Eskom	MEDUPI POWER STATION PROJECT	SPF Doc No : 200-81502 Date: 2014-03-14 Revision: 01

Title:

MEDUPI POWER STATION
DESIGN CALCULATION FILE
Structural: Weighbridge

Document type: Design Calculation File

Consultant nr: 14133-DC-AE1

Distribution:

Compiled by:

Marcia Maidi
Nyeleti Consulting

Approved by:

Marelize Mostert Pr Eng
20010150
Nyeleti Consulting

Revision	Description of Revisions	Approval	Date
01	Crack width calculations, stairs design and KKS code added	MM	2014-03-14

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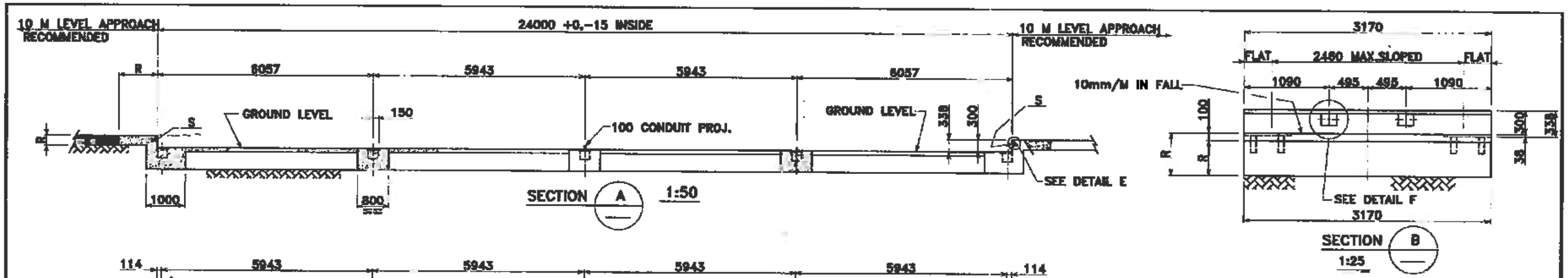
Promotion of the Access to Information Act 2000	**DATE OF NEXT REVIEW: Not applicable** *NOTE: - These dates can be changed without effecting the revision status of the document*

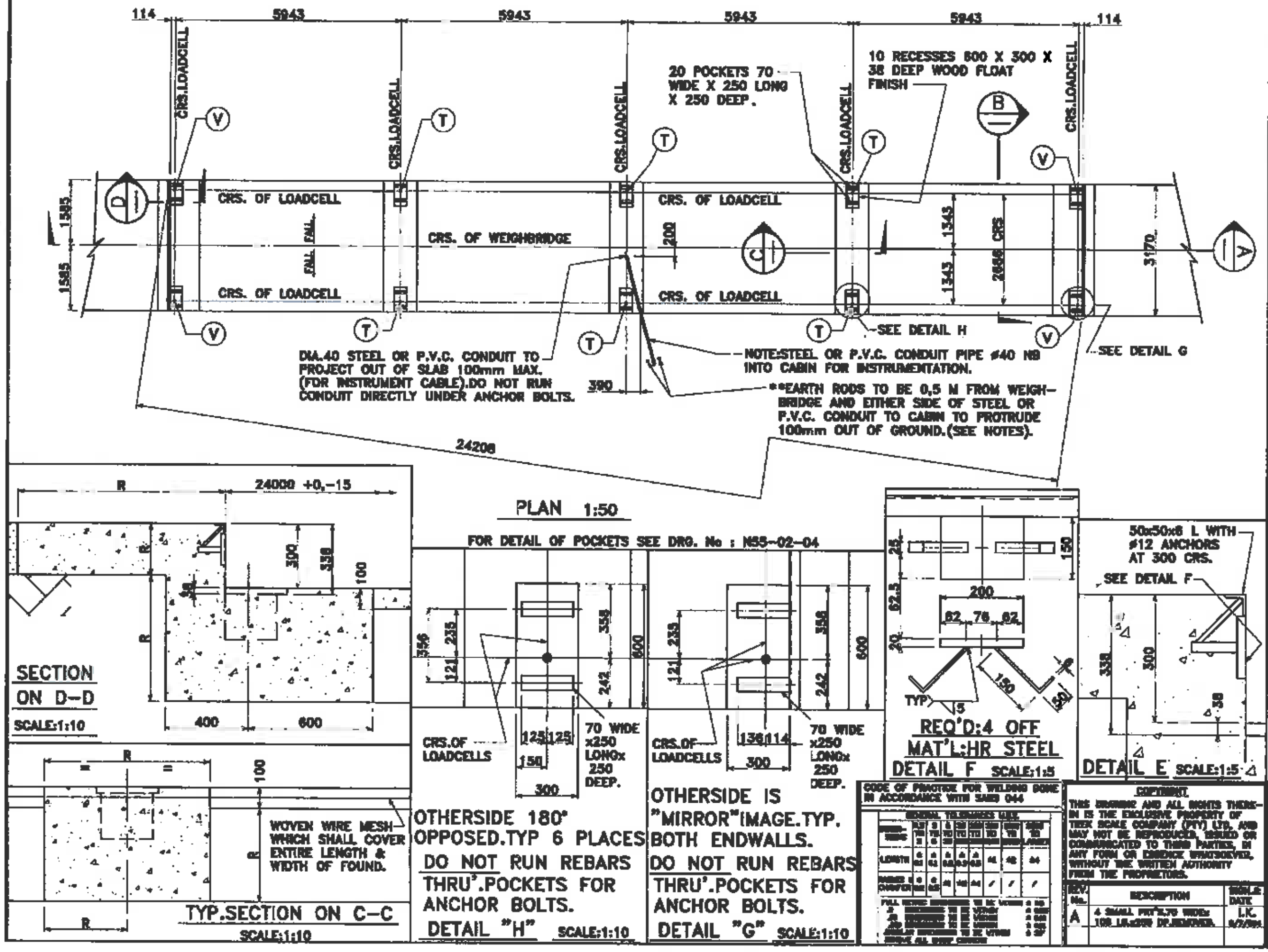
TABLE OF CONCTENTS

NOTE:

- The weigh bridge has been shelved; the calculations are for information only.
- Bases are in compression, crack width calculations not required.
- KKS code: 0 1UC

1. DRAWINGS





SPECIAL NOTE:

SPECIAL EARTH CONNECTION (CEP) TO BE PROVIDED LOCAL TO THE WB,TO BE DRIVEN 1700mm INTO GROUND.** 2 OFF EARTH RODS ø 16 x 1800mm LONG.(SEE ON PLAN).

1).DIMENSIONS MK'D "R" ARE TO SUIT SOIL CONDITIONS ; FOUNDATION TO BE REINFORCED AND/OR INCREASED IN THICKNESS TO SUIT LOCAL SITE CONDITIONS AND LOADS STATED.
2).DRAINAGE OF FOUNDATION TO BE PROVIDED BY THE CLIENT.
3).ALL BENDS IN THE CABLE DUCT MUST BE LONG RADIUS BEND.ALL DUCT ENDS TO BE PLUGGED BEFORE "CASTING IN".
4).THE CONTRACTOR MUST HOLD THE DIAGONAL MEASUREMENTS FROM ENDWALL TO ENDWALL OF 24208.
5).PLEASE NOTE,WOVEN WIRE MESH (W.W.M) WHICH SHALL COVER THE ENTIRE LENGTH AND WIDTH OF FOUNDATION.
6).STEEL OR P.V.C. CONDUIT PIPE (ø 40) INTO CABIN FOR INSTRUMENTATION.
7).DRAW ROPE TO BE INSTALLED IN AND THROUGH THE PIPE FROM FOUNDATION INTO CABIN.
8).TOLERANCES SHOWN ARE ABSOLUTE MAX./MIN.
9).NOTE:ALL CONDUIT BENDS TO BE LONG RADIUS AND DRAW ROPE TO BE INSTALLED FROM FOUNDATION INTO CABIN.

(T) MAX.FORCE=300 KN (VERTICAL)(EACH).

(V) MAX.FORCE=250 KN (VERTICAL)(EACH).

S MAX.PUSH-PULL=100 KN (LONGITUDINAL).

KKS CODE: 0 1UC

27-02-2009

Trevor Hewitt
011 626-3491
* Patrick
083 375541

CODE OF PRACTICE FOR WELDING DONE IN ACCORDANCE WITH SABS 044

REV. No.	DESCRIPTION	INITIALS DATE
A	4 SMALL PKT'S,70 WIDE: 100 LG.x250 DP.REMOVED.	I.K. 9/7/2001

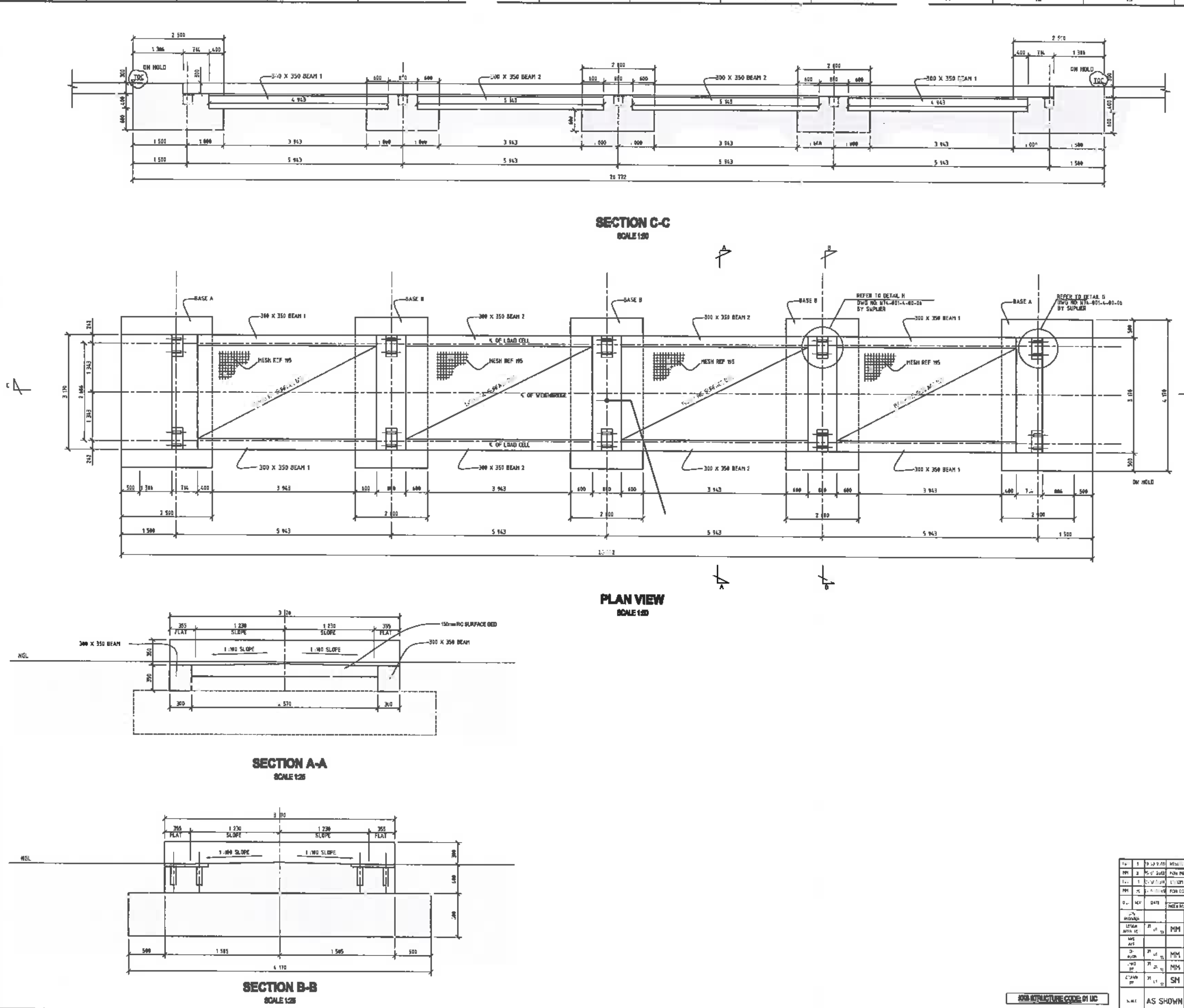
TREK SCALE COMPANY (PTY) LTD.

TITLE: FOUNDATION LAYOUT OF 24000 x 3000 x 80 TON MAX.CAPACITY STEEL DECK DIGITOL ROAD WEIGHBRIDGE – SURFACE MOUNTED

DRAWN: I.K. 15/04/04

SCALE: AS SHOWN ON PLAN & DETAILS

DRAWING NUMBER: N74-001-4-00-06 REV. A



STATION DATUM: 902.25

GENERAL NOTES:

1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH THE ARCHITECT'S DRAWINGS
2. ALL CONCRETE MIXES TO BE APPROVED BY THE ENGINEER PRIOR TO COMMENCEMENT OF CONSTRUCTION
3. FOUNDATION EXCAVATIONS TO BE APPROVED BY THE ENGINEER PRIOR TO CASTING OF THE BLINDING LAYER

CONCRETE NOTES:

1. CONCRETE CLASS.

REINFORCED CONCRETE AND SURFACE BEDS	CLASS 35/19
BLINDING AND SCREED	15 MPa CONCRETE (28 DAYS)
NO-FINES	NF19

2. ALL VISIBLE CONCRETE FACE FINISHES SHOULD BE SMOOTH OFF-SHUTTER AND ALL IMPERFECTIONS SHALL BE REPAIRED. FOR CONCEALED MEMBERS, DEFECTS SHALL BE REPAIRED BUT NO SPECIAL TREATMENT IS REQUIRED.
3. ALL EXPOSED CONCRETE EDGES TO HAVE A 25 X 25mm CHAMFER
4. ALL DIMENSIONS TO BE CONFIRMED ON SITE.
5. CONCRETE TO COMPLY WITH SANS 1200G AND ESKOM CONCRETE SPECIFICATION NO 84CIVL053.
6. CURING METHOD STATEMENT TO BE APPROVED BY THE ENGINEER

DESIGN:

1. SOIL BEARING CAPACITY = 175 kPa (SLS)
2. MAXIMUM WEIGH BRIDGE CAPACITY =80 TON
3. MAXIMUM VERTICAL FORCE EACH =300KN (SLS)
4. MAXIMUM PUSH - PULL = 100KN (LONGITUDINAL)

CODE OF PRACTICE FOR WELDING DONE IN ACCORDANCE WITH SABS 044

GENERAL TOLERANCES U.D.S

DIMENSIONS	0.5 to 3	3 to 6	6 to 30	30 to 120	120 to 315	315 to 1000	1000 to 2500	2500 to LARGER
LENGTH	± 0.1	± 0.1	± 0.2	± 0.3	± 0.5	± 1	± 2	± 4
RADIUS & CHAMFER	± 0.2	± 0.5	± 1	± 2	± 4	/	/	/

FULL METRIC DIMENSIONS TO BE WITHIN ±0.5
.0 DIMENSIONS TO BE WITHIN ± 0.5
.00 DIMENSIONS TO BE WITHIN ±0.10
.000 DIMENSIONS TO BE WITHIN ±0.01
ANGULAR DIMENSIONS TO BE WITHIN ±30°
REMOVE ALL SHARP CORNERS

ISSUED FOR INFORMATION

KKS STRUCTURE CODE: 01 UC

MEDUPI POWER STATION
WEIGHBRIDGE
CONCRETE DETAILS

Nyeleti

WG BEETGE

MM | MM | SH

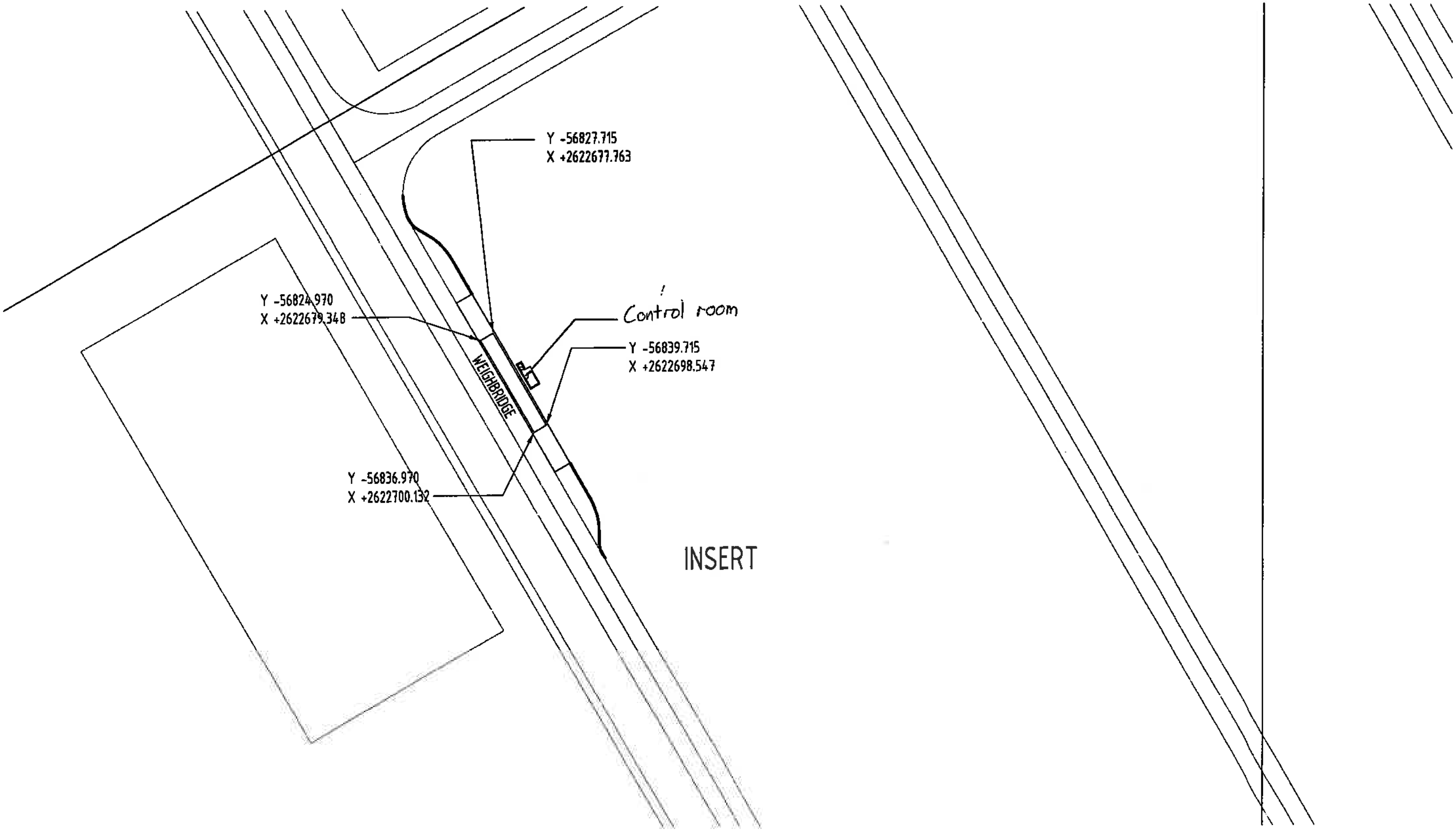
14133 - AE - 100

AS SHOWN

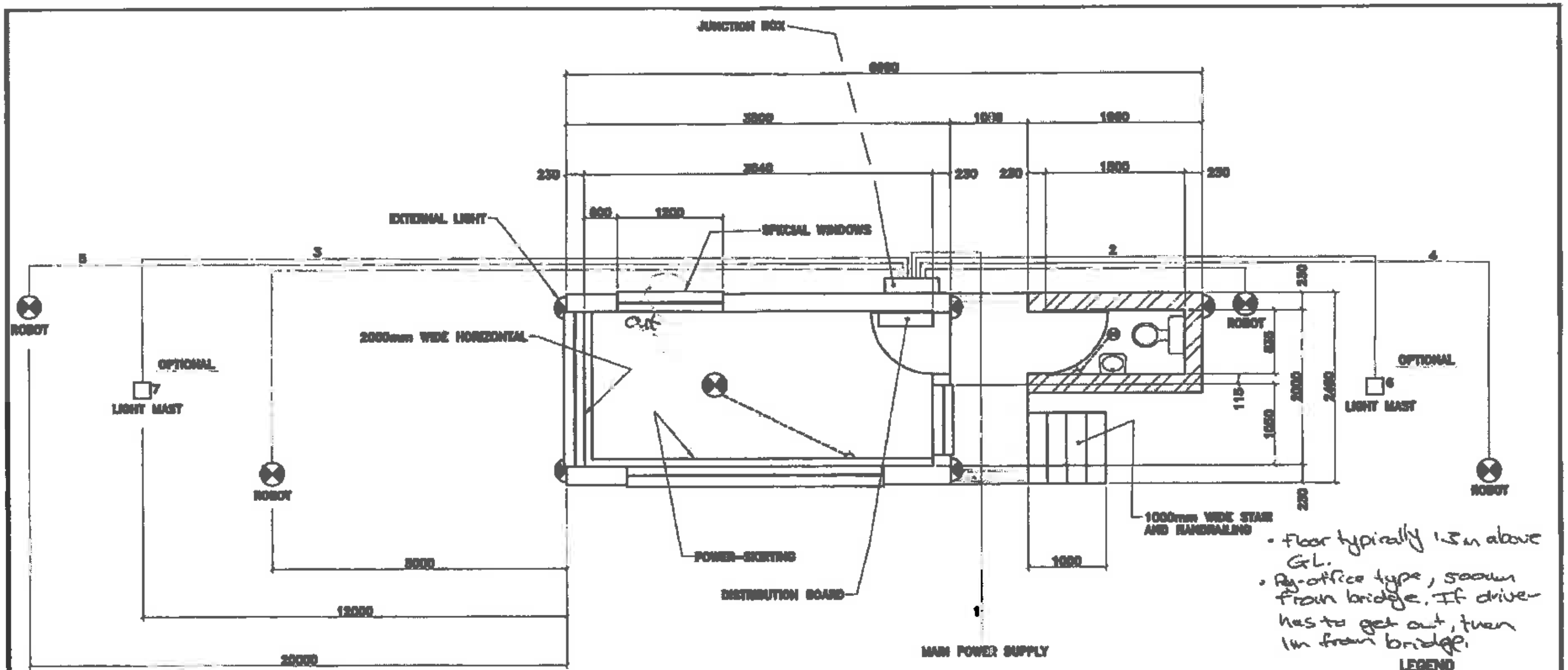
Eskom

0.84 / 12776 | 01 | 03

A1L



Y -56827.715
X +2622677.763
Control room
Y -56824.970
X +2622679.348
WEIGHBRIDGE
Y -56839.715
X +2622698.547
Y -56836.970
X +2622700.132
INSERT



· Floor typically 1.5m above GL.
· Py-office type, 500mm from bridge. If driver has to get out, then 1m from bridge.

LEGEND

CABLE

1. MAIN SUPPLY
2. LIGHT MAST 30 AMP 1.5x3 ARMOUR CABLE
3. LIGHT MAST 30 AMP 1.5x3 ARMOUR CABLE
4. ROBOTS 10 AMP 1.5x3 ARMOUR CABLE
5. ROBOTS 10 AMP 1.5x3 ARMOUR CABLE
6. LIGHT MAST 3x400 WATT HPS LIGHT FITTINGS
7. LIGHT MAST 3x400 WATT HPS LIGHT FITTINGS

D.B.

1. EARTH LEAKAGE
2. 15 AMP ISOLATION LIGHTS
3. 15 AMP ISOLATION PLUGS
4. 30 AMP ISOLATION LIGHT MAST
5. 15 AMP ISOLATION ROBOTS
6. 15 AMP ISOLATION AIR CONDITION

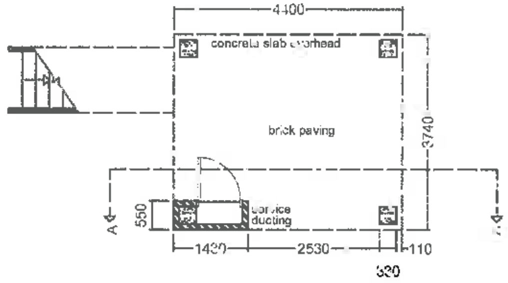
TREK SCALE COMPANY (PTY) LTD.

CLIENT:

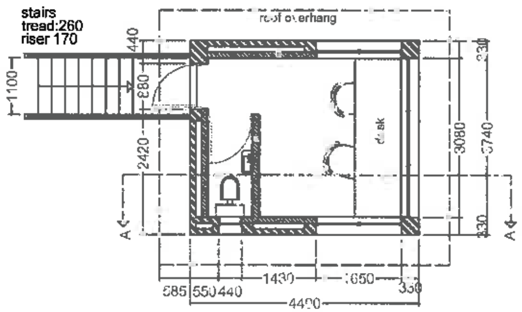
TITLE: SITE LAYOUT OF CABIN WITH ROBOTS AND LIGHTS IN POSITION AS SHOWN

DRAWING NUMBER
33674-00000-03

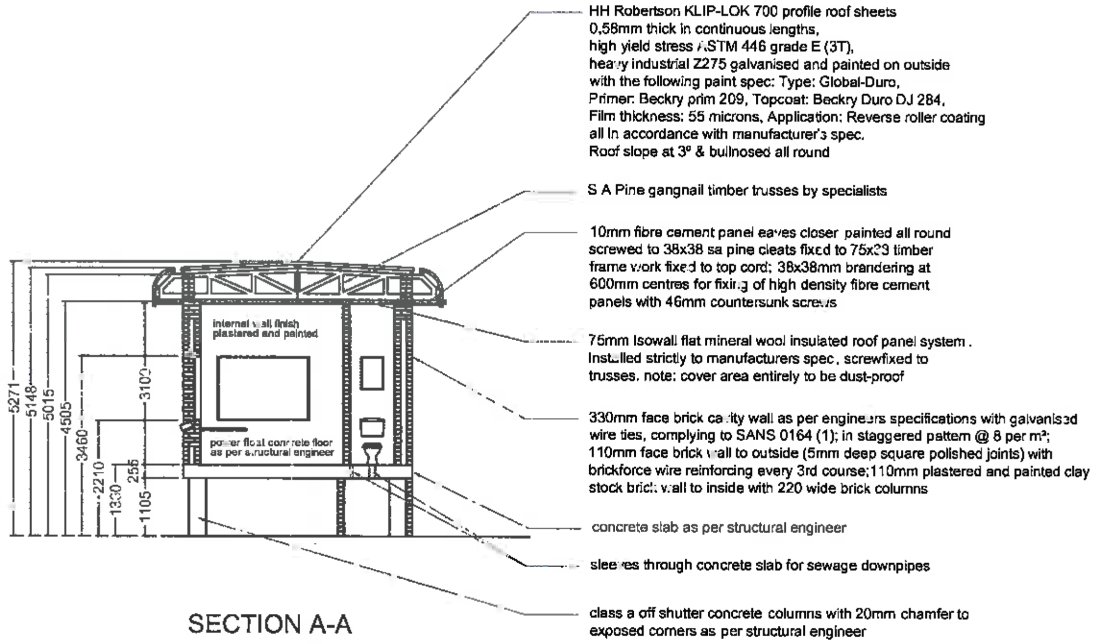
KKS CODE:0 1 UC



GROUND FLOOR
Scale 1:100



GROUND LEVEL
1:100



SECTION A-A
Scale 1:100

WMS

MEDUPI POWER STATION
WEIGH STATION
CONTROL ROOM
PLANS AND SECTION

07015 - AD - 23.1

0.90/

A3L

2. DESIGN INFORMATION, LOADS AND ASSUMPTIONS

DESIGN CODES USED

BS 8007:	Design of concrete structures retaining aqueous liquids	☒
SANS 0100 Part 1:	Structural use of concrete	☒
SANS 0160:	General procedures and loadings	☒
SANS 0162 Part 1:	Structural use of steel (hot rolled)	☐
SANS 01642 Part 2:	Structural use of steel (cold formed)	☐
SANS 0164 Part 1:	Structural use of masonry	☐
TMH 7 Parts 1 & 2:	Design of highway bridges and culverts	☐

Marelize Mostert

From:	Alan Parrock [Alan@arq.co.za]
Sent:	30 January 2009 07:37 AM
To:	Marelize Mostert
Subject:	RE: Medupi bearing capacity

Marelize

I confirm that the design soil bearing capacity for all P35 buildings on Medupi should be 175kPa at SLS.

