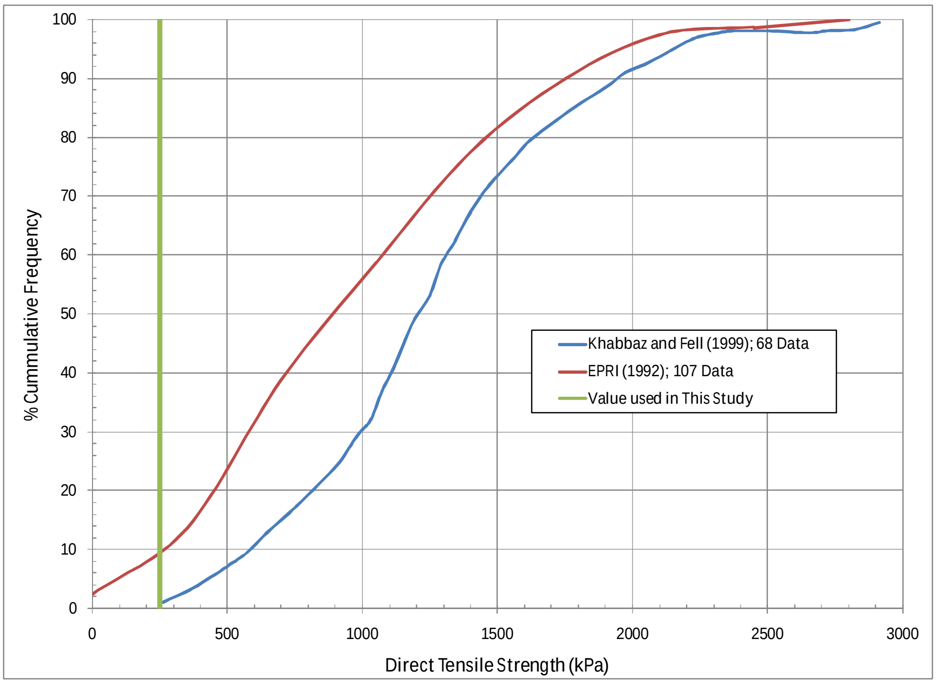
Table 3 Summary of Concrete Properties (extracted from PSM 2006)

SOURCE	SPECIMEN	TYPE[(a)]	ρ_{dry} [kg/m^3]	UCS [MPa]	E [Gpa]	μ m	COMMENT[(b)]
PWS (1995)	DDH1	Direct	2150	12	26.7	0.06	
PWS (1995)	DDH1	Indirect	2400	-	24.8	0.33	
PWS (1995)	DDH2	Direct	2280	26	25.3	0.17	
PWS (1995)	DDH2	Indirect	2570	-	38.3	0.31	
PWS (1995)	DDH3	Direct	2130	21	20.4	0.45	
PWS (1995)	DDH3	Indirect	2465	-	17.4	0.33	
GHD (2002)	Ch.1 (5)	Direct	2330	24.5	-	-	10% boulder
GHD (2002)	Ch.1 (7)	Direct	-	21.5	26.6	-	
GHD (2002)	Ch.10 (1)	Direct	-	44.5	23.1	-	
GHD (2002)	Ch.10 (2)	Direct	2240	26	-	-	25% boulder
GHD (2002)	Ch.10 (2)	Direct	2260	20.5	-	-	25% boulder
GHD (2002)	Ch.10 (3)	Direct	2644	62.5	-	-	100% boulder
GHD (2002)	Ch.11 (5)	Direct	2090	17.5	-	-	
GHD (2002)	Ch.11 (7)	Direct	-	28.5	26.5	-	
GHD (2002)	Ch.2 (5)	Direct	2510	27.5	-	-	50% boulder
GHD (2002)	Ch.3 (3)	Direct	2521	59	-	-	40% boulder
GHD (2002)	Ch.3 (5)	Direct	2360	26	-	-	30% boulder
GHD (2002)	Ch.4 (1)	Direct	2260	28.5	-	-	
GHD (2002)	Ch.5 (1)	Direct	-	32.5	29.8	-	
GHD (2002)	Ch.5 (3)	Direct	2210	28.5	-	-	5% boulder
GHD (2002)	Ch.5 (4)	Direct	2370	24.5	-	-	10% boulder
GHD (2002)	Ch.6 (1)	Direct	2170	18.5	-	-	
GHD (2002)	Ch.6 (6)	Direct	2330	42	-	-	15% boulder
GHD (2002)	Ch.7 (3)	Direct	1950	4	-	-	30% boulder
GHD (2002)	Ch.7 (5)	Direct	2669	60	-	-	100% boulder
GHD (2002)	Ch.8 (1)	Direct	2220	17.5	-	-	
GHD (2002)	Ch.8 (3)	Direct	2660	52	-	-	100% boulder
GHD (2002)	Ch.8 (6)	Direct	2360	24	-	-	40% boulder
GHD (2002)	Ch.9 (2)	Direct	-	24.5	21.1	-	
GHD (2002)	Ch.9 (6)	Direct	2681	58.5	-	-	100% boulder
GHD (2002)	BH2	Direct	2330	22.5	-	-	
GHD (2002)	BH3	Direct	2400	16.5	-	-	
GHD (2002)	BH5	Direct	2280	24	-	-	
GHD (2002)	BH6	Direct	2170	28	-	-	
GHD (2002)	BH7	Direct	2450	15	-	-	

(a) Direct tests undertaken in accordance with AS1012.4, AS4133.4.3-1993 or ISRM. Indirect tests undertaken ultrasonically.

(b) Some samples included a proportion of volcanic boulder as indicated.

Figure 8 Tensile strength of concrete with lift joints, summary of data from Khabbaz and Fell (1999) and EPRI (1992)



3.2.3 Shear Strength

The shear strength of the concrete is represented by the angle of friction and asperity angle of the plums, or the cohesion of the concrete mass. DoC (2008) adopted a shear strength equal to an angle of friction of 45^{o} and no cohesion for the concrete mass. While it is rational to take the angle of friction as 45^{o}, the assumption of zero cohesion and no asperity angle of the plums is considered to be conservative.

Based on the Mohr Coulomb failure criterion theory and assuming a linear Mohr Coulomb failure envelope as shown in the red line in Figure 9 below, the cohesion (*c*) of the concrete and horizontal joints can be estimated as:

$$c = \sigma_t \tan \phi$$

where:

c = cohesion

σ_t = apparent tensile strength

ϕ = frictional angle = 45^{o}

Given that the apparent tensile strength of the intact concrete and horizontal joint is 1,100 kPa and 340 kPa respectively, the corresponding cohesion is estimated to be 1,100 kPa and 340 kPa. The estimate of shear strength for horizontal joints is within the limit of Khabbaz and Fell (1999) and EPRI (1992) as presented in Figure 10 and Figure 11 respectively.

Figure 9 **Mohr Coulomb Diagram –** Relationship between shear strength and normal (compressive and tensile) strength

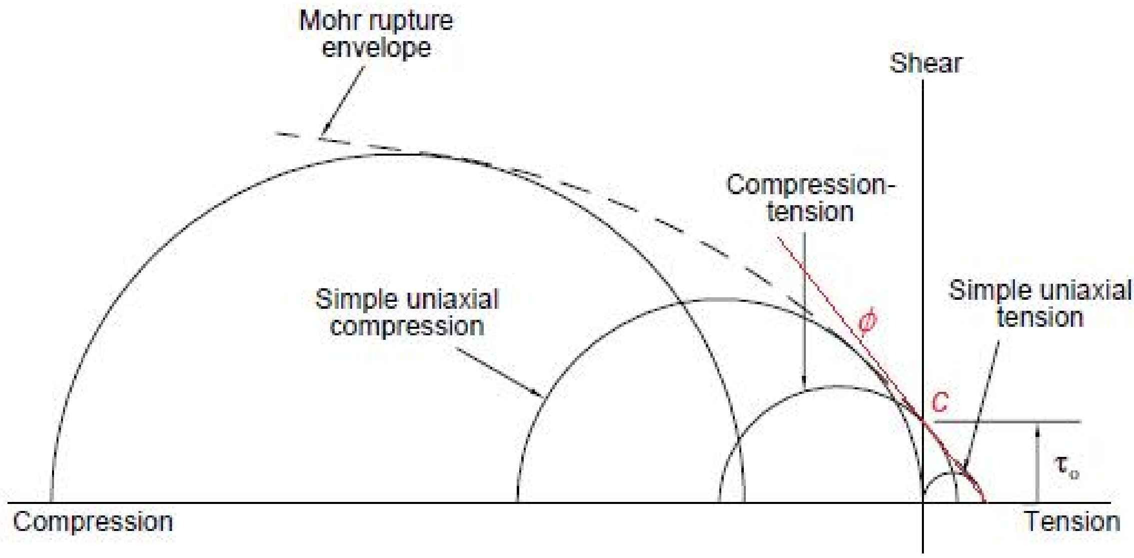


Figure 10 **Redbank Creek Dam: sliding friction shear strength of concrete with lift joints compared to the strength results of USA dams built before 1940 (Khabbaz and Fell, 1999)**

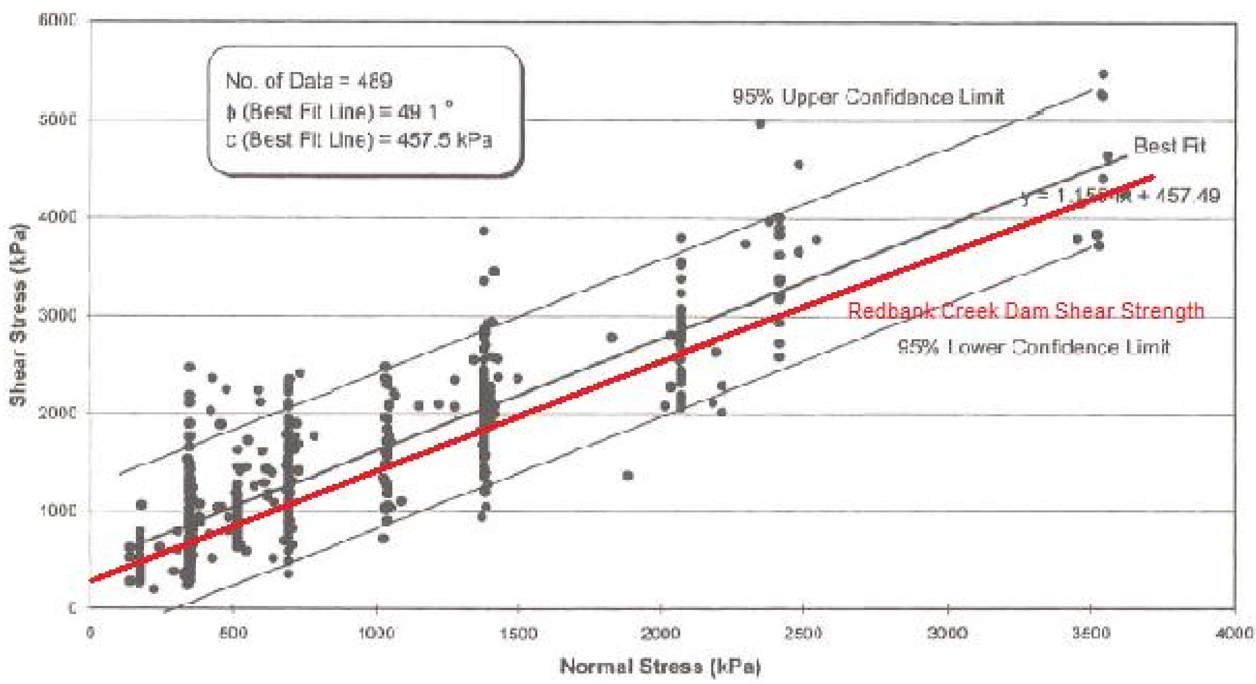
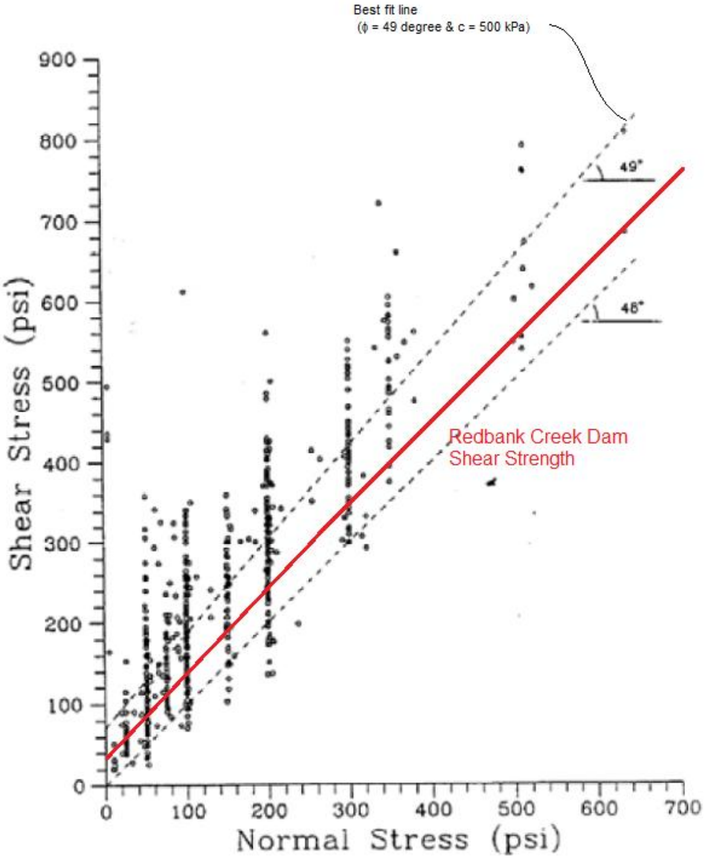


Figure 11 Redbank Creek Dam: sliding friction shear strength of concrete with lift joints compared to the 154 unbounded joint shear strength results obtained from 10 dams constructed between 1906 to 1973 (EPRI, 1992)



3.2.4 Durability and permeability of the concrete

Concrete is a permeable medium. The extent of cracks and voids in the concrete and their ability to link up with each other to form seepage paths will allow water to seep out on the downstream face, sometimes at distinct locations.

With continuous seepage through the concrete, there will be gradual leaching of the chemical components of the cement matrix. Seepage water picks up excess lime as it percolates through concrete, forming calcium hydroxide. As the water exits the concrete, the calcium hydroxide combines with carbon dioxide in the air to deposit calcium carbonate as calcite which, being insoluble, precipitates. Calcite is evidenced in several locations on the downstream face of the dam. It is however believed that any excess lime within the concrete can be considered to have now leached out completely, given that the dam was constructed more than 100 years ago. Similarly, there is no evidence of alkali aggregate reaction within the concrete. Even if there is, the reactivity action should all have by now taken place.

One of the major threats to the integrity of the concrete dam is weathering. The weathering process is accelerated when the concrete is subjected to excessive changes in temperature gradient and wet and dry environment, as is presently occurring. Therefore, the degree of weathering of the dam concrete should be closely monitored in the future.

3.3 Foundation

3.3.1 Geology

The dam is founded within a unit of the Devonian Burrundulla Shales. The Burrundulla Shales comprise siltstones and shales that are red-brown in colour and typically massively bedded. The units are typically well jointed with close spaced joints that are often tight with iron / manganese staining.

3.3.2 Foundation treatment

No data is available on foundation treatment (grouting) for the original construction of the dam. However, given that the original dam was constructed in the late 1800's, it is unlikely that any foundation treatment or grouting was undertaken.

It is believed however that some additional effort was put into excavation of the dam foundation at the time of construction. Compared to the original design drawings, which show that the dam was designed to be 13.6 m high, the actual dam is 16 m high. This indicates that a rock socket may have been excavated. This appears to have been confirmed during the GHD LongMac (2002) site investigations, where it was found that the dam is generally founded on fresh to slightly weathered siltstone except on the right abutment where a splay fault or imbricate is located.

3.3.3 Properties

As indicated, the dam is generally founded on fresh to slightly weathered, medium strong to strong intact siltstone. However, slightly to highly weathered sitstones/shales were observed on the right abutment (in BH5 of the GHD LongMac (2002) site investigation).

The rock mass is typically fractured to highly fractured. The GHD LongMac (2002) investigation report stated that within 5 metres below the base of the dam, the fracture frequencies are in the range of 3 to 18 per metre, with a mean of 10. A similar finding was reported by PSM (2006).

Sixty point load tests were conducted by GHD LongMac (2002) on the rock to estimate the strength of the intact rock. The mean compressive strength was found to be 47 MPa. In addition, four samples of rock core were tested. The average compressive strengths were 18 MPa, due to the fact that each of the rock cores had fractures present. The compressive strength values thus indicate the rock mass strength rather than that of the intact rock. Similarly, the elastic modulus measured via strain gauges attached on the rock core yielded an average value of 27.7 GPa while the modulus measured by the deformation over the height of the cores (or in reality the rock mass) gave a mean value of 3.5 GPa.

Using the Hoek and Brown rock mass failure criterion, PSM (2006) estimated the shear strength parameters of the rock mass as shown in Table 4 below.

Table 4 Summary of Rock Strength Properties recommended by PSM (2006)

Property	Value	PSM Comments
Compressive Strength	10 MPa or slightly higher	Similar to concrete and possibly slightly better
Angle of Friction	41^{o}	Based on potential failure though 30% of structure joints and 70% of rock mass
Cohesion	120 kPa	
Deformation Modulus	1.0 GPa	

3.3.4 Adopted Strength

For this study, the rock strength properties recommended by PSM (2006) were generally adopted, with the following exceptions.

The deformation modulus of 2.5 GPa was adopted. This value falls between the 3.5 GPa measured by GHD LongMac (2002) and the 1.0 GPa estimated by PSM (2006).

In addition, the value recommended by PSM has ignored the effect of asperity within the dam foundation interface. As some excavation work has been undertaken on the dam foundation (onto slightly weathered rock), the profile of the foundation is likely to be irregular. For this reason, the adopted shear strength is taken as follows:

Angle of friction + angle of asperity = 51^{o}

The tensile strength of the rock foundation interface estimated by PSM (2006) is deemed to be over conservative (i.e. too low). A preliminary finite element analysis showed that if the PSM (2006) recommended tensile strength of the dam foundation interface was adopted, full crack propagation will occur beneath the dam and the dam will not be stable, even at FSL. It is obvious that for the dam to have performed as it has, both the concrete and the foundation must be capable of sustaining a level of tensile stress in order to prevent full crack propagation with the loading imposed when the water level is at FSL. In addition, the dam has performed satisfactorily in excess of 80 years and has been overtopped on several occasions. For this reason, GHD has estimated the tensile strength based on the recommended shear strength using Mohr Coulomb failure criterion theory as discussed in Section 3.2.3. The apparent tensile strength at the interface is estimated to be 150 kPa. This will translate to a true tensile strength for the finite element analysis of 200 kPa at the foundation interface.

3.4 Previous Study of the Existing Dam

3.4.1 1995 PWD Assessment

A stability review report of Redbank Creek Dam was undertaken by NSW Department of Public Works in 1995. The review included drilling of three boreholes through the foundation and concrete and a finite element analysis of the existing structure. The following conclusions were drawn from the safety review:

- The dam did not satisfy current design standards set by ANCOLD or the USBR in terms of sliding resistance and maximum allowable stresses in the concrete.

- The crest length/height ratio was considered too high, which resulted in cracking of the cantilevers and load transfer to the horizontal arches. The report indicates that the cantilevers should be taking most of the load.
- The limited geotechnical investigations indicated that the foundation rock is highly fractured which would prejudice the sliding resistance of the dam.
- The limited investigations comprising only three unconfined compressive strength tests of the concrete indicated strengths as low as 12 MPa. There was concern that weaker zones may be present but due to the limited number of tests, they may not have been identified.
- There was concern in regard to further degradation of concrete and foundations due to efflorescence and weathering of the foundations.
- There was concern over the effect of the downstream horizontal crack on the performance of the structure during loading.

3.4.2 2002 GHD Assessment

GHD undertook a detailed structural integrity assessment of the dam in 2002. A comprehensive site investigation was carried out. It was found that the dam foundations comprise siltstone, predominantly fresh to slightly weathered, with medium strong to strong intact rock. The foundations are generally highly fractured, with no discernible change within the depth investigated.

Finite element analyses were performed using the following assumptions:

- The adopted shear strength parameter was an angle of friction of 50^{o} and 0 kPa cohesion.
- Tensile capacity in concrete and dam/foundation interface was taken as 10% of the design compressive strength of the concrete;
- Elastic modulus of the dam/ foundation contact was taken as 10 GPa.

The analyses included the effect of the horizontal cracks located at about 3m below the present crest level. It was found that the crack had a marginal effect on the stress distribution and insignificant effect on the sliding factor of safety of the dam.

The results of the analyses confirmed the 1995 assessment that the dam does not satisfy current acceptable safety criteria in terms of sliding. The earlier assessment did not incorporate the existing vertical cracks and the horizontal downstream crack, which would further destabilise the arch.

The assessment of the existing structure concluded that the sliding resistance of the dam is inadequate and that the compressive stresses imposed on the structure and the fractured foundations exceeded acceptable limits according to both USBR and ANCOLD guidelines. In addition, it was found that at full supply level, the dam is likely to crack through approximately 65% of the base width.

4. FINITE ELEMENT ANALYSIS OF PRESENT DAM

4.1 General

Past studies including the PWD (1995) and GHD (2002) have demonstrated that the dam does not satisfy the current acceptable safety criteria guidelines. It was found that the dam has a deficient spillway discharge capacity and an inadequate factor of safety against sliding, even for the reservoir level at FSL. Despite this, the dam has successfully stored water to the existing crest level and has passed floodwaters over the crest without failing or showing significant signs of distress. It is believed that the previous analyses have adopted conservative assumptions and have under-estimated the material strength of the dam concrete and its foundation.

The present study has reviewed all available information from previous investigations and a finite element analysis was performed to estimate the likely stresses to which the dam has been subjected. The estimated material strengths used in the present study have been based on previous investigations and studies, while at the same time taking account of the construction technology in the 1890's. Guidance has also been obtained from published literature on similar dams, as discussed in Sections 3.2 and 3.3 above.

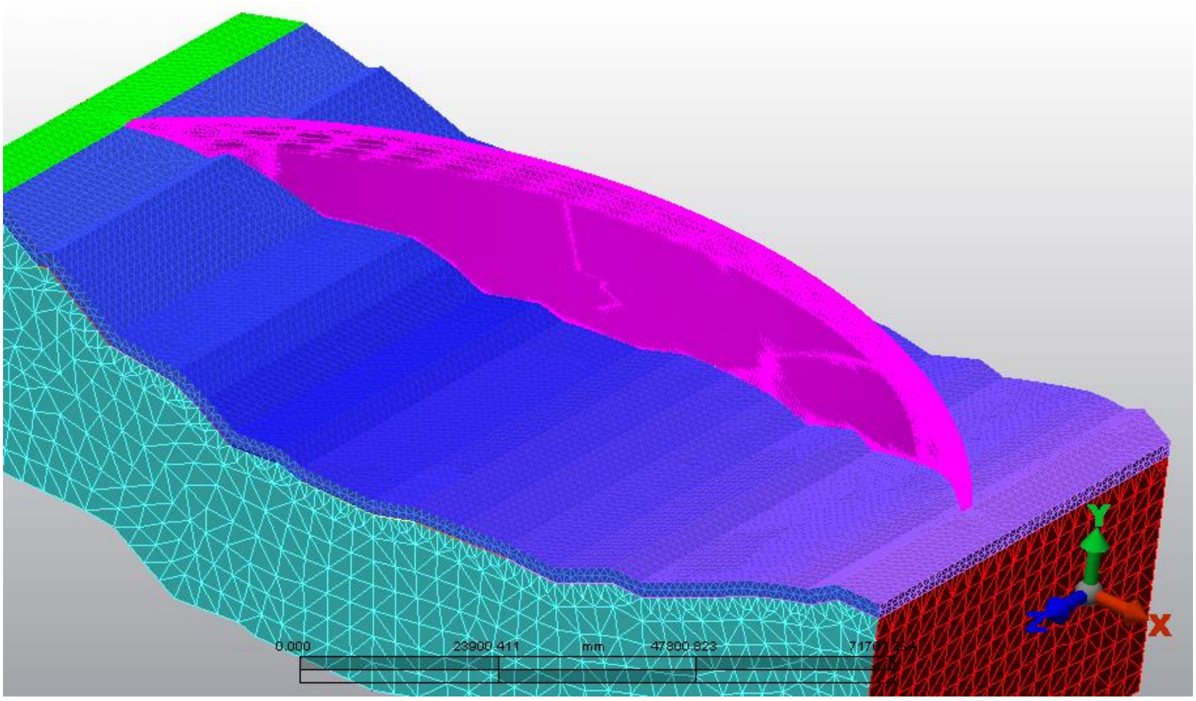
4.2 Methodology

The structural integrity of the existing concrete arch dam was assessed using a 3-D linear elastic finite element analysis with the aid of a commercially available software package – ALGOR.

The foundation profile as shown in Appendix A and cross-section of the dam was taken from the original longitudinal section drawing dated about 1898 and the DoC (2008) design drawings. The dam model was meshed into 3-dimensional, 2nd order tetrahedral elements. Figure 12 shows the dam and rock foundation model used for this analysis.

Since the dam was first built, it has overtopped on several occasions. However, the maximum flood level to-date was not recorded in any of the documents reviewed. For the purpose of this study and in order to determine the operating stresses conservatively, the stresses at FSL were estimated.

Figure 12 Redbank Creek Dam Finite Element Model



4.3 Material Properties used in the Analysis

The adopted material properties used in this present analysis are summarised in Table 5. The basis for the selection of these values is discussed in Sections 3.2 and 3.3 above.

Table 5 Adopted Material Properties

Property	Value
Concrete	
Density (average density of the dam concrete and plum aggregate)	2 400 kg/m^3
Characteristic Compressive Strength, f'_c	11 MPa
Apparent Tensile Strength of Intact Concrete, f'_{ct}	1.0 MPa
Apparent Tensile Strength in the horizontal joints, $f'_{ct,\ joint}$	0.34 MPa
Modulus of Elasticity, E_c	25 GPa
Poisson's Ratio, ν_c	0.2
Concrete frictional angle, ϕ_c	45^o
Intact Concrete cohesion, $c_{c(intact)}$	1,100 kPa
Lift Joint Concrete cohesion, $c_{c(lift\_joint)}$	340 kPa

Property	Value
Dam/Rock Foundation Interface	
Apparent Tensile Strength $f'_{ct,\,rock}$	0.2 MPa [1]
Frictional Angle, ϕ_{rc}	41 ^{o}
Cohesion, c_r	0.12 MPa

[1] derived from linear Mohr Coulomb failure envelope.

4.4 Applied Loading

4.4.1 Structural Self-Weight

The structural self-weight of the dam body was considered in both finite element and gravity analyses.

4.4.2 Hydrostatic Pressure

The hydrostatic pressure was calculated depending on the water level. The water pressure due to the water storage in the reservoir was applied normal to the upstream face of the dam.

4.4.3 Silt Loading

Significant silt deposition has occurred within Redbank Creek Dam since it was constructed. The silt acts as an additional static pressure load on the upstream face of the dam. The recent survey by R.J. Crooks & Associates indicated an average of 4 m of top soil including silt deposition at the lowest section of the reservoir floor. The soil bulk unit weight is taken as 15 kN/m^3 and the frictional angle is taken as 25^o.

4.5 Calculation of Uplift Pressure

Uplift pressures or seepage pressures under a dam arise when water seeps through the foundations. Over time a steady seepage condition results. Ideally, with no drains under the dam, the steady seepage condition would show that the drop in pressure from dam heel to toe would be a straight line from the reservoir level to tailwater level. For the downstream stilling basin, the steady state seepage condition is roughly at the groundwater level profile.

4.6 Cracking Propagation at Dam/Foundation Interface

When loads on the dam cause tensile stresses to increase above the limit for the concrete or foundation rock a crack will form. During the computerized 3-D stress evaluation, the crack is inserted based upon the resultant stresses. Full reservoir pressure is then applied to the crack length. The stresses in the dam are then re-analysed until all the tensile stresses within the dam foundation/concrete interface are within the acceptance range.

4.7 Assumptions

The analyses were based on the following assumptions:

- The profile of the dam was assumed to be in accordance with that shown in the drawings included under Appendix A and Appendix B;
- The materials, both concrete and rock foundation, act isotropically, are essentially homogeneous, and behave in a linear-elastic manner;
- Artificial crack elements were modelled between the dam and foundation interface. Uplift was applied within the upstream cracked zone. Note: since the finite element analysis was conducted using the linear elastic theory, the extent of cracking is approximate only.
- The analysis is based on the total factor of safety concept. Therefore, material reduction factors and load factors (as used for partial factor of safety analysis or ultimate limit state computations) were not applied to the loadings acting on the concrete dam – instead un-factored loads were applied.

4.8 Results

This section presents the results of the finite element analyses for the existing concrete dam. In the assessment of the stresses in the dam, concentrations of stresses that were the result of the mesh configuration (e.g. acute angle in the mesh, thin slices and sudden changes in geometry) have been ignored. Instead, more uniform values from the model were considered for comparison with the maximum stress capacity of the dam.

The results of the finite element analyses are shown in Table 6, while the stress plots are presented in Appendix C. The maximum tensile stresses at the dam/foundation interface occurred at the deeper sections of the dam for all load cases. The elements in the model were allowed to crack in the areas of high tensile stresses at the dam/foundation interface, until all the remaining tensile stresses fell below the design strengths given in Table 5.

Table 6 Computed Stresses within the Existing Dam at FSL

	Computed Value	Most Likely Strength Capacity[(a)]	Likely Factor of Safety
Concrete dam			
Maximum principal tensile stress (intact concrete)	0.8 MPa	1.0 MPa	1.25
Maximum principal tensile stress (lift joint)	0.3 MPa	0.34 MPa	1.1
Maximum principal compressive stress	2.3 MPa	11 MPa	4.8
Maximum vertical tensile stress (intact concrete)	0.4 MPa	1.0 MPa	2.5
Maximum vertical tensile stress (lift joint)	0.1 MPa	0.34 MPa	3.4

	Computed Value	Most Likely Strength Capacity[(a)]	Likely Factor of Safety
<u>Dam-foundation interface</u>			
Maximum vertical tensile stress	0.2 MPa*	0.2 MPa	1
Maximum vertical compressive stress	2.3 MPa	11 MPa	4.8

[(a)] refer to Table 5 and Sections 3.2 and 3.3

** denotes computed value after cracking*

Table 7 Likely Stability State of the Existing Dam at FSL

	Value
Frictional Sliding Resistance	**1.21**
Maximum extent of crack through thickness along foundation	~ 80%
Maximum Displacement	11.3 mm

4.9 Discussion

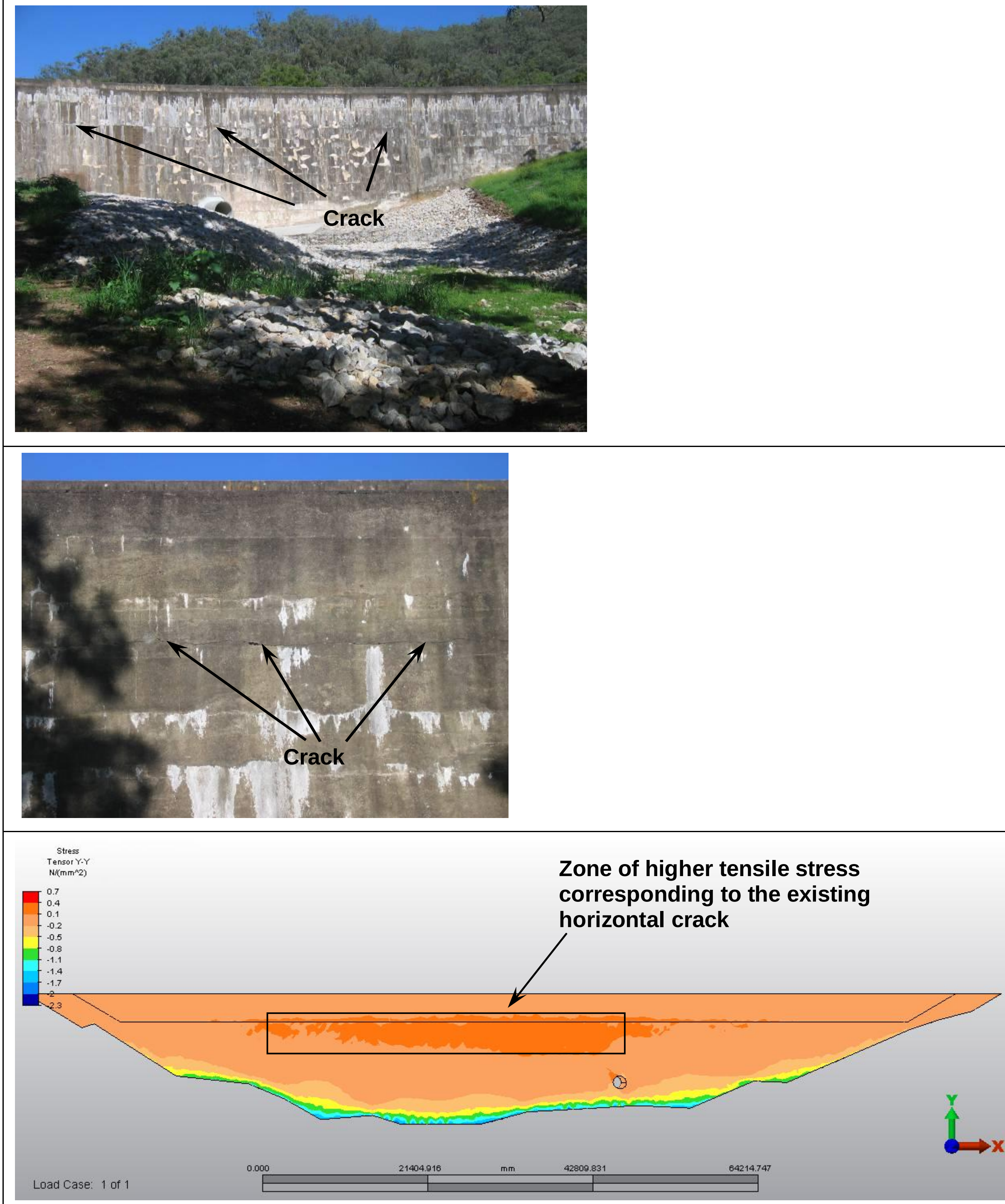
The structural integrity and stability assessment confirms that the existing dam is unable to satisfy current acceptance criteria, particularly the stability limit state. This present study concluded that the frictional sliding resistance is approximately 1.21 which is below that recommended by most dam authorities and regulators, as shown in Table 9 (Section 5.5 below). The factor of safety estimated from this study is below that of GHD (2002), as that study adopted a tensile strength of 1.0 MPa across the dam foundation interface, which resulted in less crack propagation along the interface. The value estimated for the present study however appears to be reasonable for the FSL loading case, since the dam has remained intact even though it has been overtopped.

For strength limit state, whilst all the stresses are below the most likely strength capacity of the dam and satisfy the requirements of dam authorities, the factor of safety for the dam foundation interface tensile strength is nevertheless considered to be marginal.

Based on the finite element analysis, it appears that the visible horizontal crack at about 3.7 m below the crest (refer to Figure 13 below) is likely to be as a result of the loading imposed by the impounded water.

