Hanson Construction Materials

martens consulting engineers

Flood Study:

Concept Plan for the redevelopment of Lot 11 DP558723, Lot 1 DP400697 and Lot 2 DP262213 Eastern Creek, NSW.

ENVIRONMENTAL

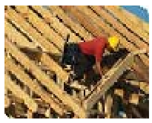
WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT MANAGEMENT



P0601396JR03_v2 Final Report
October 2006

Limitations Statement

The sole purpose of this report and the associated services performed by Martens & Associates Pty Ltd is to undertake a Flood Study in accordance with the scope of services set out in the contract / quotation between Martens & Associates Pty Ltd and Hanson Australia Pty Ltd (hereafter known as the Client). That scope of works and services were defined by the requests of the Client, by the time and budgetary constraints imposed by the Client, and by the availability of access to the site.

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Acknowledgements

Survey data and development proposal layout plans have been provided by Hanson Australia Pty Ltd.

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All enquiries regarding this project are to be directed to the Project Manager.

Contents

1 Introduction

1.1 Background

This report provides the findings of a flood study completed in association with the proposed redevelopment of the Hanson owned Eastern Creek Quarry.

The development proposal includes filling of several current stormwater control ponds, construction of access roads, site regrading, and realignment of several existing access roads.

The flood study provides baseline flood information for pre and post-development site watercourses and is intended to assist with site strategic planning, particularly in relation to stormwater management and on-site effluent disposal.

1.2 Objectives

Primary study objectives are as follows:

- Hydrologic modelling to determine 100 year ARI flow rates.
- Pre- and post-Development hydraulic modelling to determine 100 year ARI flood levels.

2 Existing Site Hydrology

2.1 Existing Watercourses

Two watercourses occur on the site, both draining to Ropes Creek. Watercourse locations and catchment boundaries are provided in Attachment A. Both watercourses are unnamed but are referred to as follows:

1. Upper tributary of Ropes Creek which starts from the catchment south of the site and flows from the property immediately to the south of the site and on to the south west corner of the site.

2. Local stormwater drainage depression which flows from properties to the east, across the eastern edge of the site, into the south eastern corner of the site and into one of the site's sedimentation ponds. Flows form the upper portion of the site also flow into this pond. Ultimately, flows from the stormwater ponds are delivered to the upper Ropes Creek tributary in the south western corner of the site.

Existing watercourses are discussed in further detail below.

2.1.1 Upper Tributary of Ropes Creek

The upper Ropes Creek tributary flows enters the site from the south west where it flows across rural residential allotments through a small grassed depression and two (2) dams (Figure 1). On entering the site it forms a narrow, shallow well-vegetated channel. The channel is characterised as follows:

- Bankfull width of approximately 2 – 3.5 m

- Banks of approximately 500 – 900 mm depth and grades of 1 H in 1 V. Banks are stable showing little to no evidence of recent erosion or instability.

- Dense reeds of 1.0 – 1.5 m height. Bed vegetation is a near monoculture with approximately 95% this single species.

- Banks are vegetated with a mixture of kikuyu, couch grasses and swamp oaks.

- At the time of inspection, surface dampness and some minor shallow (< 20 mm) ponding was visible on the site (attributed to the flow from the Hanson site via the last three detention ponds).

Figure 1: View to southeast from site boundary indicating upper tributary catchment.

Figure 2 indicates pertinent aspects of the channel's structure, vegetation and characteristics.

Figure 2: View to north west at upper Ropes Creek tributary channel.

Confluence of tributary and the drainage depression is at the southwestern corner of the site dam approximately 10 m east of the western boundary property boundary. Downstream of the site, the tributary continues with a similar channel morphology and vegetation.

2.1.2 Local Stormwater Drainage Depression

The stormwater drain is typical of a zero order stormwater drainage system contained within a catchment of extensive industrial and past rural activities (Figure 2). The stormwater drain has been dammed on the adjacent site, as well as on the site. As a result, the drain appears to only infrequently flow, has no evidence of past significant flows such as flood debris and no significant variation in vegetation indicating significant periods of flow (Figure 3). No defined bed or banks occur. Areas of reeds and vegetation observed in the lower reach where it flows into the dam are a consequence of the standing water within the stormwater pond.

Figure 3: View to north east along stormwater drainage depression.

The pond/dam also receives flow from the entire Hanson's site (approximately 23 Ha) via a large sedimentation pond and spillway north west of the pond (Attachment B Plate 1). From here the combined flows are directed into three (3) detention ponds before flowing into the upper tributary of Ropes Creek (Plate 2).

Figure 4: View to west along stormwater drain showing no formed channel.

2.2 Stormwater Ponds

The site currently has approximately 12 sedimentation ponds/dams. All of these are being drained and filled to allow for the regarding as well as the construction of a new sedimentation pond/dam in the lower south west corner of the site. Locations of existing ponds and the proposed new pond are provided in Attachment A.

3 Flood Modelling

3.1 Pre and Post Development Flows

Whilst considerable re-organisation of the development site is proposed, there will be no net change in the extent of hard-stand areas. Many trafficable areas will be consolidated and several stormwater quality management measures (eg. permeable vegetated road side swales) will be implemented. On this basis, there will be no expected change to pre-development flood hydrology on the upper tributary of Ropes Creek.

The re-development involves the redirection and construction of a suitable channel to redirect the existing stormwater drain away from the existing toe embankment. This will allow better stabilisation of the embankment while also disconnecting the stormwater drain from the water quality ponds. The redirected stormwater drain will flow down the eastern boundary and southern boundary and will connect to the existing upper tributary of Ropes Creek in the south west corner of the site (see Attachment A). The drain will need detailed design at CC stage however in summary the proposed new channel will consist of the following:

- 9m channel width from top of banks.
- Minimum 2m channel bed width.
- Minimum channel slope of 1%.
- 1m buffer from boundary to top of left bank.
- Channel banks constructed with rock filled gabions for slopes greater than 2V:1H.
- Channel banks stabilised with planting of appropriate vegetation where appropriate.
- Energy dissipater at outlet prior to joining upper tributary.

3.2 Modelling Approach

The approach to modelling has been to undertake the following:

1. Determine catchment areas.
2. Obtain creek cross-sectional survey information.
3. Undertake hydrological modelling
4. Undertake pre and post-development hydraulic modelling
5. Preparation of a flood extents plan.

3.3 Hydrologic Modelling

Hydrological modelling was undertaken using the Probablistic Rational Method (Australian Rainfall and Runoff, 1987). Time of concentration for the downstream cross-sections was 37.63 minutes (Attachment C). This was used to determine 100 year ARI discharge estimates (Table 1).

Table 1: Cross-section locations, catchment areas and 100 year ARI discharges.

X -Section	Chainage (M)		Catchment Area (Ha)	100 Yr ARI (m^3/s)
	Pre-Development	Post-Development		
1A	574.02	634.17	10.508	3.22
1	543.51	603.45	10.508	3.22
2	443.63	-	11.687	3.52
3	390.63	-	12.26	3.72
1B	-	576.45	10.508	3.22
1C	-	535.98	10.508	3.22
1D	-	497.90	10.508	3.22
1E	-	478.45	10.508	3.22
1F	-	422.70	10.508	3.22
1G	-	315.53	10.508	3.22
1H	-	255.00	22.79	6.99
4	171.48	171.48	54.298	12.95
5	93.20	93.20	56.298	13.43
6	50.29	50.29	58.298	13.91
7	3.28	3.28	60.298	14.38
7A	0	0	60.298	14.38
AA1	181.09	181.09	12.282	3.77
AA	176.09	176.09	12.282	3.77

3.4 Hydraulic Modelling

3.4.1 Cross-sectional Data

Cross sectional data were obtained from the following resources:

- Detailed site survey provided by Lovegrove Oxley Consultants
- 1:25000 scale topographic map data (Prospect 9030-2-N)

The primary Ropes Creek tributary and drainage depression were divided into 11 pre-development cross sections from approximately 40 m north east of the eastern boundary for the drainage depression and extending a further 570 m downstream where Ropes Creek crosses through the western property approximately 150m past the western boundary. Cross sections are located in Attachment A.

Post-development modelling involved the realignment of the stormwater drain along the eastern (flowing north to south) and southern (flowing east to west) boundaries so that it joins the upper tributary of Ropes Creek. The realignment involves the regrading and construction of a suitable open channel to convey the flows. Further detailed design of the channel will be needed at CC stage of the proposal. The new alignment was broken up into 7 post-development cross sections. Cross sections are located in Attachment A.

3.4.2 Hydraulic Modelling

Hydraulic modelling was conducted using the hydraulic analysis computer package HEC-RAS Ver3.1.3 to estimate flow extents, flow heights and velocity distribution at pre-determined cross–sections along the pre and post-development channel. The model (based on Bernulli's energy equation) provides sufficient detail to determine the velocity and shear strength of the flow at all relevant channel cross-sections.

3.4.3 Results

Detailed hydraulic modelling data and results are provided in Attachment D with a summary of findings provided in Table 2. The 100 year Event has been plotted on the site plan provided in Attachment A.

Table 2: Summary of Pre and Post-Development HECRAS Modelling.

X -Section	Pre-Development 100 year ARI level (mAHD)	Post-Development 100 year ARI level (mAHD)
1A	70.34	70.06
1	68.76	69.65
2	65.21	-
3	63.95	-
1B	-	68.66
1C	-	68.50
1D	-	67.51
1E	-	67.35
1F	-	66.62
1G	-	64.20
1H	-	61.86
4	60.14	60.14
5	58.76	58.76
6	58.20	58.20
7	57.53	57.53
7A	57.09	57.09

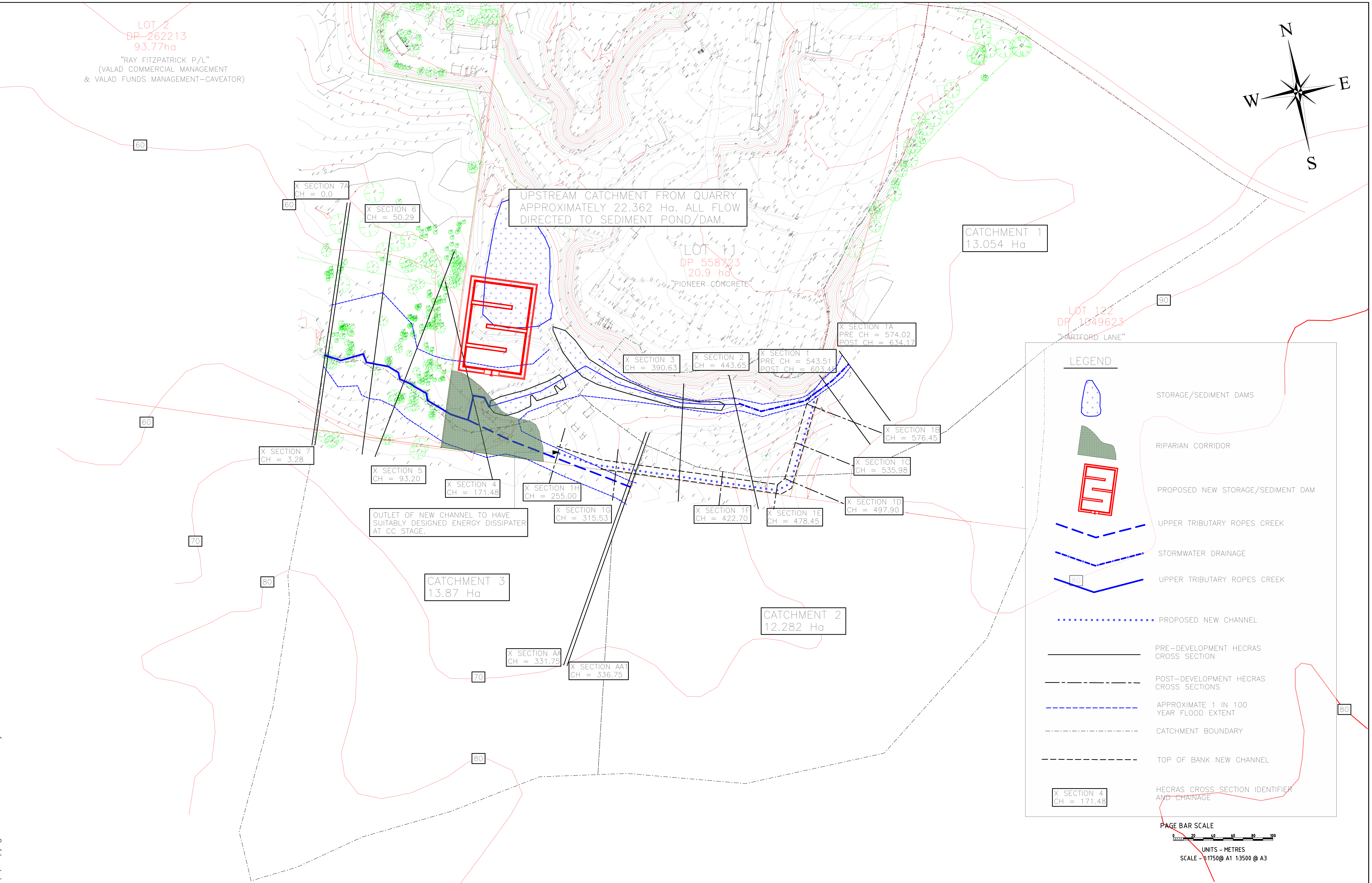
4 References

Bannerman S.M and Hazelton P.A (1990) *Soil Landscapes of the Penrith 1:100,000 Sheet*

Land and Property Information (2001) *NSW Topographic Maps - Prospect 9030-2-N*

The Institution of Engineers, Australia (1987) *Australian Rainfall and Runoff – A Guide tom Flood Estimation Volume 1 and 2*

5 Attachment A – Catchment and Site Plan



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CLIENT/ PROJECT: HANSON CONSTRUCTION MATERIALS EASTERN CREEK QUARRY

TITLE: FLOOD ASSESSMENT - CATCHMENT AND HECRAS SITE PLAN

THIS PLAN MUST NOT BE USED FOR CONSTRUCTION UNLESS SIGNED AS APPROVED BY PRINCIPAL CERTIFYING AUTHORITY
All measurements in m unless otherwise specified.

DESIGNED: GT	DATUM: AHD	SHEET 1 OF 1 SHEETS
DRAWN: GT	HORIZONTAL RATIO: 1:1750 @ A1, 1:3500 @ A3	
REVIEWED: DDM	VERTICAL RATIO: 1:1750 @ A1, 1:3500 @ A3	PAPER SIZE: A1 / A3

PROJECT MANAGER: DR DANIEL MARTENS

DRAWING NUMBER: P0601396JD07_V1

REV.	DESCRIPTION	DATE	ISSUED
1.0	FLOOD ASSESSMENT	25.09.2006	GT
2.0	FLOOD ASSESSMENT- FINAL REPORT	04.10.2006	GT

6 Attachment B - Plates

Plate 1: View to north west from spillway showing large sedimentation pond.

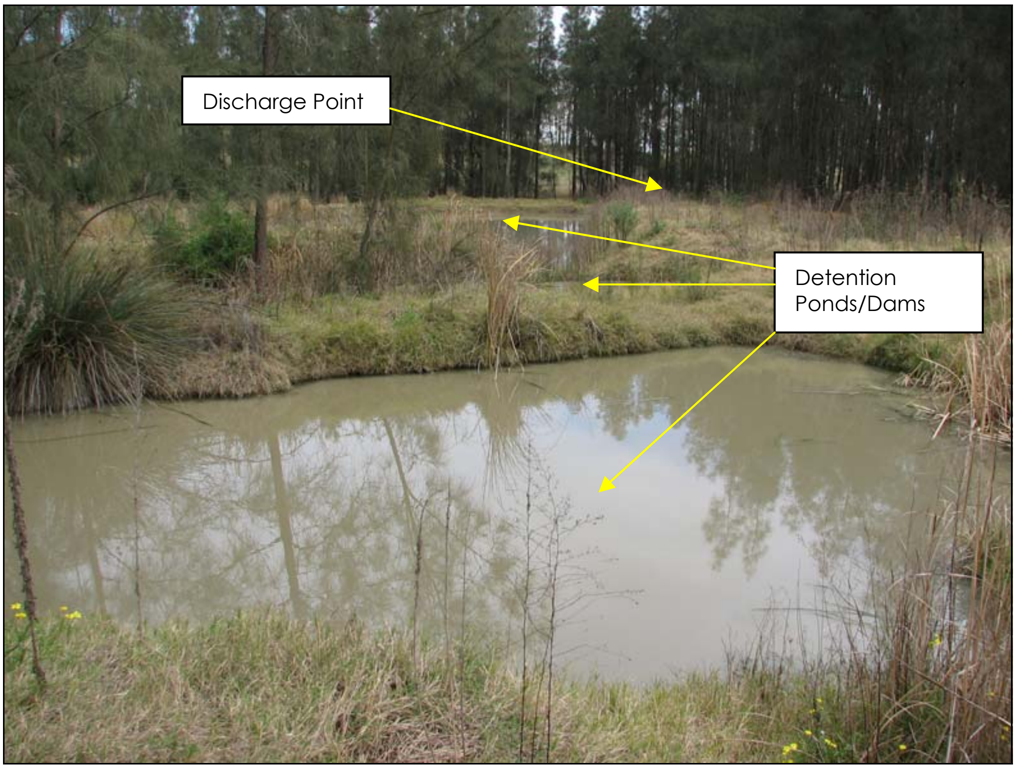

Plate 2: View to south west from minor pond showing the 3 detention ponds.

Plate 3 View to south east from spillway looking at minor pond/dam showing the typical vegetation of the pond/dam.

Plate 4: View to north east from retention ponds looking at first of the three retention ponds showing the typical vegetation of the pond/dam.

