#### **Microwave Link Installation Report**

#### **And Structural Calculation Report**

For Microwave Antenna Fixing At So Lo Pun

#### (Re-submission A)

Chan Lit/Ming (RSE 18/00) Registered Structural Engineer

Project No.	:	J8009 – S2693-LANDS	Prepared	Checked	Approved
Client	:	HGC Global Communications Limited	Λ	Л	1
Structural Engineer	:	JEG Engineering Company Limited	Ø	n	
Date	:	April 2024	M	Tim	A

#### Contents:

1	Introduction	1
2	Design Calculation for Equipment	S1 ~ S28
3	Design Calculation for Antenna	A1~A39

4 Site Photos

The following are the Drawings to be submitted with report.

<u>Drawing No.</u>	<u>Rev</u>	Description
S2693-LANDS-1	А	Details of Transmission Microwave Antenna Fixing
S2693-LANDS-2	А	Details of Transmission Microwave Antenna Fixing

#### **Introduction**

This report is to present the omni antenna installation work to be located at So Lo Pun for HGC Global Communications Limited (HGC).

The objective of the structural design calculation is to prove design check for the structural adequacy of antenna posts and equipment.

#### **Microwave Link Installation Works**

#### Installation of Equipment

1 group of BTS equipment (which include 3 nos. of S.S. Cabinet) will be installed.

The proposed equipment location and details of mounting are shown in the structural drawing and in the attached marked photos.

#### Installation of Antennae

6 nos. of microwave antenna (A1 to A6) mounted on 2 nos. of antenna posts (P1 to P2) will be installed.

The proposed antennae location and details of mounting are shown in the structural drawing and in the attached marked photos.

#### Installation of Power Cable

Total 200A three phase electrical power source will be required for the proposed equipment. Application will be submitted to China Light & Power Company Ltd. (CLP) to install additional power meter. In this case, a diameter of 50mm armored power cable from the proposed power meter to the proposed BTS equipment will be installed without blocking any access.

#### Installation of Fiber Cable

A diameter of 50mm armored fiber cable from the proposed BTS equipment to the proposed antenna will be installed without blocking any access.

#### **Conclusion**

Based on our calculation and site inspection, the proposed works will not induce adverse effect to the existing structure and the existing structure is adequate to sustain the proposed load.

#### **Design Code and Reference**

- I. Code of Practice for the Structural Use of Steel 2011;
- II. Code of Practice on Wind Effects Hong Kong 2019;
- III. Code of Practice for Dead and Imposed Loads 2011;
- IV. Hong Kong Building (Construction) Regulation;
- V. Hilti Fastening Technology Manual;

#### Material Strength

- I. All structural steel shall be class 1 of Grade S275 minimum to BS EN 10025 part 1 to 6:
   2004 standards or equivalent, and complying with the relevant reference material standards in Annex A1.1 of the Code of Practice for the Structural Use of Steel 2011;
- All welds shall comply with BS EN 1011 Part 1: 2009 & Part 2: 2001 with weld strength of 220 N/mm<sup>2</sup>;
- III. All structural steelworks shall be hot dip galvanized in compliance with BS EN ISO 1461:
   2009, with minimum thickness not less than 85µm unless otherwise stated;
- IV. All steel bolt/stud to be grade 8.8 comply with BS EN ISO 3560 part 1&2: 2009 ( $P_{tb} = 560 \text{N/mm}^2$ ,  $P_{sb} = 375 \text{N/mm}^2$ )

#### **Design Calculation**

<b>JEG</b>	ob				<b>Job No.</b> J8009 52693	Page S
CALCULATION C	Calculations by		Checked by	A. Chan	Date	pr 2024
Design Wind Pres	ssure Calcula	ation und	ler CoP on	Wind Effect	cs 2019	
Max. Actual Height , Z	Z, above ground le	vel =	2.5 m			
Wind reference pressu	re, Qo,z	=	3.7 ( 2.5/ 50	0)^0.16	==>	1.59 kPa
Directionality factor o	on pressure, Sø	=	0.85 (assi	ume cricital value	e)	
Max. Slope Height =	= 0 m;	Max. Slope	Length =	0 m;		
Upwind slope of topog	raphic feature	<u></u>	010	==>	#DIV/0!	<0.03
The topography factor	r, St	=	1.00 Outs	ide the topograp	ohy signific	ant zone
Net pressure on surfa	nce, Qz =	Qo,z St Se	. ==>	1.35 kPa		
Building Size	=	1.50 (V	V)× 3.40	(D) x 2.30 (	(H)	
Force coefficient (W), (	Cf <sub>1</sub> =	1.11	$C_f = 1.1 + \frac{1}{2}$	0.055 / p{ log_[(0.6B/D)(1 - 0.01	H <sub>e</sub> /D	113(H,/D) <sup>2</sup> ]}
Force coefficient (D), (		1.18	Where	μ( <i>μαg</i> ει(0.05) σ γ(1 → 0.01.		Equation 4-1
The size factor, Ss	=	1.00	where H <sub>e</sub>	effective building height, surroundings.	, based on <i>H</i> , tak	ing account of
The finial design wind p	pressure (W) =	Qz x Cf x S	B D	breadth of building depth of building		
nie iniai aceign wina p			Equation	m 4-1 can be used for $H_e/D$ s	≤ 12.	
	=	1.35 x 1.11 x	1		······	- Conser zone
		1.50 k	Pa	1.3		– Edge zone – Other
The finial design wind p	pressure (D) =	Qz x Cf x S	6	SC 112		
	=	1.35 x 1.18	×1	1.0 0.9		· · · · · · · · · · · · · · · · · · ·
	=	1.60 k	Pa	0.8		
				0.7		
				1 10 Half-perimeter of t	100 he loaded area,	1000 L <sub>0.5p</sub> (m)
				Figure 5-2	Size factor, S,	