Only modest tensile strengths are required in the existing dam (particularly at joints and foundation interface) to give stability at FSL. The dam has superior strengths to those calculated, since it has survived the higher loading to which it has been subjected during overtopping events.

Figure 13 Redbank Creek Dam Horizontal Crack



5. FINITE ELEMENT ANALYSIS OF THE PROPOSED DESIGN

5.1 General

As required by the scope of work, this study conducted a finite element stress analyses for a 3.76 m reduction in spillway level (and therefore a reduced overall height of the dam), as designed by the Department of Commerce (DoC), apart from the installation of vertical post tensioned anchors, which were not included in the evaluation. The DoC (2008) design drawings are included in Appendix B. The proposed design included 21 sub-vertical post tensioned anchors installed at 60^{o} from the horizontal through the downstream face of the dam. The first anchor is to be installed at chainage 116.398m and last anchor is located at chainage 39.698m. The locations of the post-tensioned anchors were obtained from DoC Design Drawing 0800112-05 Revision A and are tabulated in Table 8 below:

Table 8 Location of Post Tensioned Anchors

No.	Level RL m
1	530.65
2	529.4
3	528.35
4	528.22
5	528.37
6	528.01
7	527.75
8	527.59
9	527.68
10	527.66
11	527.84

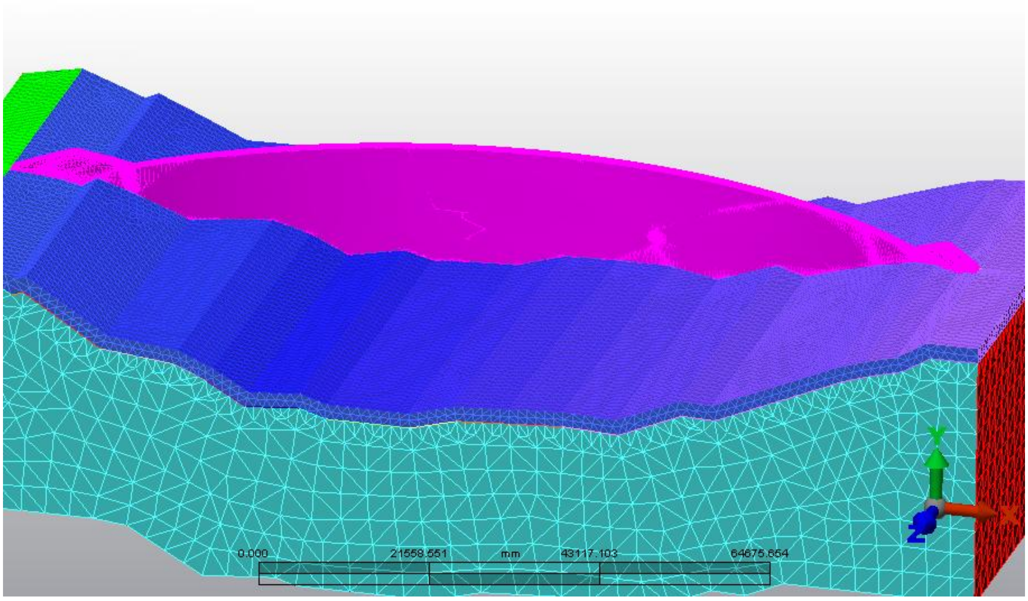
No.	Level RL m
12	528.41
13	528.26
14	528.4
15	528.53
16	528.51
17	528.85
18	529.02
19	528.55
20	530.3
21	530.4

The safety of the dam was evaluated for 1 in 100,000 AEP and PMP floods.

5.2 Methodology, Adopted Material Parameters, Assumptions and Loadings

The methodology, adopted material parameters, assumptions and loadings used in the analysis are similar to those described in Section 4.2, 4.3, 4.4 and 4.7 respectively. The model used in Section 4.2 was modified and the top 3.76 m of the dam was removed, as shown in Figure 14 below.

Figure 14 Redbank Creek Dam – 3.76 m Crest Cut Down Finite Element Model



5.3 Calculation of Uplift Pressure

When floods occur, they are usually short term events. The traditional conservative approach is to assume there is an instantaneous change along the whole of the dam/foundation interface resulting in a straight line pressure distribution from the reservoir flood level to the tailwater flood level. This assumption, however, is unrealistically conservative as uplift pressures at the dam/foundation interface are unlikely to respond instantaneously to rises in water level and the flood water levels are unlikely to be high for a sufficiently long period for full uplift pressures to develop.

In addition, a relatively thick layer of silt has been deposited in the reservoir bed adjacent to the upstream face of the dam. The reservoir silt acts as a barrier to the seepage of water into the dam foundation and thus potentially reduces the uplift pressures under the dam. This fact has been recognized by FERC (2002). The true uplift beneath the dam can be determined through a transient seepage analysis depending on the reservoir impounding duration.

In this study, two cases for uplift were considered:

- Uplift Pressure Distribution Type 1

  The first case assumes that full uplift (equivalent to the water head) is developed in the cracked section of the dam foundation interface.

- Uplift Pressure Distribution Type 2

  The second case assumes a straight line uplift pressure distribution, equivalent to the reservoir flood level on the upstream heel, while at the downstream toe, pressure is equal to the tailwater level, regardless of whether the interface was cracked or not.

5.4 Loading Cases

Three loading cases were considered, namely:

Case 1: loading corresponding to maximum 1 in 100,000 AEP flood peak water level with type 1 uplift profile;

Case 2: loading corresponding to maximum 1 in 100,000 AEP flood maximum peak water level with type 2 uplift profile; and

Case 3: loading corresponding to PMP flood peak water level with type 2 uplift profile.

5.5 Assessment Criteria

The structural integrity of the dam was assessed in accordance with recommendations made by national and international authorities for dam engineering. Table 5 presents the factors of safety proposed by selected dam authorities.

Table 9 Factors of Safety Required by Various Dam Authorities

Property	Guideline and Recommended Factors of Safety				
	USBR (1977)	USACE (1994)	FERC (1999) (Note 4)	FERC (2002)	
				(Note 4)	(Note 5)
Compression					
Usual	3.0 (Note 1) 4.0 (Note 2)	4.0	2.0	< Design Strength	
Unusual	2.0 (Note 1) 2.7 (Note 2)	2.5	1.5	< Design Strength	
Extreme	1.0 (Note 1) 1.3 (Note 2)	1.5	1.1	< Design Strength	
Tension					
Usual	< 1 MPa	N/A	1.0	1.0	< Design Strength
Unusual	< 1.55 MPa	N/A	1.0	1.0	< Design Strength
Extreme	< Design Strength	2.0	1.0	1.0	< Design Strength
Shear strength					
Usual	3.0 (Notes 1 & 2) 4.0 (Note 3)	2.0	2.0	< Design Strength	
Unusual	2.0 (Notes 1 & 2) 2.7 (Note 3)	1.3	1.5	< Design Strength	
Extreme	1.0 (Notes 1 & 2) 1.3 (Note 3)	2.0	1.1	< Design Strength	

Property	Guideline and Recommended Factors of Safety				
	USBR (1977)	**USACE (1994)**	**FERC (1999)** (Note 4)	**FERC (2002)**	
				(Note 4)	(Note 5)
Sliding Stability					
Usual	3.0 (Notes 1 & 2) 4.0 (Note 3)	2.0	1.5	1.5	3.0
Unusual	2.0 (Notes 1 & 2) 2.7 (Note 3)	1.3	1.5	1.5	2.0
Extreme	1.0 (Notes 1 & 2) 1.3 (Note 3)	1.1	1.1	1.3	1.3

Notes:

(1) Concrete
(2) Foundation contact
(3) Foundation weak plane
(4) With no cohesion
(5) With cohesion

5.5.1 Sliding Stability

The basic shear-friction sliding safety factor (*SSF*) of the dam along its foundation was assessed using the formula given as follows:

$$SSF = \frac{(F_N * \tan\phi) + C * Ac}{H}$$

where,

F_N: Normal force

ϕ: Angle of friction (peak value or residual value)

C: Cohesion

A_c: Area in compression

H: Sum of horizontal forces

In accordance with FERC (1999) for arch dams, the sliding safety factor is computed based on a frictional analysis, while the cohesion within the sliding plane is not relied upon.

5.6 Analysis Results and Evaluation

This section presents the results of the finite element analyses for each loading case. In the assessment of the stresses in the dam, concentrations of stresses that were the result of the mesh configuration (e.g. acute angle in the mesh, thin slices and sudden changes in geometry) have been ignored. Instead, more uniform values from the model were considered for comparison with the maximum stress capacity of the dam.

The results of the finite element analyses are shown in Appendix D. The maximum tensile stresses at the dam/foundation interface occurred at the deeper sections of the dam for all load cases. The elements in the model were allowed to crack in the areas of high tensile stresses at the dam/foundation interface, until all the remaining tensile stresses fell below the design strengths given in Section 5.5.

5.6.1 Case 1: 1 in 100,000 AEP Flood Peak Water Level with Type 1 Uplift Profile

Table 10 Results for Case 1: 1 in 100,000 AEP flood peak water level with type 1 uplift profile

	Computed Value	Strength Capacity[(a)]	Factor of Safety	Required Factor of Safety[(b)]	
Concrete dam					
Maximum principal tensile stress (intact concrete)	0.4 MPa	1.0 MPa	2.5	1.0	OK
Maximum principal tensile stress (lift joint)	0.3 MPa	0.34 MPa	1.1	1.0	OK
Maximum principal compressive stress	1.8 MPa	11 MPa	6.1	1.5	OK
Maximum vertical tensile stress	<0.1 MPa				
Dam-foundation interface					
Maximum vertical tensile stress	0	0.2 MPa	-	-	OK
Maximum vertical compressive stress	1.2 MPa	11 MPa	9.2	1.5	OK
Frictional Sliding Resistance			1.75	1.5	OK
Maximum Displacement				3.3 mm	

[(a)] refer to Table 5 and Sections 3.2 and 3.3

[(b)] adopted factor of safety based on FERC (1999)

** denotes computed value after cracking*

5.6.2 Case 2: 1 in 100,000 AEP Flood Peak Water Level with Type 2 Uplift Profile

Table 11 Results for Case 2: 1 in 100,000 AEP flood peak water level with type 2 uplift profile

	Computed Value	Strength Capacity[a]	Factor of Safety	Required Factor of Safety[b]	
Concrete dam					
Maximum principal tensile stress (intact concrete)	0.4 MPa	1.0 MPa	2.5	1.0	OK
Maximum principal tensile stress (lift joint)	0.2 MPa	0.34 MPa	1.7	1.0	OK
Maximum principal compressive stress	1.5 MPa	11 MPa	7.3	1.5	OK
Maximum vertical tensile stress	<0.1 MPa				
Dam-foundation interface					
Maximum vertical tensile stress	0*	0.2 MPa	-	-	OK
Maximum vertical compressive stress	1.14 MPa	11 MPa	9.6	1.5	OK
Frictional Sliding Resistance			2.25	1.5	OK
Maximum Displacement				2.3 mm	

[a] refer to Table 5 and Sections 3.2 and 3.3

[b] adopted factor of safety based on FERC (1999)

** denotes computed value after cracking*

5.6.3 Case 3: PMP Flood Peak Water Level with Type 2 Uplift Profile

Table 12 Results for Case 2: PMP flood peak water level with type 2 uplift profile

	Computed Value	Strength Capacity[(a)]	Factor of Safety	Required Factor of Safety[(b)]	
Concrete dam					
Maximum principal tensile stress (intact concrete)	0.41 MPa	1.0 MPa	2.4	1.0	OK
Maximum principal tensile stress (lift joint)	0.2 MPa	0.34 MPa	1.7	1.0	OK
Maximum principal compressive stress	1.95 MPa	11 MPa	5.6	1.5	OK
Maximum vertical tensile stress	<0.1 MPa				
Dam-foundation interface					
Maximum vertical tensile stress	0	0.2 MPa	-	-	OK
Maximum vertical compressive stress	1.3 MPa	11 MPa	8.5	1.5	OK
Frictional Sliding Resistance			2.08	1.5	OK
Maximum Displacement				3.1 mm	

[(a)] refer to Table 5 and Sections 3.2 and 3.3

[(b)] adopted factor of safety based on FERC (1999)

** denotes computed value after cracking*

5.7 Discussion

After completion of the proposed remedial works, which include lowering the full supply level of the dam by 3.76 m and installing inclined anchors at the toe of the dam, the finite element analysis shows that the dam will satisfy the modern design criteria set by FERC, USACE and USBR. The results of this study are generally in agreement with those of DoC (2008). The results indicate the following:

- The transient stresses that develop during flooding at the horizontal lift joints under the relatively conservative uplift assumptions are typically less than or equal to those which the dam has previously withstood, The stresses that the dam has previously withstood have not resulted in significant distress.
- The installation of the inclined anchors results in no tension development at the upstream heel of the dam.

- The structure will comply with sliding resistance criteria.
- Vertical tensile stresses at joints are typically less than 0.1 MPa, which the concrete should easily be able to cope with.

It is concluded that the vertical post tensioned anchors in the body of the dam are not required for the following reasons:

- The transient stresses during flooding do not exceed and are generally less than those previously withstood for long periods when the dam reservoir was at FSL,
- The reservoir level during PMP inflows rises to a level which is approximately 2.1m below the original full supply level and therefore stresses will never exceed those previously experienced by the dam,
- Vertical tensile stresses at joints are typically less than 0.1 MPa,
- Flood events will result in short-term loading of the structure,
- The dam profile is generally thicker, with the removal of the thinnest sections, which will result in a more robust dam section.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

An evaluation of the stresses that the dam has historically withstood without signs of significant distress has been undertaken. These stresses have been compared to the stresses that the dam may be expected to experience under the 1:100,000 AEP and PMP flood, subsequent to remedial works, which will include:

- Reducing the height of the spillway and dam crest by 3.76 m, and
- Installing inclined anchors (60^{o} to the horizontal) at the downstream toe of the dam.

The evaluation has concluded that the stresses that the dam will experience under extreme flood events after implementation of the remedial works will remain lower than those previously experienced during operation of the dam.

Associated with an early warning system, it is recommended that the works listed above are implemented.

6.2 Recommendations Regarding Implementation of Balance of DoC Design

Additional works that were included in the DoC design (2008) are the following:

1. Install vertical anchors between the lowered crest and foundation over the highest sections of the dam,
2. Install a foundation drainage system, consisting of inclined holes drilled from the downstream toe into the dam foundations,
3. Install vertical riser overflow pipe,
4. Repair of horizontal construction joints by gouging out and filling with Xypex slurry and grouting of vertical cracks,
5. Painting the upstream face of the dam with a Xypex concentrate slurry.
6. Provide a protective apron downstream of the dam within the central river section,
7. Crest Concrete Capping,
8. Installation of a safety fence on the crest of the dam

6.2.1 Recommended Works

It is recommended that the following items are implemented:

- Item 7 – **Concrete capping**: This work is required purely for aesthetic reasons, and need only be done to provide the crest with a neat appearance.
- Item 8 – **security fence and gate**: These are required to prevent unauthorised access onto the dam crest.

6.2.2 Works Not Recommended

It is recommended that items 1 though 5 are not implemented at this time for the following reasons:

1. **Vertical Anchors**: The tensile stresses in the dam remain below those previously experienced, even at PMP inflows, and the dam no longer stores water. Stresses in construction joints during extreme flood events are typically < 0.1 MPa. It is therefore concluded that the vertical anchors are not required.
2. **Foundation drainage system**: Since the dam does not normally store water, the pressures in the foundation will be low. When floods pass through the dam and the water level rises, the uplift pressures in the foundation may increase but, since the water level in the dam will remain high for a relatively short period, the uplift pressures are unlikely to increase to the full hydrostatic head. The silt on the foundation upstream of the dam will also reduce the rate of seepage into the foundations, resulting in lower foundation interface pressures. The prestress force introduced onto the foundations will close any cracks which may have previously formed, reducing the area over which uplift pressures could develop. The drains will therefore be of limited value and may block in view of the expectation that no flow will exit from them under normal operating conditions.
3. **Vertical Riser Overflow Pipe**: The vertical overflow riser pipe is not required in view of the fact that the dam is no longer required to store water,
4. **Repair of horizontal construction joints and vertical cracks**: Since the reservoir is no longer required to permanently store water, there is no need to seal the joints and cracks.
5. **Painting of upstream face with Xypex**: Since the reservoir is not required to store water, there is no need to seal the upstream face of the dam.
6. **Provision of Apron**: The dam will only overtop at floods with a return period in excess of 950 years, with the result that the apron will seldom be required. However, it is recommended that rip rap (which could include rubble concrete obtained from cutting down the crest of the dam) should be placed in a 4.0 m wide strip immediately downstream of the dam to provide protection against erosion of the fill and foundation rock in the event the dam does overtop. Council should inspect the dam after storm events that overtop the dam and repair any resulting erosion.

6.3 Additional Recommendations

It is further recommended that a crack monitoring programme be implemented, in association with a programme to monitor the degradation of the concrete of the dam wall.

The crack monitoring programme could consist of the taking of photographs of the upstream and downstream faces of the dam on a hot day during summer and a cold day during winter and comparing successive sets of photographs to evaluate the number of cracks and similar deficiencies noted and whether these have increased in number or size.

Monitoring of the concrete degradation could consist of annual monitoring of pre-selected areas of the dam (say 3 upstream and 3 downstream) and comparing degradation over time. Monitoring for degradation should include photographing each area and tapping selected spots within the area lightly with a hammer. This may be supported by a concrete coring and testing programme undertaken once every ten to twenty years.

7. References

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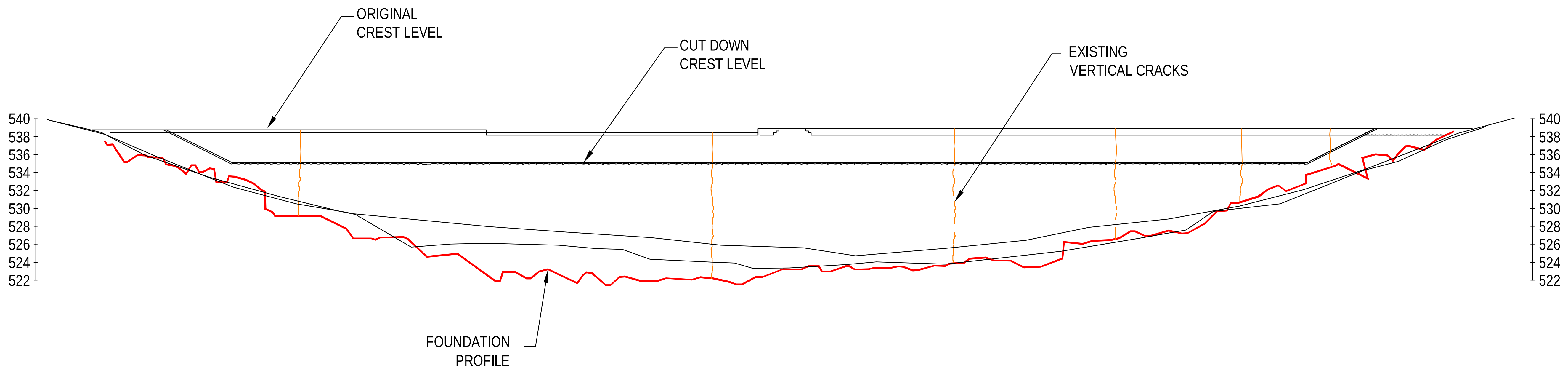
Appendix A

Dam Foundation Profile



SECTION
SCALE 1:500

0 5 10 15m
SCALE 1:500 AT ORIGINAL SIZE

CLIENTS | PEOPLE | PERFORMANCE

Redbank Creek Dam

Finite Element Analysis
Section

Job Number	21-19151
Revision	A
Date	June 2010

Figure 1

57-63 Herbert Street Artarmon NSW 2064 Australia T 61 2 9462 4700 F 61 2 9462 4710 E atnmail@ghd.com.au W www.ghd.com.au

Plot Date: 2 June 2010 - 3:14 PM Plotted by: Helen Warr/Sydney/GHD/AU Cad File No: G:\21\19151\CADD\Drawings\21-19151-FIG001.dwg

Appendix B

Design Drawings (DoC) 2008

Department of Commerce (2008) Drawing No:

0800112-01 Revision A – General Arrangement

0800112-02 Revision A – Concrete & Reinforcement Details

0800112-03 Revision A – Primary Spillway Arrangement & Details

0800112-04 Revision A – Vertical Post Tensioning from Crest

0800112-05 Revision A – Sub Vertical Post Tensioning from DS

0800112-06 Revision A – Treatment of US face

0800112-07 Revision A – Trash Rack and Safety Screen Details

0800112-08 Revision A – Existing Pipe Outlet Refurbishment

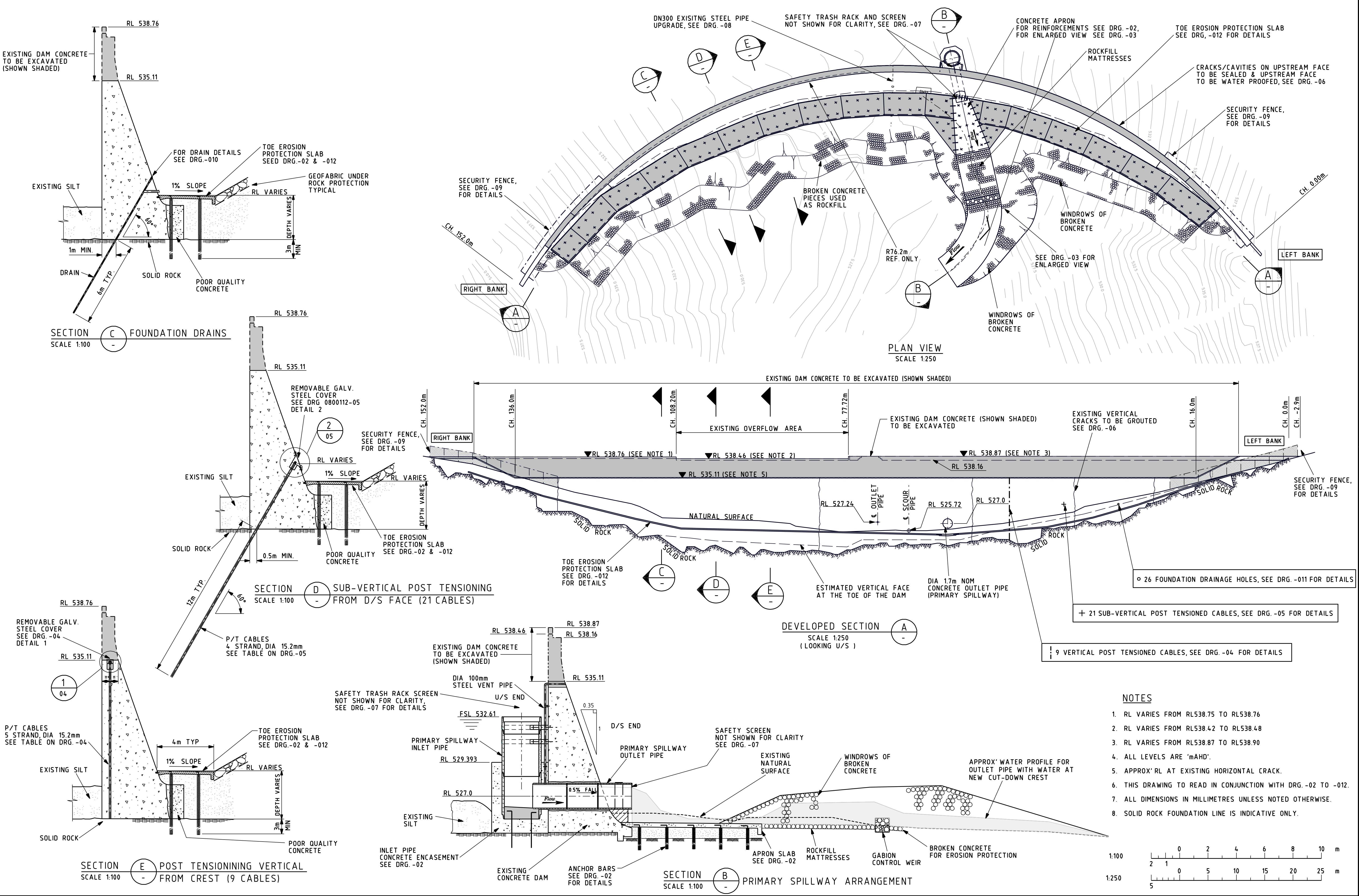
0800112-09 Revision A – Security Fence and Gate Details

0800112-10 Revision A – Foundation Drains

0800112-11 Revision A – Toe Erosion Protection Slab

0800112-12 Revision B – Crest Concrete Capping

0800112-13 Revision A – Standard Detail for Top of P/T Cable



NOTES

1. RL VARIES FROM RL538.75 TO RL538.76
2. RL VARIES FROM RL538.42 TO RL538.48
3. RL VARIES FROM RL538.87 TO RL538.90
4. ALL LEVELS ARE 'mAHD'.
5. APPROX' RL AT EXISTING HORIZONTAL CRACK.
6. THIS DRAWING TO READ IN CONJUNCTION WITH DRG.-02 TO -012.
7. ALL DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.
8. SOLID ROCK FOUNDATION LINE IS INDICATIVE ONLY.

MK	DETAILS OF AMENDMENTS	APPROVED	DATE
A	ISSUED FOR TENDER.	J.L.	04.11.08

DAVID CALLAHAN
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THE ORIGINAL OF THIS DRAWING WAS SIGNED BY THE PERSONS WHOSE NAMES APPEAR IN THE ADJACENT BLOCK

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TECHNOLOGY CONTROLLER
WATER TECHNOLOGIES

PLAN RM NO
DESIGNED A. UN-NABI
MANAGER DAMS AND CIVIL TECHNOLOGIES J.LENEHAN
DRAFTED D.T.
CHECKED B.COOPER

NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS
GENERAL ARRANGEMENT

NO IN SET
FILE NO.
DRAWING NO. 0800112-01
REVISION A

B1 0 960 100 200 300

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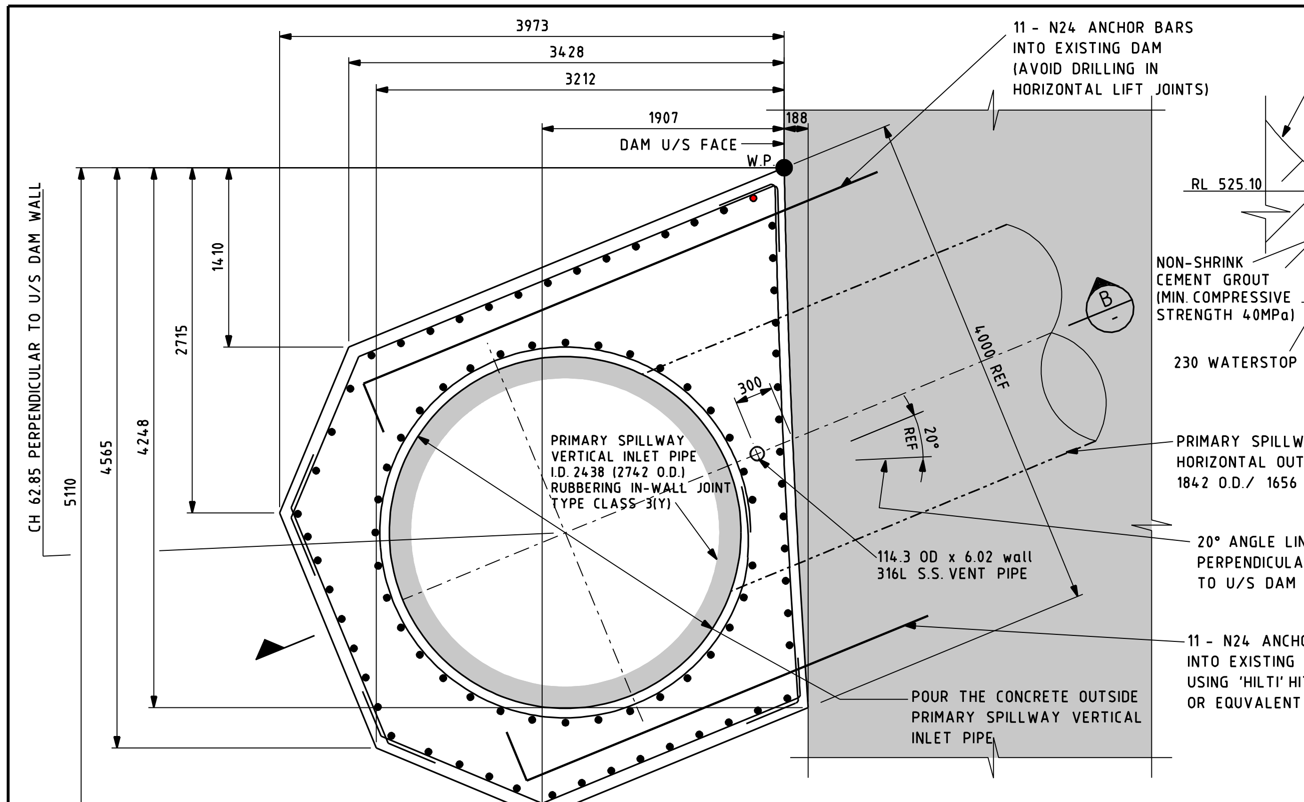
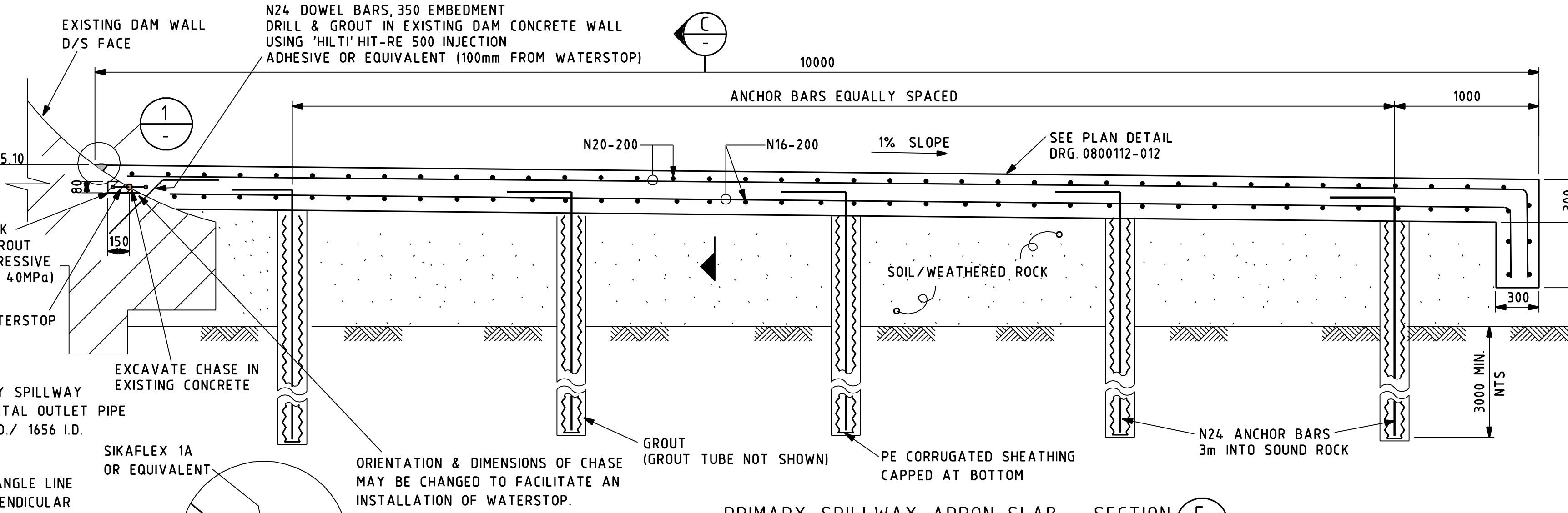
679



