REVISION	DATE	DESCRIPTION	DESIGN	DRAWN	APPROVED
H	12.09.24	DETAIL AMENDED	NK	NK	
G	08.07.24	WALL HEIGHTS AMENDED	BJC	BJC	
F	06.06.24	WALL HEIGHTS ADJUSTED, WALL ADDED, SSD ADDED	BJC	BJC	
E	22.05.24	RETAINING WALLS AND CHANNEL AMENDED	NK	NK	
D	14.05.24	CHANNEL ADDED	BJC	NK	
C	06.05.24	PARKING BAY AMENDED	BJC	BJC	
B	24.04.24	BATTER ADDED, TYPICAL SECTION ADDED	BJC	BJC	

REVISION HISTORY

100 MILLIMETRES ON ORIGINAL DRAWING

0 2 4 6 8 10
SCALE OF METRES 1:200 AT A1

DESIGN	NK	DESIGN CHECK	BJC
DRAWN	NK	DRAFTING CHECK	BJC
APPROVED		DATE	

CAD FILE: 22-3021-C048

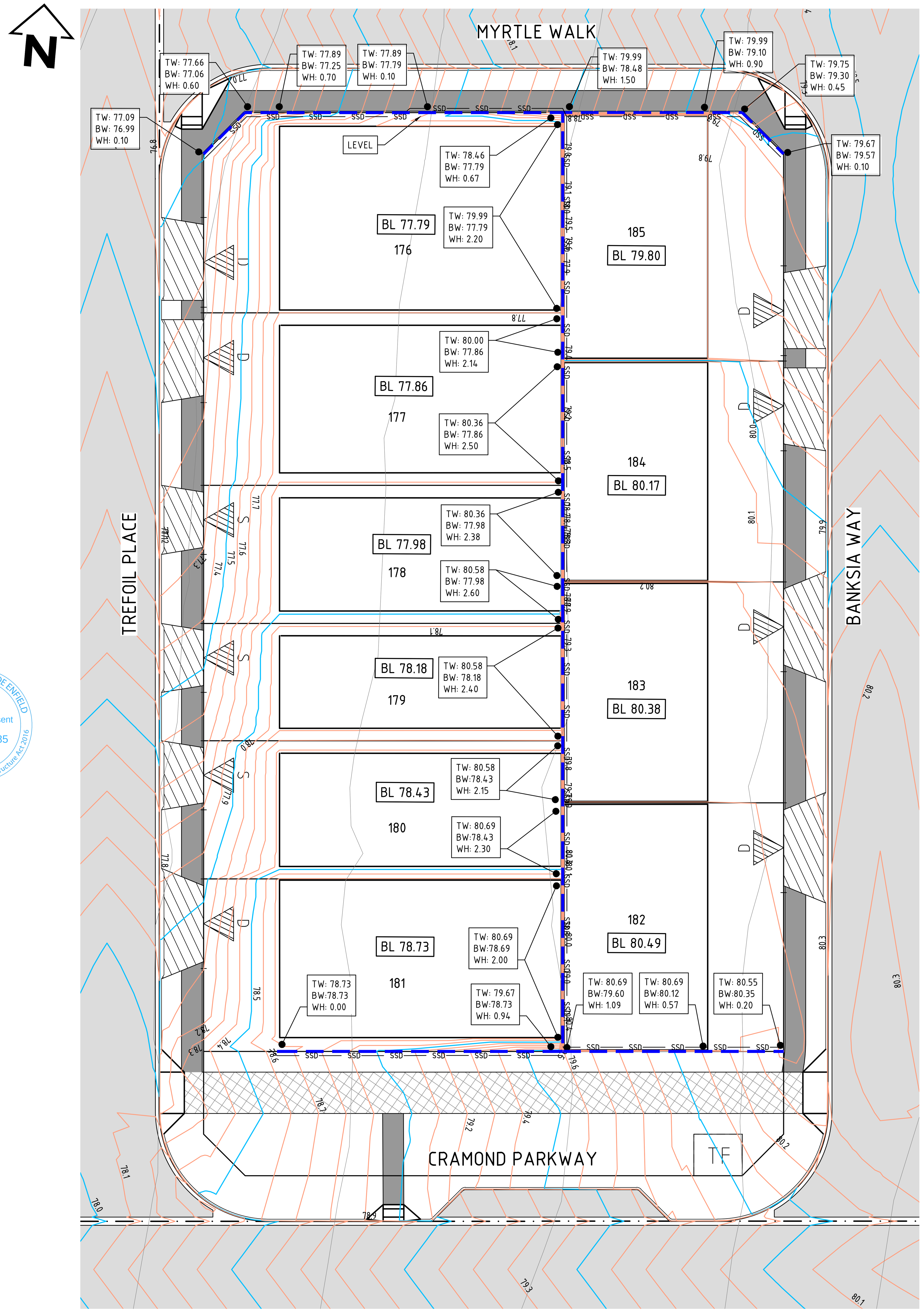
A1

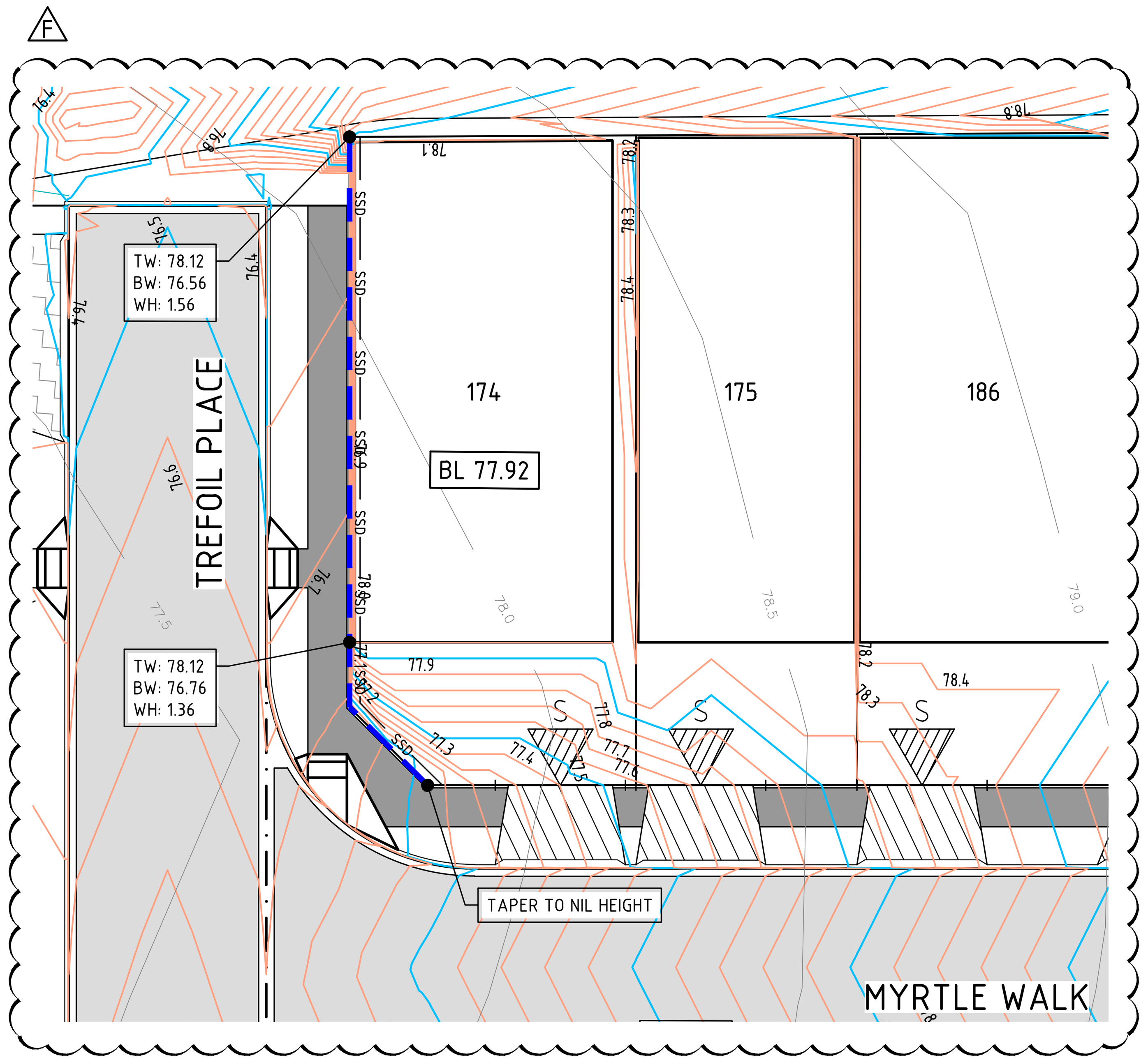
GREENHILL
Level 1, 178 Fullarton Road
Dulwich, SA 5065
T: 08 8406 1300
ABN 39 061 222 964

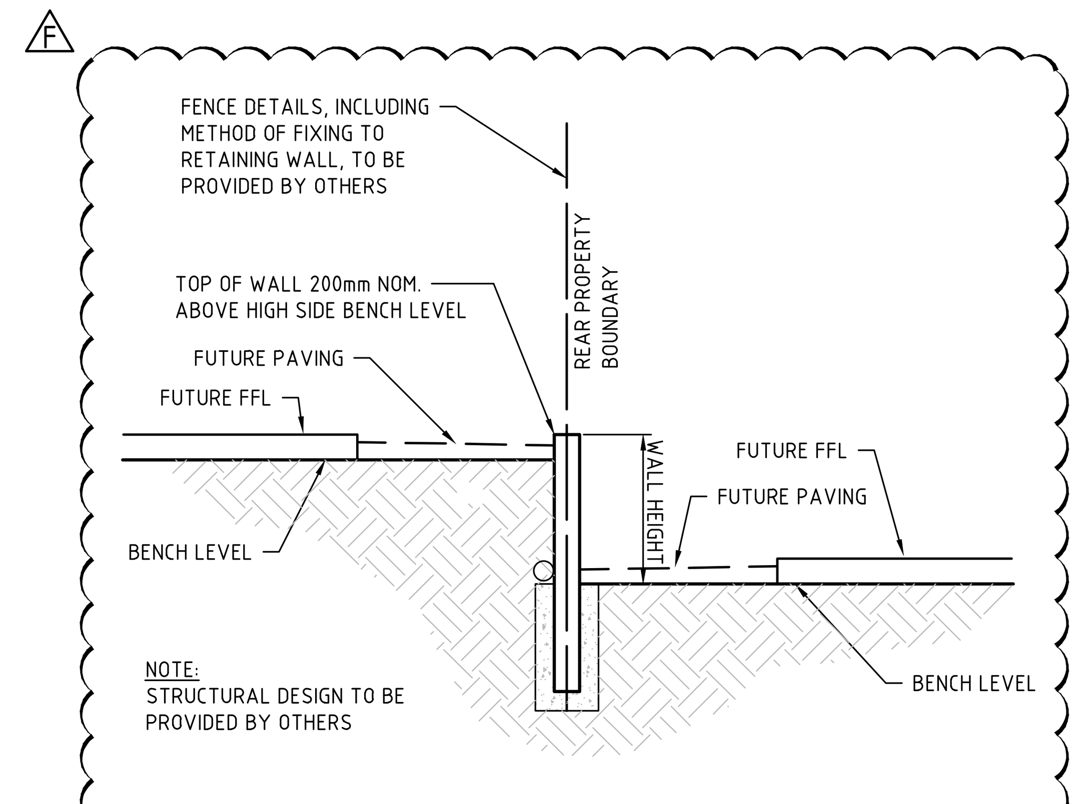
OAKDEN DEVELOPMENTS Pty Ltd
OAKDEN RISE STAGE 2

RETAINING WALL LAYOUT
SHEET 1 OF 3

DRAWING NUMBER: 22-3021-C048
REVISION: H

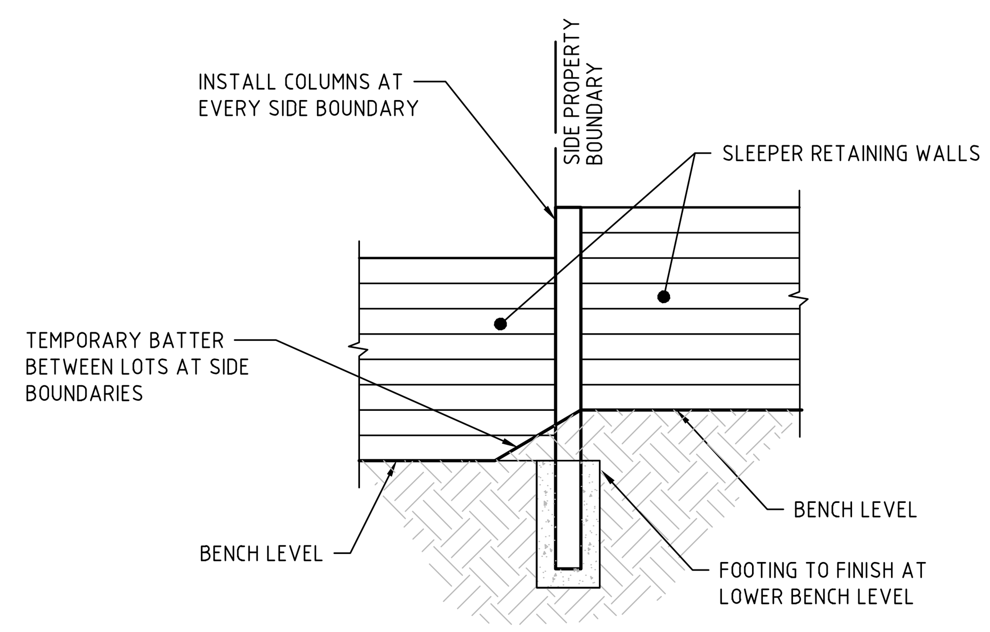






RETAINING WALL SECTION LOTS 176-181

SCALE 1:50



RETAINING WALL ELEVATION AT SIDE BOUNDARY

SCALE 1:50

LEGEND

(red outline)	LOT BENCH
BL 78.62	LOT BENCH LEVEL
(blue dashed line)	RETAINING WALL
—SSD— SSD—	SUB SOIL DRAIN
TW: 80.75 BW: 78.65 WH: 2.10	TOP OF WALL BOTTOM OF WALL NOMINAL WALL HEIGHT
78.0	EXISTING CONTOUR (0.5m INCREMENT)
30.50	DESIGN CONTOURS (0.5m INCREMENT)
30.10	DESIGN CONTOURS (0.1m INCREMENT)

NOT FOR CONSTRUCTION

REVISION	DATE	DESCRIPTION	DESIGN	DRAWN	APPROVED
F	12.09.24	DETAIL AMENDED. LOT 174 ADDED	NK	NK	
E	12.06.24	WALL LOT 176 AMENDED	BJC	BJC	
D	06.06.24	WALL HEIGHTS ADJUSTED, WALL ADDED, SSD ADDED	BJC	BJC	
C	10.05.24	FOOTPATH GRADING AND WALL HEIGHTS AMENDED	BJC	NK	
B	15.04.24	ROAD NAMES AMENDED	BJC	NK	
A	06.03.24	PREVIOUSLY SKC27, TYPICAL SECTION ADDED	BJC	BJC	

REVISION HISTORY

100 MILLIMETRES ON ORIGINAL DRAWING

0 2 4 6 8 10
SCALE OF METRES
1:200 AT A1

DESIGN	NK	DESIGN CHECK	BJC
DRAWN	NK	DRAFTING CHECK	BJC
APPROVED		DATE	

CAD FILE: 22-3021-C049

A1

GREENHILL
Level 1, 178 Fullarton Road
Dulwich, SA 5065
T: 08 8406 1300
ABN 39 061 222 964

OAKDEN DEVELOPMENTS Pty Ltd
OAKDEN RISE STAGE 2

RETAINING WALL LAYOUT
SHEET 2 OF 3

DRAWING NUMBER: 22-3021-C049

REVISION: F

GENERAL

G1 THESE DRAWINGS SHALL BE READ IN CONJUNCTION WITH ALL OTHER CONSULTANTS DRAWINGS AND SPECIFICATIONS. ALL DISCREPANCIES SHALL BE REFERRED TO THE ENGINEER FOR DECISION BEFORE PROCEEDING WITH THE WORK.

G2 DURING CONSTRUCTION THE CONTRACTOR SHALL BE RESPONSIBLE FOR MAINTAINING THE STRUCTURE IN A SAFE AND STABLE CONDITION AND SHALL ENSURE THAT NO PART IS OVERSTRESSED DUE TO CONSTRUCTION ACTIVITIES.

G3 ALL WORKMANSHIP AND MATERIALS SHALL BE IN ACCORDANCE WITH THE RELEVANT SAA CODES AND LOCAL STATUTORY AUTHORITIES.

G4 NO SUBSTITUTIONS SHALL BE MADE OR SIZES OF STRUCTURAL MEMBERS VARIED WITHOUT THE WRITTEN APPROVAL OF THE ENGINEER. THE APPROVAL OF A SUBSTITUTION FROM THE ENGINEER SHALL NOT BE AN AUTHORISATION FOR AN EXTRA. ANY EXTRA INVOLVED SHALL BE APPROVED WITH THE ENGINEER BEFORE WORK COMMENCES.

G5 ALL DIMENSIONS ARE IN MILLIMETRES UNLESS STATED OTHERWISE. ALL LEVELS ARE EXPRESSED IN METRES, AND ARE TO THE STRUCTURAL SURFACE UNLESS NOTED OTHERWISE.

G6 ALL PROPRIETARY PRODUCTS SHALL BE USED STRICTLY IN ACCORDANCE WITH THE MANUFACTURER'S INSTRUCTIONS AND RECOMMENDATIONS.

G7 CONSTRUCTION USING THESE STRUCTURAL DRAWINGS SHALL ONLY COMMENCE IF THE STRUCTURAL DRAWINGS ARE DESIGNATED "ISSUED FOR CONSTRUCTION".

G8 DIMENSIONS SHALL BE TAKEN FROM ARCHITECTURAL DRAWINGS ONLY. STRUCTURAL DRAWINGS SHALL NOT BE SCALED TO OBTAIN DIMENSIONS.

RETAINING WALL

RW1 THE BUILDER IS TO ENSURE RETAINING WALL CUT DOES NOT CAUSE INSTABILITY TO ANY EXISTING STRUCTURES OR TREES. EXISTING STRUCTURE OR TREE LOCATIONS TO BE CONFIRMED ON SITE PRIOR TO CONSTRUCTION. THE BUILDER IS TO MAINTAIN STABILITY OF EXCAVATION DURING CONSTRUCTION BY SHORING, PROPPING, ETC. AS REQUIRED.

RW2 CONCRETE SLEEPERS/PANELS SHALL BE SUPPLIED BY AN APPROVED MANUFACTURER. THE MANUFACTURER SHALL PROVIDE SPECIFIC STRUCTURAL CALCULATIONS DEMONSTRATING STRUCTURAL ADEQUACY FOR USE IN RETAINING WALLS OF THIS HEIGHT AND LOAD CONDITIONS. TIMBER SLEEPERS ARE NOT RECOMMENDED DUE TO LIMITED DURABILITY. SLEEPERS/PANELS SHALL HAVE EQUAL BEARING LENGTH, NO LESS THAN 30mm, EACH END.

RW3 STEEL POSTS TO BE GRADE 300 PLUS. CORROSION PROTECTION SHALL BE PROVIDED IN THE FORM OF HOT DIP GALVANIZING. ENSURE NO LESS THAN 75mm CLEAR CONCRETE COVER TO EMBEDDED STEEL.

RW4 STEEL POSTS TO BE GRADE 300 PLUS. CORROSION PROTECTION SHALL BE PROVIDED IN THE FORM OF HOT DIP GALVANIZING. ENSURE NO LESS THAN 75mm CLEAR CONCRETE COVER TO EMBEDDED STEEL.

RW5 THE RETAINING WALL, INCLUDING BORED PIER EXCAVATIONS, SHALL NOT EXTEND OVER THE BOUNDARY LINE.

RW6 BACKFILL MATERIAL SHALL BE FREE-DRAINING GRANULAR MATERIAL, FREE OF FINES AND CLAY LUMPS. COMPACTION AGAINST THE BACK OF THE WALL SHALL BE LIGHT-DUTY PLANT IN LAYERS NO GREATER THAN 150mm THICK. DO NOT OVERLOAD THE WALL BY HEAVY COMPACTION OF FILL BEHIND THE WALL WITH LARGE EQUIPMENT.

RW7 PROVIDE ADEQUATE DRAINAGE OF WATER BEHIND THE WALL, SURFACE AND SUBSURFACE. FOR SUB-SURFACE WATER THIS MAY BE IN THE FORM OF THE PROVISION OF AGRICULTURAL DRAINS WITH ADEQUATE GRADE (AS SHOWN ON THE TYPICAL DETAILS). WATER COLLECTED FROM UNDERGROUND AGRICULTURAL DRAINS SHALL BE DIRECTED OUT AND AWAY FROM THE RETAINING WALL AND PREFERABLE DISCHARGED TO THE STREET WATER-TABLE VIA A PIPE SYSTEM INDEPENDENT OF THE MAIN STORMWATER SYSTEM. FOR SURFACE WATER, ABOVE GROUND DIVERSION DRAINS OR PAVED SPOON DRAINS ETC. BEHIND THE WALL ON THE HIGH SIDE ARE ALSO CONSIDERED GOOD PRACTICE.

RW8 EXCAVATIONS IN FRONT OF THE WALL HAVE THE POTENTIAL TO DESTABILISE THE STRUCTURE. ALL EXCAVATIONS IN FRONT OF THE WALL SHALL COMPLY WITH THE FOLLOWING IF THEY ARE UNABLE TO BE RELOCATED TO OTHER AREAS. REFER TO THE TYPICAL DETAILS FOR ZONE LOCATIONS:

EXCAVATIONS INTO ZONE 'A'	NO RESTRICTIONS ON EXCAVATION.
EXCAVATIONS INTO ZONE 'B'	TEMPORARY EXCAVATIONS ONLY. BACKFILL WITH COMPACTED MATERIAL.
EXCAVATIONS INTO ZONE 'C'	CAN ENDANGER THE WALL AND MAY REQUIRE ENGINEERED PROPPING DURING EXCAVATION. CONTACT THIS OFFICE FOR ADVICE.

SAFETY-IN-DESIGN REVIEW

MAGRYN & ASSOCIATES (MAGRYN) HAVE CONDUCTED A PRELIMINARY SAFETY-IN-DESIGN REVIEW OF THE DESIGN SHOWN ON THESE DRAWINGS. THE REVIEW IS BASED GENERALLY ON THE PROCEDURE OUTLINED IN THE SAFE WORK AUSTRALIA PUBLICATION "SAFE DESIGN OF STRUCTURES CODE OF PRACTICE" (JULY 2012).

THE DESIGN HAS NOT BEEN REVIEWED WITH A CONTRACTOR/BUILDER AT THE TIME OF ISSUE FOR TENDER OR CONSTRUCTION. CONSTRUCTION METHODS VARY BETWEEN CONTRACTORS SO IT IS NOT POSSIBLE FOR MAGRYN TO PERFORM AN EXHAUSTIVE SAFETY-IN-DESIGN OR SAFETY-IN-CONSTRUCTION REVIEW. ONCE APPOINTED, THE CONTRACTOR IS REQUIRED TO UNDERTAKE A THOROUGH REVIEW OF THE DESIGN WITH THEIR SUB-CONTRACTORS TO IDENTIFY SAFETY RISKS DURING CONSTRUCTION AND DURING THE LIFE OF THE BUILDING.

CONTRACTORS ARE RESPONSIBLE TO REVIEW THEIR PROPOSED ERECTION PROGRAMS/SEQUENCE AND FOR THE TEMPORARY FRAMING TO SUPPORT STRUCTURAL ELEMENTS.
CONTRACTORS SHALL PROVIDE DOCUMENTATION THAT OUTLINE HOW THE PROJECT WAS BUILT SO THAT THE DEMOLITION CONTRACTOR CAN ADEQUATELY EVALUATE RISKS DURING DEMOLITION PLANNING AT THE END OF THE LIFE OF THE BUILDING.

LOCATION OF UNDERGROUND AND ABOVE GROUND SERVICES

1 MAGRYN HAS NOT CARRIED OUT A DIAL-BEFORE-YOU-DIG REVIEW DURING THE DESIGN PHASE. THE CONTRACTOR SHALL UNDERTAKE A REVIEW OF THEIR OWN TO VERIFY SERVICES ENTERING THE PROPERTY AND IN PROXIMITY TO THE BOUNDARY IN THE STREET/SURROUNDING THE PROPERTY.

2 DIAL-BEFORE-YOU-DIG DOES NOT CONFIRM THE LAYOUT OF SERVICES WITHIN THE SITE. THE CONTRACTOR SHALL ALLOW TO ENGAGE A SERVICES LOCATION CONTRACTOR TO CONDUCT A SURVEY OF ALL SERVICES ON THE SITE.

EXCAVATIONS

1 BATTER SLOPES SHALL BE IN ACCORDANCE WITH GEOTECHNICAL RECOMMENDATIONS AND SHALL BE INSPECTED BY THE GEOTECHNICAL ENGINEER TO CONFIRM ADEQUACY (INCLUDING REVIEW OF PROPOSED DURATION OF BATTER).

2 EXCAVATIONS GREATER THAN 1.0m DEEP REQUIRE SHORING AND SHALL NOT BE ACCESSED BY PERSONNEL WITHOUT APPROPRIATE CONFINED SPACE TRAINING.

3 PROVIDE BARRIERS TO ALL EXCAVATIONS TO PREVENT FALLS.

4 ENSURE MEASURES TO PROTECT ADJACENT PROPERTY / STRUCTURES ARE FOLLOWED STRICTLY IN ACCORDANCE WITH THESE DRAWINGS. IF IN DOUBT CONTACT MAGRYN.

5 CONTACT MAGRYN IF GROUND WATER IS ENCOUNTERED DURING EXCAVATION.

BLOCKWORK

BL1 ALL BLOCKWORK AND ITS TESTING SHALL COMPLY WITH THE CURRENT AS 3700 SAA MASONRY CODE.

BL2 ALL BLOCKS USED ON THIS PROJECT SHALL HAVE A MINIMUM CHARACTERISTIC COMPRESSIVE STRENGTH OF 15MPa UNLESS OTHERWISE NOTED.

BL3 ALL BED AND PERPENDICULAR JOINTS SHALL BE SOLIDLY FILLED WITH MORTAR, WITHOUT FURROWING, TO A NOMINAL THICKNESS OF 10mm.

BL4 BRICK TIES SHALL HAVE A MINIMUM EMBEDMENT OF 50mm IN THE MORTAR JOINT IN EACH LEAF OF BLOCKWORK, AND SHALL COMPLY WITH THE REQUIREMENTS OF AS 2699. SIZE, TYPE, HORIZONTAL AND VERTICAL SPACING SHALL COMPLY WITH AS 3700.

BL5 GROUT FOR CORE FILLING SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 20MPa WITH A SLUMP OF 230mm AND A MAXIMUM AGGREGATE SIZE OF 6mm UNLESS NOTED OTHERWISE. TESTING SHALL COMPLY WITH AS 3600 FOR PROJECT ASSESSMENT.

BL6 'CLEAN OUT' OPENINGS SHALL BE PROVIDED AT THE BASE OF ALL CORES TO BE CORE FILLED. ADDITIONAL 'CLEAN OUT' OPENINGS ARE REQUIRED AT ALL HORIZONTAL BREAKS IN CONSTRUCTION.

BL7 MORTAR FINS PROTRUDING FROM JOINTS SHALL BE REMOVED BEFORE GROUTING CORES.

BL8 GROUT SHALL NOT BE POURED INTO CORES FROM ANY HEIGHT GREATER THAN 1000mm. STOP POUR 50mm BELOW TOP OF BLOCK TO PROVIDE KEY FOR THE FOLLOWING POUR. RODDING OR OTHER APPROVED MEANS SHALL BE USED TO ENSURE PROPER COMPACTION OF GROUT IN CORES.

CONCRETE

C1 ALL WORKMANSHIP AND MATERIALS SHALL BE IN ACCORDANCE WITH AS 3600. ALL CONCRETE SHALL BE TESTED BY AN APPROVED NATA INDEPENDENT TESTING LABORATORY.