JEG	Job			<b>Job No.</b> J8009 52693	Page S2
CALCULATION	Calculations by	Checked by	A. Chan	Data	pr 2024
Design Loading	g For Equipment Unit				
Max. Overall Size of	Cabinet or Antenna = 1.00	m (W) × 1.0	0 m(D) x 2.5	30 m(H	)
Max. Frontal Area o = 3.40 x 2.30		= 7.82	m²(Ax) □ Y.		
Max. Frontal Area c = 1.50 x 2.30	f Cabinet in y dir + 0.65 x 0.60	= 3.84	m <sup>2</sup> (Ay)	x <u>PLA</u>	
<b>Wind Load:</b> W.L. = 1.35 kPa	a ( the building height und	der 3 m)	 		
Force Coefficient, G	f = 1.2		r 77777777	*****	
Live Load:				<u>eleva</u>	<u>TION</u>
Lyupmone son worg	ht = Equipment s/w of conc. Plinth = 24		) x 2 Nos.		400 kg 5832 kg
			Total Wei	ght =	6233 kg
Unit Weight =	1ax Weight <u>62.</u> oaded area 3.45 x		= 5.24 < 50	0.00 kPa	a

JEG						<b>Job No.</b> J8009 52693	Page S3
ALCULATION Cal	culations by	Checked b	P <sup>y</sup> A	. Chan	ł	Date /	\pr 2024
Checking for Equ	iipment Cab	inet		_			
Y X				- PI	-AN		
Design Data		Wc	,				
Design Wind Pressure, Force Coefficient, Cf Max. Overall Size of Ca Proposed Plinth Size	•	= 1.35 kPa = 1.20 na = 1.00 m (W) = = 3.45 m (Wc) =					n (H) n (Hc)
Check Stability of Equ	lipment Cabinet	;					
Check Overturning (wind	•						
Overturning Moment, Mo	•	(H/2) .82 x ( 2.3 / 2 )	=	14.6	kNm		
Resisting Moment, Mr		of cabinet × (Wc / 2)		07.5			
Factor of Safety		(1075) 1110	. =	7.4	> =	1.5	OK!
Check Overturning (wind	y dir)						
Overturning Moment, M		(H/2) 9.84 x (2.3 / 2)	=	7.2	kNm		
Resisting Moment, Mr	= total weight = 6233 / 100	of cabinet x (Lc / 2) x 3.45 / 2	. = 1	07.5	kNm		
Factor of Safety	= Mr / Mo	= (107.5) / 7.2	= '	15.03	> =	1.5	OKI
Check Sliding Case (wind		by 3.2m height existin	lg R.C. W	vali			
Sliding Force, Fs	= q x Cf x Ax = 1.35 x 1.2 x 7	.82	=	12.7	kN		
Resisting Force, Fr	= total weight = 6233 / 100	of cabinet x $\mu$	+1	24.9	kN		
Factor of Safety	= 62557100 = Fr / Fø	= ( 24.9 ) / 12.7		1.97		1.5	OK!
Check Sliding Case (wind	d y dir) – enclosed	by 3.2m height existir	1g R.C. V	vall			
Sliding Force, Fs	$= q \times Cf \times Ay$ = 1.35 x 1.2 x 3		100	6.2	kN		
Resisting Force, Fr	= total weight						
	= 6233 / 100	~ .		24.9	kN		