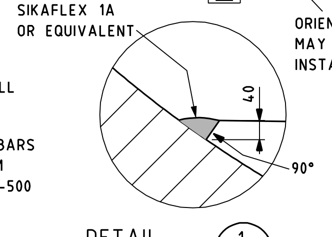
PRIMARY SPILLWAY VERTICAL PIPE CONCRETE ENCASEMENT A

SCALE 1:25



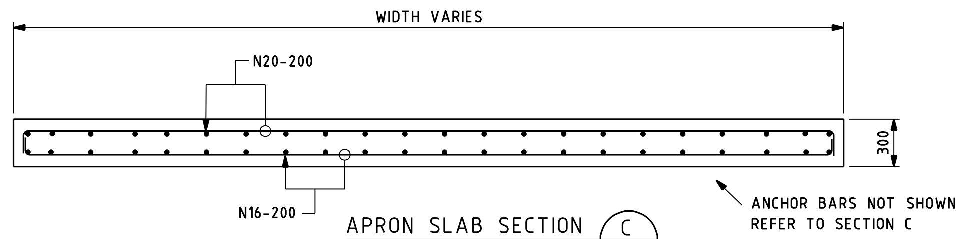
PRIMARY SPILLWAY APRON SLAB - SECTION F 012

SCALE 1:20



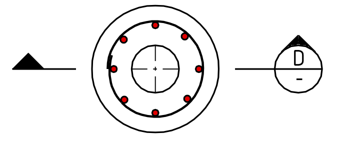
DETAIL 1

NOT TO SCALE



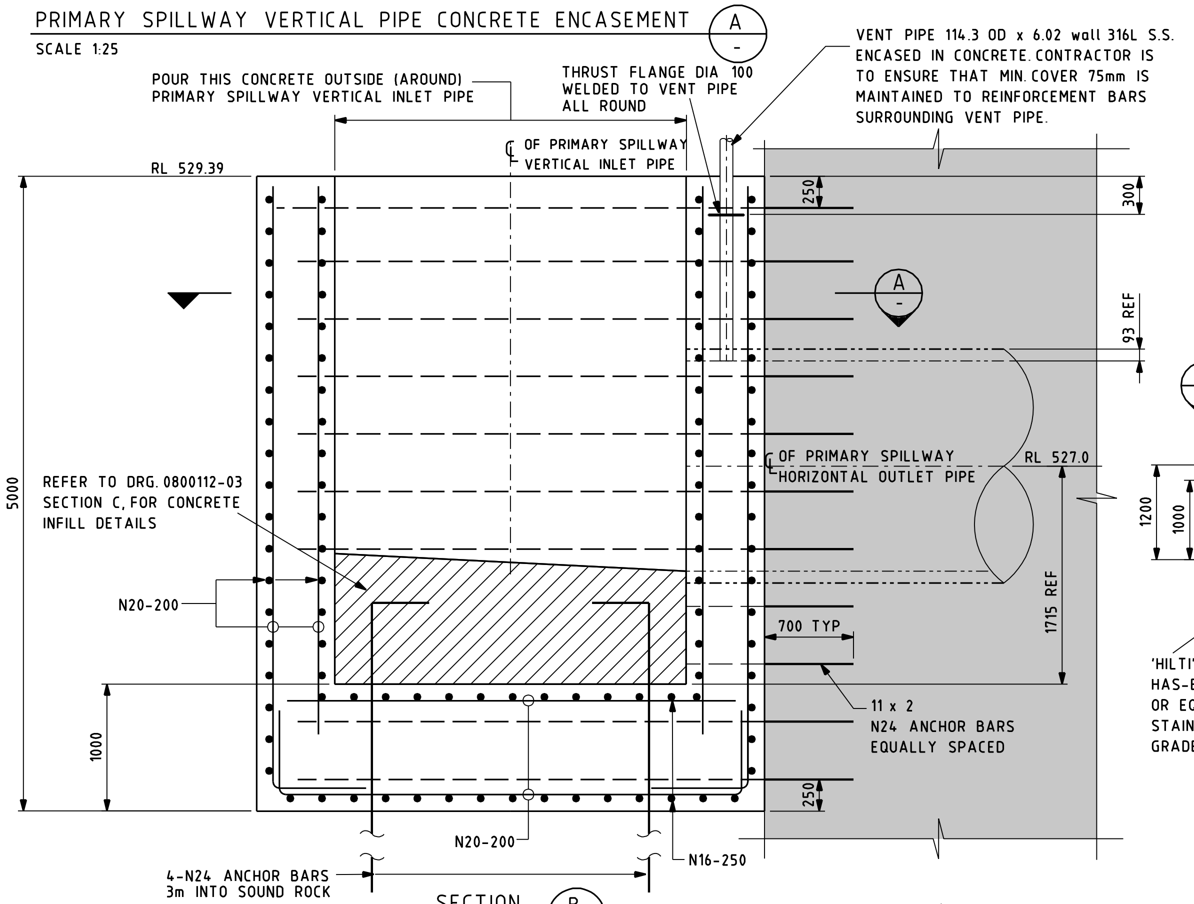
APRON SLAB SECTION C

SCALE 1:20



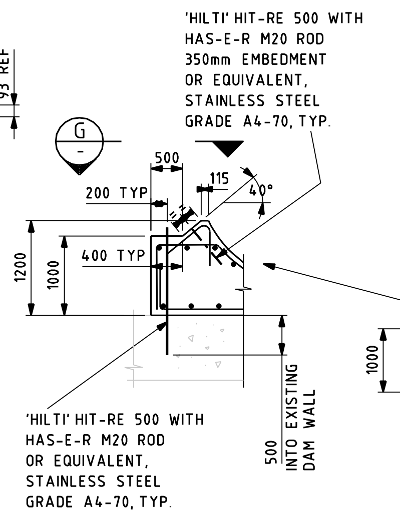
PRECAST P/T HEAD BLOCK REINFORCEMENT DETAIL E

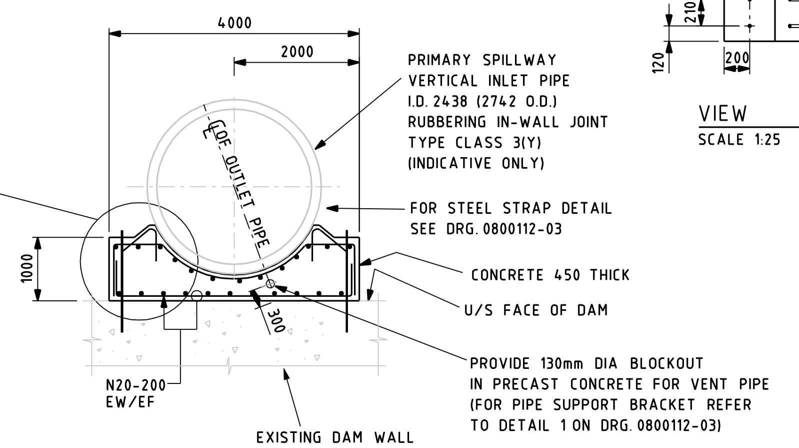
SCALE 1:5



SECTION B

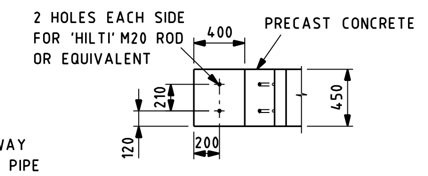
SCALE 1:25





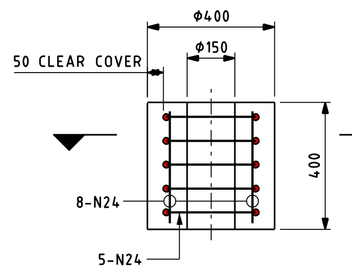
PRIMARY SPILLWAY INLET VERTICAL PRECAST CONCRETE PIPE SUPPORT

SCALE 1:50



VIEW G

SCALE 1:25

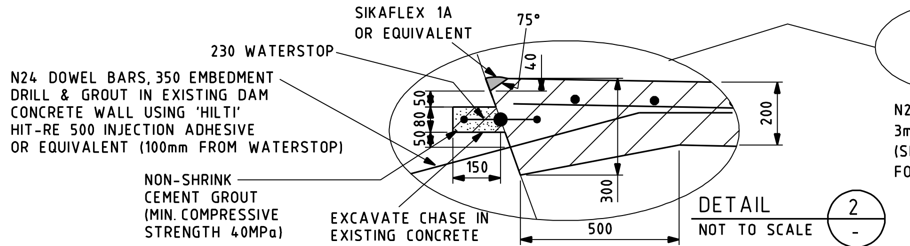


SECTION D

SCALE 1:5

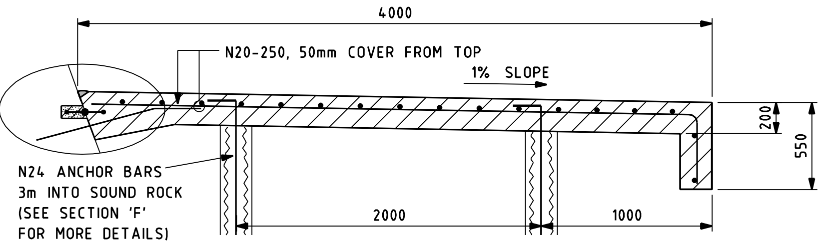
CONCRETE NOTES:

1. ALL LEVELS ARE TO AUSTRALIAN HEIGHT DATUM IN METRES.
2. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
3. REINFORCED CONCRETE GRADE SHALL BE S32 IN ACCORDANCE WITH THE SPECIFICATION, U.N.O. PRE-CAST REINFORCED CONCRETE GRADE SHALL BE S40 IN ACCORDANCE WITH THE SPECIFICATION, U.N.O.
4. INTERNAL AND EXTERNAL CORNERS SHALL HAVE TYPICAL 20mm FILLETS OR CHAMFERS, UNLESS NOTED OTHERWISE. ALL CONCRETE RECESSES FOR GRID DECKING SHALL BE PROVIDED WITHOUT FILLETS.
5. M.J. DENOTES MOVEMENT JOINT C.J. DENOTES CONSTRUCTION JOINT
6. THE LOCATION OF ANY ADDITIONAL CONSTRUCTION JOINTS TO BE DETERMINED BY THE CONTRACTOR AND TO BE SATISFACTORY TO THE PRINCIPAL.
7. PROVIDE 75mm CLEAR COVER FROM CONCRETE SURFACE TO NEAREST BAR, UNLESS OTHERWISE SHOWN.
8. REINFORCEMENT SHALL BE GRADE 500 & DUCTILITY CLASS 'N' IN ACCORDANCE WITH AS/NZS 4671, UNLESS NOTED OTHERWISE.
9. DISPLACE BARS WHERE NECESSARY TO AVOID EMBEDDED ITEMS, WHILST MAINTAINING THE CLEAR COVER.
10. REINFORCEMENT DEVELOPMENT LENGTH, HOOKS, COGS AND SPLICES ARE TO BE IN ACCORDANCE WITH AS3600 FOR THE FULL YIELD STRENGTH OF THE BAR, UNLESS OTHERWISE SHOWN. BENDS, HOOKS AND COGS SHALL BE FORMED WITHOUT STRAIGHTENING AND/OR REBENDING.
11. ALL REINFORCEMENT IS NOT NECESSARILY SHOWN IN ALL VIEWS.



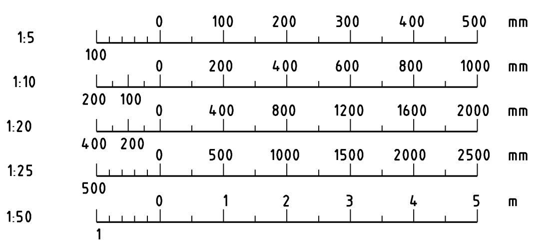
DETAIL 2

NOT TO SCALE



TOE EROSION PROTECTION SLAB

SCALE 1:20



THIS DRAWING TO BE READ IN CONJUNCTION WITH GENERAL ARRANGEMENT DRG. 0800112-01

MK	DETAILS OF AMENDMENTS	APPROVED	DATE
A	ISSUED FOR TENDER.	J.L.	04.11.08

DAVID CALLAHAN
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PLAN RM NO
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CHECKED B.COOPER

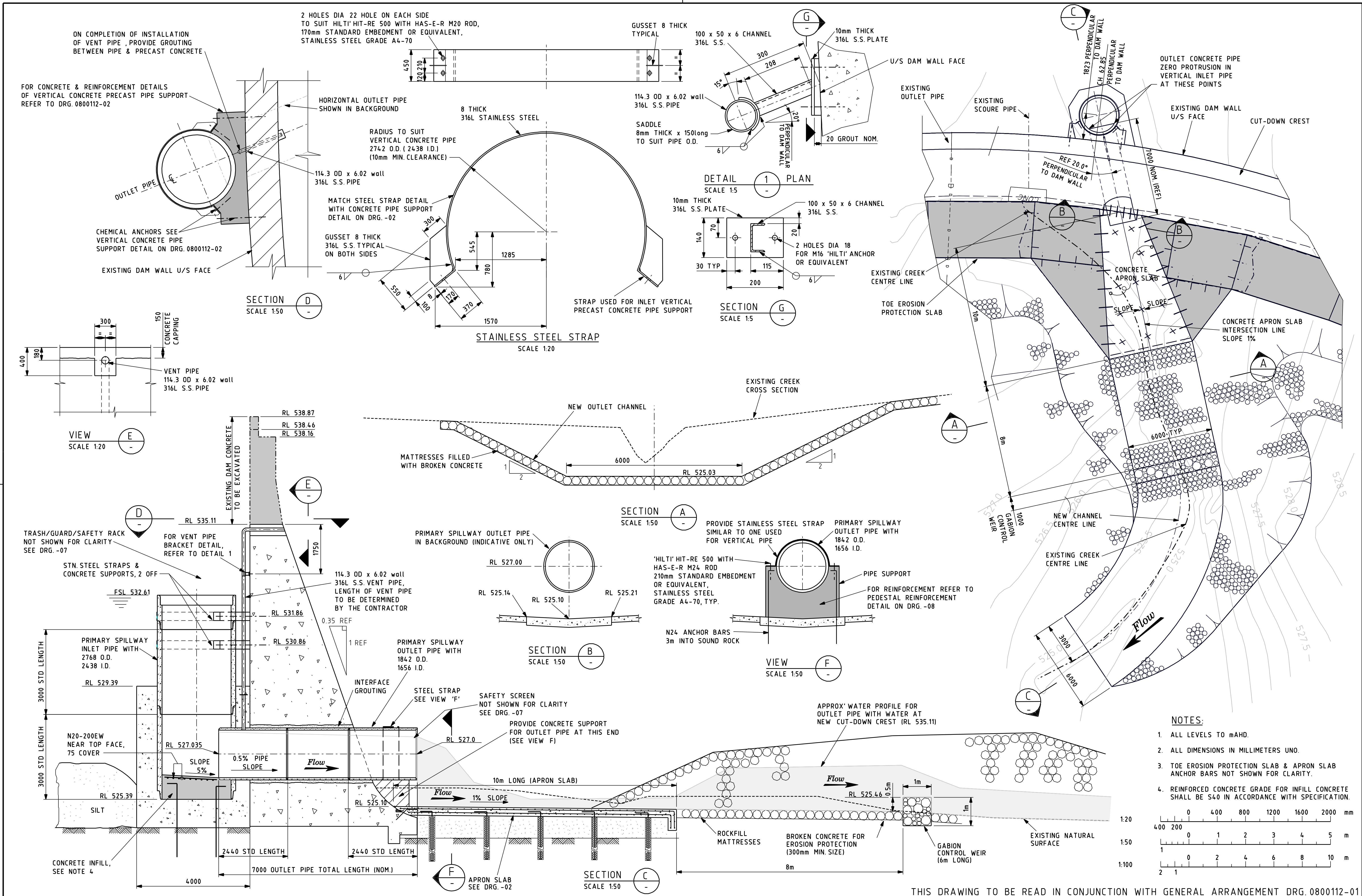
NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS
CONCRETE & REINFORCEMENT DETAILS

NO IN SET
FILE NO.
DRAWING NO. 0800112-02
REVISION A

B1 0 960 100 200 300 679

5/11/2008 1:13:13 PM g:\IS\Water-tec\Pcad\RedbankCk Dam06\2008MAR\PRC002.dgn



NOTES:

1. ALL LEVELS TO mAHD.
2. ALL DIMENSIONS IN MILLIMETERS UNO.
3. TOE EROSION PROTECTION SLAB & APRON SLAB ANCHOR BARS NOT SHOWN FOR CLARITY.
4. REINFORCED CONCRETE GRADE FOR INFILL CONCRETE SHALL BE S40 IN ACCORDANCE WITH SPECIFICATION.

1:20 0 400 800 1200 1600 2000 mm

1:50 0 1 2 3 4 5 m

1:100 0 2 4 6 8 10 m

THIS DRAWING TO BE READ IN CONJUNCTION WITH GENERAL ARRANGEMENT DRG. 0800112-01

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DRAFTED	D.T.
CHECKED	B.COOPER

NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS
PRIMARY SPILLWAY ARRANGEMENT & DETAILS

DRAWING No. 0800112-03
REVISION A

B1

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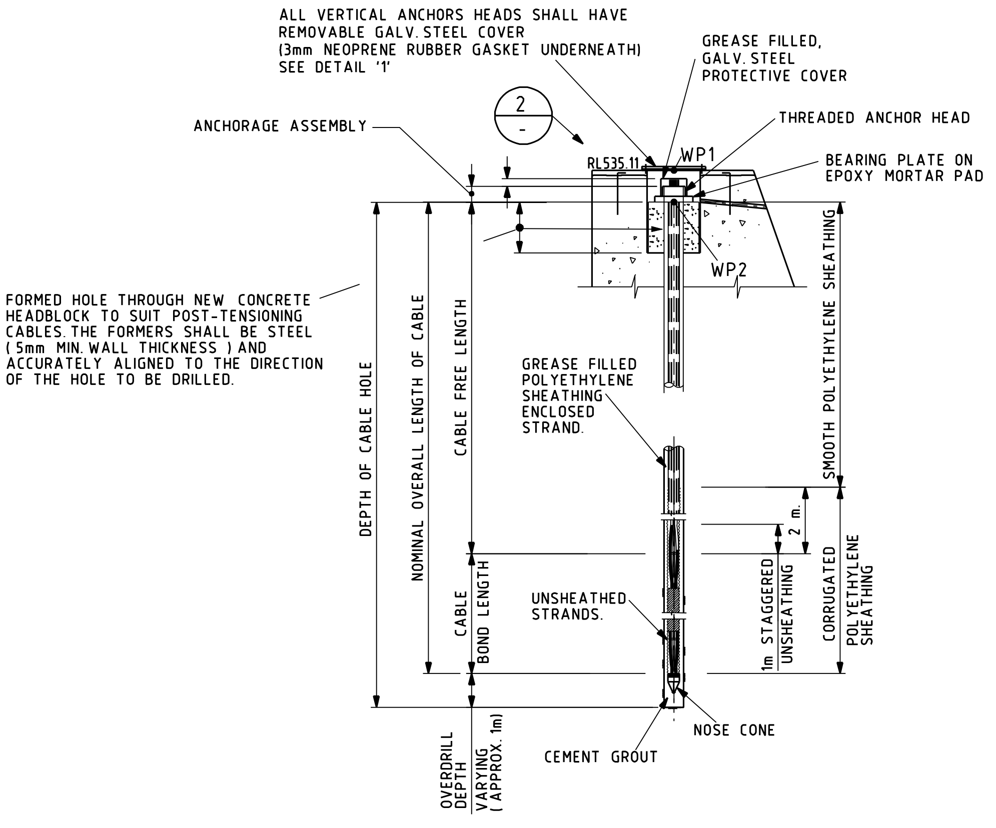
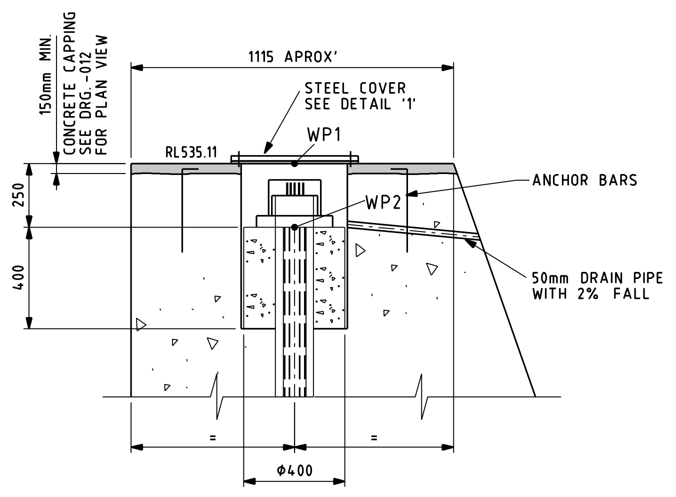
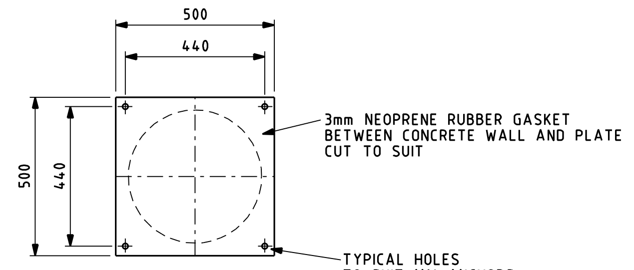


TYPICAL DETAIL
SHOWING CABLE COMPONENTS
(NOTE:- CABLE COMPONENTS ARE TYPICAL FOR ALL CABLES)
N.T.S



ENLARGED DETAIL 2
NOT TO SCALE



ANCHOR HEAD REMOVABLE GALV. STEEL COVER 1/01
SCALE 1:10

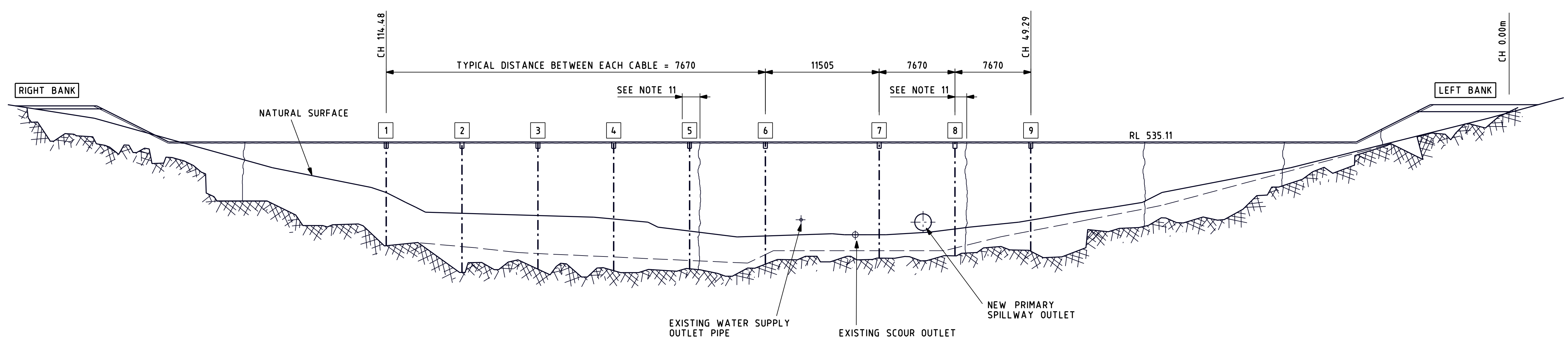
TABLE 1

P/T CABLES FROM CREST OF DAM (ALSO SEE NOTE 1)

CABLE LOCATION No.	REFERENCE RL FOR CABLE LAYOUT AT EXISTING CONC. SURFACE "WP1"	TOP OF CABLE HOLE "WP2" (RL)	ESTIMATED DRILL HOLE LENGTH (m) TOTAL LENGTH OF HOLE (m)	NEW CONCRETE HEADBLOCK FORMED HOLE	APPROX. DRILLING LENGTH THROUGH ROCK (m)	CABLE NOMINAL OVERALL LENGTH (m)	STRESSING ORDER
1	RL 535.11		12	0.15m	6	11	
2	RL 535.11		14	0.15m	6	13	
3	RL 535.11		14	0.15m	6	13	
4	RL 535.11		14	0.15m	6	13	
5	RL 535.11		14	0.15m	6	13	
6	RL 535.11		13	0.15m	6	12	
7	RL 535.11		13	0.15m	6	12	
8	RL 535.11		12	0.15m	6	11	
9	RL 535.11		12	0.15m	6	11	

NOTES

1. 9 POST-TENSIONING CABLES REQUIRED AT DAM CREST, WITH LOCATION, SIZE AND LOADS BEING:-
   i) CABLE LOCATIONS 1 TO 9 EACH CONSISTING OF 5-15.2mm DIA SUPERGRADE STRANDS, HAVING AN M.B.L. OF 1250 kN. DESIGN WORKING LOAD IS 750 kN (60% M.B.L.)
2. THE MINIMUM BOND LENGTHS SHALL BE 3m.
3. ALL RL's ARE TO mAHD.
4. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
5. ALL CABLES SHALL BE IDENTIFIED BY THE CABLE CODING ASSIGNED.
6. CONCRETE CHARACTERISTIC COMPRESSIVE STRENGTH FOR THE POST-TENSIONED CABLES HEAD BLOCKS SHALL BE 40 MPa AT 28 DAYS.
7. MINIMUM CLEAR COVER TO REINFORCEMENT SHALL BE 50 mm UNLESS OTHERWISE SHOWN.
8. WP1 DENOTES "WORKING POINT". THIS IS AN ESTIMATED RL ONLY. CONTRACTOR IS TO CONFIRM AT SITE FOR ACCURATE DRILLING ALIGNMENT.
9. "TABLE 1", DETAILS FOR "WP2" (RL) TO BE FILLED IN DURING CONSTRUCTION.
10. FOR TYPICAL REINFORCED CONCRETE CONSTRUCTION NOTES, SEE DRG. 0800112-02.
11. HOLE POSITION SHOULD BE ADJUSTED TO GIVE A MINIMUM CENTRLINE DISTANCE OF 0.5m FROM CRACK.
12. STRESSING ORDER (SEE IN TABLE '1') TO BE PROVIDED BY THE CONTRACTOR AND TO BE SATISTAFACTORY TO THE PRINCIPAL.



VERTICAL POST TENSIONING FROM CREST
SCALE 1:250

1:20 0 400 800 1200 1600 2000 mm 400 200
1:250 0 5 10 15 20 25 m 5

THIS DRAWING TO BE READ IN CONJUNCTION WITH GENERAL ARRANGEMENT DRG. 0800112-01

MK	DETAILS OF AMENDMENTS	APPROVED	DATE
A	ISSUED FOR TENDER.	J.L.	04.11.08

DAVID CALLAHAN
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PLAN RM NO
DESIGNED A. UN-NABI
MANAGER DAMS AND CIVIL TECHNOLOGIES J.LENEHAN
DRAFTED D.T.
CHECKED B.COOPER

NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS
VERTICAL POST TENSIONING FROM CREST

NO IN SET
FILE NO.
DRAWING NO. 0800112-04
REVISION A

B1 0 960 100 200 300

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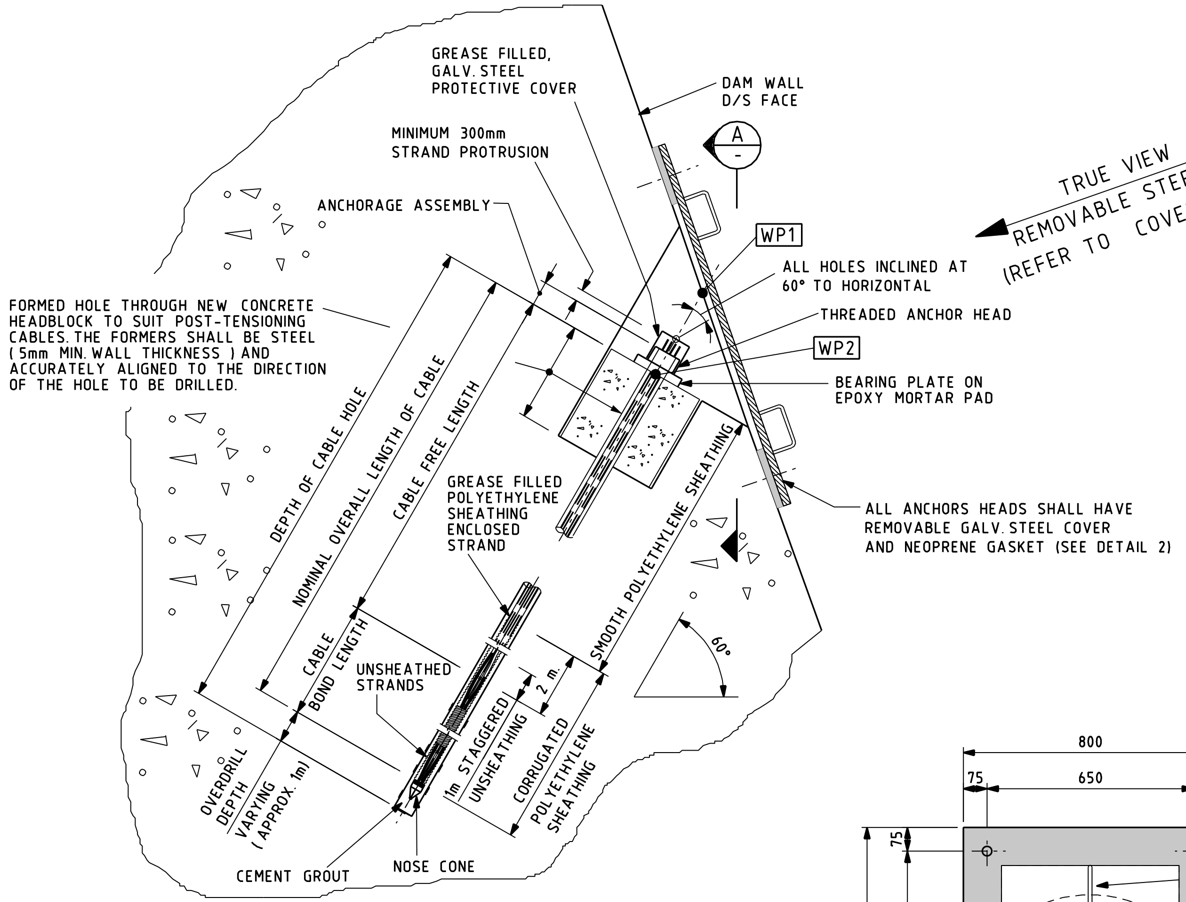


TYPICAL DETAIL

SHOWING CABLE COMPONENTS

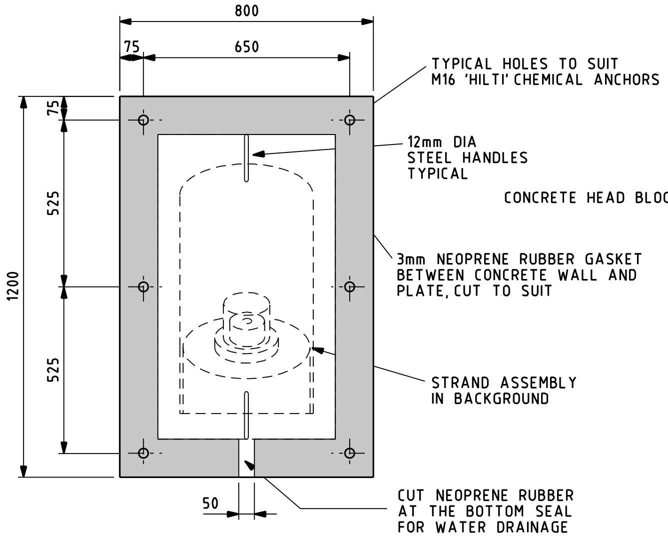
(NOTE:- CABLE COMPONENTS ARE TYPICAL FOR ALL CABLES)

N.T.S

TABLE 1

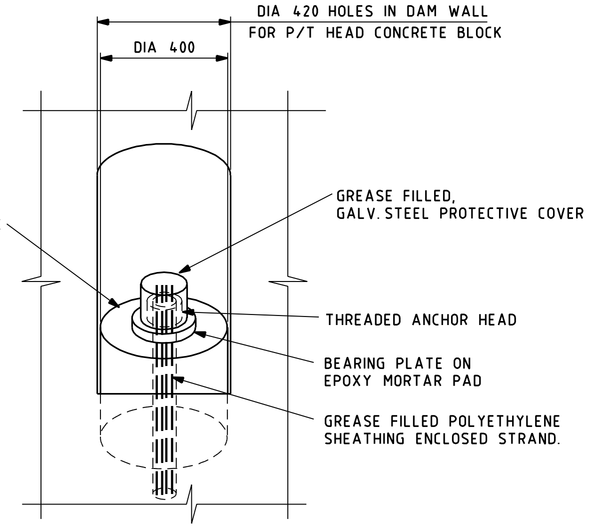
SUB-VERTICAL P/T CABLES AT D/S FACE (ALSO SEE NOTE 1)

CABLE LOCATION No.	HOLE BEARING	REFERENCE RL FOR CABLE LAYOUT AT EXISTING CONC. SURFACE "WP1"	TOP OF CABLE HOLE "WP2" (RL)	ESTIMATED DRILL HOLE LENGTH (m)			CABLE NOMINAL OVERALL LENGTH (m)	STRESSING ORDER
				TOTAL LENGTH OF HOLE (m)	NEW CONCRETE HEADBLOCK FORMED HOLE	APPROX. DRILLING LENGTH THROUGH ROCK (m)		
10		RL 530.65		17	0.15m	12	16	
11		RL 529.40		18	0.15m	12	17	
12		RL 528.35		19	0.15m	12	18	
13		RL 528.22		19	0.15m	12	18	
14		RL 528.37		19	0.15m	12	18	
15		RL 528.01		19	0.15m	12	18	
16		RL 527.75		19	0.15m	12	18	
17		RL 527.59		19	0.15m	12	18	
18		RL 527.68		19	0.15m	12	18	
19		RL 527.66		19	0.15m	12	18	
20		RL 527.84		19	0.15m	12	18	
21		RL 528.41		19	0.15m	12	18	
22		RL 528.26		19	0.15m	12	18	
23		RL 528.40		19	0.15m	12	18	
24		RL 528.53		19	0.15m	12	18	
25		RL 528.51		19	0.15m	12	18	
26		RL 528.85		19	0.15m	12	18	
27		RL 529.02		18	0.15m	12	17	
28		RL 528.55		19	0.15m	12	18	
29		RL 530.30		17	0.15m	12	16	
30		RL 530.40		17	0.15m	12	16	



REMOVABLE GALV. STEEL COVER 2/01

SCALE 1:10

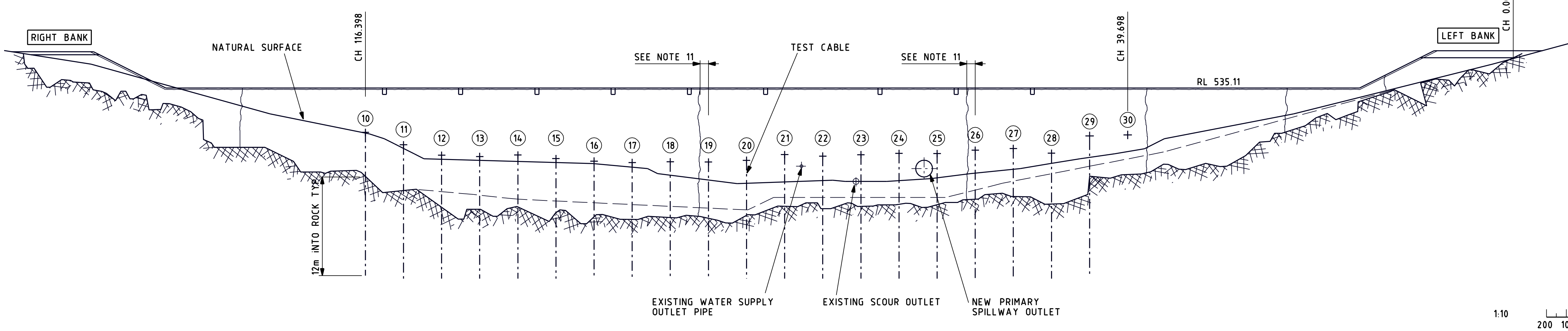


SECTION A/- (COVER NOT SHOWN)

SCALE 1:10

NOTES

1. 21 POST-TENSIONING CABLES REQUIRED AT DAM DOWNSTREAM FACE, WITH LOCATION, SIZE AND LOADS BEING:-
   i) CABLE LOCATIONS 10 TO 21 EACH CONSISTING OF 4-15.2mm DIA SUPERGRADE STRANDS, HAVING AN M.B.L. OF 1000kN. DESIGN WORKING LOAD IS 600 kN (60% M.B.L.)
2. THE MINIMUM BOND LENGTHS SHALL BE 3m.
3. ALL RL's ARE TO mAHD.
4. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
5. ALL CABLES SHALL BE IDENTIFIED BY THE CABLE CODING ASSIGNED.
6. CONCRETE CHARACTERISTIC COMPRESSIVE STRENGTH FOR THE POST-TENSIONED CABLES HEAD BLOCKS SHALL BE 40 MPa AT 28 DAYS
7. MINIMUM CLEAR COVER TO REINFORCEMENT SHALL BE 50 mm UNLESS OTHERWISE SHOWN.
8. WP1 DENOTES "WORKING POINT". THIS IS AN ESTIMATED RL ONLY. CONTRACTOR IS TO CONFIRM AT SITE FOR ACCURATE DRILLING ALIGNMENT.
9. "TABLE 1", DETAILS FOR "WP2" (RL) TO BE FILLED IN DURING CONSTRUCTION.
10. FOR TYPICAL REINFORCED CONCRETE CONSTRUCTION NOTES, SEE DRG. 0800112-02
11. HOLE POSITION SHOULD BE ADJUSTED TO GIVE A MINIMUM CENTRLINE DISTANCE OF 0.5m FROM CRACK.
12. STRESSING ORDER (SEE IN TABLE '1') TO BE PROVIDED BY THE CONTRACTOR AND TO BE SATISTAFACTORY TO THE PRINCIPAL.