7 Attachment C – Hydrologic Modelling

Flow Rate Using Probablistic Rational Method

Method based on Australian Rainfall and Runoff Volumes 1 & 2 (1987)

Method T13 Revised 15.7.2005

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PROJECT

Project	**Eastern Creek Quarry**			Ref. No.	**P0601396JS10-V1 Rational Method**
Author	**Dr Daniel Martens**	Reviewed	**Mr G. Harlow, Mr M. Dumas**	Date Created	

STEP 1 : CATCHMENT DATA

	Enter Data	Unit
Catchment Area	**60.2980**	ha
Enter zone (A,B,C,D,E,F)	**B**	
Above 500 m (Y/N)	**N**	
Time of Concentration	37.63	minutes

Catchment is near to the following zone boundaries **Zone 1** **Zone 2** **Zone 3** **Zone 4**

Enter Zones in Alphabetical Order

STEP 2: RAINFALL DATA

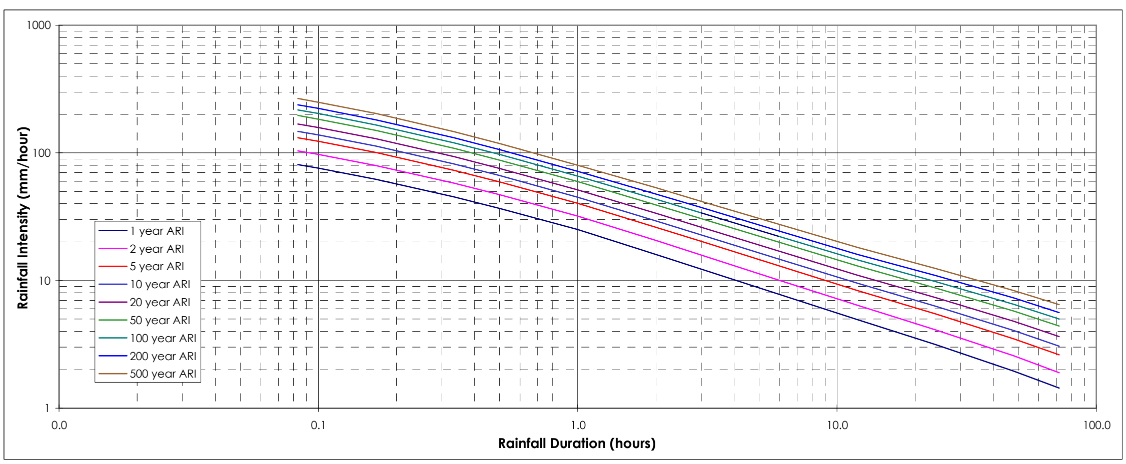
Known Rainfall Intensity	Recurrence	Enter mm/hr
map 1 rainfall intensity	2yr 1hr ($^{2}i_{1}$)	**32**
map 2 rainfall intensity	2yr 12 hr ($^{2}i_{12}$)	**6.4**
map 3 rainfall intensity	2yr 72hr ($^{2}i_{72}$)	**1.9**
map 4 rainfall intensity	50yr 1hr ($^{50}i_{1}$)	**59.5**
map 5 rainfall intensity	50yr 12hr ($^{50}i_{12}$)	**13**
map 6 rainfall intensity	50yr 72hr ($^{50}i_{72}$)	**4.4**
map 7 skewness	G (skewness)	**0.02**
map 8	F2	**4.29**
map 9	F50	**15.85**
figure 5	C_{10}	**0.7**

ARI	^{Y}K (using G, see Table 10.2, p208)
2	**0.000**
5	**0.842**
10	**1.282**
20	**1.645**
50	**2.054**
100	**2.326**
200	**2.576**
500	**2.878**

Design Rainfall Intensities

		Return Period (years)								
Duration	**Duration (hrs)**	**1**	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
5 mins	0.083	80.8	103.5	131.5	147.5	168.8	196.4	217.3	238.4	266.7
6 mins	0.10	75.8	97.1	123.3	138.2	158.1	184.0	203.5	223.3	249.7
10 mins	0.17	62.0	79.4	100.7	112.8	129.0	150.0	165.8	181.9	203.3
20 mins	0.33	45.2	57.8	73.1	81.8	93.5	108.6	120.0	131.5	146.9
30 mins	0.50	36.7	47.0	59.3	66.4	75.7	87.9	97.1	106.4	118.8
1 hr	1	25.1	32.0	40.3	45.0	51.3	59.5	65.7	71.9	80.2
2 hr	2	16.1	20.6	26.2	29.4	33.7	39.2	43.4	47.7	53.4
3 hr	3	12.3	15.8	20.2	22.8	26.2	30.6	33.9	37.3	41.8
6 hr	6	7.8	10.0	13.0	14.7	17.0	19.9	22.1	24.4	27.5
12 hr	12	4.9	6.4	8.4	9.5	11.0	13.0	14.5	16.1	18.1
24 hr	24	3.1	4.1	5.5	6.3	7.3	8.8	9.8	10.9	12.5
48 hr	48	2.0	2.6	3.5	4.1	4.8	5.8	6.5	7.3	8.4
72 hr	72	1.4	1.9	2.6	3.1	3.6	4.4	5.0	5.6	6.5

		1	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Design	0.627	**33.8**	**43.2**	**54.5**	**60.9**	**69.5**	**80.7**	**89.1**	**97.6**	**108.9**

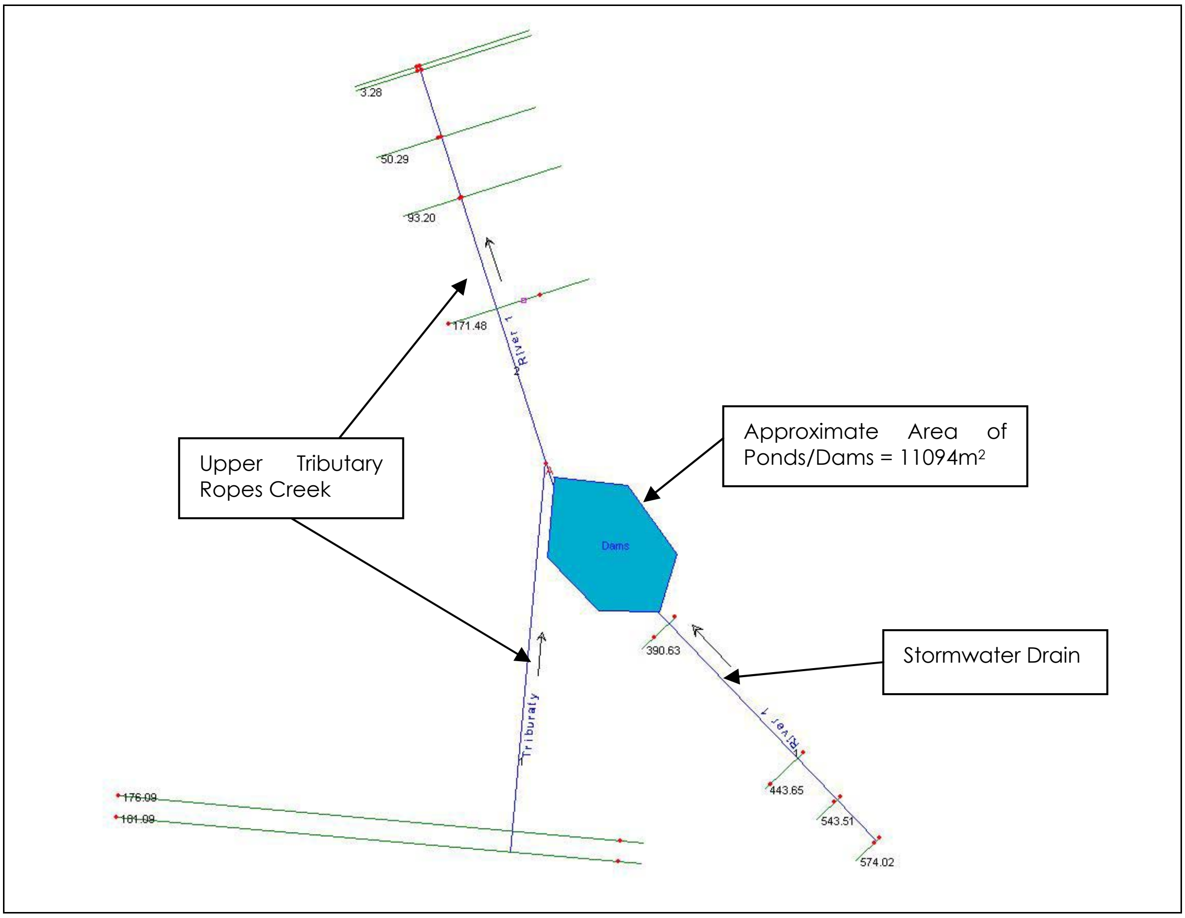


STEP 3: FLOW RATE CALCULATIONS

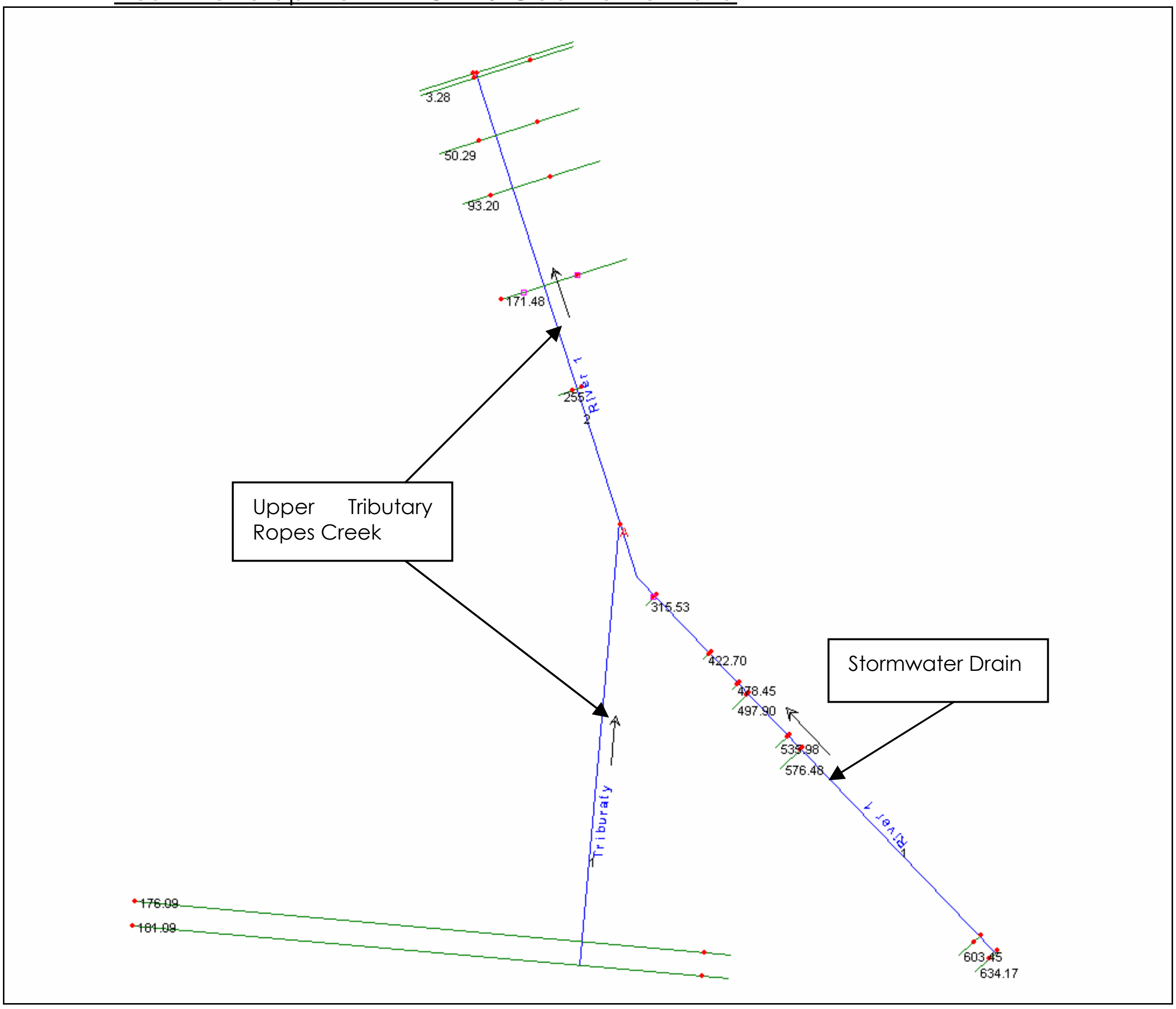
	Return Period (years)								
Results	**1**	**2**	**5**	**10**	**20**	**50**	**100**	**200**	**500**
Frequency Factor (FF_Y)	0.62	0.74	0.88	1.00	1.12	1.25	1.38	1.51	1.65
Runoff Coefficient C	0.43	0.52	0.62	0.70	0.78	0.87	0.96	1.05	1.15
Rainfall Intensity (mm/hr)	33.8	43.2	54.5	60.9	69.5	80.7	89.1	97.6	108.9
Q (m^3/s)	2.457	3.748	5.628	7.149	9.139	11.805	14.383	17.251	21.049
Q (L/s)	2456.6	3748.4	5628.1	7149.3	9138.7	11804.8	14383.2	17250.5	21049.3

8 Attachment D – HECRAS Modelling Results

Pre- Development HECRAS Geometric Data



Post- Development HECRAS Geometric Data



Pre – Development HECRAS RESULTS

HEC-RAS Plan: Plan 06 Profile: PF 1

River	Reach	River Station	X Section	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Shear Total (N/m2)	Sta W.S. Lft (m)	Sta W.S. Rgt (m)
Stormwater Drain	1	574.02	2	3.22	69.75	70.39	70.06	70.4	0.018581	0.5	6.41	15.42	0.25	75.01	66.44	81.86
Stormwater Drain	1	543.51	3	3.22	68.45	68.76	68.76	68.87	0.380242	1.46	2.21	10.45	1.01	785.51	70.69	81.13
Stormwater Drain	1	443.65	4	3.58	64	65.21		65.22	0.003275	0.3	11.95	16.43	0.11	22.38	98.78	115.21
Stormwater Drain	1	390.63	5	3.76	63.7	63.95	63.95	64.07	0.3661	1.53	2.45	10.35	1.01	841.68	93.83	104.18
Tributary Ropes Creek	2	171.48	6	12.95	59.25	60.14	59.65	60.14	0.007365	0.32	40.87	99.11	0.16	29.77	22.32	121.43
Tributary Ropes Creek	2	93.2	7	13.43	57.5	58.76		58.78	0.074445	0.65	20.64	62.25	0.36	240.45	54.59	116.85
Tributary Ropes Creek	2	50.29	7A	13.91	56.85	58.2		58.2	0.005474	0.26	55.38	107.57	0.11	27.54	56.76	164.33
Tributary Ropes Creek	2	3.28	AA1	14.38	56.7	57.53		57.55	0.074476	0.69	21.38	66.03	0.37	235.81	76.47	142.5
Tributary Ropes Creek	2	0	AA	14.38	55.75	57.09	57.09	57.19	0.172723	1.74	12.64	56.44	0.58	373.43	81.08	137.52

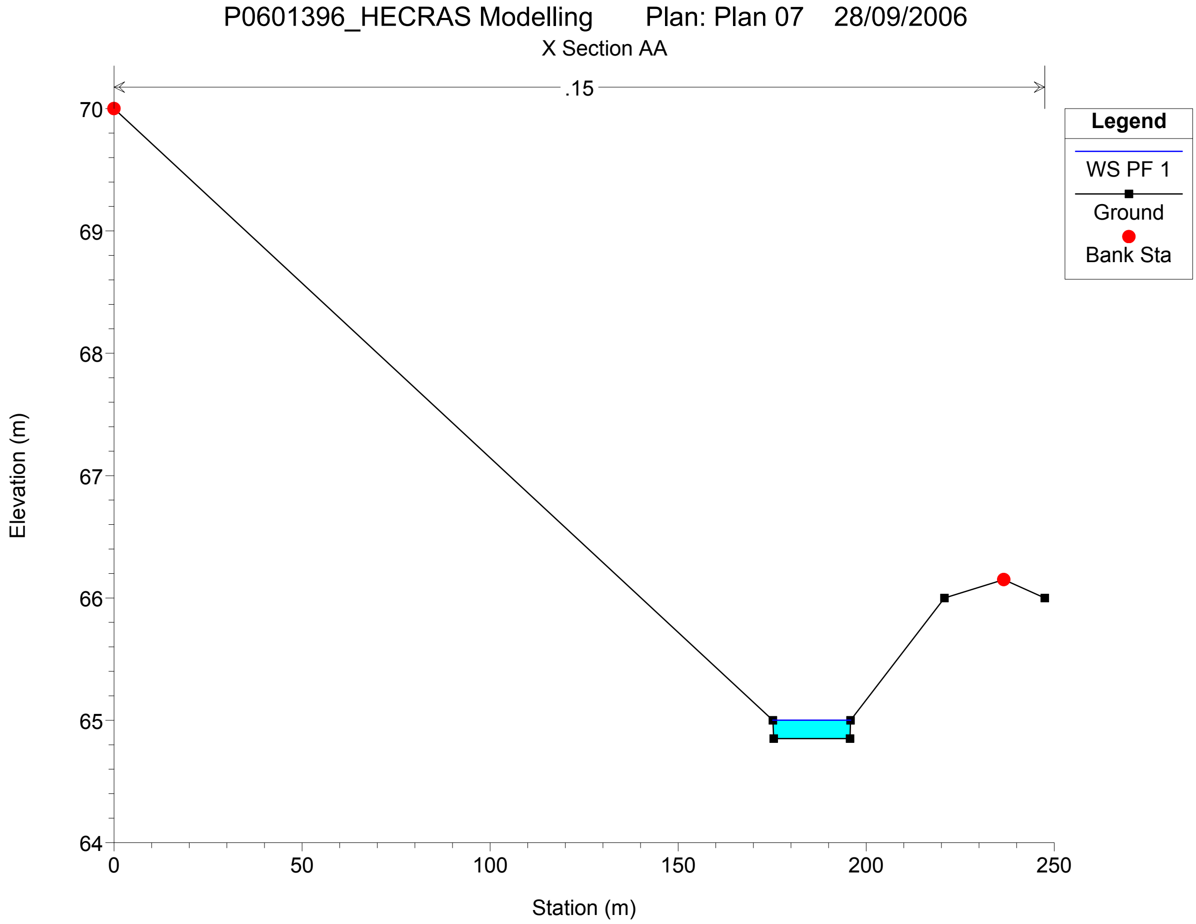
Post – Development HECRAS RESULTS

HEC-RAS Plan: Plan 06 Profile: PF 1

River	Reach	River Station	X Section	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl	Shear Total (N/m2)	Sta W.S. Lft (m)	Sta W.S. Rgt (m)
Stormwater Drain	1	634.17	1A	3.22	69.75	70.06	70.06	70.17	0.380191	1.46	2.21	10.45	1.01	785.4	70.69	81.13
Stormwater Drain	1	603.45	1	3.22	68.45	69.65		69.65	0.001168	0.19	17.33	23.58	0.07	8.3	59.1	82.68
Stormwater Drain	1	576.48	1B	3.22	67.85	68.66		68.7	0.011629	0.84	3.82	7.37	0.37	54.82	74.02	81.39
Stormwater Drain	1	535.98	1C	3.22	67.27	68.5		68.54	0.004256	0.87	3.7	4	0.29	29.54	43.8	47.8
Stormwater Drain	1	497.9	1D	3.22	66.9	67.51	67.5	67.77	0.037788	2.24	1.44	2.69	0.98	156.37	58.75	61.44
Stormwater Drain	1	478.45	1E	3.22	66.65	67.35	67.25	67.53	0.005308	1.87	1.72	2.92	0.78	24.29	18.65	21.57
Stormwater Drain	1	422.7	1F	3.22	66.05	66.62	66.62	66.86	0.04694	2.15	1.5	3.22	1.01	185	21	24.22
Stormwater Drain	1	315.53	1G	3.22	63	64.2	63.44	64.21	0.001149	0.5	6.45	7.79	0.18	8.71	30.43	38.21
Tributary Ropes Creek	2	255	1H	6.99	61.5	61.86	61.86	62	1.046883	1.62	4.38	17.65	0.94	2541.72	20.03	37.68
Tributary Ropes Creek	2	171.48	4	12.95	59.25	60.14	59.65	60.14	0.007365	0.32	40.87	99.11	0.16	29.77	22.32	121.43
Tributary Ropes Creek	2	93.2	5	13.43	57.5	58.76		58.78	0.074445	0.65	20.64	62.25	0.36	240.45	54.59	116.85
Tributary Ropes Creek	2	50.29	6	13.91	56.85	58.2		58.2	0.005474	0.26	55.38	107.57	0.11	27.54	56.76	164.33
Tributary Ropes Creek	2	3.28	7	14.38	56.7	57.53		57.55	0.074476	0.69	21.38	66.03	0.37	235.81	76.47	142.5
Tributary Ropes Creek	2	0	7A	14.38	55.75	57.09	57.09	57.19	0.172723	1.74	12.64	56.44	0.58	373.43	81.08	137.52