Alan Parrock

__________ Information from ESET NOD32 Antivirus, version of virus signature database 3811 (20090129) __________

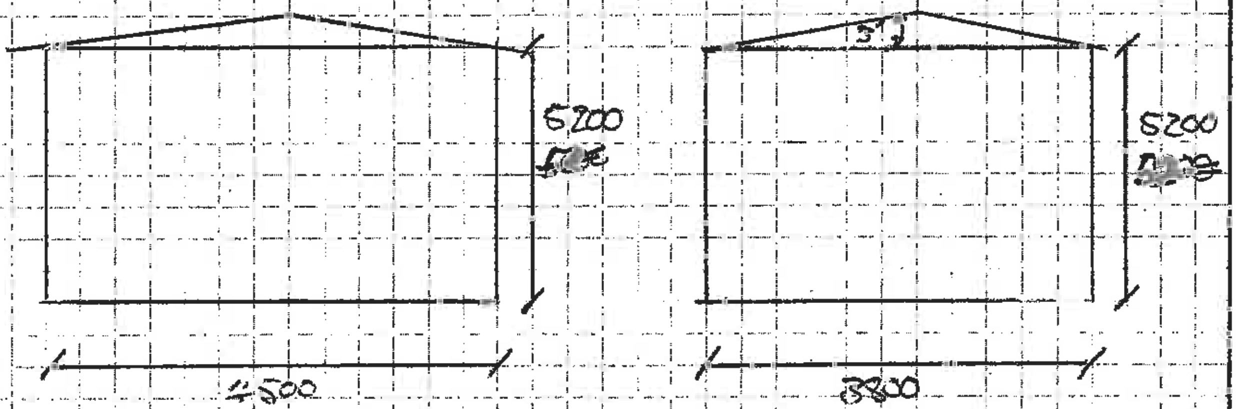
The message was checked by ESET NOD32 Antivirus.

http://www.eset.com

WEIGH BRIDGE CONTROL ROOM

Project: Medupi (Weigh bridge control room) Project No: 14133
Component: Wind loads Sheet No: 1
Compiled By: DJJ vd Merwe Date: 18-05-08 Checked By: Date:

Rev SABS 0160	Calculations



5.5.2.3	Mean return period = 50 years
5.5.2.4	Terrain catagory = 2
Fig 3	Regional basic windspeed = 40
5.5.2.6	Class of structure = A
Tables	$k_z = 0{,}942$ $\quad z = 5200$
5.5.2.6	$v_z = v \cdot k_z$ $= 37{,}68$ m/s
Given	Altitude = 902,25 m
5.5.3.1	$k_p = 0{,}536$
	$q_z = k_p \cdot v_z^2$ $= 761{,}00\ N/m^2 = 0{,}761\ kN/m^2$

<u>Cpe on walls</u>

$h = 5200$ $\quad \frac{h}{w} = 1{,}368$ $\quad \frac{1}{2} \leq \frac{h}{w} \leq \frac{3}{2}$

$w = 3800$ $\quad \frac{b}{w} = 1{,}184$ $\quad 1 \leq 1{,}184 \leq \frac{3}{2}$

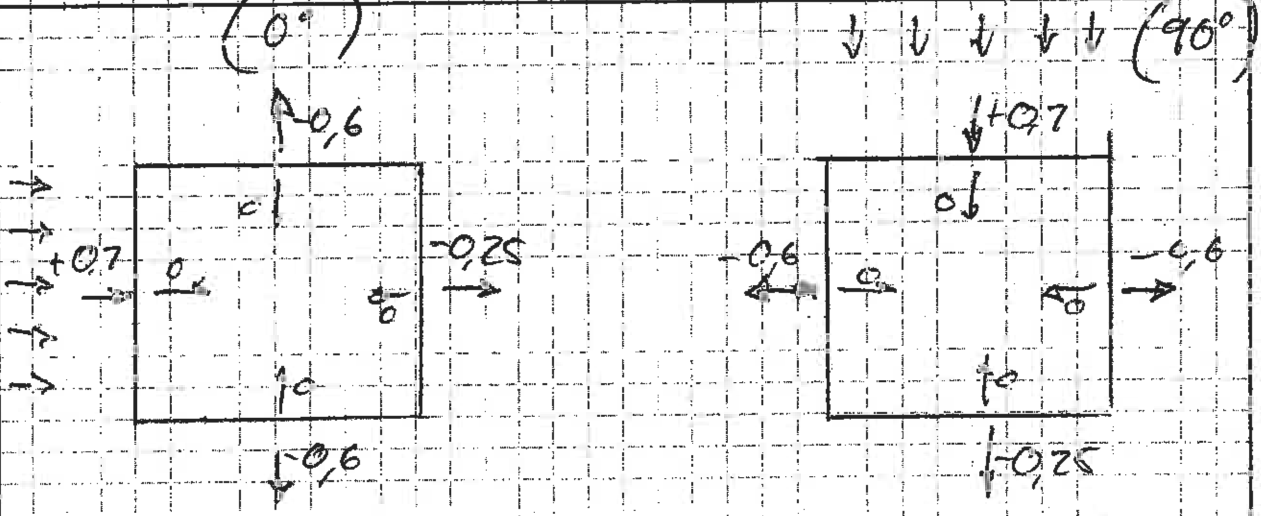
$b = 4500$

Project:	Medupi (weigh bridge control room)	Project No:	14133
Component:	Wind loads	Sheet No:	2
Compiled By:	DJJ vd M Date: 18-05-09	Checked By:	Date:

Rev: SABS 0160

Calc

Table 6.



1) Design pressures $= (C_{pe} + C_{pi}) q_z$

$= 0{,}7 \cdot 0{,}761$

$= 0{,}5327$ kPa

2.) $-0{,}25 \cdot 0{,}761 = -0{,}1903$ kPa

3) $-0{,}6 \cdot 0{,}761 = -0{,}4566$ kPa

4) $-0{,}6 \cdot 0{,}761 = -0{,}4566$ kPa

C_{pe} on roofing

Slope = 3°

EF = ~~0,54~~ $-0{,}86$ GH = $-0{,}54$ EG = $-0{,}9$ FH = $-0{,}7$

Project: Medupi Weigh bridge control room | Project No: 14133
Component: Wind loads | Sheet No:
Compiled By: DJJ vd Merwe Date: 18-05-09 Checked By: Date:

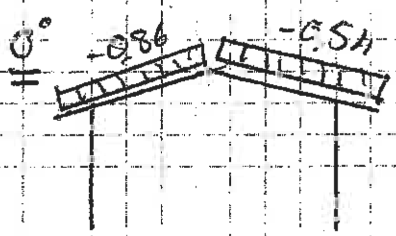
Ref SABS 0160

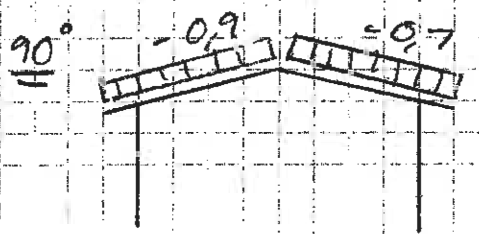
Calc

<u>Roof loads + design pressures</u>

Wind $0° \Rightarrow EF = +0{,}86$
$GH = -0{,}54$

Wind $90° \Rightarrow EG = -0{,}9$
$FH = -0{,}7$





<u>Live loadings</u>

5.4.4.3 Inaccesible roof

$\left(0{,}3 + \frac{15 - A}{60}\right)$ kN/m

$A = 0{,}6 \cdot 4.4 = 2{,}64 \text{ m}^2$

$0{,}3 + \frac{15 - 2{,}64}{60} = 0{,}506 \text{ kN/m}^2$

Truss spacing = 600mm spacing

<u>Dead load</u>

- Selfweigh to be calculated by timber roof specialist
- Insullation + ceiling loads = 1 kN/m^2

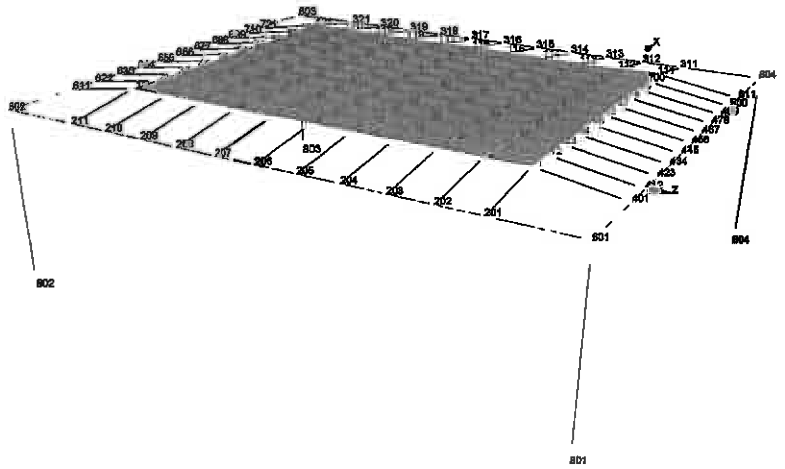
3. DETAILED DESIGN CALCULATIONS AND MODELS

3.1 Concrete

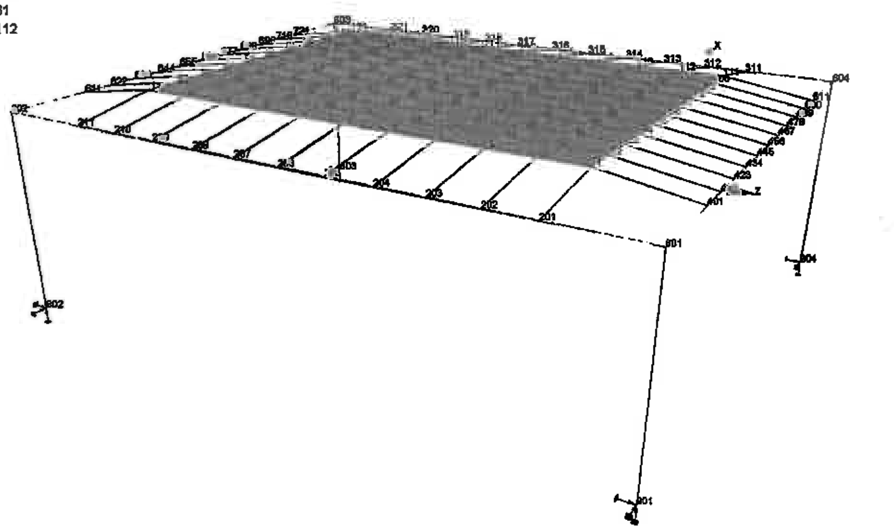
PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title		
	Client		
	Calcs by	Checked by	Date

Space Frame Analysis Ver W2.4.52 - 25 May 2009
Input file:I:\14133_Medupi PS and WTP\28 Weigh bridge\Design\Control Room\Slab 09-06-01 (2).A03
Created : 01/06/2009 05:46:48 PM

A03

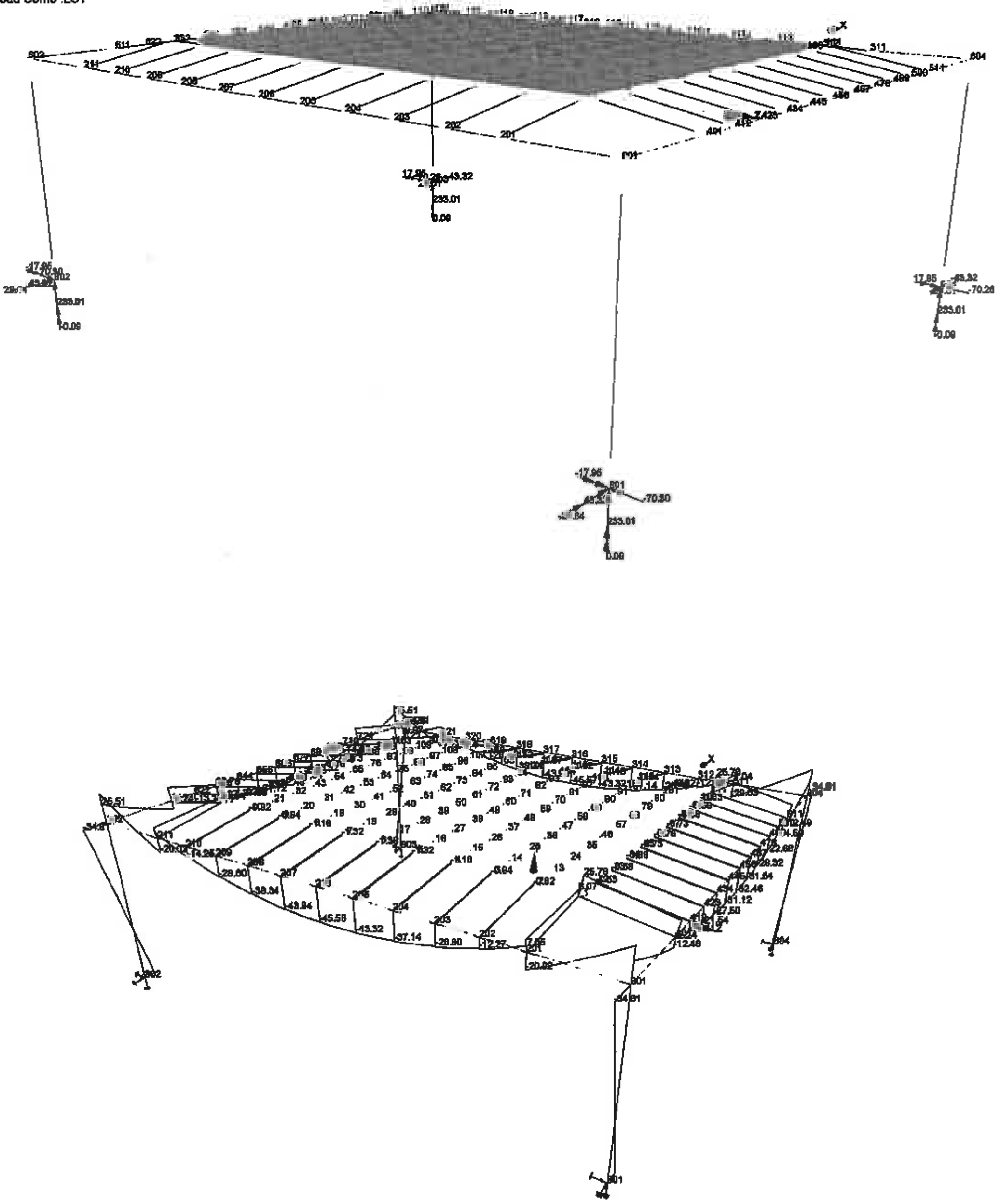


Maximum Deflections for Load Combination LC1:
X :-0.01 mm at node 110
Y :-0.37 mm at node 61
Z :-0.01 mm at node 112

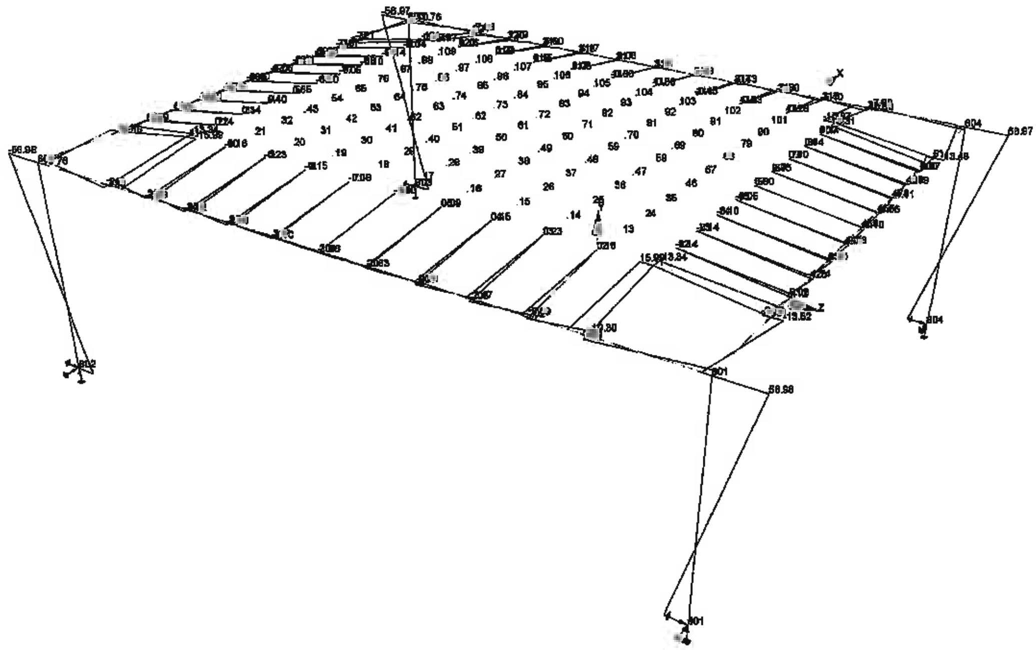


PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title		
	Client		
	Calcs by	Checked by	Date

Reactions: Load Comb :LC1



PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title		
	Client		
	Calcs by	Checked by	Date



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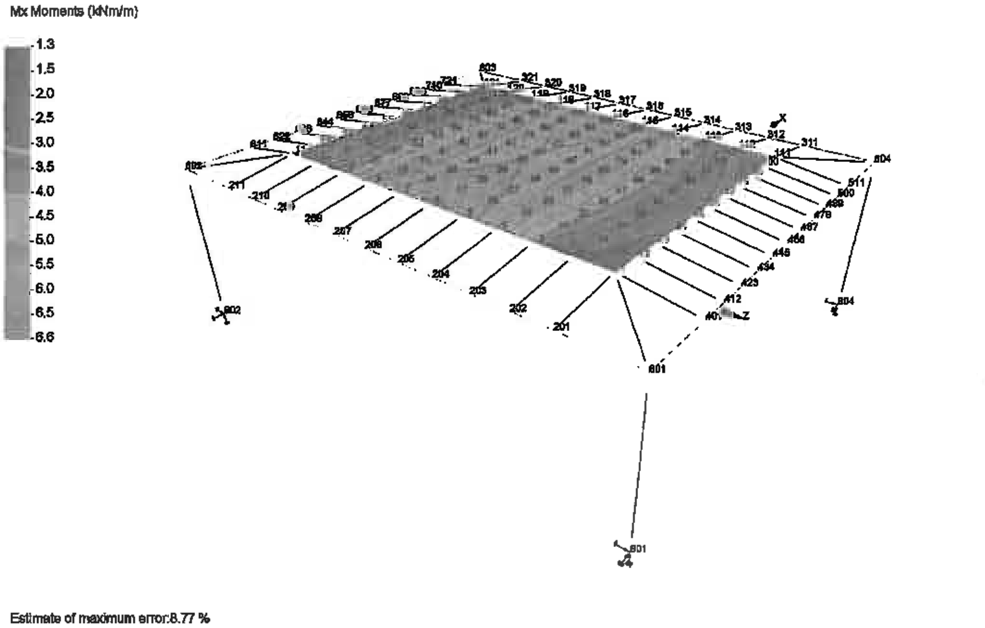
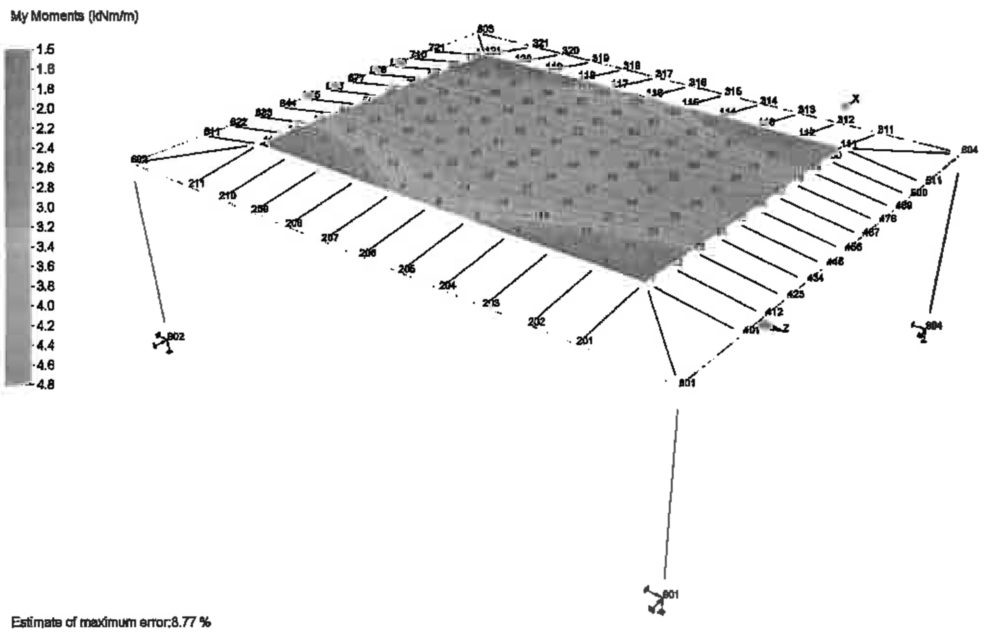
A03

Bending Moments for Load Case DL :Mx Moments (kNm/m)



Bending Moments for Load Case DL :My Moments (kNm/m)



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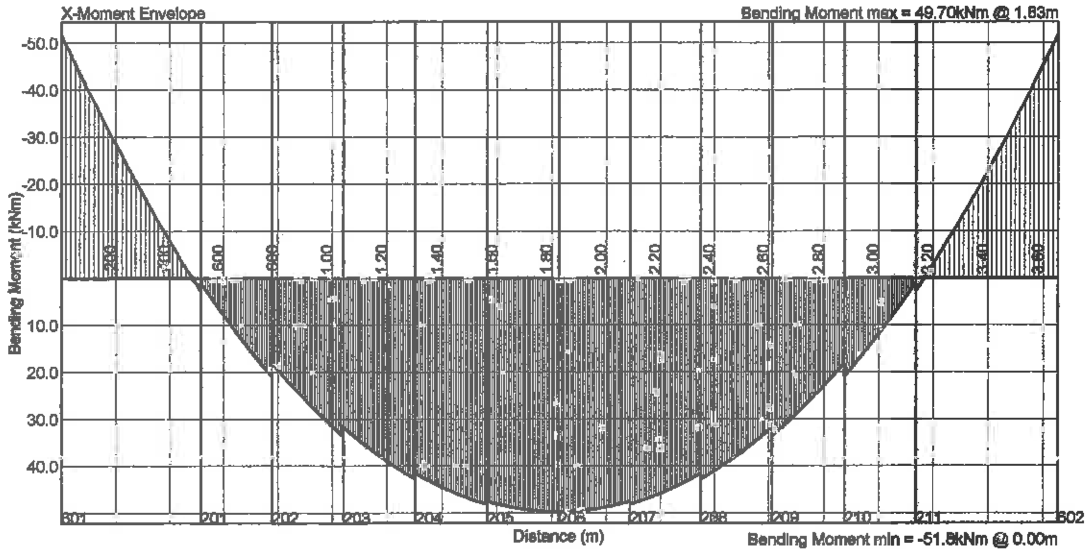
A03

Envelope for Load Cases:
DL, WL, LL, LC1, LC2, LC3, LC4,



PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title		
	Client		
	Calcs by	Checked by	Date

Space Frame Analysis Ver W2.4.52 - 25 May 2009
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A03

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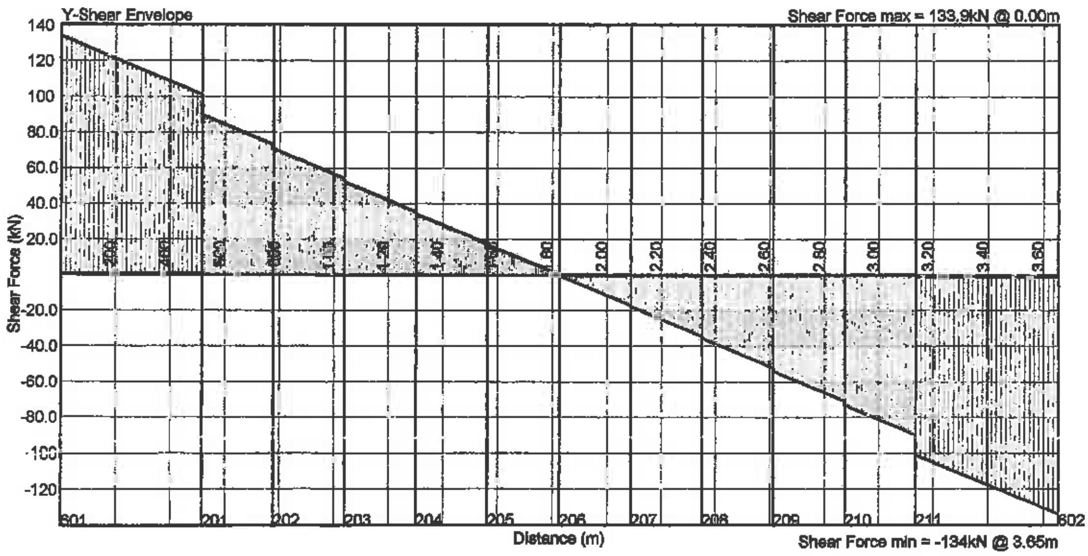


Project:	Medupi Buildings	Project No:	14133
Component:	Weigh bridge control room Shear – down stand	Sheet No:	
Compiled By:	DJJ vd M Date: 09-06-01	Checked By:	Date:

Rev SABS 0100	Calculations

V at centre point of support:

Shear-envelope — $V = 134\ kN$

V at edge of support:

$$V = \frac{134}{1{,}827} \cdot (1{,}827 - 0{,}225)$$

$$= 117{,}5\ kN$$

From shear envelope: V at support edge = 118 kN

4.3.4.1.1 — $d = 450\ mm$, $b_v = 450\ mm$ $\therefore$ $v = \frac{118000}{450 \cdot 450} = 0{,}583\ MPa$

4.3.4.1.1. — v_u = lesser of $\begin{cases} 0{,}75\sqrt{f_{cu}} = 3{,}75 \\ 4{,}75 \end{cases}$ (for $f_{cu} = 25$)

$\therefore v_u = 3{,}75$

$\therefore$ $v_u > v$ $\therefore$ OK

4.3.4.1.2 — $v_c = \frac{0{,}75}{\gamma_m} \cdot \left(\frac{f_{cu}}{25}\right)^{\frac{1}{3}} \cdot \left(\frac{100 A_s}{d \cdot b}\right)^{\frac{1}{3}} \cdot \left(\frac{400}{d}\right)^{\frac{1}{4}}$ $\left(A_s = 942 \atop 3Y20's\right)$

$= 0{,}403$

$v_c < v$ $\therefore$ Shear failure

(but beam wil never fail at support edge)

Project: Medupi Buildings | Project No: 14133
Component: Weigh bridge control room | Sheet No:
Shear - downstand
Compiled By: DJJ vd M | Date: 01-06-09 | Checked By: | Date:

Shear at d from support $d = 450\,mm$

Y-shear envelope

$V = 78\,kN$ $v = \frac{78000}{450 \cdot 450}$

$= 0{,}385\ MPa$

Prev. calculated

$v_c = 0{,}403\ MPa$

4.3.4.2.1 • Shear enhancement at d from support: $\frac{2d}{a_v} = 2$

$\therefore v_c = 2 \cdot 0.403 = 0{,}806\ MPa$

$v_c > v \ \therefore$ OK!!

Shear at 2d from support $2d = 900\,mm$

Y-shear envelope

$V = 45{,}1\,kN$ $v = 0{,}223\ MPa$

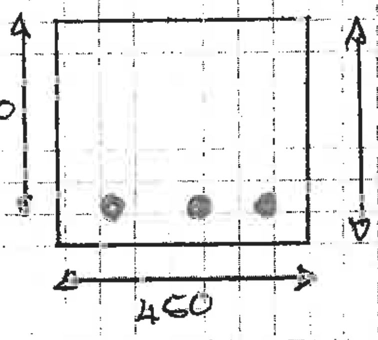
$v_c = 0{,}403\ MPa$

$v_c > v \ \therefore$ OK!!

Project:	Medupi Buildings	Project No:	14133
Component:	Weigh Bridge control Room Downstand reinforcement	Sheet No:	
Compiled By: DJJ vd M	Date: 01-06-09	Checked By:	Date:

Rev SABS 0100	Calculations

X-moment envelope



Cover = 40mm

Max Moment = 52 kN.m

$f_{cu} = 25$ MPa

4.3.3.4.1 ~~3.8~~

$K = \frac{M}{f_{cu} \cdot b \cdot d^2} = 0{,}023 < 0{,}156 \therefore$ No top reinf. use min As.

" "

$z = d\left(0{,}5 + \sqrt{0{,}25 - \frac{K}{0{,}9}}\right)$

$= 438{,}2$ mm $\quad 0{,}95d = 427{,}5$

$z > 0{,}95d$

$\therefore z = 0{,}95d = 427{,}5$

" "

$A_{st} = \frac{52\,000\,000}{0{,}87 \cdot f_y \cdot z}$

$= 310{,}7 \therefore 3\ Y12's$

3 Y20's are used to help for shear

$\therefore A_{st} = 942\ mm^2 \therefore$ OK!!