SUB-VERTICAL POST TENSIONING FROM D/S FACE

SCALE 1:250

1:10 0 200 400 600 800 1000 mm 200 100

1:250 0 5 10 15 20 25 m 5

THIS DRAWING TO BE READ IN CONJUNCTION WITH GENERAL ARRANGEMENT DRG. 0800112-01

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A	ISSUED FOR TENDER.	J.L.	04.11.08

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PLAN RM NO
DESIGNED: A. UN-NABI — MANAGER DAMS AND CIVIL TECHNOLOGIES: J.LENEHAN
DRAFTED: D.T.
CHECKED: B.COOPER

NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS
SUB-VERTICAL POST TENSIONING FROM D/S FACE

NO IN SET
FILE NO.
DRAWING NO. 0800112-05
REVISION A

B1 0 960 100 200 300 679

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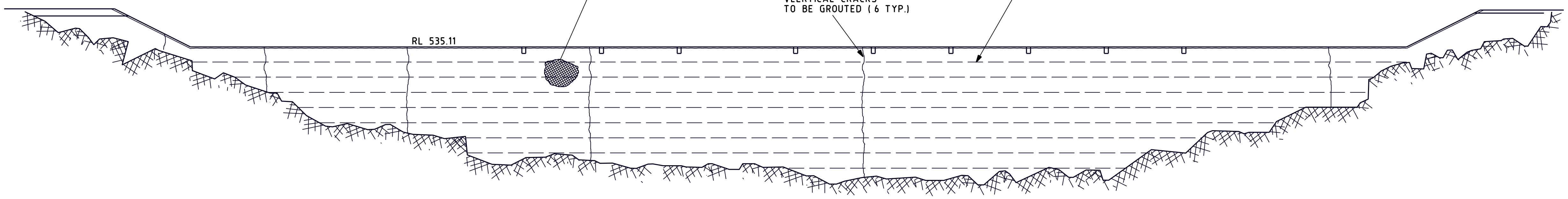
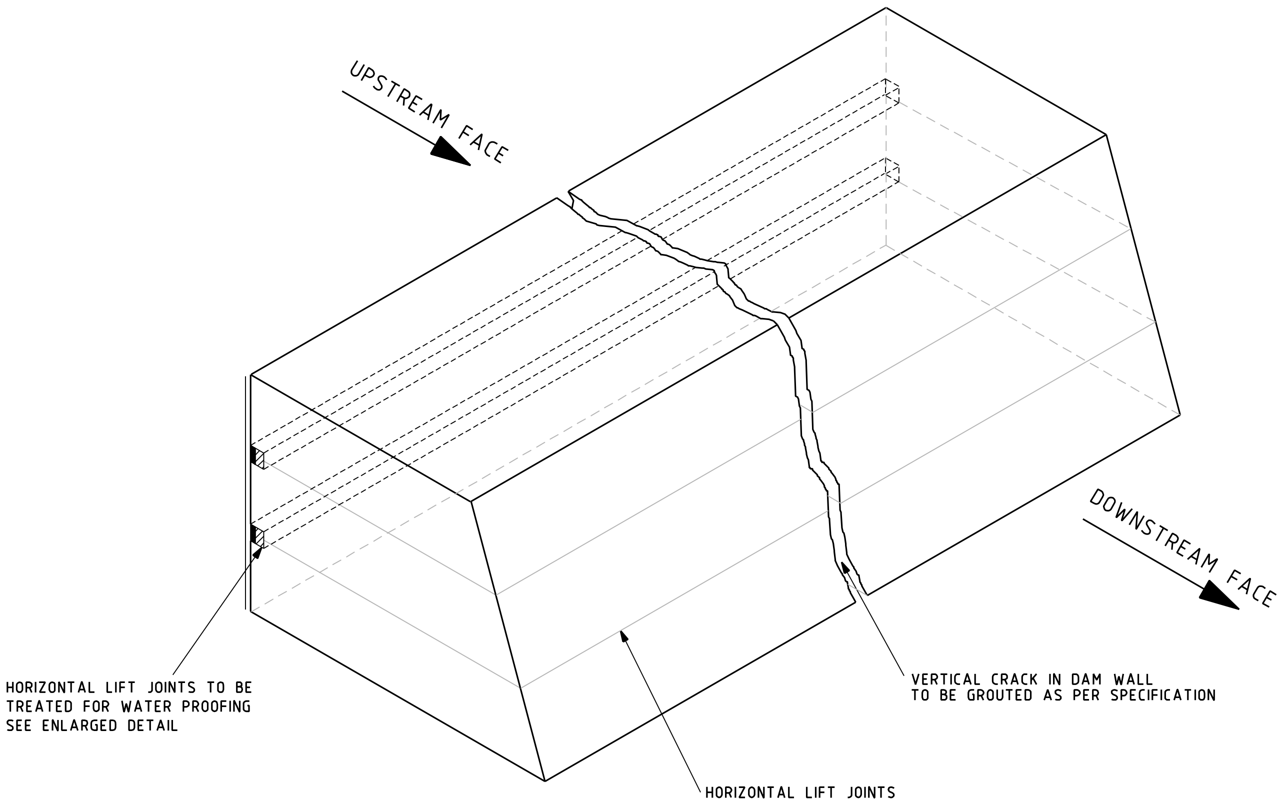


DAM U/S FACE (LOOKING D/S)

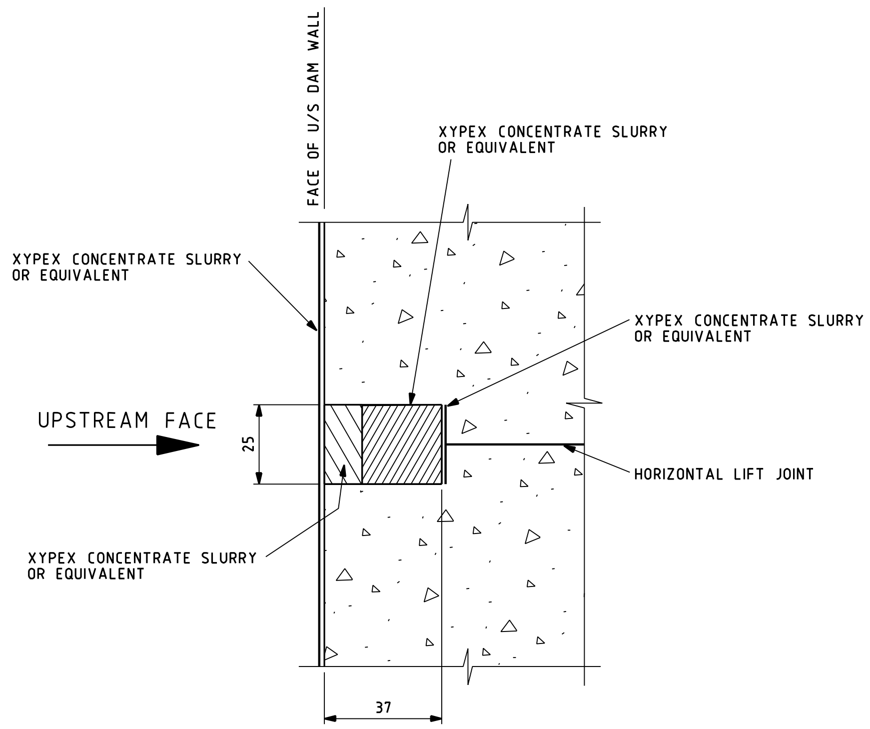
SCALE 1:200



ISOMETRIC SKETCH OF DAM SECTION SHOWING

VERTICAL CRACK & HORIZONTAL LIFT JOINTS

NOT TO SCALE



TYPICAL HORIZONTAL LIFT JOINT TREATMENT DETAIL

NOT TO SCALE

NOTES:

1. ALL LEVELS TO mAHD.
2. ALL DIMENSIONS IN MILLIMETERS UNO.

1:200 4 2 0 4 8 12 16 20 m

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REDBANK CREEK DAM
STABILISATION WORKS
VERTICAL CRACKS AND HORIZONTAL LIFT JOINTS TREATMENT DETAILS & U/S FACE WATER PROOFING

NO IN SET

FILE NO.

DRAWING NO. 0800112-06

REVISION A

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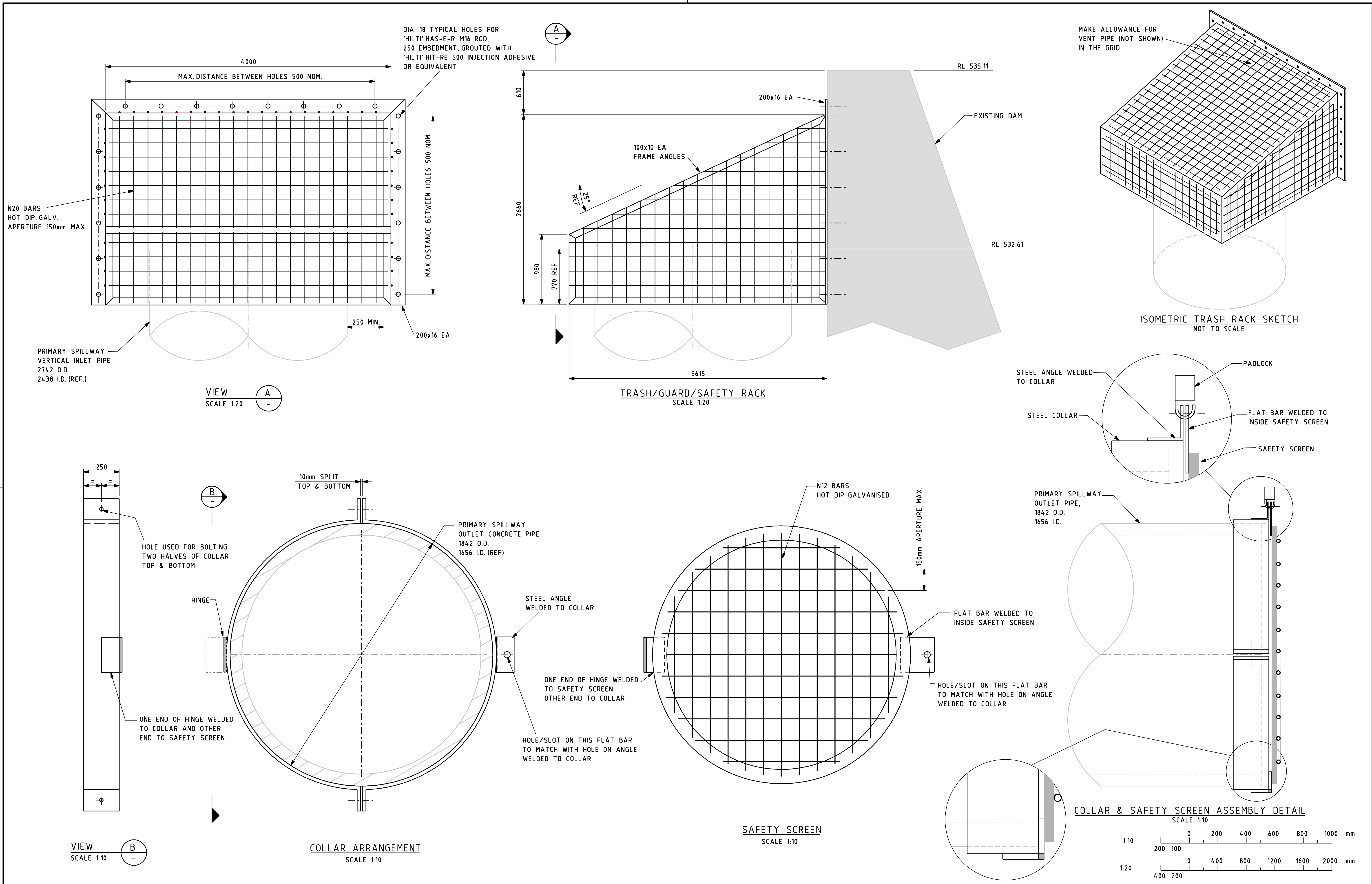

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MK	DETAILS OF AMENDMENTS	APPROVED	DATE
A	ISSUED FOR TENDER.	J.L.	04.11.08

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DRAFTED D.T.	
CHECKED B. COOPER	

NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS

PRIMARY SPILLWAY TRACH RACK AND SAFETY SCREEN DETAILS

NO IN SET
FILE NO.
DRAWING NO. 0800112-07
REVISION A

B1

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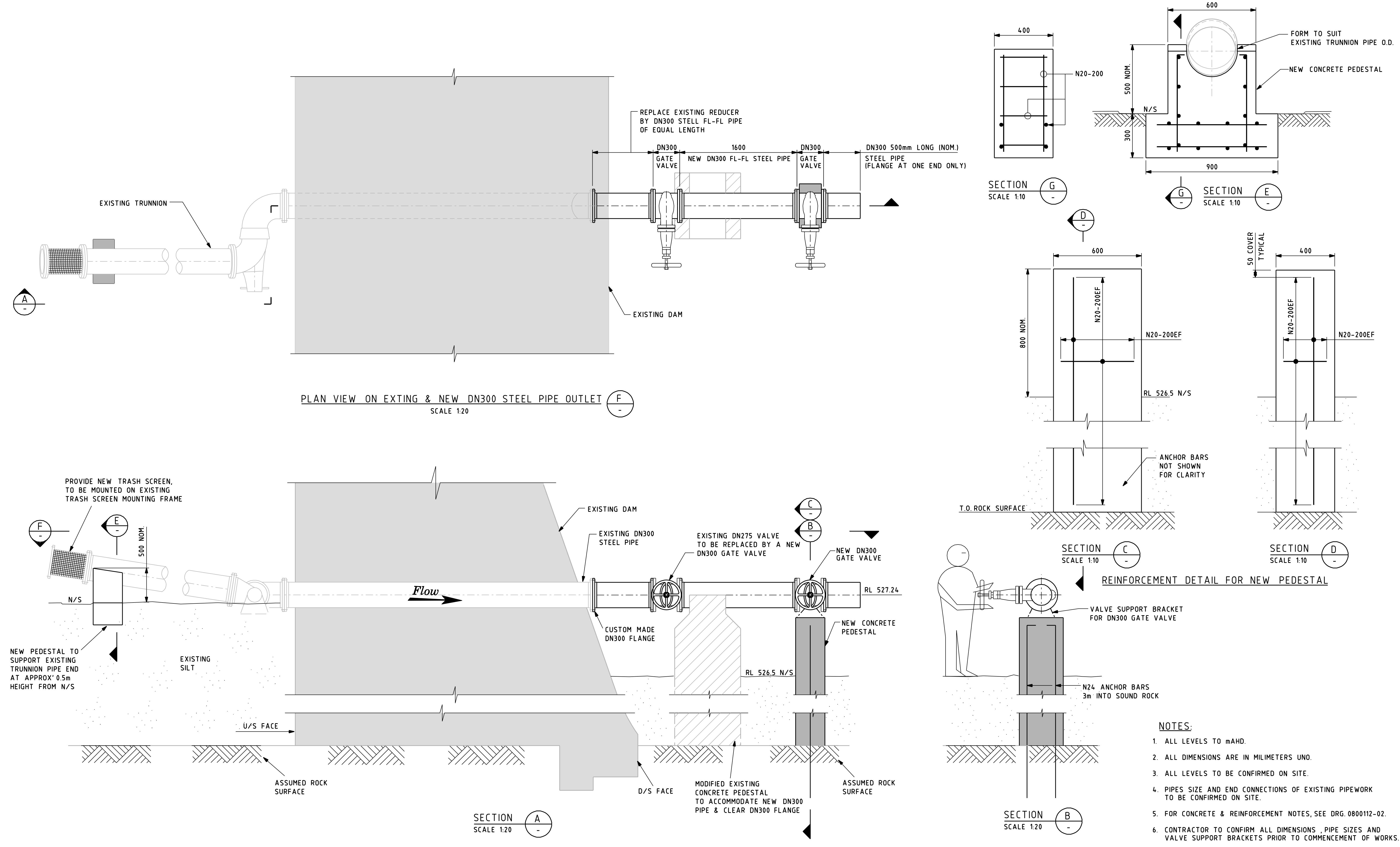

NOTES:

1. ALL LEVELS TO mAHD.
2. ALL DIMENSIONS ARE IN MILIMETERS UNO.
3. ALL LEVELS TO BE CONFIRMED ON SITE.
4. PIPES SIZE AND END CONNECTIONS OF EXISTING PIPEWORK TO BE CONFIRMED ON SITE.
5. FOR CONCRETE & REINFORCEMENT NOTES, SEE DRG. 0800112-02.
6. CONTRACTOR TO CONFIRM ALL DIMENSIONS, PIPE SIZES AND VALVE SUPPORT BRACKETS PRIOR TO COMMENCEMENT OF WORKS.

1:10 0 200 400 600 800 1000 mm
1:20 200 100 0 400 800 1200 1600 2000 mm 400 200

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MK	DETAILS OF AMENDMENTS	APPROVED	DATE
A	ISSUED FOR TENDER.	J.L.	04.11.08

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DRAFTED D.T.
CHECKED B.COOPER

NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS
EXISTING PIPE OUTLET REFURBISHMENT

NO IN SET
FILE NO.
DRAWING NO. 0800112-08
REVISION A

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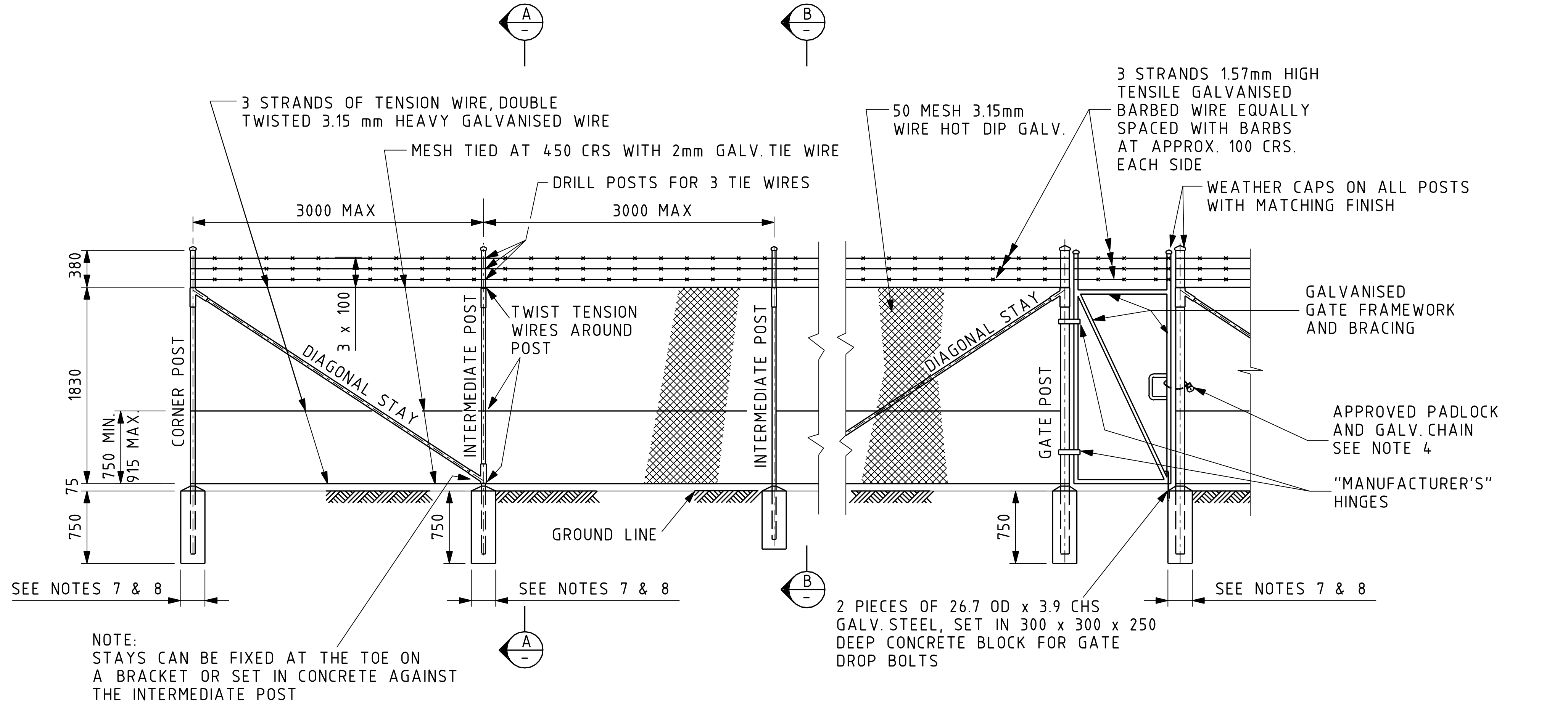
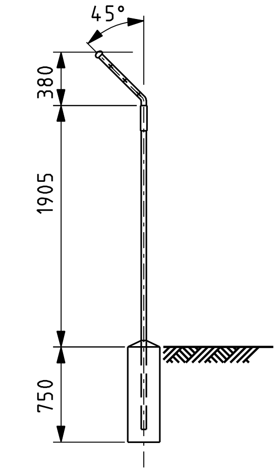
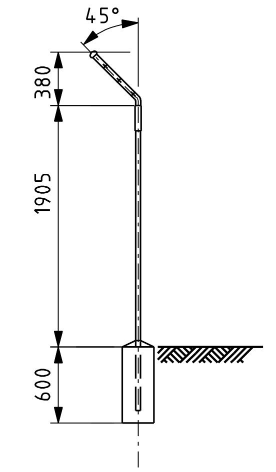


SECURITY FENCING WITH SINGLE GATE
SCALE 1:25



SECTION A
SCALE 1:25

INTERMEDIATE STAY POST, CORNER POST, GATE POST ETC. SEE NOTE 10



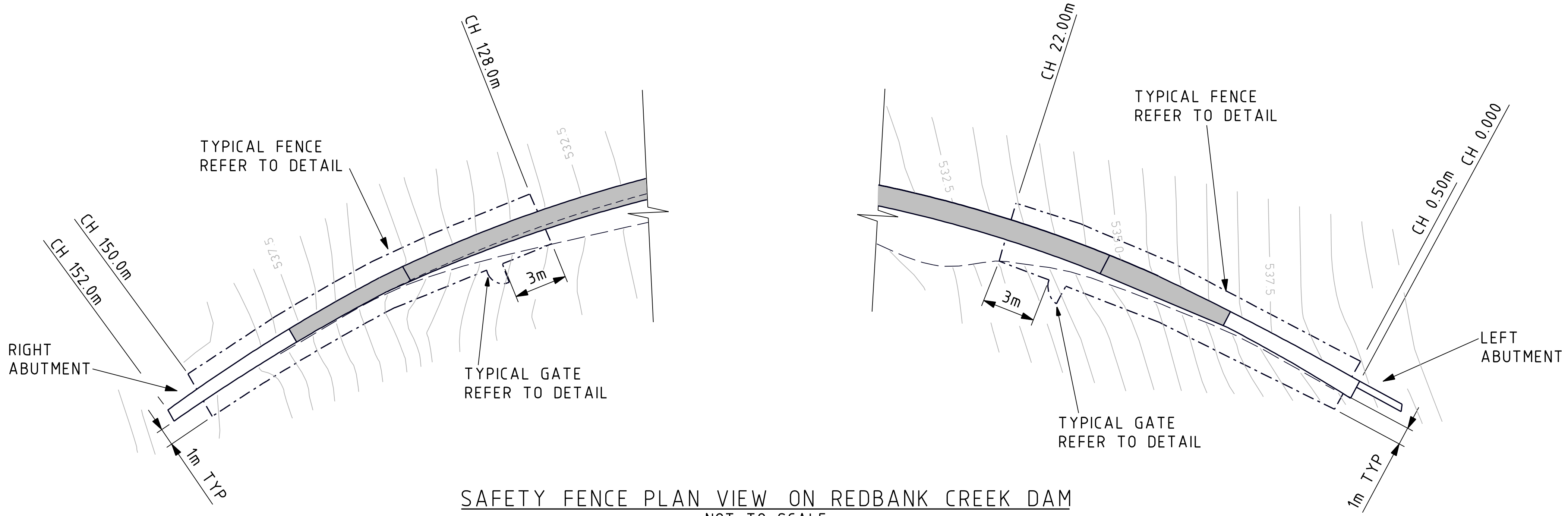
SECTION B
SCALE 1:25

INTERMEDIATE POST

SECURITY FENCING

MEMBER	POST SIZE O.D. mm	WALL THICK mm	FOOTING DEPTH IN EARTH	IN ROCK
CORNER POST	60.3	3.6	750	450
INTERMEDIATE POST	42.4	3.2	600	300
DIAGONAL STAY	42.4	3.2	*450	*300
GATE POST	88.9	4.0	750	450
GATE FRAMEWORK	42.4	3.2	-	-
GATE BRACING	42.4	3.2	-	-

*DIMENSION MEASURED VERTICALLY



SAFETY FENCE PLAN VIEW ON REDBANK CREEK DAM
NOT TO SCALE

NOTES

1. CLEAR OPENING OF GATES TO BE A MINIMUM OF 1000mm.
2. CLEAR 300 EACH SIDE OF FENCE AND REMOVE ALL ROCKS, TREES, ROOTS ETC.
3. CABLE WIRES AND MESH TO BE HOT DIP GALV. WIRE TO A.S.2423 TYPE 'A' QUALITY.
4. GATES TO HAVE A 10 GALVANISED CHAIN WHICH IS TO BE WELDED TO GATE POST FOR PADLOCK.
5. ALL FITTINGS , BOLTS AND NUTS ETC. TO BE GALV. STEEL OF APPROVED TYPE. NUTS TO BE BURRED AFTER ERECTION.
6. ALL VERTICAL POSTS SHALL BE WEATHER CAPPED.
7. POST HOLES IN ROCK SHALL BE 50mm LARGER IN DIAMETER THAN THE OUTSIDE DIA. OF THE PIPE AND THE PIPE SHALL BE GROUTED INTO HOLE WITH 3 : 1 SAND/CEMENT MIX MORTAR.
8. POST HOLES IN OTHER THAN ROCK SHALL BE 250 DIA. MIN. BACKFILLED WITH GRADE 20 CONCRETE.
9. POST HOLES TO BE 750mm DEEP x 400mm DIAMETER BACKFILLED WITH GRADE 20 CONCRETE.
10. POSTS, STAYS, ETC. TO BE GALVANISED STEEL PIPES.
11. PROVIDE 3 RUNS OF HIGH TENSILE 1.57mm BARBED WIRE ON EACH GATE AND FENCE PANEL.

NOTE: THIS IS AN EXAMPLE ONLY ACTUAL DESIGN AND CONSTRUCTION DRAWING SHALL SUIT PROJECT REQUIREMENT.

1:25 500 0 500 1000 1500 2000 2500 mm

THIS DRAWING TO BE READ IN CONJUNCTION WITH GENERAL ARRANGEMENT DRG. 0800112-01

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DRAFTED D.T.
CHECKED B. COOPER

NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS
SECURITY FENCE AND GATE DETAILS

NO IN SET
FILE NO.
DRAWING NO. 0800112-09
REVISION A

B1

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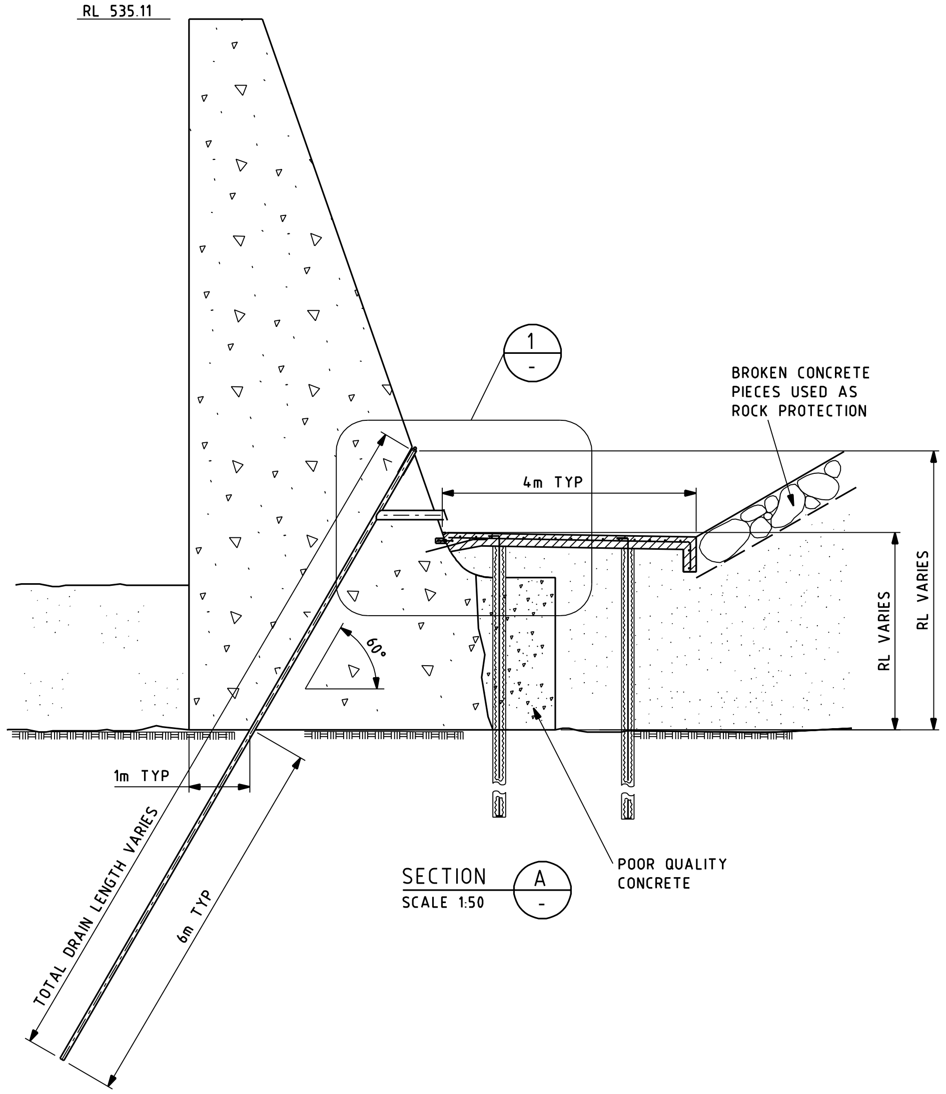
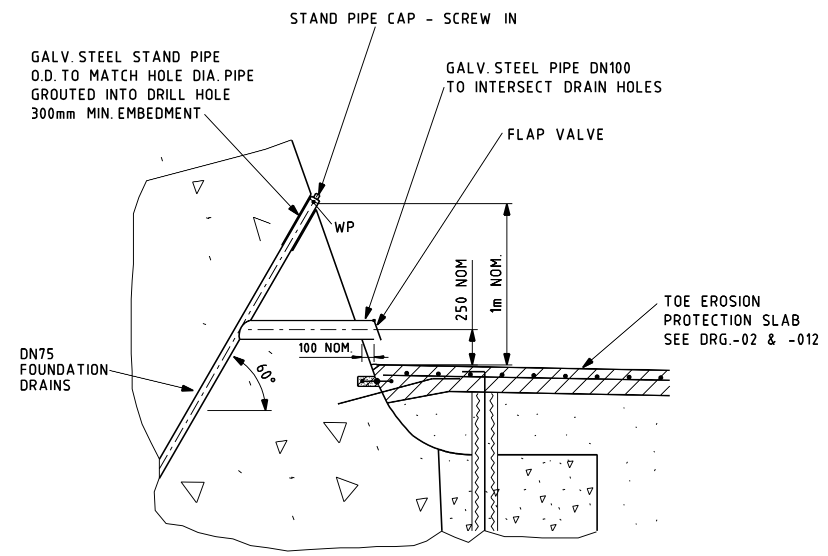


SECTION A
SCALE 1:50



DETAIL 1
SCALE 1:20

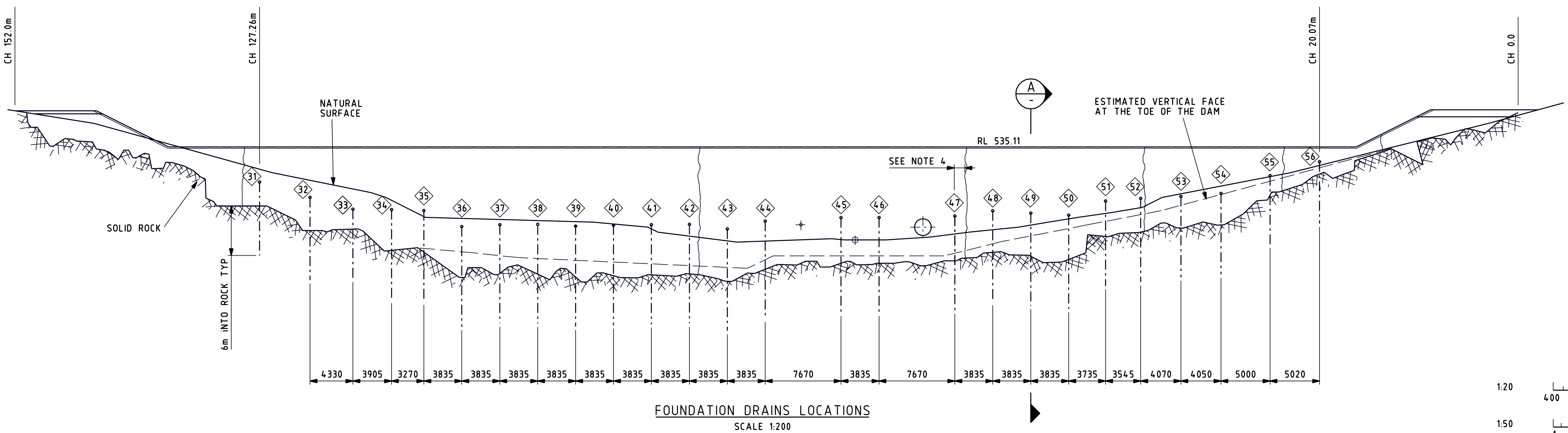
TABLE 1

FOUNDATION DRAINS

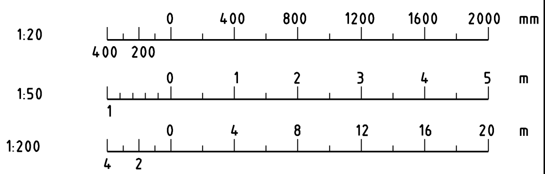
DRAIN No.	REFERENCE RL FOR CABLE LAYOUT AT EXISTING CONC. SURFACE "WP"	ESTIMATED DRILL HOLE LENGTH (m)	
		TOTAL LENGTH OF HOLE (m)	APPROX. DRILLING LENGTH THROUGH ROCK (m)
31	RL 531.56	9	6
32	RL 530.00	10	6
33	RL 530.12	10	6
34	RL 528.78	11	6
35	RL 528.67	11	6
36	RL 527.07	12	6
37	RL 527.24	12	6
38	RL 527.27	12	6
39	RL 527.10	12	6
40	RL 527.18	12	6
41	RL 527.24	12	6
42	RL 527.29	12	6
43	RL 526.83	12.5	6
44	RL 527.59	12	6
45	RL 527.96	11.5	6
46	RL 527.96	11.5	6
47	RL 528.14	11.5	6
48	RL 528.65	11	6
49	RL 528.40	11.5	6
50	RL 528.20	12	6
51	RL 529.65	10.5	6
52	RL 529.92	10	6
53	RL 530.18	10	6
54	RL 530.37	10	6
55	RL 532.24	8.5	6
56	RL 533.57	7.5	6

NOTES

1. ALL RL's ARE TO mAHD.
2. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
3. 'WP' DENOTES 'WORKING POINT". THIS IS AN ESTIMATED RL ONLY. CONTRACTOR IS TO CONFIRM AT SITE FOR ACCURATE DRILLING ALIGNMENT.
4. HOLE POSITION SHOULD BE ADJUSTED TO GIVE A MINIMUM CENTRLINE DISTANCE OF 0.5m FROM CRACK.



