

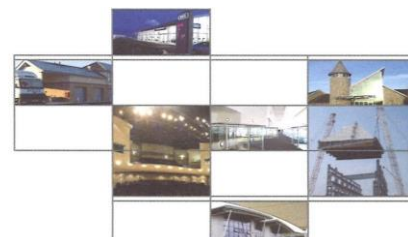
DRAINAGE CALCULATIONS

COACH DEPOT

FOR

DJ THOMAS COACHES

JOB REF	ISSUE	PREPARED BY	CHECKED BY	APPROVED BY	DATE
20.11	/	KJ	KJT	KJT	08.02.2021



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1. INTRODUCTION

1.1 BACKGROUND

1.1.1 Brief

J&N have been appointed to design and detail a drainage system for a coach depot located in Baglan Industrial Estate. The depot will comprise of a maintenance workshop together with parking for 50 coaches and vehicles.



Fig. 1.1.1 Site Map: DJ Thomas Coaches, Port Talbot

Grid ref (6 figure): SS749910

Altitude: 6m

1.2 REFERENCES

The design is carried in accordance with the following documents: -

- CIRIA publication C753 - The SUDS Manual
- The Wallingford Procedure – Volume 4 The Modified Rational Method

1.3 SCHEDULE OF SOFTWARE USED

- Tekla TEDDS

1.4 REFERENCE DRAWINGS

James & Nicholas Drawing – 20.17 C01 Proposed Site Layout

James & Nicholas Drawing – 20.17 C02 Proposed Drainage Plan

1.5 DESIGNERS

Designed by: Karl Jones BSc (Hons) MSc

Checked by: Kevin Tobin MSc DIC CEng MICE

Approved by: Kevin Tobin MSc DIC CEng MICE

2. EXISTING DRAINAGE

2.0 EXISTING DRAINAGE

2.1 EXISTING DRAINAGE DESCRIPTION

The area outlined is has no existing drainage infrastructure located within the site boundary. There are two existing rising mains located to the west and east of the boundary. All surface water around the site is captured and drained via a gully system which is disposed of to a 450mm concrete DCWW sewer located northwest of the site boundary.