Crack Width Design

	0.3	mm
	Crack width Limit	

* 0.3mm for General Structures, 0.2mm for Water Retaining Structures, 0.1mm for Chemicals
* Linked to Spreadsheet, "SUMMARY"

Project:	Medupi Power station-
Building:	Weigh bridge -slab downstand beam (sagging)
Date:	13-Mar-14
Designer:	Trevor Mathabatha

Crack Widths due to Moment @ SLS (SABS 0100-1, ANNEX A)

* Only edit "Yellow" cells

[illegible]	[illegible]	ACCEPTABLE

Type of Beam	T-Beam	* If I-Beam, Select in Flange in Tension For Both Sagging and Hogging Moments.
Type of Bending Moment	Web in Tension	If Rectangular, Select Either Type of Bending Moment

Beam Cross Section Height (h)	500	mm
	0.5	m
Flange Height (hf)	245	mm
	0.245	m
Web Width (bw)	450	mm
	0.45	m
Flange Width (bf)	670	mm
	0.67	m
Applicable Width for Calculation (bt)	450	mm
	0.45	m

Characteristic Cube Strength (f_{cu})	35	MPa
	35000000	Pa
Characteristic Yield Strength (f_y)	450	MPa
	450000000	Pa
Cover	25	mm
Diameter of Stirrups (Taken as 0; 40mm Cover is taken to Main Steel)	12	mm
Diameter of Reinforcing (Maximum)	20	mm
Spacing of Bars	150	mm
Tensile Reinforcement (A_{st})	942	mm^2
Effective Depth of Tensile Rebar (A_{st})	453	mm
	0.453	m

BENDING MOMENT (SLS)	33.430	kN.m
	33430.000	N.m
Cover (c_{min})	25	mm
	0.025	m

Potential Crack Length (a_{cr})	83.63	mm
Characteristic Cube Strength (f_{cu})	35	MPa
	35000000	Pa

Young's Modulus for Steel (E_s)	200	GPa
K_0 For Normal Weight Concrete	20	GPa
Young's Modulus for Concrete (E_c) = K_0 + 0.2GPa(f_{cu}/MPa)	27	GPa
Long Term Creep Coefficient	1	
Effective Young's Modulus for Concrete (E_{eff})	13.5	GPa
x = NA Depth for Cracked Section	[illegible]	mm
	[illegible]	m

Found by: $ax^2 + bx + c = 0$		
Web in Tension, NA Assumed in Web		
Quadratic Equation (a):	bw/2	225
Quadratic Equation (b):	[(Es/Eeff)*Ast]+ [hf * (bf - bw)]	67855.55556
Quadratic Equation (c):	[-0.5*(bf - bw)*hf^2] - [(Es/Eeff) * Ast * d]	-12924616.67
x1		132.371
Web in Tension, NA Assumed in Flange		
Quadratic Equation (a):	bf/2	335
Quadratic Equation (b):	(Es/Eeff)*Ast	13955.55556
Quadratic Equation (c):	-(Es/Eeff) * Ast * d	-6321866.667
x2		118.114

$x = [-b +- sqrt(b^2 - 4ac)]/2a$		
$\varepsilon_1 = (M/[E_s*A_{st}*(d-x/3)])*[(h-x)/(d-x)]$	0.000489194	mm/mm
$\varepsilon_{1s} = [b_t*(h-x)^2]/[3*E_s*A_{st}*(d-x)]$	0.000346722	Units to use: mm, Mpa, N
$\varepsilon_m = \varepsilon_1 - \varepsilon_{1s}$	0.000142472	mm/mm

Crack Width = $w_{max} = (3a_{cr} * \varepsilon_m)/(1+2(a_{cr}-c_{min})/(h-x))$	[illegible]	mm

Project: Medupi Buildings | Project No: 14133
Component: Slab reinforcement | Sheet No:

Compiled By: DJJ vd M | Date: 01-06-09 | Checked By: | Date:

Rev SABS 0100	Calculations

MaxMoment on slab = 10,5kN.m

$K = \frac{10500000}{25 \cdot 1000 \cdot 205^2} = \cancel{0.009}\ 0{,}01 < 0{,}156$ ∴ Min As for top steel

$z = 0{,}95\,d = 194{,}75$

$Ast = \frac{10500000}{0{,}87 \cdot 450 \cdot 194{,}75} = 137{,}71$ ∴ use Min As.

Min Ast : $\frac{100\,As}{Ac} = 0{,}13$

$As = 331{,}5$ ∴ use Y12 @ 200

$As = 565\ mm^2/m$

Crack Width Design

0.3	mm

Crack width Limit

** 0.3mm for General Structures, 0.2mm for Water Retaining Structures, 0.1mm for Chemicals*

* Linked to Spreadsheet, "SUMMARY"

Project:	Medupi Power station-
Building:	Weigh bridge -slab (sagging)
Date:	13-Mar-14
Designer:	Trevor Mathabatha

Crack Widths due to Moment @ SLS (SABS 0100-1, ANNEX A)

** Only edit "Yellow" cells*

CRACK WIDTH (mm)	0.0000	ACCEPTABLE

Slab Thickness (h_{slab})	255	mm
	0.255	m

Characteristic Cube Strength (f_{cu})	35	MPa
	35000000	Pa
Characteristic Yield Strength (f_y)	450	MPa
	450000000	Pa
Cover	25	mm
Diameter of Reinforcing (Minimum)	16	mm
Spacing of Bars	200	mm
Tensile Reinforcement (A_{st})	1005	mm^2/m
Effective Depth of Tensile Rebar (A_{st}) (davg)	222	mm
	0.222	m

BENDING MOMENT (SLS)	8.000	kN.m/m
	8000.000	N.m/m
	Sagging Moment	Type
Cover (c_{min})	25	mm
	0.025	m
Slab Depth (h)	255	mm
Area of Steel (A_{st})	1005	mm^2/m

Potential Crack Length (a_{cr})	103.08	mm
Characteristic Cube Strength (f_{cu})	35	MPa
	35000000	Pa
Effective Depth of Tensile Rebar (A_{st}) (d)	222	mm
	0.222	m
Width Section (B) = b_t	1000	mm
Young's Modulus for Steel (E_s)	200	GPa
K_0 For Normal Weight Concrete	20	GPa
Young's Modulus for Concrete (E_c) = K_0 + 0.2GPa(f_{cu}/MPa)	27	GPa
Long Term Creep Coefficient	1	
Effective Young's Modulus for Concrete (E_{eff})	13.5	GPa
x = NA Depth for Cracked Section	67.77	mm
	0.06777	m
Found by: $(bx^2)/2 = (E_s/E_{eff}) * A_{st} * (d-x)$		
Quadratic Equation (a): B/2	500	
Quadratic Equation (b): $(E_s/E_{eff})*A_{st}$	14888.88889	
Quadratic Equation (c): $-(E_s/E_{eff}) * A_{st} * d$	-3305333.333	
$x = [-b +- sqrt(b^2 - 4ac)]/2a$		
$\varepsilon_1 = (M/[E_s*A_{st}*(d-x/3)])*[(h-x)/(d-x)]$	0.000242299	mm/mm
$\varepsilon_{ts} = [b_t*(h-x)^2]/[3*E_s*A_{st}*(d-x)]$	0.000376935	Units to use: mm, Mpa, N
$\varepsilon_m = \varepsilon_1 - \varepsilon_{ts}$	-0.000134635	mm/mm

Crack Width = $w_{max} = (3a_{cr} * \varepsilon_m)/(1+2(a_{cr}-c_{min})/(h-x))$	0.0227	mm

Crack Width Design

0.3 mm Crack width Limit

** 0.3mm for General Structures, 0.2mm for Water Retaining Structures, 0.1mm for Chemicals*

* Linked to Spreadsheet, Sheet 1

Project:	
Building:	
Date:	
Designer:	

Using Spreadsheet
1) Only edit "Yellow" cells
2) Input various Areas of steel to achieve your required crack width Table A.3
3) Factors such as T2 and T1 are site specific and have some effect on the cracks
4) Remember to change your φ value when your are refering to larger bar sizes

Crack Widths due to Thermal Shrinkage (BS 8007: APPENDIX A)

This provides crack width based on temperature (Thermal Shrinkage Considerations)

This is primary consideration when determining side reinforcing in members exceeding 750mm in height. It is also used to check crack widths for general members (Slabs, beams etc.)

Section Properties

h	[illegible]	mm	Slab Thickness
b	1000	mm	Width
Type of Member	Suspended Slab		
Top Zone			
h1	127.5	mm	Height of Surface Zone
Ac1	127500	mm^2	Area Concrete for Surface Zone
Bottom Zone			
h2	127.5	mm	Height of Surface Zone
Ac2	127500	mm^2	Area Concrete for Surface Zone

Check 1
Minimum Reinforcement

Pcrit	0.003556		
Fct	1.6	N/mm^2	Direct tensile strength of immature concrete at 3 days
Fy	450	N/mm^2	Refer to BS8110 table 3.1
Top Zone: Reinforcing for Side 1			
As1	453.33	mm^2/m	Minimum reinforcement to distribute thermal shrinkage cracks
Bottom Zone: Reinforcing for Side 2			
As2	453.33	mm^2/m	Minimum reinforcement to distribute thermal shrinkage cracks

Check 2
Crack Spacing

Top Zone: Reinforcing Provided				
Area steel	**As1**	1005	mm^2/m	OK
Type of Rebar	Deformed Bars (Y)			
fct/fb	0.67			Refer to BS8007 table A.1
φ1	16	mm		Diameter of reinforcing bar
Spacing of Bars	200	mm		
ρ1	0.007882353			Steel Ratio: Top Zone
smax1	680	mm		Max spacing of cracks
Bottom Zone: Reinforcing Provided				
Area steel	**As2**	1005	mm^2/m	OK
Type of Rebar	Deformed Bars (Y)			
fct/fb	0.67			Refer to BS8007 table A.1
φ2	16	mm		Diameter of reinforcing bar
Spacing of Bars	200	mm		
ρ2	0.007882353			Steel Ratio: Top Zone
smax2	680	mm		Max spacing of cracks

Check 3
Crack width

α	0.00001		See Fulton's Concrete Technology, Chapter 13
R	0.8		Value Ranges between 0 and 0.5. If unsure, use 0.5. Refer to BS 8007, Appendix A, Figure A.3. Can also refer to BS 8110 Part 2. Allows for Restraint Factor Greater than 0.5 (R > 0.5)
T1	28	°C	Fall in temperature between hydration peak temperature and ambient temperature. Refer to Table A.2 - interpolate between values for your slab thickness
T2	15	°C	Fall in temperature due to seasonal variations

[illegible]

Wmax	Short Term			** T1 Only - Early Thermal Cracking*
Top Zone	[illegible]	mm	OK	
Bottom Zone	[illegible]	mm	OK	

Wmax	Seasonal Variation Only			** T2 Only - Seasonal Variation Only*
Top Zone	[illegible]	mm	OK	
Bottom Zone	[illegible]	mm	OK	

Wmax	Long Term			** T1 & T2 - Long Term Thermal Cracking*
Top Zone	[illegible]	mm	OK	
Bottom Zone	[illegible]	mm	OK	

Project: Medupi Building — **Project No:** 14133
Component: Weigh Bridge control room — **Sheet No:**
Bearing pressure
Compiled By: DJJ vd M — **Date:** 01-06-09 — **Checked By:** — **Date:**

SABS 0160	Calculations

Bearing pressure of footings (Quick check (No moment))

Load of roof:

- Live load:

5.4.4.3 Inaccessible roof

$\left(0,3 + \frac{15 - (0,6 \cdot 4,4)}{60}\right) = 0,506\ kN/m$

Truss spacing = 0,6 m

- Dead Load = 1 kN/m (Insullation + ceiling)
- Selfweight = 1 kN/m (Assumed)

Area of roof $= (4.4 + 2 \cdot 0,585) \cdot (3,74 + 2 \cdot 0,585)$
$= 27,349$

Load on wall $= (0,506 \cdot 2) \cdot 20 \cdot 0,6$
$= 3,308\ kN$

Load of Brickwork (Worst case - No windows)

$3,655 \cdot 0,33 \cdot 1 \cdot 26 = 31,56$

Load of column

$0,45 \cdot 0,45 \cdot 1,360 \cdot 26 = 7,160$

Project:	Medupi Building
Component:	Weigh Bridge control room Bearing Pressure
Compiled By:	DJJ vd M
Date:	01 - 06 - 09
Checked By:	
Date:	
Project No:	14133
Sheet No:	

SABS 0160

Calculations

Load on footing

Roof: $3{,}308\,(4{,}4 \cdot 2 + 3{,}74 \cdot 2)$

$= 53{,}85$

Brick work : $3{,}56 \cdot (4{,}4 + 3{,}74)2$

$= 513{,}8$

Floor slab $= 0{,}255 \cdot (4{,}4 \cdot 3{,}74) \cdot 26$

$= 109{,}103$

Down stand $= 0{,}245 \cdot 0{,}45 \cdot (4{,}4 \cdot 2 + 3{,}74 \cdot 2) \cdot 26$

$= 46{,}67$

Column $= 0{,}45 \cdot 0{,}45 \cdot 1{,}36 \cdot 26$

$= 7{,}16 \cdot 4$

$= 28{,}64$

Footing $= (0{,}45 \cdot 0{,}45 \cdot 0{,}5 \cdot 26 + 0{,}4 \cdot 2{,}3 \cdot 2{,}3 \cdot 26)\,4$

$= 57{,}649$

Total $= 809{,}712$

Total/footing $= \dfrac{809{,}712}{4} = \dfrac{202{,}5}{2{,}6 \cdot 2{,}6}$

$= 29{,}95$ Fine

(Moments will require big footing)

Project:	Medupi Buildings
Project No:	14133
Component:	Weigh Bridge Control room slab calc.
Sheet No:	
Compiled By:	DJJ vd M
Date:	09-06-01
Checked By:	
Date:	

Rev SABS 0160

Calculations

Previously calc. (Bearing Pt) — Load on Beam : 34 kN

Table 4. — Live load on shells : 2,5 kN (General office use)

Selfweight of slab + Beam = Calculated and added by Prokon.

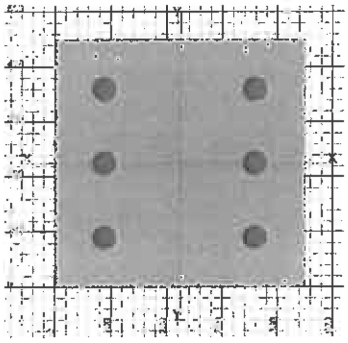
Rectangular column design

C11

Rectangular column design by *PROKON*. (RecCol Ver W2.4.12 - 27 Mar 2009)

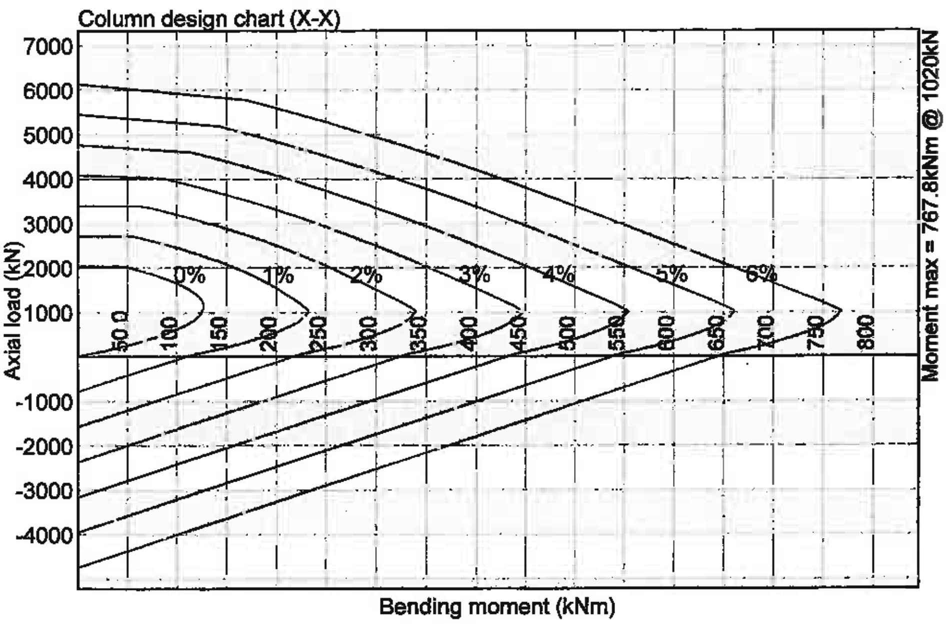
Design code : SABS 0100 - 2000

General design parameters:

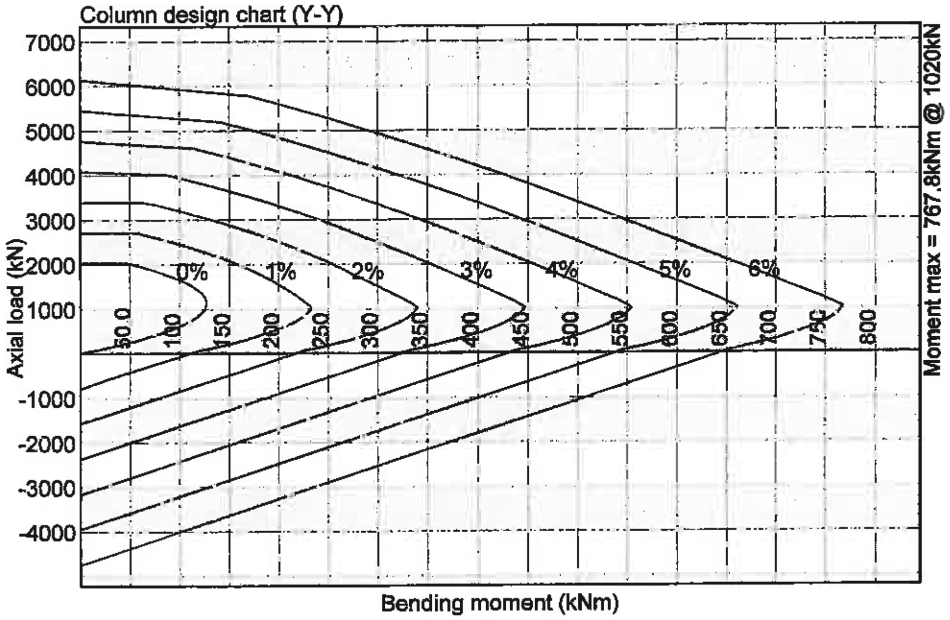


Given:
- h = 450 mm
- b = 450 mm
- d'x = 90 mm
- d'y = 90 mm
- Lo = 1.200 m
- fcu = 25 MPa
- fy = 450 MPa

Design chart for bending about the X-X axis:



Design chart for bending about the Y-Y axis:



Therefore:
$A_c = b \cdot h = 202500.00\ mm^2$
$h' = h - d'x = 360$ mm
$b' = b - d'y = 360$ mm

Assumptions:
(1) The general conditions of clause 4.7.1 are applicable.
(2) The section is symmetrically reinforced.
(3) The specified design axial loads include the self-weight of the column.
(4) The design axial loads are taken constant over the height of the column.

Design approach:

The column is designed using the following procedure:
(1) The column design charts are constructed.
(2) The design axis and design ultimate moment is determined .
(3) The steel required for the design axial force and moment is read from the relevant design chart.
(4) The area steel perpendicular to the design axis is read from the relevant design chart.
(5) The procedure is repeated for each load case.
(6) The critical load case is identified as the case yielding the largest steel area about the design axis.

Through inspection:
Load case 1 is critical.

Check column slenderness:

End fixity and bracing for bending about the X-X axis:
At the top end: Condition 2 (partially fixed).
At the bottom end: Condition 1 (fully fixed).
The column is unbraced.
∴ ßx = 1.30 *Table 19*

End fixity and bracing for bending about the Y-Y axis:
At the top end: Condition 2 (partially fixed).
At the bottom end: Condition 1 (fully fixed).
The column is unbraced.
∴ ßy = 1.30 *Table 19*

Effective column height:
lex = ßx· Lo = 1.560 m
ley = ßy· Lo = 1.560 m

Check if the column is slender: *4.7.1.4*
lex/h = 3.5 < 10
ley/b = 3.5 < 10
∴ The column is short.

Initial moments:

The column is bent in double curvature about the X-X axis:
M1 = Smaller initial end moment = 12.7 kNm
M2 = Larger initial end moment = 24.6 kNm

The initial moment near mid-height of the column : *4.7.3.2.2*
∴ Mi = -0.4M1 + 0.6M2 ≥ 0.4M2 = 24.6 kNm

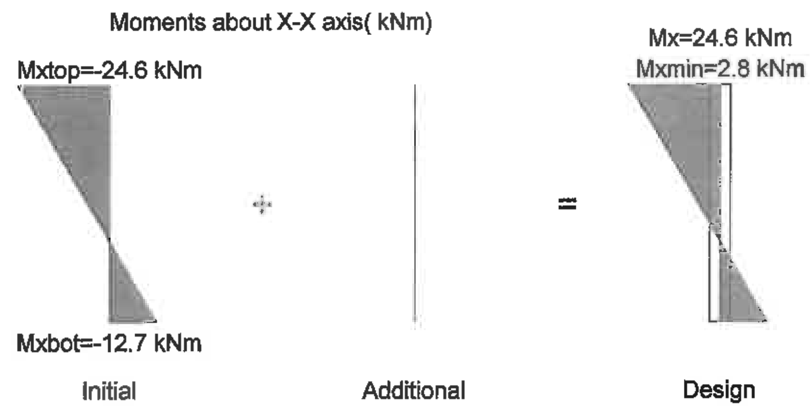
The column is bent in double curvature about the Y-Y axis:
M1 = Smaller initial end moment = 20.1 kNm
M2 = Larger initial end moment = 40.0 kNm

The initial moment near mid-height of the column : *4.7.3.2.2*
∴ Mi = -0.4M1 + 0.6M2 ≥ 0.4M2 = 40.0 kNm

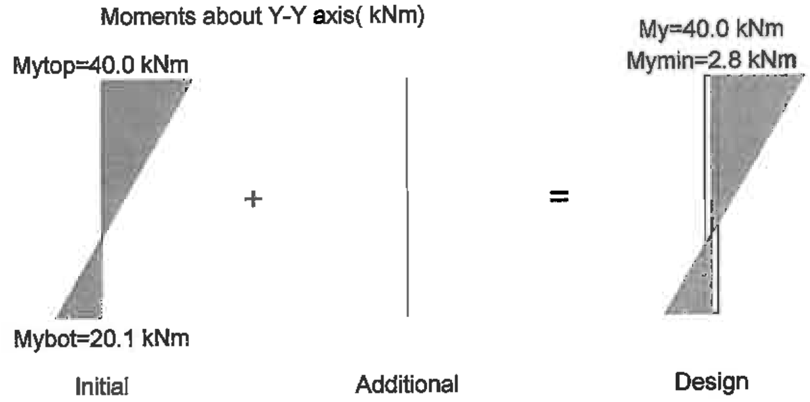
Design ultimate load and moment:

Design axial load:
P_u = 155.3 kN

Moment distribution along the height of the column for bending about the X-X:
At the top, Mx = 24.6 kNm
Near mid-height, Mx = 9.8 kNm
At the bottom, Mx = 12.7 kNm



Moment distribution along the height of the column for bending about the Y-Y:
At the top, My = 40.0 kNm
Near mid-height, My = 16.0 kNm
At the bottom, My = 20.1 kNm



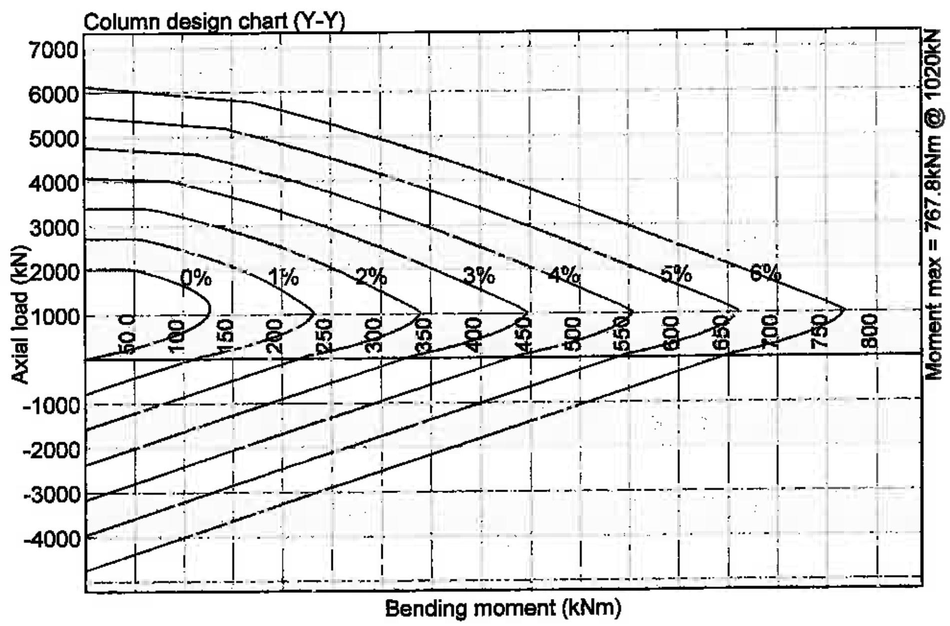
Check for miminum eccentricity: 4.7.2.3
For bi-axial bending, it is only necessary to ensure that the eccentricity excceeds the minimum about one axis at a time.

For the worst effect, apply the minimum eccentricity about the minor axis:
Use emin = 20mm
∴ Mmin = 3.1 kNm about the Y-Y axis.

Design of column section for ULS:

Through inspection:
The critical section lies at the top end of the column.

The column is bi-axially bent. The moments are added vectorially to obtain the design moment:
For bending about the design axis - use the Y-axis:

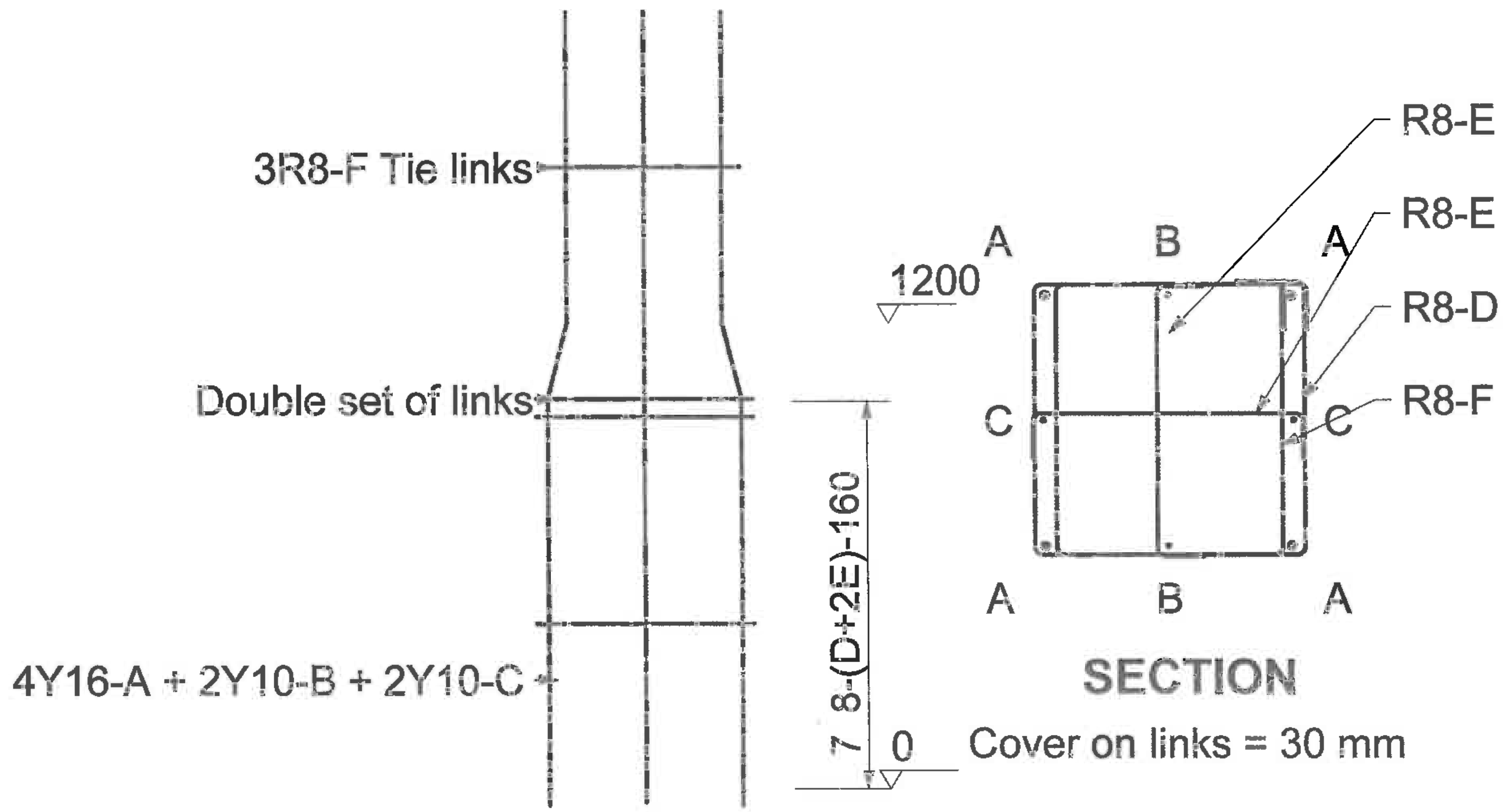


Summary of design calculations:

Design results for all load cases:

Load case	Axis	N (kN)	M1 (kNm)	M2 (kNm)	Mi (kNm)	Madd (kNm)	Design	M (kNm)	M' (kNm)	Asc (mm²)
DL	X-X Y-Y	155.3	12.7 20.1	24.6 40.0	24.6 40.0	0.0 0.0	Y-Y Top	24.6 40.0	53.3	826 (0.41%) 826 (0.41%)
WL	X-X Y-Y	-0.9	1.4 0.1	1.5 0.1	1.5 0.1	0.0 0.0	X-X Bottom	1.5 0.1	1.5	826 (0.41%) 826 (0.41%)
LL	X-X Y-Y	3.2	0.7 0.9	1.4 1.8	1.4 1.8	0.0 0.0	Y-Y Top	1.4 1.8	2.5	826 (0.41%) 826 (0.41%)
LC1	X-X Y-Y	233.0	19.0 30.2	36.9 60.0	36.9 60.0	0.0 0.0	Y-Y Top	36.9 60.0	80.7	826 (0.41%) 826 (0.41%)
LC2	X-X Y-Y	191.6	16.3 25.6	31.7 50.8	31.7 50.8	0.0 0.0	Y-Y Top	31.7 50.8	68.3	826 (0.41%) 826 (0.41%)
LC3	X-X Y-Y	186.9	13.6 24.6	28.4 48.8	28.4 48.8	0.0 0.0	Y-Y Top	28.4 48.8	64.4	826 (0.41%) 826 (0.41%)
LC4	X-X Y-Y	138.7	9.4 18.1	20.3 35.9	20.3 35.9	0.0 0.0	Y-Y Top	20.3 35.9	46.8	826 (0.41%) 826 (0.41%)

Load case 1 is critical.