FOUNDATION DRAINS LOCATIONS
SCALE 1:200



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A	ISSUED FOR TENDER.	J.L.	04.11.08

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DRAFTED	
D.T.	
CHECKED	
B. COOPER	

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REDBANK CREEK DAM
STABILISATION WORKS
FOUNDATION DRAINS

NO IN SET
FILE NO.
DRAWING NO. 0800112-10
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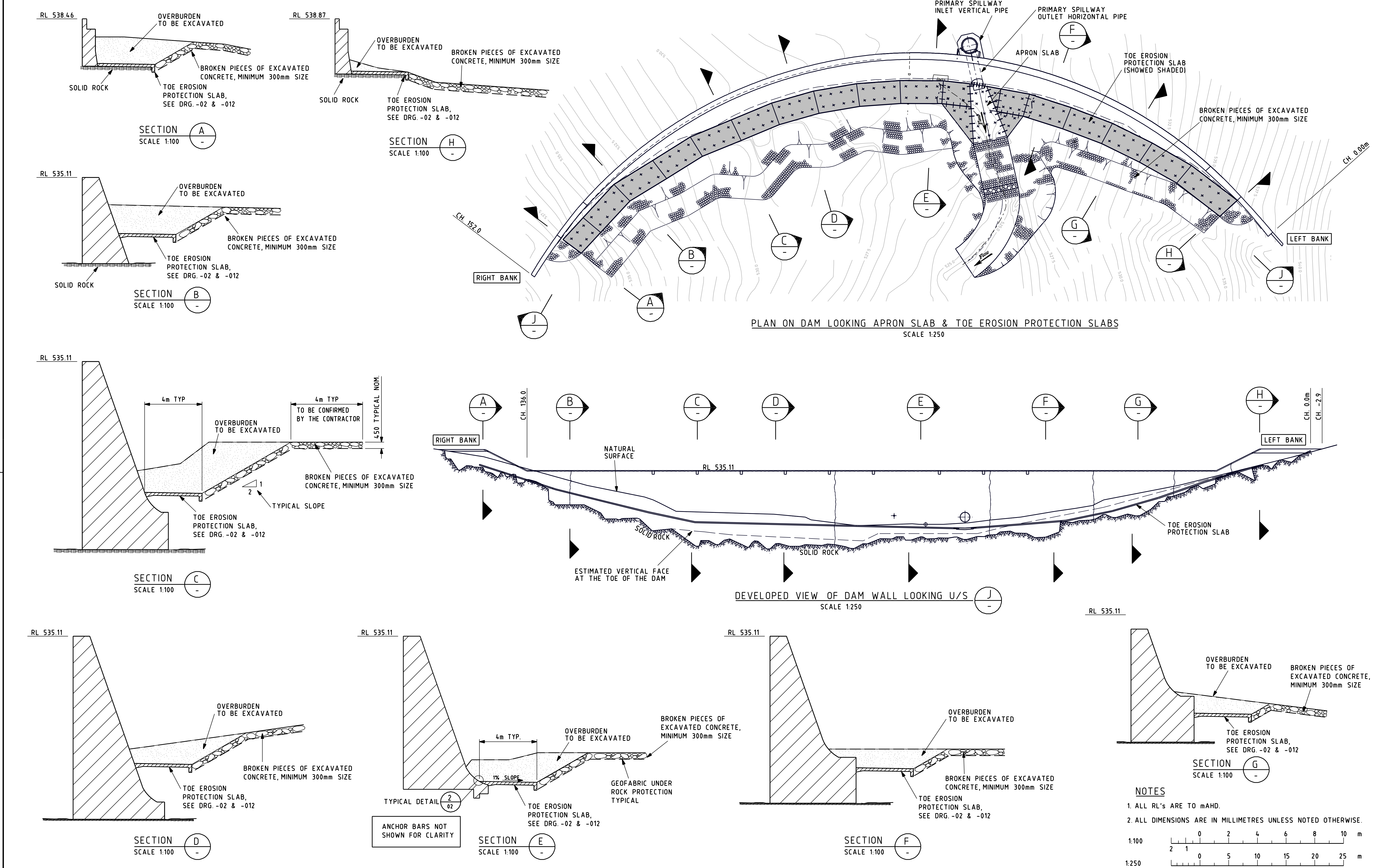

NOTES

1. ALL RL's ARE TO mAHD.
2. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.

1:100 0 2 4 6 8 10 m

1:250 0 5 10 15 20 25 m

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A	ISSUED FOR TENDER.	J.L.	04.11.08

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MANAGER DAMS AND CIVIL TECHNOLOGIES: J.LENEHAN
DRAFTED: D.T.
CHECKED: B. COOPER

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REDBANK CREEK DAM
STABILISATION WORKS
TOE EROSION PROTECTION SLAB

DRAWING NO. 0800112-11
REVISION A

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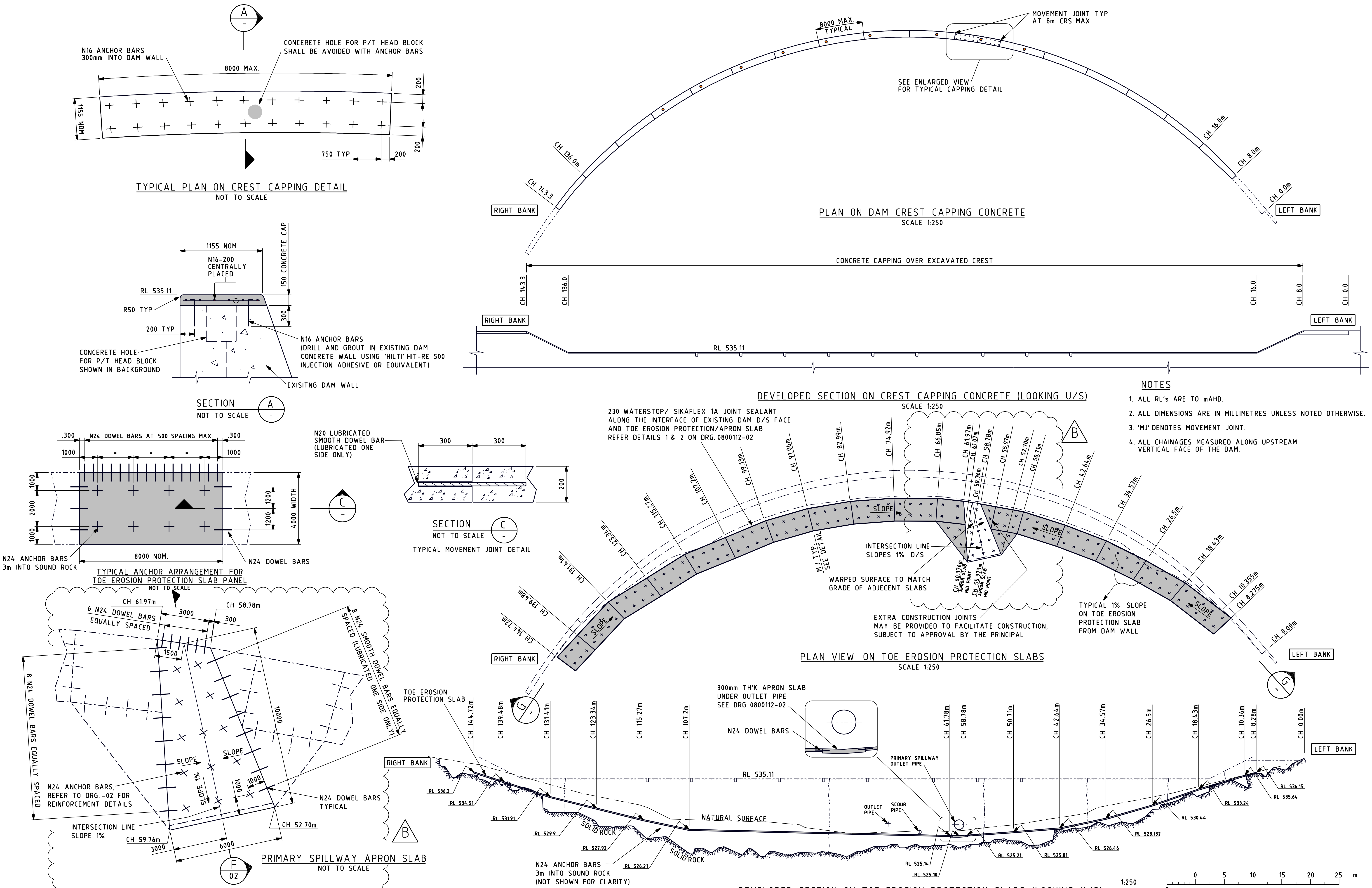

THIS DRAWING TO BE READ IN CONJUNCTION WITH GENERAL ARRANGEMENT DRG. 0800112-01

MK	DETAILS OF AMENDMENTS	APPROVED	DATE
B	DIMENSIONS AND CHANGES REVISED, NOTE 4 ADDED.	A.U.	29.07.09
A	ISSUED FOR TENDER.	J.L.	05.11.08

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DRAFTED: D.T.	
CHECKED: B.COOPER	

NSW Department of Commerce

REDBANK CREEK DAM
STABILISATION WORKS
CREST CONCRETE CAPPING AND TOE EROSION PROTECTION SLAB

NO IN SET
FILE NO.
DRAWING NO. 0800112-12
REVISION B

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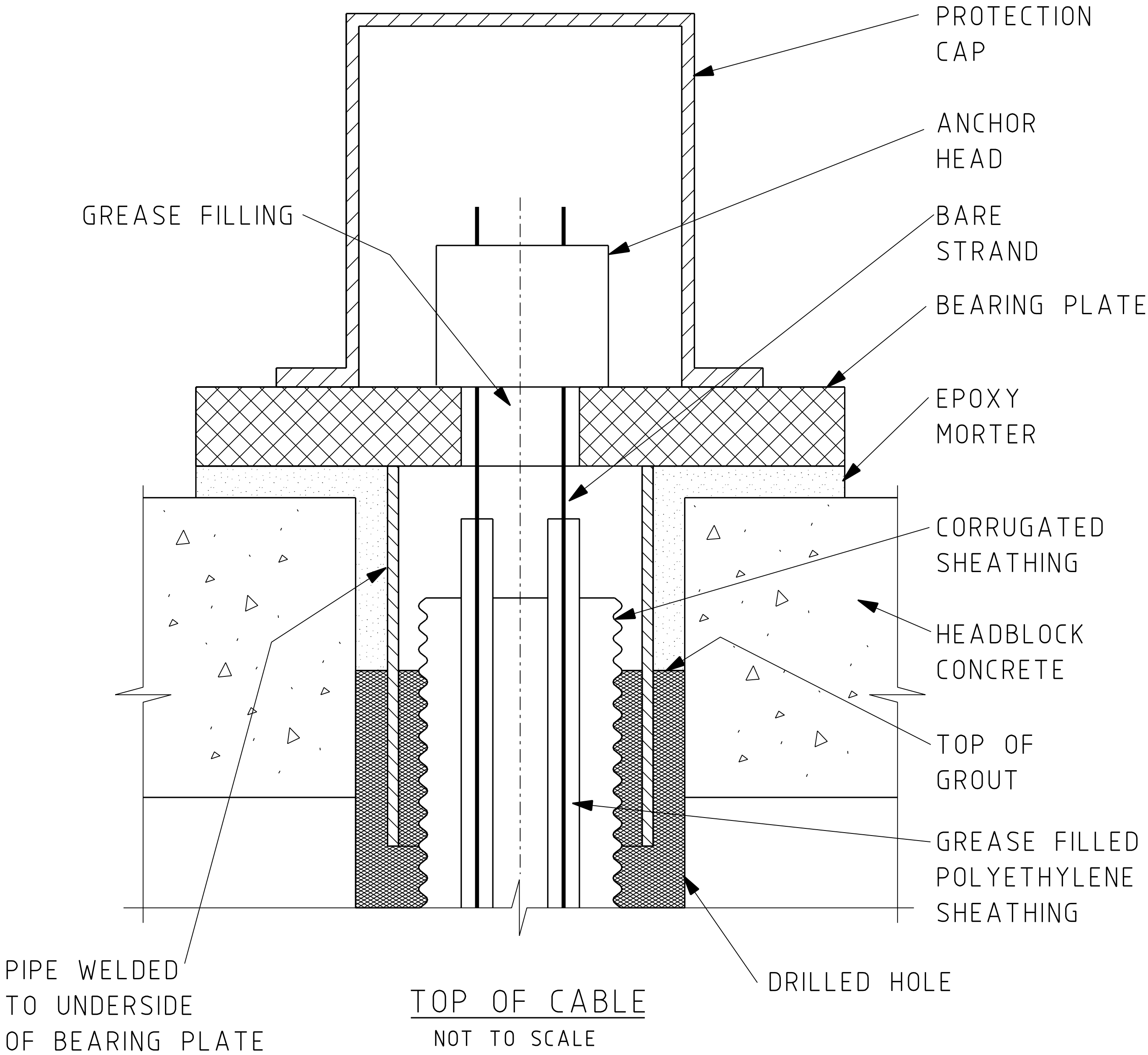

TOP OF CABLE

NOT TO SCALE

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D.T.	
CHECKED	
B.COOPER	

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REDBANK CREEK DAM

STABILISATION WORK

STANDARD DETAIL FOR TOP OF P/T CABLE

JOB No.

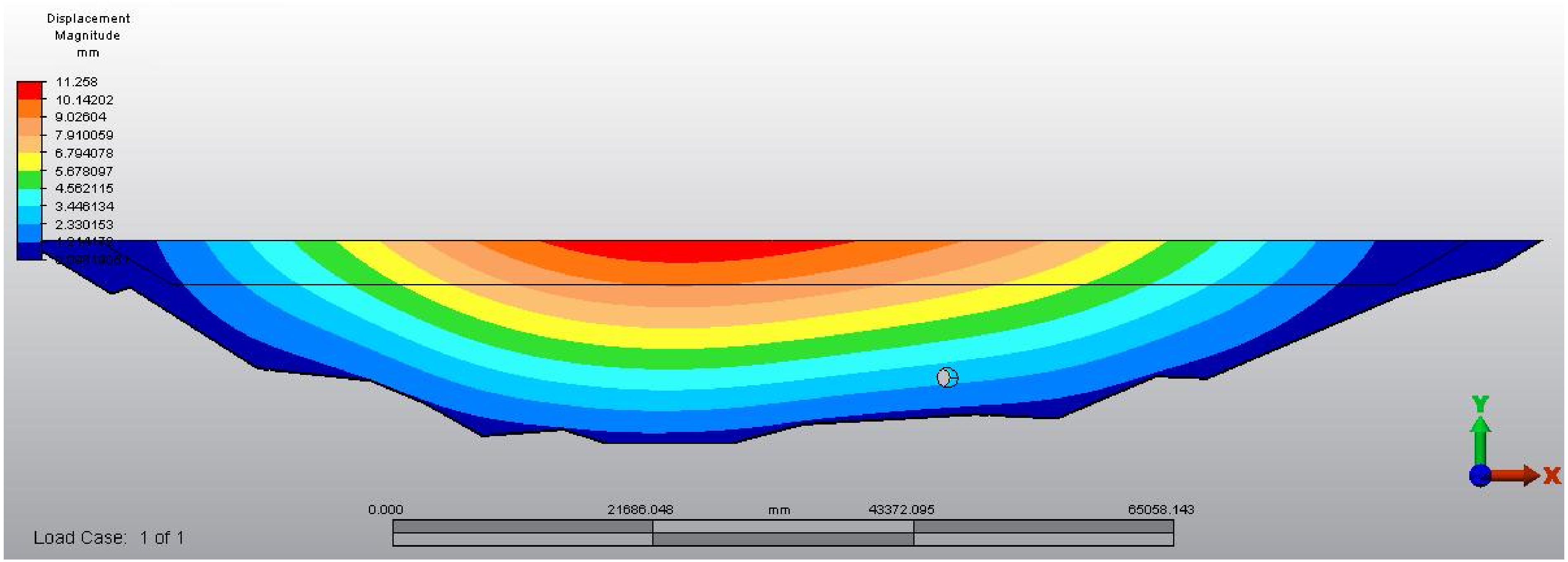
DRAWING No. 0800112–13

REVISION A

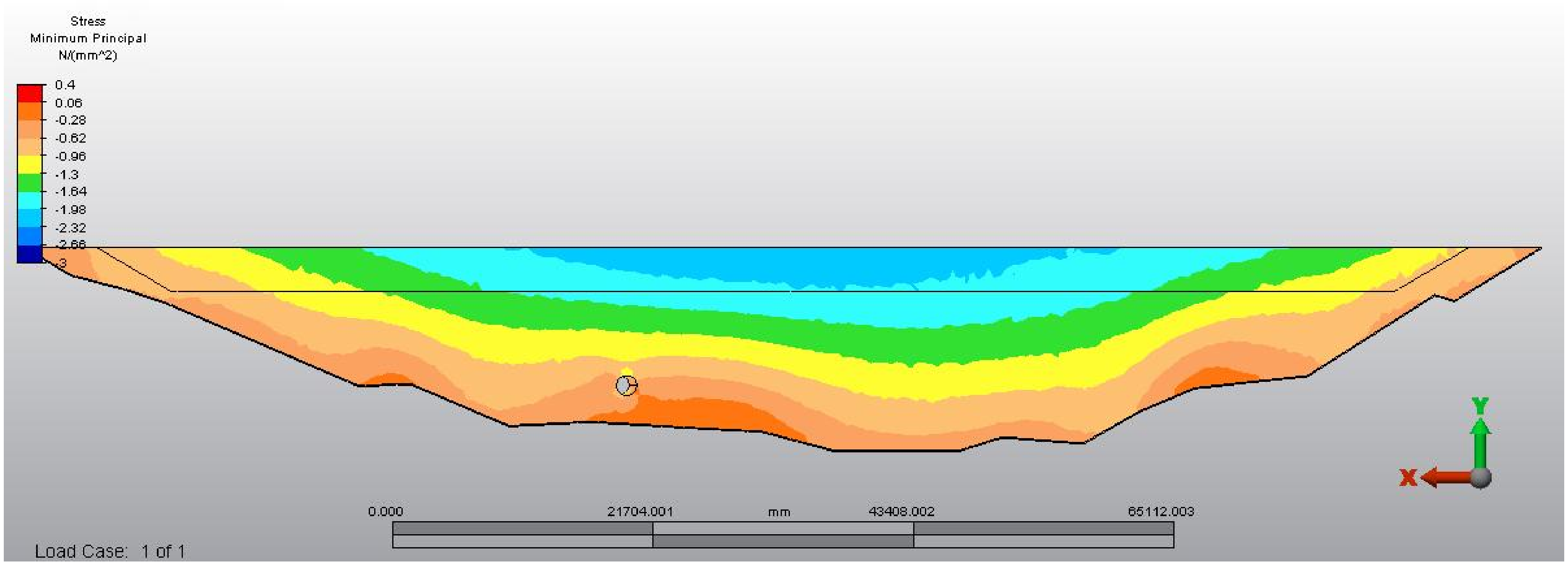
Appendix C

Finite Element Stress Analysis Results for Existing Dam

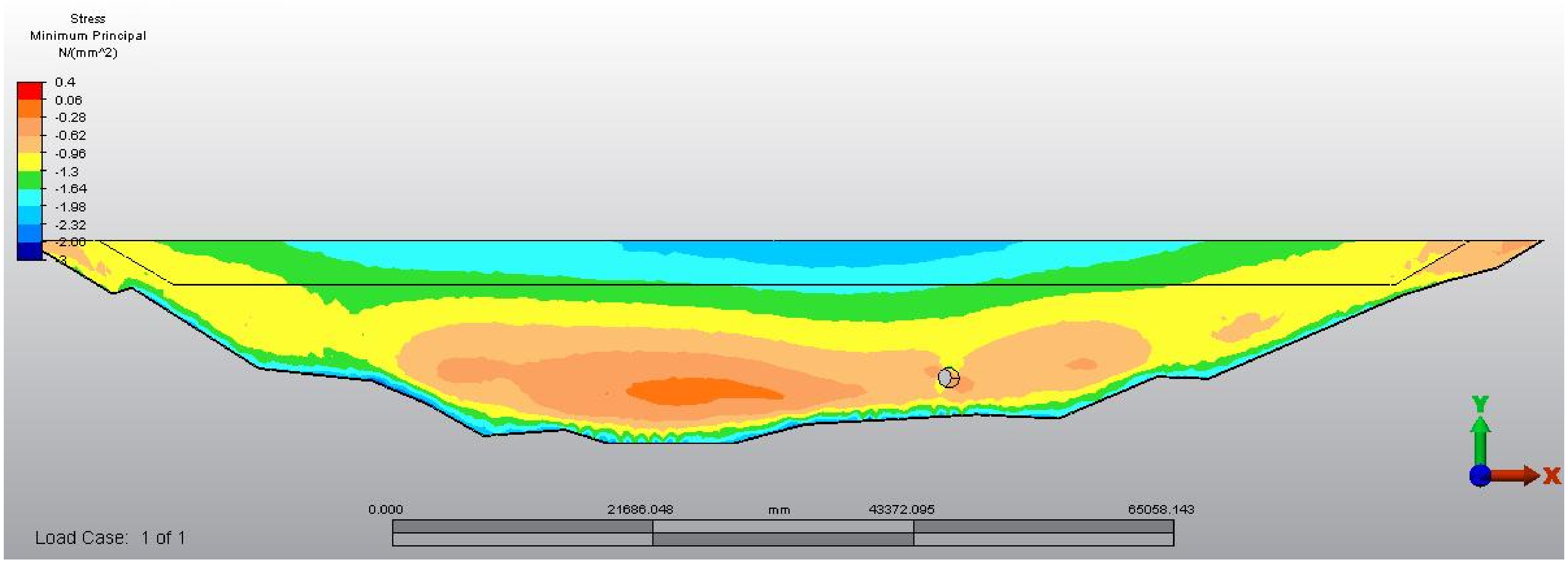
Displacement Contour Plot (Downstream Face)



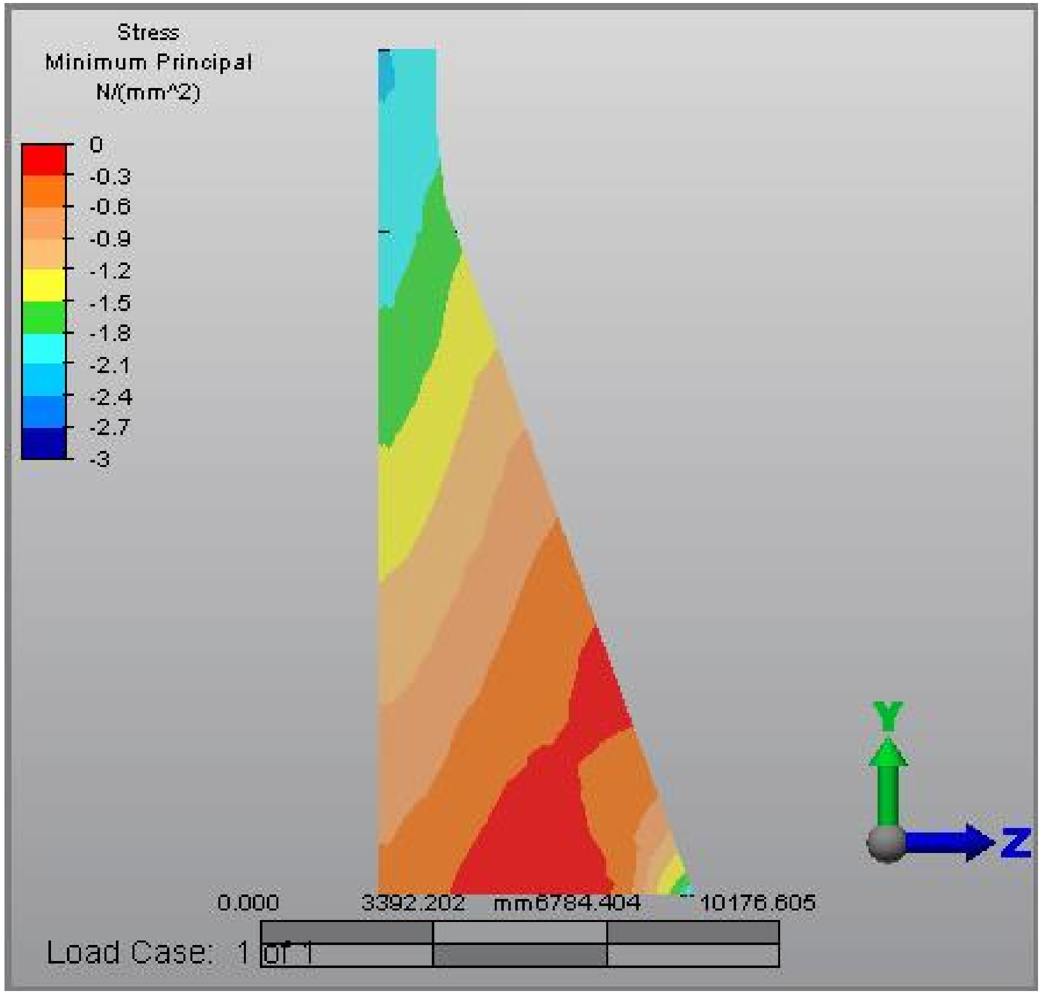
Minimum Principal Stress Contour Plot (Upstream Face)



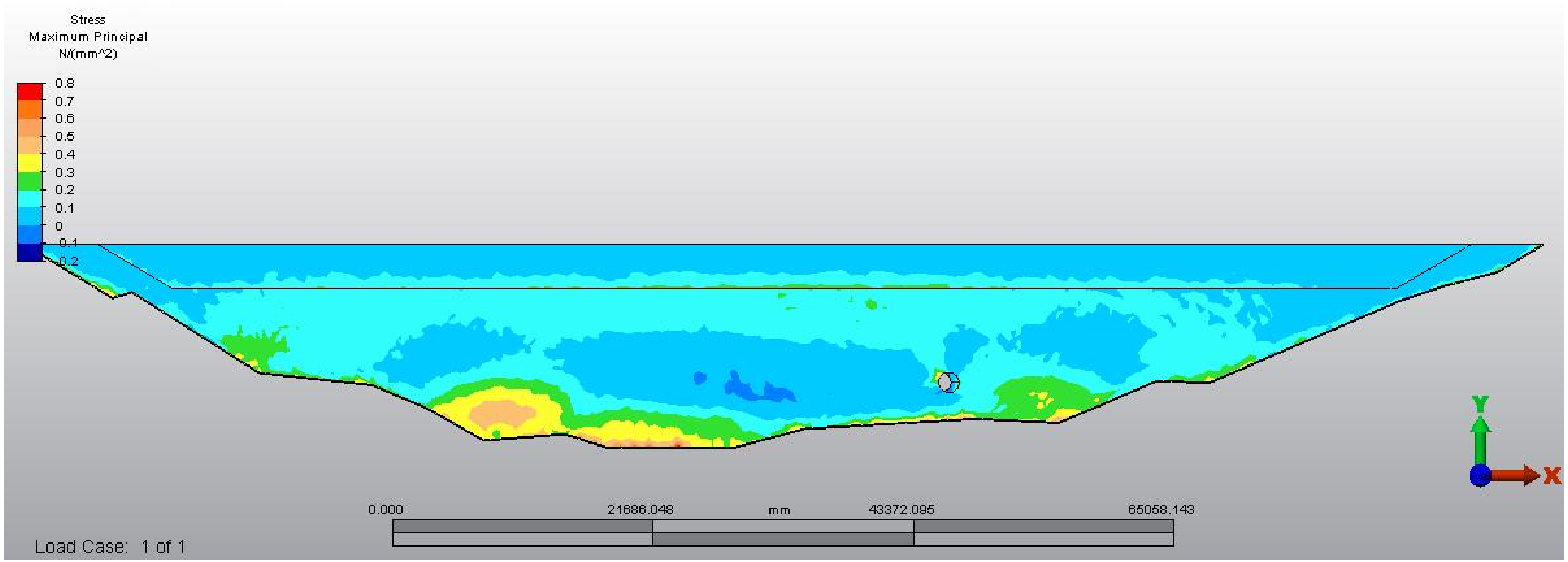
Minimum Principal Stress Contour Plot (Downstream Face)



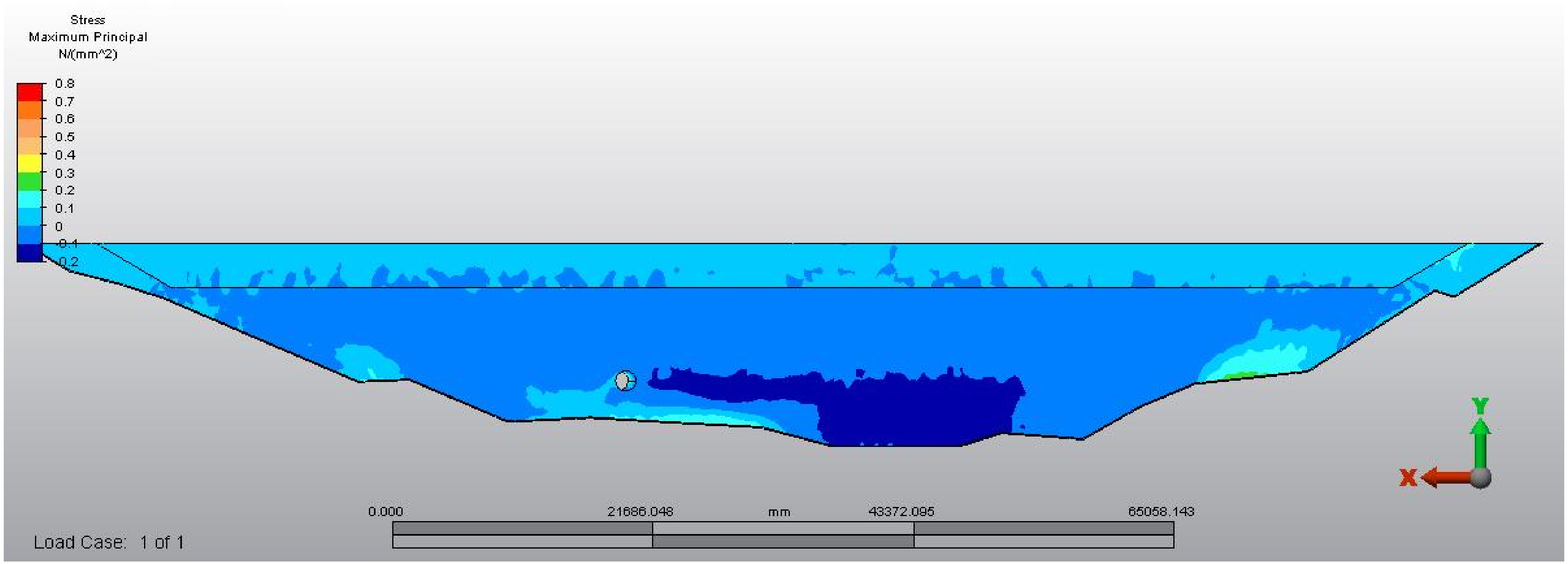
Minimum Principal Stress Contour Plot (Highest Section)



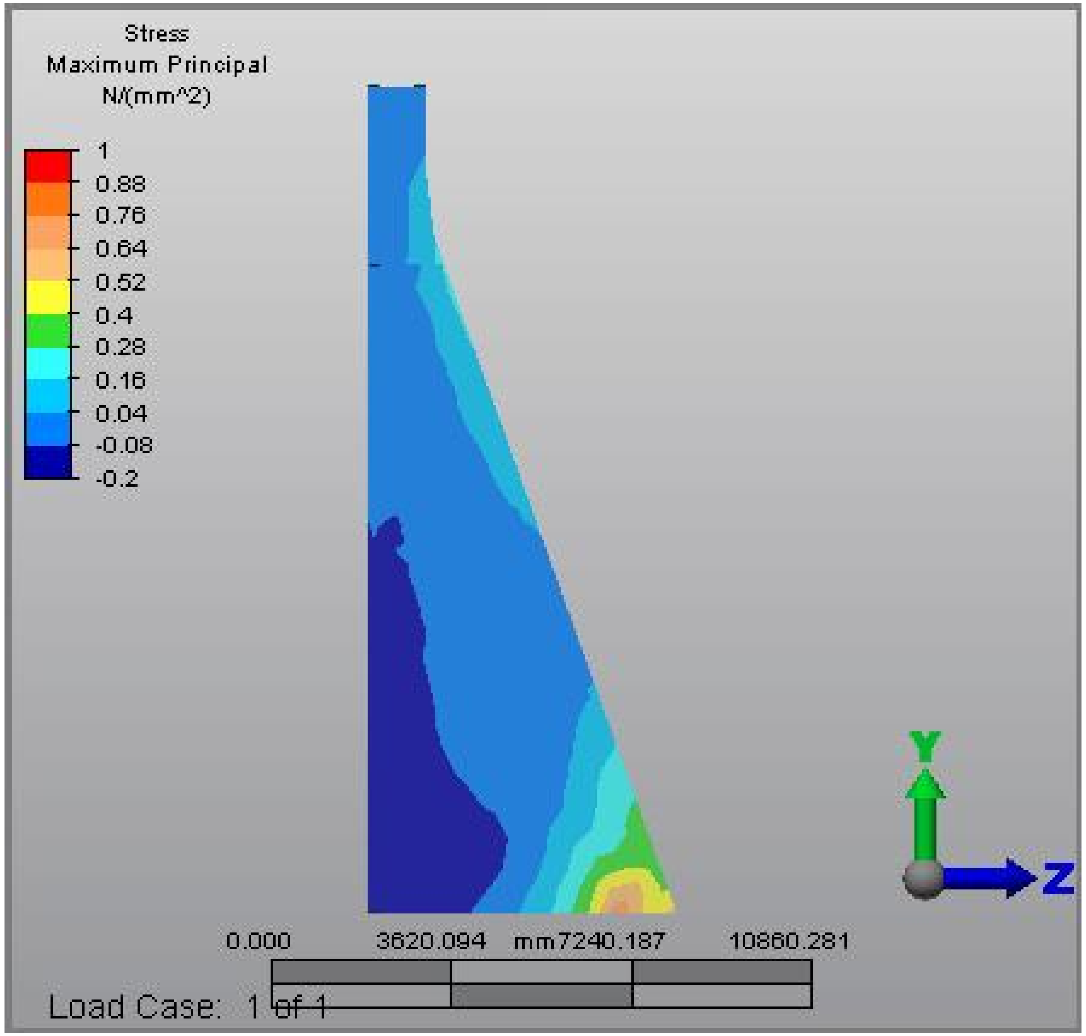
Maximum Principal Stress Contour Plot (Upstream Face)