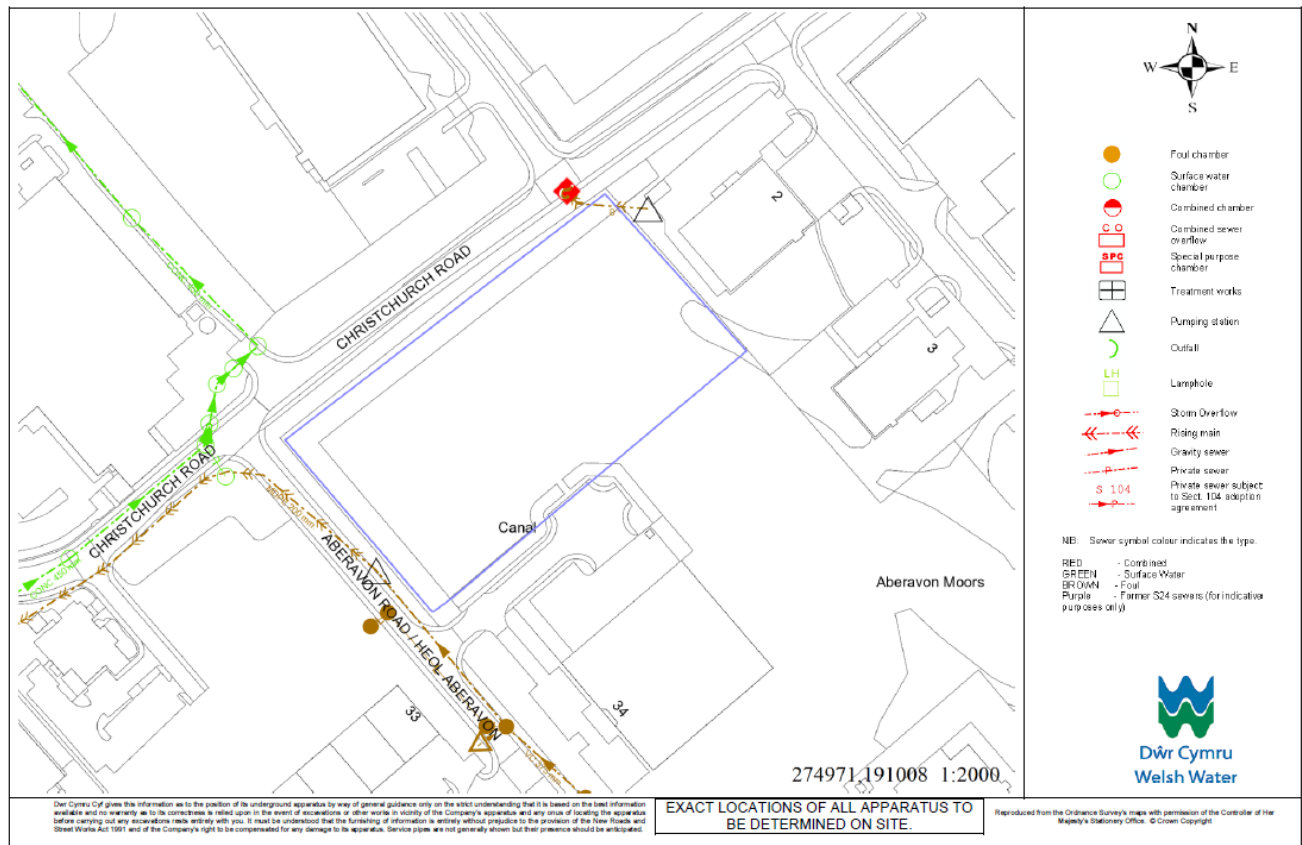


Fig. 2.1 Existing DCWW drainage infrastructure.

3. PROPOSED DRAINAGE

3.1 PROPOSED DRAINAGE LAYOUT

The site is currently being used as a coach depot that stores and maintains the fleet of coaches for DJ Thomas Coaches Ltd. Half of the site is currently open space where the other half has been stripped back and a temporary stone surface installed. It is proposed that the site will be entirely stripped back and then covered with a tarmac/concrete surface to allow vehicular movement across the site.

3.1.1 SURFACE WATER DRAINAGE

It is proposed that hardstanding surface water will be collected via ACO drainage system and attenuated prior to discharge into the existing mains drainage. Before the water is attenuated it will pass through a separator to remove any potential containments. Water collected from the maintenance workshop roof will go through this system.

Any surface water collected from the vehicle wash down area will be collected via a separate system, and disposed of within the foul water system. Water from the wash down area will be run through a class 1 retention separator to remove any potential oil/fuel spills from the vehicles.

3.1.2 FOUL WATER DRAINAGE

The building will be served by a foul water drain which will be connected to a manhole that is drained to a DCWW pumping station. See Appendix A for proposed drainage layout.

3.2 DRAINAGE CALCULATIONS

3.2.1 SURFACE WATER ATTENUATION

The proposed discharge point for the surface water is the DCWW 450mm concrete surface water sewer that is located on Christchurch Road. The drainage will be designed to be discharged to DCWW at the green field run off rate (subject to confirmation from DCWW).

To restrict the discharge rate an attenuation tank with a hydrobrake will be implemented to achieve the green field run off rate.

Hardstanding areas to be drained:

Tarmac surfacing	= 8163m ²
Concrete surfacing	= 1268m ²
Roof	= <u>700m²</u>
Total Area	= 10171m ²

The required amount of attenuation will be designed to for the following parameters:
1 in 100 year storm plus 30% climate change

From TEDDS Output the required amount of attenuation is 697m³. See Appendix B for full analysis.

3.2.1 SURFACE WATER PIPE SIZE CHECK

The most critical pipe run is run S1-S3 as this picks up the majority of surface water from the car park. Below are the design flow rates that will entering each manhole.