C2 UNLESS NOTED OTHERWISE, CONCRETE GRADE SHALL BE AS FOLLOWS:

STRIP FOOTINGS	32MPa
FOOTING SLABS	32MPa
BORED PIERS	32MPa
BLOCKWORK CORE-FILLING	20MPa
OTHER NON-SPECIFIED	32MPa

C3 COVER TO STEEL REINFORCEMENT SHALL BE AS FOLLOWS UNLESS SHOWN OTHERWISE ON THE DRAWINGS:

STRIP FOOTINGS	50mm
FOOTING SLABS	50mm
BORED PIERS	50mm
OTHER NON-SPECIFIED	50mm

C4 CONDUITS SHALL NOT BE PLACED WITHIN THE CONCRETE COVER.

C5 CONCRETE ADDITIVES SHALL NOT BE USED WITHOUT THE APPROVAL OF THE ENGINEER.

C6 FINISHES TO ALL CONCRETE SURFACES SHALL BE STEEL FLOAT.

C7 SIZES OF CONCRETE ELEMENTS DO NOT INCLUDE THICKNESS OF APPLIED FINISHES AND SHALL NOT BE ALTERED WITHOUT THE APPROVAL OF THE ENGINEER.

C8 CONSTRUCTION JOINTS SHALL BE PROPERLY FORMED AND USED ONLY WHERE SHOWN OR SPECIFICALLY APPROVED BY THE ENGINEER.

C9 FREE DROPPING OF CONCRETE FROM A HEIGHT GREATER THAN 1200mm SHALL NOT BE PERMITTED.

C10 CONCRETE SHALL BE COMPACTED WITH SUITABLE MECHANICAL VIBRATORS TO AS3600.

C11 CONCRETE SHALL BE PLACED IN ONE CONTINUOUS POURING OPERATION BETWEEN SPECIFIED CONSTRUCTION JOINTS.

C12 REINFORCEMENT SHALL BE SUPPORTED ON APPROVED PLASTIC STOOLS OR MORTAR BLOCKS OF EQUAL STRENGTH AND DURABILITY TO THE CONCRETE MIX, AT NOT MORE THAN 800mm CENTRES.

C13 CURE CONCRETE (WATER TO BE POTABLE) FOR 7 DAYS AFTER FINAL POUR. WATER CURE WITH PLASTIC MEMBRANE, FLOODED. APPROVED SPRAYED-ON CURING COMPOUNDS WHICH COMPLY WITH AS 3799 MAY BE USED WHERE EXPOSED FINISHES WILL NOT BE AFFECTED (REFER MANUFACTURERS SPECIFICATION).

C14 CONCRETE SHALL NOT BE PLACED IN THE WORKS IF THE TEMPERATURE OF THE SURROUNDING AIR FALLS BELOW 5 DEGREES(°) CELSIUS(C), OR IS HIGHER THAN 32°C OR WIND SPEEDS EXCEED 25km/h.

C15 REINFORCING SYMBOLS:

N	GRADE D500N, HOT ROLLED DEFORMED BAR, COMPLYING WITH AS4671.
R	GRADE R250N, HOT ROLLED PLAIN ROUND BAR, COMPLYING WITH AS4671.
SL/RL	GRADE 500L, SQUARE/RECTANGULAR MESH, COMPLYING WITH AS4671.
W	GRADE R500L, COLD DRAWN ROUND WIRE, COMPLYING WITH AS4671.

REINFORCING ABBREVIATIONS:

EF	EACH FACE	B	BOTTOM
NF	NEAR FACE	HORIZ	HORIZONTAL
FF	FAR FACE	VERT	VERTICAL
T	TOP	EW	EACH WAY

17-N20-250 EW
INFORMATION FOR PLACING
BAR SPACING (mm)
BAR SIZE (mm)
BAR STRUCTURAL PROPERTIES
NUMBER OF BARS IN SET

C16 THE CONTRACTOR SHALL ARRANGE FOR THE ENGINEER TO INSPECT THE REINFORCEMENT AND OBTAIN APPROVAL PRIOR TO POURING CONCRETE.

C17 FORMWORK DESIGN, CONSTRUCTION, TOLERANCES AND STRIPPING TIMES SHALL COMPLY WITH AS 3600 AND AS 3610 UNLESS OTHERWISE APPROVED BY THE ENGINEER. CONCRETE FORMED SURFACES TO HAVE FINISHES IN ACCORDANCE WITH AS 3610 AS SPECIFIED BY THE ARCHITECT.

C18 REFER TO ARCHITECTURAL DRAWINGS FOR DETAILS OF CHAMFERS AND REBATES. MAINTAIN COVER TO REINFORCEMENT AT THESE LOCATIONS.

C19 REINFORCEMENT IS REPRESENTED DIAGRAMATICALLY AND NOT NECESSARILY IN TRUE PROJECTION.

BW1 RETAINING WALL BORED PIER SCHEDULE

WALL HEIGHT (m)	PIER DIAMETER (mm)	PIER DEPTH (m)	UNDERMINING ALLOWANCE (m)	REINFORCEMENT VERTICAL	REINFORCEMENT LIGATURES
0.4	450	2.0	0.6	6-N20	N10-200
1.0	450	2.5	0.6	6-N20	N10-200
1.6	450	3.1	0.6	6-N20	N10-200

NOTE: PIER SPACING = 2.00m

BW2 RETAINING WALL BORED PIER SCHEDULE

WALL HEIGHT (m)	PIER DIAMETER (mm)	PIER DEPTH (m)	UNDERMINING ALLOWANCE (m)	REINFORCEMENT VERTICAL	REINFORCEMENT LIGATURES
0.9	450	1.6	0.0	6-N20	N10-200

NOTE: PIER SPACING = 2.00m

SW1 RETAINING WALL SCHEDULE

WALL HEIGHT (m)	PIER DIAMETER (mm)	PIER DEPTH (m)	UNDERMINING ALLOWANCE (m)	STEEL POST INTERMEDIATE	STEEL POST CORNER
0.20	450	0.8	0	150UB14	150 PFC
0.40	450	0.9	0	150UB14	150 PFC
0.60	450	1.1	0	150UB14	150 PFC
0.80	450	1.4	0	150UB14	150 PFC
1.00	450	1.6	0	150UB14	150 PFC
1.20	450	1.7	0	150UB14	150 PFC
1.40	450	1.9	0	150UB14	150 PFC
1.60	450	2.1	0	180UB16	150 PFC
1.80	450	2.3	0	200UB18	150 PFC
2.00	450	2.5	0	200UB22	150 PFC
2.20	450	2.8	0	200UB25	150 PFC
2.40	600	3.0	0	250UB31	180 PFC

NOTE: POST/PIER SPACING = 2.43m

SW2 RETAINING WALL SCHEDULE

WALL HEIGHT (m)	PIER DIAMETER (mm)	PIER DEPTH (m)	UNDERMINING ALLOWANCE (m)	STEEL POST INTERMEDIATE	STEEL POST CORNER
0.20	450	1.5	0.6	150UB14	150 PFC
0.40	450	1.7	0.6	150UB14	150 PFC
0.60	450	1.9	0.6	150UB14	150 PFC
0.80	450	2.1	0.6	150UB14	150 PFC
1.00	450	2.3	0.6	180UB16	150 PFC
1.20	450	2.5	0.6	200UB18	150 PFC
1.40	450	2.7	0.6	200UB22	150 PFC
1.60	450	2.9	0.6	200UB22	150 PFC
1.80	600	3.1	0.6	200UB31	180 PFC
2.00	600	3.3	0.6	250UB31	200 PFC

NOTE: POST/PIER SPACING = 2.43m

SW3 RETAINING WALL SCHEDULE

WALL HEIGHT (m)	PIER DIAMETER (mm)	PIER DEPTH (m)	UNDERMINING ALLOWANCE (m)	STEEL POST INTERMEDIATE	STEEL POST CORNER
0.20	450	1.5	0.6	150UB14	150 PFC
0.40	450	1.7	0.6	150UB14	150 PFC
0.60	450	1.9	0.6	150UB14	150 PFC
0.80	450	2.1	0.6	150UB14	150 PFC
1.00	450	2.3	0.6	180UB16	150 PFC
1.20	450	2.5	0.6	200UB18	150 PFC
1.40	450	2.7	0.6	200UB22	150 PFC
1.60	450	2.9	0.6	200UB22	150 PFC
1.80	600	3.1	0.6	250UB31	180 PFC
2.00	600	3.3	0.6	250UB31	200 PFC
2.20	600	3.5	0.6	310UB40	250 PFC
2.40	600	3.7	0.6	310UB40	250 PFC
2.60	600	4.0	0.6	360UB51	250 PFC
2.80	600	4.2	0.6	410UB54	300 PFC

NOTE: POST/PIER SPACING = 2.43m

SW4 RETAINING WALL SCHEDULE

WALL HEIGHT (m)	PIER DIAMETER (mm)	PIER DEPTH (m)	UNDERMINING ALLOWANCE (m)	STEEL POST INTERMEDIATE	STEEL POST CORNER
0.20	450	0.8	0	150UB14	150 PFC
0.40	450	0.9	0	150UB14	150 PFC
0.60	450	1.1	0	150UB14	150 PFC
0.80	450	1.4	0	150UB14	150 PFC
1.00	450	1.6	0	150UB14	150 PFC
1.20	450	1.7	0	150UB14	150 PFC
1.40	450	1.9	0	150UB14	150 PFC
1.60	450	2.1	0	180UB16	150 PFC
1.80	450	2.3	0	200UB18	150 PFC
2.00	450	2.5	0	200UB22	150 PFC

NOTE: POST/PIER SPACING = 2.43m

BW1 RETAINING WALL STRIP FOOTING SCHEDULE

WALL HEIGHT (m)	FOOTING SIZE (mm)	PRIMARY REINFORCMENT	REINFORCEMENT LIGATURES
0.4	450Dx500W	8-N12	N8-100
1.0	450Dx500W	8-N12	N8-100
1.6	450Dx500W	8-N12	N8-100

BW2 RETAINING WALL STRIP FOOTING SCHEDULE

WALL HEIGHT (m)	FOOTING SIZE (mm)	PRIMARY REINFORCMENT	REINFORCEMENT LIGATURES
0.9	450Dx500W	8-N12	N8-100

ISSUED FOR APPROVAL
NOT FOR CONSTRUCTION

ISSUE	AMENDMENTS	INT./DATE
J	AMENDMENTS	TH 20.09.24
H	REVISION CLOUDS REMOVED	TH 11.09.24
G	AMENDMENTS CLOUDED	TH 06.09.24
F	NO CHANGES	AW 29.08.24
E	AMENDMENTS	TH 26.08.24
D	AMENDMENTS CLOUDED	AW 30.07.24
C	AMENDMENTS CLOUDED	AW 18.06.24
B	AMENDMENTS CLOUDED	AW 04.06.24
A	FOR APPROVAL	RP 24.05.24

MAGRYN
ENGINEERING CONSULTANTS
267 BRIGHTON ROAD
SOMERTON PARK, SA 5044
TELEPHONE: (08) 8295 8677
www.magryn.com.au
> MINING
> STRUCTURAL
> COASTAL
> CIVIL

CLIENT:
OAKDEN DEVELOPMENTS

PROJECT:
RETAINING WALL DESIGN

PROJECT ADDRESS:
OAKDEN DEVELOPMENT STAGE 2, GRAND JUNCTION ROAD, OAKDEN

TITLE:
NOTES

CONTRACTORS MUST VERIFY ALL DIMENSIONS PRIOR TO ANY OFF SITE FABRICATION.

DESIGN: RP	SCALE: AS SHOWN	DATE: MAY 2024
SHEET SIZE: A1	DRAWING NUMBER: 24167-N1	REVISION: J

SITE PLAN
SCALE 1:500

REFER TO GREENHILL DRAWINGS 22-3021-C004, 22-3021-C048 & 22-3021-C049 FOR WALL HEIGHTS.

NOTE: THESE DRAWINGS ARE IN COLOUR. TO READ THE INFORMATION CONTAINED IN THE DRAWING SET IT MUST BE PRINTED IN COLOUR.

NOTE: BOUNDARY LOCATIONS SHOWN INDICATIVELY ONLY. TO BE CONFIRMED BY A LICENSED SURVEYOR.

DESIGN LOADS:
- 5.0kPa LIVE SURCHARGE
- 1.8m HIGH SOLID FENCE OVER (EXCEPT FOR BW2 WHICH HAS NO ALLOWANCE FOR SOLID FENCE).
- NO ALLOWANCE FOR LOADS FROM ADJACENT STRUCTURES.

CITY OF PORT ADELAIDE ENFIELD
Planning Consent GRANTED
DA 24031085
04/10/24
Planning Development & Infrastructure Act 2016

ISSUE	AMENDMENTS	INT./DATE
J	NO CHANGES	TH 20.09.24
H	REVISION CLOUDS REMOVED	TH 11.09.24
G	NO CHANGES	TH 06.09.24
F	NO CHANGES	AW 29.08.24
E	AMENDMENTS CLOUDED	TH 26.08.24
D	AMENDMENTS CLOUDED	AW 30.07.24
C	AMENDMENTS CLOUDED	AW 18.06.24
B	AMENDMENTS CLOUDED	AW 04.06.24
A	FOR APPROVAL	RP 24.05.24

LEGEND

(red line)	SW1. PROPOSED CONCRETE SLEEPER RETAINING WALL. REFER DWG. 24167-2 FOR DETAILS, REFER DWG. 24167-N1 FOR RETAINING WALL NOTES AND SCHEDULES.	**Fencing Type: 1.8M Good Neighbour Fencing. Colour: Dune**
(blue line)	SW2. PROPOSED CONCRETE SLEEPER RETAINING WALL. REFER DWG. 24167-2 FOR DETAILS, REFER DWG. 24167-N1 FOR RETAINING WALL NOTES AND SCHEDULES.	**Fencing Type: 1.8M Good Neighbour Fencing. Colour: Dune**
(yellow line)	SW3. PROPOSED CONCRETE SLEEPER RETAINING WALL. REFER DWG. 24167-2 FOR DETAILS, REFER DWG. 24167-N1 FOR RETAINING WALL NOTES AND SCHEDULES.	**Fencing Type: 1.8M Good Neighbour Fencing. Colour: Dune**
(orange line)	SW4. PROPOSED CONCRETE SLEEPER RETAINING WALL. REFER DWG. 24167-2 FOR DETAILS, REFER DWG. 24167-N1 FOR RETAINING WALL NOTES AND SCHEDULES.	
(purple line)	BW1. PROPOSED BLOCKWORK RETAINING WALL. REFER DWG. 24167-3 FOR DETAILS, REFER DWG. 24167-N1 FOR RETAINING WALL NOTES AND SCHEDULES.	
(green line)	BW2. PROPOSED BLOCKWORK RETAINING WALL. REFER DWG. 24167-3 FOR DETAILS, REFER DWG. 24167-N1 FOR RETAINING WALL NOTES AND SCHEDULES.	
- - -	APPROXIMATE BOUNDARY LOCATION.	

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ENGINEERING CONSULTANTS

267 BRIGHTON ROAD
SOMERTON PARK, SA 5044
TELEPHONE: (08) 8295 8677
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CLIENT:
OAKDEN DEVELOPMENTS

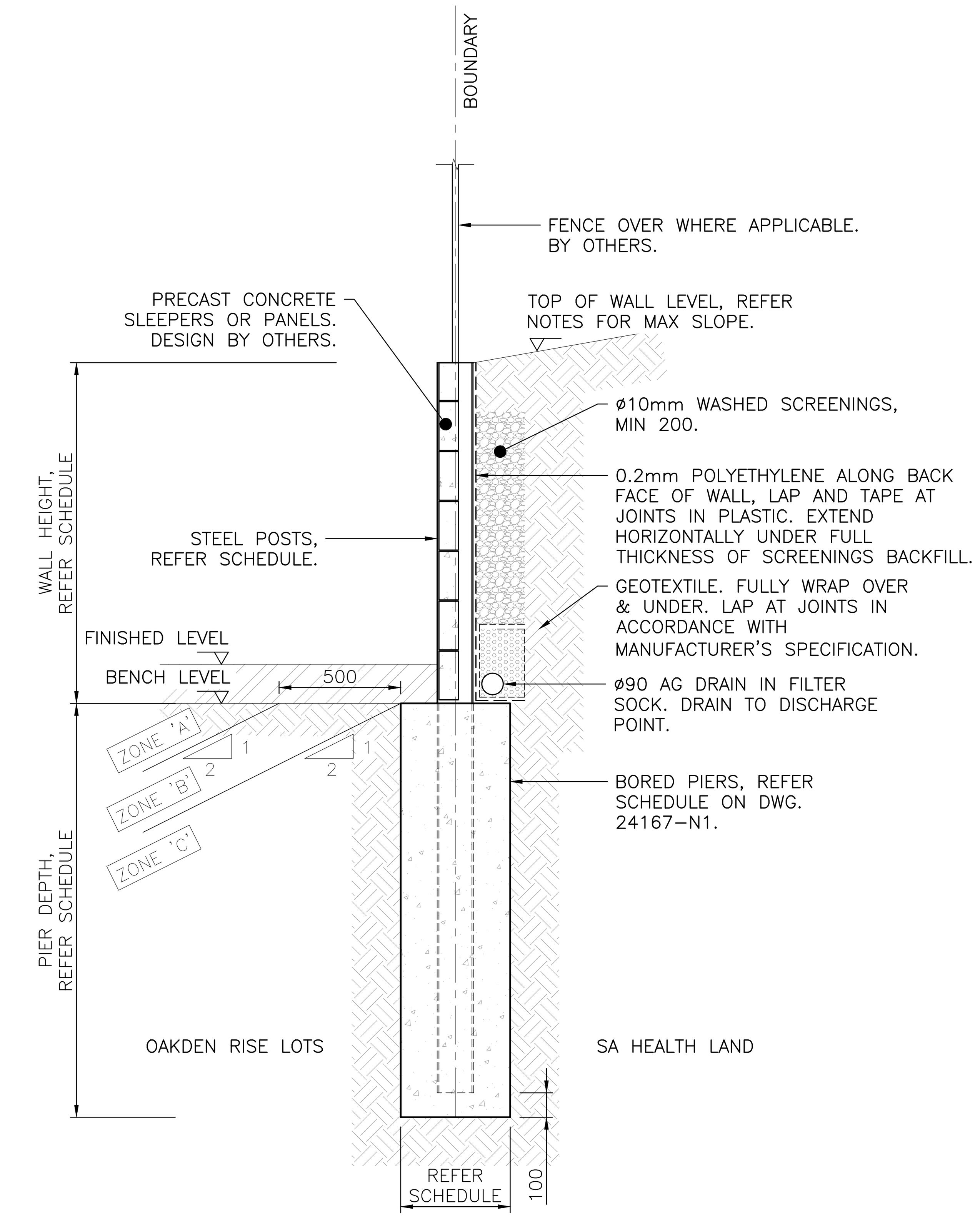
PROJECT:
RETAINING WALL DESIGN

PROJECT ADDRESS:
OAKDEN DEVELOPMENT STAGE 2, GRAND JUNCTION ROAD, OAKDEN

TITLE:
SITE PLAN

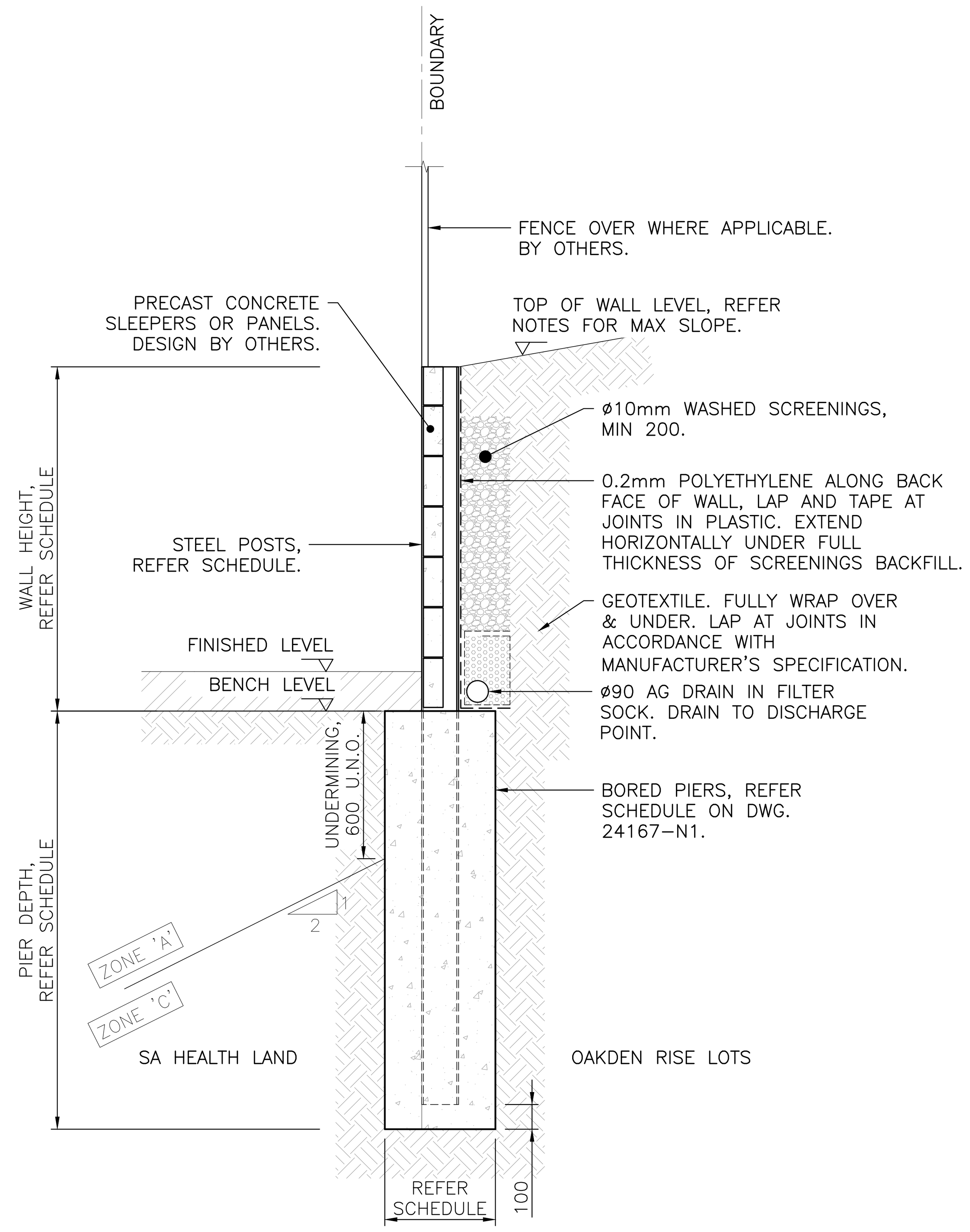
CONTRACTORS MUST VERIFY ALL DIMENSIONS PRIOR TO ANY OFF SITE FABRICATION.