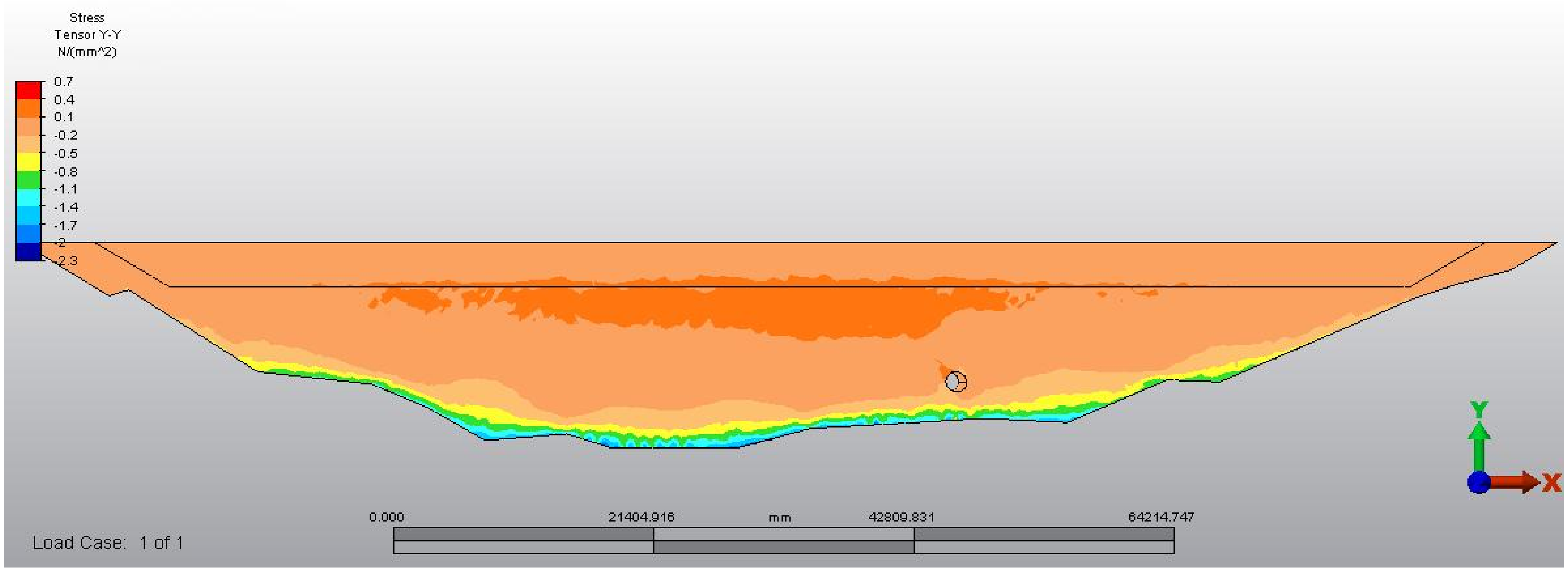
Maximum Principal Stress Contour Plot (Upstream Face)



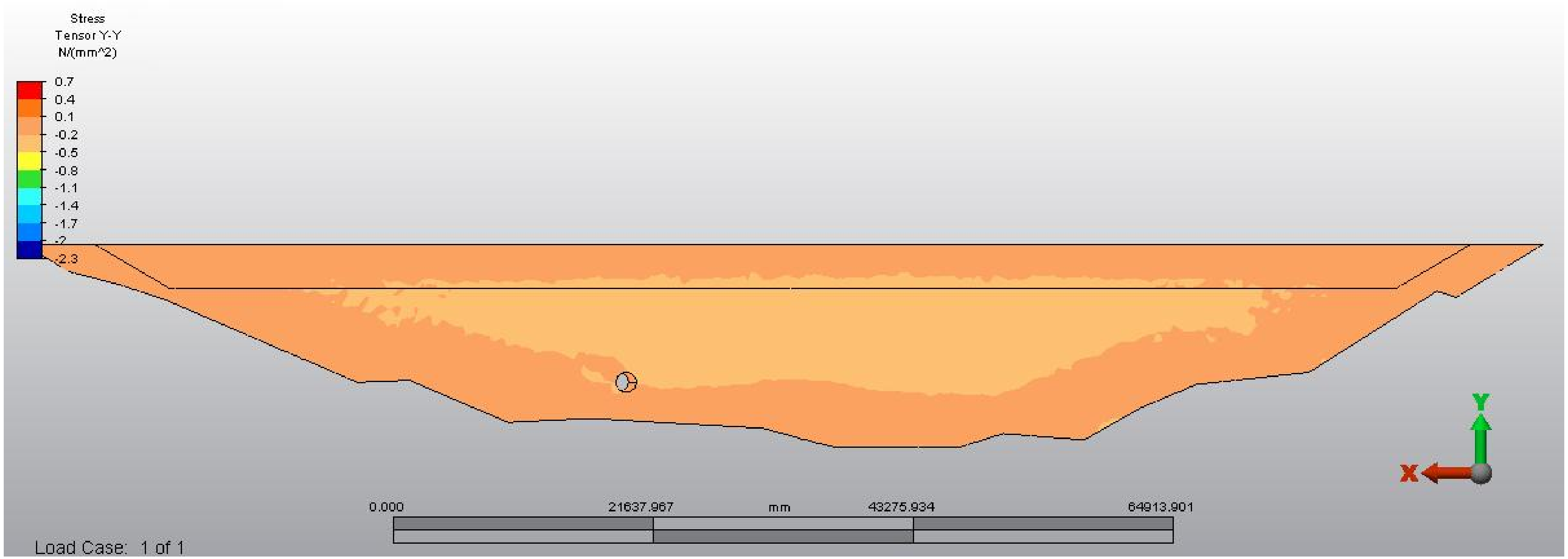
Maximum Principal Stress Contour Plot (Highest Section)



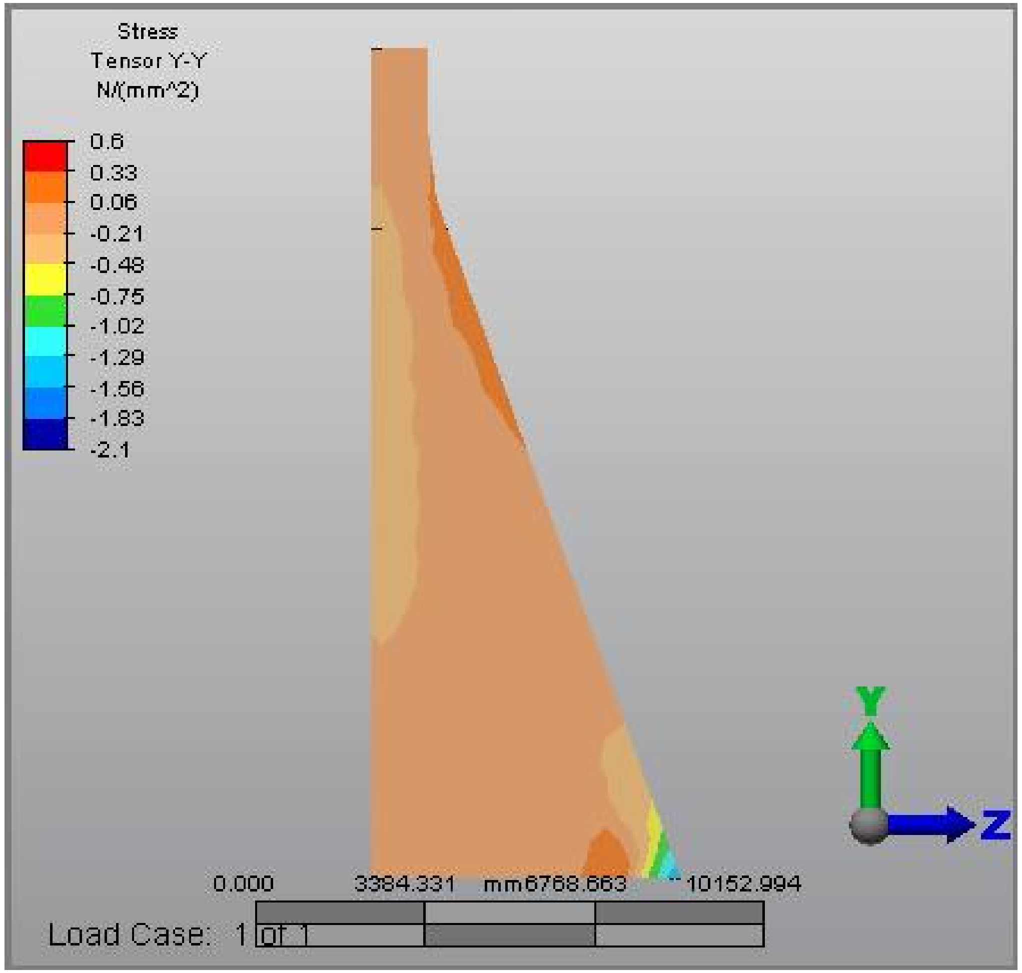
Vertical Stress Contour Plot (Downstream Face)



Vertical Stress Contour Plot (Upstream Face)



Vertical Stress Contour Plot (Highest Section)



Appendix D

Finite Element Stress Analysis Results for the Proposed Remedial Works

Stress Distributions and Displacements for:

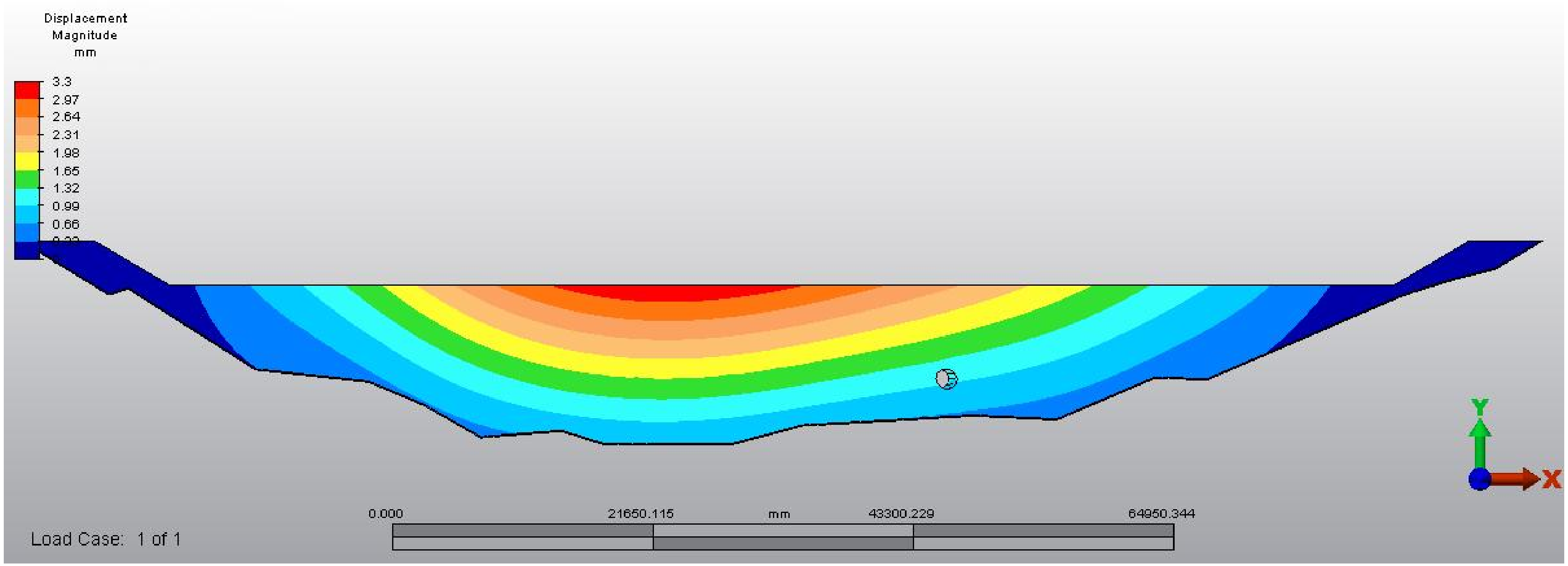
i) 1 in 100,000 AEP Flood loading with Type 1 Uplift Pressure Distribution

ii) 1 in 100,000 AEP Flood loading with Type 2 Uplift Pressure Distribution

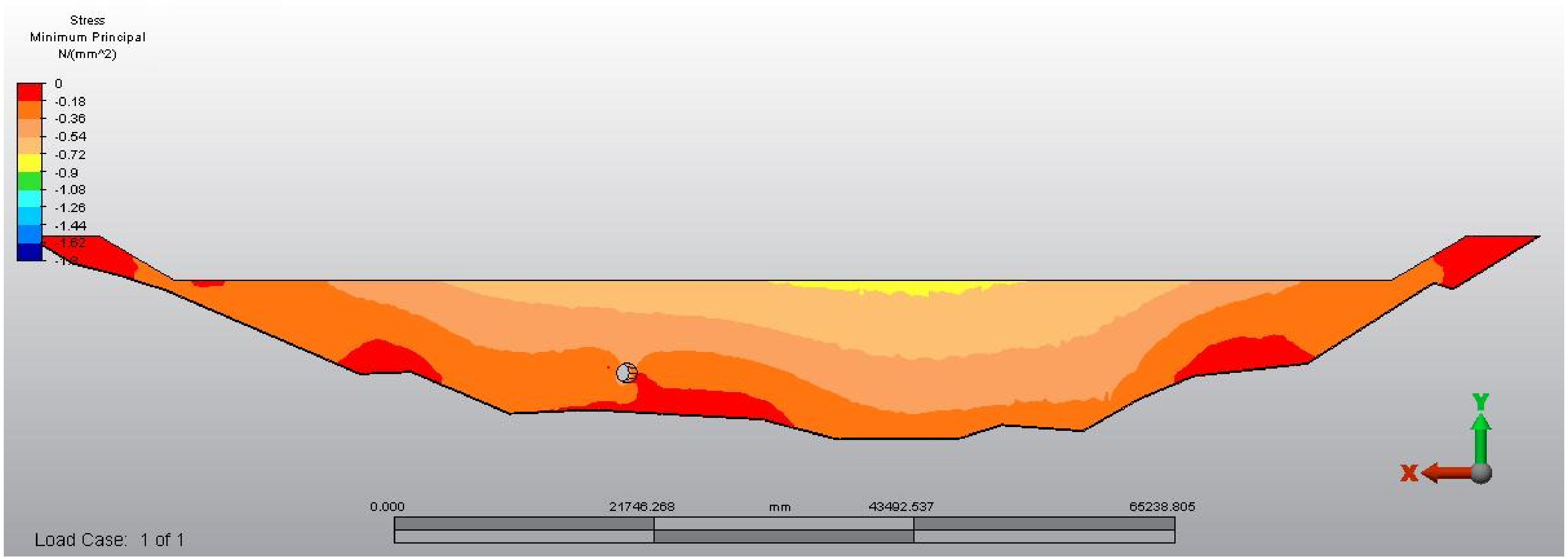
iii) PMP Flood loading with Type 2 Uplift Pressure Distribution

1 in 100,000 AEP Flood loading with Type 1 Uplift Pressure Distribution

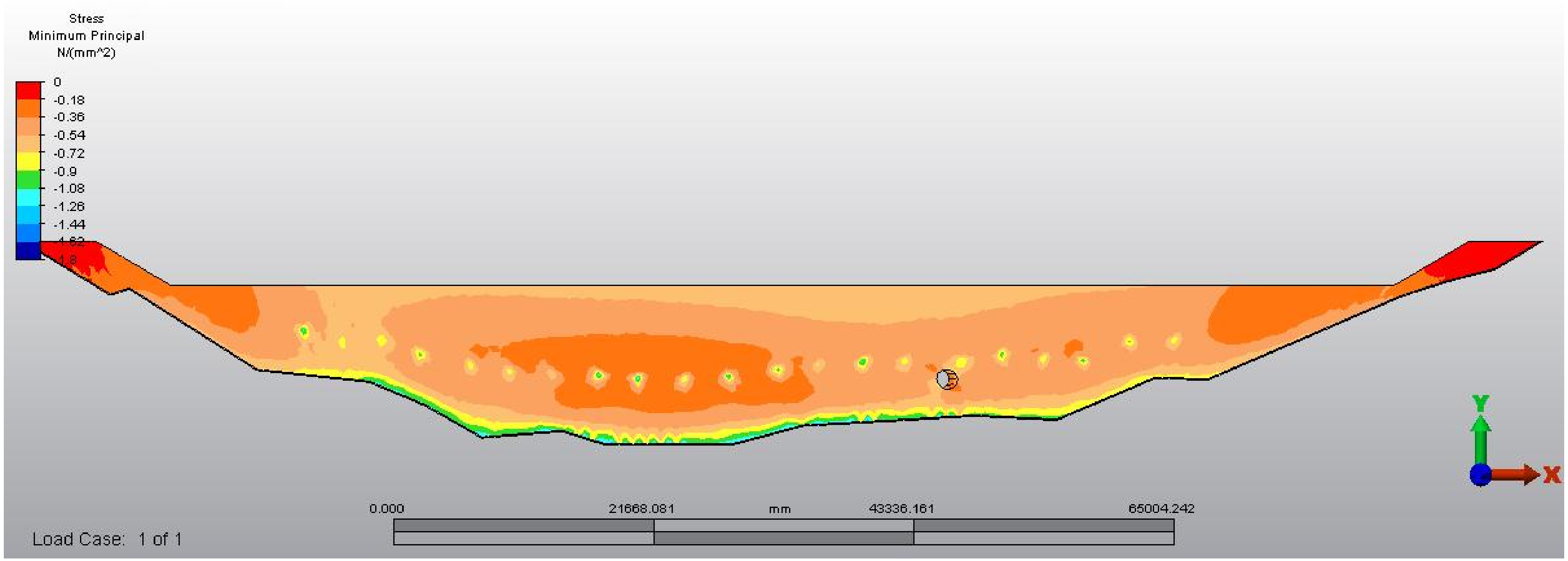
Displacement Contour Plot (Downstream Face)



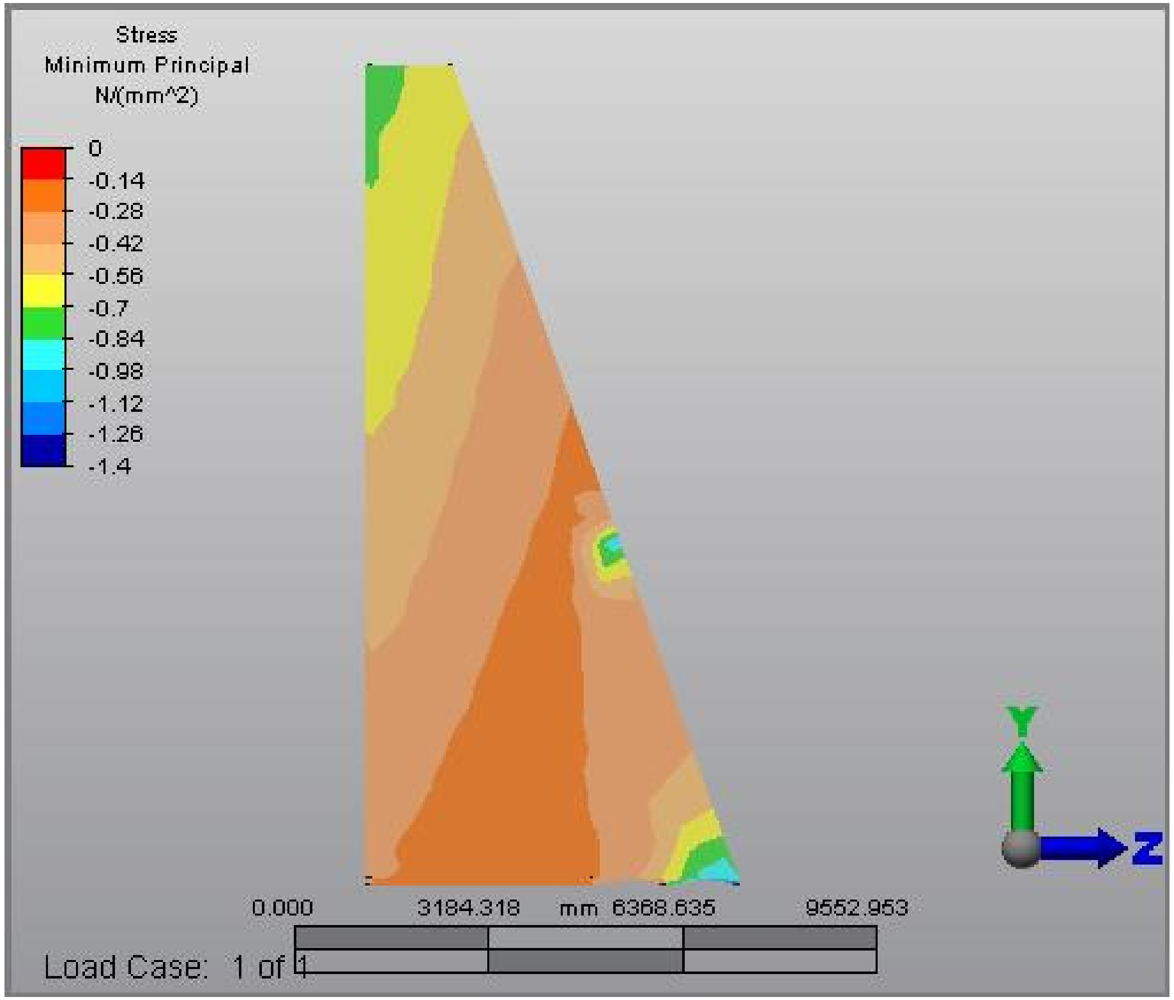
Minimum Principal Stress Contour Plot (Upstream Face)



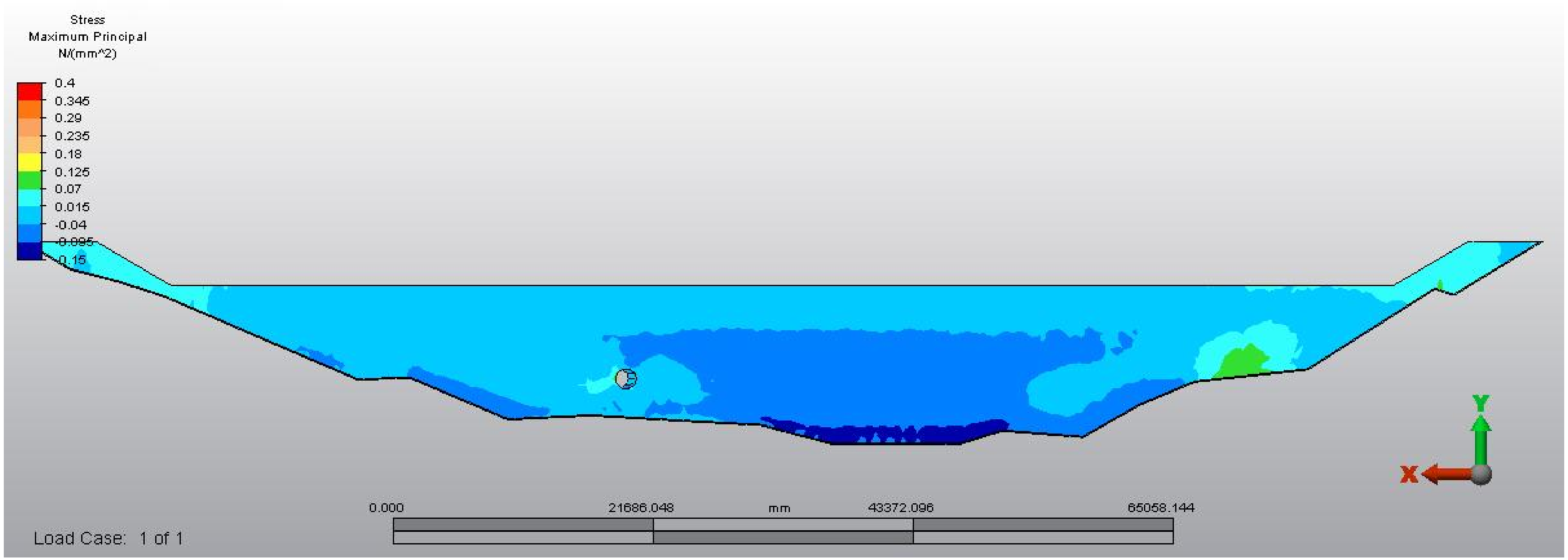
Minimum Principal Stress Contour Plot (Downstream Face)



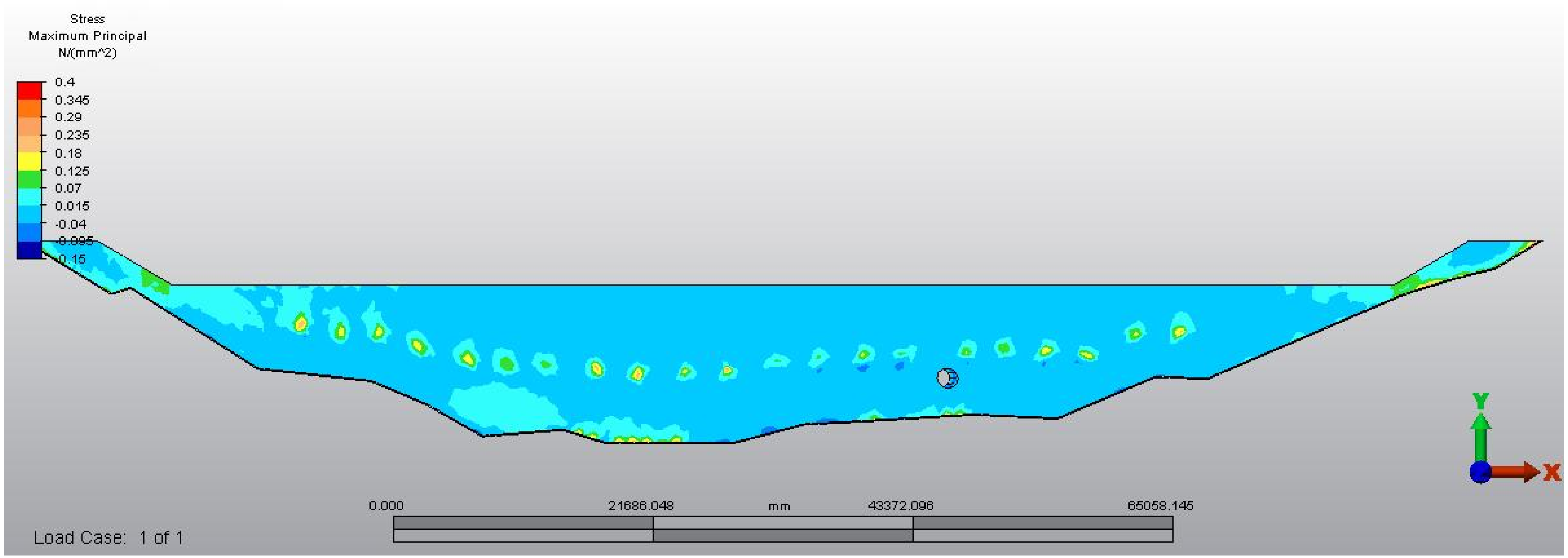
Minimum Principal Stress Contour Plot (Highest Section)



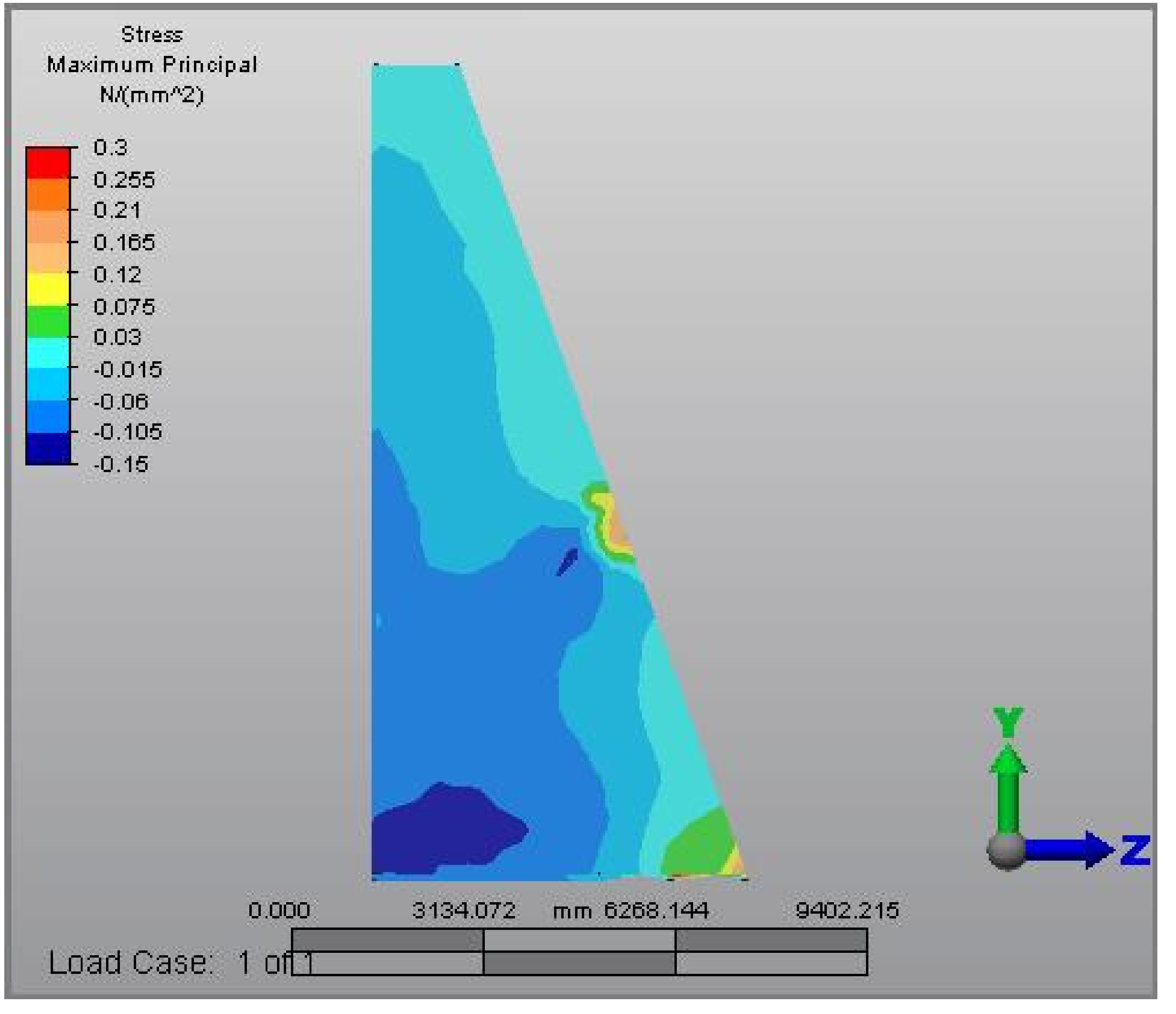
Maximum Principal Stress Contour Plot (Upstream Face)



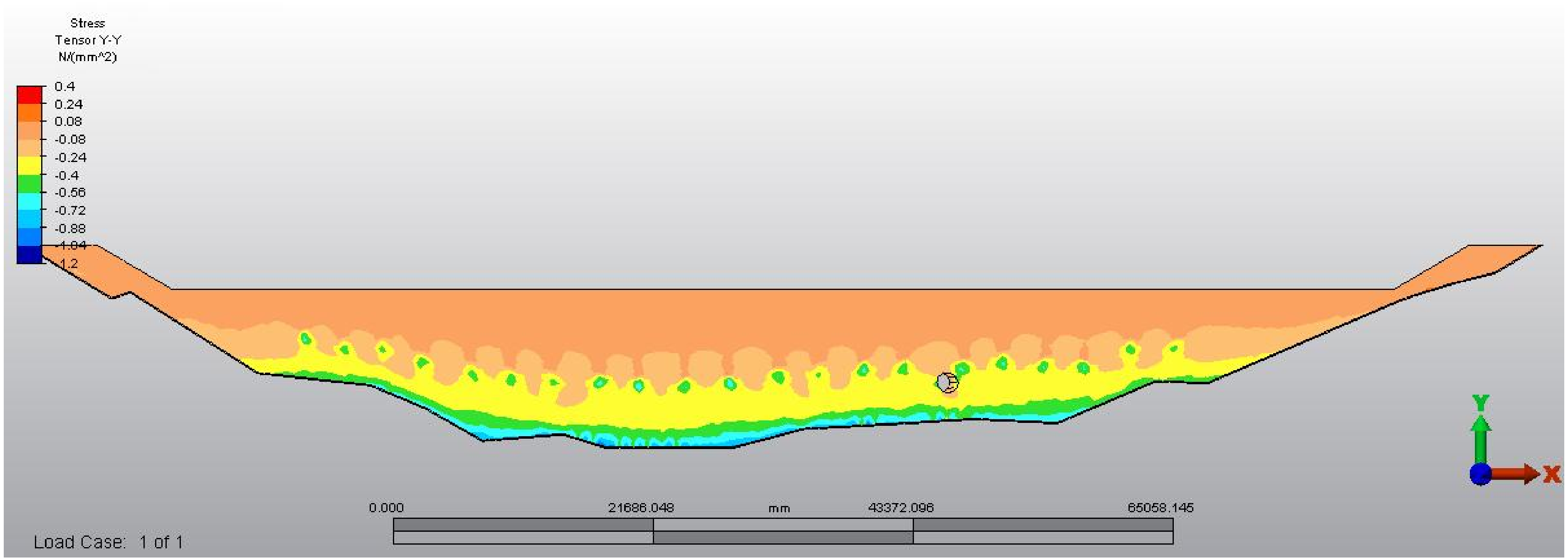
Maximum Principal Stress Contour Plot (Downstream Face)



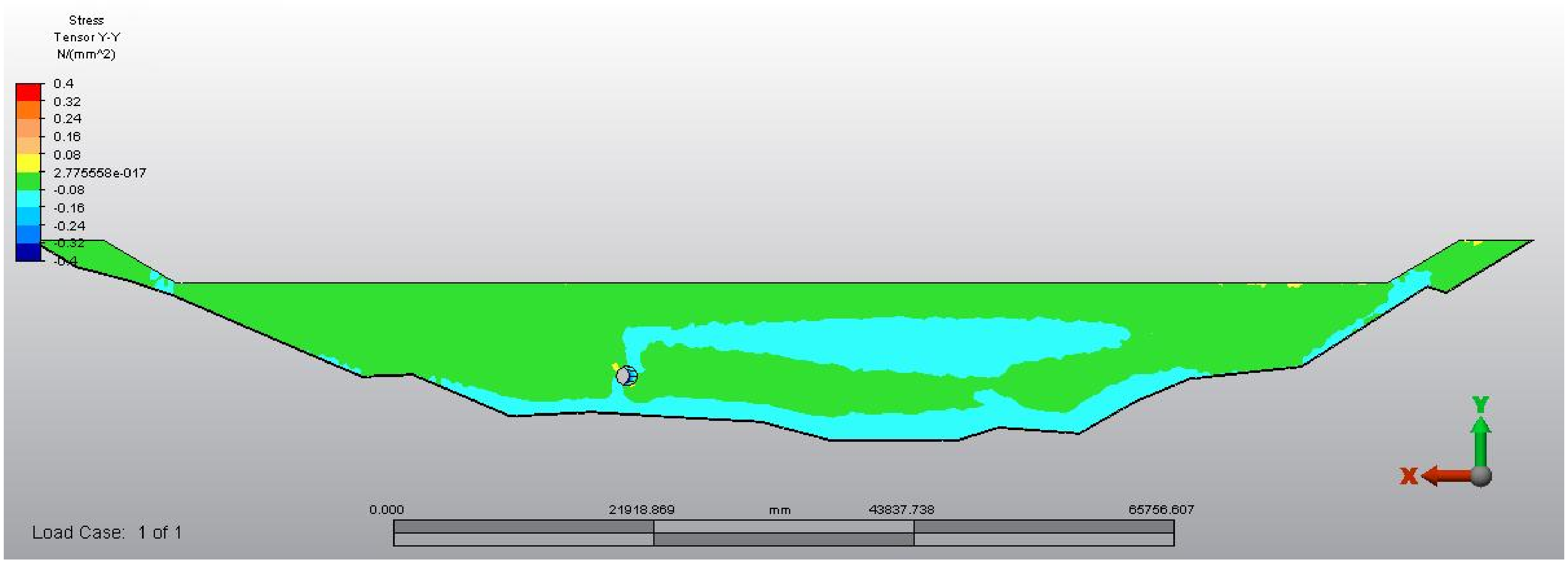
Maximum Principal Stress Contour Plot (Highest Section)