9 Attachment E – HECRAS Plots

Pre – Development HECRAS Plots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Station (m)
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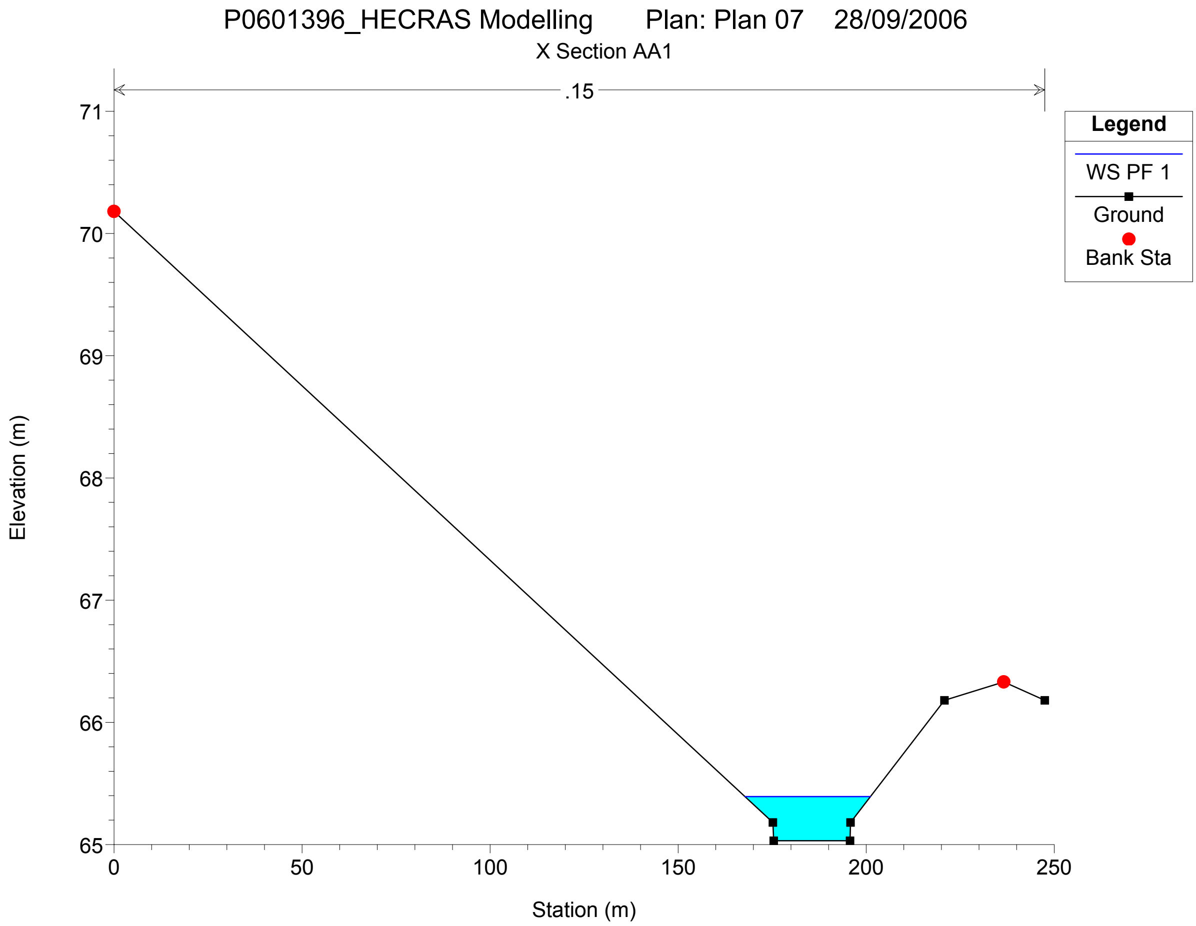
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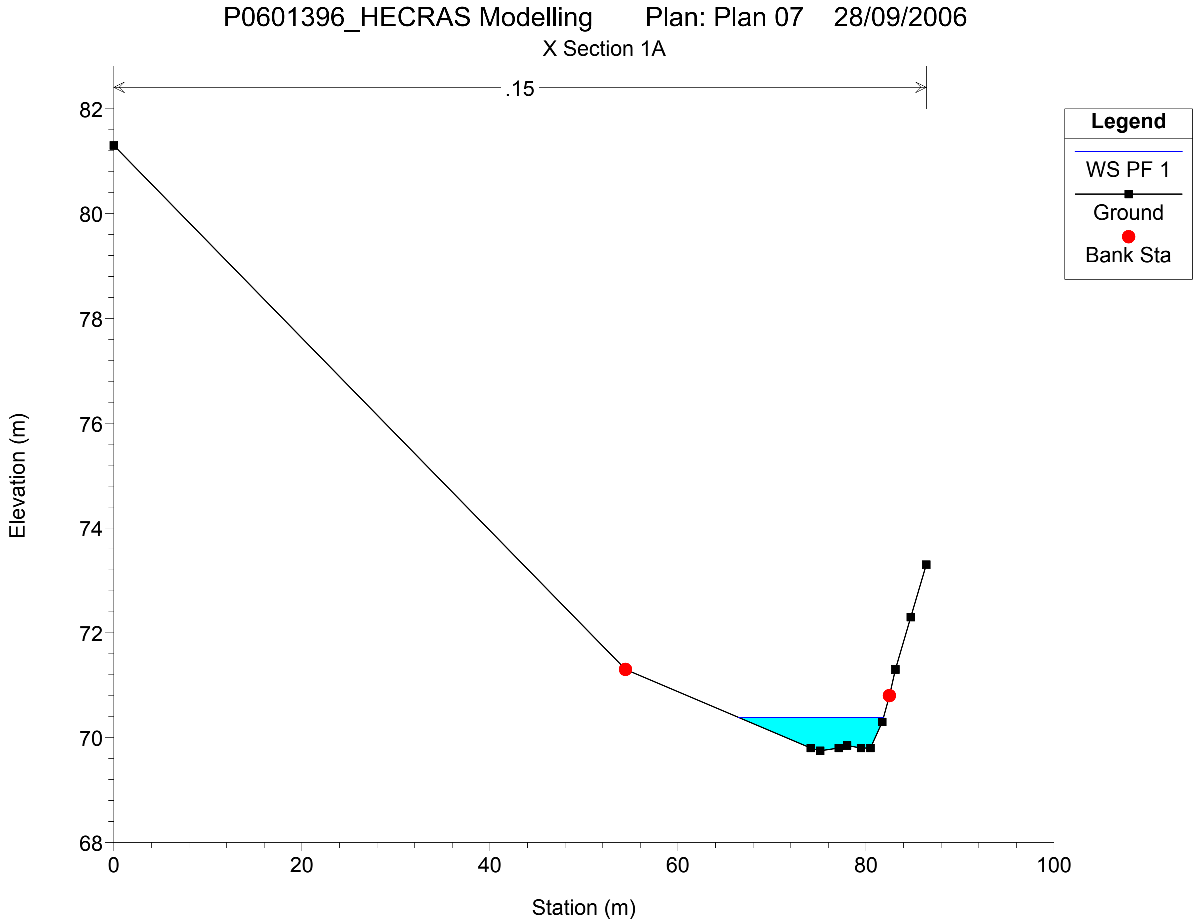
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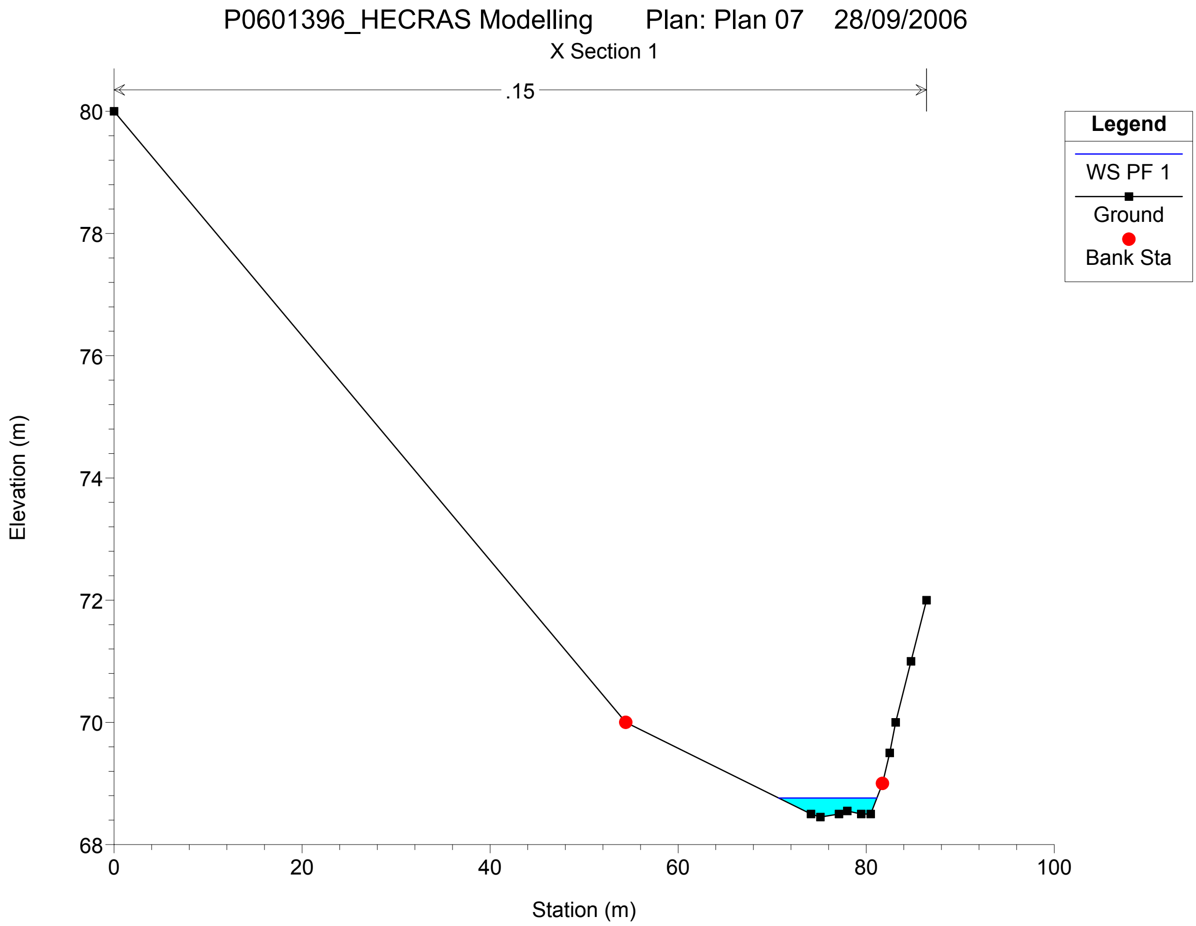
40
60
80
100
Station (m)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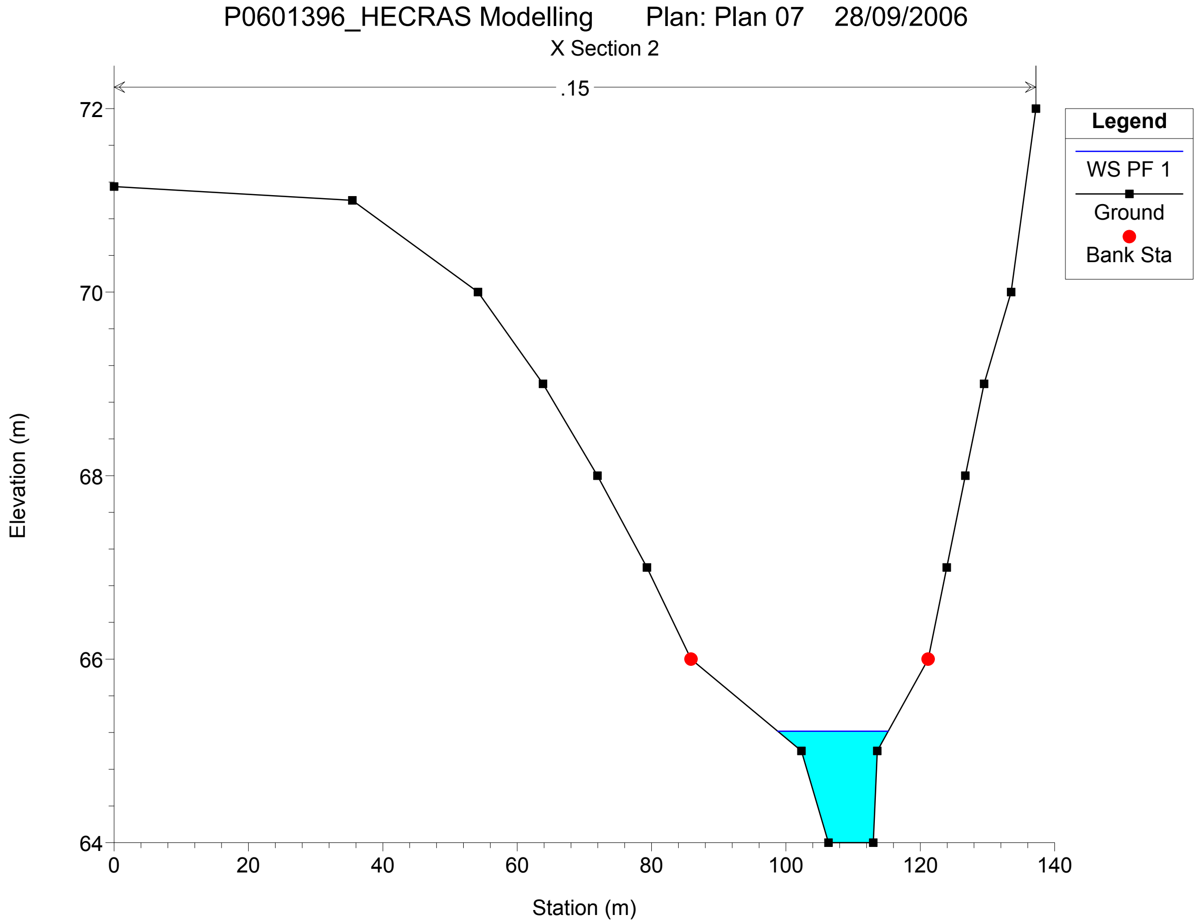

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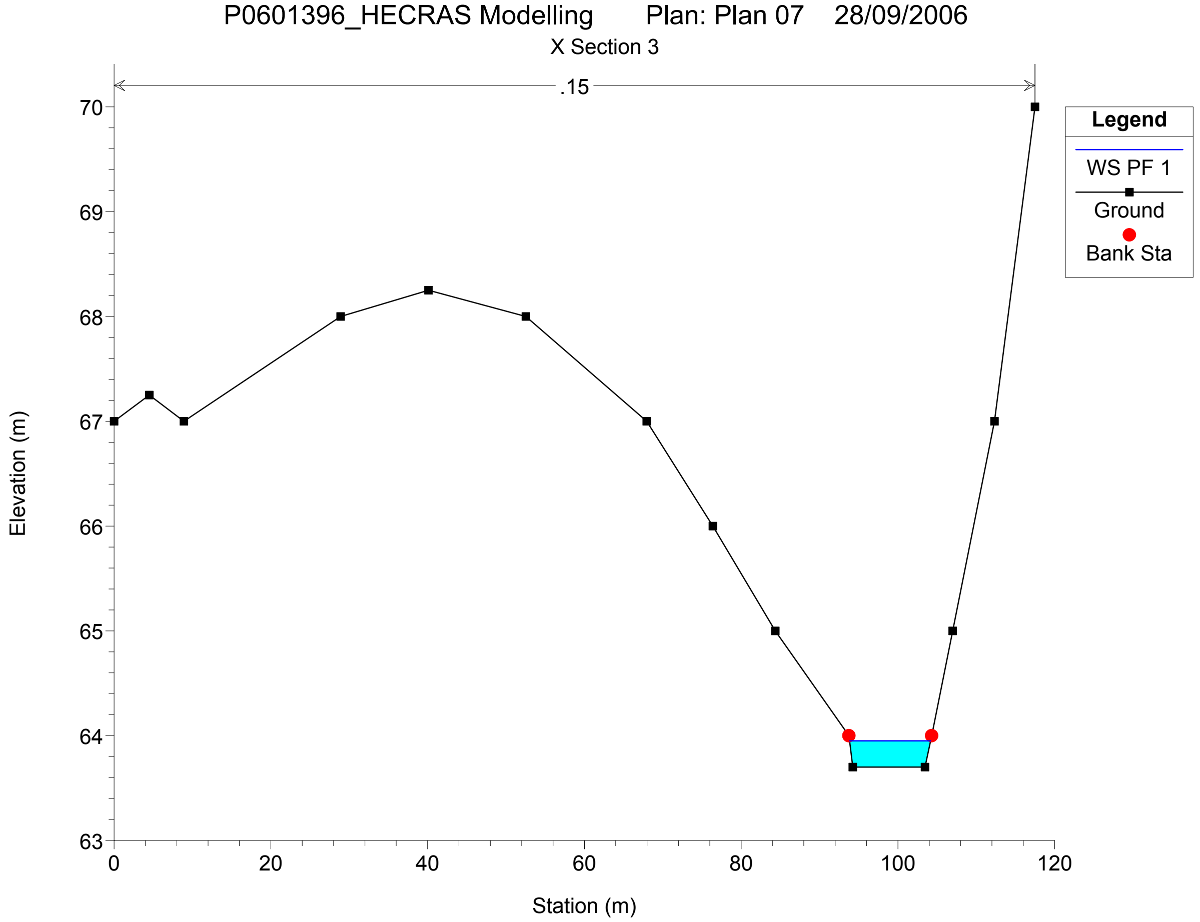
120
140
Station (m)