Flow Rate of Rainfall Runoff

Permeability Factor (P) = 0.95 (Table 2 BSEN754 Part 4)

Rainfall intensity (R) = 50 mm/hour
(Building Regs Part H – Section H3 Cl. 2.4 states normal rainfall intensity can be taken as 0.014 litres/sec/m²) which is the equivalent of the above figure)

$$\text{Flow rate } Q \text{ (m}^3\text{/s)} = \frac{A \times P \times R}{3600}$$

3.2.1.1 S1 FLOW RATE

Hardstanding area draining to manhole = 4081.5m²

$$\text{S1 Flow rate } Q \text{ (m}^3\text{/s)} = \frac{4081.5 \times 0.95 \times 50 \times 10^{-3}}{3600}$$

$$\text{S1 Flow rate } Q \text{ (m}^3\text{/s)} = 0.053 \text{ m}^3\text{/s}$$

3.2.1.2 S2 FLOW RATE

Hardstanding area draining to manhole = 8163m²

$$\text{S2 Flow rate } Q \text{ (m}^3\text{/s)} = \frac{8163 \times 0.95 \times 50 \times 10^{-3}}{3600}$$

$$\text{S2 Flow rate } Q \text{ (m}^3\text{/s)} = 0.108 \text{ m}^3\text{/s}$$

3.2.1.3 S3 FLOW RATE

Hardstanding area draining to manhole = 10371m²

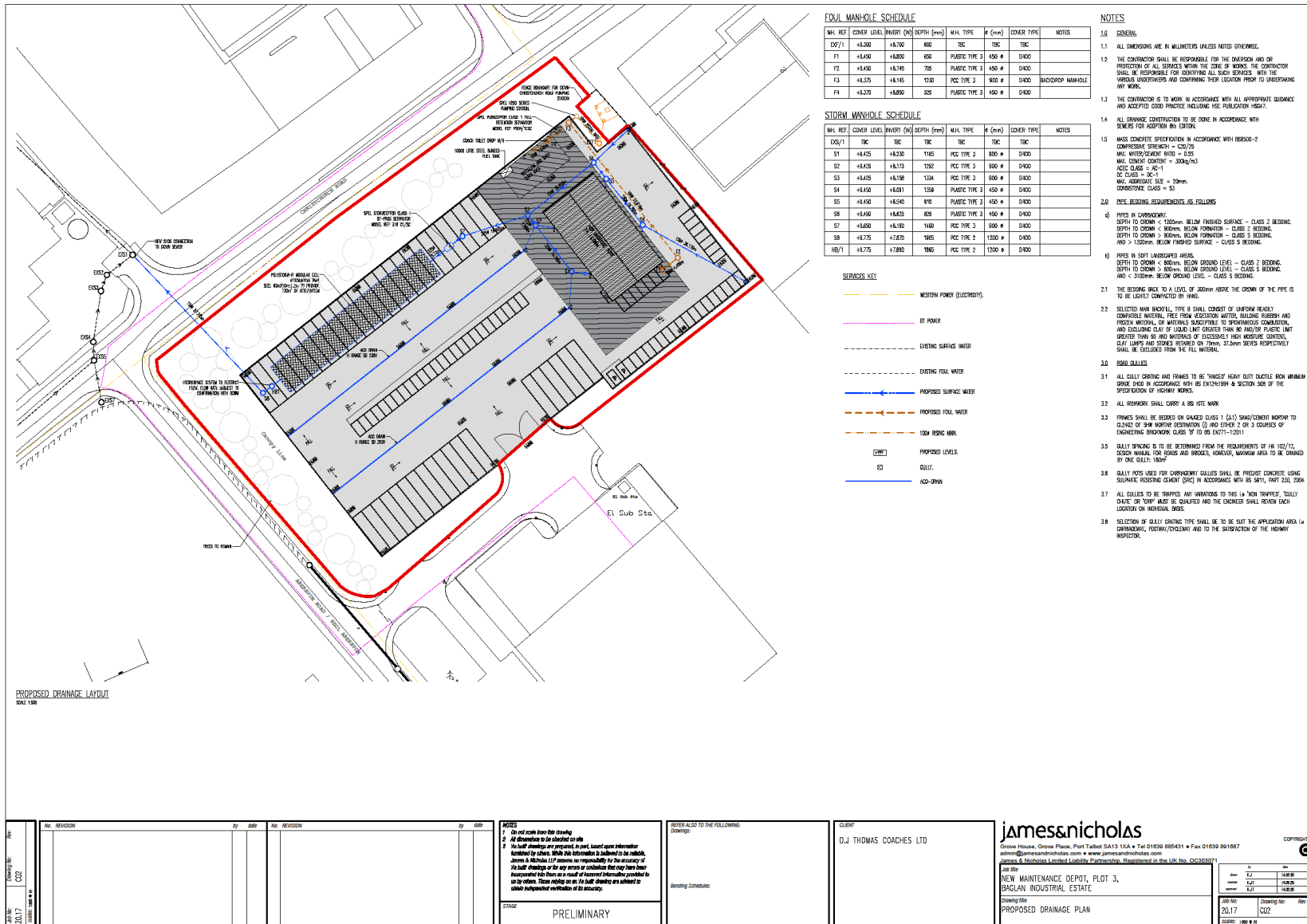
$$\text{S3 Flow rate } Q \text{ (m}^3\text{/s)} = \frac{10371 \times 0.95 \times 50 \times 10^{-3}}{3600}$$