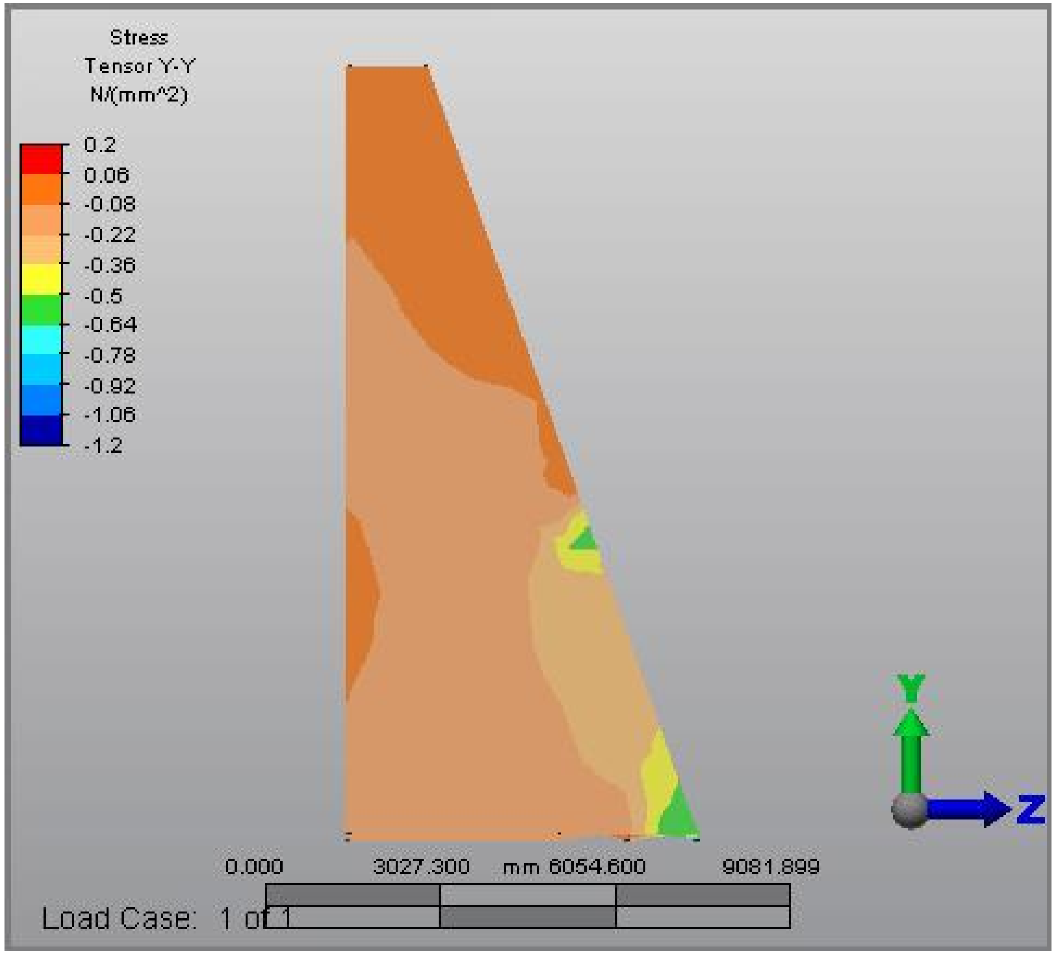
Vertical Stress Contour Plot (Downstream Face)



Vertical Stress Contour Plot (Upstream Face)

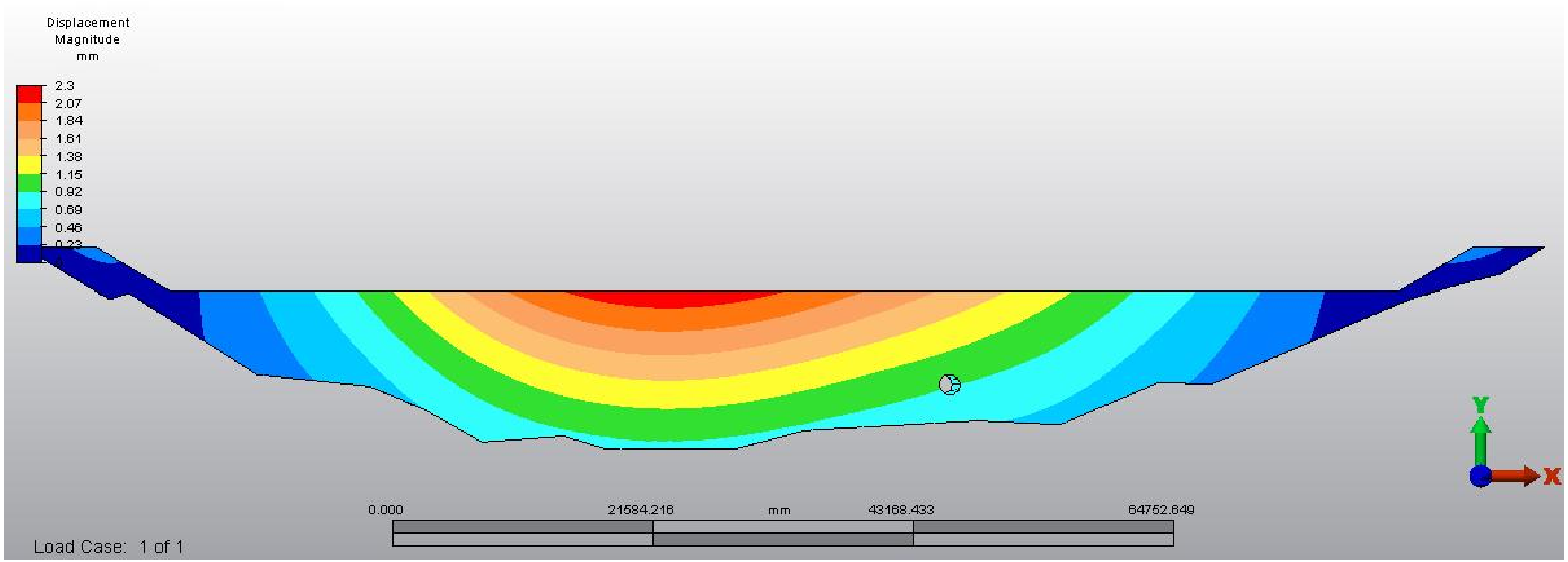


Vertical Stress Contour Plot (Highest Section)

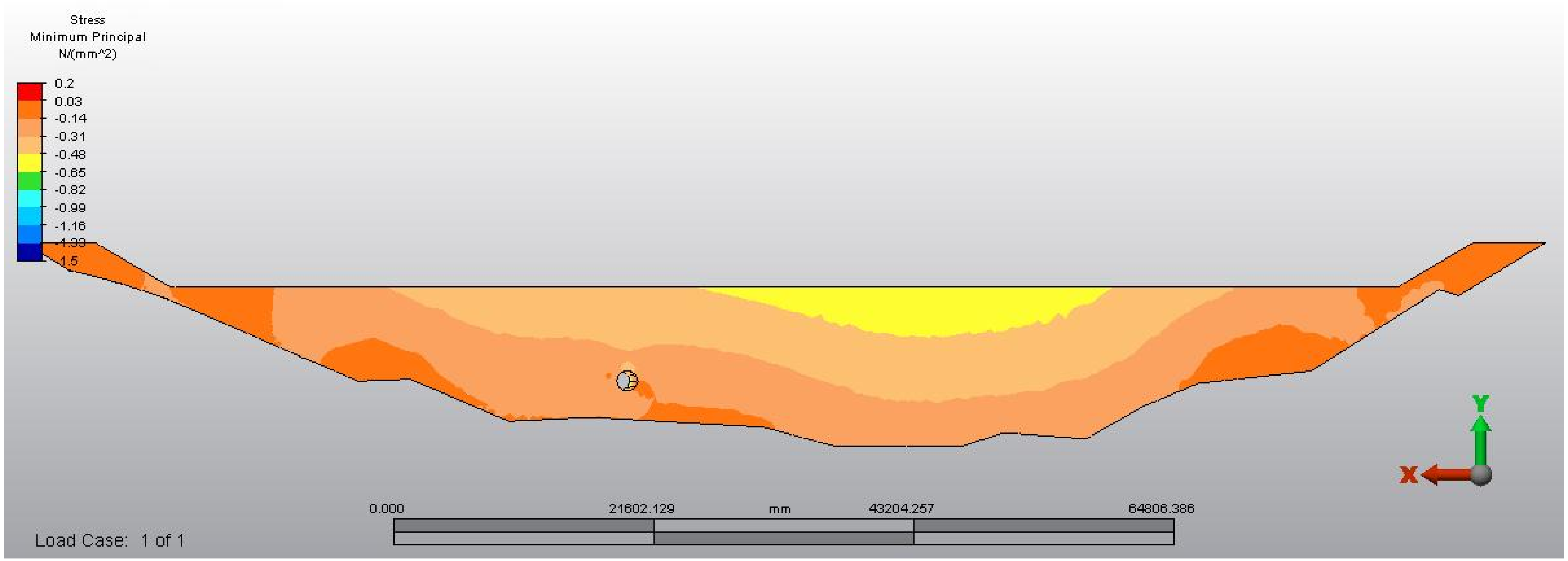


1 in 100,000 AEP Flood loading with Type 2 Uplift Pressure Distribution

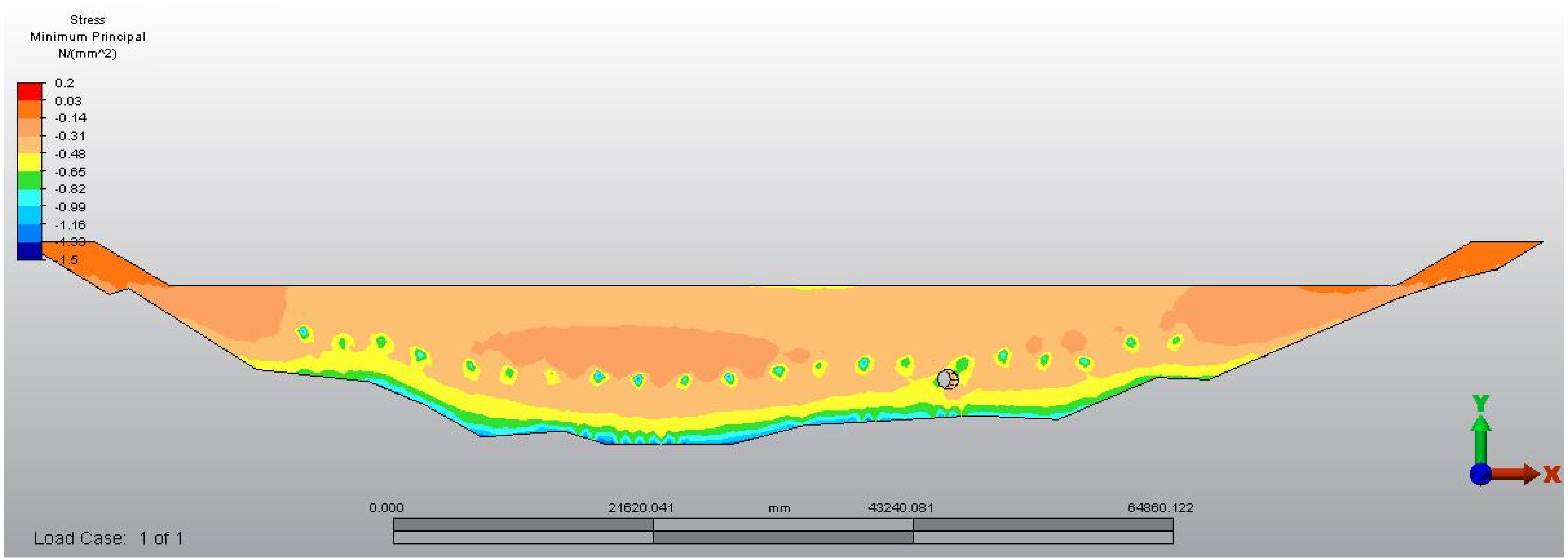
Displacement Contour Plot (Downstream Face)



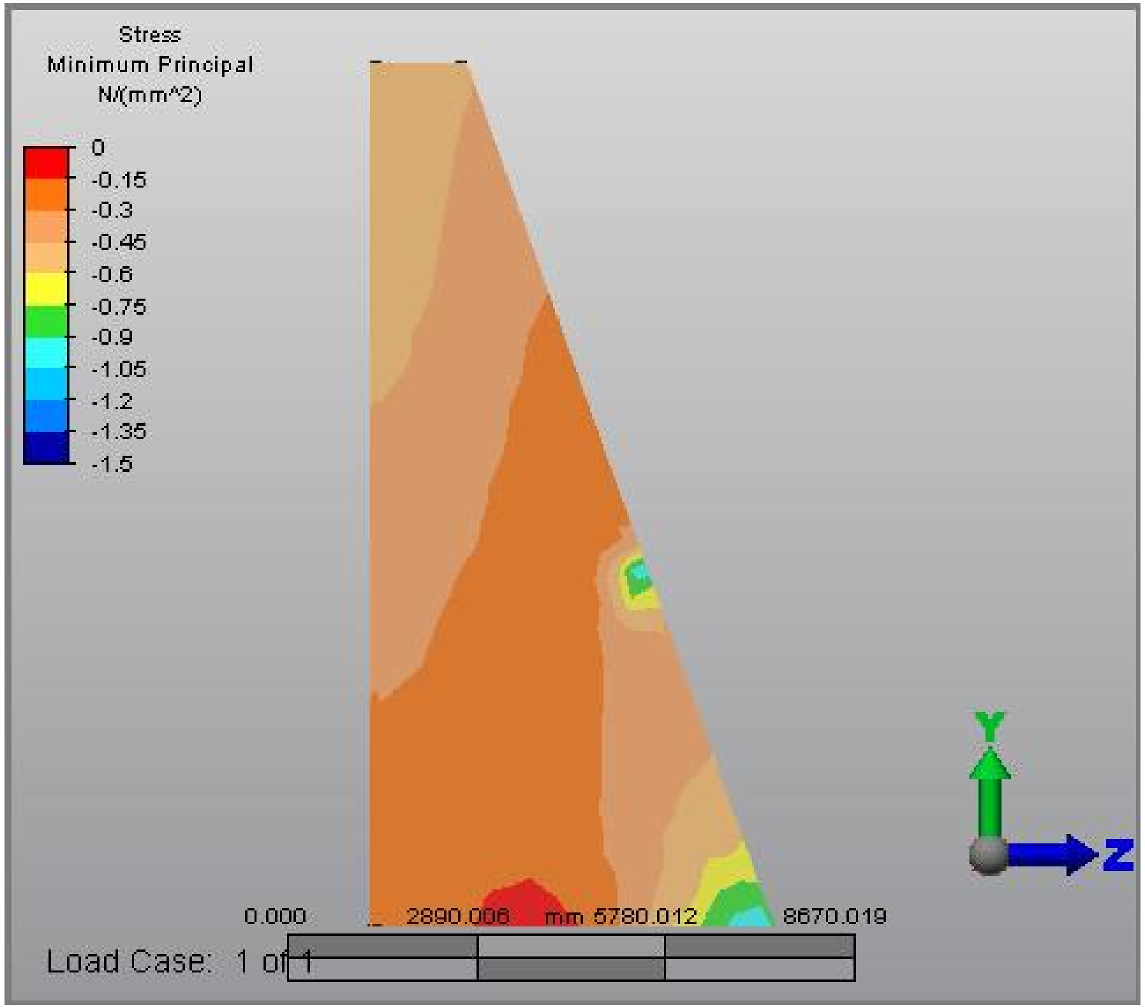
Minimum Principal Stress Contour Plot (Upstream Face)



Minimum Principal Stress Contour Plot (Downstream Face)

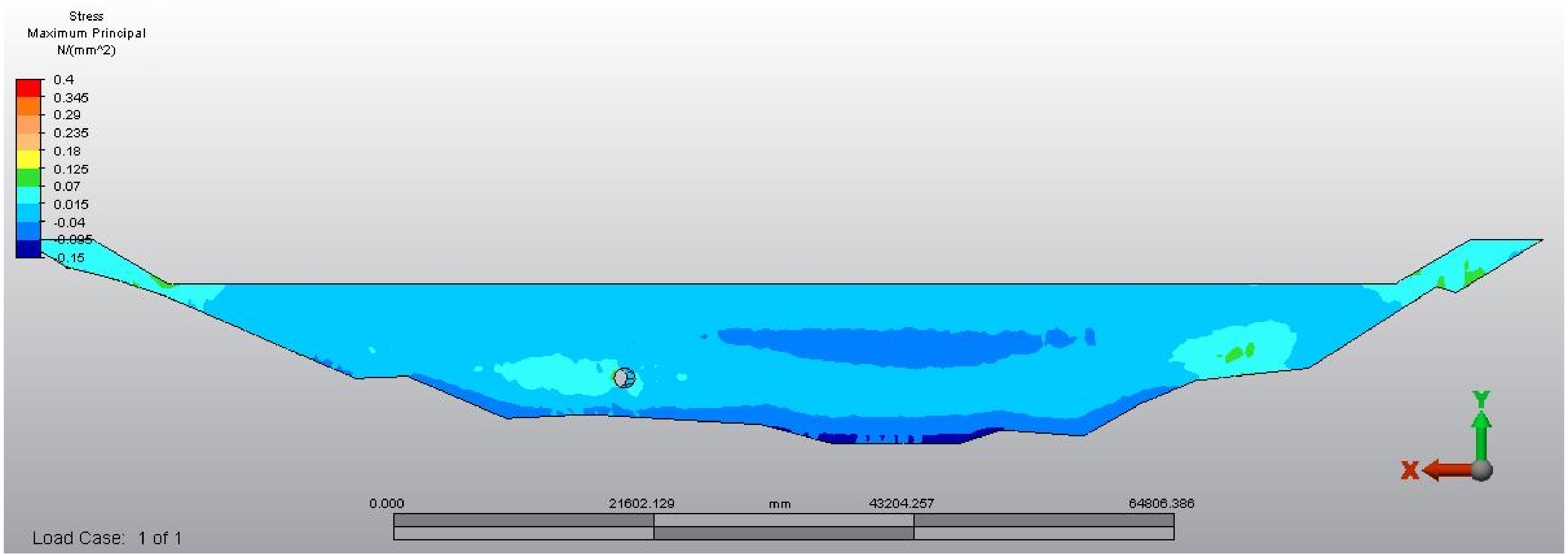


Minimum Principal Stress Contour Plot (Highest Section)



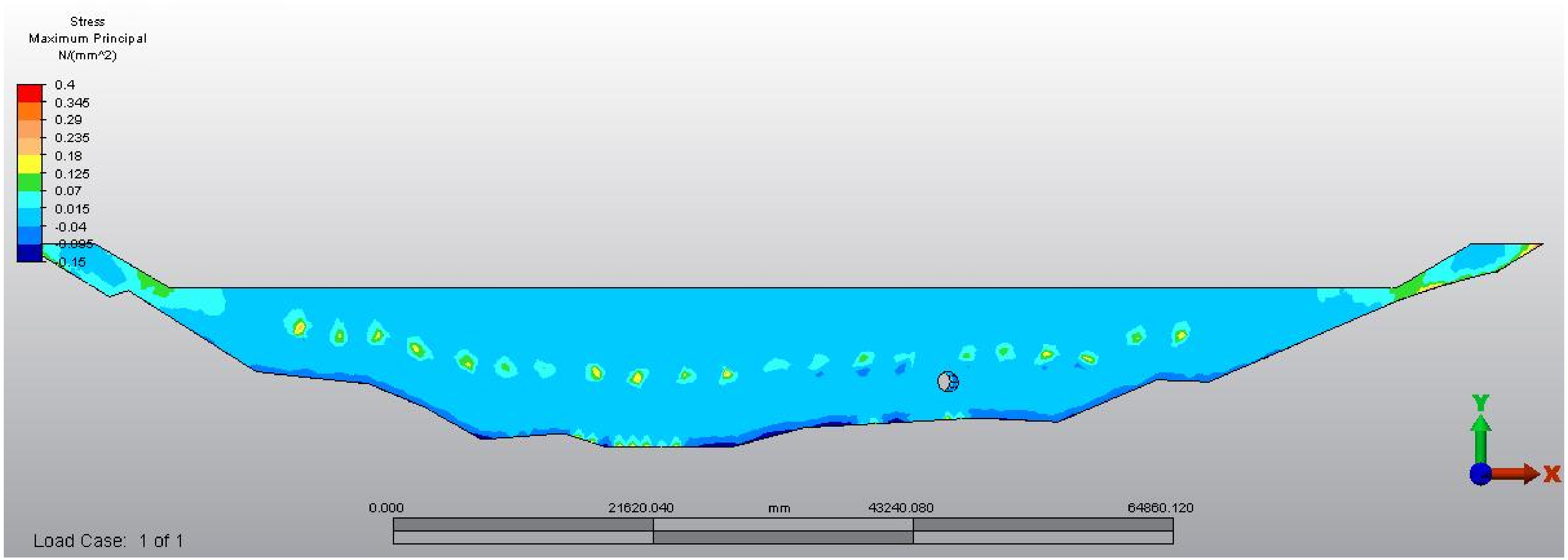
Maximum Principal Stress Contour Plot (Upstream

Face)

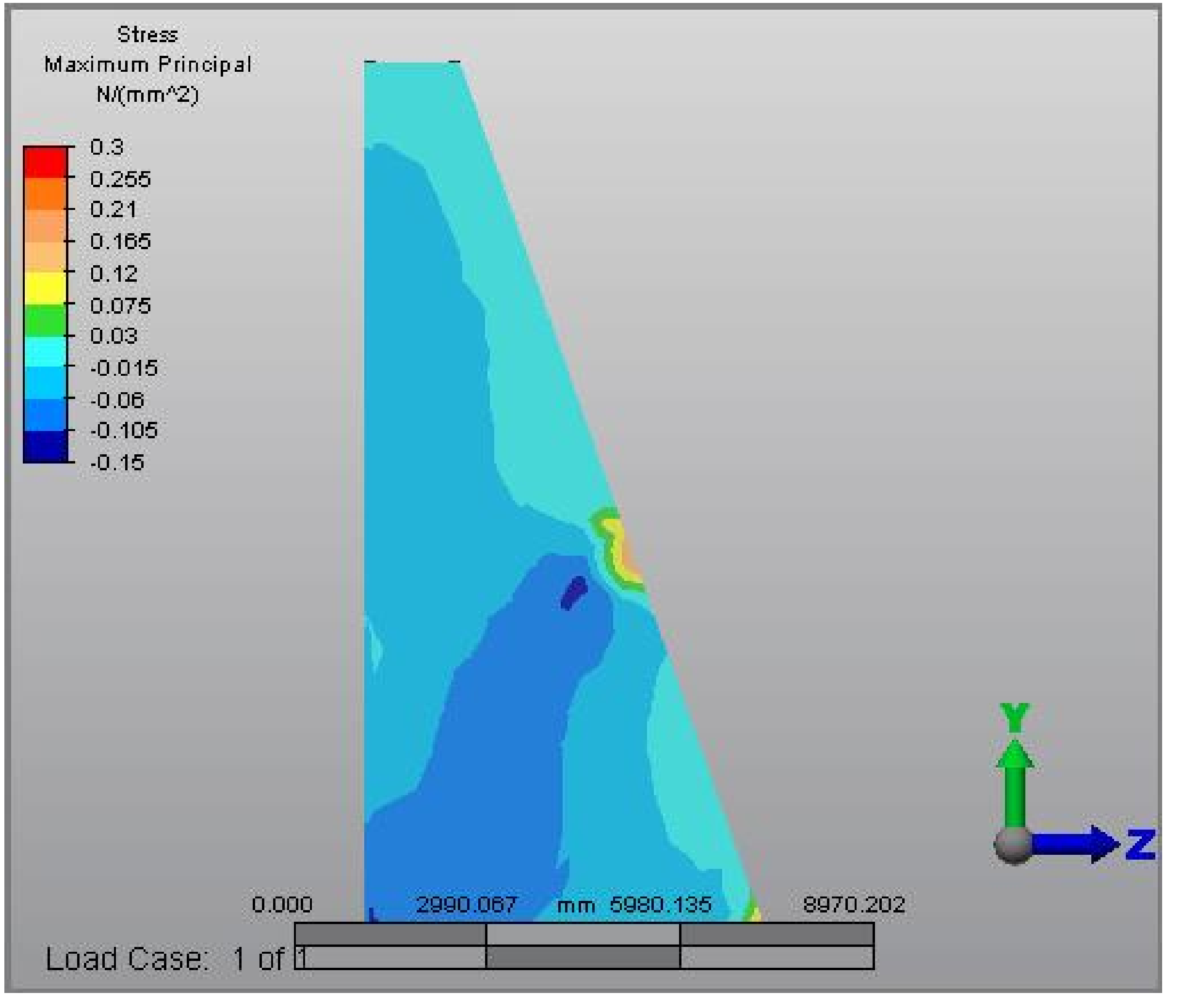


Stress
Maximum Principal
N/(mm^2)
0.4
0.345
0.29
0.235
0.18
0.125
0.07
0.015
-0.04
-0.095
-0.15
0.000
21602.129
mm
43204.257
64806.386
Y
X
Load Case: 1 of 1

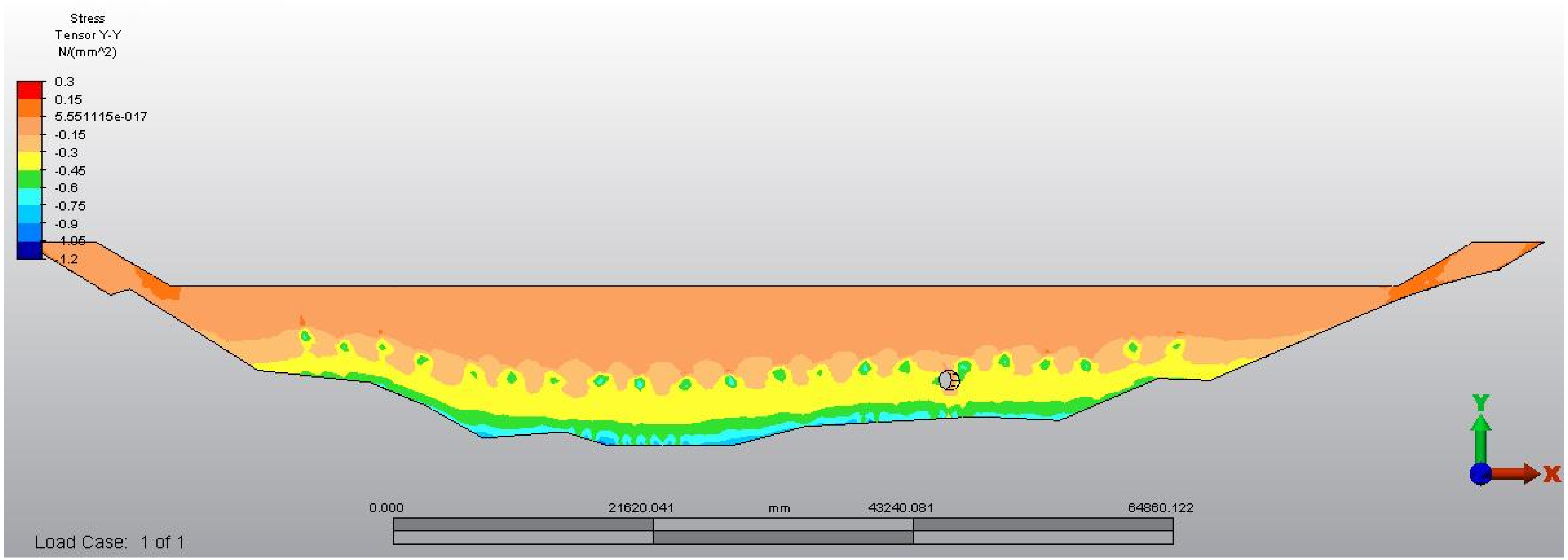
Maximum Principal Stress Contour Plot (Downstream Face)



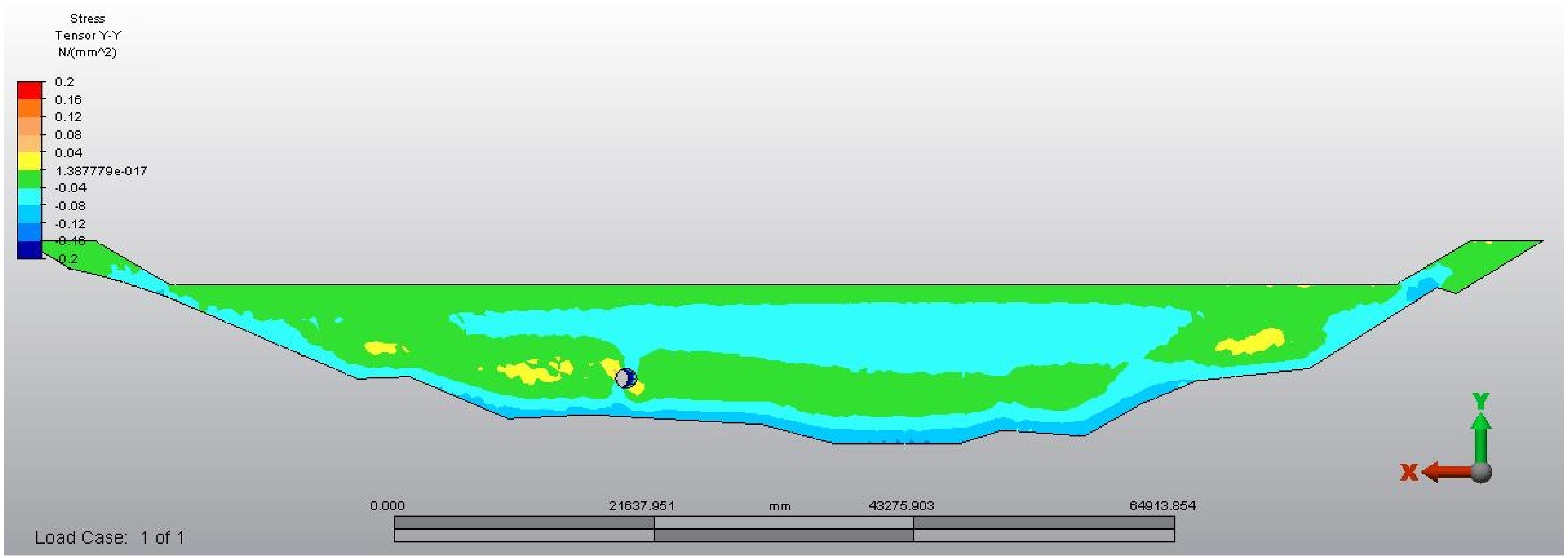
Maximum Principal Stress Contour Plot (Highest Section)



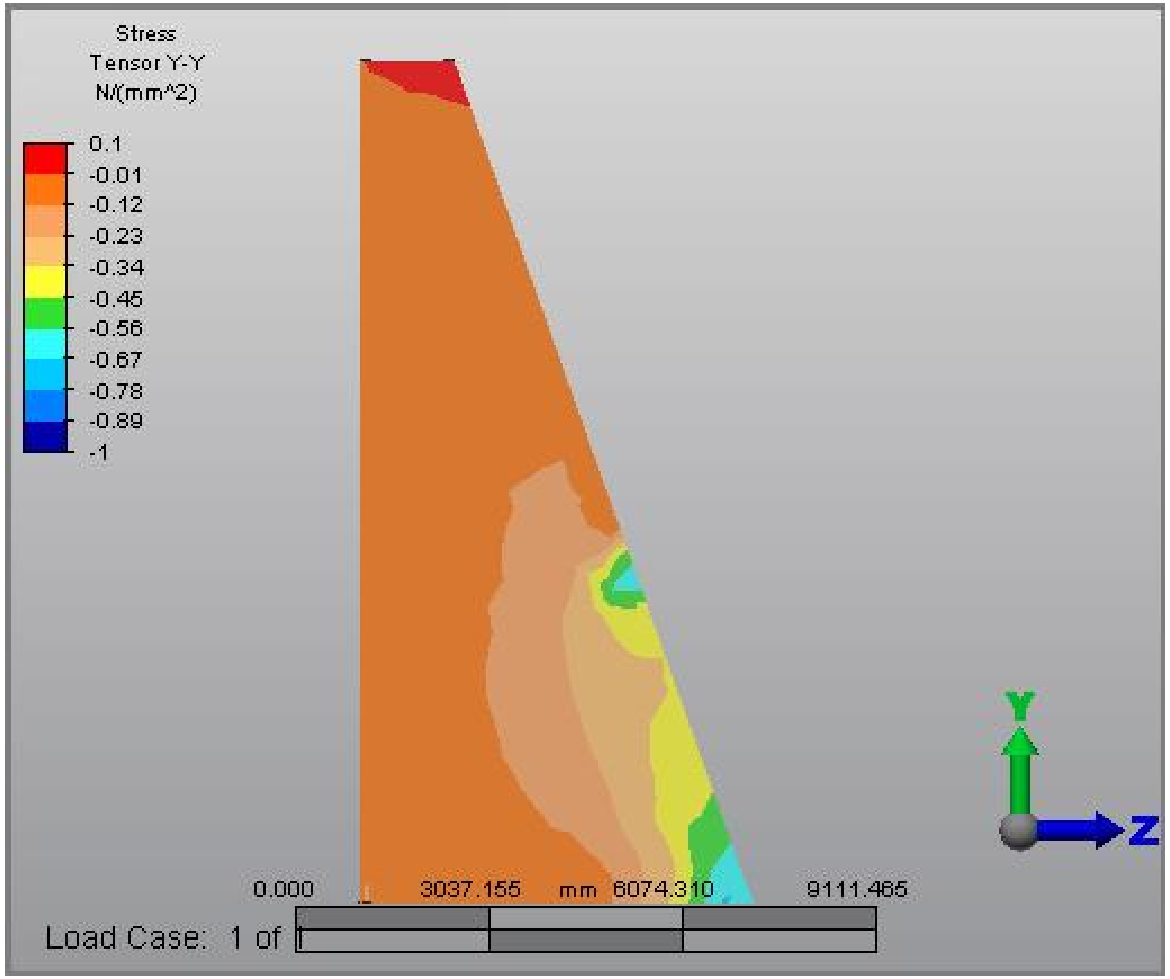
Vertical Stress Contour Plot (Downstream Face)



Vertical Stress Contour Plot (Upstream Face)