DESIGN: RP	SCALE: AS SHOWN	DATE: MAY 2024
SHEET SIZE: A1	DRAWING NUMBER: 24167-1	REVISION: J



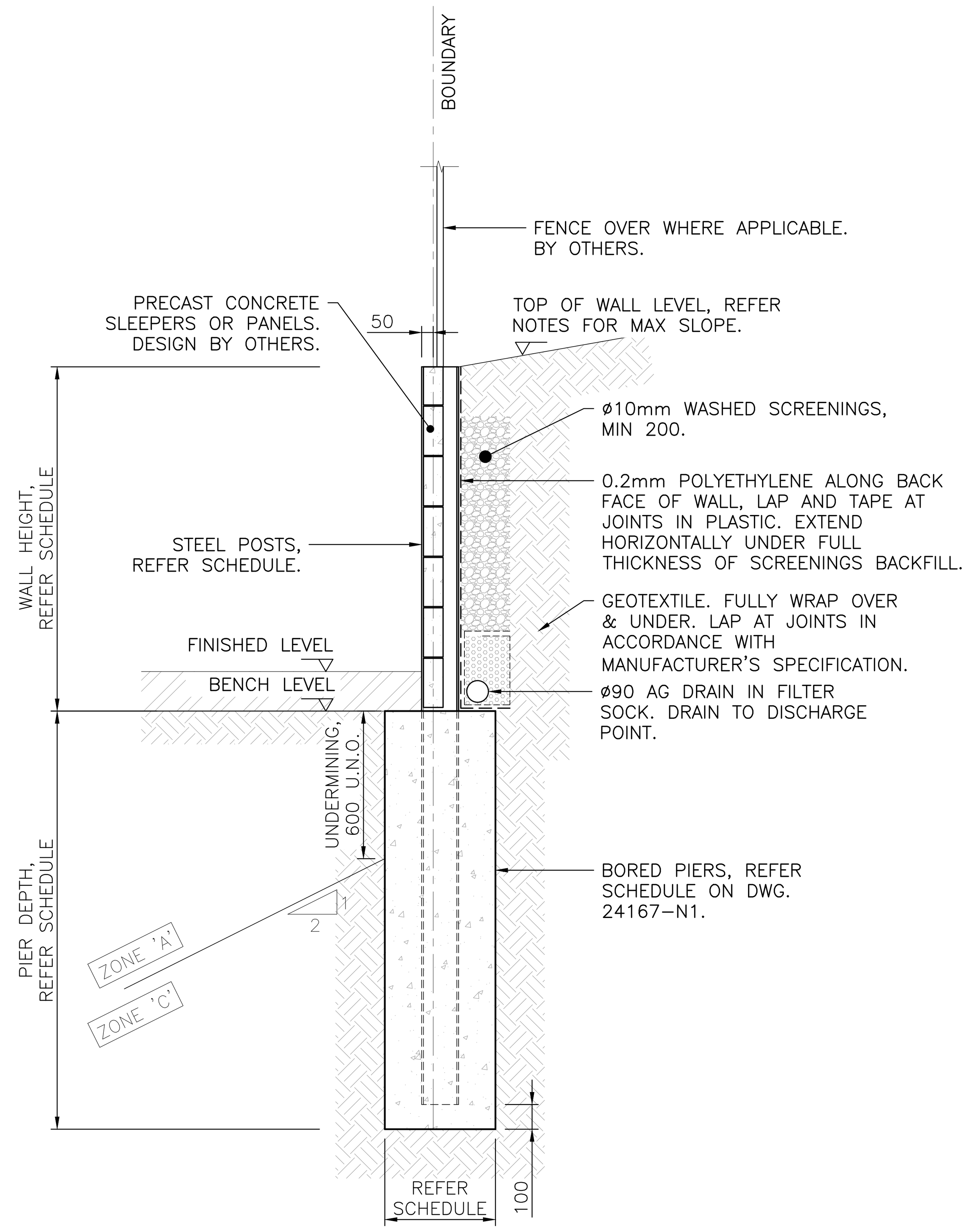
SW1 CONCRETE SLEEPER RETAINING WALL, TYPICAL SECTION

SCALE 1:20



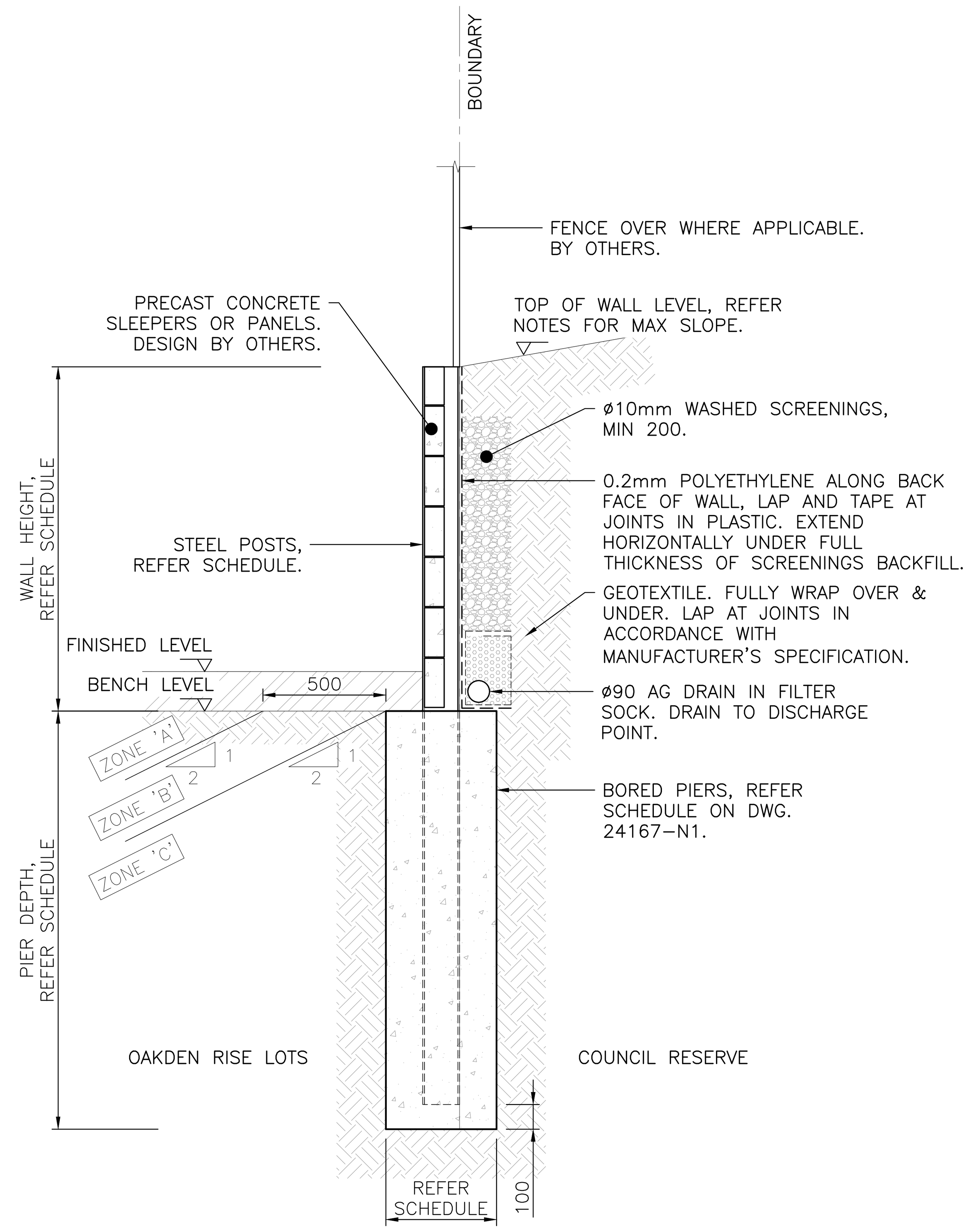
SW2 CONCRETE SLEEPER RETAINING WALL, TYPICAL SECTION

SCALE 1:20



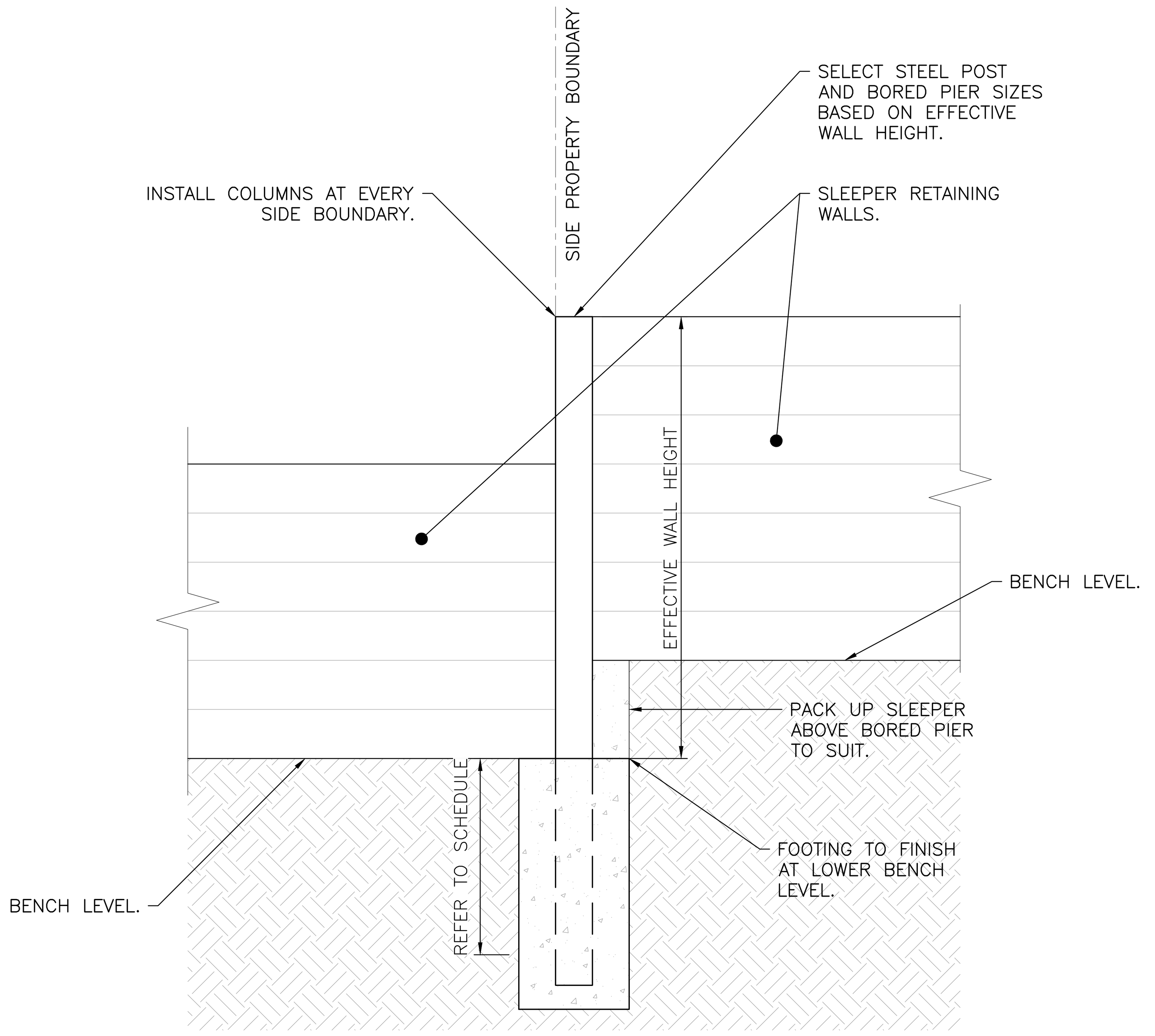
SW3 CONCRETE SLEEPER RETAINING WALL, TYPICAL SECTION

SCALE 1:20



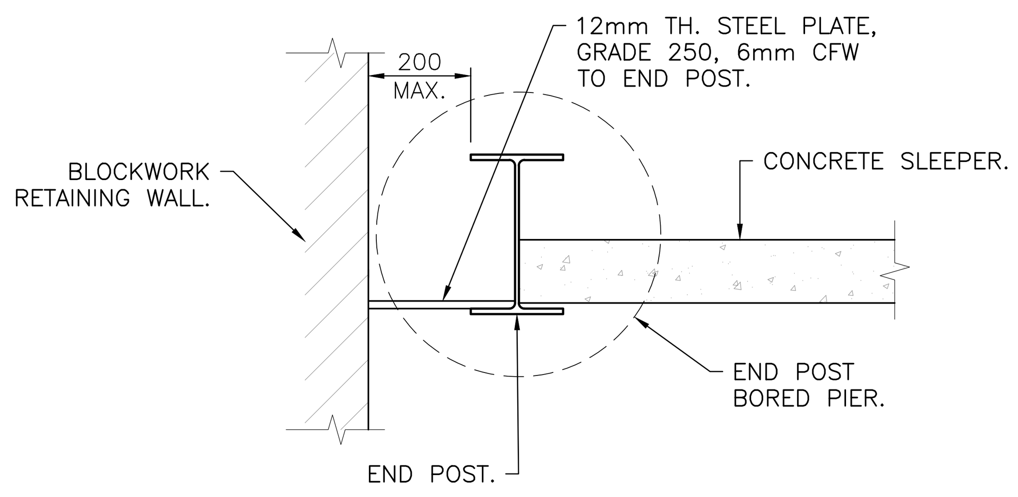
SW4 CONCRETE SLEEPER RETAINING WALL, TYPICAL SECTION

SCALE 1:20



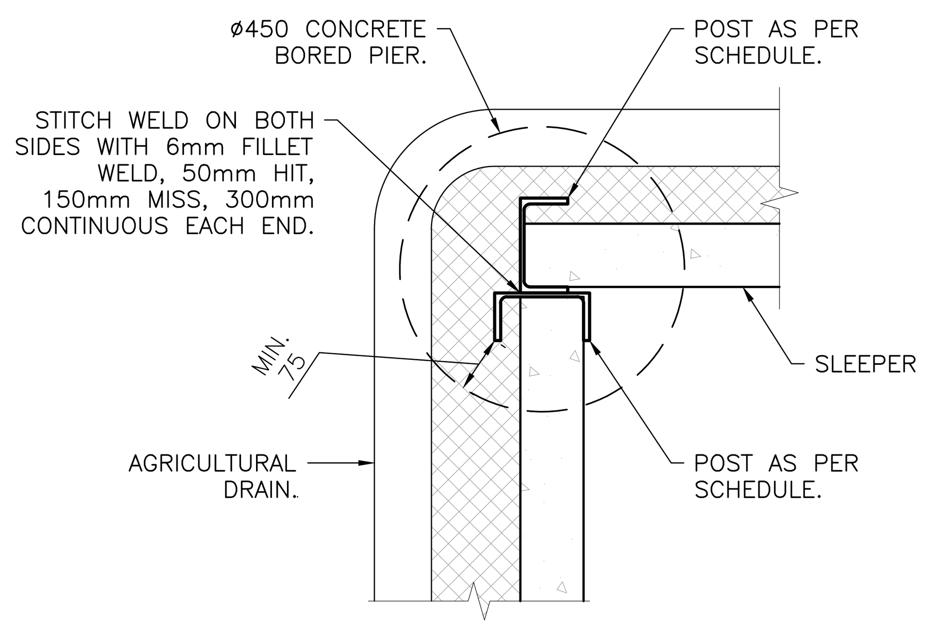
RETAINING WALL ELEVATION AT SIDE BOUNDARY (RWA)

N.T.S.



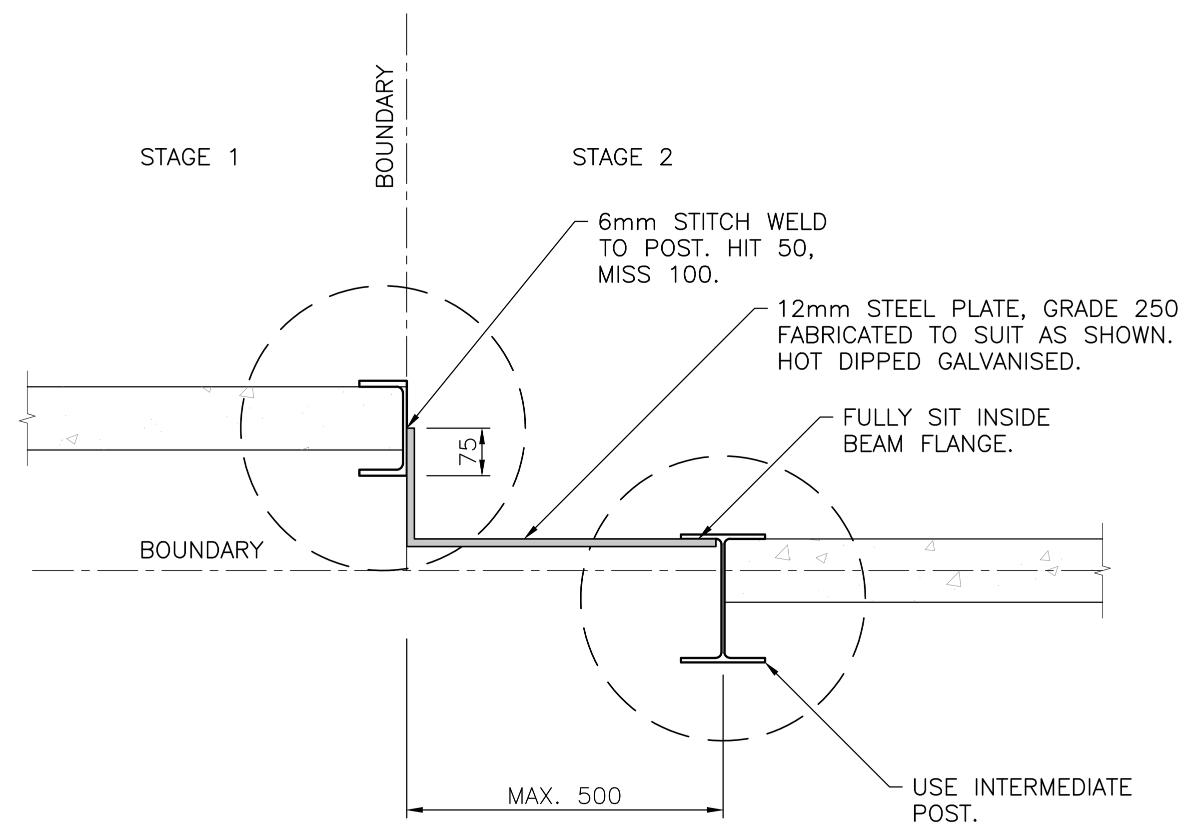
TYPICAL SLEEPER WALL TO BLOCKWORK WALL JUNCTION

SCALE 1:10



TYPICAL CORNER DETAIL

SCALE 1:10



TYPICAL EXISTING TO PROPOSED RETAINING WALL CONNECTION DETAIL BETWEEN LOT 211 & LOT 212

SCALE 1:10

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ISSUE	AMENDMENTS	INT./DATE
J	NO CHANGES	TH 20.09.24
H	REVISION CLOUDS REMOVED	TH 11.09.24
G	AMENDMENTS CLOUDED	TH 06.09.24
F	AMENDMENTS CLOUDED	AW 29.08.24
E	AMENDMENTS CLOUDED	TH 26.08.24
D	AMENDMENTS	AW 30.07.24
C	AMENDMENTS CLOUDED	AW 18.06.24
B	AMENDMENTS CLOUDED	AW 04.06.24
A	FOR APPROVAL	RP 24.05.24

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SOMERTON PARK, SA 5044
TELEPHONE: (08) 8295 8677
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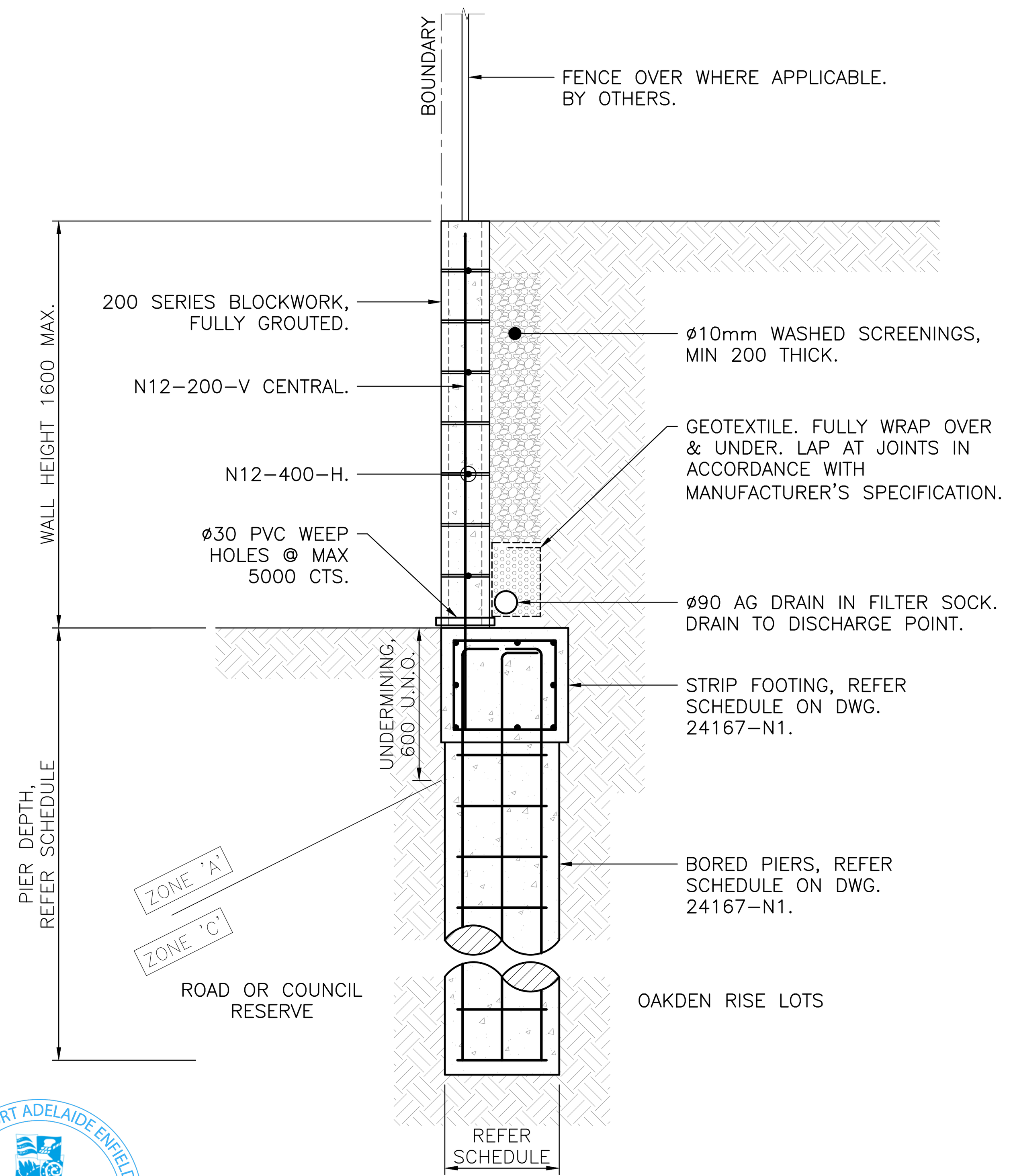
PROJECT:
RETAINING WALL DESIGN

PROJECT ADDRESS:
OAKDEN DEVELOPMENT STAGE 2, GRAND JUNCTION ROAD, OAKDEN

TITLE:
CONCRETE SLEEPER RETAINING WALL DETAILS

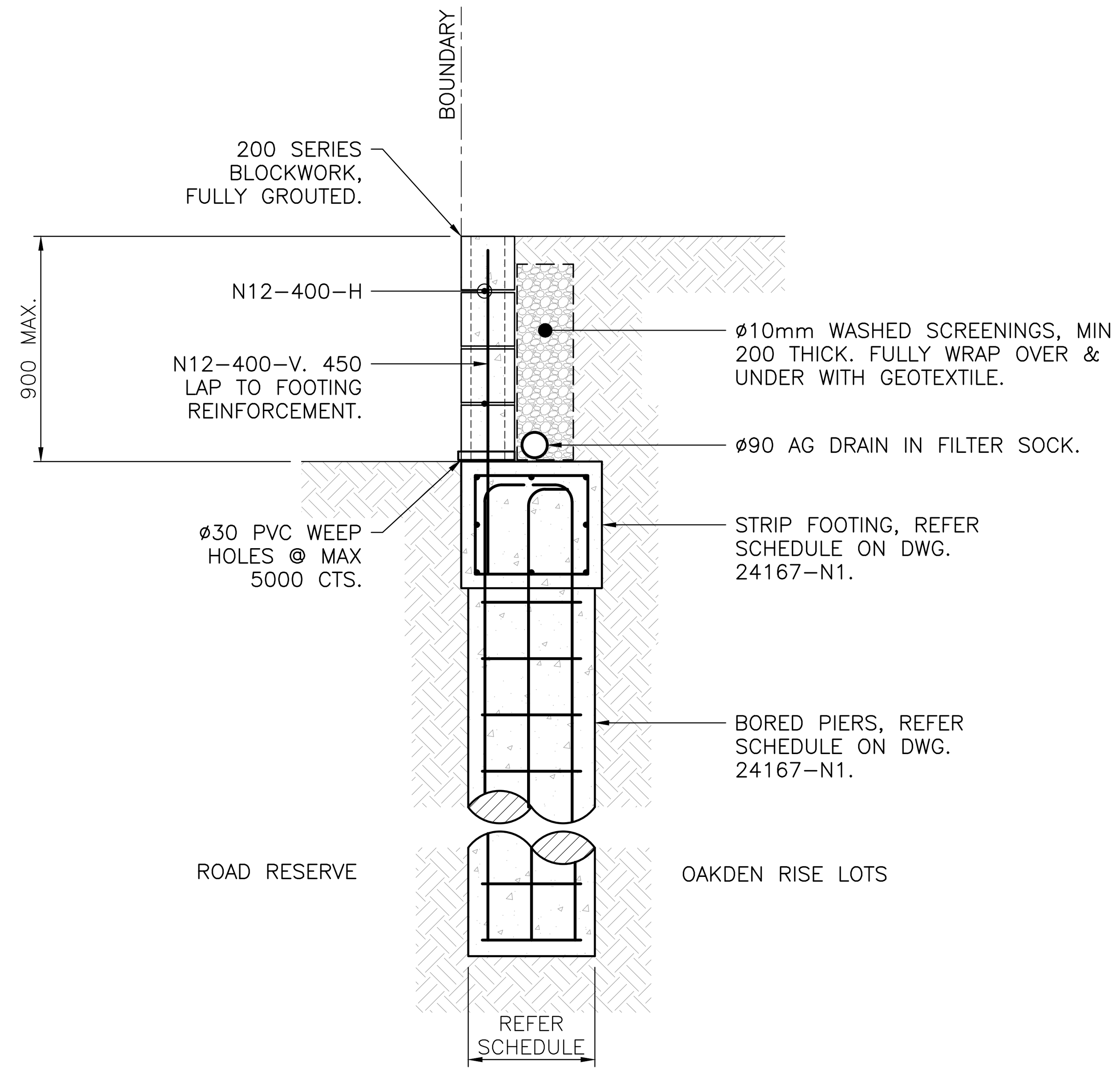
CONTRACTORS MUST VERIFY ALL DIMENSIONS PRIOR TO ANY OFF SITE FABRICATION.

DESIGN: RP	SCALE: AS SHOWN	DATE: MAY 2024
SHEET SIZE: A1	DRAWING NUMBER: 24167-2	REVISION: J



BW1 BLOCKWORK RETAINING WALL, (<1600 HIGH)

SCALE 1:20



BW2 BLOCKWORK RETAINING WALL, TYPICAL SECTION

SCALE 1:20

ISSUED FOR APPROVAL
NOT FOR CONSTRUCTION

ISSUE	AMENDMENTS	INT./DATE
J	NO CHANGES	TH 20.09.24
H	REVISION CLOUDS REMOVED	TH 11.09.24
G	AMENDMENTS CLOUDED	TH 06.09.24
F	NO CHANGES	AW 29.08.24
E	AMENDMENTS CLOUDED	TH 26.08.24
D	ISSUED FOR APPROVAL	AW 30.07.24
C	NOT ISSUED	AW 18.06.24
B	NOT ISSUED	AW 04.06.24
A	NOT ISSUED	RP 24.05.24

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267 BRIGHTON ROAD
SOMERTON PARK, SA 5044
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RETAINING WALL DESIGN

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OAKDEN DEVELOPMENT STAGE 2, GRAND JUNCTION ROAD, OAKDEN

TITLE:
BLOCKWORK RETAINING WALL DETAILS

CONTRACTORS MUST VERIFY ALL DIMENSIONS PRIOR TO ANY OFF SITE FABRICATION.

DESIGN: RP	SCALE: AS SHOWN	DATE: MAY 2024
SHEET SIZE: A1	DRAWING NUMBER: 24167–3	REVISION: J

STRUCTURAL CALCULATIONS

FOR

RETAINING WALL DESIGNS

FOR

OAKDEN DEVELOPMENTS PTY LTD

CITY OF PORT ADELAIDE ENFIELD
Planning Consent
GRANTED
DA 24031085
04/10/24
Planning Development & Infrastructure Act 2016

Project No. 24167
September 2024

Doc. No.	Revision	Issue Date	Author	Checked
SC24167-1	A	24/05/2024	RP	WS
SC24167-1	B	3/06/2024	RP	RP
SC24167-1	C	25/09/2024	BJ	RP

ENGINEERING CONSULTANTS
267 BRIGHTON ROAD
SOMERTON PARK SA 5044
TELEPHONE +61 (0)8 8295 8677
FACSIMILE +61 (0)8 8295 5104
admin@magryn.com.au
www.magryn.com.au
ABN 64 090 560 099

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RETAINING WALL CALCULATIONS - STEEL POST & BORED PIER

RETAINING WALL MARK SW1

Common Geometry

Calculation increment		0.2	m
Height range, lower (multiple of increment)		0.2	m
Height range, upper (multiple of increment)		2.4	m
Ineffective soil depth (undermining)		0.00	m
Post Spacing	a	2.43	m
Backfill slope	β	10	o
		0.175	rad
Bored pier diameter, primary	B	0.45	m
Bored pier diameter, larger alternative		0.6	m
Wall height threshold for pier size		2.4	

Additional Loads

Surcharge pressure, dead	q_G	0	kPa
Surcharge pressure, live	q_Q	5	kPa
Hydrostatic pressure		0	kN/m^3
Fence height	h_f	1.8	m
Site wind speed, ultimate	V_{sit}	37.4	m/s
Site wind pressure	q_z	0.839	kPa
Aerodynamic shape factor C,pn on fence	$C_{p,n}$	1.2	
Design wind pressure	p_w	1.01	kPa
Balustrade handrail height		0.00	m
Balustrade, top edge UDL		0.00	kN/m
Balustrade, top edge PL		0.00	kN
Balustrade, infill UDL		0.00	kPa
Include balustrade forces for H < 1.0m?		No	

Parameters

Founding material		Cohesive (i.e. clay)	
Soil bearing capacity (Rutledge method)	SS	100	kPa
Soil undrained shear strength (Broms Method)	Su	50	kPa
Soil unit weight	g_{soil}	18	kN/m^3
Soil characteristic internal friction angle	f	30	o
		0.524	rad
Soil backfill uncertainty level		Uncontrolled fill	
	F_{uf}	0.75	
Design Internal Friction Angle	f*	0.41	rad
External friction angle	d	0.175	rad
Wedge Angle	J	0.990	rad
Active Pressure Coefficient	K_{ah}	0.460	
Active Pressure Coefficient, Surcharge	$K_{a,q}$	0.589	

Factors of Safety

Earth Thrust FOS	1.25
Surcharge, dead FOS	1.25
Surcharge, live FOS	1.50
Surcharge Yc	0.60
Hydrostatic FOS	1.00
Wind FOS	1.00

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RETAINING WALL MARK: SW1

Wall Height							
Level difference h =	m	0.20	0.40	0.60	0.80	1.00	1.20

Working Forces and Moments							
Additional lever arm (ineffective depth)	m	0.00	0.00	0.00	0.00	0.00	0.00
Active Thrust P_a = 0.5 * Ka * gsoil * h² * a	kN	0.40	1.61	3.62	6.44	10.07	14.50
M_{pa} = Pa * h/3	kNm	0.03	0.21	0.72	1.72	3.36	5.80
Dead Surcharge Force P_{qG} = q_G * Ka * h * a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{pqG} = Pq_G * h/2	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	1.43	2.86	4.29	5.72	7.15	8.58
M_{pqQ} = Pq_Q * h/2	kNm	0.14	0.57	1.29	2.29	3.58	5.15
Balustrade Force P_b = Max(p*a, p, p*a*h_b)	kNm	0.00	0.00	0.00	0.00	0.00	0.00
M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)	kN	0.00	0.00	0.00	0.00	0.00	0.00
Hydrostatic Force P_h = gwater * h²/2 *a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{ph} = ph * h/3	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Wind force P_w = pw * hf * a	kN	4.41	4.41	4.41	4.41	4.41	4.41
M_w = Pw * (h + hf/2)	kNm	4.85	5.73	6.61	7.49	8.37	9.25

Ultimate Moment at Top of Bored Pier							
1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	0.25	1.13	2.84	5.58	9.56	14.97
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	0.03	0.27	0.91	2.15	4.19	7.25
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	4.96	6.34	8.29	11.01	14.71	19.59
max M_d	**kNm**	**4.96**	**6.34**	**8.29**	**11.01**	**14.71**	**19.59**

Ultimate Shear Force at Top of Bored Pier							
1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	2.65	6.30	10.97	16.64	23.31	30.99
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	0.50	2.01	4.53	8.05	12.58	18.12
3) 1.25(Pa+Pq_G)+ Pw + Yc Pq_Q + 1.0 Ph	kN	5.77	8.13	11.51	15.89	21.28	27.68
max P_d	**kN**	**5.77**	**8.13**	**11.51**	**16.64**	**23.31**	**30.99**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))
2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5
3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		1	1	1	1	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**
Required Pier Depth, D =	**m**	**0.75**	**0.90**	**1.10**	**1.40**	**1.58**	**1.75**