$$\text{S3 Flow rate } Q \text{ (m}^3\text{/s)} = 0.137 \text{ m}^3\text{/s}$$

Full TEDDS analysis the largest pipe size required is 300mm. See Appendix C for full pipe size analysis.

APPENDIX A

PROPOSED DRAINAGE LAYOUT PLAN



APPENDIX B

ATTENUATION TANK TEDDS ANALYSIS

ATTENUATION DESIGN

In accordance with CIRIA publication C753 - The SUDS Manual

Tedds calculation version 1.0.04

EA_Defra method

Site characteristics

Location	Swansea
Hydrological region	9
Soil type (Wallingford Procedure W.R.A.P map)	3
Standard percentage runoff	SPR = 0.37
Average annual rainfall	SAAR = 1200 mm
5 year return period rainfall of 60 minute duration	M5_60min = 18.0 mm
Ratio 60-minute to 2 day rainfalls of 5 year return	r = 0.28
Rainfall intensity increase due to global warming	p _{climate} = 0%
Impervious area req. attenuation storage	α = 100.0 %

Catchment details

Subcatchment	Name	Area (ha)	PIMP (%)	Impermeable area (ha)
1;	Hardstanding;	0.94;	100.0	0.94;
2;	Roof Area;	0.07;	100.0	0.07;
Total		1.01;	100.0	1.01;

Greenfield runoff rates

Catchment area	AREA = 50.00 hectare
Greenfield runoff rate (50 hectare site)	$\bar{Q}_{\text{rural}} = 0.00108 \text{ m}^3/\text{s} \times (\text{AREA}/1\text{km}^2)^{0.89} \times (\text{SAAR}/1\text{mm})^{1.17} \times \text{SPR}^{2.17} = \mathbf{269.9 \text{ l/s}}$
Greenfield runoff rate	$\bar{Q} = \bar{Q}_{\text{rural}} / \text{AREA} \times A = \mathbf{5.5 \text{ l/s}}$
Greenfield runoff rate per unit area	$\bar{Q}_A = \bar{Q} / A = \mathbf{5.4 \text{ l/s/hectare}}$

Estimated site discharges

FSR growth rate (1 year)	FSR _{1yr} = 0.88
Discharge (1 year)	Q _{1yr} = $\bar{Q} \times \text{FSR}_{1\text{yr}} = \mathbf{4.8 \text{ l/s}}$
FSR growth rate (30 year)	FSR _{30yr} = 1.78
Discharge (30 year)	Q _{30yr} = $\bar{Q} \times \text{FSR}_{30\text{yr}} = \mathbf{9.7 \text{ l/s}}$
FSR growth rate (100 year)	FSR _{100yr} = 2.18
Discharge (100 year)	Q _{100yr} = $\bar{Q} \times \text{FSR}_{100\text{yr}} = \mathbf{11.9 \text{ l/s}}$

Estimated attenuation volume - 1 year

Attenuation storage vol (fig A7.1 - A7.8)	Uvol _{1yr} = 180.9 m ³ / hectare
Basic storage volume	BSV _{1yr} = Uvol _{1yr} × α × A = 182.70 m ³
FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4)	FF _{1yr} = 0.85
Storage volume ratio (fig A8.1 - A8.8)	SVR _{1yr} = 1.26
Adjusted storage volume	ASV _{1yr} = SVR _{1yr} × BSV _{1yr} = 230.57 m ³
Hydrological regional volume ratio (fig A9.1)	HR _{1yr} = 1.00
Final estimated attenuation storage	Vol _{1yr} = HR _{1yr} × ASV _{1yr} = 230.94 m ³