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100
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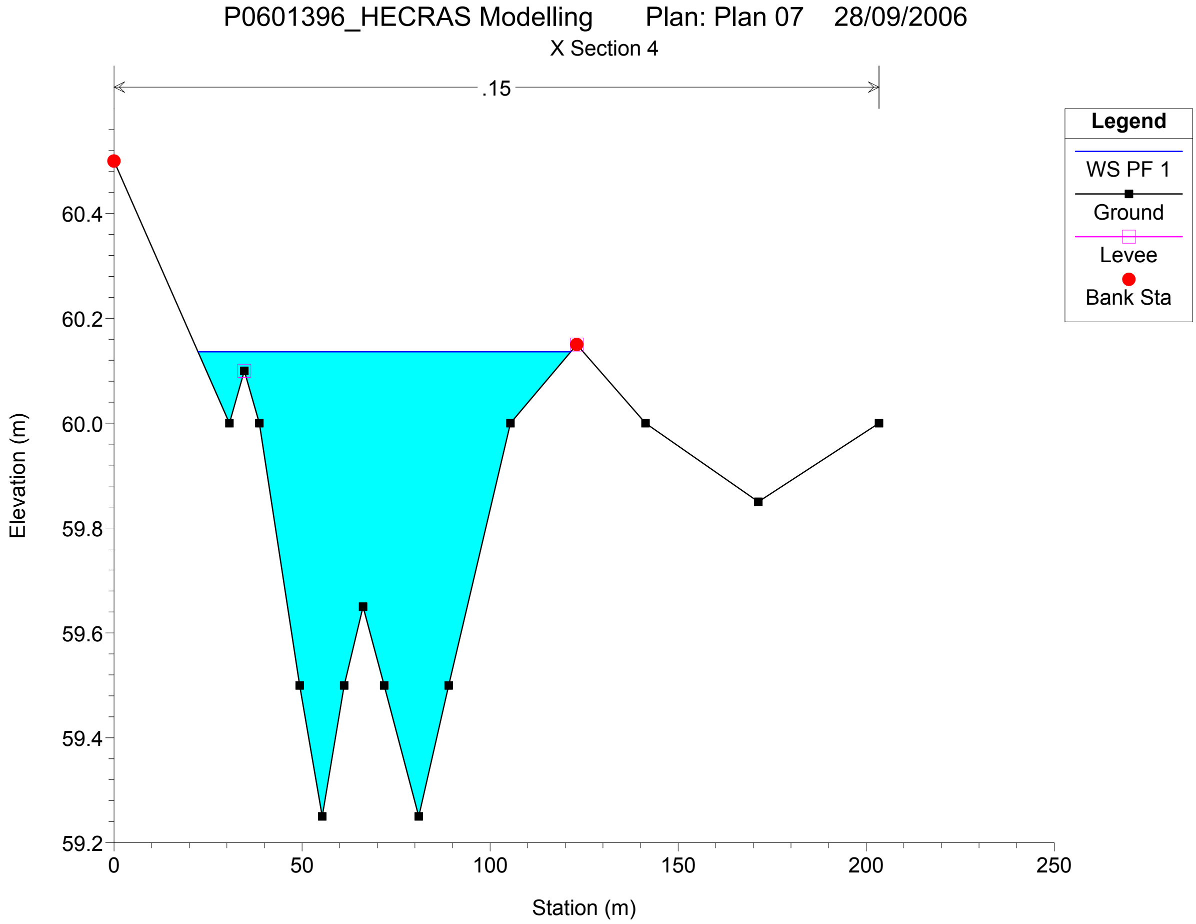
Station (m)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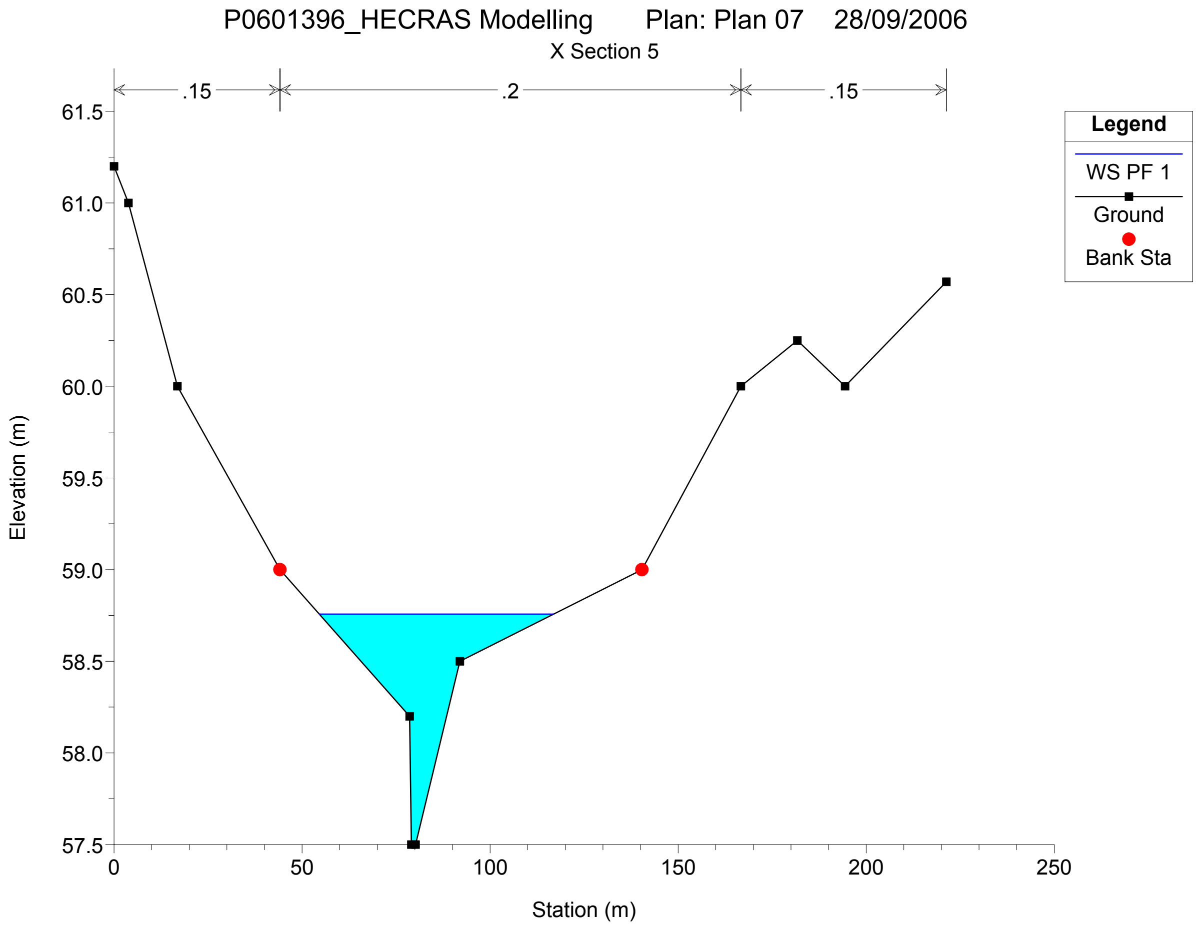
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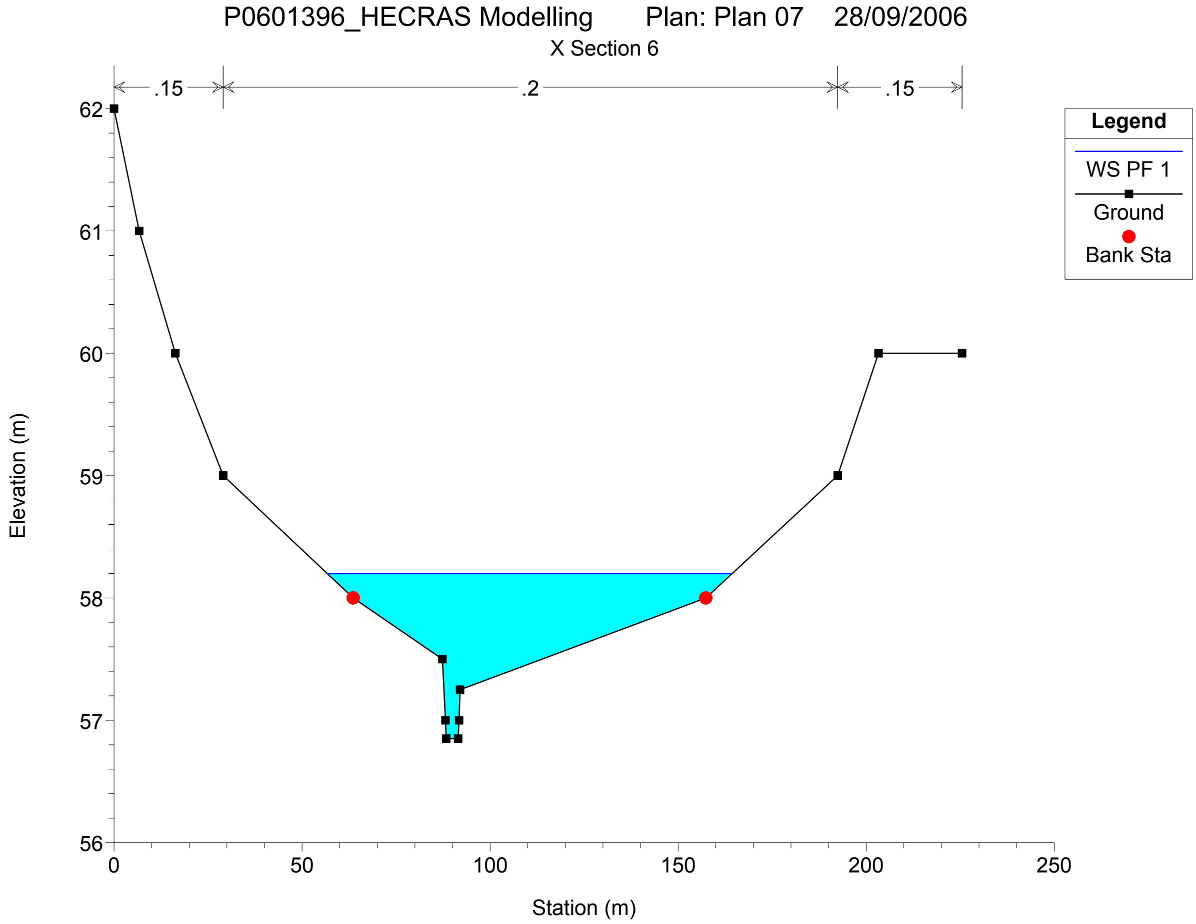

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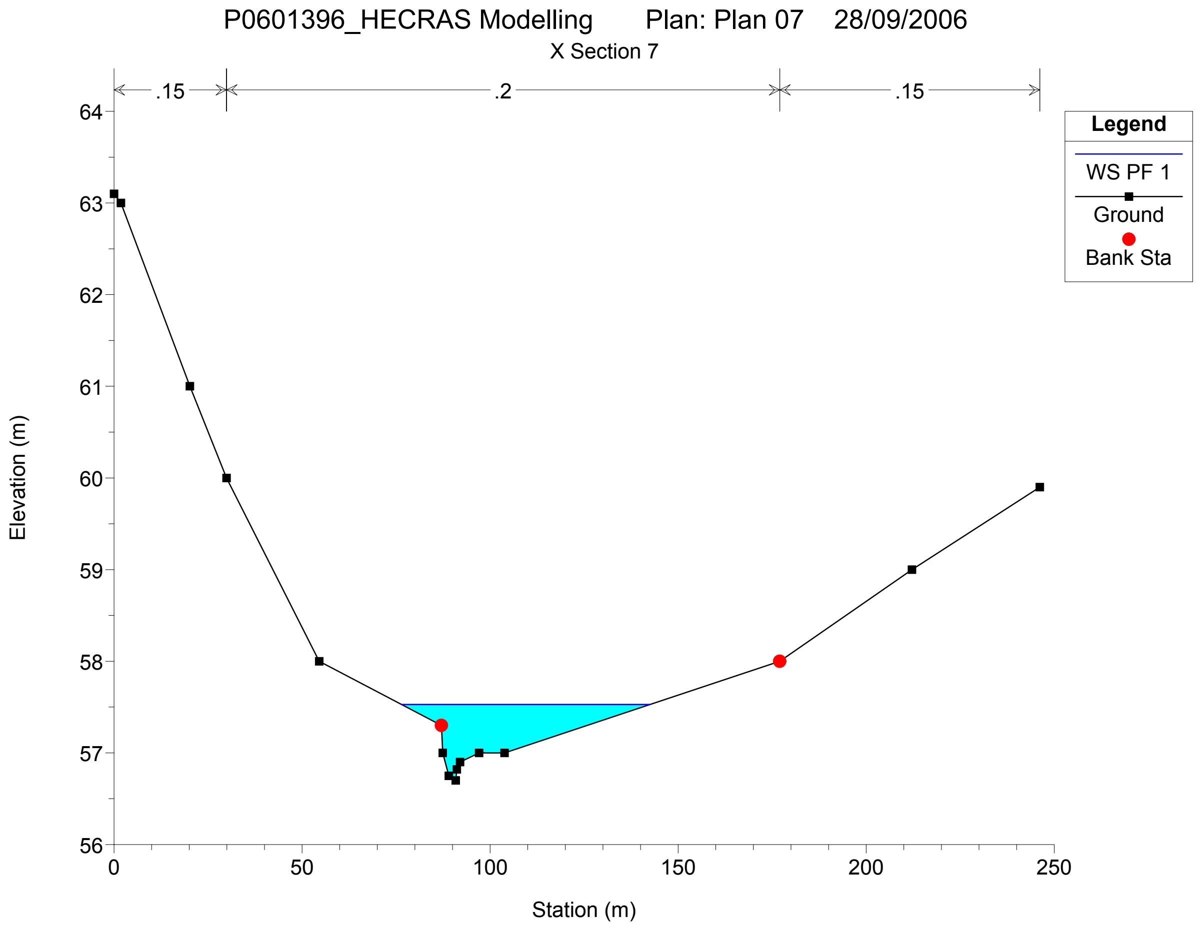
Station (m)



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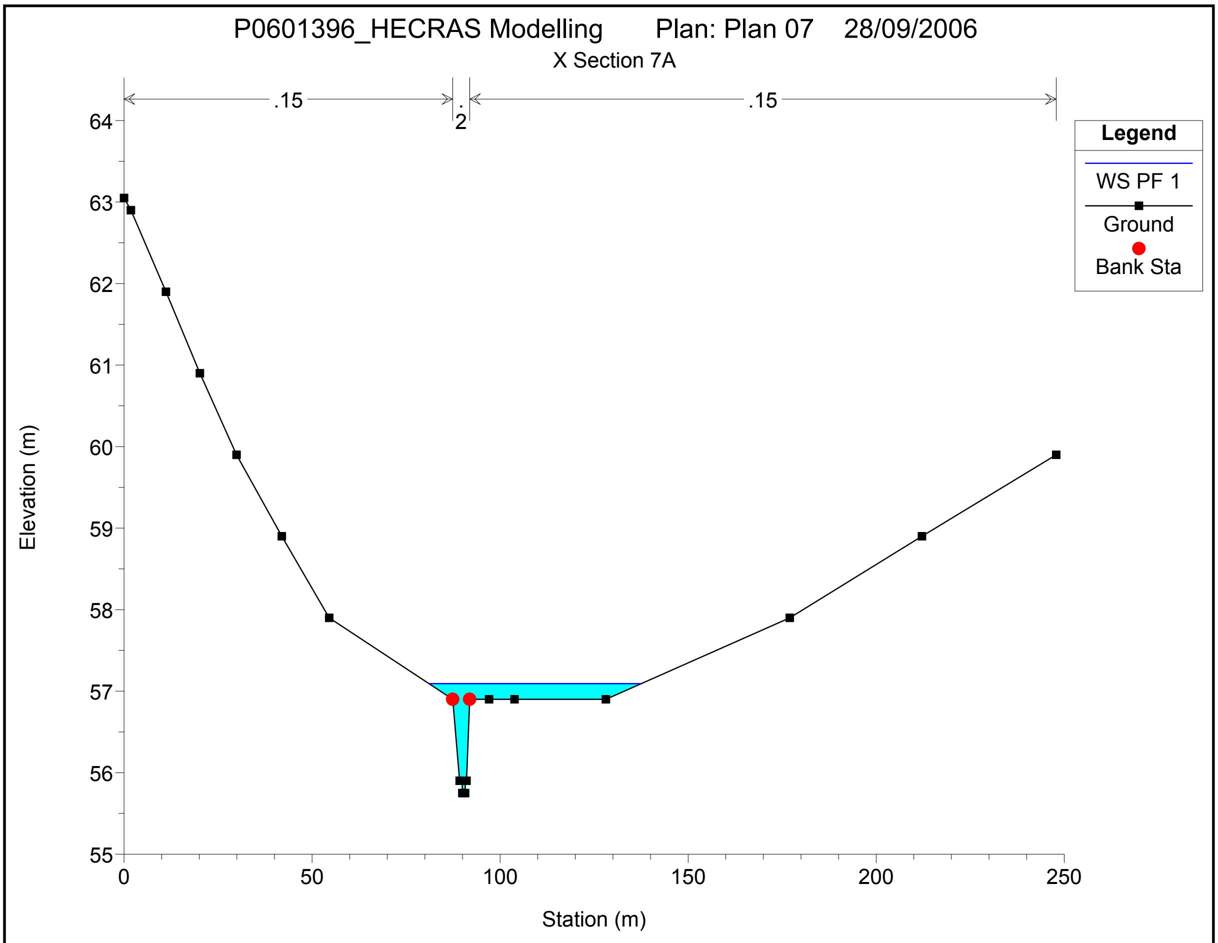
Station (m)



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200
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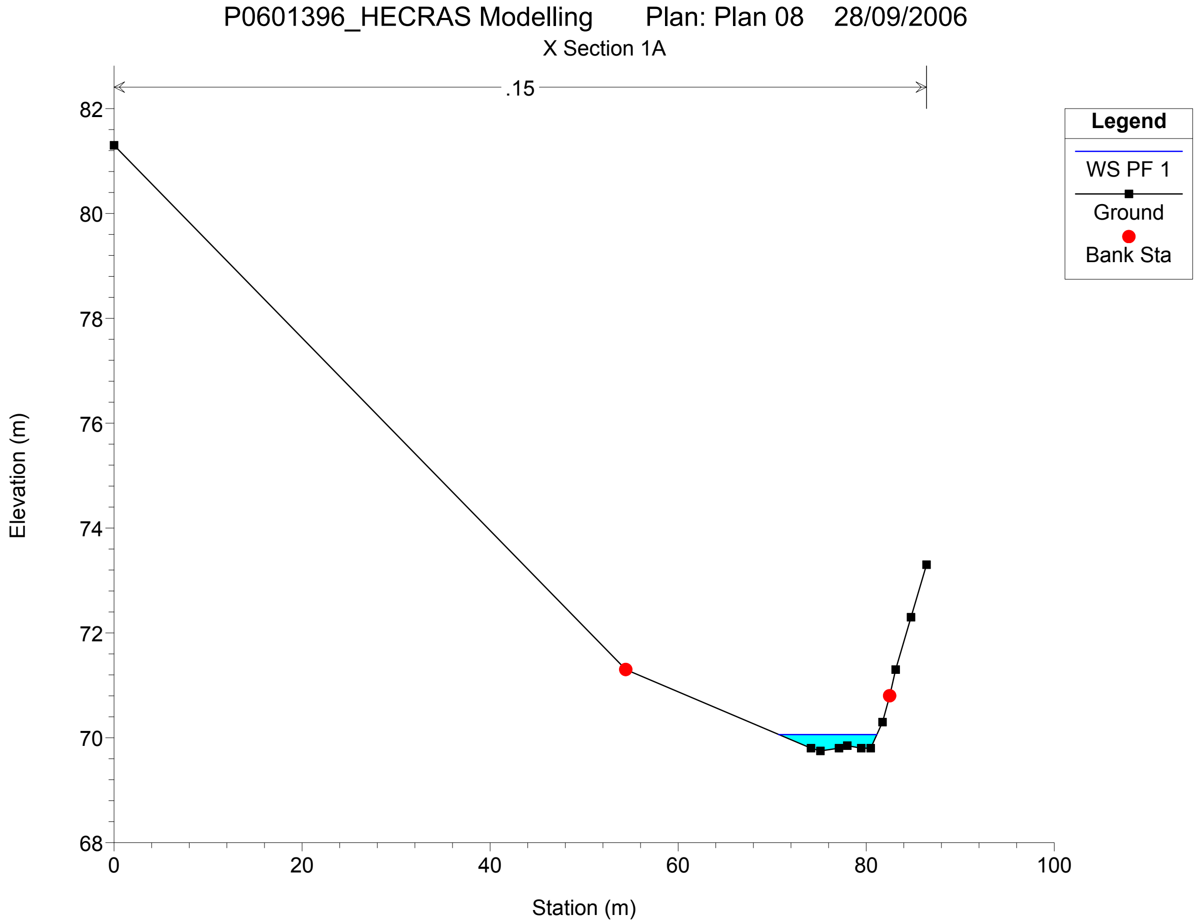
Station (m)