Steel Design							
Design moment, M* =	kNm	4.96	6.34	8.29	11.01	14.71	19.59
Post design length, l =	m	0.20	0.40	0.60	0.80	1.00	1.20
Post effective length factor, kl =		2.00	2.00	2.00	2.00	2.00	2.00
Post moment modification factor =		2.25	2.25	2.25	2.25	2.25	2.25
Intermediate post =		**150 UB 14**	**150 UB 14**	**150 UB 14**	**150 UB 14**	**150 UB 14**	**150 UB 14**
Corner/end post =		**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**

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RETAINING WALL MARK: SW1

Wall Height							
Level difference h =	m	1.40	1.60	1.80	2.00	2.20	2.40

Working Forces and Moments							
Additional lever arm (ineffective depth)	m	0.00	0.00	0.00	0.00	0.00	0.00
Active Thrust P_a = 0.5 * Ka * gsoil * h^2 * a	kN	19.73	25.77	32.62	40.27	48.73	57.99
M_{pa} = Pa * h/3	kNm	9.21	13.75	19.57	26.85	35.73	46.39
Dead Surcharge Force P_{qG} = q_G * Ka * h * a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{pqG} = Pq_G * h/2	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	10.01	11.44	12.87	14.30	15.73	17.16
M_{pqQ} = Pq_Q * h/2	kNm	7.01	9.15	11.58	14.30	17.31	20.59
Balustrade Force P_b = Max(p*a, p, p*a*h_b)	kNm	0.00	0.00	0.00	0.00	0.00	0.00
M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)	kN	0.00	0.00	0.00	0.00	0.00	0.00
Hydrostatic Force P_h = gwater * h^2/2 *a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{ph} = ph * h/3	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Wind force P_w = pw * hf * a	kN	4.41	4.41	4.41	4.41	4.41	4.41
M_w = Pw * (h + hf/2)	kNm	10.13	11.01	11.89	12.77	13.66	14.54

Ultimate Moment at Top of Bored Pier							
1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	22.02	30.91	41.84	55.01	70.62	88.88
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	11.51	17.18	24.46	33.56	44.67	57.99
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	25.85	33.69	43.31	54.91	68.71	84.88
max M_d	**kNm**	**25.85**	33.69	43.31	55.01	70.62	88.88

Ultimate Shear Force at Top of Bored Pier							
1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	39.68	49.38	60.08	71.79	84.51	98.23
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	24.67	32.22	40.77	50.34	60.91	72.49
3) 1.25(Pa+Pq_G)+Pw + Yc Pq_Q + 1.0 Ph	kN	35.08	43.49	52.90	63.32	74.75	87.19
max P_d	**kN**	**39.68**	49.38	**60.08**	**71.79**	**84.51**	**98.23**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))
2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5"
3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		2	2	2	2	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**	**0.60**
Required Pier Depth, D =	**m**	**1.93**	**2.12**	**2.33**	**2.54**	**2.79**	**2.96**

Steel Design							
Design moment, M* =	kNm	25.85	33.69	43.31	55.01	70.62	88.88
Post design length, l =	m	1.40	1.60	1.80	2.00	2.20	2.40
Post effective length factor, kl =		2.00	2.00	2.00	2.00	2.00	2.00
Post moment modification factor =		2.25	2.25	2.25	2.25	2.25	2.25
Intermediate post =		**150 UB 14**	**180 UB 16**	**200 UB 18**	**200 UB 22**	**200 UB 25**	**250 UB 31**
Corner/end post =		**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**180 PFC**

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RETAINING WALL CALCULATIONS - STEEL POST & BORED PIER
RETAINING WALL MARK SW2

Common Geometry

Calculation increment		0.2	m
Height range, lower (multiple of increment)		0.2	m
Height range, upper (multiple of increment)		2	m
Ineffective soil depth (undermining)		0.60	m
Post Spacing	a	2.43	m
Backfill slope	β	0	o
		0.000	rad
Bored pier diameter, primary	B	0.45	m
Bored pier diameter, larger alternative		0.6	m
Wall height threshold for pier size		1.8	

Additional Loads

Surcharge pressure, dead	q_G	0	kPa
Surcharge pressure, live	q_Q	5	kPa
Hydrostatic pressure		0	kN/m³
Fence height	h_f	1.8	m
Site wind speed, ultimate	V_{sit}	37.4	m/s
Site wind pressure	q_z	0.839	kPa
Aerodynamic shape factor C,pn on fence	$C_{p,n}$	1.2	
Design wind pressure	p_w	1.01	kPa
Balustrade handrail height		0.00	m
Balustrade, top edge UDL		0.00	kN/m
Balustrade, top edge PL		0.00	kN
Balustrade, infill UDL		0.00	kPa
Include balustrade forces for H < 1.0m?		No	

Parameters

Founding material		Cohesive (i.e. clay)	
Soil bearing capacity (Rutledge method)	SS	100	kPa
Soil undrained shear strength (Broms Method)	Su	50	kPa
Soil unit weight	g_{soil}	18	kN/m³
Soil characteristic internal friction angle	f	30	o
		0.524	rad
Soil backfill uncertainty level		Uncontrolled fill	
	F_{uf}	0.75	
Design Internal Friction Angle	f*	0.41	rad
External friction angle	d	0.175	rad
Wedge Angle	J	0.990	rad
Active Pressure Coefficient	K_{ah}	0.431	
Active Pressure Coefficient, Surcharge	$K_{a,q}$	0.589	

Factors of Safety

Earth Thrust FOS	1.25
Surcharge, dead FOS	1.25
Surcharge, live FOS	1.50
Surcharge Yc	0.60
Hydrostatic FOS	1.00
Wind FOS	1.00

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RETAINING WALL MARK: SW2

Wall Height							
Level difference h =	m	0.20	0.40	0.60	0.80	1.00	1.20

Working Forces and Moments							
Additional lever arm (ineffective depth)	m	0.60	0.60	0.60	0.60	0.60	0.60
Active Thrust P_a = 0.5 * Ka * gsoil * h² * a	kN	0.38	1.51	3.40	6.04	9.43	13.58
M_{pa} = Pa * h/3	kNm	0.25	1.11	2.72	5.23	8.80	13.58
Dead Surcharge Force P_{qG} = q_G * Ka * h * a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{pqG} = Pq_G * h/2	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	1.43	2.86	4.29	5.72	7.15	8.58
M_{pqQ} = Pq_Q * h/2	kNm	1.00	2.29	3.86	5.72	7.87	10.30
Balustrade Force P_b = Max(p*a, p, p*a*h_b)	kNm	0.00	0.00	0.00	0.00	0.00	0.00
M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)	kN	0.00	0.00	0.00	0.00	0.00	0.00
Hydrostatic Force P_h = gwater * h²/2 *a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{ph} = ph * h/3	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Wind force P_w = pw * hf * a	kN	4.41	4.41	4.41	4.41	4.41	4.41
M_w = Pw * (h + hf/2)	kNm	7.49	8.37	9.25	10.13	11.01	11.89

Ultimate Moment at Top of Bored Pier							
1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	1.82	4.82	9.19	15.12	22.80	32.42
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	0.31	1.38	3.40	6.54	11.00	16.98
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	8.40	11.13	14.96	20.10	26.74	35.05
max M_d	**kNm**	**8.40**	**11.13**	**14.96**	**20.10**	**26.74**	**35.05**

Ultimate Shear Force at Top of Bored Pier							
1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	2.62	6.18	10.68	16.13	22.52	29.85
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	0.47	1.89	4.24	7.55	11.79	16.98
3) 1.25(Pa+Pq_G)+ Pw + Yc Pq_Q + 1.0 Ph	kN	5.73	8.01	11.22	15.38	20.49	26.53
max P_d	**kN**	**5.73**	**8.01**	**11.22**	**16.13**	**22.52**	**29.85**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))

2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5

3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		1	1	1	2	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**
Required Pier Depth + Undermining, D =	**m**	**1.50**	**1.65**	**1.90**	**2.15**	**2.31**	**2.49**

Steel Design							
Design moment, M* =	kNm	8.40	11.13	14.96	20.10	26.74	35.05
Post design length, l =	m	0.80	1.00	1.20	1.40	1.60	1.80
Post effective length factor, kl =		2.00	2.00	2.00	2.00	2.00	2.00
Post moment modification factor =		2.25	2.25	2.25	2.25	2.25	2.25
Intermediate post =		**150 UB 14**	**150 UB 14**	**150 UB 14**	**150 UB 14**	**180 UB 16**	**200 UB 18**
Corner/end post =		**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**

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RETAINING WALL MARK: *SW2*

Wall Height					
Level difference h =	m	1.40	1.60	1.80	2.00

Working Forces and Moments					
Additional lever arm (ineffective depth)	m	0.60	0.60	0.60	0.60
Active Thrust P_a = 0.5 * Ka * gsoil * h² * a	kN	18.49	24.15	30.56	37.73
M_{pa} = Pa * h/3	kNm	19.72	27.37	36.67	47.79
Dead Surcharge Force P_{qG} = q_G * Ka * h * a	kN	0.00	0.00	0.00	0.00
M_{pqG} = Pq_G * h/2	kNm	0.00	0.00	0.00	0.00
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	10.01	11.44	12.87	14.30
M_{pqQ} = Pq_Q * h/2	kNm	13.01	16.02	19.31	22.88
Balustrade Force P_b = Max(p*a, p, p*a*h_b)	kNm	0.00	0.00	0.00	0.00
M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)	kN	0.00	0.00	0.00	0.00
Hydrostatic Force P_h = gwater * h²/2 *a	kN	0.00	0.00	0.00	0.00
M_{ph} = ph * h/3	kNm	0.00	0.00	0.00	0.00
Wind force P_w = pw * hf * a	kN	4.41	4.41	4.41	4.41
M_w = Pw * (h + hf/2)	kNm	12.77	13.66	14.54	15.42

Ultimate Moment at Top of Bored Pier					
1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	44.17	58.23	74.80	94.06
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	24.65	34.21	45.84	59.74
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	45.23	57.47	71.96	88.88
max M_d	**kNm**	**45.23**	58.23	74.80	94.06

Ultimate Shear Force at Top of Bored Pier					
1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	38.13	47.34	57.51	68.61
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	23.11	30.18	38.20	47.16
3) 1.25(Pa+Pq_G)+ Pw + Yc Pq_Q + 1.0 Ph	kN	33.52	41.45	50.33	60.15
max P_d	**kN**	**38.13**	47.34	**57.51**	**68.61**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))
2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5"
3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		2	2	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.60**	**0.60**
Required Pier Depth + Undermining, D =	**m**	**2.68**	**2.88**	**3.12**	**3.32**

Steel Design					
Design moment, M* =	kNm	45.23	58.23	74.80	94.06
Post design length, l =	m	2.00	2.20	2.40	2.60
Post effective length factor, kl =		2.00	2.00	2.00	2.00
Post moment modification factor =		2.25	2.25	2.25	2.25
Intermediate post =		**200 UB 22**	**200 UB 22**	**250 UB 31**	**250 UB 31**
Corner/end post =		**150 PFC**	**150 PFC**	**180 PFC**	**200 PFC**

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RETAINING WALL CALCULATIONS - STEEL POST & BORED PIER

RETAINING WALL MARK SW3

Common Geometry

Calculation increment		0.2	m
Height range, lower (multiple of increment)		0.2	m
Height range, upper (multiple of increment)		2.8	m
Ineffective soil depth (undermining)		0.60	m
Post Spacing	a	2.43	m
Backfill slope	β	0	o
		0.000	rad
Bored pier diameter, primary	B	0.45	m
Bored pier diameter, larger alternative		0.6	m
Wall height threshold for pier size		1.8	

Additional Loads

Surcharge pressure, dead	q_G	0	kPa
Surcharge pressure, live	q_Q	5	kPa
Hydrostatic pressure		0	kN/m³
Fence height	h_f	1.8	m
Site wind speed, ultimate	V_{sit}	37.4	m/s
Site wind pressure	q_z	0.839	kPa
Aerodynamic shape factor C,pn on fence	$C_{p,n}$	1.2	
Design wind pressure	p_w	1.01	kPa
Balustrade handrail height		0.00	m
Balustrade, top edge UDL		0.00	kN/m
Balustrade, top edge PL		0.00	kN
Balustrade, infill UDL		0.00	kPa
Include balustrade forces for H < 1.0m?		No	

Parameters

Founding material		Cohesive (i.e. clay)	
Soil bearing capacity (Rutledge method)	SS	100	kPa
Soil undrained shear strength (Broms Method)	Su	50	kPa
Soil unit weight	g_{soil}	18	kN/m³
Soil characteristic internal friction angle	f	30	o
		0.524	rad
Soil backfill uncertainty level		Uncontrolled fill	
	F_{uf}	0.75	
Design Internal Friction Angle	f*	0.41	rad
External friction angle	d	0.175	rad
Wedge Angle	J	0.990	rad
Active Pressure Coefficient	K_{ah}	0.431	
Active Pressure Coefficient, Surcharge	$K_{a,q}$	0.589	

Factors of Safety

Earth Thrust FOS	1.25
Surcharge, dead FOS	1.25
Surcharge, live FOS	1.50
Surcharge Yc	0.60
Hydrostatic FOS	1.00
Wind FOS	1.00

RETAINING WALL MARK: SW3

Wall Height							
Level difference h =	m	0.20	0.40	0.60	0.80	1.00	1.20

Working Forces and Moments							
Additional lever arm (ineffective depth)	m	0.60	0.60	0.60	0.60	0.60	0.60
Active Thrust P_a = 0.5 * Ka * gsoil * h² * a	kN	0.38	1.51	3.40	6.04	9.43	13.58
M_{pa} = Pa * h/3	kNm	0.25	1.11	2.72	5.23	8.80	13.58
Dead Surcharge Force P_{qG} = q_G * Ka * h * a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{pqG} = Pq_G * h/2	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	1.43	2.86	4.29	5.72	7.15	8.58
M_{pqQ} = Pq_Q * h/2	kNm	1.00	2.29	3.86	5.72	7.87	10.30
Balustrade Force P_b = Max(p*a, p, p*a*h_b)	kNm	0.00	0.00	0.00	0.00	0.00	0.00
M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)	kN	0.00	0.00	0.00	0.00	0.00	0.00
Hydrostatic Force P_h = gwater * h²/2 *a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{ph} = ph * h/3	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Wind force P_w = pw * hf * a	kN	4.41	4.41	4.41	4.41	4.41	4.41
M_w = Pw * (h + hf/2)	kNm	7.49	8.37	9.25	10.13	11.01	11.89

Ultimate Moment at Top of Bored Pier							
1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	1.82	4.82	9.19	15.12	22.80	32.42
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	0.31	1.38	3.40	6.54	11.00	16.98
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	8.40	11.13	14.96	20.10	26.74	35.05
max M_d	**kNm**	**8.40**	**11.13**	**14.96**	**20.10**	**26.74**	**35.05**

Ultimate Shear Force at Top of Bored Pier							
1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	2.62	6.18	10.68	16.13	22.52	29.85
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	0.47	1.89	4.24	7.55	11.79	16.98
3) 1.25(Pa+Pq_G)+ Pw + Yc Pq_Q + 1.0 Ph	kN	5.73	8.01	11.22	15.38	20.49	26.53
max P_d	**kN**	**5.73**	**8.01**	**11.22**	**16.13**	**22.52**	**29.85**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))
2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5
3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		1	1	1	2	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**
Required Pier Depth + Undermining, D =	**m**	**1.50**	**1.65**	**1.90**	**2.15**	**2.31**	**2.49**

Steel Design							
Design moment, M* =	kNm	8.40	11.13	14.96	20.10	26.74	35.05
Post design length, l =	m	0.80	1.00	1.20	1.40	1.60	1.80
Post effective length factor, kl =		2.00	2.00	2.00	2.00	2.00	2.00
Post moment modification factor =		2.25	2.25	2.25	2.25	2.25	2.25
Intermediate post =		**150 UB 14**	**150 UB 14**	**150 UB 14**	**150 UB 14**	**180 UB 16**	**200 UB 18**
Corner/end post =		**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**

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RETAINING WALL MARK: SW3

Wall Height							
Level difference h =	m	1.40	1.60	1.80	2.00	2.20	2.40

Working Forces and Moments							
Additional lever arm (ineffective depth)	m	0.60	0.60	0.60	0.60	0.60	0.60
Active Thrust P_a = 0.5 * Ka * gsoil * h² * a	kN	18.49	24.15	30.56	37.73	45.65	54.33
M_{pa} = Pa * h/3	kNm	19.72	27.37	36.67	47.79	60.87	76.06
Dead Surcharge Force P_{qG} = q_G * Ka * h * a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{pqG} = Pq_G * h/2	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	10.01	11.44	12.87	14.30	15.73	17.16
M_{pqQ} = Pq_Q * h/2	kNm	13.01	16.02	19.31	22.88	26.74	30.89
Balustrade Force P_b = Max(p*a, p, p*a*h_b)	kNm	0.00	0.00	0.00	0.00	0.00	0.00
M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)	kN	0.00	0.00	0.00	0.00	0.00	0.00
Hydrostatic Force P_h = gwater * h²/2 *a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{ph} = ph * h/3	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Wind force P_w = pw * hf * a	kN	4.41	4.41	4.41	4.41	4.41	4.41
M_w = Pw * (h + hf/2)	kNm	12.77	13.66	14.54	15.42	16.30	17.18

Ultimate Moment at Top of Bored Pier							
1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	44.17	58.23	74.80	94.06	116.20	141.41
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	24.65	34.21	45.84	59.74	76.08	95.07
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	45.23	57.47	71.96	88.88	108.43	130.79
max M_d	**kNm**	**45.23**	58.23	74.80	94.06	116.20	141.41

Ultimate Shear Force at Top of Bored Pier							
1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	38.13	47.34	57.51	68.61	80.66	93.65
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	23.11	30.18	38.20	47.16	57.06	67.91
3) 1.25(Pa+Pq_G)+Pw + Yc Pq_Q + 1.0 Ph	kN	33.52	41.45	50.33	60.15	70.91	82.61
max P_d	**kN**	**38.13**	47.34	**57.51**	**68.61**	**80.66**	**93.65**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))
2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5"
3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		2	2	2	2	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.60**	**0.60**	**0.60**	**0.60**
Required Pier Depth + Undermining, D =	**m**	**2.68**	**2.88**	**3.12**	**3.32**	**3.52**	**3.74**

Steel Design							
Design moment, M* =	kNm	45.23	58.23	74.80	94.06	116.20	141.41
Post design length, l =	m	2.00	2.20	2.40	2.60	2.80	3.00
Post effective length factor, kl =		2.00	2.00	2.00	2.00	2.00	2.00
Post moment modification factor =		2.25	2.25	2.25	2.25	2.25	2.25
Intermediate post =		**200 UB 22**	**200 UB 22**	**250 UB 31**	**250 UB 31**	**310 UB 40**	**310 UB 40**
Corner/end post =		**150 PFC**	**150 PFC**	**180 PFC**	**200 PFC**	**250 PFC**	**250 PFC**

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RETAINING WALL MARK: SW3

Wall Height

Level difference h =	m	2.60	2.80

Working Forces and Moments

Additional lever arm (ineffective depth)	m	0.60	0.60
Active Thrust $P_a = 0.5 * Ka * gsoil * h^2 * a$	kN	63.76	73.95
$M_{pa} = Pa * h/3$	kNm	93.51	113.38
Dead Surcharge Force $P_{qG} = q_G * Ka * h * a$	kN	0.00	0.00
$M_{pqG} = Pq_G * h/2$	kNm	0.00	0.00
Live Surcharge Force $P_{qQ} = q_Q * Ka * h * a$	kN	18.59	20.02
$M_{pqQ} = Pq_Q * h/2$	kNm	35.33	40.05
Balustrade Force $P_b = Max(p*a, p, p*a*h_b)$	kNm	0.00	0.00
$M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)$	kN	0.00	0.00
Hydrostatic Force $P_h = gwater * h^2/2 * a$	kN	0.00	0.00
$M_{ph} = ph * h/3$	kNm	0.00	0.00
Wind force $P_w = pw * hf * a$	kN	4.41	4.41
$M_w = Pw * (h + hf/2)$	kNm	18.06	18.94

Ultimate Moment at Top of Bored Pier

1) $1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)$	kNm	169.88	201.80
2) $1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)$	kNm	116.89	141.73
3) $1.25(MPa+MPq_G)+MPw+Yc\ MPq_Q+1.0(MPh)$	kNm	156.15	184.70
max M_d	**kNm**	169.88	201.80

Ultimate Shear Force at Top of Bored Pier

1) $1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)$	kN	107.59	122.47
2) $1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph$	kN	79.70	92.43
3) $1.25(Pa+Pq_G)+ Pw + Yc\ Pq_Q + 1.0\ Ph$	kN	95.26	108.85
max P_d	**kN**	**107.59**	**122.47**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : $P/SS = BD^2 / (2.34D + 2.64(Md/Pd))$
2) BROMS METHOD, COHESIVE : $D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))$^0.5"
3) BROMS METHOD, COHESIONLESS: $\gamma B(1/ka)D^3 - 2Pd(Md/Pd) - 2PdD = 0$

Method of analysis used =		2	2
Pier Diameter, B =	**m**	**0.60**	**0.60**
Required Pier Depth + Undermining, D =	**m**	**3.96**	**4.197229827**

Steel Design

Design moment, M* =	kNm	169.88	201.80
Post design length, l =	m	3.20	3.40
Post effective length factor, kl =		2.00	2.00
Post moment modification factor =		2.25	2.25
Intermediate post =		**360 UB 51**	**410 UB 54**
Corner/end post =		**250 PFC**	**300 PFC**

RETAINING WALL CALCULATIONS - STEEL POST & BORED PIER
RETAINING WALL MARK SW4

Common Geometry

Calculation increment		0.2	m
Height range, lower (multiple of increment)		0.2	m
Height range, upper (multiple of increment)		2	m
Ineffective soil depth (undermining)		0.00	m
Post Spacing	a	2.43	m
Backfill slope	β	10	o
		0.175	rad
Bored pier diameter, primary	B	0.45	m
Bored pier diameter, larger alternative		0	m
Wall height threshold for pier size		N/A	

Additional Loads

Surcharge pressure, dead	q_G	0	kPa
Surcharge pressure, live	q_Q	5	kPa
Hydrostatic pressure		0	kN/m³
Fence height	h_f	1.8	m
Site wind speed, ultimate	V_{sit}	37.4	m/s
Site wind pressure	q_z	0.839	kPa
Aerodynamic shape factor C,pn on fence	$C_{p,n}$	1.2	
Design wind pressure	p_w	1.01	kPa
Balustrade handrail height		0.00	m
Balustrade, top edge UDL		0.00	kN/m
Balustrade, top edge PL		0.00	kN
Balustrade, infill UDL		0.00	kPa
Include balustrade forces for H < 1.0m?		No	

Soil Parameters

Founding material		Cohesive (i.e. clay)	
Soil bearing capacity (Rutledge method)	SS	100	kPa
Soil undrained shear strength (Broms Method)	Su	50	kPa
Soil unit weight	g_{soil}	18	kN/m³
Soil characteristic internal friction angle	f	30	o
		0.524	rad
Soil backfill uncertainty level		Uncontrolled fill	
	F_{uf}	0.75	
Design Internal Friction Angle	f*	0.41	rad
External friction angle	d	0.175	rad
Wedge Angle	J	0.990	rad
Active Pressure Coefficient	K_{ah}	0.460	
Active Pressure Coefficient, Surcharge	$K_{a,q}$	0.589	

Factors of Safety

Earth Thrust FOS	1.25
Surcharge, dead FOS	1.25
Surcharge, live FOS	1.50
Surcharge Yc	0.60
Hydrostatic FOS	1.00
Wind FOS	1.00

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RETAINING WALL MARK: SW4

Wall Height							
Level difference h =	m	0.20	0.40	0.60	0.80	1.00	1.20

Working Forces and Moments							
Additional lever arm (ineffective depth)	m	0.00	0.00	0.00	0.00	0.00	0.00
Active Thrust P_a = 0.5 * Ka * gsoil * h² * a	kN	0.40	1.61	3.62	6.44	10.07	14.50
M_{pa} = Pa * h/3	kNm	0.03	0.21	0.72	1.72	3.36	5.80
Dead Surcharge Force P_{qG} = q_G * Ka * h * a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{pqG} = Pq_G * h/2	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	1.43	2.86	4.29	5.72	7.15	8.58
M_{pqQ} = Pq_Q * h/2	kNm	0.14	0.57	1.29	2.29	3.58	5.15
Balustrade Force P_b = Max(p*a, p, p*a*h_b)	kNm	0.00	0.00	0.00	0.00	0.00	0.00
M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)	kN	0.00	0.00	0.00	0.00	0.00	0.00
Hydrostatic Force P_h = gwater * h²/2 *a	kN	0.00	0.00	0.00	0.00	0.00	0.00
M_{ph} = ph * h/3	kNm	0.00	0.00	0.00	0.00	0.00	0.00
Wind force P_w = pw * hf * a	kN	4.41	4.41	4.41	4.41	4.41	4.41
M_w = Pw * (h + hf/2)	kNm	4.85	5.73	6.61	7.49	8.37	9.25

Ultimate Moment at Top of Bored Pier							
1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	0.25	1.13	2.84	5.58	9.56	14.97
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	0.03	0.27	0.91	2.15	4.19	7.25
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	4.96	6.34	8.29	11.01	14.71	19.59
max M_d	**kNm**	**4.96**	**6.34**	**8.29**	**11.01**	**14.71**	**19.59**

Ultimate Shear Force at Top of Bored Pier							
1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	2.65	6.30	10.97	16.64	23.31	30.99
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	0.50	2.01	4.53	8.05	12.58	18.12
3) 1.25(Pa+Pq_G)+ Pw + Yc Pq_Q + 1.0 Ph	kN	5.77	8.13	11.51	15.89	21.28	27.68
max P_d	**kN**	**5.77**	**8.13**	**11.51**	**16.64**	**23.31**	**30.99**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))
2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5
3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		1	1	1	1	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**	**0.45**
Required Pier Depth, D =	**m**	**0.75**	**0.90**	**1.10**	**1.40**	**1.58**	**1.75**

Steel Design							
Design moment, M* =	kNm	4.96	6.34	8.29	11.01	14.71	19.59
Post design length, l =	m	0.20	0.40	0.60	0.80	1.00	1.20
Post effective length factor, kl =		2.00	2.00	2.00	2.00	2.00	2.00
Post moment modification factor =		2.25	2.25	2.25	2.25	2.25	2.25
Intermediate post =		**150 UB 14**	**150 UB 14**	**150 UB 14**	**150 UB 14**	**150 UB 14**	**150 UB 14**
Corner/end post =		**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**

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RETAINING WALL MARK: *SW4*

Wall Height					
Level difference h =	m	1.40	1.60	1.80	2.00
Working Forces and Moments					
Additional lever arm (ineffective depth)	m	0.00	0.00	0.00	0.00
Active Thrust P_a = 0.5 * Ka * gsoil * h² * a	kN	19.73	25.77	32.62	40.27
M_{pa} = Pa * h/3	kNm	9.21	13.75	19.57	26.85
Dead Surcharge Force P_{qG} = q_G * Ka * h * a	kN	0.00	0.00	0.00	0.00
M_{pqG} = Pq_G * h/2	kNm	0.00	0.00	0.00	0.00
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	10.01	11.44	12.87	14.30
M_{pqQ} = Pq_Q * h/2	kNm	7.01	9.15	11.58	14.30
Balustrade Force P_b = Max(p*a, p, p*a*h_b)	kNm	0.00	0.00	0.00	0.00
M_b = Max(P_b*a*(h+h_b, P_b*(h+h_b), P_b*a*(h+h_b/2)	kN	0.00	0.00	0.00	0.00
Hydrostatic Force P_h = gwater * h²/2 *a	kN	0.00	0.00	0.00	0.00
M_{ph} = ph * h/3	kNm	0.00	0.00	0.00	0.00
Wind force P_w = pw * hf * a	kN	4.41	4.41	4.41	4.41
M_w = Pw * (h + hf/2)	kNm	10.13	11.01	11.89	12.77
Ultimate Moment at Top of Bored Pier					
1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	22.02	30.91	41.84	55.01
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	11.51	17.18	24.46	33.56
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	25.85	33.69	43.31	54.91
max M_d	**kNm**	**25.85**	33.69	43.31	55.01
Ultimate Shear Force at Top of Bored Pier					
1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	39.68	49.38	60.08	71.79
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	24.67	32.22	40.77	50.34
3) 1.25(Pa+Pq_G)+ Pw + Yc Pq_Q + 1.0 Ph	kN	35.08	43.49	52.90	63.32
max P_d	**kN**	**39.68**	49.38	**60.08**	**71.79**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))
2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5"
3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		2	2	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.45**	**0.45**
Required Pier Depth, D =	**m**	**1.93**	**2.12**	**2.33**	**2.54**