3R8-F Tie links
Double set of links
4Y16-A + 2Y10-B + 2Y10-C
7 8-(D+2E)-160
1200
0
A
B
A
C
C
A
B
A
R8-E
R8-E
R8-D
R8-F
SECTION
Cover on links = 30 mm

Column Base Design :

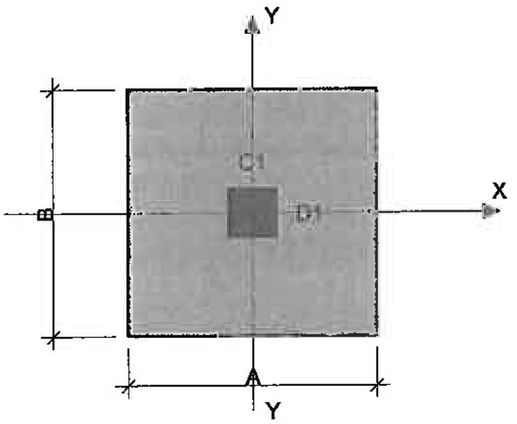
C15

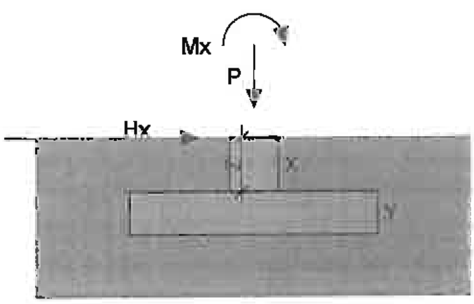
Input Data

Base length A (m)	2.3	
Base width B (m)	2.3	
Column(s)	Col 1	Col 2
C (m)	.45	
D (m)	.45	
E (m)		
F (m)		
Stub column height X (m)	.5	
Base depth Y (m)	.4	
Soil cover Z (m)	.5	
Concrete density (kN/m^3)	25	
Soil density (kN/m^3)	18	
Soil friction angle (°)	30	
Base friction constant	0.5	
Rebar depth top X (mm)	80	
Rebar depth top Y (mm)	96	
Rebar depth bottom X (mm)	80	
Rebar depth bottom Y (mm)	96	
ULS ovt. LF: Self weight	0.9	
ULS LF: Self weight	1.2	
Max. SLS bearing pr. (kN/m^2)	175	
S.F. Overturning (ULS)	2	
S.F. Slip (ULS)	1.5	
fcu base (MPa)	25	
fcu columns (MPa)	30	
fy (MPa)	450	

		Unfactored Loads						
Load Case	Col no.	LF ULS ovt	LF ULS	P (kN)	Hx (kN)	Hy (kN)	Mx (kNm)	My (kNm)
DL	1	1	1	155.34	-30.60	49.36	-12.66	20.14
WL	1	1	1	-0.88	2.39	-0.09	1.51	-0.05
LL	1	1	1	3.21	-1.68	2.22	-0.69	0.91
LC1	1	1	1.50	155.34	-30.60	49.36	-12.66	20.14
LC2	1	1	1.20	155.34	-30.60	49.36	-12.66	20.14
	1	1	1.60	3.21	-1.68	2.22	-0.69	0.91
LC3	1	1	1.20	155.34	-30.60	49.36	-12.66	20.14
	1	1	1.30	-0.88	2.39	-0.09	1.51	-0.05
	1	1	0.50	3.21	-1.68	2.22	-0.69	0.91
LC4	1	1	0.90	155.34	-30.60	49.36	-12.66	20.14
	1	1	1.30	-0.88	2.39	-0.09	1.51	-0.05

Sketch of Base

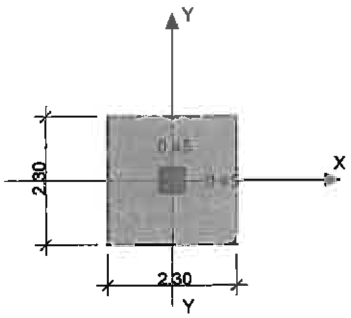




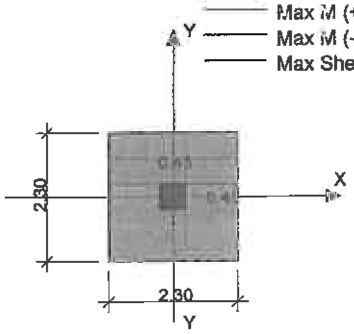
Output for Load Case DL

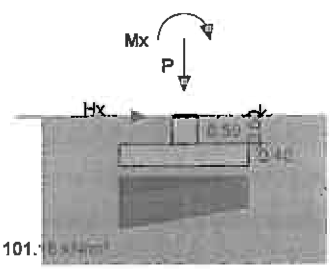
Output for Load Case DL	
Soil pressure (ULS) (kN/m²)	103.98
Soil pressure (SLS) (kN/m²)	101.18
SF overturning (SLS)	3.88
SF overturning (ULS)	3.73
Safety Factor slip (ULS)	3.21
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	18.83
Reinforcement X (mm²/m)	158
Design moment Y (kNm/m)	22.59
Reinforcement Y (mm²/m)	200
Top	
Design moment X (kNm/m)	0.00
Reinforcement X (mm²/m)	0
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.065
vc (MPa)	0.301
Linear Shear Y (MPa)	0.083
vc (MPa)	0.305
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	0.081
vc (MPa)	0.303
Cost	226.48

Load Case:DL

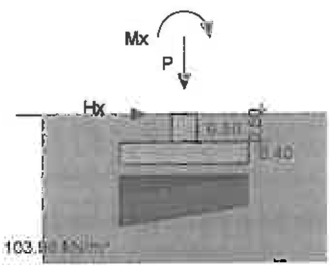


Legend
Max M (+)
Max M (-)
Max Shear





Soil Pressures at SLS

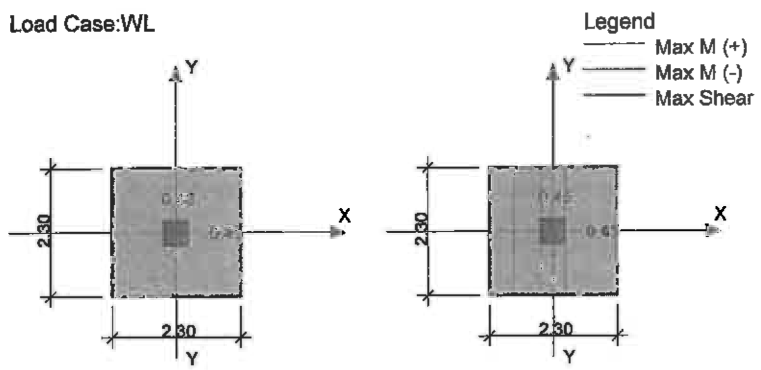


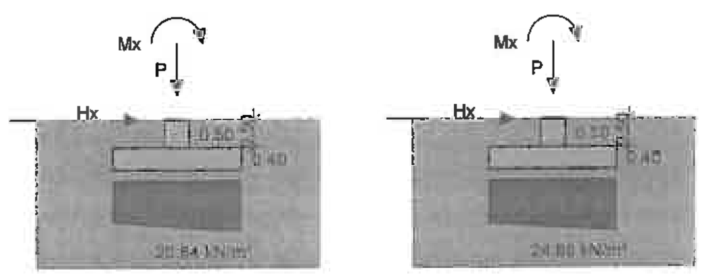
Soil Pressures at ULS

PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title		
	Client		
	Calcs by	Checked by	Date

Output for Load Case WL

Output for Load Case WL	
Soil pressure (ULS) (kN/m²)	24.66
Soil pressure (SLS) (kN/m²)	20.84
SF overturning (SLS)	31.50
SF overturning (ULS)	28.32
Safety Factor slip (ULS)	40.29
Safety Factor uplift (ULS)	103.52
Bottom	
Design moment X (kNm/m)	0.56
Reinforcement X (mm²/m)	5
Design moment Y (kNm/m)	0.02
Reinforcement Y (mm²/m)	0
Top	
Design moment X (kNm/m)	-0.57
Reinforcement X (mm²/m)	5
Design moment Y (kNm/m)	-0.02
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.002
vc (MPa)	0.301
Linear Shear Y (MPa)	0.000
vc (MPa)	0.305
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	0.002
vc (MPa)	0.303
Cost	211.82





Soil Pressures at SLS

Soil Pressures at ULS

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	Job Title		
	Client		
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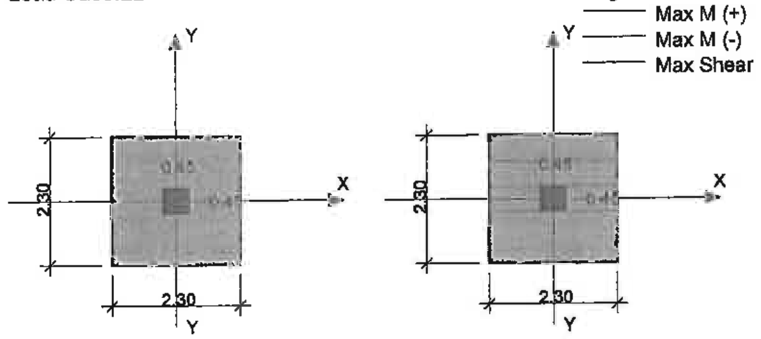
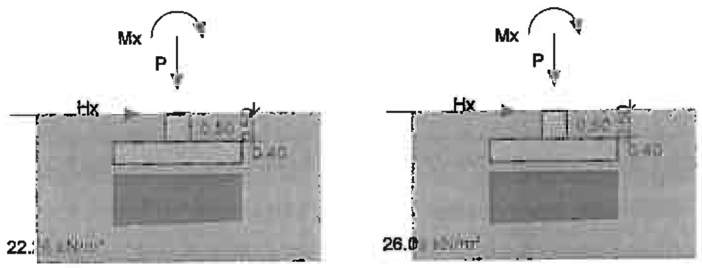
Output for Load Case LL

Output for Load Case LL	
Soil pressure (ULS) (kN/m²)	26.09
Soil pressure (SLS) (kN/m²)	22.26
SF overturning (SLS)	32.92
SF overturning (ULS)	29.73
Safety Factor slip (ULS)	39.89
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	0.67
Reinforcement X (mm²/m)	6
Design moment Y (kNm/m)	0.78
Reinforcement Y (mm²/m)	7
Top	
Design moment X (kNm/m)	-0.01
Reinforcement X (mm²/m)	0
Design moment Y (kNm/m)	-0.12
Reinforcement Y (mm²/m)	1
Linear Shear X (MPa)	0.002
vc (MPa)	0.301
Linear Shear Y (MPa)	0.003
vc (MPa)	0.305
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	0.003
vc (MPa)	0.303
Cost	212.12

Load Case:LL

Legend



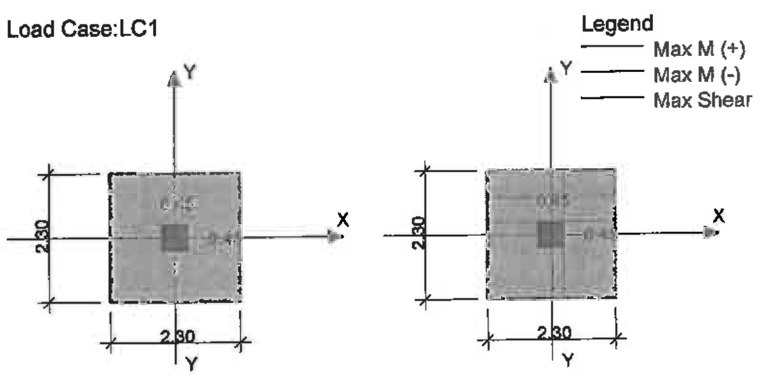


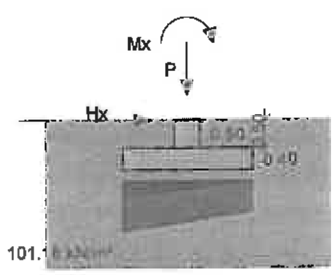
Soil Pressures at SLS

Soil Pressures at ULS

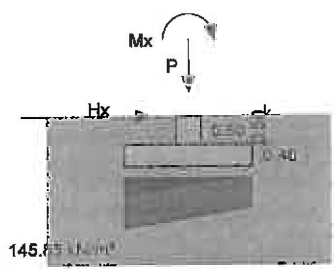
Output for Load Case LC1

Output for Load Case LC1	
Soil pressure (ULS) (kN/m²)	145.83
Soil pressure (SLS) (kN/m²)	101.18
SF overturning (SLS)	3.88
SF overturning (ULS)	3.73
Safety Factor slip (ULS)	2.58
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	28.85
Reinforcement X (mm²/m)	243
Design moment Y (kNm/m)	34.47
Reinforcement Y (mm²/m)	305
Top	
Design moment X (kNm/m)	0.00
Reinforcement X (mm²/m)	0
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.100
vc (MPa)	0.301
Linear Shear Y (MPa)	0.127
vc (MPa)	0.305
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	0.124
vc (MPa)	0.303
Cost	234.34





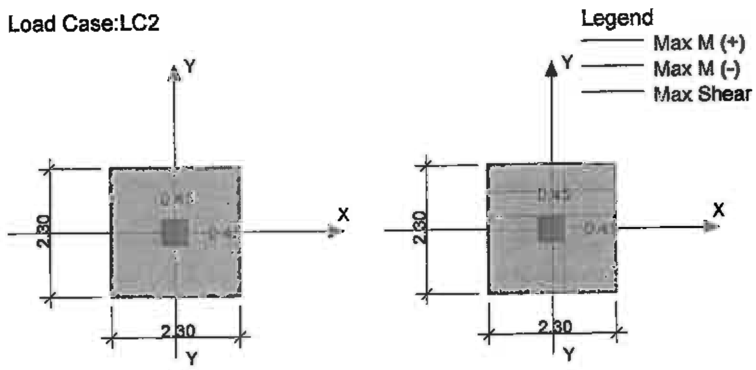
Soil Pressures at SLS

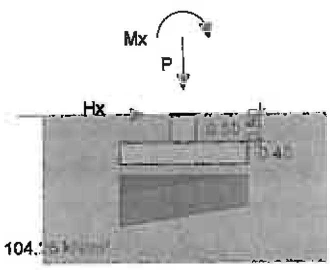


Soil Pressures at ULS

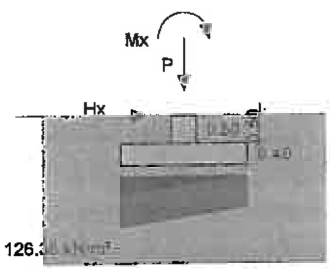
Output for Load Case LC2

Output for Load Case LC2	
Soil pressure (ULS) (kN/m^2)	126.36
Soil pressure (SLS) (kN/m^2)	104.26
SF overturning (SLS)	3.75
SF overturning (ULS)	3.60
Safety Factor slip (ULS)	2.76
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	24.02
Reinforcement X (mm^2/m)	202
Design moment Y (kNm/m)	28.71
Reinforcement Y (mm^2/m)	254
Top	
Design moment X (kNm/m)	0.00
Reinforcement X (mm^2/m)	0
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm^2/m)	0
Linear Shear X (MPa)	0.083
vc (MPa)	0.301
Linear Shear Y (MPa)	0.106
vc (MPa)	0.305
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	0.103
vc (MPa)	0.303
Cost	230.54





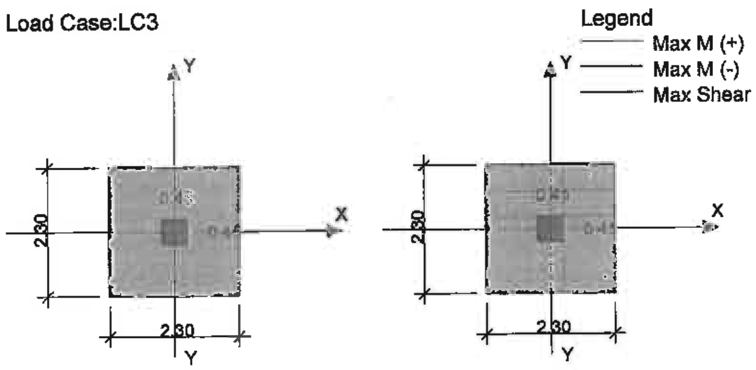
Soil Pressures at SLS

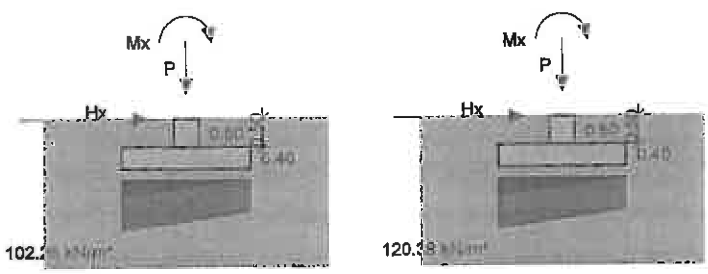


Soil Pressures at ULS

Output for Load Case LC3

Output for Load Case LC3	
Soil pressure (ULS) (kN/m²)	120.38
Soil pressure (SLS) (kN/m²)	102.26
SF overturning (SLS)	3.83
SF overturning (ULS)	3.68
Safety Factor slip (ULS)	2.91
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	22.36
Reinforcement X (mm²/m)	188
Design moment Y (kNm/m)	27.71
Reinforcement Y (mm²/m)	245
Top	
Design moment X (kNm/m)	0.00
Reinforcement X (mm²/m)	0
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.077
vc (MPa)	0.301
Linear Shear Y (MPa)	0.102
vc (MPa)	0.305
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	0.099
vc (MPa)	0.303
Cost	229.59





Soil Pressures at SLS

Soil Pressures at ULS

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	Client		
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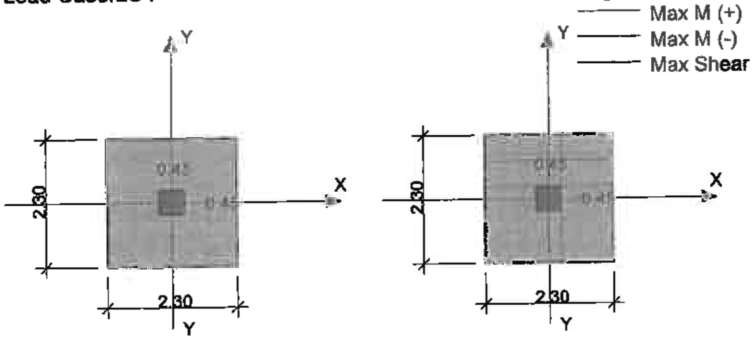
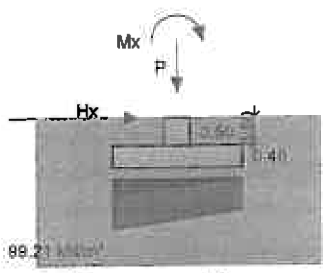
Output for Load Case LC4

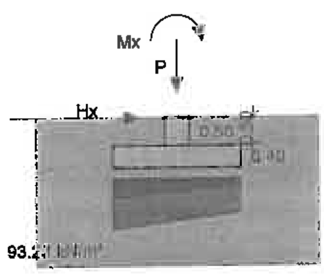
Output for Load Case LC4	
Soil pressure (ULS) (kN/m²)	93.23
Soil pressure (SLS) (kN/m²)	99.21
SF overturning (SLS)	3.97
SF overturning (ULS)	3.81
Safety Factor slip (ULS)	3.50
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	16.13
Reinforcement X (mm²/m)	136
Design moment Y (kNm/m)	20.22
Reinforcement Y (mm²/m)	179
Top	
Design moment X (kNm/m)	0.00
Reinforcement X (mm²/m)	0
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.056
vc (MPa)	0.301
Linear Shear Y (MPa)	0.074
vc (MPa)	0.305
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	0.073
vc (MPa)	0.303
Cost	224.66

Load Case:LC4



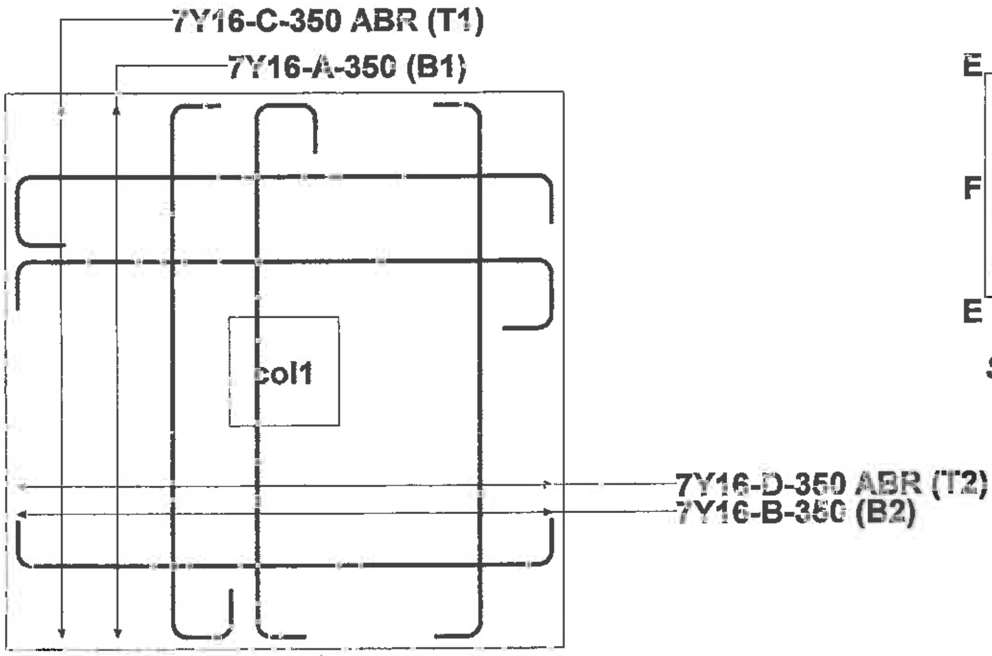


Soil Pressures at SLS

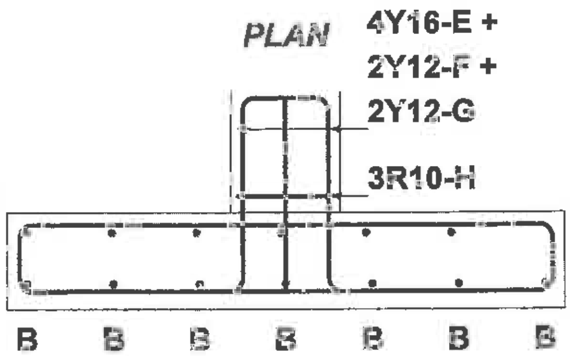


Soil Pressures at ULS

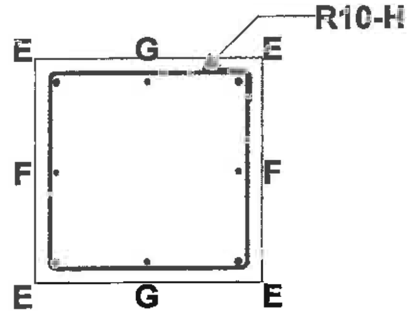
Footing A + B



PLAN



SECTION



SECTION: col1

Column Base Design :

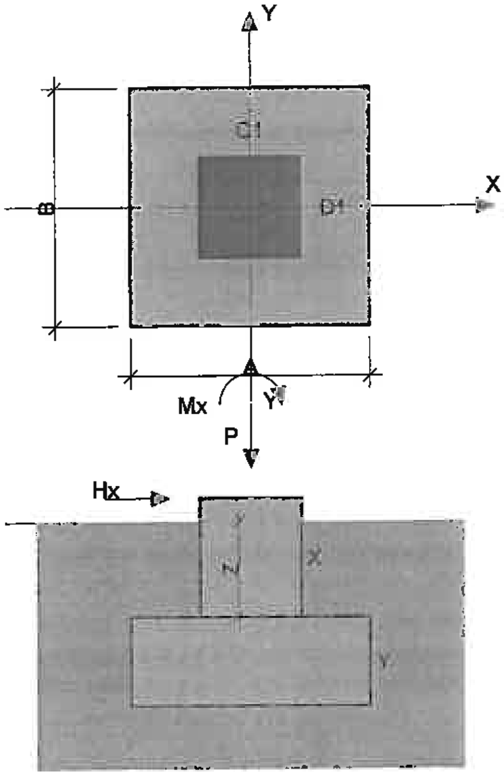
C15

Input Data

Base length A	(m)	.8	
Base width B	(m)	.8	
Column(s)		Col 1	Col 2
C	(m)	0.34	
D	(m)	0.34	
E	(m)		
F	(m)		
Stub column height X	(m)	.4	
Base depth Y	(m)	0.30	
Soil cover Z	(m)	.32	
Concrete density	(kN/m^3)	25	
Soil density	(kN/m^3)	18	
Soil friction angle (°)		30	
Base friction constant		0.5	
Rebar depth top X	(mm)	80	
Rebar depth top Y	(mm)	96	
Rebar depth bottom X	(mm)	80	
Rebar depth bottom Y	(mm)	96	
ULS ovt. LF: Self weight		0.9	
ULS LF: Self weight		1.2	
Max. SLS bearing pr.	(kN/m^2)	175	
S.F. Overturning (ULS)		2	
S.F. Slip (ULS)		1.5	
fcu base	(MPa)	25	
fcu columns	(MPa)	30	
fy	(MPa)	450	

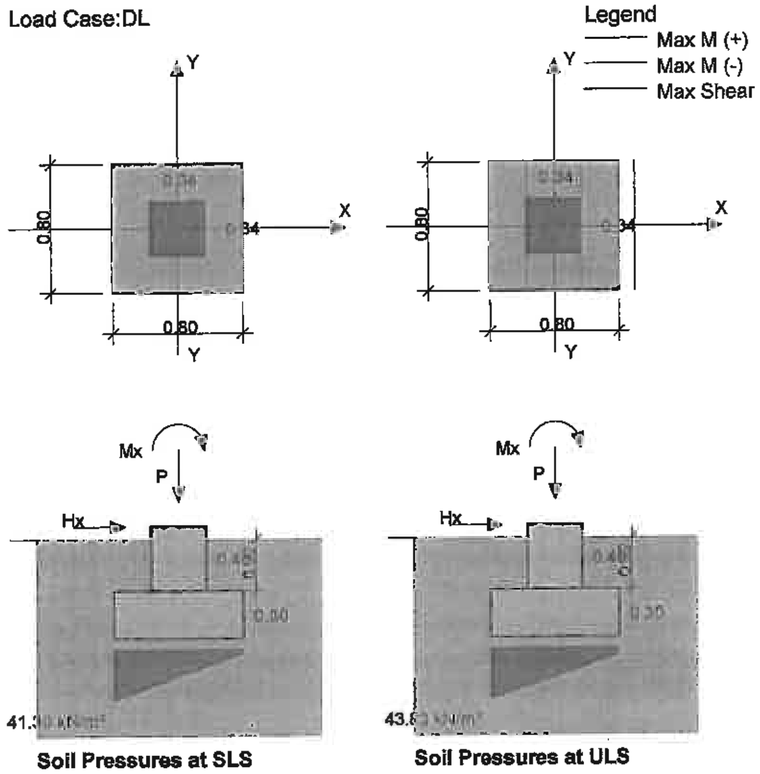
		Unfactored Loads						
Load Case	Col no.	LF ULS ovt	LF ULS	P (kN)	Hx (kN)	Hy (kN)	Mx (kNm)	My (kNm)
DL	1	1	1	3.86	-3.35	-0.25	0.74	-0.01
LL	1	1	1	4.64	-3.65	-0.40	1.00	-0.02
C1	1	1	1.50	3.86	-3.35	-0.25	0.74	-0.01
C2	1	1	1.20	3.86	-3.35	-0.25	0.74	-0.01
	1	1	1.60	4.64	-3.65	-0.40	1.00	-0.02