JEG	Job			Job No. J8009 52963	Page 54
CALCULATION	Calculations by	Date Apr 2024			
Design for S.S. Cabin	et				
Ĭ	ting into				
Max. Equipment size	e: 1500 (W) x 1700	(D) x 2300 (H)			9 3 107 
Max Weight of Equip	ement, s/w = 200 kg				ی بو بو
Wind pressure, qz	= 1.35 kP	a (Building under 2	2.5 m(max))		
Force coefficient, Cf	= 2.0				1 1 1 1
Shear due to Wind lo	pad,Vx = qz x Cf x D	xH = 1.35 x 2 x 1	.7 x 2.3	= 10.56	kN
Shear due to Wind L	.oad,Vy = qz × Cf × W	′×H = 1.35×2×1	.5 x 2.3 :	= 9.32	kN
Moment due to Wind	d Load, Mx = Vy x H / 2	= 9.315 x 2.3	5/2	= 10.71	kNm
Moment due to Wind	4 Load, My = Vx x H / 2	= 10.557 × 2	.3/2 :	= 12.14	kNm

Adopt 8 nos. HST3-R-M10 Hilti Bolt

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Company:		Page:	0
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	09-2693-S.S. Cabinet	Date:	17/4/2024
Fastening point			
Specifier's comments:			

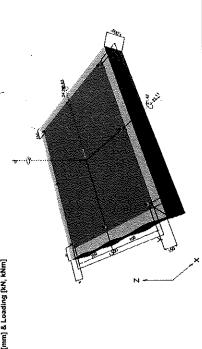
Specifier

			10		n), h <sub>nem</sub> = 68.0 mm				Aechanical	: 3.0 mm	$l_x x l_y x t = 1,500.0 \text{ mm x} 1,700.0 \text{ mm x} 3.0 \text{ mm;}$ (Recommended plate thickness: not calculated)		cracked contrete, C25/30, $f_{\rm cor}$ = 25.00 N/mm²; h =1,000.0 mm, User-defined partial material safety factor $r_{\rm c}$ = 1.500	ation condition: Dry	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)	with longitudinal edge reinforcement d >= 12.0 [mm] + close mesh (stimups, hangers) s <= 100.0 [mm]
	HST3-R M10 hef2	50	2105864 HST3-R M10x90 30/10	ap filling solution	$h_{at,act} = 60.0 \text{ mm} (h_{at,limit} = - \text{mm}), h_{nom} = 68.0 \text{ mm}$	A4	ET'A 98/0001	20/7/2023   -	SOFA based on EN 1992-4, Mechanical	$e_b = 0.0 \text{ mm}$ (no stand-off); t = 3.0 mm	l <sub>x</sub> x l <sub>y</sub> x t = 1,500.0 mm x 1,700	no profile	cracked concrete, C25/30, $f_{coy}$ factor $\gamma_c=1.500$	hammer drilled hole, installation condition: Dry	no reinforcement or reinforcem	with longitudinal edge reinforci [mm]
1 Input data	Anchor type and diameter:	Return period (service life in years):	item number:	Filling set or any suitable annular gap filling solution	Effective embedment depth:	Material:	Evaluation Service Report:	Issued I Valid:	Proof;	Stand-off installation:	Anchor plate <sup>R</sup> :	Profile:	Base material:	Installation:	Reinforcement:	

Application also possible with HST4-R M10 under the selected boundary conditions. More information in section Alternative fastening data of this report.

 $^{\rm R}$  - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [mm] & Loading [kN, kNm]



Input data and seelite must be checked for conformity with the existing conditions and for plaustbillty! PROPIS Engineering ( c ) 2003-2004 Hits AG. FL-9494 Schwan Hitl is a registreed trademark of Hits AG. Schwan

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	Program. Specifier:	-
	E-Mail:	
	09-2693-S.S. Cabinet	17/4/2024
1.1 Load combination		
Case Description	Forces [kN] / Moments [kNm] Seismic	Fire Max. Util. Anchor [%]
1 Combination 1	$N = 0.000$ ; $V_x = 21.110$ ; $V_y = 18.630$ ; no	no 79
	$M_x = -21.420$ ; $M_y = 24.280$ ; $M_z = 0.000$ ;	

## 2 Load case/Resulting anchor forces

Compression

Ä

 $\bullet_{\rm Tension}$ 

Ν

1         5,843         3,519         2,639         2,329           2         3,764         3,519         2,639         2,329           3         1,724         3,519         2,639         2,329           4         4,827         3,519         2,639         2,329           5         0,708         3,519         2,639         2,329           6         3,811         3,519         2,639         2,329           7         1,751         3,519         2,639         2,329           7         1,751         3,519         2,639         2,329           8         0,000         3,519         2,639         2,329
3.519 2.639 3.519 2.639 3.519 2.633 3.519 2.633 3.519 2.633 3.519 2.633 3.519 2.633
3.519 2.639 3.519 2.639 3.519 2.639 3.519 2.639 3.519 2.639 3.519 2.639
3.519 2.639 3.519 2.639 3.519 2.639 3.519 2.639 3.519 2.639
3.519 2.639 3.519 2.639 3.519 2.639 3.519 2.639
3.519 2.639 3.519 2.639 3.519 2.639
3.519 2.639 3.519 2.639
3.519 2.639

Anchor forces are calculated based on the assumption of a rigid anchor plate.