Steel Design

Design moment, M* =	kNm	25.85	33.69	43.31	55.01
Post design length, l =	m	1.40	1.60	1.80	2.00
Post effective length factor, kl =		2.00	2.00	2.00	2.00
Post moment modification factor =		2.25	2.25	2.25	2.25
Intermediate post =		**150 UB 14**	**180 UB 16**	**200 UB 18**	**200 UB 22**
Corner/end post =		**150 PFC**	**150 PFC**	**150 PFC**	**150 PFC**

ENGINEERING CONSULTANTS
276 BRIGHTON ROAD SOMERTON PARK
TELEPHONE +61 (0)8 8295 8677
admin@magryn.com.au
www.magryn.com.au
ABN 64 090 560 099

RETAINING WALLS V5.08 Magryn & Associates

Error - Footing too small

Wall:	**(BW1 - Wall Geometry, Upper (200 Series Block)) 1600mm retaining ht., 190mm thick (No key)**	
Loads:	**Surcharge = 5.00kPa, Drained, Additional overturning applied**	
Footing:	**450mm long (overall), 260mm overhang (behind wall), 450mm deep**	
Stability:	**M*o = 23.6kNm > øMr = -533.4kNm**	**No Good (-0.04)**
Sliding:	**V*slide = 28.0kN > øVr = 10.4kN**	**No Good (2.70)**
Wall:	**M*wall = 13.2kNm, V*wall = 18.6kN, Design factor Φn = 1.0**	
Bearing:	**Max. bearing pressure = 99999kPa > Allowable = 0kPa**	**Error - Footing too small**

Geometry

Wall assumed fully drained

Retaining height (z) =	1600	mm (= Wall height)
Risk class =	B	(A),(B),(C) - Moderate damage and loss of services - Table 1.1
Backfill type =	U	Class(1), Class(2), (U)ncontrolled, (I)n-situ - Table 5.1(A)

Soil parameters

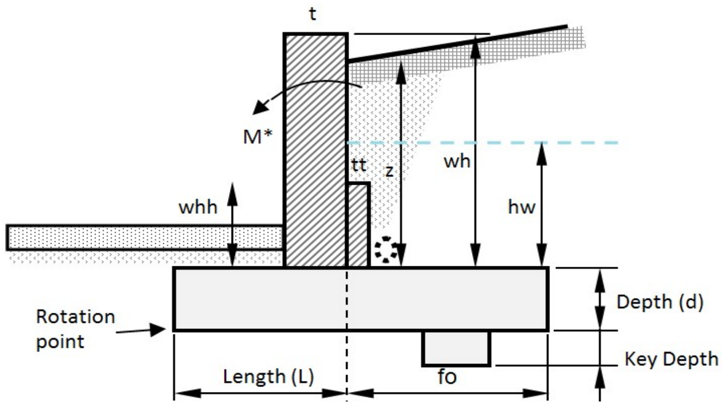
Internal friction (ø) =	30	° (0° ≤ ø ≤ 45°)
Incline of soil at top (β) =	0	° (0° ≤ β ≤ 45°) 1 in 0.0
Soil weight (γ) =	18.0	kN/m³
Cohesion (c) =	0	kPa
Friction coefficient (μ) =	0.433	(-1 to ignore sliding)
Use passive =	Y	(Y)es,(N)o
Manual Kp value =	0.00	0 for default
Soil depth to Ignore =	0	mm (for passive)
Allowable bearing =		kPa
Slab overlay (ts) =		mm
Soil overlay (tsl) =		mm

Wall

Wall thickness (t) =	190	mm
Wall height (wh) =	1600	mm (= Retaining height)
Add'l thickness (tt) =		mm (behind wall)
Add'l th. height (whh) =		mm
Add'l th. on soil side =	Y	(Y)es,(N)o
Wall Wt (Wwt) =	25	kN/m³

Footing

Footing depth (d) =	450	mm
End thickness (d2) =	0	mm (0 = no taper)
Footing length (L) =	190	mm excluding overhang
Overhang (fo) =	260	mm additional to length
Key depth (key) =	0	mm (sliding only)
Key length (klen) =	0	mm
Key position from heel/back (kpos) =	0	mm (to face)



Design loads - Appendix J - AS4678 & Section 4

Surcharge Min. =	5.0	kPa min. (Table 4.1)
Surcharge (S) =	5.0	kPa
Water height (hw) =	0	mm (over footing, 0=no water)
Earth factor =	1.25	(1.25)
Water pressure =	1.00	(1.00)
Live load factor =	1.50	(1.50 - 0.0 for stabilising effect)
Stabilising factor (øs) =	0.80	AS 4678 - Appendix J2

Additional Loads at top of wall: `

Overturning (Ma) =	0.88	kNm/m
Thrust (Va) =	0.98	kN/m
Load factor =	1.00	
Ma* =	0.88	kNm/m
Va* =	0.98	kN/m
(Moment/Thrust about top wall)		
Additional dead load (Pv) =	0.00	kN/m

Material design factors - Section 5

Φuc =	0.50	c*=c*Φuc =	0 kPa	Cl 5.2.1			
Φuo =	0.75	ø* =	23.4 °	Cl 5.2.2		ø* =	0.409 rad
Design factor (Φn) =	1.00 Table 5.2						
	Ka =	0.431		Depth of tension zone =	0 mm		
	Kp =	2.319		Acting soil height (ha) =	1600 mm		

BW1 - Wall Geometry, Upper (200 Series Block)

Error - Footing too small

Design Moment in wall (about top of footing, per m of wall)

	Pressure kPa	Thrust kN/m	Thrust* kN/m	Lever arm mm	Mw kNm/m	Mw* kNm/m	Comments (At wall base)
Soil = Ka*γ*ha =	12.42	9.94	12.42	533	5.30	6.62	Max. pressure at base
Water = None =	0.00	0.00	0.00	0	0.00	0.00	Max. pressure at base
Surcharge = Ka*S =	2.16	3.45	5.18	800	2.76	4.14	Uniform pressure dist.
Additional =		0.98	0.98	1600	2.45	2.45	
	Total	**14.37**	**18.58**		**10.51**	**13.21**	

Total thrust (Vw) = 14.37 kN/m
Total ult. thrust (Vw*) = 18.58 kN/m
Total ult. design thrust (Vw/Φn) = 18.58 kN/m
Total moment (Mw) = 10.51 kNm/m
Total ult. moment (Mw*) = 13.21 kNm/m
Total ult. design moment (Mw*/Φn) = 13.21 kNm/m

Overturning (Limit mode U2) (about base of footing, per m of wall)

Depth for rotation (deff) = 450 mm (Footing depth)

	Pressure kPa	Thrust kN/m	Thrust* kN/m	Lever arm mm	Mo kNm/m	Mo* kNm/m	Comments (At footing base)
Soil = Ka*γ*(ha+deff) =	15.91	16.31	20.39	683	11.15	13.93	Max. pressure at base
Water = None =	0.00	0.00	0.00	0	0.00	0.00	Max. pressure at base
Surcharge = Ka*S =	2.16	4.42	6.63	1025	4.53	6.80	Uniform pressure dist.
Additional =		0.98	0.98	2050	2.89	2.89	
	Total	**21.71**	**28.00**		**18.57**	**23.62**	

Total thrust = 21.71 kN/m
Total ult. thrust = 28.00 kN/m
Total moment (Mo) = 18.57 kNm/m
Total ult. moment (Mo*) = 23.62 kNm/m
Total ult. (Mo*/Φn) = 23.62 kNm/m

Restoring (Limit mode U2) (about base of footing, per m of wall)

	Force kN/m	Lever arm mm	Mr kNm/m	øMr kNm/m	Ecc mm	Mb kNm/m	
Footing =	5.06	-33108	-167.61	-134.09	0	0.00	
Soil overlay =	0.00	-33333	0.00	0.00	225	0.00	
Slab overlay =	0.00	-33333	0.00	0.00	225	0.00	
Wall =	7.60	-33238	-252.61	-202.09	130	0.99	
Lower wall (back) =	0.00	-33143	0.00	0.00	35	0.00	Using Wwt-Soil dentity (γ)
Soil behind wall =	7.49	-33013	-247.20	-197.76	-95	-0.71	
Dead load (Pv) =	0.00	-33238	0.00	0.00	130	0.00	
Tapered =	0.00	0	0.00	0.00	225	0.00	
Key =	0.00	-32883	0.00	0.00	-225	0.00	
Passive pressure =			0.63	0.51			
Total	**20.15**		**-666.79**	**-533.43**		**0.28**	

Total force (Wt) = 20.15 kN/m
Total bearing overturning (Mb) = 0.28 kN/m
Total restoring (Mr) = -666.79 kNm/m
Total ult. Restoring (øs(0.80)*"Mr) = -533.43 kNm/m No Good (-0.04)

BW1 - Wall Geometry, Upper (200 Series Block)

RETAINING WALLS V5.08 Magryn & Associates

Error - Footing too small

Sliding (Limit mode U1)

Sliding (V*slide/Φn) =	28.00	kN/m at base	
Passive resistance of footing and key (Rp) =	4.23	kN/m	
Sliding resistance (Rs = µ*Wt) =	8.73	kN/m	
Total resistance øVr = øs(0.80)*(Rp+Rs) =	10.36	kN/m	No Good (2.70)

Key Forces (Not applicable)

Bearing

Total (Wt) =	20.15	kN/m	
Overturning moment (Mot = Mo + Mb) =	18.84	kNm/m	
Ecc = Mot / Wt =	935	mm	
Maximum bearing pressure (q.max) =	99999.0	kPa	
Minimum bearing pressure (q.min) =	99999.0	kPa	Error - Footing too small
Centroid from left/front edge of footing (y) =	99999	mm	(Entire footing under pressure)

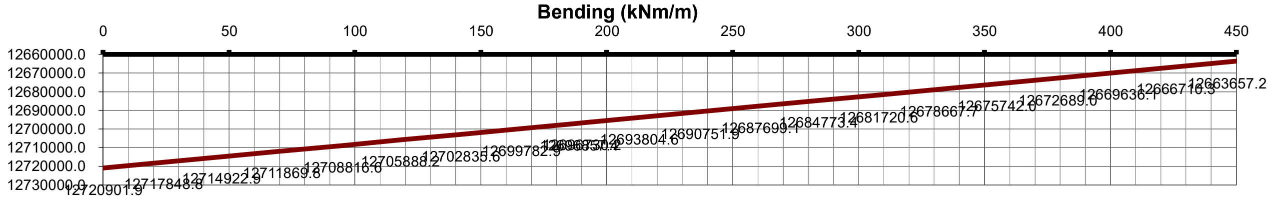
Design Moment and Shear in footing due to bearing (per m of wall)

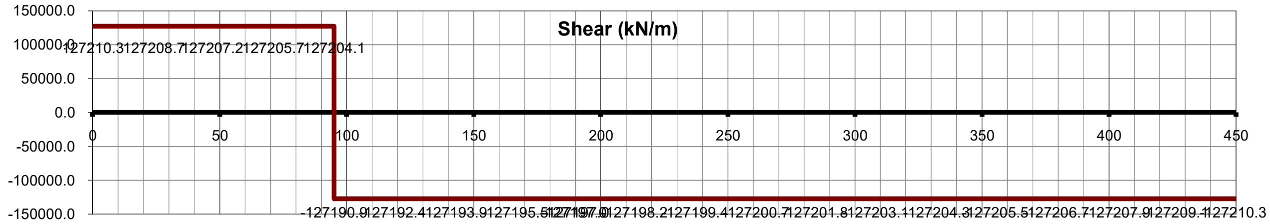
Manual force position =	225	mm (from left/front edge of footing)
Equivalent load factor =	1.27	

	Moment* kNm/m	Shear* kN/m	Lever arm mm	Position mm
Front face of wall =	12720901.9	127210.3	99999	0
Centre of wall (from the left) =	12708816.6	127204.1	99909	95
Centre of wall (from the right) =	12708813.6	-127190.9	99919	95
Back face of wall (soil side) =	12696730.2	-127197.0	99819	190
Max. moment (+ve) =	12720901.9	127210.3	-	0
Max. moment (-ve) =	12663657.2	-127210.3	-	450
Max. shear =	12720901.9	127210.3	-	0
Manual position =	12692278.3	-127198.8	-	225

Graphs (Footing)

Error - Footing too small





BW1 - Wall, Upper (200 Series Block)

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Unit:	**(BW1 - Wall, Upper (200 Series Block)) 20.01 Hollow block (All cores grouted) [Wall]**
Reinf't:	**N12-200 cts**
Grout:	**All cores grouted**

Hollow block properties (Face-shell bedded joints)

Masonry unit =	20.01		Custom =	N (Y)es,(N)o
Unit type =	H	(S)olid, (H)ollow	Solidity ratio =	0.51 (51%)
Unit material =	C	C(L)ay, Calcium (S)ilicate with basalt ag., (C)oncrete with basalt ag.		
Percentage basaltic aggregate > 45% =	N	(Y)es,(N)o		
Thickness of unit (tu) =	190	mm		
Height of unit (hu) =	190	mm		
Length of unit (lu) =	390	mm		
Shell thickness (ts) =	30	mm		
Overlap of units (sp) =	190	mm		
Self weight =	2.2	kN/m²	Eff. width (t) =	190 mm
AAC masonry =	N	(Y)es,(N)o	Core cts =	200 mm
Stack bonded =	N	(Y)es,(N)o		
Density of block =	2250	kg/m³		

Compressive strength - Cl 3.3.2

Char. comp. strength of unit (f'uc) =	15.0	MPa	
Compressive strength factor (km) =	1.6		
Char. comp. strength of masonry (f'mb = km*√f'uc) =	6.20	MPa	Cl 3.3.2(a)(i)
hu/tj =	19.0		Cl 3.3.2(a)(i)
Joint thickness factor (kh) =	1.30		Table 3.2
Compressive strength (f'm = kh*f'mb) =	8.06	MPa	Cl 3.3.2(a)(i)

Flexural tensile strength - Cl 3.3.3

Char. lateral modulus of rupture (f'ut) =	0.8	MPa	Cl 3.2

Joint

Joint thickness (tj) =	10	mm	Cl 4.9.1
Rake (face in comp. under +ve bending) =	0	mm (Unraked)	
Rake (face in comp. under -ve bending) =	0	mm (Unraked)	

Grouting (All cores grouted)

Grouted every =	1	(All cores)	Density of grout =	2400 kg/m³
Compressive strength of grout (f'c) =	20.0	MPa		
Design comp. strength of grout (f'cg = max(12,f'c)) =	20.0	MPa	Cl 3.5	
Grout strength factor (kc) =	1.4		Cl 7.3.2 & Cl 8.5 - Hollow & ρ>2000kg/m³ =1.4 else 1.2	

Cross section properties

Bedded thickness (tb = leafs*tu) =	190	mm	Cl 4.5.1 with rake ≤ 3mm		
Bedded area (Hollow) (Ab = ts*2*1000) =	60000	mm²/m	Cl 4.5.4(b)		
Combined cross-sectional area (Grouted/reinf.) (Ac) =	190000	mm²/m	Cl 4.5.5		
Design cross section (Ad) =	190000	mm²/m	Cl 4.5.6		
Grout area (Ag = Ad - Ab) =	130000	mm²/m	Cl 4.5.7	Wall S.Wt =	4.56 kN/m²

Reinforcement

Vertical reinforcement =	**N12-200 cts**			
Bar size (Dia) =	12	mm	Ast.min = 0.002*Ad =	380 mm²/m
Reinf't cts (cts) =	200	mm	Area steel (Ast) =	565 mm²/m
Reinf't strength (fsy) =	500	MPa - Table 3.7		
Depth to steel in +ve bending from unit face (ds) =	95	mm	Reinf't central (ds) =	95 mm
Depth to steel in +ve bending (dt = d - rep) =	95	mm	Reinf't cover in block =	59.0 mm
Depth to steel in -ve bending (dc) =	95	mm		

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BW1 - Wall, Upper (200 Series Block)

MASONRY MEMBER V5.11 Magryn & Associates

Unit:	**(BW1 - Wall, Upper (200 Series Block)) 20.01 Hollow block (All cores grouted) [Wall]**	
Reinf't:	**N12-200 cts**	
Geometry:	**10000mm long x 1500mm high panel**	
Capacity:	**Mv* = 13.20kNm/m < øMd+ = 16.71kNm/m**	**OK (0.79)**
	Mv-* = 0.56kNm/m < øMd- = 16.71kNm/m	**OK (0.03)**
Combined:	**Compression & Mv+* = 0.79, Tension & Mv+* = 0.79**	**OK (0.79)**
	Compression & Mv-* = 0.00, Tension & Mv-* = 0.00	**OK (0.00)**

Loadings - Wall

Robustness (Refer Geometry):

Vertical bending (Mv+*) =	13.20	kNm/m	RMv* =	0.56	kNm/m
Vertical bending (Mv-*) =		kNm/m			
			RMh* =	0.00	kNm/m
Check ductility =	Y	(Y)es,(N)o	Mecc* =	0.00	kNm (see RComp)

Design for Reinforced Vertical Bending Capacity (øMd) - Cl 8.6

Strength reduction factor (reinforced) (ø) =	0.75	Table 4.1	
Effective comp. width per bar =	772	mm (2*tu past reinf't) - Cl 8.6 (i)	
Reinf't centres =	200	mm	
Effective comp. width (b) =	1000	mm/m	
Area steel (Ast) =	565	mm²/m	
Earthquake ductility Astd.min = 0.0013*Ad =	247	mm²/m	
Depth to steel in +ve bending (dt) =	95	mm	
Asd = min(Ast & 0.29*1.3*f'm*b*dt/fsy) =	565	mm²/m	
ku =	0.393	Cl 8.3(c) - Based on Asd	
øMd+ = ø*fsy*Asd*dt*[1-0.6*fsy*Asd/(1.3*f'm*b*dt)] =	16.71	kNm/m	OK (0.79)
Depth to steel in -ve bending (dc) =	95	mm	
Asd- = min(Ast & 0.29*1.3*f'm*b*dc/fsy) =	565	mm²/m	
ku- =	0.393	Cl 8.3(c) - Based on Asd-	
øMd- =	16.71	kNm/m	OK (0.00)

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Combined Bending & Compression - Cl 8.11.1

Compression (Nc*) =	0.0	kN/m	
Bending (M*) =	13.20	kNm/m	
Combined comp. capacity (øFd.comb) =	423.6	kN/m	
Combined comp. ratio (Mv+*) =	0.79		OK (0.79)
Combined comp. ratio (Mv-*) =	0.00		OK (0.00)

Combined Bending & Tension - Cl 8.11.2

Tension (Nt*) =	0.0	kN/m	
Tensile capacity (øFdt = ø*fsy*As) =	212.1	kN/m	
Combined tension ratio (Mv+*) =	0.79		OK (0.79)
Combined tension ratio (Mv-*) =	0.00		OK (0.00)

Secondary reinforcement - Cl 8.4.3

Secondary reinforcement = 0.00035*Ad =	67	mm²/m	Where required to resist non-uniform loadings or temperature or shrinkage
øMds =	2.32	kNm/m	For central bars

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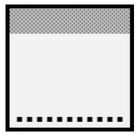
Section:	**(BW1 - Strip Footing) 450mm (D) x 600mm (W) beam, f'c=32MPa**	
Reinf't:	**3-N16 top, 5-N16 bottom (2 layers), ku = 0.12, 2 legs N12-100 cts ligs**	
Strength:	**(+ve M) M* = 31.5kNm < øMuo = 128.9kNm**	**OK (0.24)**
Cracking:	**fscr = 87MPa < Fscr = 216MPa & fscr1 = 100MPa < Fscr1 = 400MPa, crack width = 0.1mm**	**OK (0.25,0.40)**
Ast.min:	**Ast.min = 519mm² < Ast = 1005mm² (Minimum of Deemed and actual)**	**OK (0.52)**
	Shear/Torsion	OK (0.85)

Geometry Interior environment with $\varepsilon csd.b^*=800x10^{-6}$, $\varepsilon cs^*=510x10^{-6}$ L/D ratio = 4.4

Concrete strength (f'c) =	32	MPa
Depth (D) =	450	mm
Web width (W) =	600	mm, (S)lab
Flange width (Bf) =	0	mm



Comp.
Tension

Span (L) =	2000	mm (For effective flange, bef)	Fully enclosed =	N	(Y)es,(N)o
			Side cover =	50	mm
			Formwork =	S	(S)tandard,(R)igid
Concrete weight =	25.0	kN/m³	Exposure top =	B1	Table 4.10.3.2
S.Wt =	6.75	kN/m	Exposure bottom =	B1	Table 4.10.3.2
Gross area (Ag) =	270000	mm²	Span type =	S	(S)imple/cont., (C)antilever

Max. dist. b/n lat. restraints for simple/cont. (L1=min(60*bef,180*bef²/D) = 36000 mm - Cl 8.9.2

Design actions

Analysis values = M (M)anual, (L)eft, Position (X) from analysis, (R)ight

Manual values				**Manual**	**Units**
Design (M*) =	31.5	kNm	**M***	31.5	kNm
Design (Ms1*, Ψs=1) =	D	kNm	**Ms1*(Ψs=1)**	23.6	kNm
Design (Ms*, Ψs) =	D	kNm	**Ms*(Ψs)**	20.5	kNm
			Ast req'd	236	mm²
Ms1* & Ms* estimated - To be verified			**Ast**	1005	mm²
			Reinf't req'd	1.2-N16	

Reinforcement Ligs = 2 legs N12-100 cts

Bottom reinf't =	**5-N16**		**Top reinf't =**	**3-N16**	
Bar size =	16	mm	Bar size =	16	mm
Bar cts/No/mm² =	5	No	Bar cts/No/mm² =	3	No
Yield strength (fsy) =	500	MPa	Yield strength (fsyc) =	500	MPa
Ductility class =	A	(N)ormal,(L)ow,(A)uto	Ductility class =	A	(N)ormal,(L)ow,(A)uto
Reinf't ductility class =	N	(N)ormal,(L)ow	Reinf't ductility class =	N	(N)ormal,(L)ow
Steel area (Ast) =	1005	mm²	Steel area (Asc) =	603	mm²
Bottom cover to ligs =	50	mm	Top cover to ligs =	50	mm
Depth to bottom steel layer (ds.max) =	380	mm	Depth to top steel layer =	70	mm
Depth to bottom steel (ds) =	318	mm	Depth to top steel =	70	mm
D-ds =	132	mm	D-ds =	380	mm
No. bars =	5.0	No.	No. bars =	3.0	No.
Bar centres =	230	mm	Bar centres =	230	mm
Max. bars per layer =	3		Max. bars per layer =	8	
Layers required =	2		Max. bars per 2nd layer =	8	
			Layers required =	1	

Strength in +ve bending for beams - Cl 8.1

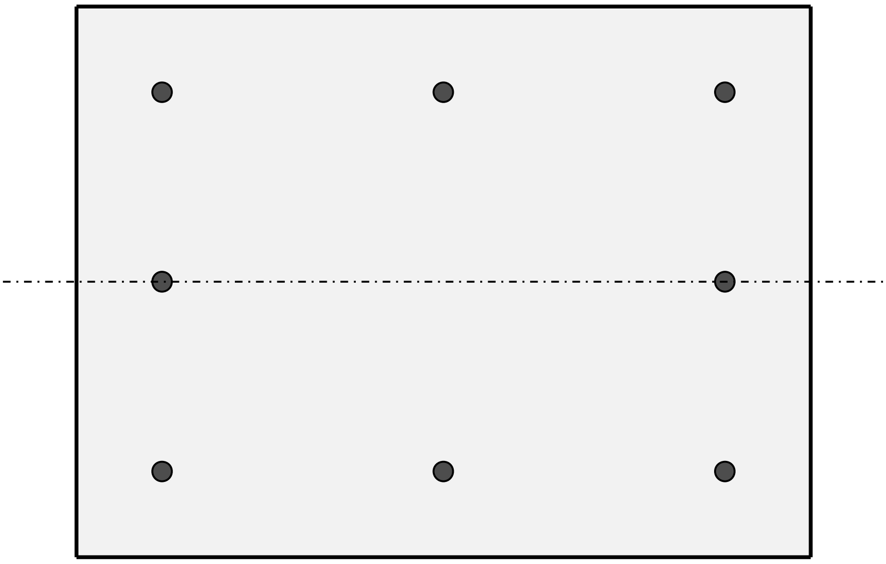
Design width (W) =	600	mm	Design flange (bef) =	600	mm
Tensile steel area (As) =	1005	mm²	Depth to tensile (ds) =	318	mm
Comp. steel area (Ac) =	0	mm²	Depth to comp. steel (dc) =	70	mm
Ultimate Moment (Mu) =	151.6	kNm	ku =	0.115	
Design capacity (øMuo) =	128.9	kNm	Factor (ø) =	0.850	Table 2.2.2
			Ast.min =	519	mm²

Crack control in +ve bending for beams - Cl 8.6

Steel stress (σscr1) =	100	MPa	Max. stress (σscr1.max) =	400	MPa	OK (0.25)
Steel stress (σscr) =	87	MPa	Max. stress (σscr.max) =	216	MPa	OK (0.40)
Max. crack width (w'max) =	0.3	mm	Calculated crack width =	0.09	mm	

Reinf't:	**5.0-N16 bottom tensile**
	3.0-N16 top compression
Ligs:	**2 legs N12-100 cts**

Preview



Section:	**(BW1 - Strip Footing) 450mm (D) x 600mm (W) beam, f'c=32MPa**	
Reinf't:	**5.0-N16 bottom tensile, 2 legs N12-100 cts anchored in compr. zone**	
	3.0-N16 top compression	
	Ligs required	
Strength:	**V* = 63.0kN > ks*øVuc = 58.2kN, V* = 63.0kN ≤ øVu = 74.1kN - Min. shear ligs Required**	**OK (0.85)**
	øVuc = 74.1kN, øVu.min = 139.2kN, øVus = 0.0kN	
Torsion:	**T* = 77.0kNm ≤ øTus = 117.4kNm - Torsion ligs required**	**OK (0.66)**
	0.25øTcr = 11.3kNm, øTcr = 45.4kNm	
Combined:	**Vcomb* = 538.4kN ≤ øVu.max = 1076.0kN**	**OK (0.50)**
Add'l reinf't:	**Additional tensile shear reinforcement not required at Manual location**	
	Error - Lig transverse spacing exceeds max. of 450mm (488mm)	

Strength of beams in shear - Cl 8.2 - Design actions

Warning - Capacity only valid for current combination of V* and M*

Analysis values = M (M)anual, (L)eft, Position (X) from analysis, (R)ight, (C)ritical

Manual shear (V*) =	63.0	kN	Design shear (V*) =	63	kN
Manual moment (M*) =	31.5	kNm	Design moment (M*) =	32	kNm (+ve or -ve)
Manual section max. moment (Mmax*) =	31.5	kNm	Section Max. (Mmax*) =	32	kNm (+ve or -ve)
				(blank = M*)	

Axial (N*) = 0 kN (Tension -ve) (Refer column for bending/axial capacity)
Torsion (T*) = 77 kNm

do (+ve bending) =	380	mm	dv (+ve bending) =	342	mm
do (-ve bending) =	380	mm	dv (-ve bending) =	342	mm

Consideration of torsion - Cl 8.2.1.2 Torsion reinf't req'd such that, T* ≤ øTus

øTcr = ø*0.33*√f'c*(Acp²/uc)√(1+(σcp/(0.33*√f'c))) =	45.4 kNm	Cl 8.2.1.2(2) - Refer below	
0.25*øTcr =	11.3 kNm	Cl 8.2.1.2(1)	
Consider torsional effects (T* > 0.25*øTcr) =	Y (Y)es,(N)o	Cl 8.2.1.2	
T*/0.25øTcr =	6.79	Cl 8.2.1.2(1)	Torsion reinf't req'd (6.79)
Torsion considered, V* =	63.0 kN		

Strength of beams in shear - Cl 8.2

Use Simplified method =	N	(Y)es,(N)o	
Max. nominal aggregate size (dg) =	16	mm	(General method)
Lightweight concrete =	N	(Y)es,(N)o	(General method)
Cracking in compression zone =	N	(Y)es,(N)o	Cl 8.2.4.5

Reinforcement - Min. shear ligs Required Error - Lig transverse spacing exceeds max. of 450mm (488mm) Note - V* ≤ øVu.min, Toggle the increa:

Description =	**2 legs N12-100 cts**		Ignore ligs =	N	(Y)es,(N)o
Lig size =	12	mm	Leg area (Asl) =	113	mm²
Spacing (s) =	100	mm	Asv.req'd =	0	mm² (2-0.0mm dia.)
Legs in cross-section =	2		s.req'd =	0	mm
Steel yield strength (fsy.f) =	500	MPa	s.max =	417	mm
Fitment class =	N	(N)ormal,(L)ow	s.max.t =	417	mm
Angle between shear & tensile reinf't (αv) =	90.0	°			

Shear spacing - Cl 8.3.2.2

Max. cts = 0.5*D =	225	mm < 300mm	Ligs area (Asv) =	226	mm²
Max. cts =	225	mm	Adopted ligs area (Asv) =	0	mm²
			Asv.min =	54	mm² (2-5.9mm dia.)