Post – Development HECRAS Plots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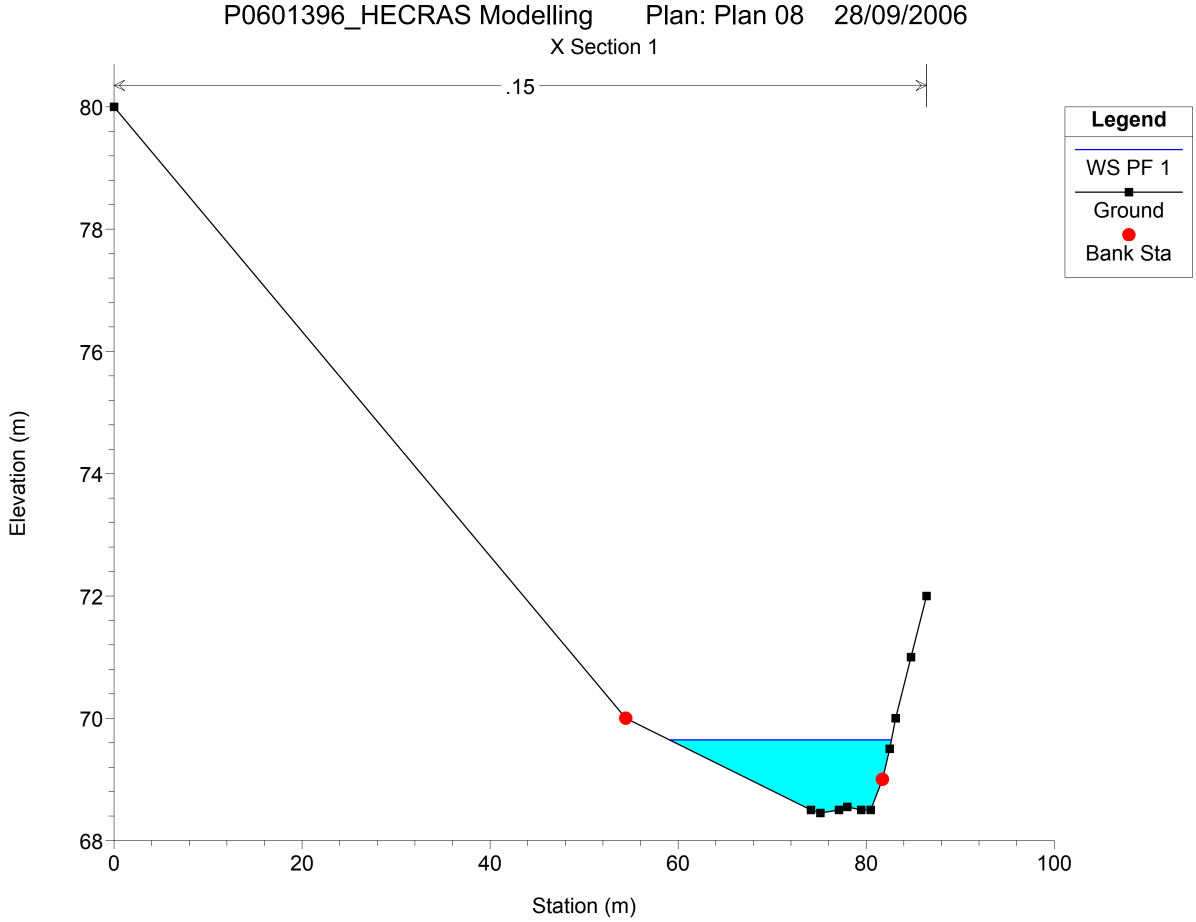
Station (m)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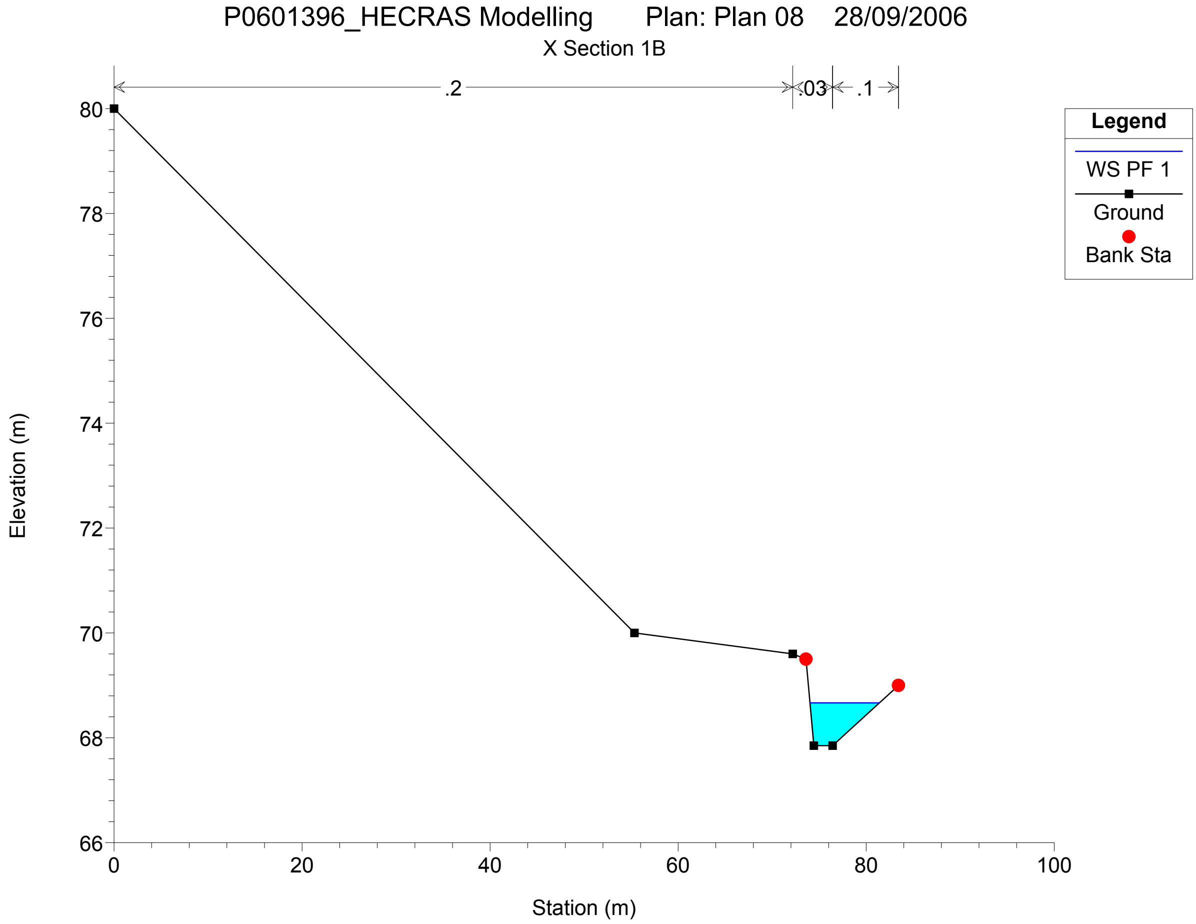
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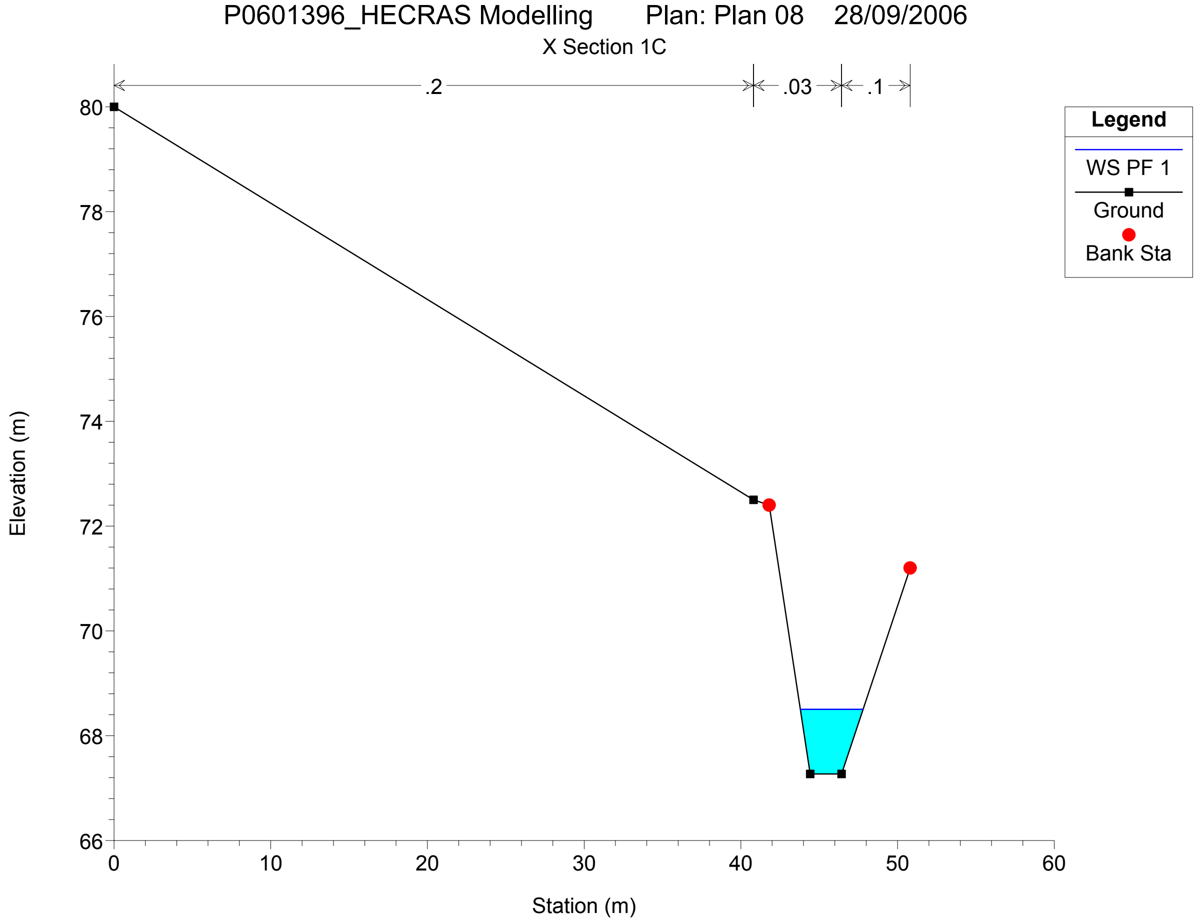
Station (m)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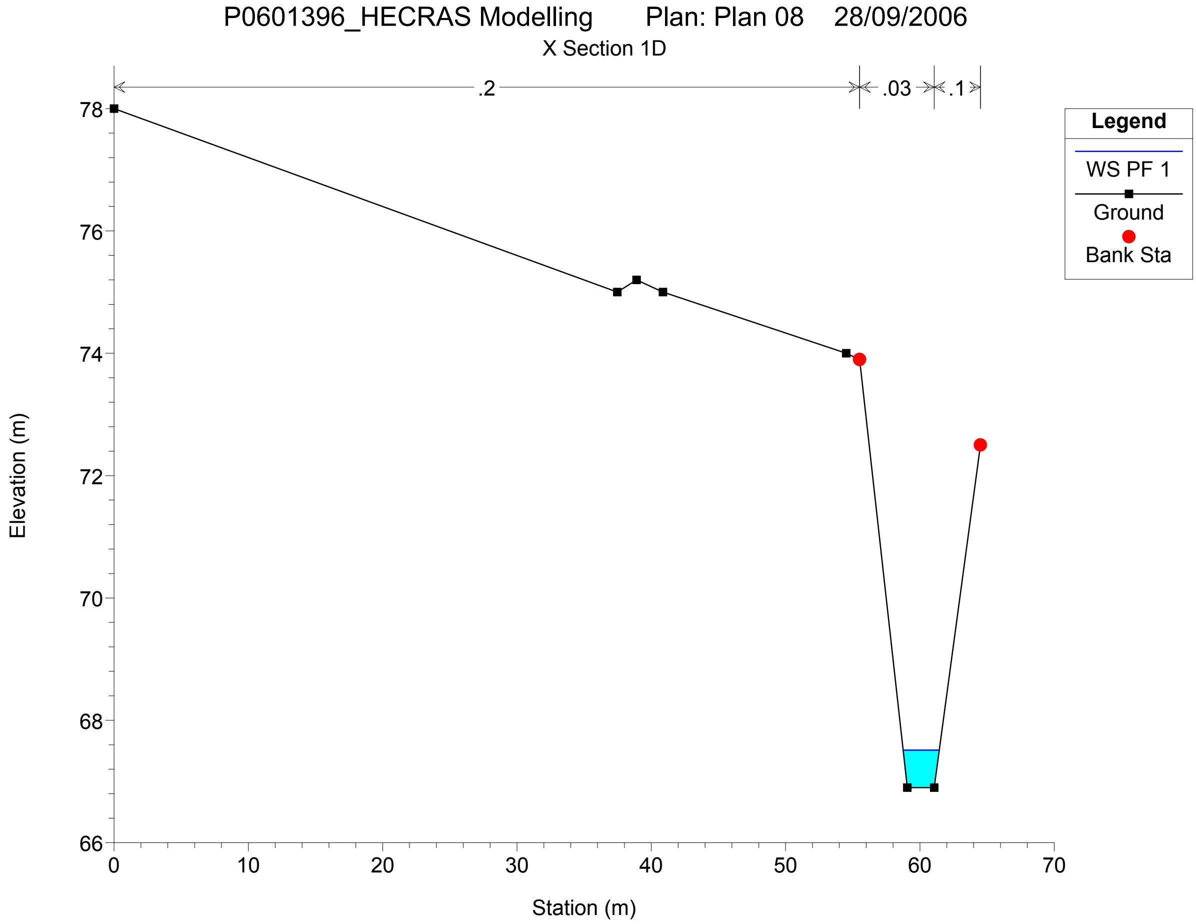
Station (m)



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Legend
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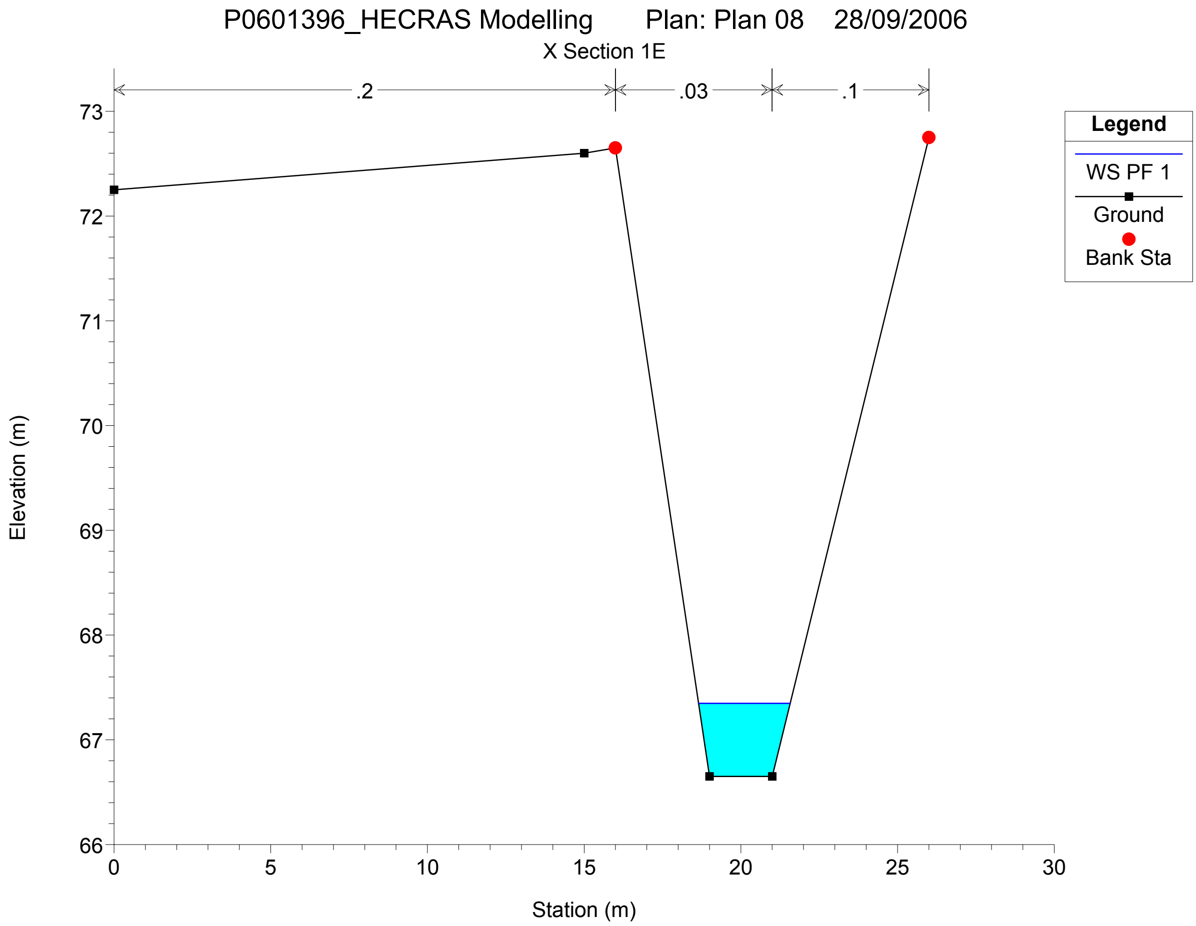
Station (m)