Library item: Estimated attenuation output

Estimated attenuation volume - 30 year

Attenuation storage vol (fig A7.1 - A7.8)	Uvol _{30yr} = 356.1 m ³ / hectare
Basic storage volume	BSV _{30yr} = Uvol _{30yr} × α × A = 359.62 m ³
FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4)	FF _{30yr} = 0.80
Storage volume ratio (fig A8.1 - A8.8)	SVR _{30yr} = 1.38
Adjusted storage volume	ASV _{30yr} = SVR _{30yr} × BSV _{30yr} = 496.41 m ³

Hydrological regional volume ratio (fig A9.1)
Final estimated attenuation storage

$$HR_{30yr} = \mathbf{1.07}$$

$$Vol_{30yr} = HR_{30yr} \times ASV_{30yr} = \mathbf{530.83 \text{ m}^3}$$

Library item: Estimated attenuation output

Estimated attenuation volume - 100 year

Attenuation storage vol (fig A7.1 - A7.8)
Basic storage volume
FEH rainfall factor (figs A11.1, A6.1.1 - A6.3.4)
Storage volume ratio (fig A8.1 - A8.8)
Adjusted storage volume
Hydrological regional volume ratio (fig A9.1)
Final estimated attenuation storage

$$Uvol_{100yr} = \mathbf{447.8 \text{ m}^3 / \text{hectare}}$$

$$BSV_{100yr} = Uvol_{100yr} \times \alpha \times A = \mathbf{452.28 \text{ m}^3}$$

$$FF_{100yr} = \mathbf{0.80}$$

$$SVR_{100yr} = \mathbf{1.38}$$

$$ASV_{100yr} = SVR_{100yr} \times BSV_{100yr} = \mathbf{624.31 \text{ m}^3}$$

$$HR_{100yr} = \mathbf{1.12}$$

$$Vol_{100yr} = HR_{100yr} \times ASV_{100yr} = \mathbf{697.34 \text{ m}^3}$$

Library item: Estimated attenuation output

Attenuation storage required

Vol. increase due to head-discharge relationship
Maximum attenuation storage required

$$p_{hydro} = \mathbf{1.25}$$

$$V_{req_max} = Vol_{30yr} \times p_{hydro} = \mathbf{663.5 \text{ m}^3}$$

Interception storage

Interception rainfall depth
Volume of interception storage required

$$d_{int} = \mathbf{5 \text{ mm}}$$

$$V_{int_req} = 0.8 \times A_{imp} \times d_{int} = \mathbf{40.40 \text{ m}^3}$$

Long term storage

Proportion of paved area draining in to network
Proportion of pervious area draining in to network
Rainfall depth for 100years, 6 hour event
Extra runoff vol of dev.runoff over greenfield runoff

$$\alpha = \mathbf{1.0}$$

$$\beta = \mathbf{0.5}$$

$$RD = M100_360 = \mathbf{64.6 \text{ mm}}$$

$$Vol_{xs} = \max(RD \times A \times (PIMP \times \alpha \times 0.8 + ((1 - PIMP) \times \beta \times SPR) - SPR), 0 \text{ m}^3) = \mathbf{280.65 \text{ m}^3}$$

Treatment volume

Treatment volume (assume 80% runoff)

$$T_{vol} = 0.8 \times A \times 15 \text{ mm} \times PIMP = \mathbf{121.20 \text{ m}^3}$$

APPENDIX C

PIPE SIZE TEDDS ANALYSIS

STORM WATER DRAINAGE DESIGN

In accordance with UK recommended structure losses

TEDDS calculation version 1.0.01

Pipe 1

Internal diameter	$D_{I_1} = 0.3 \text{ m}$
Internal area	$A_1 = \pi \times D_{I_1}^2 / 4 = 0.07 \text{ m}^2$
Length	$L_1 = 19.84 \text{ m}$
Gradient	$S_1 = 0.01 \%$
Flow	$Q_1 = 0.14 \text{ m}^3/\text{s}$
Velocity of fluid	$V_1 = Q_1 / A_1 = 1.95 \text{ m/s}$
Invert level (downstream)	$h_{in_1} = 8.23 \text{ m}$
Invert level (upstream)	$h_{out_2} = 8.23 \text{ m}$