Transverse spacing - Cl 8.3.2.2

Transverse spacing =	488	mm	Waive max. spacing =	N	(Y)es,(N)o
Max.transverse spacing =	450	mm	Waive transv. spacing =	N	(Y)es,(N)o

Error - Lig transverse spacing exceeds max. of 450mm (488mm)

Torsion spacing - Cl 8.3.3

			Lig area (Asw) =	113	mm²
Max. cts = 0.12*uh =	198	mm < 300mm	Adopted ligs area (Asw) =	113	mm²
Max. cts =	198	mm	Asw.min =	27	mm² (5.9mm dia.)

Tensile reinforcement within tensile D/2 (Used for General Method and additional longitudinal forces):

Ast Req'd for M* =	236 mm² (1.2-N16)	Bottom bars in tension		
2-N16 =	402 mm²			
Design (Ast) =	1005 mm² (5-N16)	Design (Asc) =	603 mm² (3-N16)
Manual (Ast) =	mm² (blank = design)	Manual (Asc) =	mm² (blank = design)
Adopted (Ast) =	1005 mm² (5-N16)	Adopted (Asc) =	603 mm² (3-N16)
Ast req'd for Mmax* =	236 mm² (1.2-N16)		

Concrete contribution to shear strength - Cl 8.2.4

do = ds.max =	380	mm	
Dist. to centroid of D/2 tensile reinf't (ds) =	318	mm	(Positive bending, bottom bars in tension)
Effective shear depth dv = max(0.72*D,0.9*ds) =	324	mm	Cl 8.2.1.9
Effective flange (bv) =	600	mm	

Simple method for kv & θv - Cl 8.2.4.3

Asv < Asv.min, kvo = 200/(1000+1.3*dv) =	0.141		Eq 8.2.4.3(1)
Asv < Asv.min, kv = min(kvo, 0.15) =	0.141		Eq 8.2.4.3(1)
Angle of inclination of concrete comp. strut (θv) =	36.0	°	
Long. mid-depth concrete strain (εx) = 0.85*fsy/(2*Es) =	1062.5	$x10^{-6}$	

General method for kv & θv - Cl 8.2.4.2

Tensile area of concrete (Act) =	135000	mm²	Area between mid depth and tensile fibre
Adopted (Ast) =	1005	mm²	
Shear only εx1 ≤ $3000x10^{-6}$ =	398.4	$x10^{-6}$	Eq 8.2.4.2.2(1)
Shear only $-200x10^{-6}$ ≤ εx2 (where εx1 < 0) ≤ 0 =	0.0	$x10^{-6}$	Eq 8.2.4.2.2(2)
Shear/torsion εxt1 ≤ $3000x10^{-6}$ =	2054.7	$x10^{-6}$	Eq 8.2.4.2.3(1)
Shear/torsion $-200x10^{-6}$ ≤ εxt2 (where εxt1 < 0) ≤ 0 =	0.0	$x10^{-6}$	Eq 8.2.4.2.3(2)
Long. mid-depth concrete strain (εx) =	2054.7	$x10^{-6}$	Cl 8.2.4.2.3
kdg = max(32/(16+dg), 0.8) =	1.000		Eq 8.2.4.2(3)
Asv < Asv.min, kv = (0.4/(1+1500*εx))*(1300/(1000+kdg*dv)) =	0.096		Eq 8.2.4.2(2)
Angle (θv) = (29+7000*εx) =	43.4	°	Eq 8.2.4.2(1)

Adopted kv & θv - General method - Cl 8.2.4.2

kv =	0.096	
Angle of inclination of concrete comp. strut (θv) =	43.4	°

Concrete contribution to shear stength - Cl 8.2.4.1

Strength reduction factor (ø) =	0.7		Table 2.2.2(e)
kv =	0.096		General method - Cl 8.2.4.2
Effective flange (bv) =	600	mm	
Effective shear depth (dv) =	324	mm	Cl 8.2.1.9
min(√f'c,8.0) =	5.66	MPa	
Vuc = kv*bv*dv*min(√f'c, 8.0) =	105.8	kN	Eq 8.2.4.1
300 < D < 650mm, ks = (1000-D)/700 =	0.79		
(ø=0.7)Vuc =	74.1	kN	
(ks=0.79)*(ø=0.7)Vuc =	58.2	kN	Eq 8.2.1.6(1)

Transverse shear and torsion reinforcement contribution - Cl 8.2.5

Strength reduction factor (ø) =	0.7		Table 2.2.2(e)	
øVuc =	74.1	kN		
Torsion considered, V* =	63.0	kN	Cl 8.2.1.2	
Angle between shear & tensile reinf't (αv) =	90	°		
Angle (αvr) =	1.571	rad		
Angle of inclination of concrete compression strut (θv) =	43.4	°	General method - Cl 8.2.4.2	
Angle (θvr) =	0.757	rad		
Vus = (Asv*fsy.f*dv/s)*cot(θv) =	0.0	kN	Eq 8.2.5.2(1)	
øVus =	0.0	kN		
øVu = øVuc + øVus =	74.1	kN		
Ultimate shear strength (øVu = øVuc) =	74.1	kN	Cl 8.2.3.1	OK (0.85)
øVus.req'd = V* - øVuc =	0.0	kN		
Vus.reqd =	0.0	kN		
Asv.reqd = Vus/((Asv*fsy.f*dv/s)*cot(θv)) =	0	mm² (0.0mm dia.)		Eq 8.2.5.2(1)
s.reqd = (Asv*fsy.f*dv/Vus)*cot(θv) =	0	mm		Eq 8.2.5.2(1)

Minimum transverse shear reinforcement - Cl 8.2.1.7

σmin = 0.08*√f'c =	0.45	MPa	Cl 8.2.1.7
Asv.min = σmin*bv*s/fsy.f =	54	mm²	Eq 8.2.1.7
s.max = Asv*fsy.f/(σmin*bv) =	417	mm	
Vus.min = (Asv.min*fsy.f*dv/s)*cot(θv) =	93.1	kN	Eq 8.2.1.7
øVu.min = ø(Vuc + Vus.min) =	139.2	kN	

Maximum shear strength limited by web crushing - Cl 8.2.3.3

kc =	0.55		Cl 8.2.3.3(1)
Vu.max = kc*[0.9*f'c*bv*dv*((cot(θv)+cot(αv))/(1+cot²(θv)))]	1537.2	kN	Eq 8.2.3.3(1)
øVu.max = (ø=0.7)*Vu.max =	1076.0	kN	Table 2.2.2(e)(ii)
øVus.max = øVu.max - øVuc =	1002.0	kN	
Asv.max = øVus.max/(ø(fsy.f*dv/s)*cot(θv)) =	835	mm²	Eq 8.2.5.2(2)
when s =	100	mm	

Torsional strength of a beam without reinforcement - Cl 8.2.1.2

Outside perimeter of concrete cross-section (uc) =	2100 mm	Cl 8.2.1.2	
Area enclosed by oustide perimeter (Acp) =	270000 mm²		
Torsional capacity of concrete - Eq 8.2.1.2(2):			
Strength reduction factor (ø) =	0.70	Table 2.2.2	
σcp =	0.00 MPa	Cl 8.2.1.2	
Tcr = 0.33*√f'c*(Acp²/uc)√(1+(σcp/(0.33*√f'c))) =	64.8 kNm	Eq 8.2.1.2(2)	
øTcr =	45.4 kNm		
0.25*øTcr =	11.3 kNm		
Consider torsional effects (T*>0.25*øTcr) =	Y (Y)es,(N)o	Cl 8.2.1.2	
T*/0.25øTcr =	6.79	Cl 8.2.1.2(1)	Torsion reinf't req'd (6.79)

Torsional strength of a beam including reinforcement - Cl 8.2.5.6

x = Min(D & W) =	450 mm	Dxyo =	338 mm	xo =	338 mm
y = Max(D & W) =	600 mm	Wxyo =	488 mm	yo =	488 mm

Perimeter of torsion ligs (centreline) (uh = 2*(xo + yo)) =	1652 mm	Cl 8.2.3.3	
Area enclosed by ligs (Aoh = xo*yo) =	164944 mm²	Cl 8.2.3.3	
Enclosed area by shear flow (Ao = 0.85*Aoh) =	140202 mm²	Cl 8.2.5.6	
Angle of inclination of concrete compression strut (θv) =	43.4 °		
Angle (θvr) =	0.757 rad		
Area of ligs (Asw) =	113 mm²		
Tus = 2.0*Ao*Asw*fsy.f*/s*cot(θv) =	167.8 kNm	Eq 8.2.5.6	
øTus =	117.4 kNm		
T*/øTus =	0.66		OK (0.66)

Minimum torsional reinforcement - Cl 8.2.5.5

Tcr =	64.8 kNm	
Asw.min1 = Asv.min/legs =	27 mm²	Cl 8.2.5.5(b)(i) & Eq 8.2.1.7
Asw.min2 = 0.25*Tcr/(2.0*Ao*fsy.f/s*cot(θv)) =	11 mm²	Cl 8.2.5.5(b)(ii) & Eq 8.2.5.6
Asw.min = max(Asw.min1, Asw.min2) =	27 mm²	
s.max1 = Asv*fsy.f/(σmin*bv) =	417 mm	Cl 8.2.5.5(b)(i) & Eq 8.2.1.7
s.max2 = (2.0*Ao*Asw*fsy.f*cot(θv))/0.25*Tcr =	1036 mm	Cl 8.2.5.5(b)(ii) & Eq 8.2.5.6
s.max = min(s.max1, s.max2) =	417 mm	N12-417 cts min.
Tus.min = 2.0*Ao*Asw.min*fsy.f*/s*cot(θv) =	40.3 kNm	Eq 8.2.5.6
øTu.min =	28.2 kNm	

Shear and torsional strength limited by crushing - Cl 8.2.3.3

V*/(bv*dv) =	0.324 MPa		
T*.*uh/(1.7*Aoh²) =	2.750 MPa		
Vcombσ* = √[(V*/(bv*dv))²+(T*.*uh/(1.7*Aoh²))²] =	2.769 MPa	Eq 8.2.3.3(5)	
øVu.max =	1076.0 kN		
Vcombσ.limit = øVu.max/(bv*dv) =	5.535 MPa	Eq 8.2.3.3(5)	
Vcomb* = Vcombσ*.*(bv*dv) =	538.4 kN		
Vcombσ*/Vcombσ.limit or Vcomb*/øVu.max =	0.50		OK (0.50)

Additional longitudinal tension forces caused by shear and torsion - Cl 8.2.7 (Not applicable)

M* =	31.5	kNm	
V* =	63.0	kN	
T* =	77.0	kNm	
N* =	0.0	kN	
Steel area req'd (Ast.reqd) =	236	mm²	
Adopted area (Ast) =	1005	mm²	
Lever z = ds*(1-fsy*Ast/(2*α2*f'c*bw*ds)) =	302	mm	
Compressive steel area (Asc) =	603	mm²	
uo = 0.92*uh =	1520	mm	
0.5*(V*+øVuc) =	68.5	kN > V* (Limited)	
Shear contribution ΔFtds* = V*.*cot(θv) =	66.7	kN	Eq 8.2.7(2)
Torsion contribution ΔFtdt* = 0.5*T*.*uo/(2*Ao)*cot(θv) =	220.8	kN	Eq 8.2.7(3)
ΔFtd* = ΔFtds + ΔFtt* =	287.5	kN	Eq 8.2.7(1)
Flexural tension side	Normal ductility reinforcement		
Strength reduction factor in bending (øbt) =	0.85		
Strength reduction factor on tension (øtt) =	0.85		Table 2.2.2(b)
Flexural tensile side = Ttd* = (M*/z-N*/2+ΔFtd*) =	391.9	kN	Eq 8.2.8.2(1)
Ast = ΔTtd*/(øtt*fsy) =	922	mm²	Eq 8.2.8.2(2)
Add'l Ast = min(Max. Ast.Req'd, Ast) - Ast prov'd =	0	mm²	
Add'l reinf't req'd =	0.0-N16		
Flexural compression side	Normal ductility reinforcement		
Strength reduction factor in bending (øbc) =	0.85		
Strength reduction factor on compression side (øtc) =	0.85		Table 2.2.2(b)
Flexural compression side = Tcd* = (-M*/z-N*/2+ΔFtd*) =	183.0	kN	Eq 8.2.8.3(1)
Asc = ΔTcd*/(øtc*fsy) =	431	mm²	Eq 8.2.8.3(2)
Add'l Asc = Asc - Asc prov'd =	0	mm²	
Add'l reinf't req'd =	0.0-N16		

MAGRYN

RETAINING WALL CALCULATIONS - BORED PIERS ONLY

RETAINING WALL MARK *BW1 Bored Piers*

Common Geometry

Calculation increment		0.6	m
Height range, lower (multiple of increment)		0.4	m
Height range, upper (multiple of increment)		1.6	m
Ineffective soil depth (undermining)		0.60	m
Post Spacing	a	2.03	m
Backfill slope	β	0	o
		0.000	rad
Bored pier diameter, primary	B	0.45	m
Bored pier diameter, larger alternative		0.6	m
Wall height threshold for pier size		2.2	

Additional Loads

Surcharge pressure, dead	q_G	0	kPa
Surcharge pressure, live	q_Q	5	kPa
Hydrostatic pressure		0	kN/m^3
Fence height	h_f	1.8	m
Site wind speed, ultimate	V_{sit}	37.4	m/s
Site wind pressure	q_z	0.839	kPa
Aerodynamic shape factor C,pn on fence	$C_{p,n}$	1.2	
Design wind pressure	p_w	1.01	kPa
Balustrade handrail height		0.00	m
Balustrade, top edge UDL		0.00	kN/m
Balustrade, top edge PL		0.00	kN
Balustrade, infill UDL		0.00	kPa
Include balustrade forces for H < 1.0m?		No	

Parameters

Founding material		Cohesive (i.e. clay)	
Soil bearing capacity (Rutledge method)	SS	100	kPa
Soil undrained shear strength (Broms Method)	Su	50	kPa
Soil unit weight	g_{soil}	18	kN/m^3
Soil characteristic internal friction angle	f	30	o
		0.524	rad
Soil backfill uncertainty level		Uncontrolled fill	
	F_{uf}	0.75	
Design Internal Friction Angle	f*	0.41	rad
External friction angle	d	0.175	rad
Wedge Angle	J	0.990	rad
Active Pressure Coefficient	K_{ah}	0.431	
Active Pressure Coefficient, Surcharge	$K_{a,q}$	0.589	

Factors of Safety

Earth Thrust FOS	1.25
Surcharge, dead FOS	1.25
Surcharge, live FOS	1.50
Surcharge Yc	0.60
Hydrostatic FOS	1.00
Wind FOS	1.00

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RETAINING WALL MARK: *BW1 Bored Piers*

Wall Height

Level difference h =	m	0.40	1.00	1.60

Working Forces and Moments

Depth of footing adding to load	m	0.45	0.45	0.45	0.45	0.45
Additional depth (undermining allowance)	m	0.60	0.60	0.60	0.60	0.60
Active Thrust P_a = 0.5 * Ka * gsoil * h² * a	kN	5.69	16.57	33.11		
M_{pa} = Pa * h/3	kNm	2.47	10.49	27.59		
Live Surcharge Force P_{qQ} = q_Q* Ka * h * a	kN	5.08	8.66	12.25		
M_{pqQ} = Pq_Q * h/2	kNm	2.92	7.58	14.39		
Wind force P_w = pw * hf * a	kN	3.68	3.68	3.68		
M_w = Pw * (h + hf/2)	kNm	6.99	9.20	11.41		

Ultimate Moment at Top of Bored Pier

1) 1.25(MPa+MPq_G)+1.5(MPq_Q)+1.0(MPh)	kNm	7.46	24.48	56.08
2) 1.25(MPa+MPq_G)+1.5(MPb)+1.0(MPh)	kNm	3.08	13.11	34.49
3) 1.25(MPa+MPq_G)+MPw+Yc MPq_Q+1.0(MPh)	kNm	11.83	26.86	54.53
max M_d	**kNm**	**11.83**	**26.86**	**56.08**

Ultimate Shear Force at Top of Bored Pier

1) 1.25(Pa+Pq_G)+1.5(Pq_Q)+1.0(Ph)	kN	14.73	33.70	59.76
2) 1.25(Pa+Pq_G)+1.5(Pb)+1.0Ph	kN	7.12	20.71	41.39
3) 1.25(Pa+Pq_G)+ Pw + Yc Pq_Q + 1.0 Ph	kN	13.84	29.59	52.42
max P_d	**kN**	**14.73**	**33.70**	**59.76**

Design for Lateral Loading

Methods of analysis: 1) RUTLEDGE METHOD : P/SS = BD² / (2.34D + 2.64(Md/Pd))
2) BROMS METHOD, COHESIVE : D = 1.5B + P/(9SuB) + (P((Md/Pd) + 1.5B + P/(9SuB))/(2.25SuB))^0.5
3) BROMS METHOD, COHESIONLESS: γB(1/ka)D³ - 2Pd(Md/Pd) - 2PdD = 0

Method of analysis used =		1	2	2
Pier Diameter, B =	**m**	**0.45**	**0.45**	**0.45**
Required Pier Depth + Undermining, D =	**m**	**1.95**	**2.46**	**3.01**

Concrete Pile Design - Bored Pier - cohesive soil

In accordance with AS 2159-2009

Max forces in bored pier - 1.6m wall height

Loads

Design Moment M* =	56.1	kNm			
Design Shear Force P* =	60.0	kN	Lever arm e =	0.935	m

Geometry | *Soil Parameter*

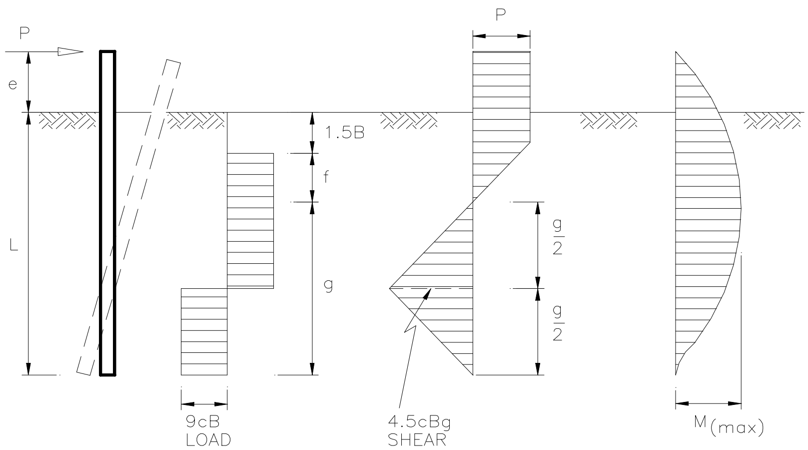
Pier Diameter d =	0.45	m	Undrained Shear Strength, s_u =	50	kPa

Design for Lateral Loading Broms Free Head Piles in Cohesive Soil

f = P/9cB	0.30	m
M_{max} = P(e + 1.5B + 0.5f) =	**105.5**	**kNm**
g =	1.44	m
V_{max} = 4.5*c*d*g =	**146.2**	**kN**

$$g = \sqrt{\frac{M_{max}}{2.25\,cB}}$$

$$L_{req} = 1.5B + f + g$$



Column: **(BW1 - Bored Pier, 1.6m high) 450mm diameter, f'c=40MPa**
Reinf't: **6-N20, 50mm cover, R10 ties at 300mm (Rest.)**
Capacity: **øNuo = 4086kN, øMuby = 165kNm, øMubx = 166kNm**
øNu = 2911kN at min. ecc., øMuy = 66kNm, øMux = 65kNm
Core confinement not required - Cl 10.7.3.1

Geometry

Refer [Confinement] for earthquake restraint - N/A

Concrete strength (f'c) =	40 MPa		Min. Ecc.y (0.05*cX) =	23 mm Cl 10.1.2
Diameter (cX=D) =	450 mm		Min. Ecc.x (0.05*cY) =	23 mm Cl 10.1.2
Circle (cY=D, > 0 for Rect.) =	mm		Tie diameter =	10 mm
			Min. tie diameter =	6 mm Cl 10.7.4.3(a)
Cover to ties (cover) =	50 mm		Max. spacing (Rest.) =	300 mm Cl 10.7.4.3(b)(i)
Bar size (db) =	20 mm			
No. bars =	6			
			Clear gap (ccx) =	142 mm
Steel yield strength (fsy) =	500 MPa (Class N)		Bar spacing (scx) =	162 mm (> 150mm *)
Ductility class =	N (N)ormal,(L)ow			*all bars to be restrained
Axis distance (as) =	70.0 mm Cl 5.2.2			
Total number of bars =	6		Dia. = 450mm	
Bar area (Ab) =	314 mm²			
Total steel area (Ast) =	1885 mm²			
Gross column area (Ag) =	159043 mm²			
Column concrete area (Ac) =	157158 mm²			
Reinforcement ratio (p=Ast/Ag) =	1.19 %			
Formwork =	S (S)tandard,(R)igid		alpha (α1) =	0.850 (0.72 ≤ α1 ≤ 0.85) Eq 10.6.2.2
Exposure classification =	B2 Table 4.10.3.2		alpha (0.95*α2) =	0.751 (α2 ≥ 0.67) Eq 10.6.2.5(1),Not
FRP =	Refer Cl 5.6		gamma (γ) =	0.870 (γ ≥ 0.67) Eq 10.6.2.5(2)
			$E_c = \rho^{1.5}*0.043*\sqrt{fcmi}$ =	32668 MPa ± 20% Cl 3.1.2

X - - - - - - X

Capacity

About X-X

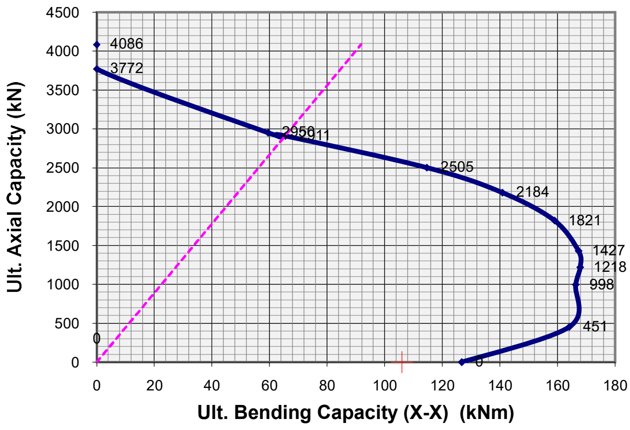
	øNuo	e.min	Decomp ku at bottom	Limit ku at dmaxx	Balanced ku=kuby	Ductile	øMuox	
øNux	4086	2911	2950	2505	998	451	0	kN
øMux	0	65.5	59.5	114.7	166.3	164.1	126.8	kNm
Ecc.x	0	23	20	46	167	364	∞	
kux	∞	1.166	1.184	1.000	0.545	0.360	0.353	kuox = kudx/deffx
						deffx =	275.5	mm

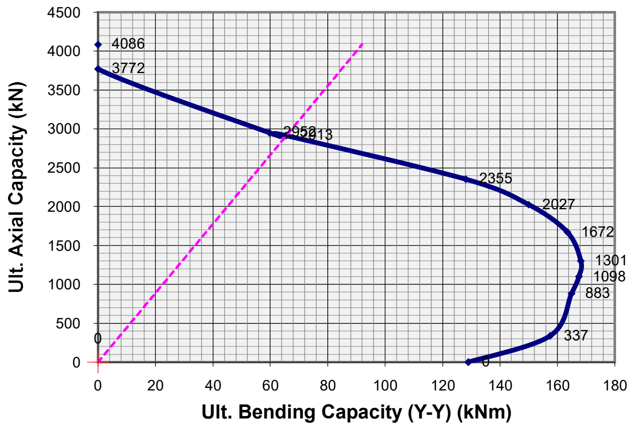
About Y-Y (Axis rotated for circular bar setout)

	øNuoy	e.min	Decomp ku at bottom	Limit ku at dmaxy	Balanced ku=kuby	Ductile	øMuoy	
øNuy	4086	2913	2952	2355	883	337	0	kN
øMuy	0	65.5	59.7	128.1	165.0	157.4	128.9	kNm
Ecc.y	0	23	20	54	187	467	∞	
kuy	∞	1.233	1.253	1.000	0.545	0.360	0.328	kuoy = kudy/deffy
						deffy =	275.5	mm

øNuo = ø(=0.65)*(α1*f'c*(Ag-Ast)+fsy*Ast) = 4086 kN øNuot = 801 kN (If all bars are anchored - Class N)

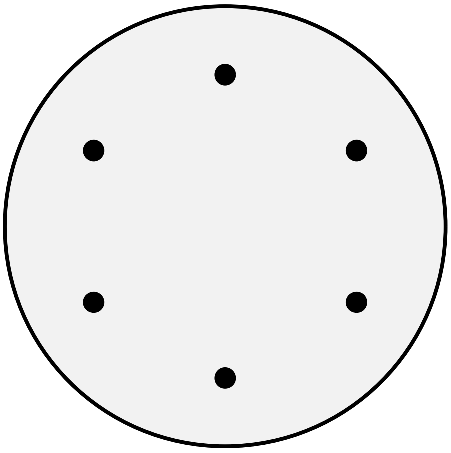
Graphs





Column: (BW1 - Bored Pier, 1.6m high) 450mm diameter, f'c=40MPa
Reinf't: 6-N20, 50mm cover, R10 ties at 300mm (Rest.)

Preview

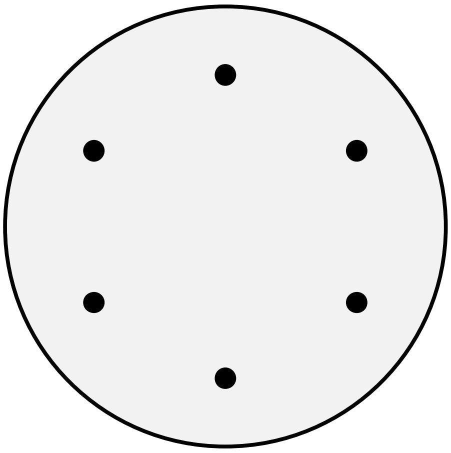


Bar	X	Y	No. Bars	Spacing	Area
	mm	mm		mm	Asi - mm2
1	225	70	1	142	314
2	359	148	1	142	314
3	359	303	1	142	314
4	225	380	1	142	314
5	91	303	1	142	314
6	91	148	1	142	314
7	0	0	0	0	0
8	0	0	0	0	0
9	0	0	0	0	0
10	0	0	0	0	0
11	0	0	0	0	0
12	0	0	0	0	0
13	0	0	0	0	0
14	0	0	0	0	0
15	0	0	0	0	0
16	0	0	0	0	0
17	0	0	0	0	0
18	0	0	0	0	0
19	0	0	0	0	0
20	0	0	0	0	0
21	0	0	0	0	0
22	0	0	0	0	0
23	0	0	0	0	0
24	0	0	0	0	0
25	0	0	0	0	0
26	0	0	0	0	0
27	0	0	0	0	0
28	0	0	0	0	0
29	0	0	0	0	0
30	0	0	0	0	0
31	0	0	0	0	0
32	0	0	0	0	0

CONCRETE COLUMN V5.31 Magryn & Associates

Column: (BW1 - Bored Pier, 1.6m high) 450mm diameter, f'c=40MPa
Reinf't: 6-N20, 50mm cover, R10 ties at 300mm (Rest.)