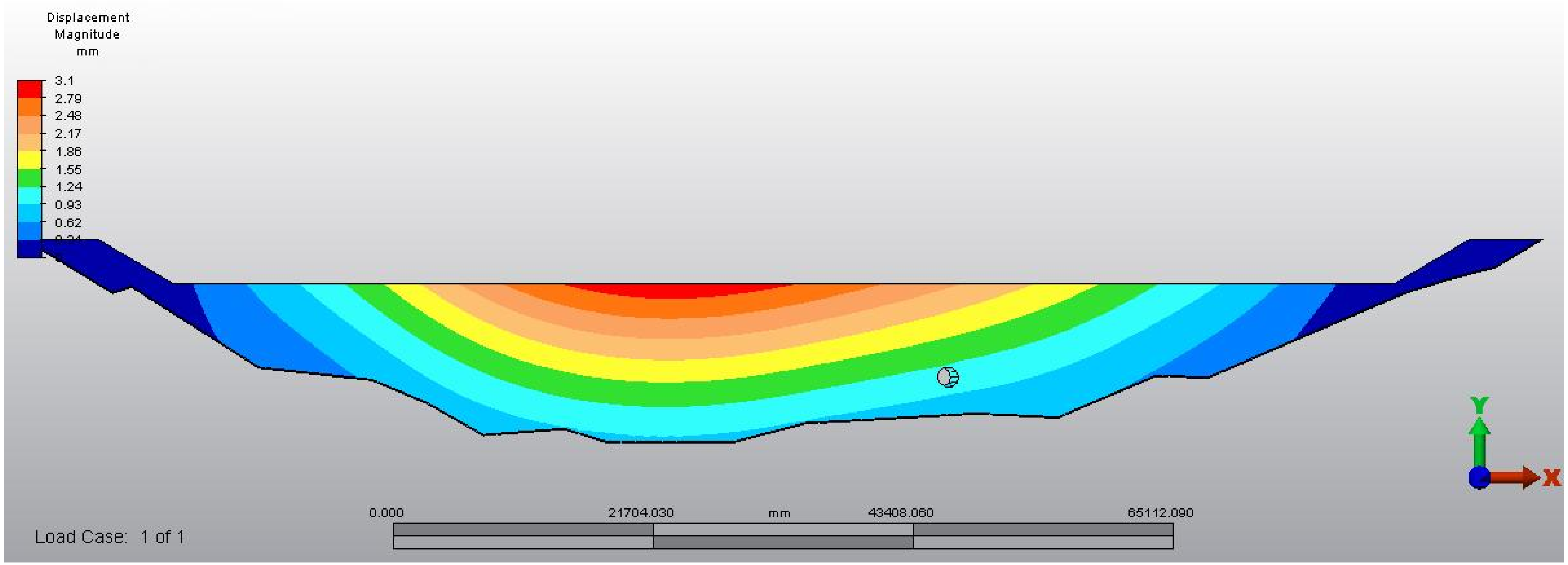


Vertical Stress Contour Plot (Highest Section)

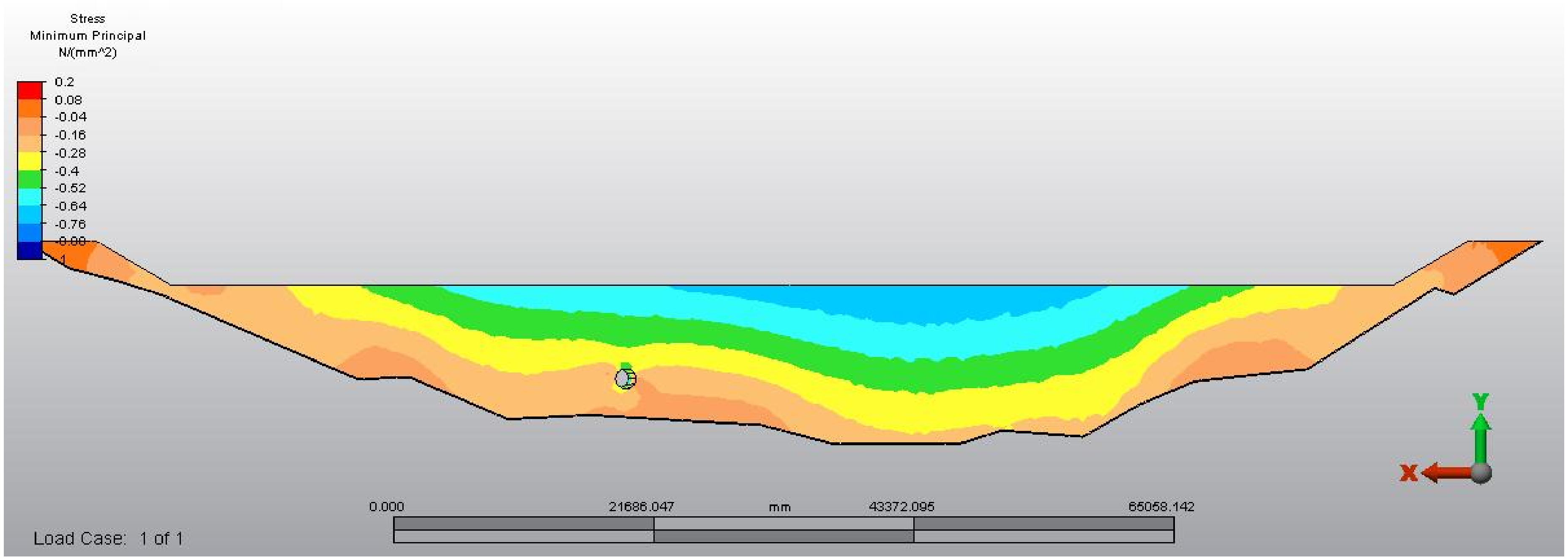


PMP Flood loading
with Type 2 Uplift Pressure Distribution

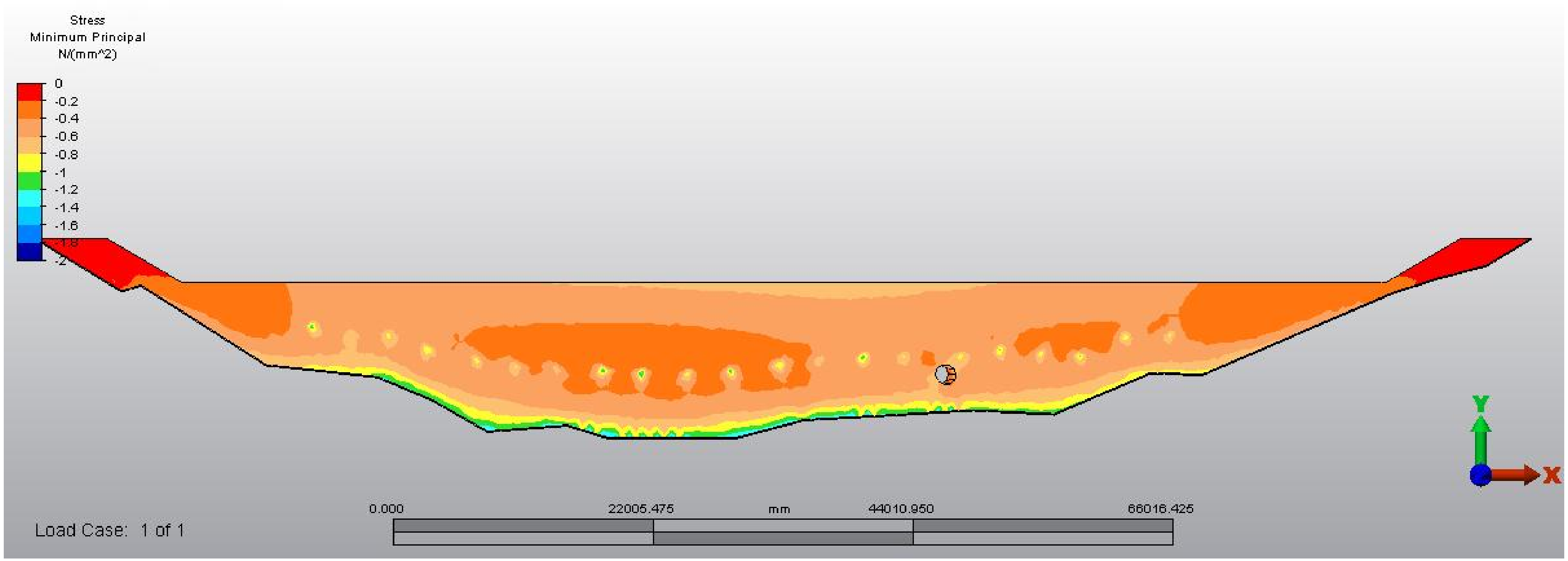
Displacement Contour Plot (Downstream Face)



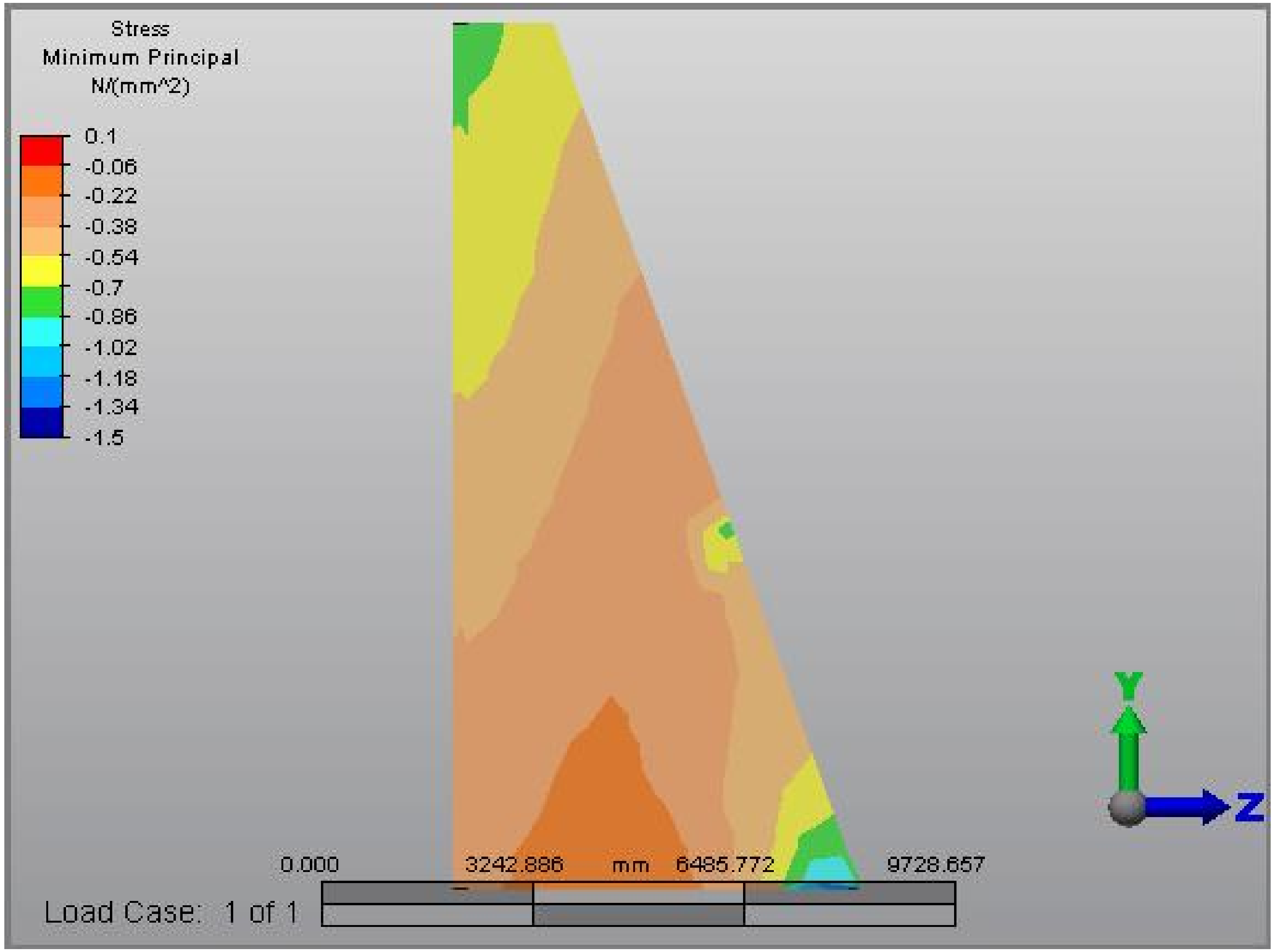
Minimum Principal Stress Contour Plot (Upstream Face)



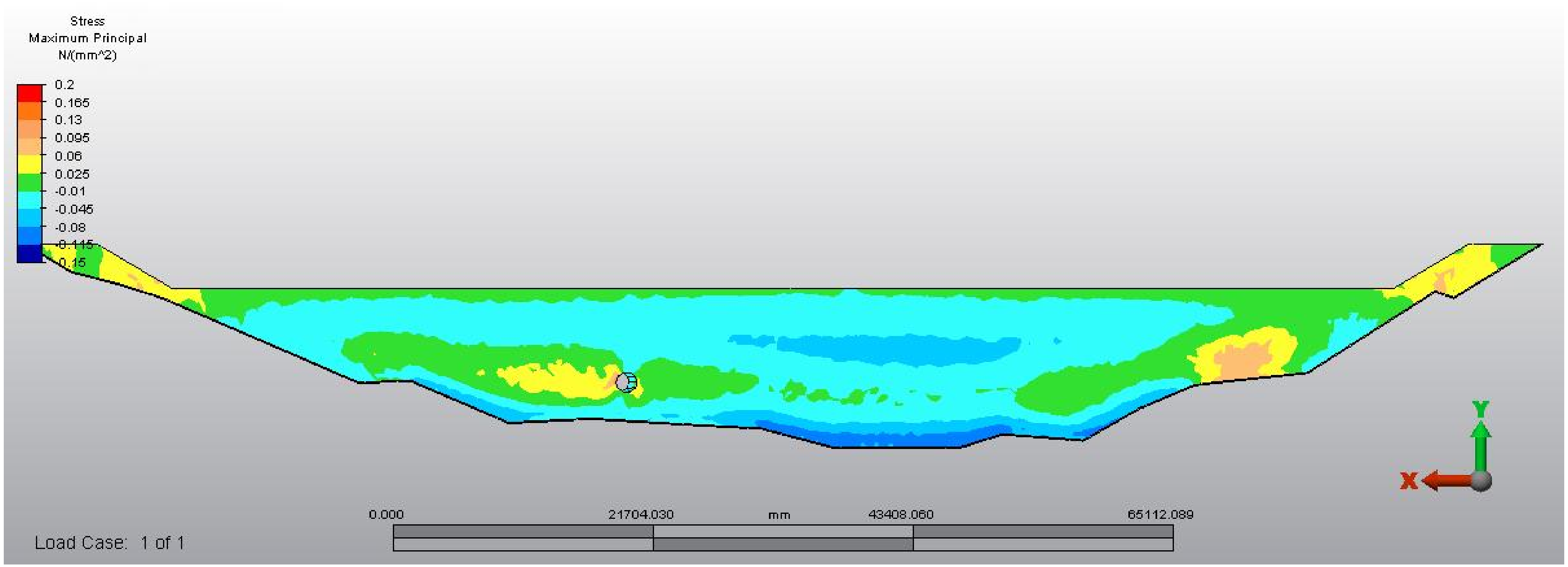
Minimum Principal Stress Contour Plot (Downstream Face)



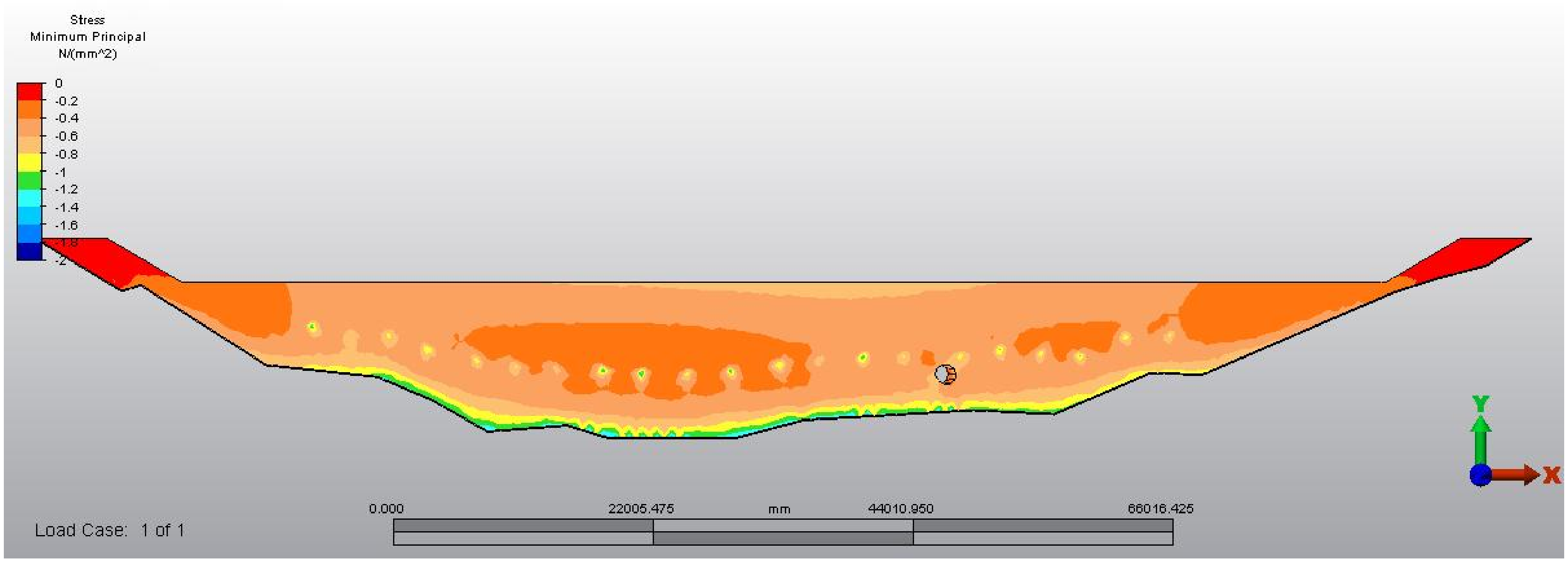
Minimum Principal Stress Contour Plot (Highest Section)



Maximum Principal Stress Contour Plot (Upstream Face)

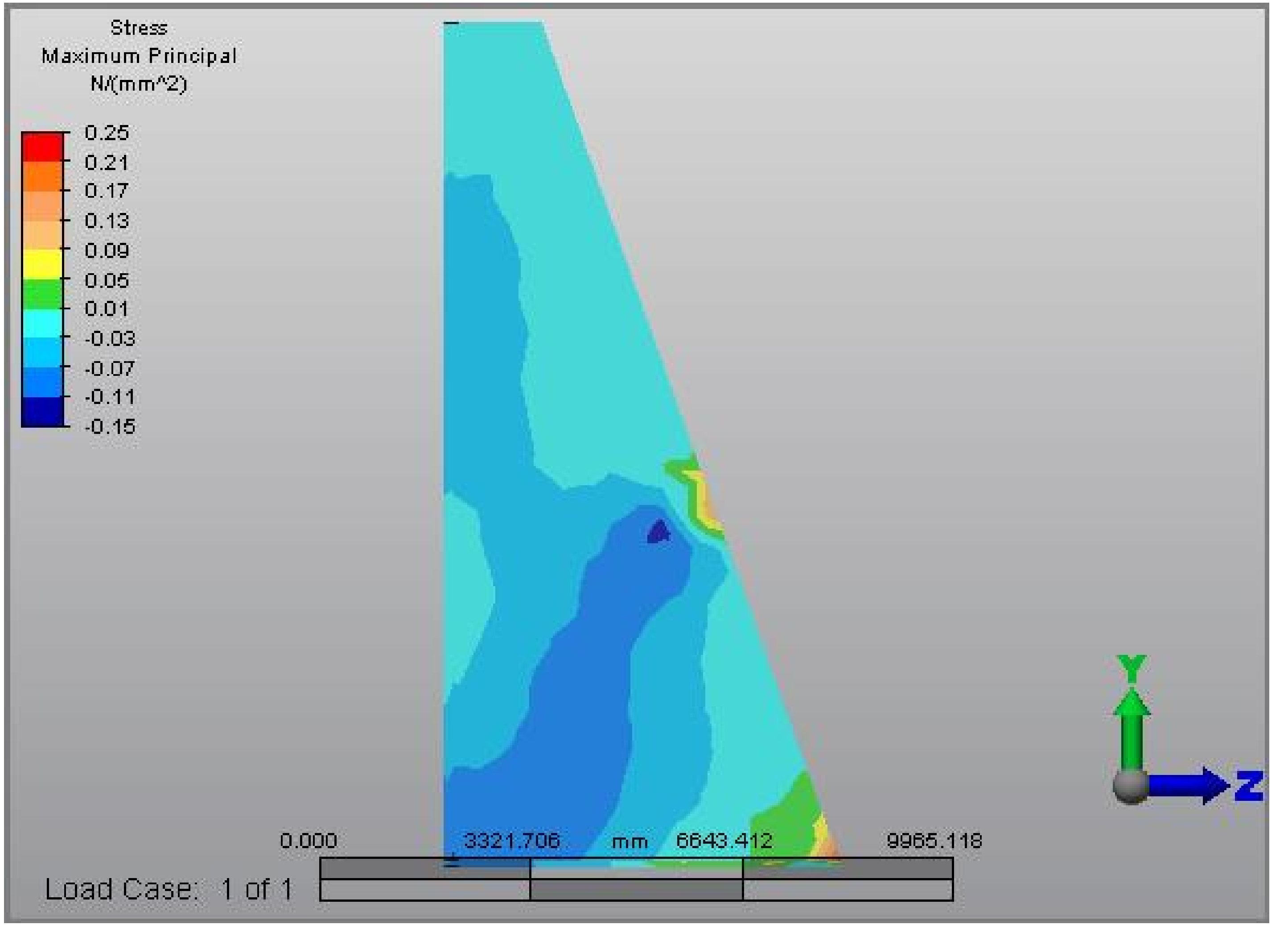


Maximum Principal Stress Contour Plot (Downstream Face)

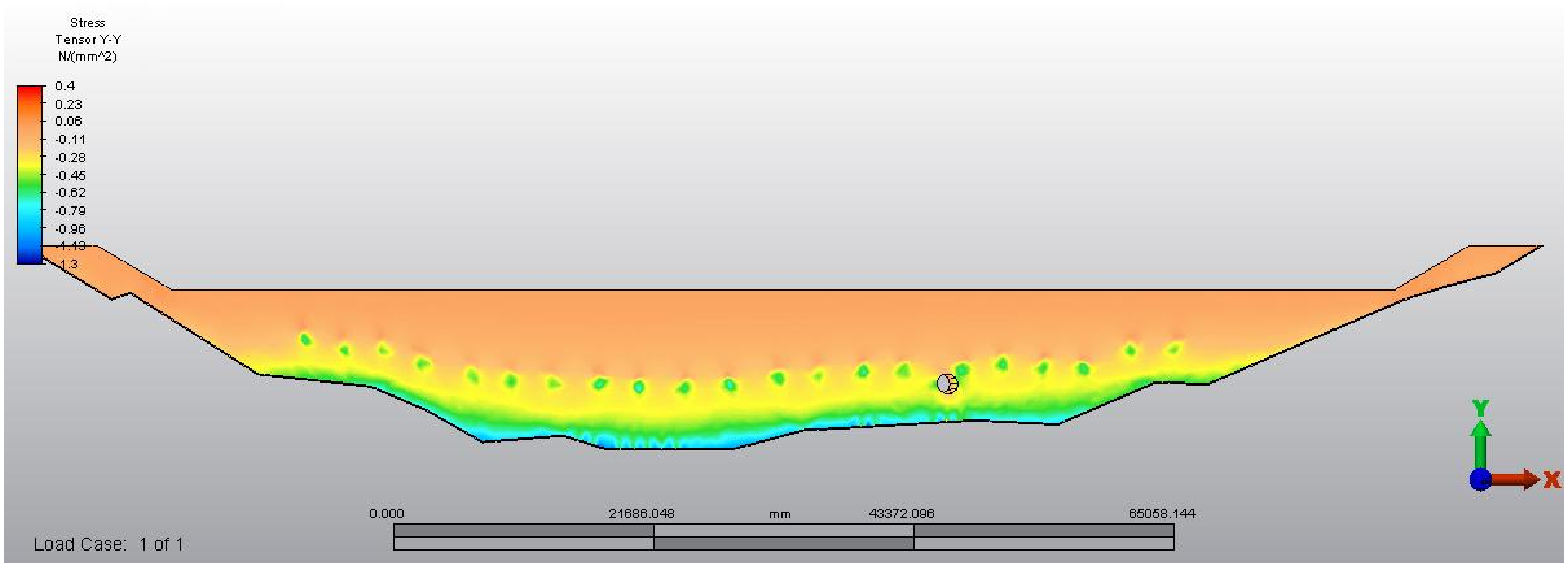


Redbank Creek Dam
Alternative Stabilisation Works

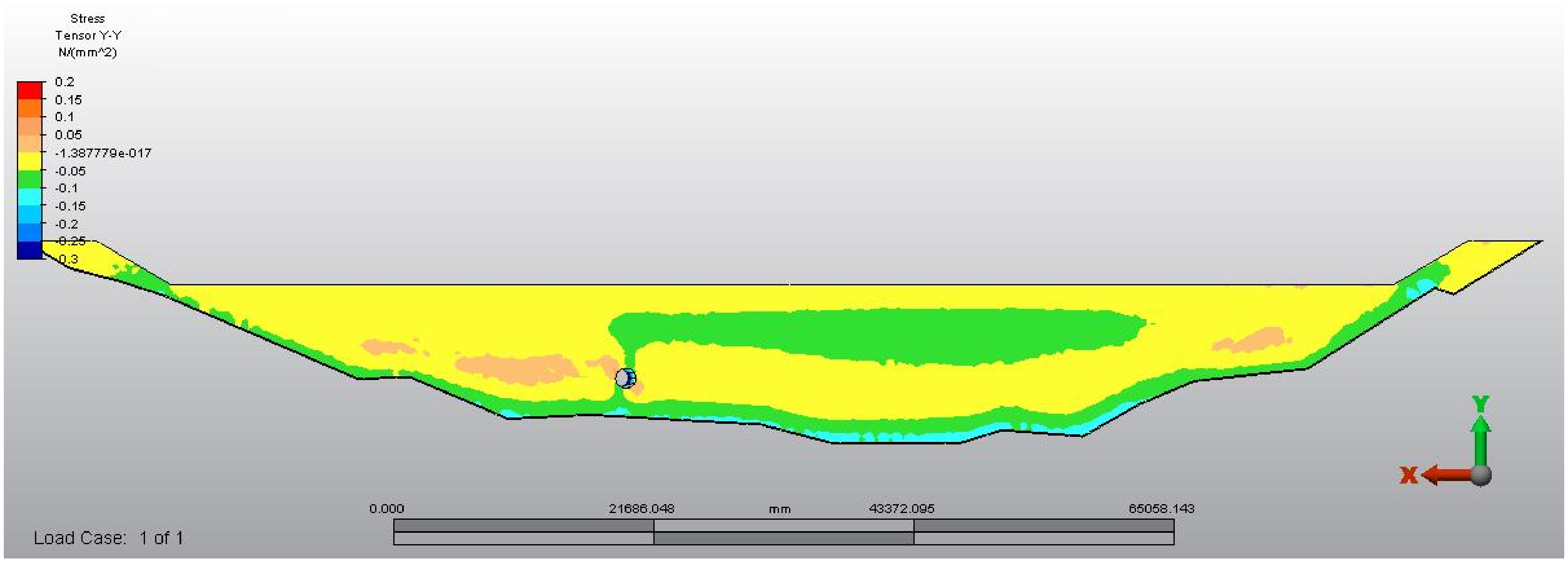
Maximum Principal Stress Contour Plot (Highest Section)



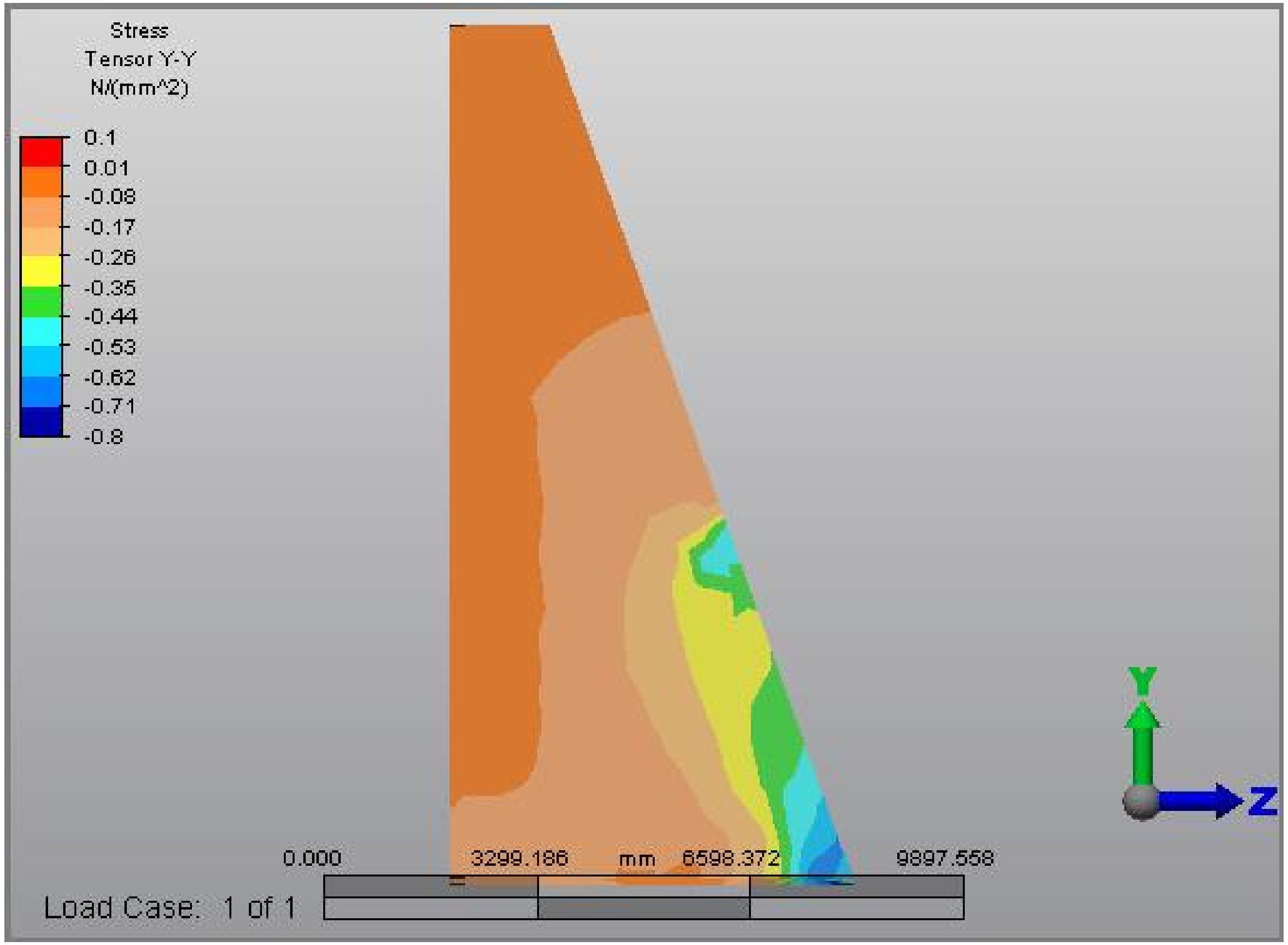
Vertical Stress Contour Plot (Downstream Face)



Vertical Stress Contour Plot (Upstream Face)



Vertical Stress Contour Plot (Highest Section)



Appendix E

Peer Review Comments

(Note: These comments have been taken into account in this report)

Review of Report by GHD

to Mid Western Regional Council

on Alternative Stabilising Works for Redbank Creek Dam

Review prepared by Norman Himsley BE, MEngSc, FIE Aust, CPEng

4 June 2010

Background

I was requested by Brett Corven of Mid Western Regional Council in late April 2010 to conduct a peer review of GHD's design process for alternative stabilising works for Redbank Creek Dam.

My understanding of my role was that, as an engineer with expertise in dam design and safety management, I was to interact with GHD at key points in its design process to review progress and provide an independent assessment of the approaches used and conclusions reached in the upgrading design report to be presented to the NSW Dams Safety Committee (DSC).

Process Review

I had an initial afternoon meeting with GHD design personnel on 28 April 2010 where GHD personnel outlined their proposed risk-based design process for evaluating and designing alternative stabilisation works for the upgrading of Redbank Creek Dam. Various stabilisation proposals were briefly discussed and I confirmed that their proposed risk-based process should be in accordance with new DSC policies.

Subsequently, on 28 May 2010, GHD emailed me a draft copy of their design report. I reviewed the initial draft report and made the following reply to GHD (copy to Brett Corven):

"After a quick read of Redbank Creek dam draft report my initial impression is that GHD have done some good work with a pleasing result. Some quick comments before I read in detail over weekend are:

Section 4.8/4.9-Stress that only modest strengths are required in the existing dam (particularly joints and foundation interface) to give stability at FSL and that the dam has better than those strengths to have survived especially during higher loading in overtopping events.

Section 5.7-Stress upgraded dam meets sliding criteria (especially with new toe anchors), there is NO tension in the foundation interface, compressive stresses are well within normal criteria. Also stress that, although joint stresses are typically below 100kPa, the max joint tensile stress of 200kPa is one third less than what dam has sustained for long periods at FSL and now would only briefly meet this level under a very rare extreme flood event.

Section 6-Stress points in 5.7 above and the fact that new PMP flood levels will be 2.5m below the previous FSL meaning significantly lower reservoir and uplift loadings with small loss of dam weight and better dam rigidity in the new "stockier" dam.

An aside is it worth looking at a cut down of only 3m to the existing crack in the dam. You would need to check stresses but it would have the advantages of easier cut down to a defined joint (and less concrete removal), better flood mitigation (eg around 1 in 1,000) and consequently less need to initially put in place toe protection for overtopping."

I further phoned GHD personnel to ensure when finalizing the report it stresses the need for toe anchorage as modeled and provides appropriate comment on the need, if any, for drainage provisions.

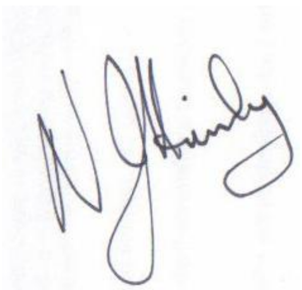
Report Review

Brett Corven forwarded to me the final draft of the alternative stabilisation works report on 3 June 2010 and the following specific comments on the report are provided for consideration by the designers in finalising their report:

Section	Specific Comments
Sentence below Table 1	Note that stresses have not been determined as yet in report and may not be below existing as there is less compressive force from cut down structure. However, it can be asserted that future water loadings (ie thrust and uplift) will be substantially lower than any previously experienced.
3	Good summary and reasonable assumptions of strengths
Tables5&6	Refer to interface tension capability of 0.2MPa whereas Section3.3 adopts 150kPa
4.9	Endorse discussion (now note that crack is placed at 3.7m compared to 3m in initial report)
5.7	First dot point (in both lots of dot points) should refer to the “transient flood” stresses
5.7	5th dot point in second lot of dot points is misleading with “less arching action to resist loading”-should refer to removal of more flexible, upper arch sections leading to stockier, stiffer remaining dam
6.2.2 Item2	Mention should be made that provision of substantial toe anchorage will provide substantial resistance to crack opening in this area and lower need for drainage

Conclusions

Subject to the specific comments above, I endorse the approaches used, and the Section 6-Conclusions and Recommendations, in the report.



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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
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