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Legend
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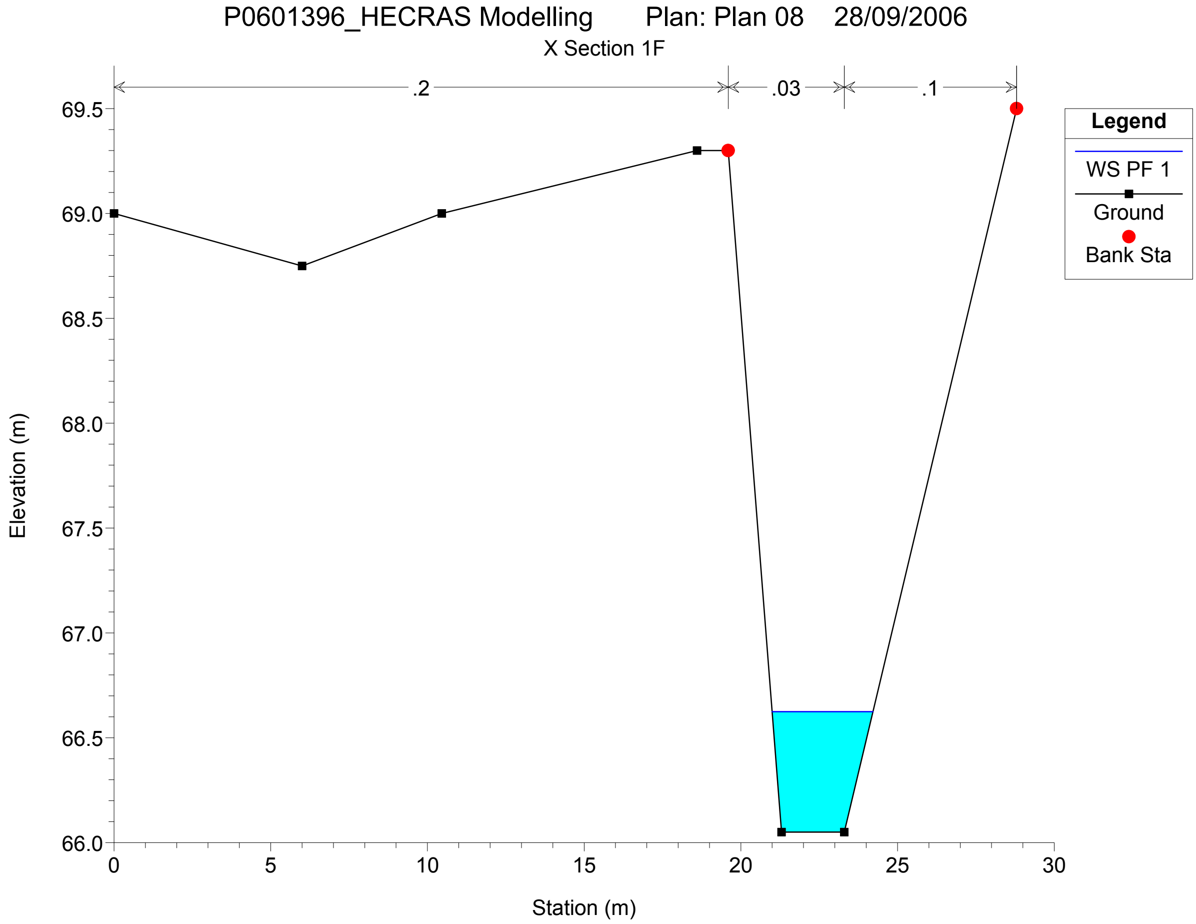
30
Station (m)



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Bank Sta
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10
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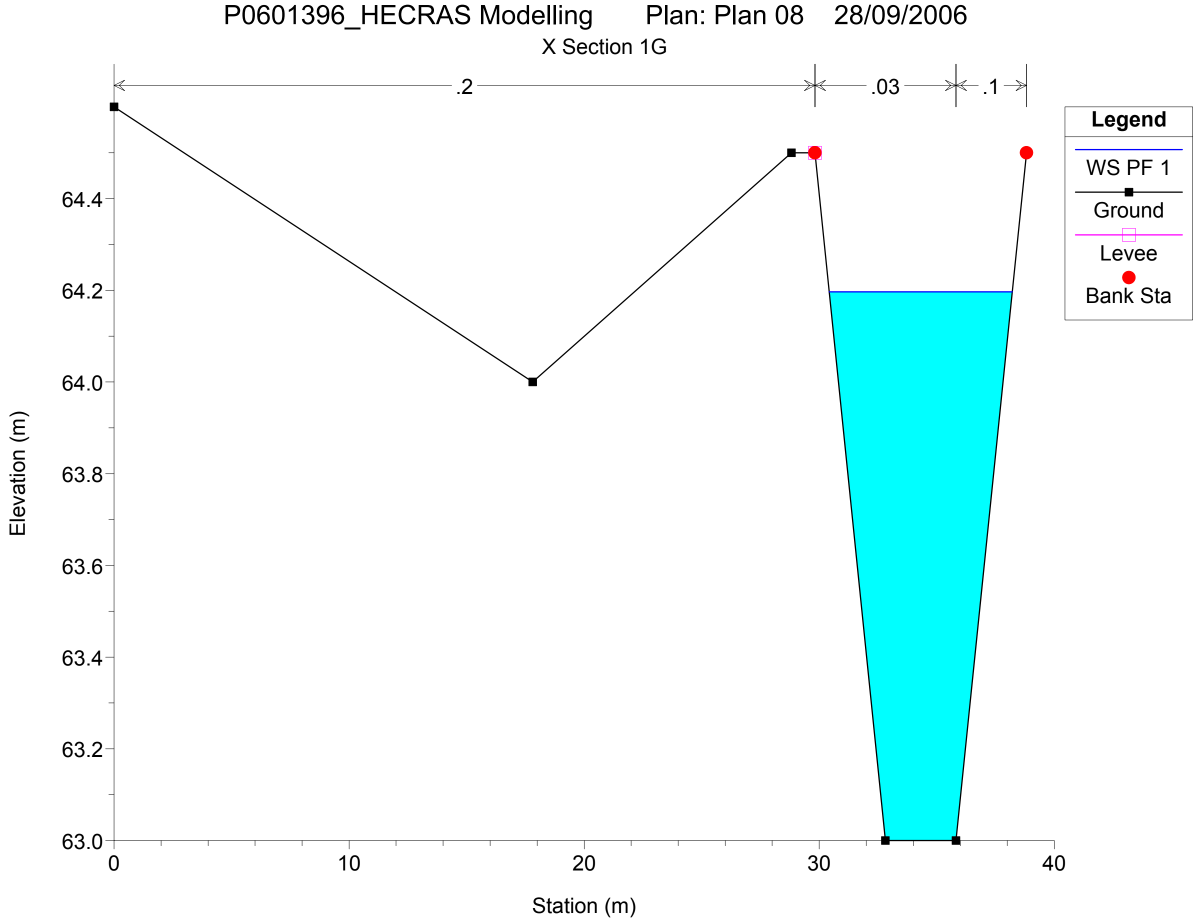
20
25
30
Station (m)



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Levee
Bank Sta
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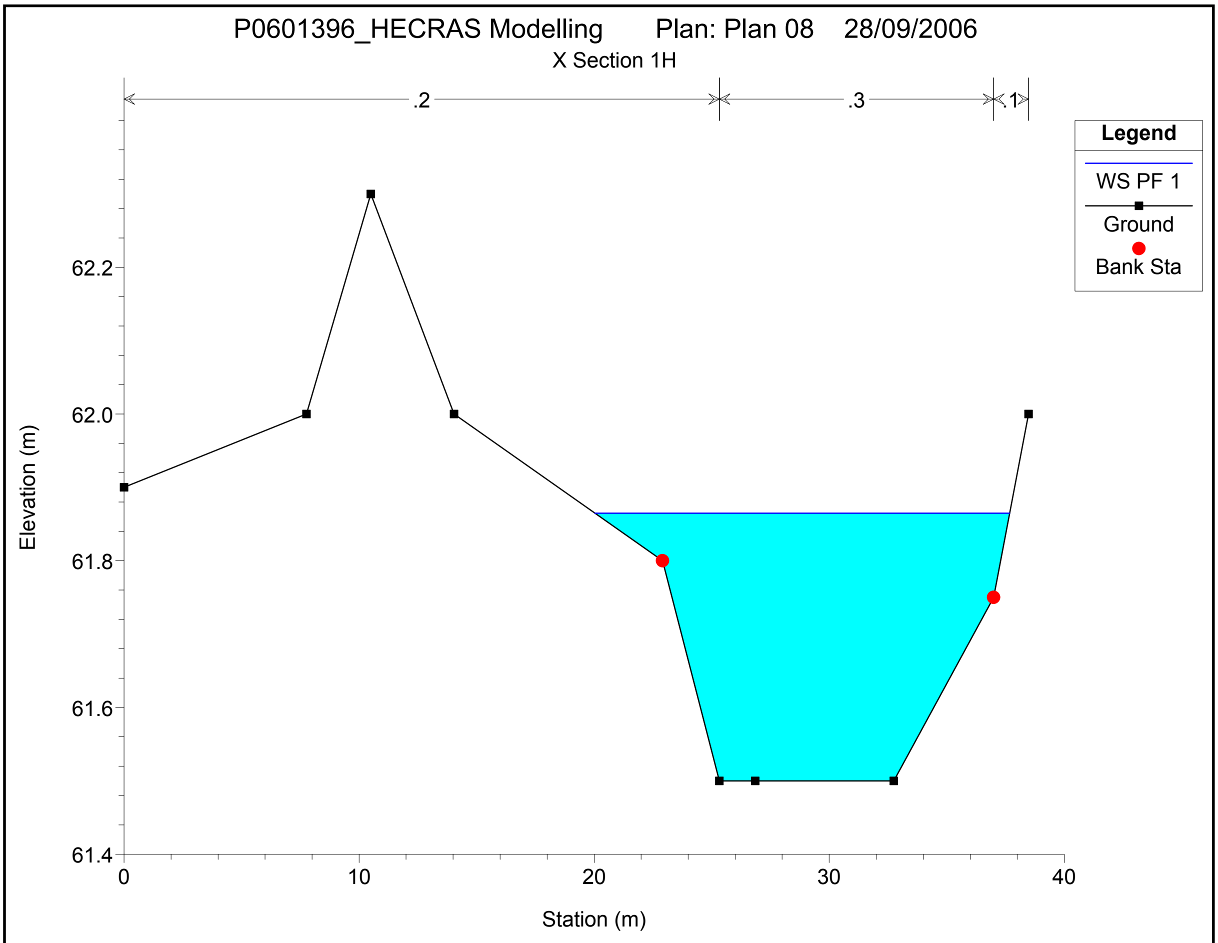
30
40
Station (m)



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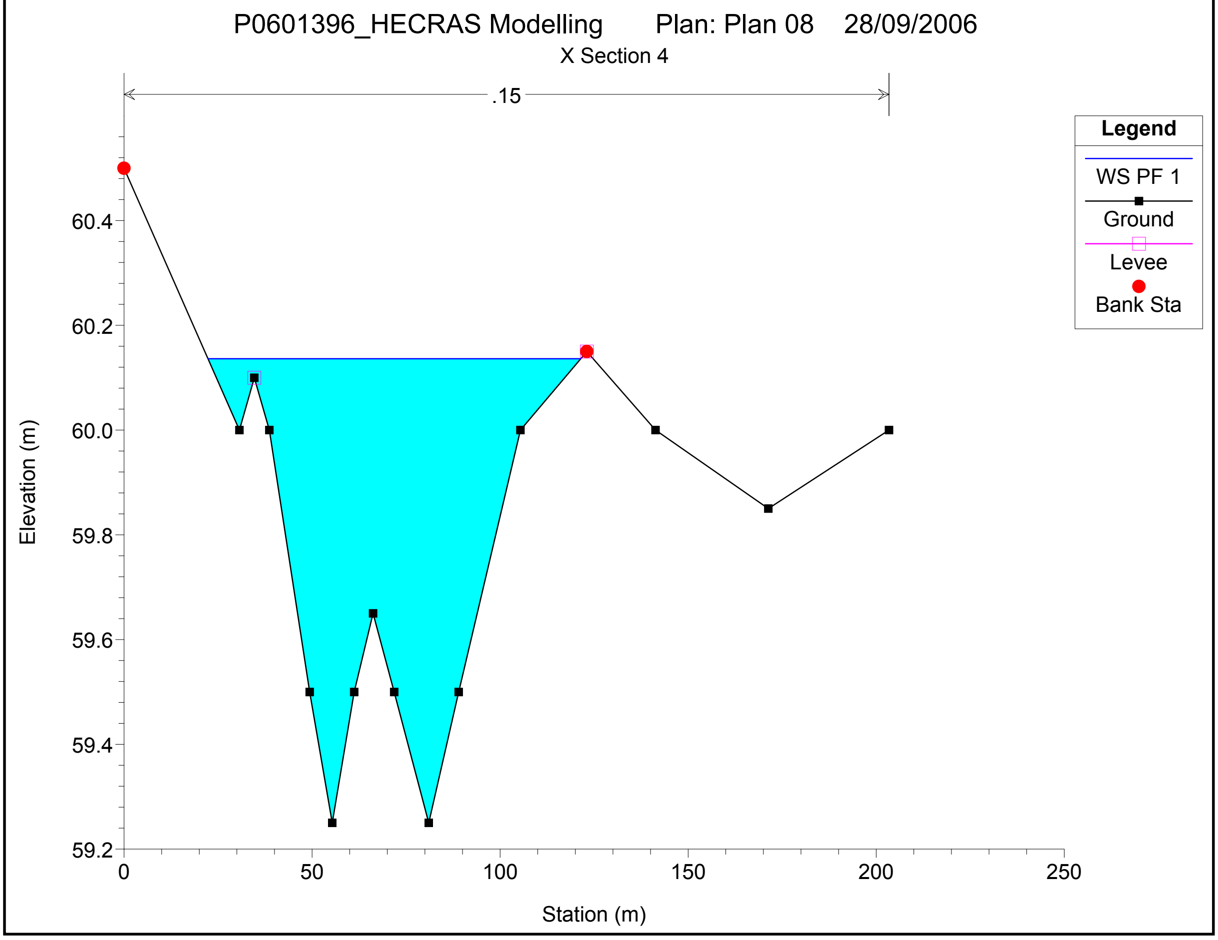
Station (m)