Preview



Bar	X	Y	No. Bars	Spacing	Area
	mm	mm		mm	Asi - mm2
33	0	0	0	0	0
34	0	0	0	0	0
35	0	0	0	0	0
36	0	0	0	0	0
37	0	0	0	0	0
38	0	0	0	0	0
39	0	0	0	0	0
40	0	0	0	0	0
41	0	0	0	0	0
42	0	0	0	0	0
43	0	0	0	0	0
44	0	0	0	0	0
45	0	0	0	0	0
46	0	0	0	0	0
47	0	0	0	0	0
48	0	0	0	0	0
49	0	0	0	0	0
50	0	0	0	0	0

BW1 - Bored Pier, 1.6m high

CONCRETE COLUMN V5.31 Magryn & Associates

Column:	**(BW1 - Bored Pier, 1.6m high) 450mm diameter, f'c=40MPa (Subject to no load)**	
Reinf't:	**6-N20, 50mm cover, R10 ties at 300mm (Rest.)**	
About X-X:	**Lx = 3000mm, kex = 1.00, N* = 0kN, Mx* = 106kNm < øMux = 127kNm**	**OK (0.84)**
About Y-Y:	**Ly = 3000mm, key = 1.00, N* = 0kN, My* = 0kNm < øMuy = 129kNm**	**OK (0.00)**
Uniaxial:	**Circular column uniaxially resolved Mrt* = 106kNm, Mrb* = 0kNm**	**No Good (1.09)**
Confinement:	**Special core confinement not required - Cl 10.7.3.1**	
Seismic:	**Not considering seismic loadings**	

Loading

Dead load (Ndl = G) =	0	kN (-ve tension)	Load type =	F	(F)loor,(S)tore,(R)oof,(O)ther
Live load (Nll = Q) =	0	kN			
Ult. earthquake (Neu*=Eu*) =	0	kN	Earthquake LL factor (Ψe) =	0.30	AS/NZS 1170.0 - Table 4.1
Ult. Wind (Nwu*=Wu*) =	0	kN			
N* = 1.2*G+1.5*Q =	0	kN	Comb. LL factor (Ψc) =	0.40	AS/NZS 1170.0 - Table 4.1
Ne* = G+Eu+Ψe*Q =	0	kN < Critical	Max. Ly =	2813	mm (Max. Ley = 2813mm, key = 1.00) (Braced)
Nw* = 1.2*G+Wu+Ψc*Q =	0	kN < Critical	Max. Lx =	2813	mm (Max. Lex = 2813mm, kex = 1.00) (Braced)
G/(G+Q) =	#DIV/0!		Min. Ast =	0	mm^2 = 0.00% Cl 10.7.1
Q/G =	1.00		N*/øNuo =	0.000	

About X-X (Weak axis), Slender column (Single c., Braced) - OK (0.84)

Braced, Slender column (Lex/rx > 25.0)

Lx =	3000	mm	rx =	113	mm	For a short column		
kex =	1.00	Cl 10.5.3	Lex/rx =	26.7		Max lex/rx =	25.0	Cl 10.3.1
Lex =	3000	mm				αc =√(1/(3.5*N*/øNuo) =	0.00	
			Min. Mx* =	0.0	kNm	βdx =	0.00	Cl 10.4.3
M.top* =	106	kNm	Mx* =	106.0	kNm	dox =	380	mm
M.bot* =		kNm	Ecc.x =	0	mm	ø(0.60)Mcx =	166	kNm
Type =	B	(B)raced/(U)nbraced				Buckling (Ncx) =	12616	kN
Transverse load =	N	(Y)es,(N)o	δbx =	1.00	Eq 10.4.2			
Circle (cY=D) =	450	mm				M1*/M2* =	0.00	Single c.
Min. Ecc. =	22.5	mm	Design Mx* =	106.0	kNm	kmx =	0.60	
			Mux =	149.2	kNm			
			øMux =	126.8	kNm	øx =	0.85	

Confinement: Special core confinement not required (X-X) - Cl 10.7.3.1 (0.00)

Confinement limit (øNconfx) =	1145	kN
Not Req'd (N*/øNconfx) =	0.00	(f'c ≤ 50MPa)
Special confinement where Mx* > 0.6*øMux =	76	kNm
1.2*cY =	540	mm

About Y-Y (Strong axis), Slender column (Single c., Braced) - OK (0.00)

Braced, Slender column (Ley/ry > 25.0)

Ly =	3000	mm	ry =	113	mm	For a short column		
key =	1.00	Cl 10.5.3	Ley/ry =	26.7		Max ley/ry =	25.0	Cl 10.3.1
Ley =	3000	mm				αc =√(1/(3.5*N*/øNuo) =	0.00	
			Min. My* =	0.0	kNm	βdy =	0.00	Cl 10.4.3
My.top* =		kNm	My* =	0.0	kNm	doy =	359	mm
My.bot* =		kNm	Ecc.y =	0	mm	ø(0.60)Mcy =	165	kNm
Type =	B	(B)raced, (U)nbraced				Buckling (Ncy) =	11829	kN
Transverse load =	N	(Y)es,(N)o	δby =	1.00	Eq 10.4.2			
Diameter (cX=D) =	450	mm				M1*/M2* =	0.00	Single c.
Min. Ecc. =	22.5	mm	Design My* =	0.0	kNm	kmy =	0.60	
			Muy =	151.7	kNm			
			øMuy =	128.9	kNm	øy =	0.85	

Confinement: Special core confinement not required (Y-Y) - Cl 10.7.3.1 (0.00)

Confinement Limit (øNconfy) =	3064	kN
Not Req'd (N*/øNconfy) =	0.00	(f'c ≤ 50MPa)
Special confinement where My* > 0.6*øMuy =	77	kNm
1.2*cX =	540	mm

Biaxial Check - Cl 10.6.3 - Circular column uniaxially resolved

cx/cy =	1.00		øbiy =	0.65
Mrt* = √(Mxt*²+Myt*²) =	106	kNm	øbix =	0.65
Mrb* = √(Mxb*²+Myb*²) =	0	kNm		
Uniaxial ratio top = Mrt*/min(øbi*Mux,øbi*Muy) =	1.09	Eq 10.6.4	< (Critical)	No Good (1.09)
Uniaxial ratio bot = Mrb*/min(øbi*Mux,øbi*Muy) =	0.00	Eq 10.6.4		OK (0.00)

Earthquake actions - Axial load limit for walls (For elements with µ > 1) - Cl 14.4.4.3 - Not applicable (Not considering seismic loading)

Earthquake loading/Seismic region =	N	(Y)es,(N)o
Structural ductility factor (µ) =	-	Table 14.3
Design as wall =	N	(Y)es,(N)o
Increased vertical load (Nve) =	0	kN (from vert. ground acceleration or frame action - Cl 14.4.4.2)
Seismic weights = Ne0* = G+Nve+Ψe*Q =	0	kN - AS 1170.4 Cl 6.2.2
Axial load limit for elements with µ > 1 = Ne0*/Ag =	0.2	*Ag - Cl 14.4.4.3
Seismic load ratio = Ne0*/Ag =	0.0	MPa
Axial load limit = 0.2*f'c =	8.0	MPa
Seismic weights = Ne0* = G+Nve+Ψe*Q =	0	kN - AS 1170.4 Cl 6.2.2
Axial load limit = øNe0.max = 0.2*f'c*Ag =	1272.345025	kN OK (0.00)

Axial shortening

Column Length =	3000	mm	
Non-domestic concent'd load =	Y	(Y)es,(N)o	
Short term LL factor (Ψs) =	1.00		AS/NZS 1170.0 - Table 4.1
Long term LL factor (Ψl) =	0.60		AS/NZS 1170.0 - Table 4.1
Modular ratio (n = Es/Ec) =	6.12		
Transformed area (At = Ag + Ast*(n-1)) =	168698	mm²	
Short term shortening (δs = (G+Ψs*Q)*L/(Ec*At) =	0.00	mm	
Long term shortening (δl = (G+Ψl*Q)*L/(Ec*At) =	0.00	mm	(Excluding creep and shrinkage)

Design parameters

Steel modulus (Es):

Custom =	N	(Y)es,(N)o
Es =	200000	MPa (Default = 200000MPa)
Es =	200000	MPa

Top fibre strain (εc):

Custom =	N	(Y)es,(N)o
εc =	0.003	(Default = 0.003)
εc =	0.0030	

Concrete Modulus (Ec): Cl 3.1.2

Ec Method =	i	(f)'c,fcm(i),(M)anual
f'c =	40	MPa
Ec(f'c) =	31975	MPa
fcmi =	43.3	MPa
Ec(fcmi) =	32668	MPa
Adopted Ec =	32668	MPa

Column:	**(BW1 - Bored Pier, 1.6m high) 450mm diameter, f'c=40MPa**	
Reinf't:	**6-N20, 50mm cover, N10 ties at 200mm cts**	
	Ligs required	
Strength:	**Vy* = 146.0kN > ksy*øVucy = 93.8kN, Vy* = 146.0kN ≤ øVuy = 267.2kN - Shear ligs required**	**OK (0.55)**
	øVucy = 128.0kN, øVuy.min = 208.7kN, øVusy = 139.3kN	
	Vx* = 0.0kN ≤ ksx*øVucx = 202.9kN, Vx* = 0.0kN ≤ øVux = 448.8kN - No shear ligs required	**OK (0.00)**
	øVucx = 276.6kN, øVux.min = 376.4kN, øVusx = 172.2kN	
Torsion:	**T* = 0.0kNm ≤ øTusx = 0.0kNm & øTusy = 0.0kNm - No torsion ligs required**	**OK (0.00,0.00)**
	0.25øTcr = 7.0kNm, øTcr = 28.0kNm	
Combined:	**Vcombx* = 0.0kN ≤ øVux.max = 856.9kN, Vcomby* = 146.0kN ≤ øVuy.max = 942.3kN**	**OK (0.15,0.00)**
Add'l reinf't:	**Additional tensile shear reinforcement not required (Y-Y)**	
	Additional tensile shear reinforcement not required (X-X)	

Geometry

	About X-X			About Y-Y	
Diameter (Dy=cY=D) =	450	mm	Diameter (Dx=cX=D) =	450	mm
Effective flange (bvy=cX) =	450	mm	Effective flange (bvx=cX) =	450	mm

Strength of beams in shear - Cl 8.2 - Design actions

	About X-X			About Y-Y		
Design shear (Vy*) =	146	kN	Design shear (Vx*) =	0	kN	
Design moment (Mx*) =	0	kNm	Design moment (My*) =	0	kNm	
Design section max. moment (Mmaxx*) =	0	kNm	Design max. (Mmaxy*) =	0	kNm	Cl 8.2.8.2
Axial (N*) =	0	kN (Tension -ve)				
Torsion (T*) =	0	kNm				
Short and Q/G ≥ 0.25 (X-X) =	N	(Y)es,(N)o	Short and Q/G ≥ 0.25 (Y-Y) =	N	(Y)es,(N)o	

Consideration of torsion - Cl 8.2.1.2 No torsion - Torsional reinf't not required

øTcr = ø*0.33*√f'c*(Acp²/uc)√(1+(σcp/(0.33*√f'c))) =	28.0 kNm	Cl 8.2.1.2(2) - Refer below	
0.25*øTcr =	7.0 kNm	Cl 8.2.1.2(1)	
Consider torsional effects (T* > 0.25*øTcr) =	N (Y)es,(N)o	Cl 8.2.1.2	
T*/0.25øTcr =	0.00	Cl 8.2.1.2(1)	No torsion (0.00)
Torsion ignored, Vy* =	146.0 kN	Vx* =	0.0 kN

Strength of beams in shear - Cl 8.2

Use Simplified method =	N	(Y)es,(N)o	
Max. nominal aggregate size (dg) =	16	mm	(General method)
Lightweight concrete =	N	(Y)es,(N)o	(General method)

Reinforcement Note - V* ≤ øVu.min, Toggle the increase spacing option - Refer to the right

Description = **2 legs N10-200 cts (X-X),2 legs N10-200 cts**

Lig size =	10	mm (Refer Capacity)			
Spacing (s) =	200	mm	Leg area (Asl) =	79	mm²
Legs in cross-section =	2		Legs in cross-section =	2	
Lig yield strength (fsy.f) =	500	MPa			
Fitment class =	N	(N)ormal,(L)ow	Waive max. spacing =	N	(Y)es,(N)o
Angle b/n shear and long. reinf't (αv) =	90.0	°	Waive transv. spacing =	N	(Y)es,(N)o
Ligs area (Asvy) =	157	mm²	Asvx =	157	mm²
Adopted ligs area (Asvy) =	157	mm²	Asvx =	157	mm²
Asvy.min =	91	mm² (Asv/Asv.min = 1.72)	Asvx.min =	91	mm² (Asv/Asv.min = 1.72)
Asvy.req'd =	20	mm² (2-3.6mm dia.)	Asvx.req'd =	0	mm² (2-0.0mm dia.)
sy.reqd =	1544	mm	sx.reqd =	0	mm
sy.max =	345	mm	sx.max =	345	mm
sy.max.t =	0	mm	sx.max.t =	0	mm

Shear spacing - Cl 8.3.2.2

Max. cts = 0.5*cY =	225 mm < 300mm	Max. cts = 0.5*cX =	225 mm < 300mm
Max. cts =	225 mm	Max. cts =	225 mm

Transverse spacing - Cl 8.3.2.2

Transverse spacing (cX) =	340 mm	Transverse spacing (cY) =	340 mm
Max. spacing (cX) = min(600, cY) =	450 mm	Max. spacing (cY) =	450 mm

Torsion spacing - Cl 8.3.3

		Ligs area (Asw) =	79 mm^2
Max. cts = 0.12*uh =	128 mm < 300mm	Adopted ligs area (Asw) =	0 mm^2
Max. cts =	128 mm		
Aswy.min =	46 mm^2 (7.6mm dia.)	Aswx.min =	46 mm^2 (7.6mm dia.)

Tensile reinforcement within tensile D/2 (Used for General Method and additional longitudinal forces):

Ast Req'd for M* =	0 mm^2 (0.0-N20)	Ast Req'd for M* =	0 mm^2 (0.0-N20)
0-N20 =	0 mm^2	0-N20 =	0 mm^2
Adopted (Asty) =	942 mm^2 (3.0-N20)	Adopted (Astx) =	628 mm^2 (2.0-N20)
Adopted (Ascy) =	942 mm^2 (3.0-N20)	Adopted (Ascx) =	628 mm^2 (2.0-N20)
Ast req'd for Mmaxx* =	0 mm^2 (0.0-N20)	Ast req'd for Mmaxy* =	0 mm^2 (0.0-N20)

Concrete contribution to shear strength - Cl 8.2.4

doy = dsy.max =	328 mm	dox =	359 mm	
Dist. to centroid of D/2 tensile reinf't (dsy) =	328 mm	dsx =	359 mm	
Effective shear depth dvy = max(0.72*D,0.9*dsy) =	324 mm	dvx =	324 mm	Cl 8.2.1.9
Effective flange (bvy) =	450 mm	bvx =	450 mm	

Simple method for kv & θv - Cl 8.2.4.3

Asv ≥ Asvy.min, kvy = 0.15 =	0.150	kvx =	0.150	Eq 8.2.4.3(2)
Angle of inclination of concrete comp. strut (θvy) =	36.0 °	θvx =	36.0 °	

General method for kv & θv - Cl 8.2.4.2

Tensile area of concrete (Acty) =	79522 mm^2	Actx =	79522 mm^2	
Tensile steel (Asty) =	942 mm^2	Astx =	628 mm^2	
Shear only εx1y ≤ 3000x10^{-6} =	774.6 x10^{-6}	εx1x =	0.0 x10^{-6}	Eq 8.2.4.2.2(1)
Shear only -200x10^{-6} ≤ εx2y (where εx1y < 0) ≤ 0 =	0.0 x10^{-6}	εx2x =	0.0 x10^{-6}	Eq 8.2.4.2.2(2)
Shear/torsion εxt1y ≤ 3000x10^{-6} =	- x10^{-6}	εx1x =	- x10^{-6}	Eq 8.2.4.2.3(1)
Shear/torsion -200x10^{-6} ≤ εxt2y (where εxt1y < 0) ≤ 0 =	- x10^{-6}	εx2x =	- x10^{-6}	Eq 8.2.4.2.3(2)
Long. mid-depth concrete strain (εxy) =	774.6 x10^{-6}	εxx =	0.0 x10^{-6}	
kdg = max(32/(16+dg), 0.8) =	1.000			Eq 8.2.4.2(3)
Asv ≥ Asvy.min, kvy = (0.4/(1+1500*εxy)) =	0.185	kvx =	0.400	Eq 8.2.4.2(5)
Angle (θvy) = (29+7000*εxy) =	34.4 °	θvx =	29.0 °	Eq 8.2.4.2(1)

Adopted kv & θv - General method - Cl 8.2.4.2

kvy =	0.185	kvx =	0.400
Angle of inclination of concrete compression strut (θvy) =	34.4 °	θvx =	29.0 °

Concrete consribution to shear stength - Cl 8.2.4.1

Capacity reduction factor (øy) =	0.70	øx =	0.70	Table 2.2.2(e)
kvy =	0.185	kvx =	0.400	
Effective flange (bvy) =	450 mm	bvx =	450 mm	
Effective shear depth (dvy) =	324 mm	dvx =	324 mm	
min(√f'c,8.0) =	6.32 MPa			
Vucy = kvy*bvy*dvy*min(√f'c, 8.0) =	170.6 kN	Vucx =	368.8 kN	Cl 8.2.4.1
300 < Dy < 650mm, ksy = (1000-Dy)/700 =	0.79	ksx = (1000-Dx)/700 =	0.79	
(ø=0.7)Vucy =	119.4 kN	(ø=0.7)Vucx =	258.2 kN	
(ksy=0.79)*(ø=0.7)Vucy =	93.8	(ksx=0.79)*(ø=0.7)Vucx =	202.9 kN	Eq 8.2.1.6(1)

Transverse shear and torsion reinforcement contribution - Cl 8.2.5

Capacity reduction factor (øy) =	0.75		øx =	0.75		Table 2.2.2(e)
øVucy =	128.0	kN	øVucx =	276.6	kN	
Torsion ignored, Vy* =	146.0	kN	Vx* =	0.0	kN	Cl 8.2.1.2
Angle between shear & tensile reinf't (αvy) =	90	°	αvx =	90	°	
Angle (αvry) =	1.571	rad	αvrx =	1.571	rad	
Angle of inclination of concrete compression strut (θvy) =	34.4	°	θvx =	29.0	°	Cl 8.2.4.2
Angle (θvyr) =	0.601	rad	θvx =	0.506	rad	
Vusy = (Asvy*fsy.f*dvy/s)*cot(θvy) =	185.7	kN	Vusx =	229.5	kN	Eq 8.2.5.2(1)
øVusy =	139.3	kN	øVusx =	172.2	kN	
øVuy = øVucy + øVusy =	267.2	kN	øVux =	448.8	kN	
Ultimate shear strength (øVuy = øVucy + øVusy) =	267.2	kN	øVux =	448.8	kN	Cl 8.2.3.1
øVusy.reqd = Vy* - øVucy =	18.0	kN	øVusx.req'd =	0.0	kN	
Vusy.reqd =	24.0	kN	Vusx.req'd =	0.0	kN	
Asvy.reqd = Vusy/((Asvy*fsy.f*dvy/s)*cot(θvy)) =	20.3	mm^2	Asvx.reqd =	0.0	mm^2	Eq 8.2.5.2(1)
sy.reqd = (Asvy*fsy.f*dvy/Vusy)*cot(θvy) =	1544	mm	sx.req'd =	0	mm	Eq 8.2.5.2(1)

Minimum transverse shear reinforcement - Cl 8.2.1.7

σmin = 0.08*√f'c =	0.51	MPa	Cl 8.2.1.7			
Asvy.min = σmin*bvy*s/fsy.f =	91	mm^2	Asvx.min =	91	mm^2	Eq 8.2.1.7
Lig min. =	7.6	mm	Lig min. =	7.6	mm	
sy.max = Asvy*fsy.f/(σmin*bvy) =	345	mm	sx.max =	345	mm	
Vusy.min = (Asvy.min*fsy.f*dvy/s)*cot(θvy) =	107.6	kN	Vusx.min =	133.1	kN	Eq 8.2.1.7
øVuy.min = ø(Vucy + Vusy.min) =	208.7	kN	øVux.min =	376.4	kN	

Maximum shear strength limited by web crushing - Cl 8.2.3.3

kc =	0.55		Cl 8.2.3.3(1)			
…ax =kc*[0.9*f'c*bvy*dvy*((cot(θvy)+cot(αvy))/(1+cot²(θvy)))] =	1346.1	kN	Vux.max =	1224.1	kN	Eq 8.2.3.3(1)
øVuy.max = (ø=0.7)*Vux.may =	942.3	kN	øVux.max =	856.9	kN	Tab 2.2.2(e)(ii)
øVusy.max = øVuy.max - øVucy =	814.3	kN	øVusx.max =	580.2	kN	
Asvy.max = øVusy.max/(ø(fsy.f*dvy/s)*cot(θvy)) =	919	mm^2	Asvx.max =	529	mm^2	Eq 8.2.5.2(2)
when s =	200	mm	when s =	200	mm	

Torsional strength of a beam without reinforcement - Cl 8.2.1.2

Outside perimeter of conc. cross-section (uc = 2*(cX+cY)) =	1414 mm	Cl 8.2.1.2	
Area enclosed by oustide perimeter (Acp = cX*cY) =	159043 mm²		
Torsional capacity of concrete - Eq 8.2.1.2(2):			
Strength reduction factor (ø) =	0.75	Table 2.2.2	
σcp =	0.00 MPa	Cl 8.2.1.2	
Tcr = 0.33*√f'c*(Acp²/uc)√(1+(σcp/(0.33*√f'c))) =	37.3 kNm	Eq 8.2.1.2(2)	
øTcr =	28.0 kNm		
0.25*øTcr =	7.0 kNm		
Consider torsional effects (T*>0.25*øTcr) =	N (Y)es,(N)o	Cl 8.2.1.2	
T*/0.25øTcr =	0.00	Cl 8.2.1.2(1)	No torsion (0.00)

Torsional strength of a beam including reinforcement - Cl 8.2.5.6

x = Min(cX & cY) =	450 mm	Dxyo =	340 mm	yo =	340 mm
y = Max(cX & cY) =	450 mm	Wxyo =	340 mm	xo =	340 mm

Perimeter of torsion ligs (centreline) (uh = π*xo) =	1068 mm	Cl 8.2.3.3		
Area enclosed by ligs (Aoh = π*xo²/4) =	90792 mm²	Cl 8.2.3.3		
Enclosed area by shear flow (Ao = 0.85*Aoh) =	77173 mm²	Cl 8.2.5.6		
Angle of inclination of concrete compression strut (θvy) =	34.4 °	θvx =	29.0 °	
Angle (θvry) =	0.601 rad	θvrx =	0.506 rad	
Area of ligs (Asw) =	0 mm²			
Tusy = 2.0*Ao*Asw*fsy.f*/s*cot(θvy) =	0.0 kNm	Tusx =	0.0 kNm	Eq 8.2.5.6
øTusy =	0.0 kNm	øTusx =	0.0 kNm	
T*/øTusy =	0.00 OK (0.00)	T*/øTusx =	0.00 OK (0.00)	

Minimum torsional reinforcement - Cl 8.2.5.5

Tcr =	37.3 kNm			
Asw.min1y = Asv.miny/legs =	46 mm²	Asw.min1x =	46 mm²	Cl 8.2.5.5(b)(i)
Asw.min2y = 0.25*Tcr/(2.0*Ao*fsy.f/s*cot(θvy)) =	17 mm²	Asw.min2x =	13 mm²	Cl 8.2.5.5(b)(ii)
Asw.miny = max(Asw.min1y, Asw.min2y) =	46 mm²	Asw.minx =	46 mm²	
s.max1y = Asv*fsy.f/(σmin*bv) =	345 mm	s.max1x =	345 mm	Cl 8.2.5.5(b)(i)
s.max2y = (2.0*Ao*Asw*fsy.f*cot(θvy))/0.25*Tcr =	0 mm	s.max2x =	0 mm	Cl 8.2.5.5(b)(ii)
s.maxy = min(s.max1y, s.max2y) =	0 mm	s.max =	0 mm	No ligs
Tus.miny = 2.0*Ao*Asw.miny*fsy.f*/s*cot(θvy) =	25.6 kNm	Tus.minx =	31.7 kNm	
øTu.miny =	19.2 kNm	øTu.minx =	23.8 kNm	

Shear and torsional strength limited by crushing - Cl 8.2.3.3

Vy*/(bvy*dvy) =	1.001 MPa	Vx*/(bvx*dvx) =	0.000 MPa	
T*.*uh/(1.7*Aoh²) =	0.000 MPa	T*.*uh/(1.7*Aoh²) =	0.000 MPa	
Vcombσy* = √[(Vy*/(bvy*dvy))²+(T*.*uh/(1.7*Aoh²))²] =	1.001 MPa	Vcombσx* =	0.000 MPa	Eq 8.2.3.3(5)
øVuy.max =	942.3 kN	øVux.max =	856.9 kN	
Vcombσy.lim = øVuy.max/(bvy*dvy) =	6.463 MPa	Vcombσx.lim =	5.877 MPa	Eq 8.2.3.3(5)
Vcomby*=Vcombσy*.*(bvy*dvy) =	146.0 kN	Vcombx* =	0.0 kN	
Vcombσy*/Vcombσy.lim or Vcombx*/øVuy.max =	0.15 OK (0.15)	x =	0.00 OK (0.00)	

Additional longitudinal tension forces caused by shear and torsion - Cl 8.2.7

Mx* =	0.0 kNm	My* =	0.0 kNm	
Vy* =	146.0 kN	Vx* =	0.0 kN	
T* =	0.0 kNm	T* =	0.0 kNm	
N* =	0.0 kN	N* =	0.0 kN	
Steel area req'd (Asty.reqd) =	0 mm²	Asty.reqd =	0 mm²	
Adopted area (Ast) =	942 mm²	Astx =	628 mm²	
Lever zy = dsy*(1-fsy*Asty/(2*α2*f'c*bwy*dsy)) =	311 mm	zx =	348 mm	
Compressive steel area (Ascy) =	942 mm²	Ascx =	628 mm²	
uo = 0.92*uh =	983 mm			
0.5*(Vy*+øVucy) =	137.0 kN < V*	'(Vx*+øVucx) =	138.3 kN > V*	
Shear contribution ΔFtdsy* = 0.5*(Vy*+øVucy)*cot(θvy) =	199.9 kN	ΔFtdsx* =	0.0 kN	Eq 8.2.7(2)
Torsion contribution ΔFtdt* = (Ignored) =	0.0 kN	ΔFtdtx* =	0.0 kN	Eq 8.2.7(3)
ΔFtd* = ΔFtds + ΔFtt* =	199.9 kN	ΔFtdx* =	0.0 kN	Eq 8.2.7(1)
Flexural tension side				
Strength reduction factor in bending (øbt) =	0.85	øbt =	0.85	
Strength reduction factor on tension side (øtt) =	0.85	øtt =	0.85	Table 2.2.2(b)
Flexural tensile side = Ttdy* = (Mx*/zy-N*/2+ΔFtdy*) =	199.9 kN	Ttdx* =	0.0 kN	Eq 8.2.8.2(1)
Asty = ΔTtdy*/(øtt*fsy) =	470 mm²	Astx =	0 mm²	Eq 8.2.8.2(2)
Add'l Asty = Asty - Asty.Req'd =	470 mm²	Add'l Astx =	0 mm²	
Add'l Asty = Asty - Asty.Prov'd =	0 mm²	Add'l Astx =	0 mm²	
Add'l reinf't req'd =	0.0-N20	Add'l reinf't req'd =	0.0-N20	
Flexural compression side				
Strength reduction factor in bending (øbc) =	0.85	øbc =	0.85	
Strength reduction factor on compression side (øtc) =	0.85	øtc =	0.85	Table 2.2.2(b)
Flexural comp. side = Tcdy* = (-Mx*/zy-N*/2+ΔFtdy*) =	199.9 kN	Tcdx* =	0.0 kN	
Ascy = ΔTcdy*/(øtc*fsy) =	470 mm²	Ascx =	0 mm²	
Add'l Asty = Asty - Ascy =	0 mm²	Add'l Astx =	0 mm²	
Add'l reinf't req'd =	0.0-N20	Add'l reinf't req'd =	0.0-N20	