Pipe 2

Internal diameter	$D_{I_2} = 0.3 \text{ m}$
Internal area	$A_2 = \pi \times D_{I_2}^2 / 4 = 0.07 \text{ m}^2$
Length	$L_2 = 2.29 \text{ m}$
Gradient	$S_2 = 0.01 \%$
Flow	$Q_2 = 0.11 \text{ m}^3/\text{s}$
Velocity of fluid	$V_2 = Q_2 / A_2 = 1.53 \text{ m/s}$
Invert level (downstream)	$h_{in_2} = 8.23 \text{ m}$
Invert level (upstream)	$h_{out_3} = 8.23 \text{ m}$

Pipe 3

Internal diameter	$D_{I_3} = 0.225 \text{ m}$
Internal area	$A_3 = \pi \times D_{I_3}^2 / 4 = 0.04 \text{ m}^2$
Length	$L_3 = 19.84 \text{ m}$
Gradient	$S_3 = 0.17 \%$
Flow	$Q_3 = 0.05 \text{ m}^3/\text{s}$
Velocity of fluid	$V_3 = Q_3 / A_3 = 1.33 \text{ m/s}$
Invert level (downstream)	$h_{in_3} = 8.23 \text{ m}$
Invert level (upstream)	$h_{out_4} = 8.27 \text{ m}$

Boundary conditions

Tailwater level	$H_{ds} = 8.190 \text{ m}$
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Upstream water level
Freeboard
Velocity beyond outlet
Hydraulic grade line – downstream of structure 1
Energy grade line – downstream of structure 1

Local headloss – Structure number 1

Structure type
Loss coefficient
Head loss through structure
Hydraulic grade line – upstream of structure 1
Energy grade line – upstream of structure 1

$$\begin{aligned}H_{us} &= \mathbf{8.200\ m} \\h_{free} &= \mathbf{0.100\ m} \\V_{out} &= \mathbf{0.00\ m/s} \\HGL_{ds_1} &= H_{ds} = \mathbf{8.19\ m} \\EGL_{ds_1} &= HGL_{ds_1} + (V_{out}^2 / (2 \times g_{acc})) = \mathbf{8.19\ m} \\Exit \\k_e &= \mathbf{1.00} \\h_{e_1} &= k_e \times (V_1^2 / (2 \times g_{acc})) = \mathbf{0.19\ m} \\HGL_{us_1} &= HGL_{ds_1} = \mathbf{8.19\ m} \\EGL_{us_1} &= HGL_{us_1} + k_e \times (V_1^2 / (2 \times g_{acc})) = \mathbf{8.38\ m}\end{aligned}$$

PASS - HGL is lower than ground level minus freeboard

Friction headloss

Effective roughness
Kinematic viscosity of fluid
Reynolds number
Friction factor
Hydraulic Gradient
Friction headloss
Energy grade line – downstream of structure 2
Hydraulic grade line – downstream of structure 2

$$\begin{aligned}k_{s_1} &= \mathbf{0.0\ mm} \\v &= \mathbf{1.31 \times 10^{-6}\ m^2/s} \\Re &= (V_1 \times D_{l_1}) / v = \mathbf{447092} \\\lambda &= (1 / (-2 \times \log((k_{s_1} / (3.7 \times D_{l_1})) + (5.1286 / Re^{0.89}))))^2 = \mathbf{0.015} \\S_f &= \lambda \times V_1^2 / (2 \times g_{acc} \times D_{l_1}) = \mathbf{0.0095} \\h_{f_1} &= S_f \times L_1 = \mathbf{0.19\ m} \\EGL_{ds_2} &= EGL_{us_1} + h_{f_1} = \mathbf{8.57\ m} \\HGL_{ds_2} &= EGL_{ds_2} - V_1^2 / (2 \times g_{acc}) = \mathbf{8.38\ m}\end{aligned}$$