Sketch of Base



Output for Load Case DL

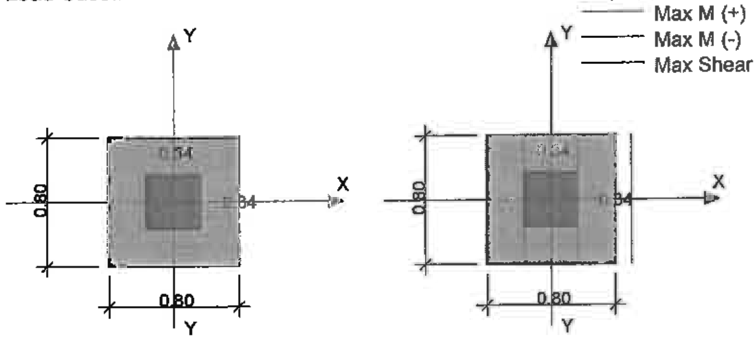
Output for Load Case DL	
Soil pressure (ULS) (kN/m²)	43.83
Soil pressure (SLS) (kN/m²)	41.30
SF overturning (SLS)	3.18
SF overturning (ULS)	2.96
Safety Factor slip (ULS)	4.12
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	0.59
Reinforcement X (mm²/m)	7
Design moment Y (kNm/m)	0.23
Reinforcement Y (mm²/m)	3
Top	
Design moment X (kNm/m)	-0.22
Reinforcement X (mm²/m)	3
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.000
vc (MPa)	0.331
Linear Shear Y (MPa)	0.000
vc (MPa)	0.337
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	N.A.
vc (MPa)	N.A.
Cost	19.25

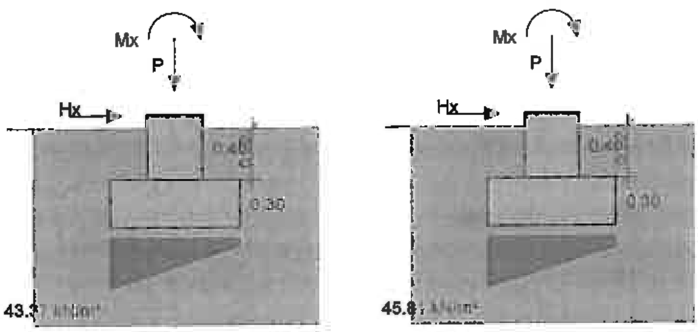


Output for Load Case LL

Output for Load Case LL	
Soil pressure (ULS) (kN/m²)	45.81
Soil pressure (SLS) (kN/m²)	43.37
SF overturning (SLS)	3.44
SF overturning (ULS)	3.21
Safety Factor slip (ULS)	3.93
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	0.61
Reinforcement X (mm²/m)	7
Design moment Y (kNm/m)	0.29
Reinforcement Y (mm²/m)	4
Top	
Design moment X (kNm/m)	-0.17
Reinforcement X (mm²/m)	2
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.000
vc (MPa)	0.331
Linear Shear Y (MPa)	0.000
vc (MPa)	0.337
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	N.A.
vc (MPa)	N.A.
Cost	19.26

Load Case:LL





Soil Pressures at SLS

Soil Pressures at ULS

Output for Load Case C1

Output for Load Case C1	
Soil pressure (ULS) (kN/m²)	58.00
Soil pressure (SLS) (kN/m²)	41.30
SF overturning (SLS)	3.18
SF overturning (ULS)	2.96
Safety Factor slip (ULS)	2.94
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	0.88
Reinforcement X (mm²/m)	11
Design moment Y (kNm/m)	0.33
Reinforcement Y (mm²/m)	4
Top	
Design moment X (kNm/m)	-0.34
Reinforcement X (mm²/m)	4
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.000
vc (MPa)	0.331
Linear Shear Y (MPa)	0.000
vc (MPa)	0.337
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	N.A.
vc (MPa)	N.A.
Cost	19.28

Job Number		Sheet
Job Title		
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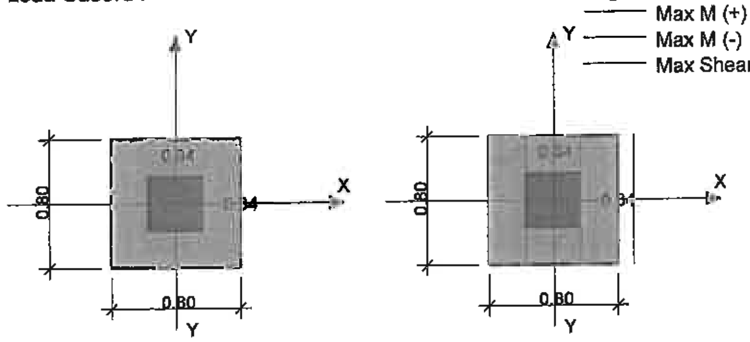
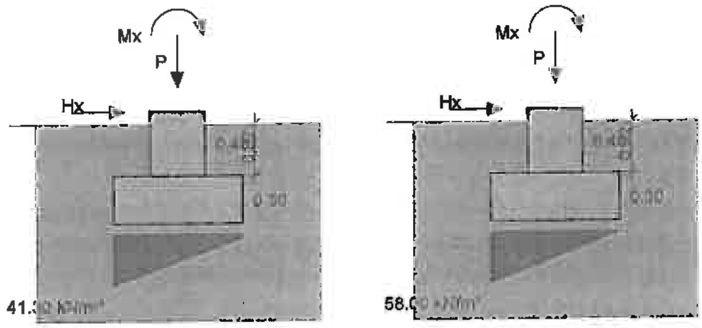
Load Case:C1

Legend
Max M (+)
Max M (-)
Max Shear





Soil Pressures at SLS

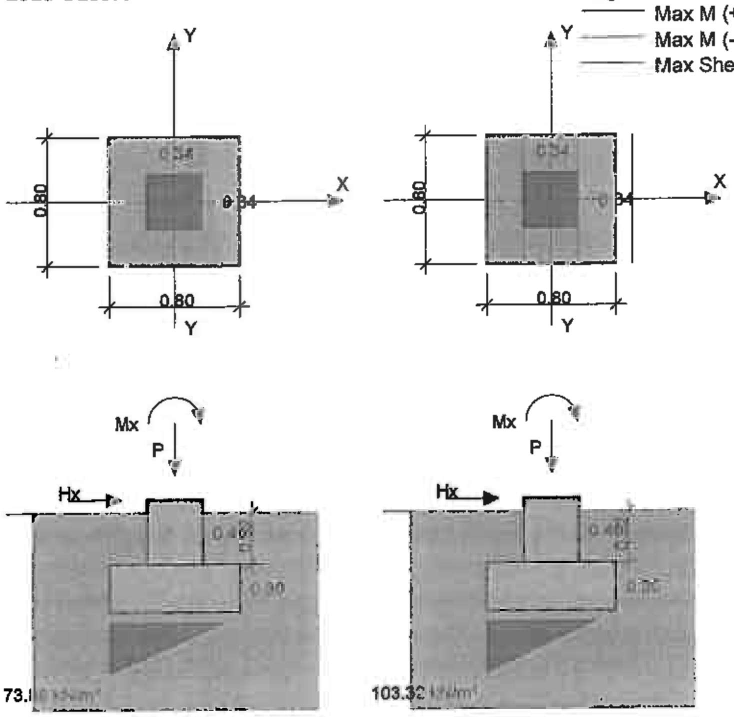
Soil Pressures at ULS

Output for Load Case C2

Output for Load Case C2	
Soil pressure (ULS) (kN/m²)	103.32
Soil pressure (SLS) (kN/m²)	73.86
SF overturning (SLS)	2.19
SF overturning (ULS)	2.07
Safety Factor slip (ULS)	1.82
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	1.74
Reinforcement X (mm²/m)	21
Design moment Y (kNm/m)	0.68
Reinforcement Y (mm²/m)	9
Top	
Design moment X (kNm/m)	-0.42
Reinforcement X (mm²/m)	5
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.000
vc (MPa)	0.331
Linear Shear Y (MPa)	0.000
vc (MPa)	0.337
Linear Shear Other (MPa)	0.000
Punching Shear (MPa)	N.A.
vc (MPa)	N.A.
Cost	19.35

Load Case:C2

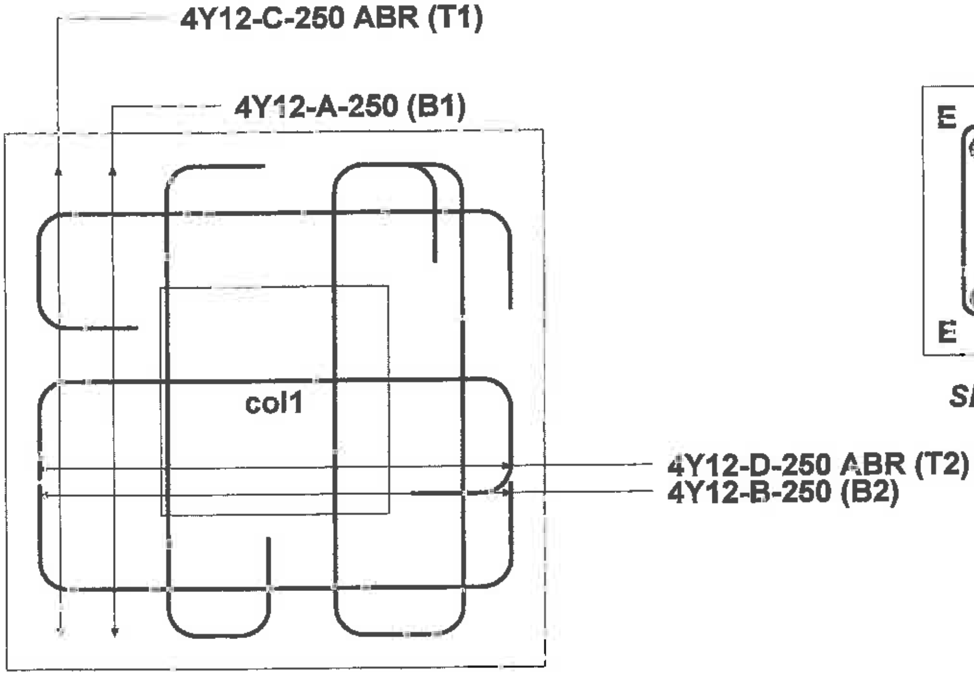
Legend
Max M (+)
Max M (-)
Max Shear



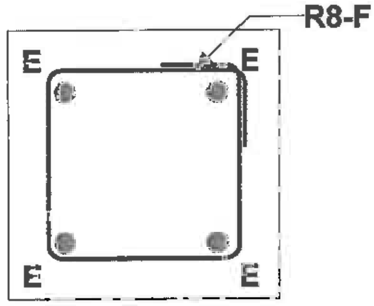
Soil Pressures at SLS

Soil Pressures at ULS

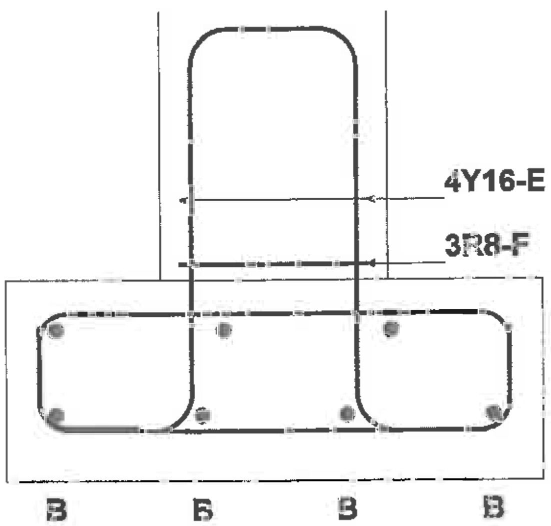
Footing C



PLAN



SECTION: col1



SECTION

3.2 Steel Members

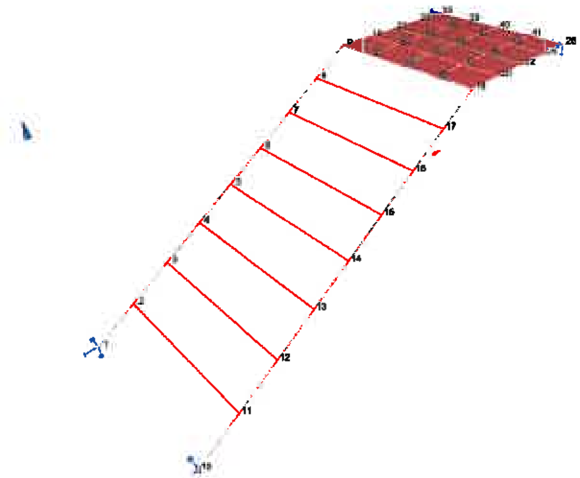
STAIRS

Frame Analysis Ver W2.6.27 - 07 Jan 2014
Input file:I:\14133_Medupi PS and WTP\28 Weigh bridge\Design\Control Room\12_03_2014\STAIRS.A03
Created : 13/03/2014 7:53:40 AM

A03

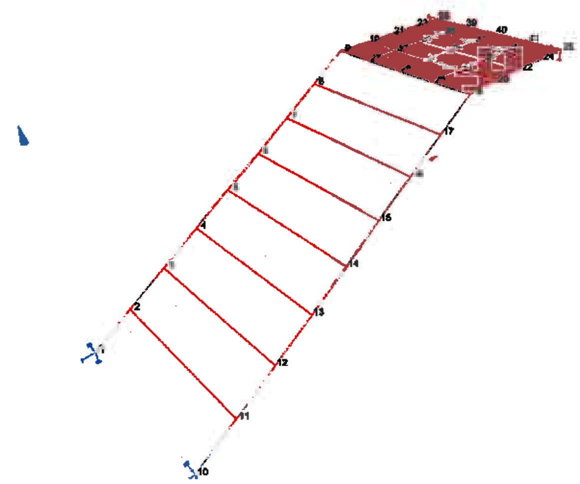
Deflections for Load Case C1

Maximum Deflections for Load Combination C1:
X :0.05 mm at node 15
Y :-0.14 mm at node 34
Z :0.00 mm at node 11

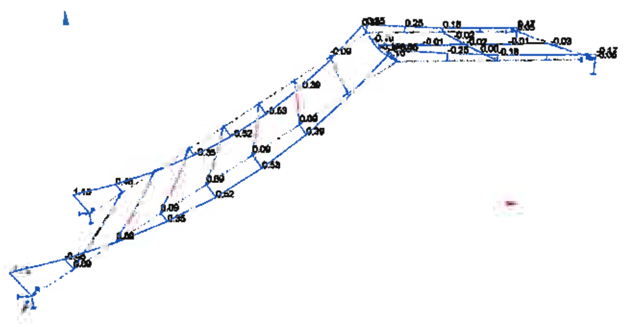


Deflections for Load Case C2

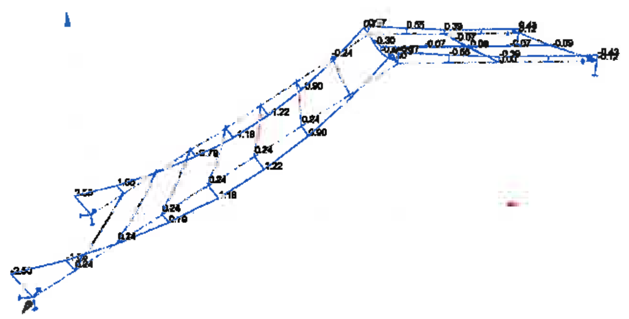
Maximum Deflections for Load Combination C2:
X :0.13 mm at node 15
Y :-0.57 mm at node 34
Z :0.00 mm at node 11



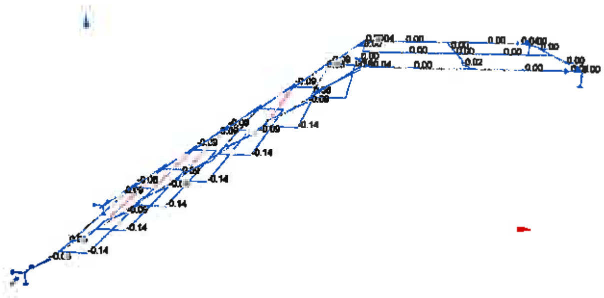
X-Moments for Load Combination C1



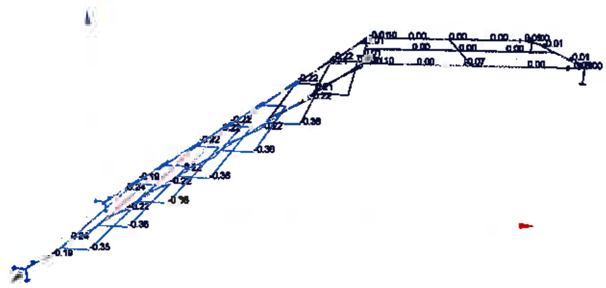
X-Moments for Load Combination C2



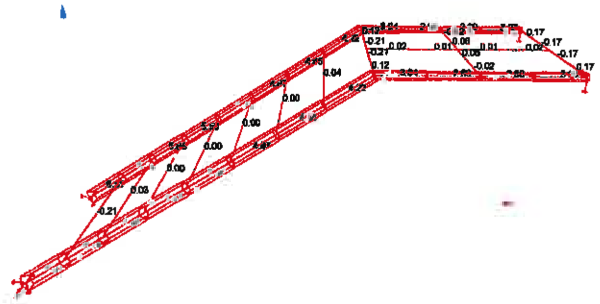
Y-Moments for Load Combination C1



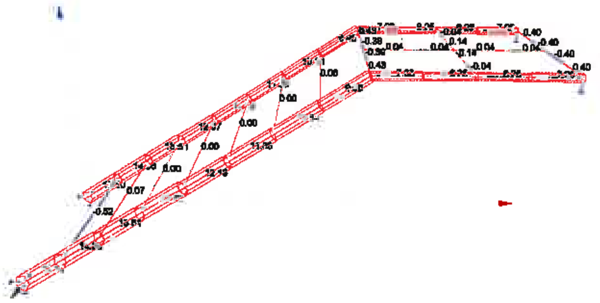
Y-Moments for Load Combination C2



Axial Forces for Load Combination C1



Axial Forces for Load Combination C2



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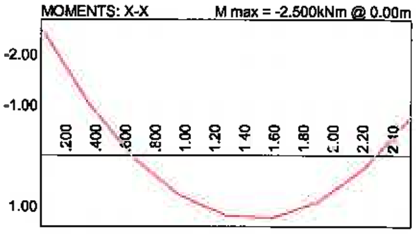
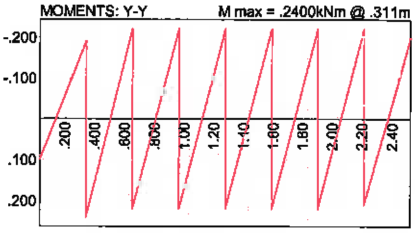
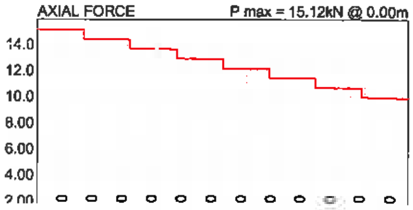
Design of steel member (C152x76)-Design for Combined Stresses

Task: Task 1








```
Combine Ver W2.6.11
Element  1-9                    Evaluate current section
Section name 1

Lx Eff  = 2.488 m          W1x = 1.00
Ly Eff  = 2.488 m          W1y = 1.00
Le Eff  = 2.488 m          W2  = 1.45
Fy = 350 MPa               Fu  = 450 MPa
Tension area factor (Ane/Ag)  = 1.00
Flange class: 3            Web class    : 3

Critical Load Case : C2

Section C152x76      Channels (Web vert)

SANS 10162 - 2005 13.8.3 :
a) Cross-sectional strength (Crit. pos.= 0.000 m)
   Cu   Mux   Muy   15.1   2.51   .104
   -- + --- + --- = ---- + ---- + ---- = 0.11                 OK
   Cr   Mrx   Mry   718    35.3   6.62

b) Overall member strength
   Cu   U1xMux   U1yMuy   15.1   2.51   .250
   -- + ------ + ------ = ---- + ---- + ---- = 0.17           OK
   Cr    Mrx      Mry     261    35.3   6.62

c) Lateral torsional buckling strength
   Cu   U1xMux   U1yMuy   15.1   2.51   .250
   -- + ------ + ------ = ---- + ---- + ---- = 0.17           OK
   Cr    Mrx      Mry     261    33.7   6.62

13.4:Shear

   Vux<Vrx       4.7 < 202.8                                  OK
   Vuy<Vry       1.5 < 237.6                                  OK


Slenderness Ratio: L/r = 112                                  OK

--------------------------------------------------------------
Combine Ver W2.6.11
Element  9-18                   Evaluate current section
Section name 1

Lx Eff  = 1.100 m          W1x = 1.00
Ly Eff  = 1.100 m          W1y = 1.00
Le Eff  = 1.100 m          W2  = 1.00
Fy = 350 MPa               Fu  = 450 MPa
Tension area factor (Ane/Ag)  = 1.00
Flange class: 3            Web class    : 3

Critical Load Case : C2

Section C152x76      Channels (Web vert)

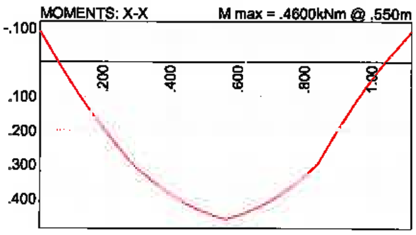
SANS 10162 - 2005 13.8.3 :
a) Cross-sectional strength (Crit. pos.= 0.275 m)
   Cu   Mux   Muy   .430   .300   .080
   -- + --- + --- = ---- + ---- + ---- = 0.02                 OK
   Cr   Mrx   Mry   718    35.3   6.62

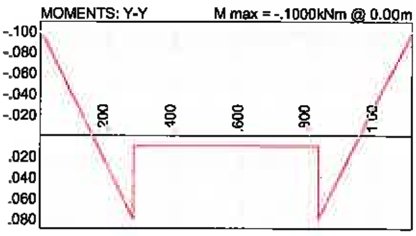
b) Overall member strength
   Cu   U1xMux   U1yMuy   .430   .460   .100
   -- + ------ + ------ = ---- + ---- + ---- = 0.03           OK
   Cr    Mrx      Mry     552    35.3   6.62

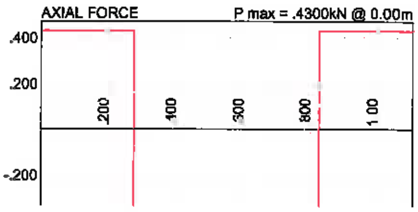
c) Lateral torsional buckling strength
   Cu   U1xMux   U1yMuy   .430   .460   .100
   -- + ------ + ------ = ---- + ---- + ---- = 0.03           OK
   Cr    Mrx      Mry     582    35.3   6.62

13.4:Shear

   Vux<Vrx       1.7 < 202.8                                  OK
   Vuy<Vry       0.7 < 237.6                                  OK
```







```
Slenderness Ratio: L/r =  49                                OK

------------------------------------------------------------

Combine Ver W2.6.11
Element 9-25                     Evaluate current section
Section name 1

Lx Eff = 1.100 m          W1x = 1.00
Ly Eff = 1.100 m          W1y = 1.00
Le Eff = 1.100 m          W2  = 1.26
Fy = 350 MPa              Fu  = 450 MPa
Tension area factor (Ane/Ag)  = 1.00
Flange class: 3           Web class   : 3

Critical Load Case : C2

Section C152x76      Channels (Web vert)

SANS 10162 - 2005 13.8.3 :
a) Cross-sectional strength (Crit. pos.= 0.000 m)
   Cu    Mux   Muy   7.03   .770   .010
   -- + --- + --- = ---- + ---- + ---- = 0.03               OK
   Cr    Mrx   Mry   718    35.3   6.62

b) Overall member strength
   Cu   U1xMux   U1yMuy   7.06   .770   .010
   -- + ------ + ------ = ---- + ---- + ---- = 0.04         OK
   Cr    Mrx      Mry     552    35.3   6.62

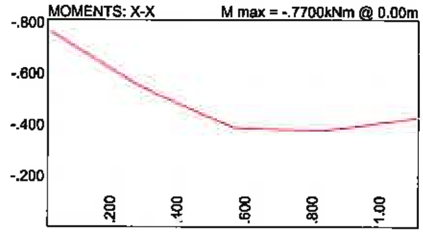
c) Lateral torsional buckling strength
   Cu   U1xMux   U1yMuy   7.06   .770   .010
   -- + ------ + ------ = ---- + ---- + ---- = 0.04         OK
   Cr    Mrx      Mry     582    35.3   6.62

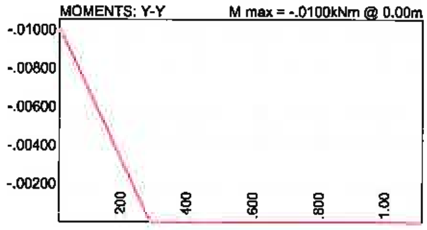
13.4:Shear

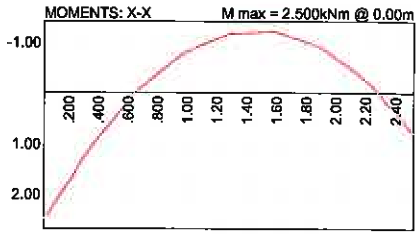
   Vux<Vrx       0.8 < 202.8                                OK
   Vuy<Vry       0.0 < 237.6                                OK

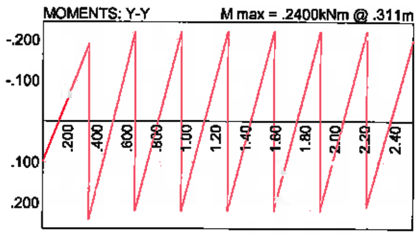
Slenderness Ratio: L/r =  49                                OK

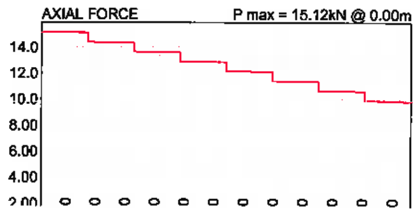
------------------------------------------------------------
```












```
Combine Ver W2.6.11
Element 10-18                    Evaluate current section
Section name 1

Lx Eff = 2.488 m           W1x = 1.00
Ly Eff = 2.488 m           W1y = 1.00
Le Eff = 2.488 m           W2  = 1.45
Fy = 350 MPa               Fu  = 450 MPa
Tension area factor (Ane/Ag)   = 1.00
Flange class: 3            Web class   = 3

Critical Load Case : C2

Section C152x76      Channels (Web vert)

SANS 10162 - 2005 13.8.3 :
a) Cross-sectional strength (Crit. pos.= 0.000 m)
   Cu   Mux   Muy   15.1   2.51   .104
   -- + --- + --- = ---- + ---- + ---- = 0.11                   OK
   Cr   Mrx   Mry   718    35.3   6.62

b) Overall member strength
   Cu   U1xMux   U1yMuy   15.1   2.51   .250
   -- + ------ + ------ = ---- + ---- + ---- = 0.17             OK
   Cr    Mrx      Mry     261    35.3   6.62

c) Lateral torsional buckling strength
   Cu   U1xMux   U1yMuy   15.1   2.51   .250
   -- + ------ + ------ = ---- + ---- + ---- = 0.17             OK
   Cr    Mrx      Mry     261    33.7   6.62

13.4:Shear

   Vux<Vrx       4.7 < 202.8                                    OK
   Vuy<Vry       1.5 < 237.6                                    OK

Slenderness Ratio: L/r = 112                                    OK

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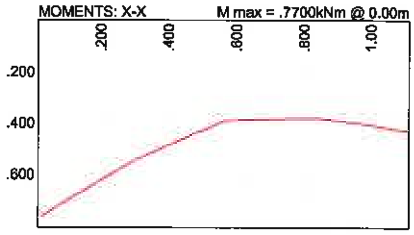

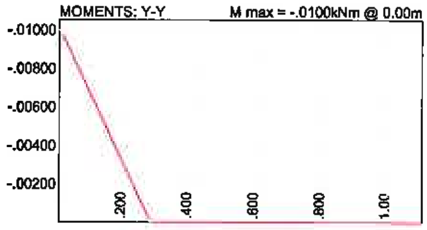

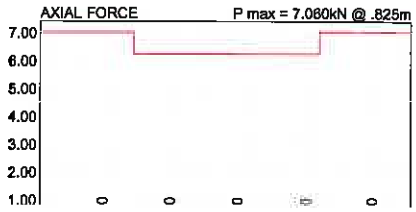


```
Combine Ver W2.6.11
Element 18-26                     Evaluate current section
Section name 1

Lx Eff = 1.100 m          W1x = 1.00
Ly Eff = 1.100 m          W1y = 1.00
Le Eff = 1.100 m          W2  = 1.26
Fy = 350 MPa              Fu  = 450 MPa
Tension area factor (Ane/Ag)  = 1.00
Flange class: 3           Web class   : 3

Critical Load Case : C2

Section C152x76      Channels (Web vert)

SANS 10162 - 2005 13.8.3 :
a) Cross-sectional strength (Crit. pos.= 0.000 m)
   Cu    Mux   Muy    7.03   .770   .010
   -- + --- + --- = ---- + ---- + ---- = 0.03                 OK
   Cr    Mrx   Mry    718    35.3   6.62

b) Overall member strength
   Cu    U1xMux   U1yMuy   7.06   .770   .010
   -- + ------ + ------ = ---- + ---- + ---- = 0.04           OK
   Cr     Mrx       Mry     552    35.3   6.62

c) Lateral torsional buckling strength
   Cu    U1xMux   U1yMuy   7.06   .770   .010
   -- + ------ + ------ = ---- + ---- + ---- = 0.04           OK
   Cr     Mrx       Mry     582    35.3   6.62

13.4:Shear

   Vux<Vrx       0.8 < 202.8                                  OK
   Vuy<Vry       0.0 < 237.6                                  OK