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Levee
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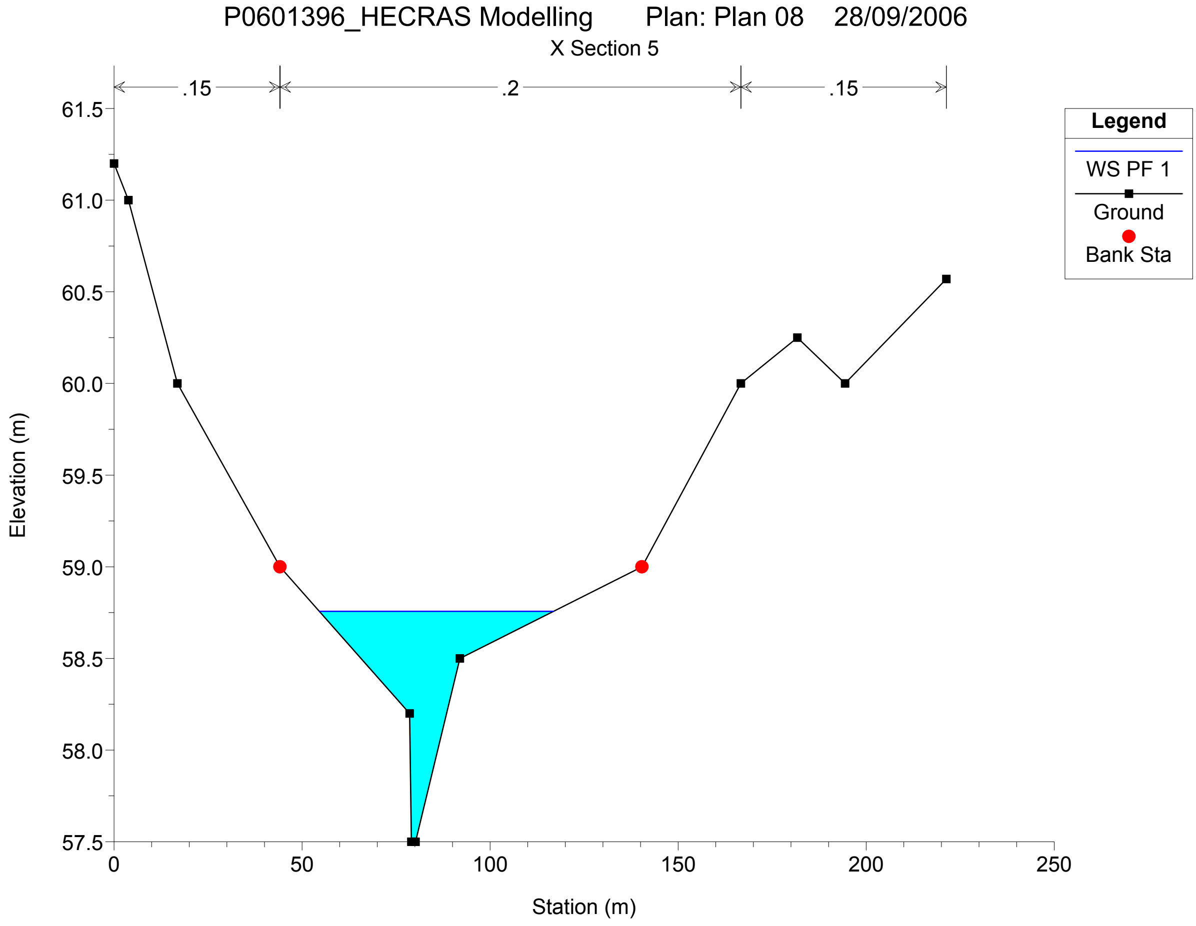
150
200
250
Station (m)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Bank Sta
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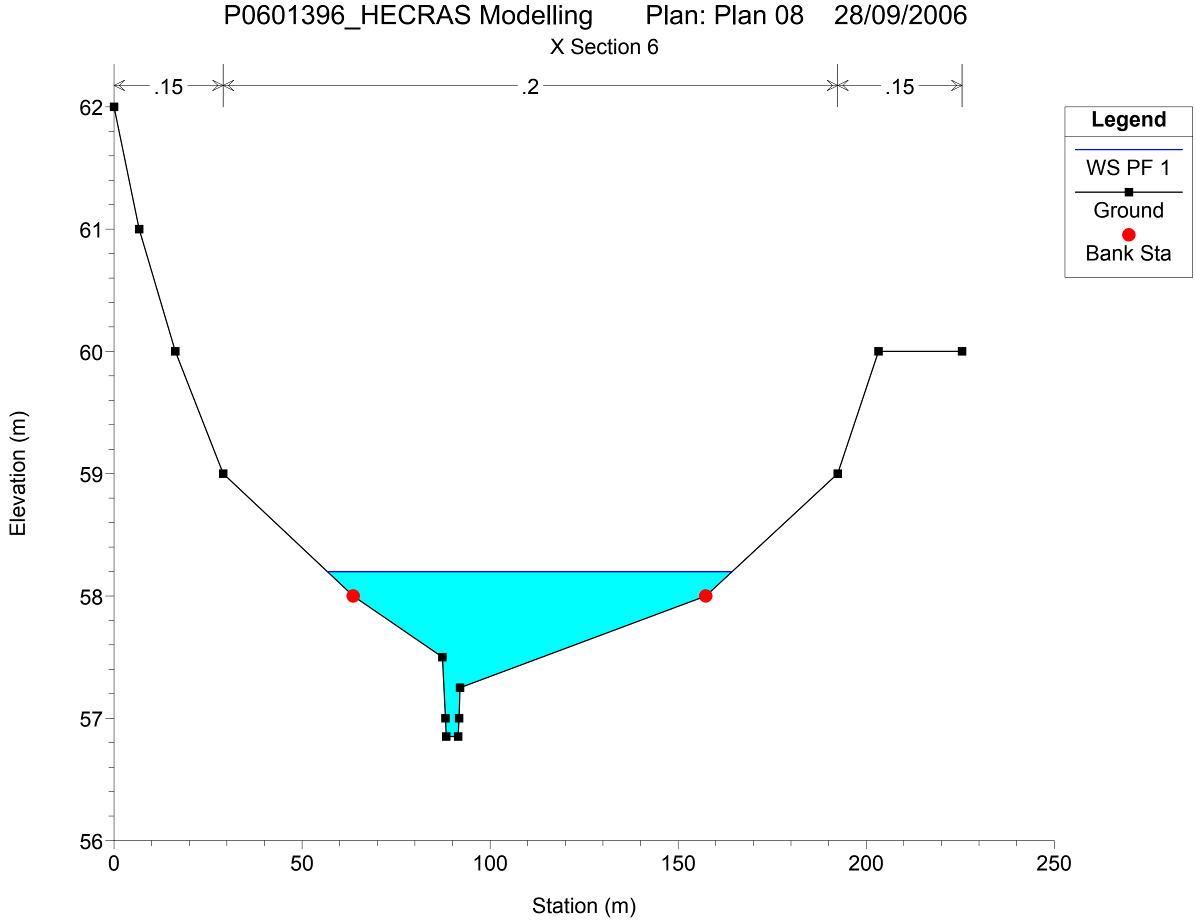
200
250
Station (m)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Legend
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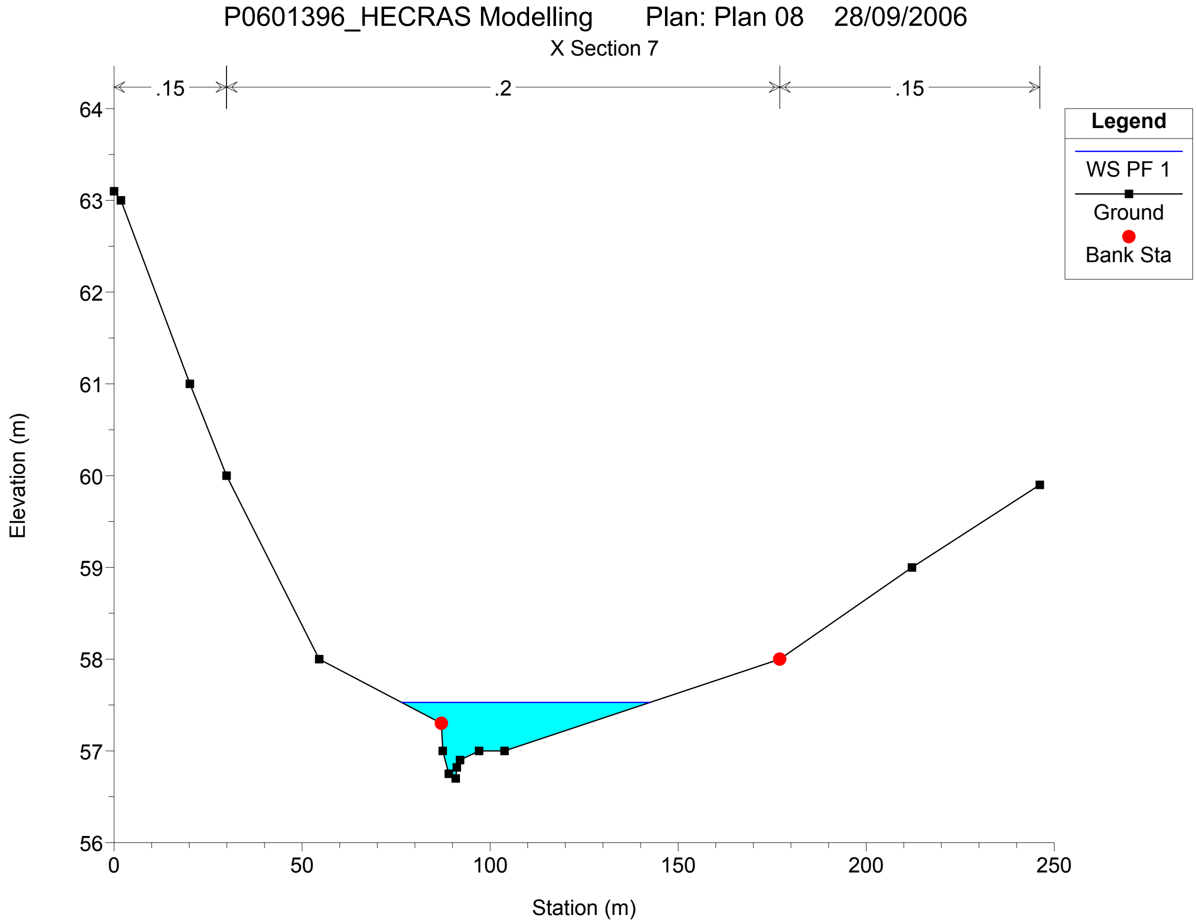
Station (m)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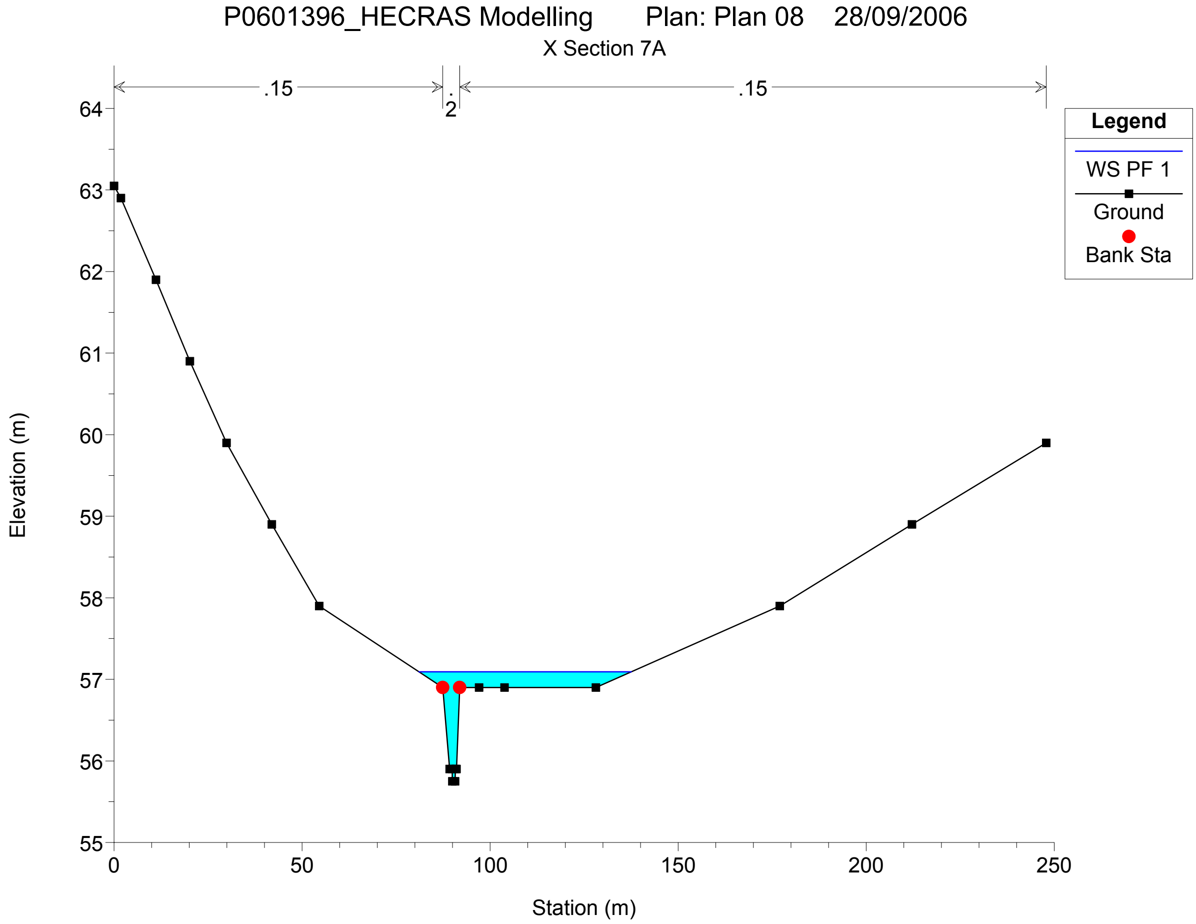
Station (m)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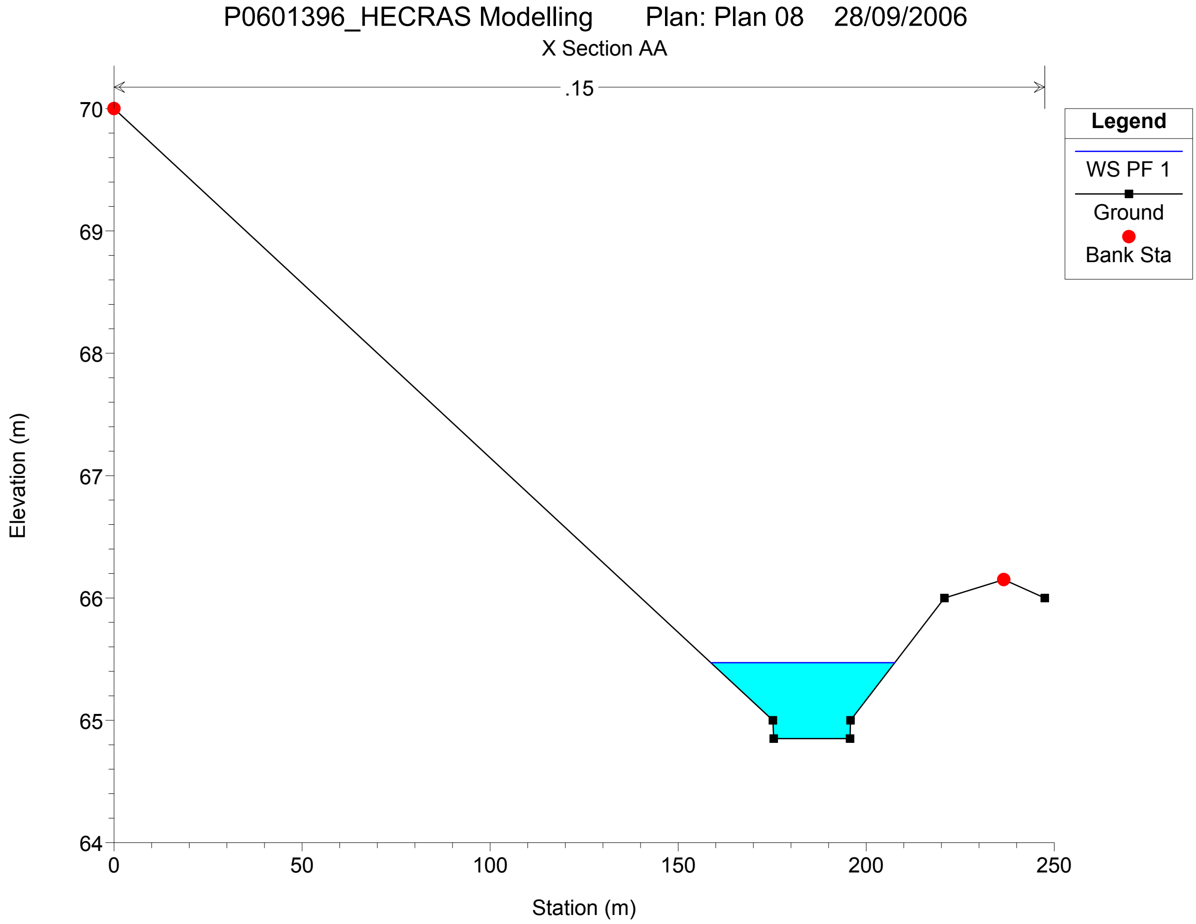
Station (m)



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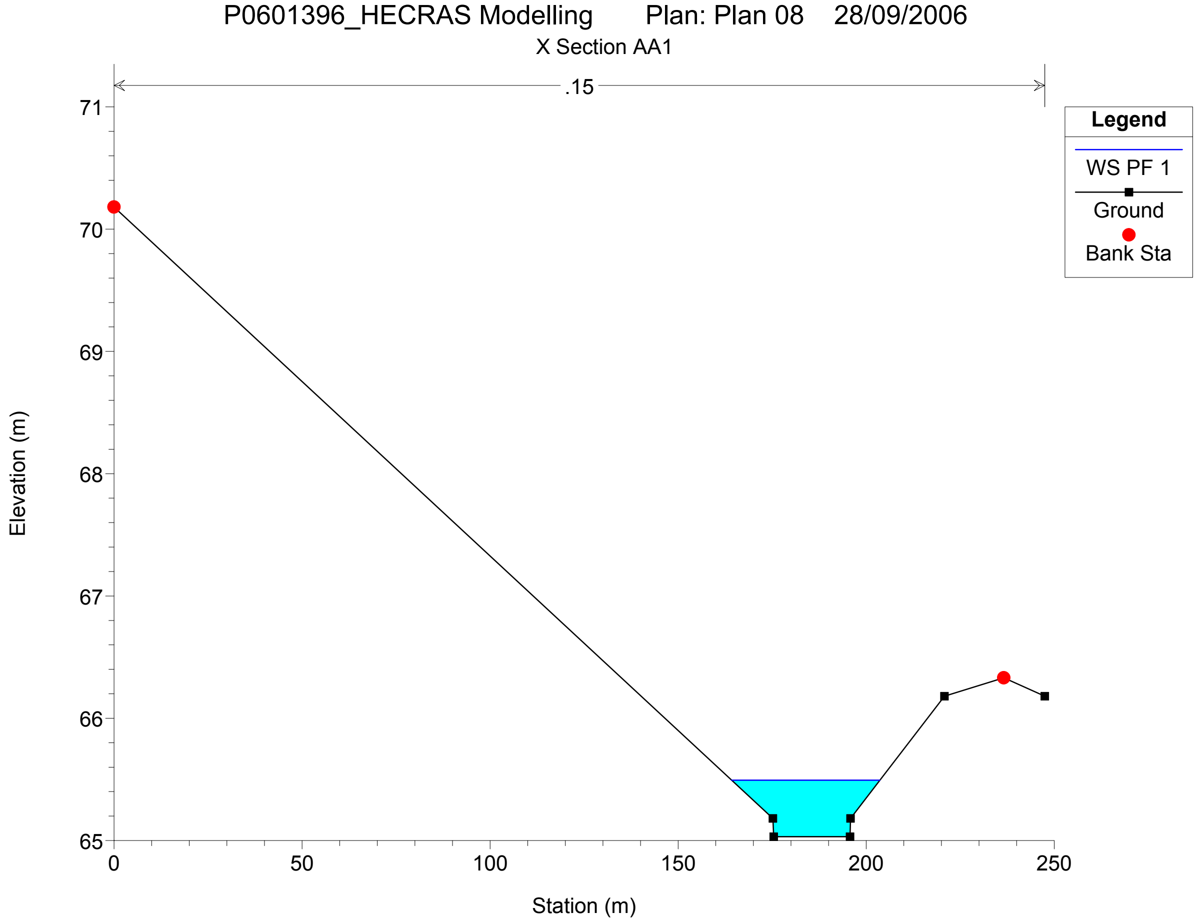
Station (m)



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Ground
Bank Sta
Elevation (m)
71
70
69
68
67
66
65
0
50
100
150
200
250
Station (m)

10 Attachment F – Report Notes

Information

Important Information About Your Report

Subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Martens to help you interpret and understand the limitations of your report. Not all of course, are necessarily relevant to all reports, but are included as general reference.

Engineering Reports - Limitations

Geotechnical reports are based on information gained from limited sub-surface site testing and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretative rather than factual documents, limited to some extent by the scope of information on which they rely.

Engineering Reports – Project Specific Criteria

Engineering reports are prepared by qualified personnel and are based on the information obtained, on current engineering standards of interpretation and analysis, and on the basis of your unique project specific requirements as understood by Martens. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the Client.

Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relative if the design proposal is changed (eg. to a twenty storey building). Your report should not be relied upon if there are changes to the project without first asking Martens to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Martens will not accept responsibility for problems that may occur due to design changes if they are not consulted.

Engineering Reports – Recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption often cannot be substantiated until project implementation has commenced and therefore your site investigation report recommendations should only be regarded as preliminary.

Only Martens, who prepared the report, are fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Martens cannot be held responsible for such misinterpretation.

Engineering Reports – Use For Tendering Purposes

Where information obtained from this investigation is provided for tendering purposes, Martens recommend that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. Attention is drawn to the document 'Guidelines for the Provision of Geotechnical Information in Tender Documents', published by the Institution of Engineers, Australia.

The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Engineering Reports – Data

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way.

Logs, figures, drawings *etc* are customarily included in a Martens report and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These data should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Engineering Reports – Other Projects

To avoid misuse of the information contained in your report it is recommended that you confer with Martens before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Subsurface Conditions - General

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects, relevant standards and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions - the potential for will depend partly on test point (eg. excavation or borehole) spacing and sampling frequency which are often limited by project imposed budgetary constraints.

- Changes in guidelines, standards and policy or interpretation of guidelines, standards and policy by statutory authorities.

- The actions of contractors responding to commercial pressures.

- Actual conditions differing somewhat from those inferred to exist, because no professional, no matter how qualified, can reveal precisely what is hidden by earth, rock and time.

  The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions

If these conditions occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Subsurface Conditions - Changes

Natural processes and the activity of man create subsurface conditions. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Reports are based on conditions which existed at the time of the subsurface exploration.

Decisions should not be based on a report whose adequacy may have been affected by time. If an extended period of time has elapsed since the report was prepared, consult Martens to be advised how time may have impacted on the project.

Subsurface Conditions - Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those that were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved at the time when conditions are exposed, rather than at some later stage well after the event.