Wall:	**(BW2 - Wall Geometry) 900mm retaining ht., 190mm thick (With key)**	
Loads:	**Surcharge = 5.00kPa, Drained**	
Footing:	**800mm long (overall), 610mm overhang (behind wall), 200mm deep**	
Stability:	**M*o = 3.7kNm < øMr = 5.1kNm**	**OK (0.74)**
Sliding:	**V*slide = 8.6kN < øVr = 10.7kN**	**OK (0.80)**
Wall:	**M*wall = 2.3kNm, V*wall = 6.2kN, Design factor Φn = 1.1**	
Bearing:	**Max. bearing pressure = 49kPa < Allowable = 100kPa**	**OK (0.49)**

Geometry

Wall assumed fully drained

Retaining height (z) = 900 mm (= Wall height)
Risk class = A (A),(B),(C) - Minimal damage and loss of access - Table 1.1
Backfill type = U Class(1), Class(2), (U)ncontrolled, (I)n-situ - Table 5.1(A)

Soil parameters

Internal friction (ø) = 30 ° (0° ≤ ø ≤ 45°)
Incline of soil at top (β) = 0 ° (0° ≤ β ≤ 45°) 1 in 0.0
Soil weight (γ) = 18.0 kN/m³
Cohesion (c) = 0 kPa
Friction coefficient (μ) = 0.433 (-1 to ignore sliding)

Use passive = Y (Y)es,(N)o
Manual Kp value = 0.00 0 for default
Soil depth to Ignore = 0 mm (for passive)

Allowable bearing = 100 kPa

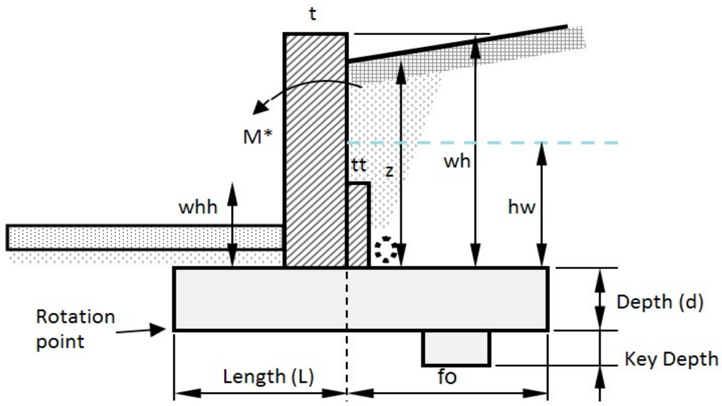
Slab overlay (ts) = mm
Soil overlay (tsl) = mm

Wall

Wall thickness (t) = 190 mm
Wall height (wh) = 900 mm (= Retaining height)
Add'l thickness (tt) = 0 mm (behind wall)
Add'l th. height (whh) = 0 mm
Add'l th. on soil side = Y (Y)es,(N)o
Wall Wt (Wwt) = 21 kN/m³

Footing

Footing depth (d) = 200 mm
End thickness (d2) = 0 mm (0 = no taper)
Footing length (L) = 190 mm excluding overhang
Overhang (fo) = 610 mm additional to length
Key depth (key) = 300 mm (sliding only)
Key length (klen) = 200 mm
Key position from heel/back (kpos) = 0 mm (to face)



Design loads - Appendix J - AS4678 & Section 4

Surcharge Min. = 2.5 kPa min. (Table 4.1)
Surcharge (S) = 5.0 kPa
Water height (hw) = 0 mm (over footing, 0=no water)

Earth factor = 1.25 (1.25)
Water pressure = 1.00 (1.00)
Live load factor = 1.50 (1.50 - 0.0 for stabilising effect)
Stabilising factor (øs) = 0.80 AS 4678 - Appendix J2

Additional Loads at top of wall: `
Overturning (Ma) = 0.00 kNm/m
Thrust (Va) = 0.00 kN/m
Load factor = 1.00
Ma* = 0.00 kNm/m
Va* = 0.00 kN/m
(Moment/Thrust about top wall)
Additional dead load (Pv) = 0.00 kN/m

Material design factors - Section 5

Φuc =	0.50	c*=c*Φuc =	0 kPa	Cl 5.2.1		
Φuo =	0.75	ø* =	23.4 °	Cl 5.2.2	ø* =	0.409 rad
Design factor (Φn) =	1.10 Table 5.2					
	Ka =	0.431		Depth of tension zone =	0 mm	
	Kp =	2.319		Acting soil height (ha) =	900 mm	

Design Moment in wall (about top of footing, per m of wall)

	Pressure kPa	Thrust kN/m	Thrust* kN/m	Lever arm mm	Mw kNm/m	Mw* kNm/m	Comments (At wall base)
Soil = Ka*γ*ha =	6.99	3.14	3.93	300	0.94	1.18	Max. pressure at base
Water = None =	0.00	0.00	0.00	0	0.00	0.00	Max. pressure at base
Surcharge = Ka*S =	2.16	1.94	2.91	450	0.87	1.31	Uniform pressure dist.
Additional =		0.00	0.00	900	0.00	0.00	
	Total	**5.08**	**6.84**		**1.82**	**2.49**	

Total thrust (Vw) =	5.08	kN/m
Total ult. thrust (Vw*) =	6.84	kN/m
Total ult. design thrust (Vw/Φn) =	6.22	kN/m
Total moment (Mw) =	1.82	kNm/m
Total ult. moment (Mw*) =	2.49	kNm/m
Total ult. design moment (Mw*/Φn) =	2.26	kNm/m

Overturning (Limit mode U2) (about base of footing, per m of wall)

Depth for rotation (deff) = 200 mm (Footing depth)

	Pressure kPa	Thrust kN/m	Thrust* kN/m	Lever arm mm	Mo kNm/m	Mo* kNm/m	Comments (At footing base)
Soil = Ka*γ*(ha+deff) =	8.54	4.70	5.87	367	1.72	2.15	Max. pressure at base
Water = None =	0.00	0.00	0.00	0	0.00	0.00	Max. pressure at base
Surcharge = Ka*S =	2.16	2.37	3.56	550	1.30	1.96	Uniform pressure dist.
Additional =		0.00	0.00	1100	0.00	0.00	
	Total	**7.07**	**9.43**		**3.03**	**4.11**	

Total thrust =	7.07	kN/m
Total ult. thrust =	9.43	kN/m
Total moment (Mo) =	3.03	kNm/m
Total ult. moment (Mo*) =	4.11	kNm/m
Total ult. (Mo*/Φn) =	3.74	kNm/m

Restoring (Limit mode U2) (about base of footing, per m of wall)

	Force kN/m	Lever arm mm	Mr kNm/m	øMr kNm/m	Ecc mm	Mb kNm/m	
Footing =	4.00	315	1.26	1.01	0	0.00	
Soil overlay =	0.00	-85	0.00	0.00	400	0.00	
Slab overlay =	0.00	-85	0.00	0.00	400	0.00	
Wall =	3.59	10	0.03	0.03	305	1.10	
Lower wall (back) =	0.00	105	0.00	0.00	210	0.00	Using Wwt-Soil dentity (γ)
Soil behind wall =	9.88	410	4.05	3.24	-95	-0.94	
Dead load (Pv) =	0.00	10	0.00	0.00	305	0.00	
Tapered =	0.00	0	0.00	0.00	400	0.00	
Key =	1.50	615	0.92	0.74	-300	-0.45	
Passive pressure =			0.06	0.04			
Total	**18.97**		**6.32**	**5.06**		**-0.29**	

Total force (Wt) =	18.97	kN/m	
Total bearing overturning (Mb) =	-0.29	kN/m	
Total restoring (Mr) =	6.32	kN/m	
Total ult. Restoring (øs(0.80)*"Mr) =	5.06	kNm/m	OK (0.74)

Sliding (Limit mode U1)

Sliding (V*slide/Φn) =	8.57	kN/m at base
Passive resistance of footing and key (Rp) =	5.22	kN/m
Sliding resistance (Rs = µ*Wt) =	8.22	kN/m
Total resistance øVr = øs(0.80)*(Rp+Rs) =	10.75	kN/m OK (0.80)

Key Forces

Pressure at top of key (k.min) =	8.3	kPa
Pressure at bottom of key (k.max) =	20.9	kPa
Top of pressure wedge =	0.0	mm (from underside of footing)
Level arm =	177.8	mm (from underside of footing)
Proportioned friction on base of key (Vf = klen/(L+fo)*Rs) =	2.05	kN/m
Key shear from passive (Vp) =	4.38	kN/m
Key shear (V*k = earth factor (1.25) * (Vp+Vf)) =	8.05	kN/m
Key moment (M*k) =	1.74	kNm/m

Bearing

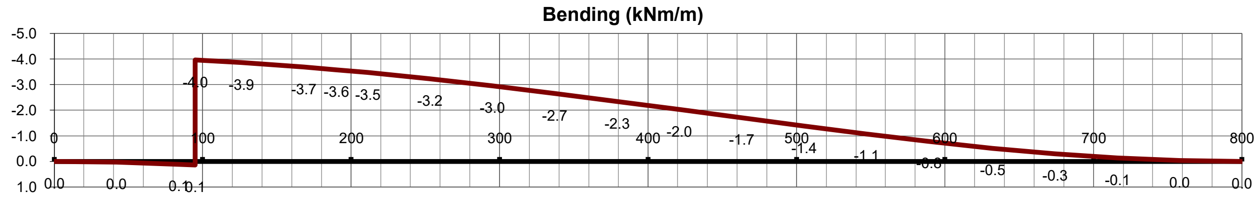
Total (Wt) =	18.97	kN/m
Overturning moment (Mot = Mo + Mb) =	2.73	kNm/m
Ecc = Mot / Wt =	144	mm
Minimum bearing pressure (q.min) =	0.0	kPa
Maximum bearing pressure (q.max) =	49.4	kPa OK (0.49)
Centroid from left/front edge of footing (y) =	256	mm (768mm of footing under pressure)

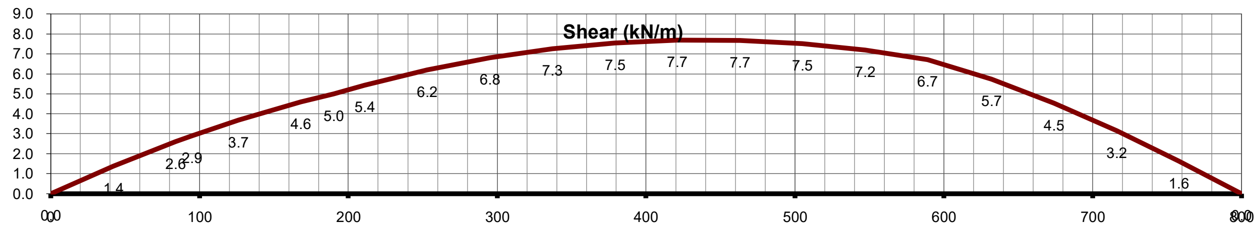
Design Moment and Shear in footing due to bearing (per m of wall)

Manual force position = 400 mm (from left/front edge of footing)
Equivalent load factor = 1.36

	Moment* kNm/m	Shear* kN/m	Lever arm mm	Position mm
Front face of wall =	0.0	0.0	-	0
Centre of wall (from the left) =	0.1	2.9	50	95
Centre of wall (from the right) =	-4.0	2.9	1369	95
Back face of wall (soil side) =	-3.6	5.0	716	190
Max. moment (+ve) =	0.1	2.9	-	95
Max. moment (-ve) =	-4.0	2.9	-	95
Max. shear =	-2.0	7.7	-	421
Manual position =	-2.2	7.6	-	400

Graphs (Footing)





MASONRY MEMBER V5.11 Magryn & Associates

Unit: **(BW2 - Wall) 20.01 Hollow block (All cores grouted) [Wall]**
Reinf't: **N12-400 cts**
Grout: **All cores grouted**

Ast < Ast.min (Comp.)

Hollow block properties (Face-shell bedded joints)

Masonry unit =	20.01		Custom =	N	(Y)es,(N)o
Unit type =	H	(S)olid, (H)ollow	Solidity ratio =	0.51	(51%)
Unit material =	C	C(L)ay, Calcium (S)ilicate with basalt ag., (C)oncrete with basalt ag.			
Percentage basaltic aggregate > 45% =	N	(Y)es,(N)o			
Thickness of unit (tu) =	190	mm			
Height of unit (hu) =	190	mm			
Length of unit (lu) =	390	mm			
Shell thickness (ts) =	30	mm			
Overlap of units (sp) =	190	mm			
Self weight =	2.2	kN/m²	Eff. width (t) =	190	mm
AAC masonry =	N	(Y)es,(N)o	Core cts =	200	mm
Stack bonded =	N	(Y)es,(N)o			
Density of block =	2250	kg/m³			

Compressive strength - Cl 3.3.2

Char. comp. strength of unit (f'uc) =	15.0	MPa	
Compressive strength factor (km) =	1.6		
Char. comp. strength of masonry (f'mb = km*√f'uc) =	6.20	MPa	Cl 3.3.2(a)(i)
hu/tj =	19.0		Cl 3.3.2(a)(i)
Joint thickness factor (kh) =	1.30		Table 3.2
Compressive strength (f'm = kh*f'mb) =	8.06	MPa	Cl 3.3.2(a)(i)

Flexural tensile strength - Cl 3.3.3

Char. lateral modulus of rupture (f'ut) =	0.8	MPa	Cl 3.2

Joint

Joint thickness (tj) =	10	mm	Cl 4.9.1
Rake (face in comp. under +ve bending) =	0	mm (Unraked)	
Rake (face in comp. under -ve bending) =	0	mm (Unraked)	

Grouting (All cores grouted)

Grouted every =	1	(All cores)	Density of grout =	2400	kg/m³
Compressive strength of grout (f'c) =	20.0	MPa			
Design comp. strength of grout (f'cg = max(12,f'c)) =	20.0	MPa	Cl 3.5		
Grout strength factor (kc) =	1.4		Cl 7.3.2 & Cl 8.5 - Hollow & ρ>2000kg/m³ =1.4 else 1.2		

Cross section properties

Bedded thickness (tb = leafs*tu) =	190	mm	Cl 4.5.1 with rake ≤ 3mm		
Bedded area (Hollow) (Ab = ts*2*1000) =	60000	mm²/m	Cl 4.5.4(b)		
Combined cross-sectional area (Grouted/reinf.) (Ac) =	190000	mm²/m	Cl 4.5.5		
Design cross section (Ad) =	190000	mm²/m	Cl 4.5.6		
Grout area (Ag = Ad - Ab) =	130000	mm²/m	Cl 4.5.7	Wall S.Wt =	4.56 kN/m²

Reinforcement

Vertical reinforcement =	**N12-400 cts**			
Bar size (Dia) =	12	mm	Ast.min = 0.002*Ad =	380 mm²/m
Reinf't cts (cts) =	400	mm	Area steel (Ast) =	283 mm²/m
Reinf't strength (fsy) =	500	MPa - Table 3.7		
Depth to steel in +ve bending from unit face (ds) =	95	mm	Reinf't central (ds) =	95 mm
Depth to steel in +ve bending (dt = d - rep) =	95	mm	Reinf't cover in block =	59.0 mm
Depth to steel in -ve bending (dc) =	95	mm		

MASONRY MEMBER V5.11 Magryn & Associates

Unit:	**(BW2 - Wall) 20.01 Hollow block (All cores grouted) [Wall]**	
Reinf't:	**N12-400 cts**	
Geometry:	**10000mm long x 1500mm high panel**	**Ast < Ast.min (Comp.)**
Capacity:	**Mv* = 2.49kNm/m < øMd+ = 9.21kNm/m**	**OK (0.27)**
	Mv-* = 0.56kNm/m < øMd- = 9.21kNm/m	**OK (0.06)**
Combined:	**Compression & Mv+* = 0.27, Tension & Mv+* = 0.27**	**OK (0.27)**
	Compression & Mv-* = 0.00, Tension & Mv-* = 0.00	**OK (0.00)**

Loadings - Wall

Robustness (Refer Geometry):

Vertical bending (Mv+*) =	2.49	kNm/m	RMv* =	0.56	kNm/m
Vertical bending (Mv-*) =		kNm/m			
			RMh* =	0.00	kNm/m
Check ductility =	Y	(Y)es,(N)o	Mecc* =	0.00	kNm (see RComp)

Design for Reinforced Vertical Bending Capacity (øMd) - Cl 8.6

Strength reduction factor (reinforced) (ø) =	0.75	Table 4.1	
Effective comp. width per bar =	772	mm (2*tu past reinf't) - Cl 8.6 (i)	
Reinf't centres =	400	mm	
Effective comp. width (b) =	1000	mm/m	
Area steel (Ast) =	283	mm²/m	
Earthquake ductility Astd.min = 0.0013*Ad =	247	mm²/m	
Depth to steel in +ve bending (dt) =	95	mm	
Asd = min(Ast & 0.29*1.3*f'm*b*dt/fsy) =	283	mm²/m	
ku =	0.197	Cl 8.3(c) - Based on Asd	
øMd+ = ø*fsy*Asd*dt*[1-0.6*fsy*Asd/(1.3*f'm*b*dt)] =	9.21	kNm/m	OK (0.27)
Depth to steel in -ve bending (dc) =	95	mm	
Asd- = min(Ast & 0.29*1.3*f'm*b*dc/fsy) =	283	mm²/m	
ku- =	0.197	Cl 8.3(c) - Based on Asd-	
øMd- =	9.21	kNm/m	OK (0.00)

Combined Bending & Compression - Cl 8.11.1

Compression (Nc*) =	0.0	kN/m	
Bending (M*) =	2.49	kNm/m	
Combined comp. capacity (øFd.comb) =	399.1	kN/m	
Combined comp. ratio (Mv+*) =	0.27		OK (0.27)
Combined comp. ratio (Mv-*) =	0.00		OK (0.00)

Combined Bending & Tension - Cl 8.11.2

Tension (Nt*) =	0.0	kN/m	
Tensile capacity (øFdt = ø*fsy*As) =	142.5	kN/m	
Combined tension ratio (Mv+*) =	0.27		OK (0.27)
Combined tension ratio (Mv-*) =	0.00		OK (0.00)

Secondary reinforcement - Cl 8.4.3

Secondary reinforcement = 0.00035*Ad =	67	mm²/m	Where required to resist non-uniform loadings or temperature or shrinkage
øMds =	2.32	kNm/m	For central bars

CONCRETE MEMBER V5.12 Magryn & Associates

Section:	**(BW2 - Footing) 200mm thk slab, f'c=32MPa**	
Reinf't:	**N12-200 cts bottom, ku = 0.12**	
Strength:	**(+ve M) M* = 3.0kNm < øMuo = 23.7kNm**	**OK (0.13)**
Cracking:	**fscr = 36MPa < Fscr = 225MPa & fscr1 = 42MPa < Fscr1 = 400MPa, crack width = 0.0mm**	**OK (0.10,0.16)**
Ast.min:	**Ast.min = 522mm² < Ast = 565mm² (Minimum of Deemed and actual)**	**OK (0.92)**

Geometry Interior environment with εcsd.b*=800x10⁻⁶, εcs*=560x10⁻⁶ L/D ratio = 0.0

Concrete strength (f'c) = 32 MPa
Depth (D) = 200 mm
Web width (W) = S mm, (S)lab
Slab type = O (O)ne way, Two way & (C)ols, (T)wo way & walls, Two-way (F)ooting

Comp.
Tension

Span (L) = mm (for moment resisting width)
Fully enclosed = N (Y)es,(N)o
Formwork = S (S)tandard,(R)igid
Concrete weight = 25.0 kN/m³
Exposure top = (B2)* Table 4.10.3.2 [*Cl 4.3.2]
S.Wt = 5.00 kN/m
Exposure bottom = (B2)* Table 4.10.3.2 [*Cl 4.3.2]
Gross area (Ag) = 200000 mm²

Design actions

Analysis values = M (M)anual, (L)eft, Position (X) from analysis, (R)ight

Manual values

Design (M*) = 3.0 kNm/m
Design (Ms1*, Ψs=1) = D kNm/m
Design (Ms*, Ψs) = D kNm/m

Ms1* & Ms* estimated - To be verified

	Manual	Units
M*	3.0	kNm
Ms1*(Ψs=1)	2.3	kNm
Ms*(Ψs)	2.0	kNm
Ast req'd	68	mm²
Ast	565	mm²
Reinf't req'd	N12-1655	

Reinforcement

Bottom reinf't =	**N12-200 cts**	**Top reinf't =**	**None**
Bar size =	12 mm	Bar size =	mm
Bar cts/No/mm² =	200 mm	Bar cts/No/mm² =	200 mm
Yield strength (fsy) =	500 MPa	Yield strength (fsyc) =	500 MPa
Ductility class =	A (N)ormal,(L)ow,(A)uto	Ductility class =	A (N)ormal,(L)ow,(A)uto
Reinf't ductility class =	N (N)ormal,(L)ow	Reinf't ductility class =	N (N)ormal,(L)ow
Steel area (Ast) =	565 mm²/m	Steel area (Asc) =	0 mm²/m
Bottom cover to steel =	90 mm	Top cover to steel =	100 mm
Depth to bottom steel layer (ds.max) =	104 mm	Depth to top steel layer =	100 mm
Depth to bottom steel (ds) =	104 mm	Depth to top steel =	100 mm
D-ds =	96 mm	D-ds =	100 mm
No. bars =	5.0 No.	No. bars =	5.0 No.
Bar centres =	200 mm	Bar centres =	0 mm
Max. bars per layer =	21	Max. bars per layer =	1
Layers required =	1	Max. bars per 2nd layer =	0
		Layers required =	0

Strength in +ve bending for slabs - Cl 9.1

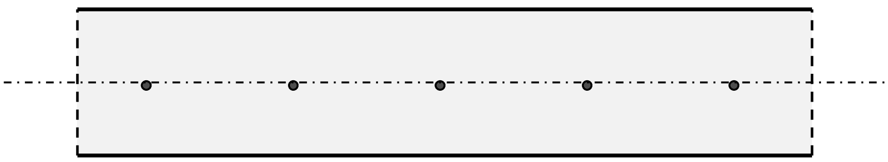
Design width (W) =	1000 mm	Design flange (bef) =	1000 mm
Tensile steel area (As) =	565 mm²/m	Depth to tensile (ds) =	104 mm
Comp. steel area (Ac) =	0 mm²/m	Depth to comp. steel (dc) =	100 mm
Ultimate Moment (Mu) =	27.8 kNm/m	ku =	0.119
Design capacity (øMuo) =	23.7 kNm/m	Factor (ø) =	0.850 Table 2.2.2
		Ast.min =	522 mm²/m

Crack control in +ve bending for slabs - Cl 9.4

Steel stress (σscr1) =	42 MPa	Max. stress (σscr1.max) =	400 MPa	OK (0.10)
Steel stress (σscr) =	36 MPa	Max. stress (σscr.max) =	225 MPa	OK (0.16)
Max. crack width (w'max) =	0.3 mm	Calculated crack width =	0.02 mm	

Reinf't:	**5.0-N12 bottom tensile**
	0.0-N20 top compression
Ligs:	**No ligs**

Preview



Michael Dickson

From: Michael Dickson
Sent: Monday, 30 September 2024 10:32 AM
To: tim.hicks@cityofpae.sa.gov.au
Subject: Response to RFI for DA 24031085

Hi, Tim.

Thank you for your correspondence regarding this application.

In regards to item 1, the purpose of the updated plans submitted on 26 September 2024 were to address the inconsistencies with the original plans that were submitted. The SW3 wall on the Greenhill plans has a maximum height of 2.6 metres, and therefore is engineered in the Magryn documentation to 2.8 metres. Similarly, the SW1 wall on the Greenhill plans has a maximum height of 2.2 metres, and therefore is engineered in the Magryn documentation to 2.4 metres. For all intents and purposes, the heights shown on the Greenhill plans are what will eventuate on the ground.

In regards to the retaining walls at the front of the allotments, identified as BW2 on the Magryn drawings, will all be less than 1.0 metre in height, and therefore, do not constitute ‘development’. For this reason, it is not considered necessary for these to also be shown on the Greenhill plans. They have, however, been shown on the Magryn drawings for complete transparency to the Council of the intended outcome.

Should you have any further queries, please do not hesitate to contact me.

MICHAEL DICKSON
Associate Director

FUTURE URBAN

M. 0403 824 468
E. michaeld@futureurban.com.au
W. www.futureurban.com.au
A. Level 1, 74 Pirie Street, Adelaide, SA, 5000

Level 1, 74 Pirie Street
Adelaide SA 5000
PH: 08 8221 5511
W: www.futureurban.com.au
E: info@futureurban.com.au
ABN: 71 651 171 630

September 17, 2024

Tim Hicks
Senior Planning Officer
City of Port Adelaide Enfield

Via: The PlanSA Portal

Dear Tim,

RE: RETAINING WALLS AND FENCES – STAGE 2, OAKDEN RISE

On behalf of our client, Oakden Developments Pty Ltd ('the Proponent'), we are seeking Planning Consent for the construction of retaining walls and fences within Stage 2 of Oakden Rise ('the Proposed Development').

Stage 2 has now been titled, and the residential allotments created. The Proposed Development is located over Allotments 174, 176-185, 189, 190, 192, 194, 196-198, 200-206 and 208-211 held within Deposited Plan 134686 ('the Site').

The Site is located wholly within the confines of the Master Planned Neighbourhood Zone ('the Zone').

Proposal

The Proponent is seeking to construct retaining walls and fencing at the Site.

The proposed retaining walls are to facilitate the benching of the Site. The proposed retaining walls that constitute development[1] vary in height from 1.2 metres to 2.6 metres.

This will result in a coordinated and consistent benching and retaining outcome which is less onerous for purchasers who ordinarily would need to liaise with their neighbours about such matters, which becomes difficult when those neighbours haven't got their own house plans finalised.

A Colorbond "Good Neighbour" fence in "Dune" colour, with a height of 1.8 metres, is also proposed on top of the retaining walls identified as SW1, SW2 and SW3.

Procedural Matters

Assessment Pathway

As the Proposed Development is not a Restricted Development, Accepted Development or Deemed-to-Satisfy Development, it shall be assessed as a Performance Assessed Development.

Applicable Policies

We note that a "fence" and "retaining wall" are both a class of development specified within Table 3 of the Zone, where all listed policies are to apply to assessment of the application, to the exclusion of all other policies of the Planning and Design Code ('the Code').

[1] *Retaining walls exceeding 1 metre in height constitute development pursuant to Schedule 4, 4(1)(f) of the Planning, Development and Infrastructure (General) Regulations 2017*

FUTURE URBAN

Public Notification

A “fence” and “retaining wall” are both identified in Table 5, Clause 7 of the Zone as being excluded from public notification.

Relevant Authority

Pursuant to Regulation 22(a)(ii) of the *Planning, Development and Infrastructure (General) Regulations 2017*, the Council’s Assessment Manager may act as the relevant authority where a development is exempt from the requirements in Section 107(3) of the *Planning, Development and Infrastructure Act 2016* to give notice of the application.

Assessment

From our assessment of the proposal against the relevant policies within Table 3 of the Zone, we wish to highlight that:

- the proposed retaining and benching will facilitate future housing consistent with the existing and emerging locality, satisfying Desired Outcome 1 of the Zone;
- despite exceeding the recommended vertical height of 1.5m for excavation and filling in DPF 16.1 of the Zone, the proposed earthworks and allotment layout has sought to minimise the height of retaining walls so as to not result in any unreasonable impacts to the visual amenity of the locality, noting that:
  - » the proposed bench levels of the Site, and therefore the height of the retaining walls, has been designed in line with the constructed road levels and the highest bench level possible that will allow compliant driveway grades in accordance with AS2890.1:2004, thereby minimising the height of retaining walls as much as possible;
  - » the highest retaining walls are generally located towards the rear of the allotments, and will not be visible from the public realm once a dwelling is constructed, and will also not result in an unreasonable degree of overshadowing (where applicable) of private open space;
  - » the retaining walls along street frontages and the reserves (BW1) will be a block wall that is rendered and painted, in order to enhance the appearance of the public realm;
  - » the retaining walls that will be forward of the future dwellings (BW2), despite being less than 1.0 metre in height and not requiring approval, will have a rendered and painted finish to enhance the appearance of the streetscape;
- the proposed retaining walls and fences are of a height and scale that is reasonably expected to occur within the sloping nature of the Site, and thus not considered to unreasonably restrict access to sunlight to adjoining land, which satisfies PO 9.1 of the Design section of the Code.

For the above reasons, we are of the opinion that the proposal satisfies the relevant provisions of the Code and thereby has sufficient merit to warrant planning consent.

Should you have any queries, please do not hesitate to contact me.

Yours sincerely,

Michael Dickson
Associate Director

Enc: *Site Plan and Elevations*