Slenderness Ratio: L/r =  49                                  OK

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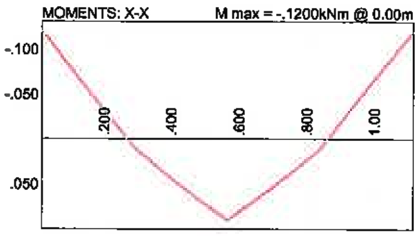
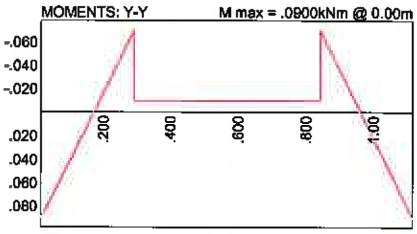
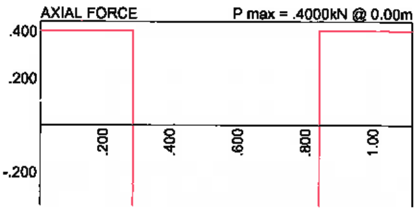
```








```
Combine Ver W2.6.11
Element 25-26                    Evaluate current section
Section name 1

Lx Eff = 1.100 m         W1x = 1.00
Ly Eff = 1.100 m         W1y = 1.00
Le Eff = 1.100 m         W2  = 1.00
Fy = 350 MPa             Fu  = 450 MPa
Tension area factor (Ane/Ag)  = 1.00
Flange class: 3          Web class   : 3

Critical Load Case : C2

Section C152x76      Channels (Web vert)

SANS 10162 - 2005 13.8.3 :
a) Cross-sectional strength (Crit. pos.= 0.000 m)
   Cu   Mux   Muy   .400   .120   .090
   -- + --- + --- = ---- + ---- + ---- = 0.02                OK
   Cr   Mrx   Mry   718    35.3   6.62

b) Overall member strength
   Cu   U1xMux   U1yMuy   .400   .120   .090
   -- + ------ + ------ = ---- + ---- + ---- = 0.02          OK
   Cr    Mrx      Mry     552    35.3   6.62

c) Lateral torsional buckling strength
   Cu   U1xMux   U1yMuy   .400   .120   .090
   -- + ------ + ------ = ---- + ---- + ---- = 0.02          OK
   Cr    Mrx      Mry     582    35.3   6.62

13.4:Shear

   Vux<Vrx      0.5 < 202.8                                  OK
   Vuy<Vry      0.6 < 237.6                                  OK