PASS - HGL is lower than ground level minus freeboard

Local headloss – Structure number 2

Structure type
Loss coefficient
Head loss through structure
Hydraulic grade line – upstream of structure 2
Energy grade line – upstream of structure 2

$$\begin{aligned}\text{Manhole} \\k_e &= \mathbf{1.00} \\h_{e_2} &= k_e \times (V_1^2 / (2 \times g_{acc})) = \mathbf{0.19\ m} \\HGL_{us_2} &= HGL_{ds_2} + h_{e_2} = \mathbf{8.57\ m} \\EGL_{us_2} &= HGL_{us_2} + (V_2^2 / (2 \times g_{acc})) = \mathbf{8.69\ m}\end{aligned}$$

PASS - HGL is lower than ground level minus freeboard

Friction headloss

Effective roughness
Kinematic viscosity of fluid
Reynolds number
Friction factor
Hydraulic Gradient
Friction headloss
Energy grade line – downstream of structure 3
Hydraulic grade line – downstream of structure 3

$$\begin{aligned}k_{s_2} &= \mathbf{0.3\ mm} \\v &= \mathbf{1.31 \times 10^{-6}\ m^2/s} \\Re &= (V_2 \times D_{l_2}) / v = \mathbf{349898} \\\lambda &= (1 / (-2 \times \log((k_{s_2} / (3.7 \times D_{l_2})) + (5.1286 / Re^{0.89}))))^2 = \mathbf{0.021} \\S_f &= \lambda \times V_2^2 / (2 \times g_{acc} \times D_{l_2}) = \mathbf{0.0082} \\h_{f_2} &= S_f \times L_2 = \mathbf{0.02\ m} \\EGL_{ds_3} &= EGL_{us_2} + h_{f_2} = \mathbf{8.71\ m} \\HGL_{ds_3} &= EGL_{ds_3} - V_2^2 / (2 \times g_{acc}) = \mathbf{8.59\ m}\end{aligned}$$

PASS - HGL is lower than ground level minus freeboard

Local headloss – Structure number 3

Structure type
Loss coefficient
Head loss through structure
Hydraulic grade line – upstream of structure 3
Energy grade line – upstream of structure 3

$$\begin{aligned}\text{Manhole} \\k_e &= \mathbf{1.00} \\h_{e_3} &= k_e \times (V_2^2 / (2 \times g_{acc})) = \mathbf{0.12\ m} \\HGL_{us_3} &= HGL_{ds_3} + h_{e_3} = \mathbf{8.71\ m} \\EGL_{us_3} &= HGL_{us_3} + (V_3^2 / (2 \times g_{acc})) = \mathbf{8.80\ m}\end{aligned}$$

PASS - HGL is lower than ground level minus freeboard

Friction headloss

Effective roughness

Kinematic viscosity of fluid

Reynolds number

Friction factor

Hydraulic Gradient

Friction headloss

Energy grade line – downstream of structure 4

Hydraulic grade line – downstream of structure 4

$$k_{s_3} = 0.3 \text{ mm}$$

$$\nu = 1.31 \times 10^{-6} \text{ m}^2/\text{s}$$

$$Re = (V_3 \times D_{L_3}) / \nu = 228946$$

$$\lambda = (1 / (-2 \times \log((k_{s_3} / (3.7 \times D_{L_3})) + (5.1286 / Re^{0.89}))))^2 = 0.022$$

$$S_f = \lambda \times V_3^2 / (2 \times g_{acc} \times D_{L_3}) = 0.0090$$

$$h_{f_3} = S_f \times L_3 = 0.18 \text{ m}$$

$$EGL_{ds_4} = EGL_{us_3} + h_{f_3} = 8.98 \text{ m}$$

$$HGL_{ds_4} = EGL_{ds_4} - V_3^2 / (2 \times g_{acc}) = 8.89 \text{ m}$$

PASS - HGL is lower than ground level minus freeboard

Local headloss – Structure number 4

Structure type

Loss coefficient

Head loss through structure

Hydraulic grade line – upstream of structure 4

Energy grade line – upstream of structure 4

Manhole

$$k_e = 1.00$$

$$h_{e_4} = k_e \times (V_3^2 / (2 \times g_{acc})) = 0.09 \text{ m}$$

$$HGL_{us_4} = HGL_{ds_4} + h_{e_4} = 8.98 \text{ m}$$

$$EGL_{us_4} = HGL_{us_4} + (V_3^2 / (2 \times g_{acc})) = 9.07 \text{ m}$$

PASS - HGL is lower than ground level minus freeboard