Report Use By Other Design Professionals

To avoid potentially costly misinterpretations when other design professionals develop their plans based on a report, retain Martens to work with other project professionals who are affected by the report. This may involve Martens explaining the report design implications and then reviewing plans and specifications produced to see how they have incorporated the report findings.

Subsurface Conditions - Geoenvironmental Issues

Your report generally does not relate to any findings, conclusions, or recommendations about the potential for hazardous or contaminated materials existing at the site unless specifically required to do so as part of the Company's proposal for works.

Specific sampling guidelines and specialist equipment, techniques and personnel are typically used to perform geoenvironmental or site contamination assessments. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Martens for information relating to such matters.

Responsibility

Geotechnical reporting relies on interpretation of factual information based on professional judgment and opinion and has an inherent level of uncertainty attached to it and is typically far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded.

To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Martens to other parties but are included to identify where Martens' responsibilities begin and end. Their use is intended to help all parties involved to recognize their individual responsibilities. Read all documents from Martens closely and do not hesitate to ask any questions you may have.

Site Inspections

Martens will always be pleased to provide engineering inspection services for aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site. Martens is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction.

Soil Data

Explanation of Terms (1 of 3)

Definitions

In engineering terms, soil includes every type of uncemented or partially cemented inorganic or organic material found in the ground. In practice, if the material does not exhibit any visible rock properties and can be remoulded or disintegrated by hand in its field condition or in water it is described as a soil. Other materials are described using rock description terms.

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726 and the S.A.A Site Investigation Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

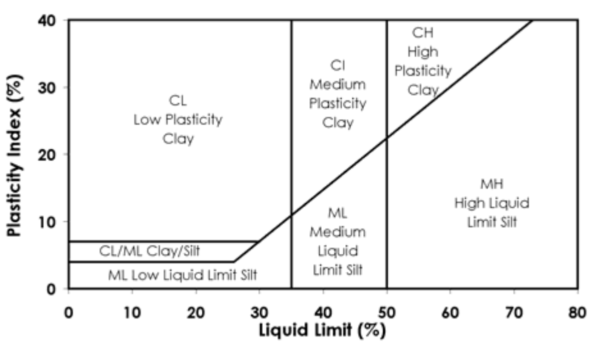
Particle Size

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay). Unless otherwise stated, particle size is described in accordance with the following table.

Division	Subdivision	Size
BOULDERS		>200 mm
COBBLES		60 to 200 mm
GRAVEL	Coarse	20 to 60 mm
	Medium	6 to 20 mm
	Fine	2 to 6 mm
SAND	Coarse	0.6 to 2.0 mm
	Medium	0.2 to 0.6 mm
	Fine	0.075 to 0.2 mm
SILT		0.002 to 0.075 mm
CLAY		< 0.002 mm

Plasticity Properties

Plasticity properties can be assessed either in the field by tactile properties, or by laboratory procedures.



Moisture Condition

Dry — Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.

Moist — Soil feels cool and damp and is darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.

Wet — As for moist but with free water forming on hands when handled.

Consistency of Cohesive Soils

Cohesive soils refer to predominantly clay materials.

Term	C_u (kPa)	Approx SPT "N"	Field Guide
Very Soft	<12	2	A finger can be pushed well into the soil with little effort.
Soft	12 - 25	2 to 4	A finger can be pushed into the soil to about 25mm depth.
Firm	25 - 50	4 – 8	The soil can be indented about 5mm with the thumb, but not penetrated.
Stiff	50 - 100	8 – 15	The surface of the soil can be indented with the thumb, but not penetrated.
Very Stiff	100 - 200	15 – 30	The surface of the soil can be marked, but not indented with thumb pressure.
Hard	> 200	> 30	The surface of the soil can be marked only with the thumbnail.
Friable	-		Crumbles or powders when scraped by thumbnail

Density of Granular Soils

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration test (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	%	SPT 'N' Value (blows/300mm)	CPT Cone Value (q_c Mpa)
Very loose	< 15	< 5	< 2
Loose	15 – 35	5 - 10	2 -5
Medium dense	35 – 65	10 - 30	5 - 15
Dense	65- 85	30 - 50	15 - 25
Very dense	> 85	> 50	> 25

Minor Components

Minor components in soils may be present and readily detectable, but have little bearing on general geotechnical classification. Terms include:

Term	Assessment	Proportion of Minor component In:
Trace of	Presence just detectable by feel or eye, but soil properties little or no different to general properties of primary component.	Coarse grained soils: < 5 % Fine grained soils: < 15 %
With some	Presence easily detectable by feel or eye, soil properties little different to general properties of primary component.	Coarse grained soils: 5 – 12 % Fine grained soils: 15 – 30 %

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Soil Agricultural Classification Scheme

In some situations, such as where soils are to be used for effluent disposal purposes, soils are often more appropriately classified in terms of traditional agricultural classification schemes. Where a Martens report provides agricultural classifications, these are undertaken in accordance with descriptions by Northcote, K.H. (1979) *The factual key for the recognition of Australian Soils*, Rellim Technical Publications, NSW, p 26 - 28.

Symbol	Field Texture Grade	Behaviour of moist bolus	Ribbon length	Clay content (%)
S	Sand	Coherence nil to very slight; cannot be moulded; single grains adhere to fingers	0 mm	< 5
LS	Loamy sand	Slight coherence; discolours fingers with dark organic stain	6.35 mm	5
CLS	Clayey sand	Slight coherence; sticky when wet; many sand grains stick to fingers; discolours fingers with clay stain	6.35mm - 1.3cm	5 - 10
SL	Sandy loam	Bolus just coherent but very sandy to touch; dominant sand grains are of medium size and are readily visible	1.3 - 2.5	10 - 15
FSL	Fine sandy loam	Bolus coherent; fine sand can be felt and heard	1.3 - 2.5	10 - 20
SCL^-	Light sandy clay loam	Bolus strongly coherent but sandy to touch, sand grains dominantly medium size and easily visible	2.0	15 - 20
L	Loam	Bolus coherent and rather spongy; smooth feel when manipulated but no obvious sandiness or silkiness; may be somewhat greasy to the touch if much organic matter present	2.5	25
Lfsy	Loam, fine sandy	Bolus coherent and slightly spongy; fine sand can be felt and heard when manipulated	2.5	25
SiL	Silt loam	Coherent bolus, very smooth to silky when manipulated	2.5	25 + > 25 silt
SCL	Sandy clay loam	Strongly coherent bolus sandy to touch; medium size sand grains visible in a finer matrix	2.5 - 3.8	20 - 30
CL	Clay loam	Coherent plastic bolus; smooth to manipulate	3.8 - 5.0	30 - 35
SiCL	Silty clay loam	Coherent smooth bolus; plastic and silky to touch	3.8 - 5.0	30- 35 + > 25 silt
FSCL	Fine sandy clay loam	Coherent bolus; fine sand can be felt and heard	3.8 - 5.0	30 - 35
SC	Sandy clay	Plastic bolus; fine to medium sized sands can be seen, felt or heard in a clayey matrix	5.0 - 7.5	35 - 40
SiC	Silty clay	Plastic bolus; smooth and silky	5.0 - 7.5	35 - 40 + > 25 silt
LC	Light clay	Plastic bolus; smooth to touch; slight resistance to shearing	5.0 - 7.5	35 - 40
LMC	Light medium clay	Plastic bolus; smooth to touch, slightly greater resistance to shearing than LC	7.5	40 - 45
MC	Medium clay	Smooth plastic bolus, handles like plasticine and can be moulded into rods without fracture, some resistance to shearing	> 7.5	45 - 55
HC	Heavy clay	Smooth plastic bolus; handles like stiff plasticine; can be moulded into rods without fracture; firm resistance to shearing	> 7.5	> 50

Soil Data

Explanation of Terms (3 of 3)

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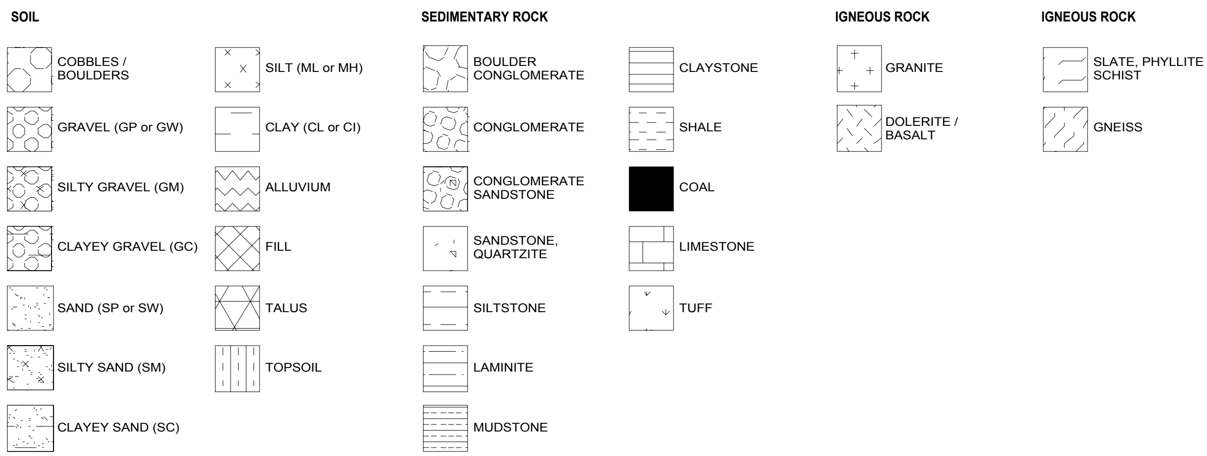
Symbols for Soil and Rock



Unified Soil Classification Scheme (USCS)

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 63 mm and basing fractions on estimated mass)					USCS	Primary Name
COARSE GRAINED SOILS More than 50 % of material less than 63 mm is larger than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than 2.0 mm.	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.	GW	Gravel
				Predominantly one size or a range of sizes with more intermediate sizes missing	GP	Gravel
			GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	Silty Gravel
				Plastic fines (for identification procedures see CL below)	GC	Clayey Gravel
		SANDS More than half of coarse fraction is smaller than 2.0 mm	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of intermediate sizes missing.	SW	Sand
				Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Sand
			SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	SM	Silty Sand
				Plastic fines (for identification procedures see CL below)	SC	Clayey Sand

		IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM					
		DRY STRENGTH (Crushing Characteristics)	**DILATANCY**	**TOUGHNESS**	**DESCRIPTION**	**USCS**	**Primary Name**
FINE GRAINED SOILS More than 50 % of material less than 63 mm is smaller than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	None to Low	Quick to Slow	None	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	Silt
		Medium to High	None	Medium	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays	CL	Clay
		Low to Medium	Slow to Very Slow	Low	Organic slits and organic silty clays of low plasticity	OL	Organic Silt
		Low to Medium	Slow to Very Slow	Low to Medium	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	Silt
		High	None	High	Inorganic clays of high plasticity, fat clays	CH	Clay
		Medium to High	None	Low to Medium	Organic clays of medium to high plasticity	OH	Organic Silt
HIGHLY ORGANIC SOILS		Readily identified by colour, odour, spongy feel and frequently by fibrous texture				Pt	Peat

Low Plasticity – Liquid Limit W_L < 35 %   Medium Plasticity – Liquid limit W_L 35 to 60 %   High Plasticity - Liquid limit W_L > 60 %

Rock Data

Explanation of Terms (1 of 2)

Definitions

Descriptive terms used for Rock by Martens are given below and include rock substance, rock defects and rock mass.

Rock Substance — In geotechnical engineering terms, rock substance is any naturally occurring aggregate of minerals and organic matter which cannot, unless extremely weathered, be disintegrated or remoulded by hand in air or water. Other material is described using soil descriptive terms. Rock substance is effectively homogeneous and may be isotropic or anisotropic.

Rock Defect — Discontinuity or break in the continuity of a substance or substances.

Rock Mass — Any body of material which is not effectively homogeneous. It can consist of two or more substances without defects, or one or more substances with one or more defects.

Degree of Weathering

Rock weathering is defined as the degree in rock structure and grain property decline and can be readily determined in the field.

Term	Symbol	Definition
Residual Soil	Rs	Soil derived from the weathering of rock. The mass structure and substance fabric are no longer evident. There is a large change in volume but the soil has not been significantly transported.
Extremely weathered	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - ie. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly weathered	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decrease compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original rock substance is no longer recognisable.
Moderately weathered	MW	Rock substance affected by weathering to the extent that staining extends throughout the whole of the rock substance and the original colour of the fresh rock is no longer recognisable.
Slightly weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh	Fr	Rock substance unaffected by weathering

Rock Strength

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance is the direction normal to the bedding. The test procedure is described by the International Society of Rock Mechanics.

Term	Is (50) MPa	Field Guide	Symbol
Extremely weak	< 0.03	Easily remoulded by hand to a material with soil properties.	EW
Very weak	0.03 - 0.1	May be crumbled in the hand. Sandstone is 'sugary' and friable.	VW
Weak	0.1 - 0.3	A piece of core 150mm long x 50mm diameter may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.	W
Medium strong	0.3 - 1	A piece of core 150mm long x 50mm diameter can be broken by hand with considerable difficulty. Readily scored with a knife.	MS
Strong	1 - 3	A piece of core 150mm long x 50mm diameter cannot be broken by unaided hands, can be slightly scratched or scored with a knife.	S
Very Strong	3 - 10	A piece of core 150mm long x 50mm diameter may be broken readily with hand held hammer. Cannot be scratched with pen knife.	VS
Extremely strong	> 10	A piece of core 150mm long x 50mm diameter is difficult to break with hand held hammer. Rings when struck with a hammer.	ES

Degree of Fracturing

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but excludes fractures such as drilling breaks.

Term	Description
Fragmented	The core is comprised primarily of fragments of length less than 20mm, and mostly of width less than core diameter.
Highly fractured	Core lengths are generally less than 20mm-40mm with occasional fragments.
Fractured	Core lengths are mainly 30mm-100mm with occasional shorter and longer sections.
Slightly fractured	Core lengths are generally 300mm-1000mm with occasional longer sections and occasional sections of 100mm-300mm.
Unbroken	The core does not contain any fractures.

Rock Core Recovery

TCR = Total Core Recovery

$$= \frac{\text{Length of core recovered}}{\text{Length of core run}} \times 100\%$$

SCR = Solid Core Recovery

$$= \frac{\sum \text{Length of cylindrical core recovered}}{\text{Length of core run}} \times 100\%$$

RQD = Rock Quality Designation

$$= \frac{\sum \text{Axial lengths of core} > 100\,\text{mm long}}{\text{Length of core run}} \times 100\%$$

Rock Strength Tests

- ▼ Point load strength Index (Is50) - axial test (MPa)
- ▶ Point load strength Index (Is50) - diametrall test (MPa)
- ● Unconfined compressive strength (UCS) (MPa)

Defect Type Abbreviations and Descriptions

Defect Type (with inclination given)		Coating or Filling		Roughness	
BP	Bedding plane parting	Cn	Clean	Po	Polished
X	Foliation	Sn	Stain	Ro	Rough
L	Cleavage	Ct	Coating	Sl	Slickensided
JT	Joint	Fe	Iron Oxide	Sm	Smooth
F	Fracture			Vr	Very rough
SZ	Sheared zone (Fault)	**Planarity**		**Inclination**	
CS	Crushed seam	Cu	Curved	The inclination of defects are measured from perpendicular to the core axis.	
DS	Decomposed seam	Ir	Irregular		
IS	Infilled seam	Pl	Planar		
V	Vein	St	Stepped		
		Un	Undulating		

Test Methods

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Sampling

Sampling is carried out during drilling or excavation to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples may be taken by pushing a thin-walled sample tube into the soils and withdrawing a soil sample in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils. Other sampling methods may be used. Details of the type and method of sampling are given in the report.

Drilling Methods

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Hand Excavation – in some situations, excavation using hand tools such as mattock and spade may be required due to limited site access or shallow soil profiles.

Hand Auger - the hole is advanced by pushing and rotating either a sand or clay auger generally 75-100mm in diameter into the ground. The depth of penetration is usually limited to the length of the auger pole, however extender pieces can be added to lengthen this.

Test Pits - these are excavated with a backhoe or a tracked excavator, allowing close examination of the *in-situ* soils if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) - the hole is advanced by a rotating plate or short spiral auger, generally 300mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling - the hole is advanced by pushing a 100mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength *etc.* is only marginally affected.

Continuous Spiral Flight Augers - the hole is advanced using 90 - 115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or *in-situ* testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface or, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling - similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling - a continuous core sample is obtained using a diamond tipped core barrel, usually 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in AS 1289 Methods of Testing Soils for Engineering Purposes - Test F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

(i) In the case where full penetration is obtained with successive blow counts for each 150mm of say 4, 6 and 7 blows:

as 4, 6, 7
N = 13
(ii) In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm

as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

CONE PENETROMETER TESTING AND INTERPRETATION

Cone penetrometer testing (sometimes referred to as Dutch Cone - abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in AS 1289 - Test F4.1.

In the test, a 35mm diameter rod with a cone tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on separate 130mm long sleeve, immediately behind the cone. Tranducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output on continuous chart recorders. The plotted results given in this report have been traced from the original records.

The information provided on the charts comprises:
Cone resistance - the actual end bearing force divided by the cross sectional area of the cone - expressed in MPA.
Sleeve friction - the frictional force of the sleeve divided by the surface area - expressed in kPa.
Friction ratio - the ratio of sleeve friction to cone resistance - expressed in percent.

There are two scales available for measurement of cone resistance. The lower (A) scale (0 - 5 Mpa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main (B) scale (0 - 50 Mpa) is less sensitive and is shown as a full line.

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%-2% are commonly encountered in sands and very soft clays rising to 4%-10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:

q_c (Mpa) = (0.4 to 0.6) N (blows/300mm)

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:

$q_c = (12 \text{ to } 18)\ c_u$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes *etc.* This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

DYNAMIC CONE (HAND) PENETROMETERS

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods. Two relatively similar tests are used.

Perth sand penetrometer - a 16 mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS 1289 - Test F 3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
Cone penetrometer (sometimes known as the Scala Penetrometer) - a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS 1289 - Test F 3.2). The test was developed initially for pavement sub-grade investigations, with correlations of the test results with California bearing ratio published by various Road Authorities.

LABORATORY TESTING

Laboratory testing is carried out in accordance with AS 1289 Methods of Testing Soil for Engineering Purposes. Details of the test procedure used are given on the individual report forms.

TEST PIT / BORE LOGS

The test pit / bore log(s) presented herein are an engineering and/or geological interpretation of the sub-surface conditions and their reliability will depend to some extent on frequency of sampling and the method of excavation / drilling. Ideally, continuous undisturbed sampling or excavation / core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variation between the boreholes.

GROUND WATER

Where ground water levels are measured in boreholes, there are several potential problems:

In low permeability soils, ground water although present, may enter the hole slowly, or perhaps not at all during the time it is left open.
A localised perched water table may lead to an erroneous indication of the true water table.
Water table levels will vary from time to time with seasons or recent prior weather changes. They may not be the same at the time of construction as are indicated in the report.
The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.