Slenderness Ratio: L/r =  49                                 OK

----------------------------------------------------------------
```

Design Code: SANS 10162 - 2005

Element : 1-9

Section: C152x76 C

Lx eff = 2488
Ly eff = 2488
Lz eff = 2488
Le eff = 2488

Section Slenderness: Table 4

Flanges

$$B_{ratio} = \frac{B}{T_f} \cdot \sqrt{f_y}$$

$$= \frac{76.20}{9.00} \times \sqrt{350}$$

$$= 158.397$$

$B_{ratio} < 200$ => Class 3 Flange

Web

$$W_{ratio\_1} = \frac{1100}{\sqrt{f_y}} \cdot \left[1 - \frac{0.39 \cdot C_u}{\phi \cdot C_y}\right]$$

$$= \frac{1100}{\sqrt{350}} \times \left[1 - \frac{0.39 \times 15.1}{0.9 \times 798.0}\right]$$

$$= 58.315$$

Hw/Tw(16.6) < Wratio_3 => Class 3 Web

=> Section class = 3

Maximum slenderness ratio 10.4.2.1

$$X_{\_ratio} = \frac{K_x \cdot L_x}{r_x}$$

$$= \frac{1.00 \times 2\,488.00}{61}$$

$$= 40.787$$

$X_{\_ratio} < 200$ => OK

$$Y_{\_ratio} = \frac{K_y \cdot L_y}{r_y}$$

$$= \frac{1.00 \times 2\,488.00}{22}$$

$$= 113.091$$

$Y_{\_ratio} < 200$ => OK

Axial compression and bending 13.8

Member strength and stability — All classes of sections except class 1 and class 2 I-shaped sections 13.8.3

Cross-sectional strength 13.8.3 a

$$C_r = \frac{\phi \cdot A \cdot f_y}{1000}$$
$$= \frac{0.9 \times 2\,280.00 \times 350.00}{1000}$$
$$= 718.200 \text{ kN}$$

$$M_{rx} = \frac{\phi \cdot Z_{ex} \cdot f_y}{1 \times 10^6}$$
$$= \frac{0.9 \times 112\,000.00 \times 350.00}{1 \times 10^6}$$
$$= 35.280 \text{ kNm}$$

$$M_{ry} = \frac{\phi \cdot Z_{ey} \cdot f_y}{1 \times 10^6}$$
$$= \frac{0.9 \times 21\,000.00 \times 350.00}{1 \times 10^6}$$
$$= 6.615 \text{ kNm}$$

End moment factors: 13.8.5

$$\kappa_x = \frac{M_{ux\_min}}{M_{ux\_max}}$$
$$= \frac{-0.79}{2.50}$$
$$= -0.3160$$

$$\kappa_y = \frac{M_{uy\_min}}{M_{uy\_max}}$$
$$= \frac{-0.10}{-0.20}$$
$$= 0.5000$$

$$\omega_x = 0.6 - 0.4 \cdot \kappa_x$$
$$= 0.6 - 0.4 \times -0.32$$
$$= 0.7280$$

$\omega_x = 1$ (loads between supports)

$$\omega_y = 0.6 - 0.4 \cdot \kappa_y$$
$$= 0.6 - 0.4 \times 0.50$$
$$= 0.4000$$

$\omega_y = \max(\omega_y, 0.4) = 0.4$
$\omega_y = 1$ (loads between supports)

$$C_{ex} = \frac{\pi^2 \cdot E \cdot I_x}{1000 \cdot (K_x \cdot L_x)^2}$$
$$= \frac{\pi^2 \times 200\ 000.00 \times 8\ 520\ 000.00}{1000 \times (1.00 \times 2\ 488.00)^2}$$
$$= 2\ 716.868 \text{ kN}$$

$$C_{ey} = \frac{\pi^2 \cdot E \cdot I_y}{1000 \cdot (K_y \cdot L_y)^2}$$
$$= \frac{\pi^2 \times 200\ 000.00 \times 1\ 140\ 000.00}{1000 \times (1.00 \times 2\ 488.00)^2}$$
$$= 363.525 \text{ kN}$$

$$U_{1x} = \frac{\omega_x}{1 - \frac{C_u}{C_{ex}}}$$
$$= \frac{1.000}{1 - \frac{15.12}{2\ 716.87}}$$
$$= 1.006$$

$$U_{1y} = \frac{\omega_y}{1 - \frac{C_u}{C_{ey}}}$$
$$= \frac{1.000}{1 - \frac{15.12}{363.52}}$$
$$= 1.043$$

$$F_{13.8.3(a)} = \frac{C_u}{C_r} + \frac{U_{1x} \cdot M_{ux}}{M_{rx}} + \frac{U_{1y} \cdot M_{uy}}{M_{ry}}$$
$$= \frac{15.1}{718.2} + \frac{1.006 \times 2.50}{35.28} + \frac{1.043 \times 0.10}{6.62}$$
$$= 0.1081$$

OK

Overall member strength 13.8.3 b

Calculate Cr (Note: K=1 will only be used if K>0.7 and K<1, because K<0.7 implies intermediate lateral supports and does not refer to restraint conditions)

$$f_{ex} = \frac{\pi^2 \cdot E}{\left[\frac{K_x \cdot L_x}{r_x}\right]^2}$$

$$= \frac{\pi^2 \times 200000}{\left[\frac{1 \times 2488}{61.1}\right]^2}$$

$$= 1\,190.451 \text{ MPa}$$

$$f_{ey} = \frac{\pi^2 \cdot E}{\left[\frac{K_y \cdot L_y}{r_y}\right]^2}$$

$$= \frac{\pi^2 \times 200000}{\left[\frac{1 \times 2488}{22.3}\right]^2}$$

$$= 158.576 \text{ MPa}$$

$$f_{ez} = \frac{\frac{\pi^2 \cdot E \cdot C_w}{(K_z \cdot L_z)^2} + G \cdot J}{A \cdot r_o^2}$$

$$= \frac{\frac{\pi^2 \times 200000 \times 3620 \times 10^6}{(1 \times 2488)^2} + 77000 \times 56900}{2280 \times 80.718^2}$$

$$= 372.643 \text{ MPa}$$

$$r_o = \sqrt{x_o^2 + y_o^2 + r_x^2 + r_y^2}$$

$$= \sqrt{-47.8^2 + 0^2 + 61.1^2 + 22.3^2}$$

$$= 80.718 \text{ mm}$$

$$\Omega = 1 - \frac{x_o^2 + y_o^2}{r_o^2}$$

$$= 1 - \frac{-47.8^2 + 0^2}{80.718^2}$$

$$= 0.6493$$

Note: fex and fey swapped in following 2 eq. because y axis taken as axis of symmetry

13.3.2(b)

$$f_{eyz} = \frac{f_{ex} + f_{ez}}{2 \cdot \Omega} \cdot \left[1 - \sqrt{1 - \frac{4 \cdot f_{ex} \cdot f_{ez} \cdot \Omega}{(f_{ex} + f_{ez})^2}}\right]$$

$$= \frac{1\,190.5 + 372.65}{2 \times .64931} \times \left[1 - \sqrt{1 - \frac{4 \times 1\,190.5 \times 372.65 \times .64931}{(1\,190.5 + 372.65)^2}}\right]$$

$$= 328.688 \text{ MPa}$$

$$f_e = \min(f_{ey}, f_{eyz}) = 158.6 \text{ MPa}$$

$$\lambda = \sqrt{\frac{f_y}{f_e}}$$

$$= \sqrt{\frac{350}{158.58}}$$

$$= 1.486$$

$$C_r = \frac{\phi \cdot A \cdot f_y \cdot \left[1 + \lambda^{2 \cdot n}\right]^{-\frac{1}{n}}}{1 \times 10^3}$$

$$= \frac{0.9 \times 2\,280.0 \times 350 \times \left[1 + 1.4856^{2 \times 1.34}\right]^{-\frac{1}{1.34}}}{1 \times 10^3}$$

$$= 260.673 \text{ kN}$$

$$U_{1x} = 1.006$$

$$U_{1y} = 1.043$$

$$F_{13.8.3(b)} = \frac{C_u}{C_r} + \frac{U_{1x} \cdot M_{ux}}{M_{rx}} + \frac{U_{1y} \cdot M_{uy}}{M_{ry}}$$

$$= \frac{15.1}{260.7} + \frac{1.006 \times 2.50}{35.28} + \frac{1.043 \times 0.24}{6.62}$$

$$= 0.1670$$

OK

Lateral torsional buckling strength 13.8.3 c

$$M_p = \frac{f_y \cdot Z_{plx}}{1 \times 10^6}$$

$$= \frac{350.000 \times 130\,000.000}{1 \times 10^6}$$

$$= 45.500 \text{ kNm}$$

$$M_y = \frac{f_y \cdot Z_{ex}}{1\times10^6}$$

$$= \frac{350.000\times112\,000.000}{1\times10^6}$$

$$= 39.200 \text{ kNm}$$

$$\omega_2 = 1.75 + 1.05\cdot\kappa_x + 0.3\cdot\kappa_x^2$$

$$= 1.75 + 1.05\times\text{-}0.316 + 0.3\times\text{-}0.316^2$$

$$= 1.448$$

$$M_{cr} = \frac{\frac{\omega_2\cdot\pi}{K_e\cdot L}\cdot\sqrt{E\cdot I_y\cdot G\cdot J + \left[\frac{\pi\cdot E}{K_e\cdot L}\right]^2\cdot I_y\cdot C_w}}{1000}$$

$$= \frac{\frac{1.448\times\pi}{1.000\times2488}\times\sqrt{200\times1140000\times77\times56900 + \left[\frac{\pi\times200}{1.000\times2488}\right]^2\times1140000\times3620000000}}{1000}$$

$$= 64.956 \text{ kNm}$$

Channel section =>
$M_{cr} > 0.67M_y$ =>

Effect of thin webs on moment resistance | 14.3.4

$$SlendernessRatioLimit = \frac{1900\cdot\left[1 - \frac{0.65\cdot C_u}{\phi\cdot C_y}\right]}{\sqrt{\frac{M_u}{\phi\cdot Z_e}}}$$

$$= \frac{1900\times\left[1 - \frac{0.65\times15120}{0.9\times798000}\right]}{\sqrt{\frac{2500000}{0.9\times112000}}}$$

$$= 376.296$$

Hw/Tw < SlendernessRatioLimit => no reduction in Mr

$$M_r = 1.15\cdot\phi\cdot M_y\left[1 - \frac{0.28\cdot M_y}{M_{cr}}\right]$$

$$= 1.15\times0.9\times39.200\times\left[1 - \frac{0.28\times39.200}{64.963}\right]$$

$$= 33.717 \text{ kNm}$$

Cr based on weak-axis bending

$$A_{vy} = \frac{2 \cdot B \cdot t_f \cdot 5}{6}$$

$$= \frac{2 \times 76.2 \times 9 \times 5}{6}$$

$$= 1\ 143.000\ \text{mm}^2$$

$$V_{ry} = \frac{0.90 \cdot A_{vy} \cdot f_{sv}}{1000}$$

$$= \frac{0.90 \times 1143 \times 231}{1000}$$

$$= 237.630\ \text{kN}$$

$V_{uy} = 1.5\text{kN}$ OK

Design Code: SANS 10162 - 2005

Element : 10-18

Section: C152x76 C

Lx eff = 2488
Ly eff = 2488
Lz eff = 2488
Le eff = 2488

Section Slenderness: Table 4

Flanges

$$B_{ratio} = \frac{B}{Tf} \cdot \sqrt{f_y}$$

$$= \frac{76.20}{9.00} \times \sqrt{350}$$

$$= 158.397$$

$B_{ratio} < 200$ => Class 3 Flange

Web

$$W_{ratio\_1} = \frac{1100}{\sqrt{f_y}} \cdot \left[1 - \frac{0.39 \cdot C_u}{\phi \cdot C_y}\right]$$

$$= \frac{1100}{\sqrt{350}} \times \left[1 - \frac{0.39 \times 15.1}{0.9 \times 798.0}\right]$$

$$= 58.315$$

Hw/Tw(16.6) < Wratio_3 => Class 3 Web

=> Section class = 3

Maximum slenderness ratio 10.4.2.1

$$X\_ratio = \frac{K_x \cdot L_x}{r_x}$$

$$= \frac{1.00 \times 2\,488.00}{61}$$

$$= 40.787$$

$X\_ratio < 200$ => OK

$$Y\_ratio = \frac{K_y \cdot L_y}{r_y}$$

$$= \frac{1.00 \times 2\,488.00}{22}$$

$$= 113.091$$

$Y\_ratio < 200$ => OK

Axial compression and bending 13.8

Member strength and stability — All classes of sections except class 1 and class 2 I-shaped sections 13.8.3

Cross-sectional strength 13.8.3 a

$$C_r = \frac{\phi \cdot A \cdot f_y}{1000}$$

$$= \frac{0.9 \times 2\,280.00 \times 350.00}{1000}$$

$$= 718.200 \text{ kN}$$

$$M_{rx} = \frac{\phi \cdot Z_{ex} \cdot f_y}{1 \times 10^6}$$

$$= \frac{0.9 \times 112\,000.00 \times 350.00}{1 \times 10^6}$$

$$= 35.280 \text{ kNm}$$

$$M_{ry} = \frac{\phi \cdot Z_{ey} \cdot f_y}{1 \times 10^6}$$

$$= \frac{0.9 \times 21\,000.00 \times 350.00}{1 \times 10^6}$$

$$= 6.615 \text{ kNm}$$

End moment factors: 13.8.5

$$\kappa_x = \frac{M_{ux\_min}}{M_{ux\_max}}$$

$$= \frac{0.79}{-2.50}$$

$$= -0.3160$$

$$\kappa_y = \frac{M_{uy\_min}}{M_{uy\_max}}$$

$$= \frac{-0.10}{-0.20}$$

$$= 0.5000$$

$$\omega_x = 0.6 - 0.4 \cdot \kappa_x$$

$$= 0.6 - 0.4 \times -0.32$$

$$= 0.7280$$

$\omega_x = 1$ (loads between supports)

$$\omega_y = 0.6 - 0.4 \cdot \kappa_y$$
$$= 0.6 - 0.4 \times 0.50$$
$$= 0.4000$$

$\omega_y = \max(\omega_y, 0.4) = 0.4$
$\omega_y = 1$ (loads between supports)

$$C_{ex} = \frac{\pi^2 \cdot E \cdot I_x}{1000 \cdot (K_x \cdot L_x)^2}$$
$$= \frac{\pi^2 \times 200\,000.00 \times 8\,520\,000.00}{1000 \times (1.00 \times 2\,488.00)^2}$$
$$= 2\,716.868 \text{ kN}$$

$$C_{ey} = \frac{\pi^2 \cdot E \cdot I_y}{1000 \cdot (K_y \cdot L_y)^2}$$
$$= \frac{\pi^2 \times 200\,000.00 \times 1\,140\,000.00}{1000 \times (1.00 \times 2\,488.00)^2}$$
$$= 363.525 \text{ kN}$$

$$U_{1x} = \frac{\omega_x}{1 - \frac{C_u}{C_{ex}}}$$
$$= \frac{1.000}{1 - \frac{15.12}{2\,716.87}}$$
$$= 1.006$$

$$U_{1y} = \frac{\omega_y}{1 - \frac{C_u}{C_{ey}}}$$
$$= \frac{1.000}{1 - \frac{15.12}{363.52}}$$
$$= 1.043$$

$$F_{13.8.3(a)} = \frac{C_u}{C_r} + \frac{U_{1x} \cdot M_{ux}}{M_{rx}} + \frac{U_{1y} \cdot M_{uy}}{M_{ry}}$$
$$= \frac{15.1}{718.2} + \frac{1.006 \times 2.50}{35.28} + \frac{1.043 \times 0.10}{6.62}$$
$$= 0.1081$$

OK

Overall member strength 13.8.3 b

Calculate Cr (Note: K=1 will only be used if K>0.7 and K<1, because K<0.7 implies intermediate lateral supports and does not refer to restraint conditions)

$$f_{ex} = \frac{\pi^2 \cdot E}{\left[\frac{K_x \cdot L_x}{r_x}\right]^2}$$

$$= \frac{\pi^2 \times 200000}{\left[\frac{1 \times 2488}{61.1}\right]^2}$$

$$= 1\,190.451 \text{ MPa}$$

$$f_{ey} = \frac{\pi^2 \cdot E}{\left[\frac{K_y \cdot L_y}{r_y}\right]^2}$$

$$= \frac{\pi^2 \times 200000}{\left[\frac{1 \times 2488}{22.3}\right]^2}$$

$$= 158.576 \text{ MPa}$$

$$f_{ez} = \frac{\frac{\pi^2 \cdot E \cdot C_w}{(K_z \cdot L_z)^2} + G \cdot J}{A \cdot r_o^2}$$

$$= \frac{\frac{\pi^2 \times 200000 \times 3620 \times 10^6}{(1 \times 2488)^2} + 77000 \times 56900}{2280 \times 80.718^2}$$

$$= 372.643 \text{ MPa}$$

$$r_o = \sqrt{x_o^2 + y_o^2 + r_x^2 + r_y^2}$$

$$= \sqrt{-47.8^2 + 0^2 + 61.1^2 + 22.3^2}$$

$$= 80.718 \text{ mm}$$

$$\Omega = 1 - \frac{x_o^2 + y_o^2}{r_o^2}$$

$$= 1 - \frac{-47.8^2 + 0^2}{80.718^2}$$

$$= 0.6493$$

Note: fex and fey swapped in following 2 eq. because y axis taken as axis of symmetry

13.3.2(b)

$$f_{eyz} = \frac{f_{ex} + f_{ez}}{2 \cdot \Omega} \cdot \left[1 - \sqrt{1 - \frac{4 \cdot f_{ex} \cdot f_{ez} \cdot \Omega}{(f_{ex} + f_{ez})^2}}\right]$$

$$= \frac{1\,190.5 + 372.65}{2 \times .64931} \times \left[1 - \sqrt{1 - \frac{4 \times 1\,190.5 \times 372.65 \times .64931}{(1\,190.5 + 372.65)^2}}\right]$$

$$= 328.688 \text{ MPa}$$

$f_e = \min(f_{ey}, f_{eyz}) = 158.6$ MPa

$$\lambda = \sqrt{\frac{f_y}{f_e}}$$

$$= \sqrt{\frac{350}{158.58}}$$

$$= 1.486$$

$$C_r = \frac{\phi \cdot A \cdot f_y \cdot \left[1 + \lambda^{2 \cdot n}\right]^{-\frac{1}{n}}}{1 \times 10^3}$$

$$= \frac{0.9 \times 2\,280.0 \times 350 \times \left[1 + 1.4856^{2 \times 1.34}\right]^{-\frac{1}{1.34}}}{1 \times 10^3}$$

$$= 260.673 \text{ kN}$$

$U_{1x} = 1.006$

$U_{1y} = 1.043$

$$F_{13.8.3(b)} = \frac{C_u}{C_r} + \frac{U_{1x} \cdot M_{ux}}{M_{rx}} + \frac{U_{1y} \cdot M_{uy}}{M_{ry}}$$

$$= \frac{15.1}{260.7} + \frac{1.006 \times 2.50}{35.28} + \frac{1.043 \times 0.24}{6.62}$$

$$= 0.1670$$

OK 13.8.3 c

Lateral torsional buckling strength

$$M_p = \frac{f_y \cdot Z_{plx}}{1 \times 10^6}$$

$$= \frac{350.000 \times 130\,000.000}{1 \times 10^6}$$

$$= 45.500 \text{ kNm}$$

$$M_y = \frac{f_y \cdot Z_{ex}}{1\times10^6}$$

$$= \frac{350.000\times112\,000.000}{1\times10^6}$$

$$= 39.200 \text{ kNm}$$

$$\omega_2 = 1.75 + 1.05\cdot \kappa_x + 0.3\cdot \kappa_x^2$$

$$= 1.75 + 1.05\times\text{-}0.316 + 0.3\times\text{-}0.316^2$$

$$= 1.448$$

$$M_{cr} = \frac{\dfrac{\omega_2\cdot \pi}{K_e\cdot L}\cdot\sqrt{E\cdot I_y\cdot G\cdot J + \left[\dfrac{\pi\cdot E}{K_e\cdot L}\right]^2\cdot I_y\cdot C_w}}{1000}$$

$$= \frac{\dfrac{1.448\times\pi}{1.000\times2488}\times\sqrt{200\times1140000\times77\times56900 + \left[\dfrac{\pi\times200}{1.000\times2488}\right]^2\times1140000\times3620000000}}{1000}$$

$$= 64.956 \text{ kNm}$$

Channel section =>
$M_{cr} > 0.67M_y$ =>

Effect of thin webs on moment resistance 14.3.4

$$SlendernessRatioLimit = \frac{1900\cdot\left[1 - \dfrac{0.65\cdot C_u}{\phi\cdot C_y}\right]}{\sqrt{\dfrac{M_u}{\phi\cdot Z_e}}}$$

$$= \frac{1900\times\left[1 - \dfrac{0.65\times15120}{0.9\times798000}\right]}{\sqrt{\dfrac{2500000}{0.9\times112000}}}$$

$$= 376.296$$

Hw/Tw < SlendernessRatioLimit => no reduction in Mr

$$M_r = 1.15\cdot \phi\cdot M_y\cdot\left[1 - \frac{0.28\cdot M_y}{M_{cr}}\right]$$

$$= 1.15\times0.9\times39.200\times\left[1 - \frac{0.28\times39.200}{64.963}\right]$$

$$= 33.717 \text{ kNm}$$

Cr based on weak-axis bending

$$\lambda_y = \frac{K_y \cdot L_y}{r_y} \cdot \sqrt{\frac{f_y}{\pi^2 \cdot E}}$$

$$= \frac{1.00 \times 2\,488.00}{22.30} \times \sqrt{\frac{350}{\pi^2 \times 200000}}$$

$$= 1.486$$

$$C_r = \frac{\phi \cdot A \cdot f_y \cdot \left[1 + \lambda^{2 \cdot n}\right]^{-\frac{1}{n}}}{1 \times 10^3}$$

$$= \frac{0.9 \times 2\,280.0 \times 350 \times \left[1 + 1.486^{2 \times 1.34}\right]^{-\frac{1}{1.34}}}{1 \times 10^3}$$

$$= 260.569 \text{ kN}$$

$$F_{13.8.3(c)} = \frac{C_u}{C_r} + \frac{U_{1x} \cdot M_{ux}}{M_{rx}} + \frac{U_{1y} \cdot M_{uy}}{M_{ry}}$$

$$= \frac{15.1}{260.7} + \frac{1.006 \times 2.50}{33.72} + \frac{1.043 \times 0.24}{6.62}$$

$$= 0.1703$$

OK

Shear 13.4

Shear buckling coefficient 13.3.4.1.1 a)

k_v=5.34

$$h_{wtw} = \frac{h_w}{t_w}$$

$$= \frac{106}{6.4}$$

$$= 16.562$$

Shear buckling factor 13.4.1.1 a)

$$S_{fac} = \sqrt{\frac{k_v}{f_y}}$$

$$= \sqrt{\frac{5.34}{350}}$$

$$= 0.1235$$

Inelastic critical plate-buckling stress in shear 13.4.1.1 b)

$$f_{cri} = \frac{290 \cdot \sqrt{f_y \cdot k_v}}{\frac{h_w}{t_w}}$$

$$= \frac{290 \times \sqrt{350 \times 5.34}}{\frac{106}{6.4}}$$

$$= 756.967 \text{ MPa}$$

Elastic critical plate-buckling stress in shear 13.4.1.1 d)

$$f_{cre} = \frac{180000 \cdot k_v}{\left[\frac{h_w}{t_w}\right]^2}$$

$$= \frac{180000 \times 5.34}{\left[\frac{106}{6.4}\right]^2}$$

$$= 3\,503.983 \text{ MPa}$$

hw/tw <= 440*sqrt(kv/fy) => 13.4.1.1 a)

$$f_{sx} = 0.66 \cdot f_y$$

$$= 0.66 \times 350$$

$$= 231.000 \text{ MPa}$$

Factored shear resistance X-axis: 13.4.1.1

$$A_{vx} = H \cdot t_w$$

$$= 152.4 \times 6.4$$

$$= 975.360 \text{ mm}^2$$

$$V_{rx} = \frac{0.9 \cdot A_{vx} \cdot f_{sx}}{1000}$$

$$= \frac{0.9 \times 975.36 \times 231}{1000}$$

$$= 202.777 \text{ kN}$$

$V_{ux} = 4.7\text{kN}$ OK

Factored shear resistance Y-axis: 13.4.1.1

$$f_{sy} = 0.66 \cdot f_y$$

$$= 0.66 \times 350$$

$$= 231.000 \text{ MPa}$$

$$A_{vy} = \frac{2 \cdot B \cdot tf \cdot 5}{6}$$

$$= \frac{2 \times 76.2 \times 9 \times 5}{6}$$

$$= 1\,143.000 \text{ mm}^2$$

$$V_{ry} = \frac{0.90 \cdot A_{vy} \cdot f_{sy}}{1000}$$

$$= \frac{0.90 \times 1143 \times 231}{1000}$$

$$= 237.630 \text{ kN}$$

$V_{uy} = 1.5\text{kN}$ OK

3.3 Connections

Project: Medupi Buildings | Project No: 14133
Component: Weigh bridge control room, Stair - slab connection | Sheet No:
Compiled By: DJJ vd M | Date: 01-05-09 | Checked By: | Date:

Ref SABS 0160	Calculations
Table 4	Loads: Live load on stairs : 1,5 kN/m^2
	Area of stairs : 1,1 × 3,2 = 3,52
	Load = 1,5 · 3,52 = 5,28 kN
Red book table 2.12	Dead load : 152×76 TFC : 17,9 kg/m
	$17{,}9 \cdot (1{,}1 + \sqrt{1{,}360^2 + 1{,}835^2}) = 60{,}57$
	= 0,59 kN = 1,181
	Vastrap plate : Density of steel : 77 kN/m^3
	Mass of treads : 77 · 0,006 · 0,43 · 1,1
	= 0,219 kN · 7 = 1,53
	+ 77 · 0,006 · 1,1 · 1,1
	= 1,53 + 0,56
	= 2,09 kN
	Load on bolts = 5,28 · 1,6 + (2,09 + 1,18) · 1,2
	= ~~11,7 kN~~ 12,37
	∴ = ~~3,9~~ 4,12 kN / bolt
	Shear resistance of M12 bolt (Class 4,8)
Red book table 7.2	V_r = 16 kN for single shear plane
	∴ $V_r > V$ ∴ M12's will work.

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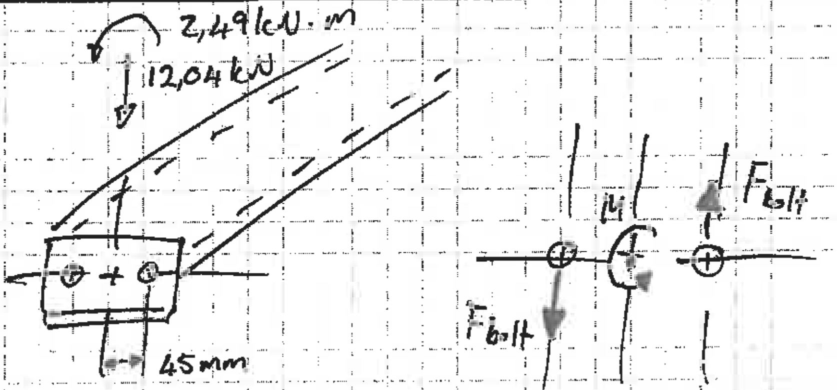
Project:	Medupi Buildings	Project No:	14133
Component:	Weigh bridge control room Stair - stub connect.	Sheet No:	
Compiled By:	DJT vd M Date: 04-06-09	Checked By:	Date:

Calculations

SANS 10162

Prokon: Stair



Shear force on bolts:

$$V = \frac{12,04}{2} = 6,02 \text{ kN / bolts}$$

Force on bolt due to moment:

$$V = \frac{2,49}{0,045} = 55,33 \text{ kN}$$

$\therefore$ 27,67 kN/bolt

Total shear force: $V = 6,02 + 27,67$
$= 33,69$ kN / bolt

Redbook table 7.2

$\therefore$ use Class 8.8 M16

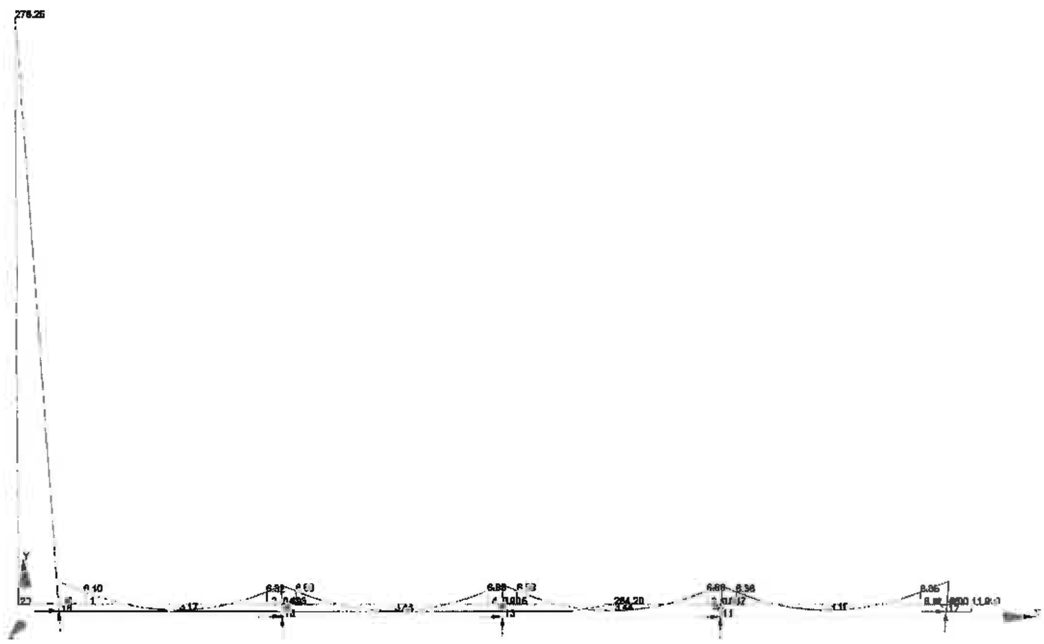
WEIGH BRIDGE

PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title		
	Client		
	Calcs by	Checked by	Date

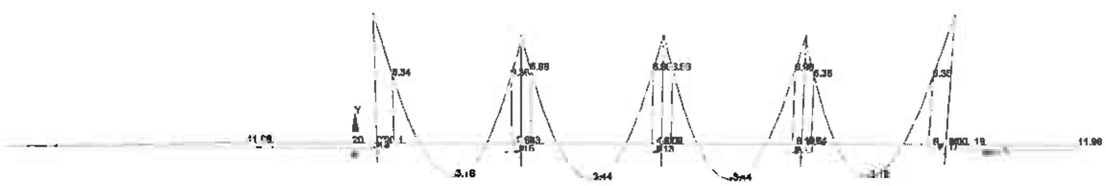
Space Frame Analysis Ver W2.4.52 - 25 May 2009
Input file:I:\14133_Medupi PS and WTP\28 Weigh bridge\Design\Bases\2D Model.A03
Created : 12/06/2009 11:11:49 AM

A03

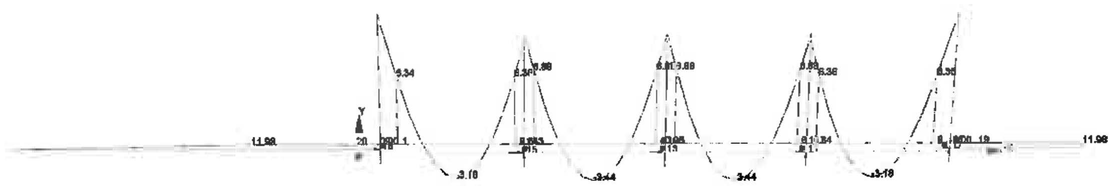
X-Moments for Load Combination C1



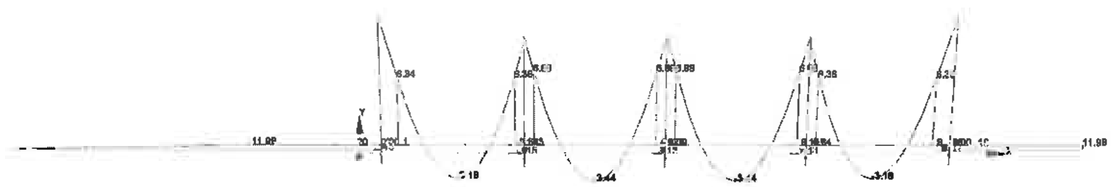
X-Moments for Load Combination C2



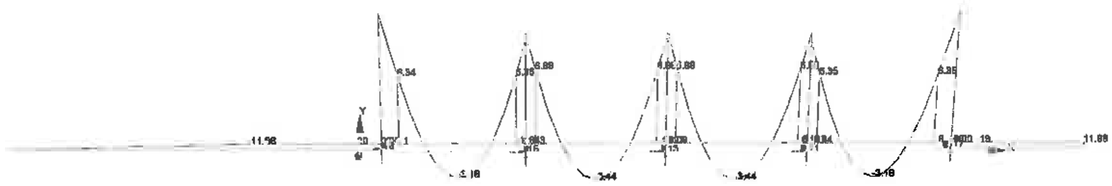
X-Moments for Load Combination C3



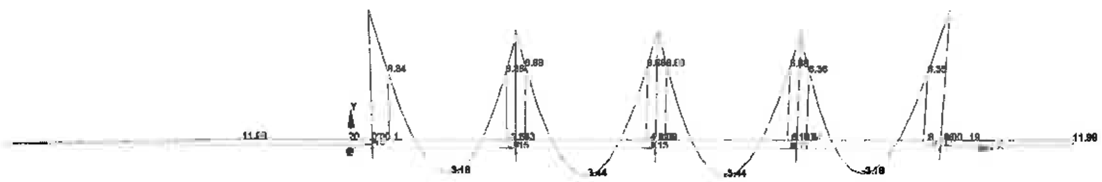
X-Moments for Load Combination C4



X-Moments for Load Combination C5



X-Moments for Load Combination C6



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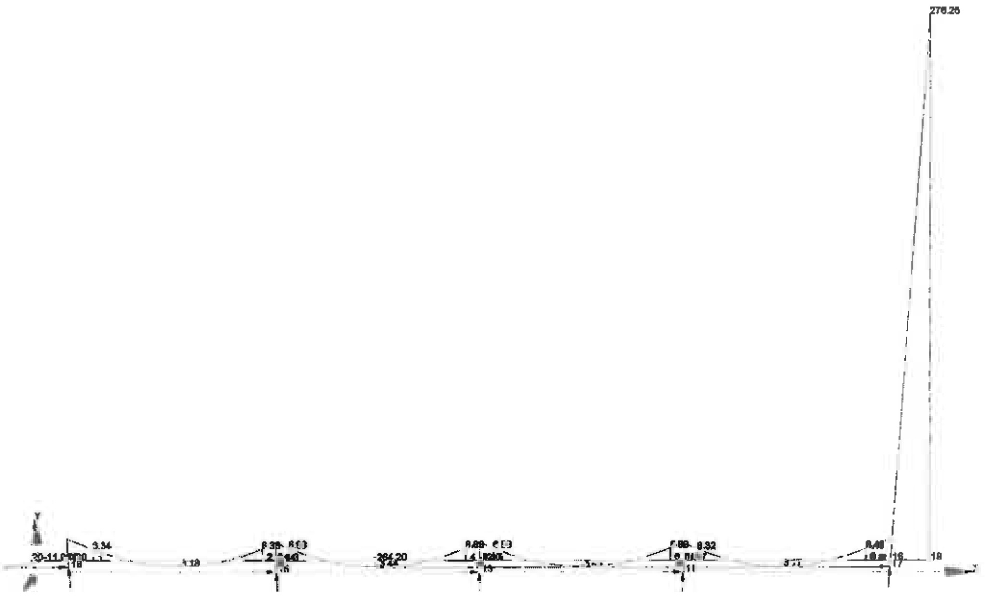
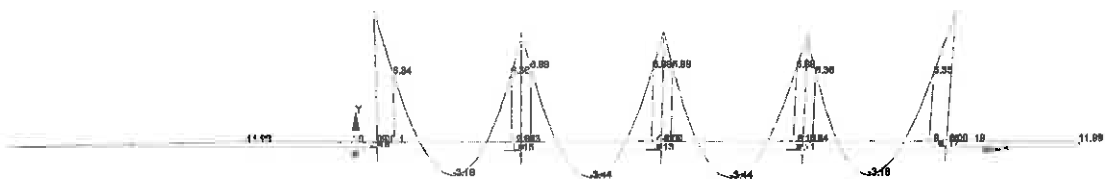
X-Moments for Load Combination C7

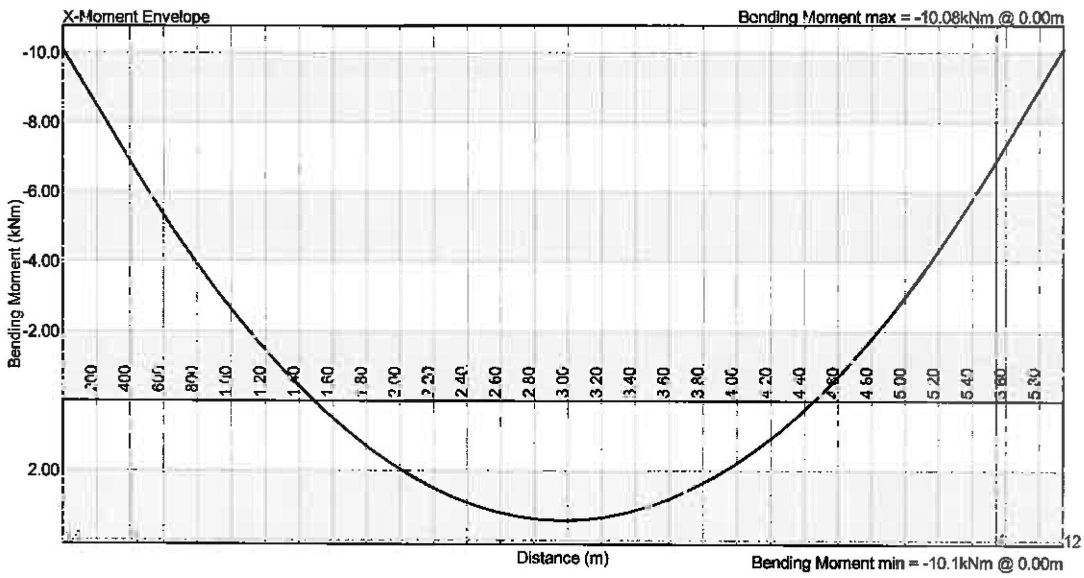


X-Moments for Load Combination C1

X-Moments for Load Combination C2



Envelope for Load Cases:
C1, C2, C3, C4, C5, C6, C7,



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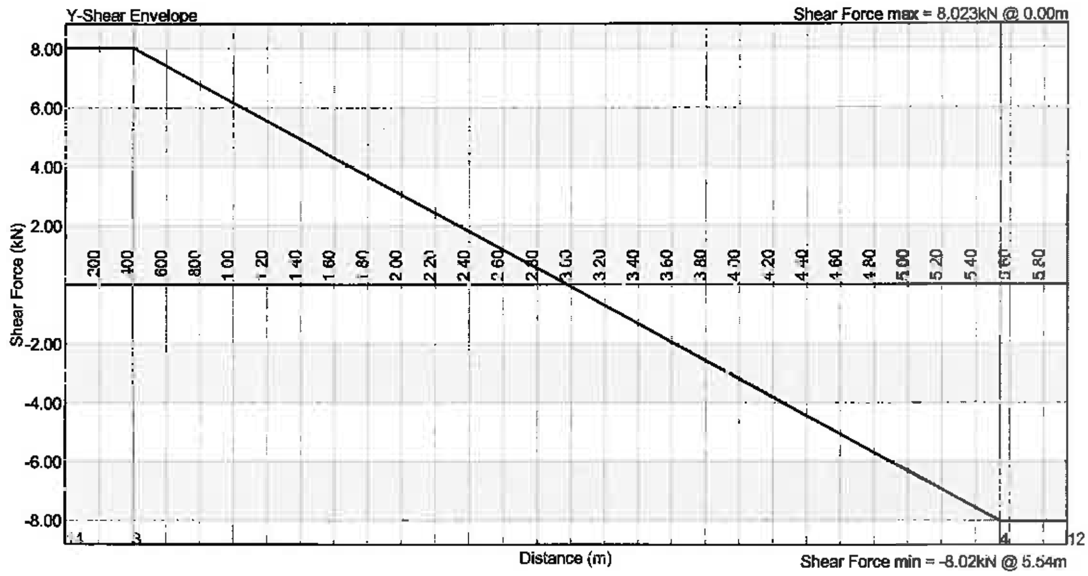
	Job Title		
	Client		
	Calcs by	Checked by	Date

Envelope for Load Cases:
C1, C2, C3, C4, C5, C6, C7,



PROKON	Job Number		Sheet
Software Consultants (Pty) Ltd	Job Title		
Internet: http://www.prokon.com	Client		
E-Mail : mail@prokon.com	Calcs by	Checked by	Date

Column Base Design :

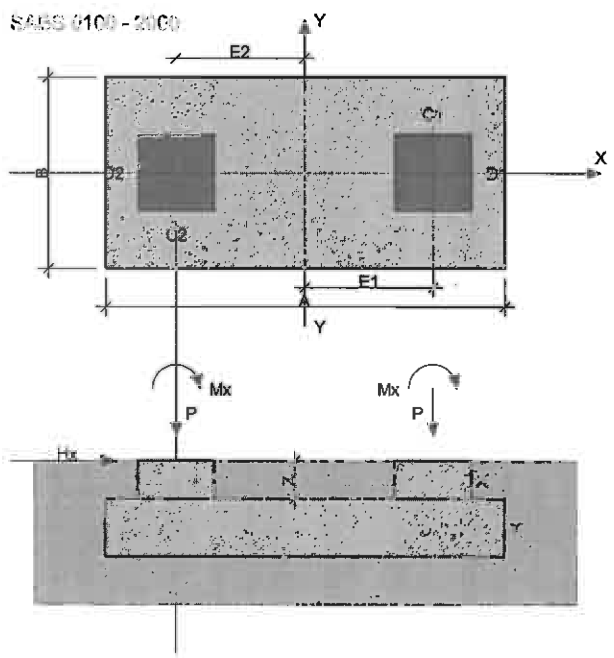
C15

Input Data

Base length A (m)	4.17	
Base width B (m)	2	
Column(s)	Col 1	Col 2
C (m)	0.8	0.8
D (m)	0.8	0.8
E (m)	1.343	-1.343
F (m)		
Stub column height X (m)	0.4	
Base depth Y (m)	0.6	
Soil cover Z (m)	0.4	
Concrete density (kN/m^3)	25	
Soil density (kN/m^3)	18	
Soil friction angle (°)	30	
Base friction constant	0.5	
Rebar depth top X (mm)	80	
Rebar depth top Y (mm)	88	
Rebar depth bottom X (mm)	80	
Rebar depth bottom Y (mm)	88	
ULS ovt. LF: Self weight	1.2	
ULS LF: Self weight	1	
Max. SLS bearing pr. (kN/m^2)	175	
S.F. Overturning (ULS)	2	
S.F. Slip (ULS)	1.5	
fcu base (MPa)	35	
fcu columns (MPa)	35	
fy (MPa)	450	

		Unfactored Loads						
Load Case	Col no.	LF ULS ovt	LF ULS	P (kN)	Hx (kN)	Hy (kN)	Mx (kNm)	My (kNm)
1	1			300		50		
	2			300		50		

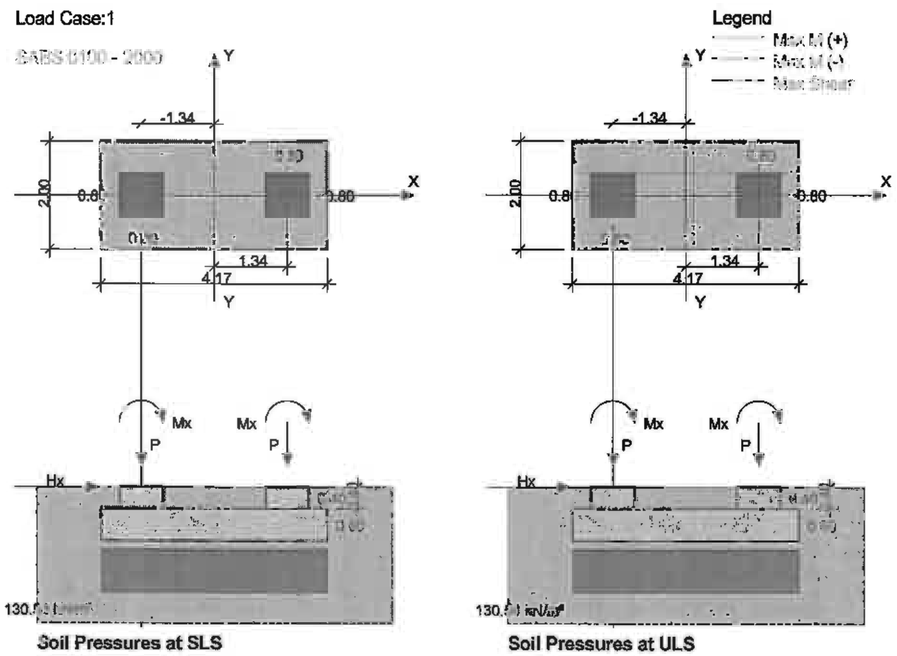
Sketch of Base



PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title		
	Client		
	Calcs by	Checked by	Date

Output for Load Case 1

Output for Load Case 1	
Soil pressure (ULS) (kN/m²)	130.54
Soil pressure (SLS) (kN/m²)	130.54
SF overturning (SLS)	7.89
SF overturning (ULS)	8.26
Safety Factor slip (ULS)	4.89
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	17.21
Reinforcement X (mm²/m)	89
Design moment Y (kNm/m)	18.56
Reinforcement Y (mm²/m)	97
Top	
Design moment X (kNm/m)	-44.16
Reinforcement X (mm²/m)	228
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.000
vc (MPa)	0.298
Linear Shear Y (MPa)	0.000
vc (MPa)	0.299
Linear Shear Other (MPa)	0.025
Punching Shear (MPa)	N.A.
vc (MPa)	N.A.
Cost	0.00



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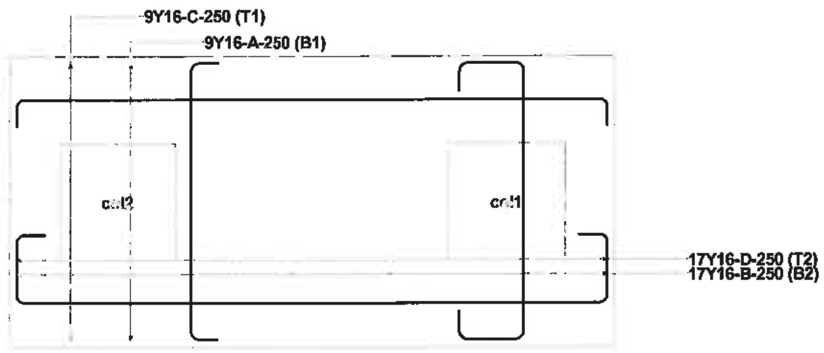
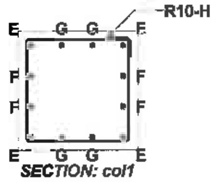
	Job Title		
	Client		
	Calcs by	Checked by	Date

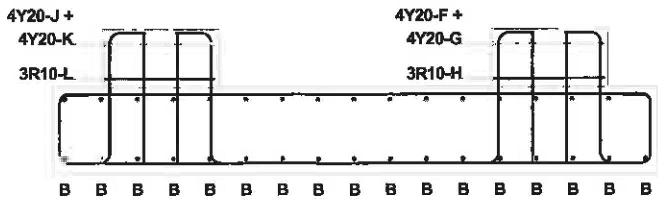
Schematic reinforcement of Base



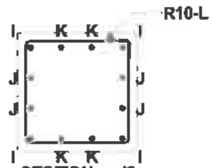


SECTION: col1

PLAN



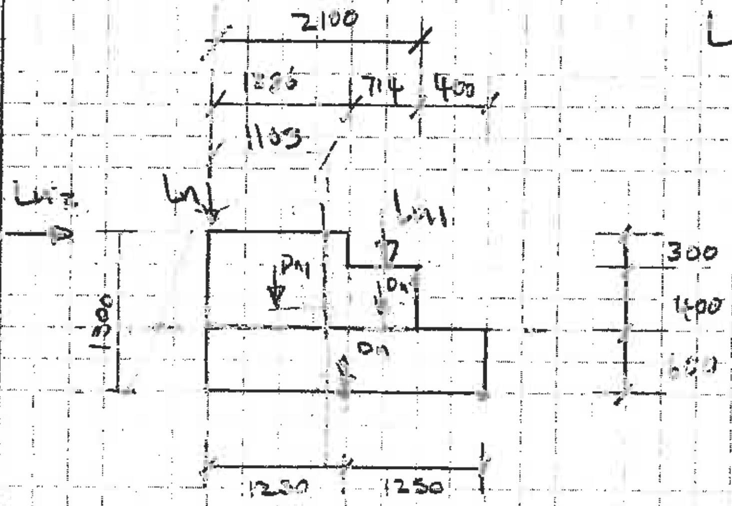
SECTION



SECTION: col2

Project: Medupi Buildings - Weighbridge | Project No: 14133
Component: Base design | Sheet No: 01
Compiled By: S Moguz | Date: 01/06 | Checked By: | Date:

TMH 7	Design Calculations	Results



L = 4.17

Design loads

$- D_{H1} = \gamma \times V = 25 \times 0.875 \times 3.17$ (γ kN/m³, A, L)

$= 69.344$ kN

$- D_{H2} = \gamma_{kN/m^3} \times V_{m^3} = 25 \times 0.381 \times 3.17$

$= 30.194$ kN

$- D_H = \gamma \times V = 25 \times 2.5 \times 0.6 \times 4.17$

$= 156.375$ kN

Note D_H = self weight of base ∴ Axial force

D_{H1} & D_{H2} = are eccentric loads causing a Moment in the base

$L_H = 500 \times 0.8 = 400$ kN

$L_{H1} = 500 \times 0.2 = 100$ kN

$L_{H2} = 100$ kN

print A form (012) 346 1923 | Jan 2005

Project: Project No:
Component: Sheet No: 02
Compiled By: Date: Checked By: Date:

Design calculations

Summary of loads
checking of max moment acting on base

Description	Force	Lever Arm	Moment
Dn	156,375	0	
Dn1	69,344	-0,625	-43,34
Dn2	30,194	0,387	11,685
Ln	400	-1,25	-500
Ln1	100	0,493	49,3
Ln2	100	0,7	70,00
Σ	Σ_v 755,913		-382,355 kN.m

$\frac{L}{6} = \frac{2,5}{6} = 0,417$

$Ecc = \frac{M}{N} = \frac{382,355}{755,913}$

$= 0,505\ m$

$Ecc > \frac{L}{6}$ ∴ Resultant outside mid third

$P_{max} = \frac{2N}{3 \times B(D/2 - e)}$

$P_{max} = 162,214\ kN/m^2$

print A form (012) 346 1923 Jar

Project:

Component:

Compiled By: Date: Checked By: Date:

Project No:

Sheet No: 03

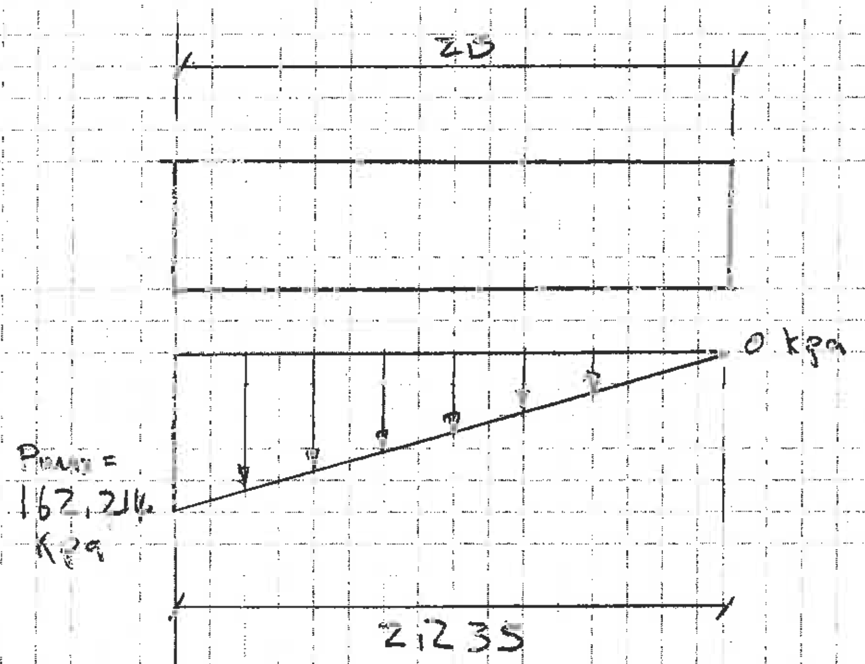
Design calculations

$P_{max} = 162{,}214 \text{ kpa}$

$$y = 3\left(\frac{D}{2} - e\right).$$

$$= 3 \times \left(\frac{2.5}{2} - 0{,}505\right)$$

$$= 2{,}235 \text{ m.}$$



PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number		Sheet
	Job Title		
	Client		
	Calcs by	Checked by	Date

Column Base Design :

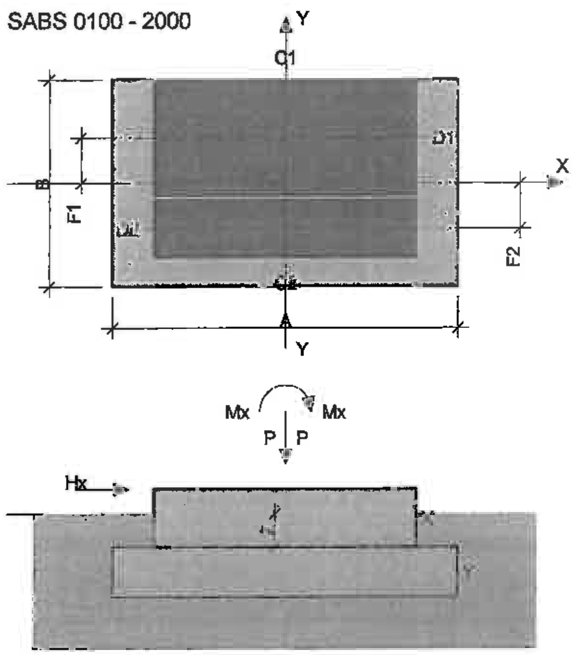
C15

Input Data

Base length A (m)	4.17	
Base width B (m)	2.50	
Column(s)	Col 1	Col 2
C (m)	3.17	3.17
D (m)	1.4	0.7
E (m)	0	0
F (m)	0.55	-0.55
Stub column height X (m)	0.7	
Base depth Y (m)	0.6	
Soil cover Z (m)	0.4	
Concrete density (kN/m³)	25	
Soil density (kN/m³)	18	
Soil friction angle (°)	30	
Base friction constant	0.5	
Rebar depth top X (mm)	80	
Rebar depth top Y (mm)	88	
Rebar depth bottom X (mm)	80	
Rebar depth bottom Y (mm)	88	
ULS ovt. LF: Self weight	1.2	
ULS LF: Self weight	1	
Max. SLS bearing pr. (kN/m²)	175	
S.F. Overturning (ULS)	2	
S.F. Slip (ULS)	1.5	
fcu base (MPa)	35	
fcu columns (MPa)	35	
fy (MPa)	450	

		Unfactored Loads						
Load Case	Col no.	LF ULS ovt	LF ULS	P (kN)	Hx (kN)	Hy (kN)	Mx (kNm)	My (kNm)
1	1					100		-383
	2			100		0		
2	1			400		10		
	2			100				

Sketch of Base

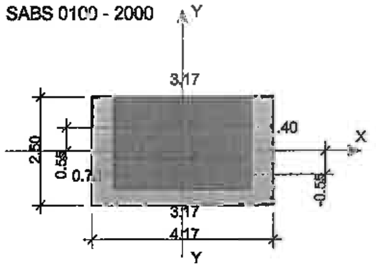


Output for Load Case 1

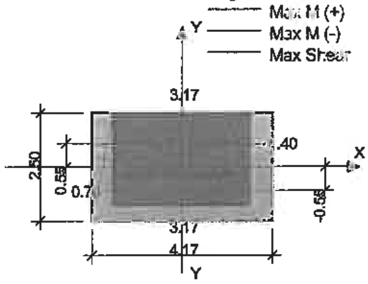
Output for Load Case 1	
Soil pressure (ULS) (kN/m²)	125.04
Soil pressure (SLS) (kN/m²)	125.04
SF overturning (SLS)	1.69
SF overturning (ULS)	1.96
Safety Factor slip (ULS)	2.95
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	44.58
Reinforcement X (mm²/m)	231
Design moment Y (kNm/m)	60.54
Reinforcement Y (mm²/m)	318
Top	
Design moment X (kNm/m)	0.00
Reinforcement X (mm²/m)	0
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.000
vc (MPa)	0.298
Linear Shear Y (MPa)	0.000
vc (MPa)	0.299
Linear Shear Other (MPa)	0.052
Punching Shear (MPa)	N.A.
vc (MPa)	N.A.
Cost	0.00

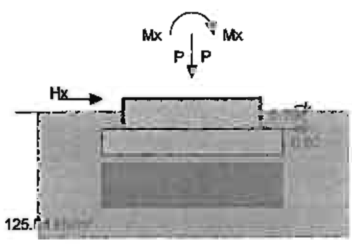
Load Case:1

SABS 0100 - 2000

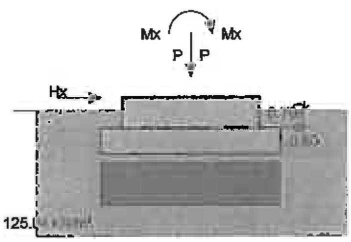


Legend
Max M (+)
Max M (-)
Max Shear





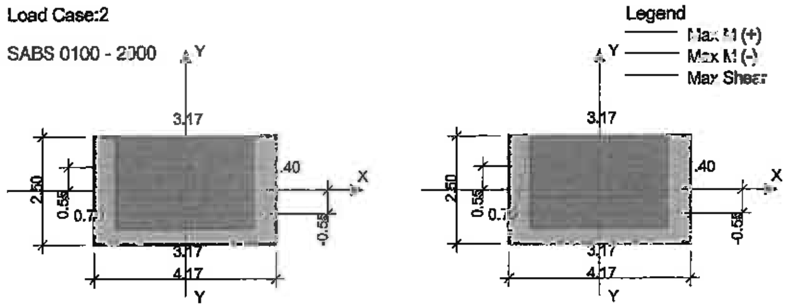
Soil Pressures at SLS

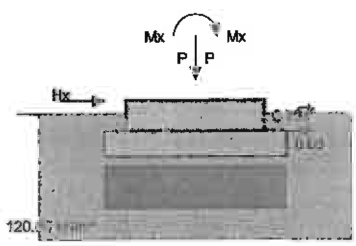


Soil Pressures at ULS

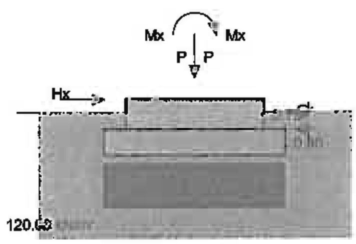
Output for Load Case 2

Output for Load Case 2	
Soil pressure (ULS) (kN/m²)	120.60
Soil pressure (SLS) (kN/m²)	120.60
SF overturning (SLS)	5.25
SF overturning (ULS)	5.57
Safety Factor slip (ULS)	49.46
Safety Factor uplift (ULS)	>100
Bottom	
Design moment X (kNm/m)	49.31
Reinforcement X (mm²/m)	255
Design moment Y (kNm/m)	30.15
Reinforcement Y (mm²/m)	158
Top	
Design moment X (kNm/m)	0.00
Reinforcement X (mm²/m)	0
Design moment Y (kNm/m)	0.00
Reinforcement Y (mm²/m)	0
Linear Shear X (MPa)	0.000
vc (MPa)	0.298
Linear Shear Y (MPa)	0.000
vc (MPa)	0.299
Linear Shear Other (MPa)	0.053
Punching Shear (MPa)	N.A.
vc (MPa)	N.A.
Cost	0.00



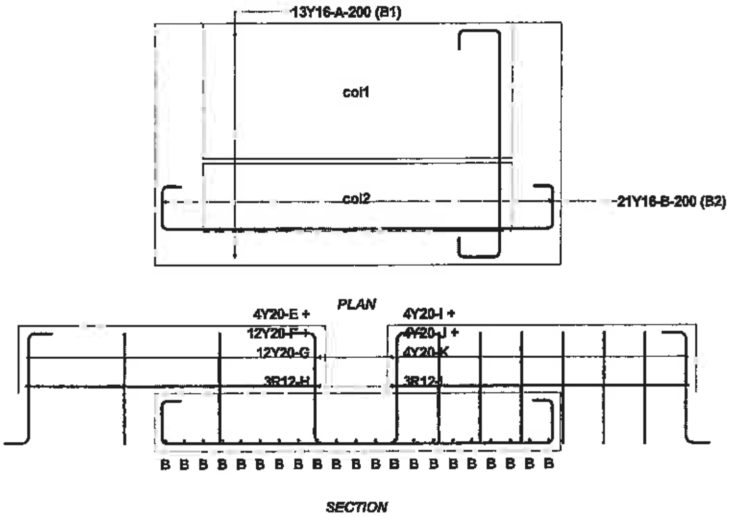


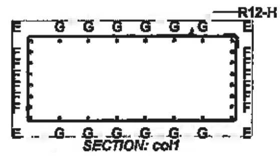
Soil Pressures at SLS

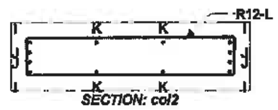


Soil Pressures at ULS

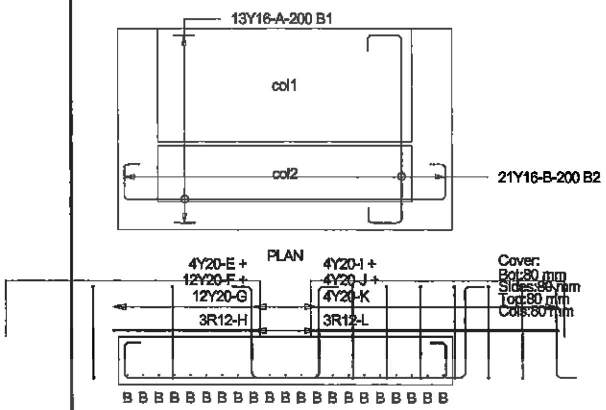
Schematic reinforcement of Base





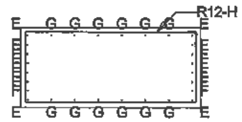


MEMBER	No OF	BARS PER MEMB	DIA	LENGTH	TOTAL NUM-BER	MARK	S C	BENDING A	BENDING B	BENDING C	BENDING D	E/r
	1	21	Y16	3400	21	B	55	190	420	2340	420	
		13	Y16	5100	13	A	55	190	440	4010	440	
		4	Y20	1550	4	E	54	240	1140			
		12	Y20	1550	12	F	54	240	1140			
		12	Y20	1550	12	G	54	240	1140			
		3	R12	8650	3	H	60	1240	3010			
		4	Y20	1550	4	I	54	240	1140			
		4	Y20	1550	4	J	54	240	1140			
		4	Y20	1550	4	K	54	240	1140			
		3	R12	7250	3	L	60	540	3010			

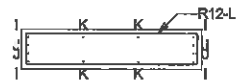


PLAN

SECTION



SECTION: col1



SECTION: col2

	8	10	12	16	20	25	32	40	TOT
R			42						42
Y				217	153				370
TOT			42	217	153				413

Date
Det by
Ref Dwg
Job No
Revision
Schedule No

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Continuous Beam: Beams (Code of Practice : SABS 0100 - 2000)

12/06/2009 11:21:52 AM

C01

Input Tables

Fcu (MPa)	30
Fy (MPa)	450
Fyv (MPa)	250
% Redistribution	0
Downward/Optimized redistr.	D
Cover to centre top steel(mm)	40
Cover to centre bot.steel(mm)	40
Dead Load Factor	1.2
Live Load Factor	1.6
Density of concrete (kN/m3)	24
% Live load permanent	25
Ø (Creep coefficient)	2
Ecs (Free shrinkage strain)	300E-6

Sec No.	Bw (mm)	D (mm)	Bf-top (mm)	Hf-top (mm)	Bf-bot (mm)	Hf-bot (mm)	Y-offset (mm)	Web offset	Flange offset
1	300	350	0	0	0	0		0	0

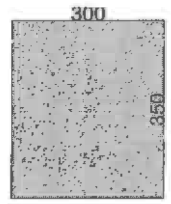
Span No	Section Length(m)	Sec No Left	Sec No Right
1	5.143	1	

Sup No.	Code C,F	Column Below			Code F,P	Column above			Code F,P
		D(mm)	B(mm)	H(m)		D(mm)	B(mm)	H(m)	
1		300							
2		300							

Case D,L	Span	Wleft (kN/m)	Wright (kN/m)	a (m)	b (m)	P (kN)	a (m)	M (kNm)	a (m)

Span	M left (kNm)	M right (kNm)

Sections



SECTION 1

Beam Elevation

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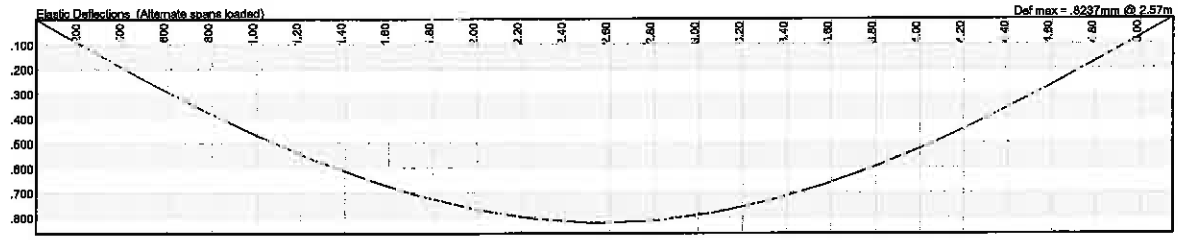
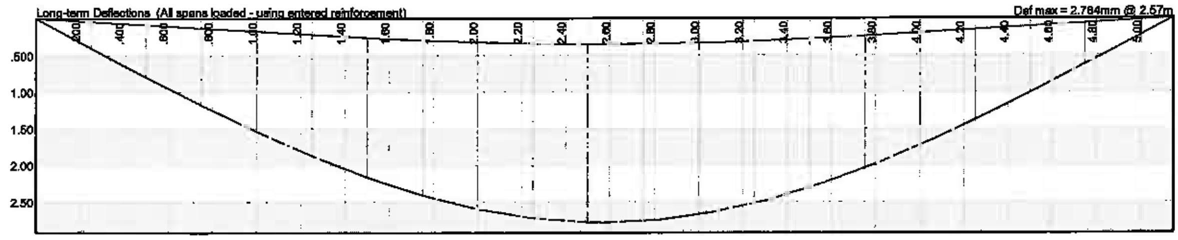
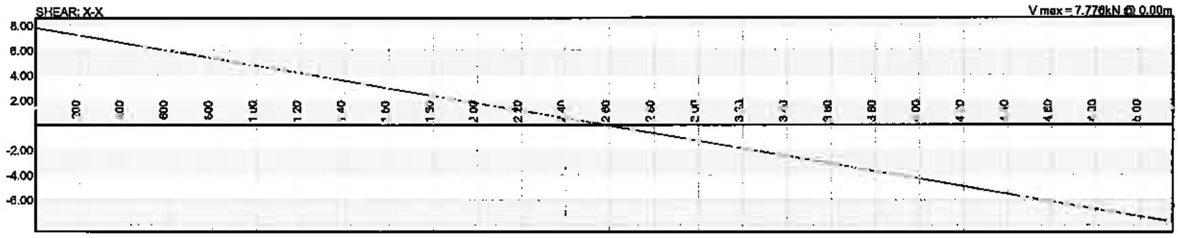
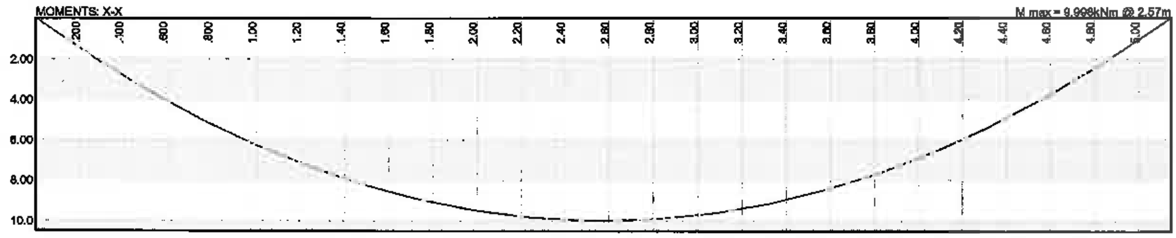
	Job Title		
	Client		
	Calcs by	Checked by	Date

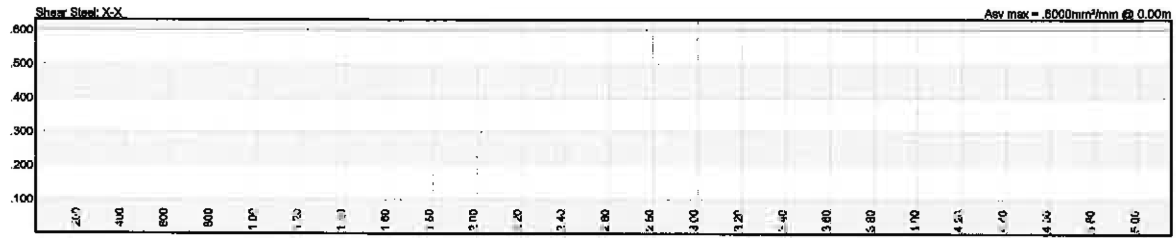
Deflection, Shear, Moment and Steel diagrams. Span 1

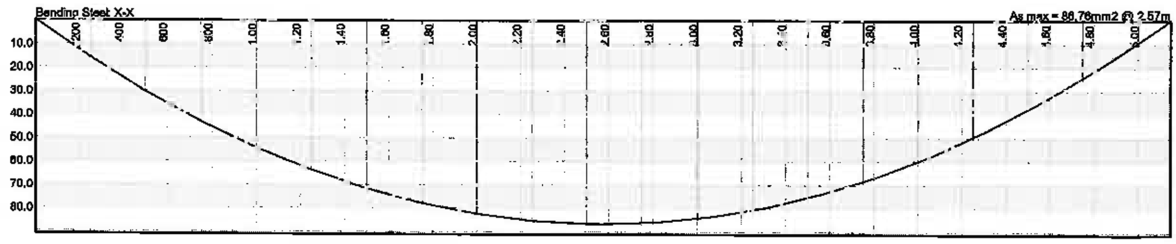












Reinforcing Tables

Beam type	(1-5)	1
Maximum bar length	(m)	13
TOP: Minimum bar diameter	(mm)	16
Maximum bar diameter	(mm)	40
BOTTOM: Minimum bar diameter	(mm)	16
Maximum bar diameter	(mm)	40
STIRRUP: Minimum diameter	(mm)	8
Maximum diameter	(mm)	16
Stirrup shape code		60
First bar mark - top		A
First bar mark - mid.[optional]		
First bar mark - bot.[optional]		
Cover to stirrups - top	(mm)	80
Cover to stirrups - bot	(mm)	80
Cover to stirrups - sides	(mm)	80
Minimum stirrups as % of nominal		100
Loose method of detailing	(Y/N)	N
Exploded elevation	(Y/N)	Y
Bending schedule scale	1:	Auto
Number bars from left to right	(Y/N)	Y
Middle bar diameter	(mm)	Auto

Bars	Mark	SC	Span	Offset	Length	Hook	Layer
2Y16	A	35	1	-0.070	5.283	R	T
2Y16	B	35	1	-0.070	5.283	R	B

Stirrup Number	Section Number	Bar Type + Diameter	Mark	Shape Code	Top Offset	Top Flange	Bottom Flange
1	1	R10	SA	60			

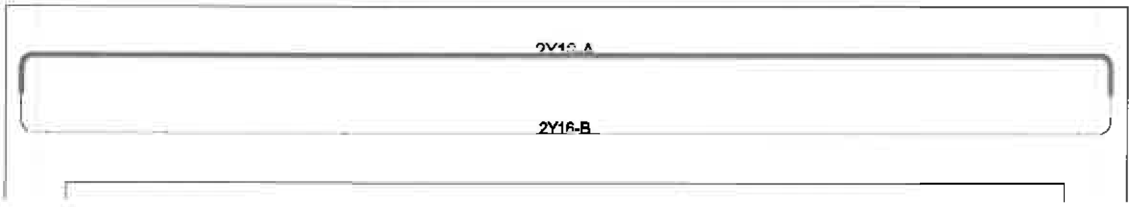
Stirrup Number	Spacing	Span	Offset	Length (m)
1	200	1	0.25	4.60

Section positions		
Label	Span	Offset(m)
A	1	2.8

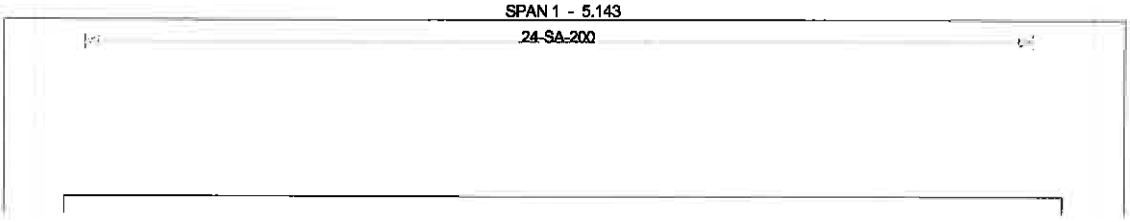
File name	Beams.PAD
First span	
Last span	1
Draw grid lines?(Y/N)	
Draw columns?(Y/N)	
Number/Letter of first grid	
Number (U)p or (D)own	
Drawing size?(A4 or A5)	
Height (normal text) mm	2.0
Height (title text) mm	2.5
Height (sec. bar mark) mm	1.8

Main and Shear Reinforcing diagrams. Span 1

SPAN 1 - 5.143



SPAN 1 - 5.143



Crack Width Design

0.3	mm

Crack width Limit

* *0.3mm for General Structures, 0.2mm for Water Retaining Structures, 0.1mm for Chemicals*
* Linked to Spreadsheet, "SUMMARY"

Project:	Medupi Power station-
Building:	Weigh bridge -300x350 ground beam
Date:	13-Mar-14
Designer:	Trevor Mathabatha

Crack Widths due to Moment @ SLS (SABS 0100-1, ANNEX A)

* *Only edit "Yellow" cells*

ACCEPTABLE

Type of Beam	Rectangular Beam	* If I-Beam, Select in Flange in Tension For Both Sagging and Hogging Moments. If Rectangular, Select Either Type of Bending Moment
Type of Bending Moment	Web in Tension	

Beam Cross Section Height (h)	350	mm
	0.35	m
Flange Height (hf)	0	mm
	0	m
Web Width (bw)	300	mm
	0.3	m
Flange Width (bf)	300	mm
	0.3	m
Applicable Width for Calculation (bt)	300	mm
	0.3	m

Characteristic Cube Strength (f_{cu})	35	MPa
	35000000	Pa
Characteristic Yield Strength (f_y)	450	MPa
	450000000	Pa
Cover	80	mm
Diameter of Stirrups (Taken as 0; 40mm Cover is taken to Main Steel)	12	mm
Diameter of Reinforcing (Maximum)	16	mm
Spacing of Bars	200	mm
Tensile Reinforcement (A_{st})	403	mm^2
Effective Depth of Tensile Rebar (A_{st})	[illegible]	mm
	0.25	m

BENDING MOMENT (SLS)	9.980	kN.m
	9980.000	N.m
Cover (c_{min})	80	mm
	0.08	m

Potential Crack Length (a_{cr})	135.88	mm
Characteristic Cube Strength (f_{cu})	35	MPa
	35000000	Pa

Young's Modulus for Steel (E_s)	200	GPa
K_0 For Normal Weight Concrete	20	GPa
Young's Modulus for Concrete (E_c) = K_0 + 0.2GPa(f_{cu}/MPa)	27	GPa
Long Term Creep Coefficient	1	
Effective Young's Modulus for Concrete (E_{eff})	13.5	GPa
x = NA Depth for Cracked Section	[illegible]	mm
	[illegible]	m

Found by: $ax^2 + bx + c = 0$		
Web in Tension: NA Assumed in Web		
Quadratic Equation (a):	bw/2	150
Quadratic Equation (b):	[(Es/Eeff)*Ast]+ [hf * (bf - bw)]	5970.37037
Quadratic Equation (c):	[-0.5*(bf - bw)*hf^2] - [(Es/Eeff) * Ast * d]	-1492592.593
x1		81.817
Web in Tension: NA Assumed in Flange		
Quadratic Equation (a):	bf/2	150
Quadratic Equation (b):	(Es/Eeff)*Ast	5970.37037
Quadratic Equation (c):	-(Es/Eeff) * Ast * d	-1492592.593
x2		81.817

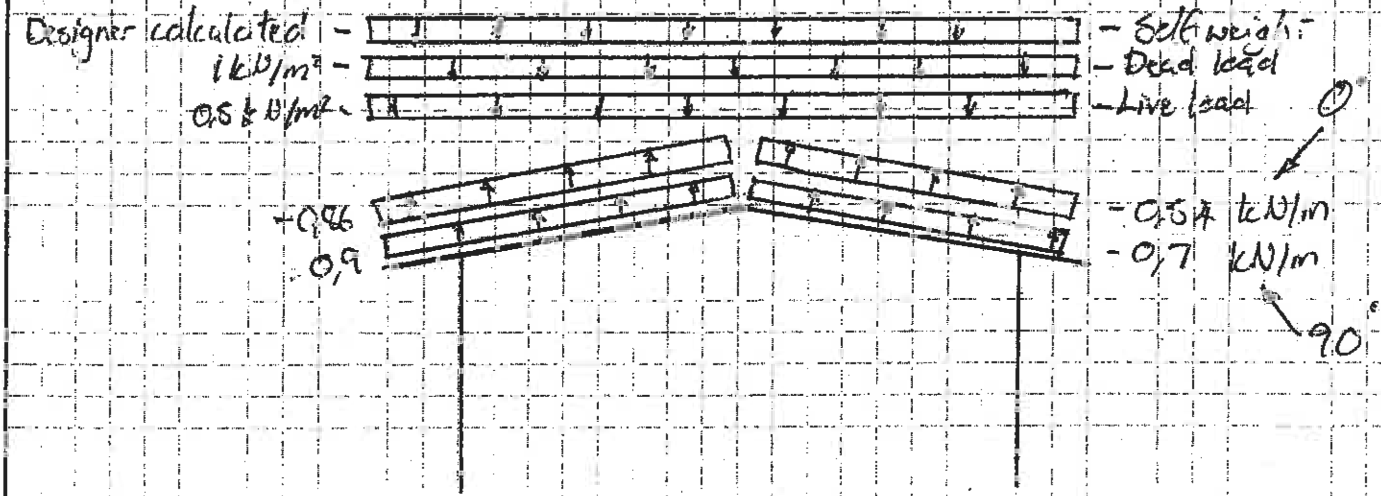
$x = [-b +- sqrt(b^2 - 4ac)]/2a$		
$\varepsilon_1 = (M/[E_s*A_{st}*(d-x/3)])*[(h-x)/(d-x)]$	0.000886484	mm/mm
$\varepsilon_{ls} = [b_t*(h-x)^2]/[3*E_s*A_{st}*(d-x)]$	0.000530573	Units to use: mm, Mpa, N
$\varepsilon_m = \varepsilon_1 - \varepsilon_{ls}$	0.000355911	mm/mm

Crack Width = $w_{max} = (3a_{cr} * \varepsilon_m)/(1+2(a_{cr}-c_{min})/(h-x))$	[illegible]	mm

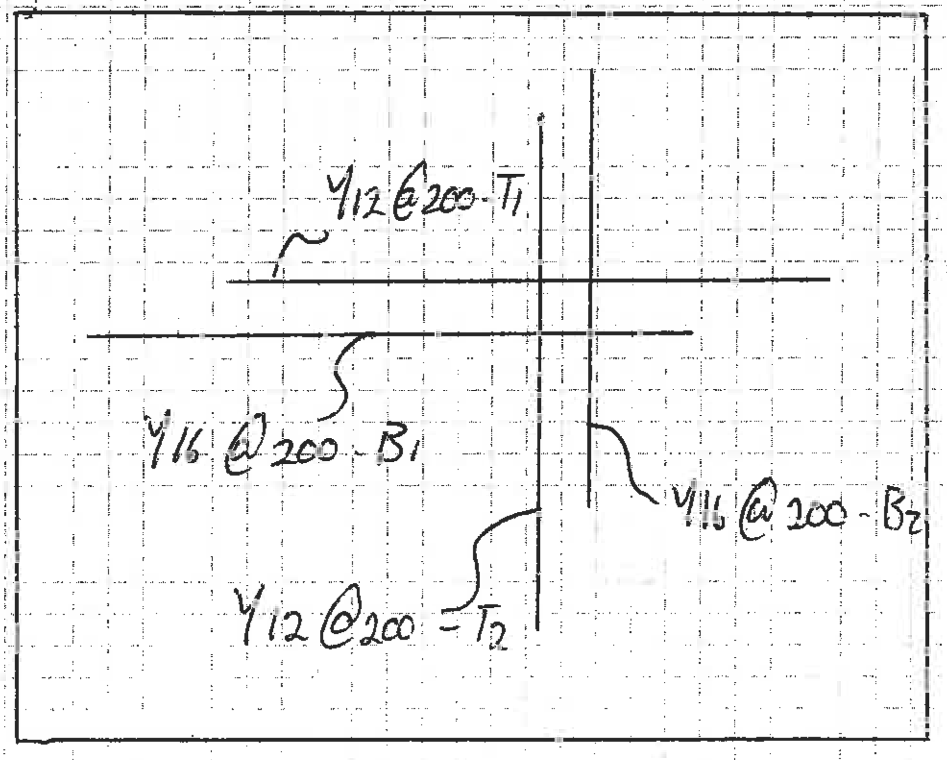
4. DESIGN SUMMARY

Project: Medupi weigh bridge control room | Project No: 14183
Component: Wind loads | Sheet No:
Compiled By: DN vd M | Date: 18-05-09 | Checked By: | Date:

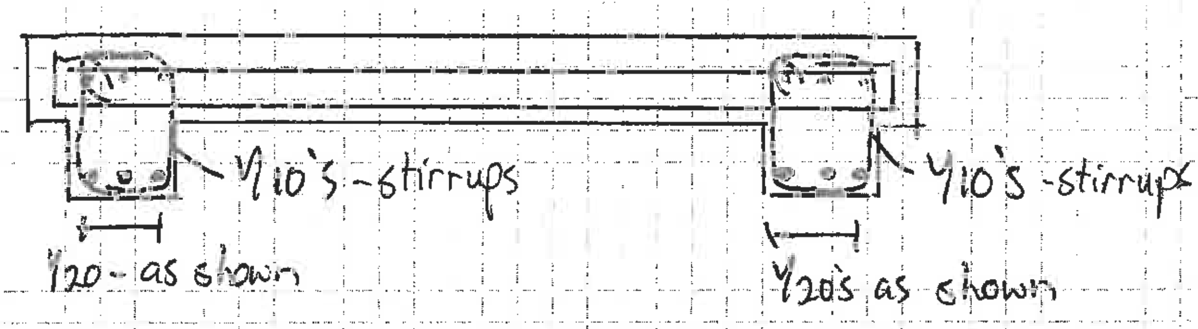
Summary



Project: Medupi Buildings | Project No: 14133
Component: weigh bridge control room slab reinforcement | Sheet No:
Compiled By: DJJ vd M | Date: 10-06-09 | Checked By: | Date:



Cover = 40 mm



5. DESIGN REVIEW MEETING MINUTES

Document Name	Revision	Title	ALTDO(	ALTDC	Create U	Creation D	Comment Nar	Comment Category	Comment Review Status	Comment
200-81502	[0,1]	STRL	14133-D	0	ngwenyd	########	KKS	KKS Review	Accepted With Comments	Add KKS code 0 1UC to the File and all Attached Drawings. 1.) detail design of steel stairs to be included in the report. 2.) concrete crack width calculations (bases, beams and slab) to be included in the report.
200-81502	[0,1]	STRL	14133-D	0	beetgew	########	WB COMMENT		Not Accepted	
200-81502	[0,1]	STRL	14133-D	0	bisschm(	########	200-81502		Not Accepted	Report incomplete. No crack width calculations submitted.