Input data and results must be checked for conformity with the existing conditions and for plausibility/ PROFIS Engineering ( c ) 2002-2024 Hitle AG, FL-9494 Schean Mitle s a registered Trademark of Hitle AG, Schean

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Company:		Page:	2
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	09-2693-S.S. Cabinet	Date:	17/4/2024
Fastening point:			
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## 3 Tension load (EN 1992-4, Section 7.2.1)

	Load [kN]	Capacity [kN]	Utilization B <sub>N</sub> [%]	Status
Steel Strength*	5.843	20.500	29	ş
Puliout Strength*	5.843	11,180	53	ð
Concrete Breakout Failure**	5.843	11.929	49	ð
Splitting failure**	NA	N/A	N/A	N/A
* highest loaded anchor **anchor group (anchors in tension)	group (anchors in tension)			

### 3.1 Steel Strength

EN 1992-4, Table 7.1 N<sub>Ed</sub> ≤ N<sub>Rd.s</sub> = <sup>N<sub>Rk.s</sub> <sup>7</sup>M.s</sup>



### 3.2 Pullout Strength

N<sub>Ed</sub> ≤ N<sub>Pd.p</sub> = <u>Ψ<sub>6</sub>. N<sub>Pk.P</sub></u> EN 1992-4, Table 7.1 <sup>Y<sub>M.P</sub></sup>

N <sub>Ed</sub> [kN]	5.843
N <sub>Rdo</sub> [kN]	11.180
1 M.P	1.500
Ψc	1.118
N <sub>Rk.0</sub> [kN]	15.000

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	Page: Specifier:	E-Mail: Date:	
		 09-2693-S.S. Cabinet	
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Failure	
Breakout	
3.3 Concrete	
.3 Concrete Break	

												N <sub>Ed</sub> [kN]	5.843	
										¥ <sub>re.N</sub>	1.000	N <sub>Rd c</sub> [kN]	11.929	
Table 7.1	EQ. (7.1)	Eq. (7.2)	Eq. (7.3)	Eq. (7.4)	Eq. (7.6)	Eq. (7.6)	Eq. (7.7)	f <sub>cevi</sub> [N/mm <sup>2</sup> ]	25,00	e.N	1.000	YMIC.	1.500	
EN 1992-4, Table 7,1	EN 1882-4, EQ. (1.1)	EN 1992-4, Eq. (7.2)	EN 1992-4, Eq. (7.3)	EN 1992-4, Eq. (7.4)	EN 1992-4, Eq. (7.6)	EN 1992-4, Eq. (7.6)	EN 1992-4, Eq. (7.7)	s <sub>er N</sub> [mm]	180.0	W oc2,N	1.000	N <sup>0</sup> <sub>Rke</sub> [kN]	17.893	
	oct.N ' Warz.N ' WARN							c <sub>orN</sub> [mm]	90.0	e <sub>c2,N</sub> [mm]	0.0	k,	7.700	
z	$= \mathbf{N}_{\mathbf{R},\mathbf{L},\mathbf{C}} \cdot \underbrace{-\frac{1}{20}}_{\mathbf{C},\mathbf{N}} \cdot \mathbf{V}_{\mathbf{E},\mathbf{N}} \cdot \mathbf{V}_{\mathbf{R},\mathbf{N}} \cdot \mathbf{V}_{\mathbf{R},\mathbf{N}} \cdot \mathbf{V}_{\mathbf{R},\mathbf{N}}$	Ъ.	Z'5	$= 0.7 + 0.3 \cdot \frac{c_{m}}{c_{crN}} \le 1.00$	$\left(\frac{1}{S_{a,N}}\right) \leq 1.00$	$\frac{1}{\left(\frac{2\cdot e_{N,2}}{s_{n,N}}\right)} \le 1.00$		A <sup>0</sup> <sub>e.N</sub> [mm <sup>2</sup> ]	32,400	W UCI.N	1.000	N'W A	1.000	
NRd.o		$N_{Rk,c}^{0} = k_{1} \cdot V_{fck} \cdot h_{af}^{1}$		Ψ' <sub>s.N</sub> = 0.7 + 0.	$\Psi_{acr,N} = \frac{1}{1 + \left(\frac{2}{3}\right)^2}$	$\psi_{\text{od2,N}} = \frac{1}{1 + \left(\frac{2}{\pi}\right)}$	ψ <sub>M,N</sub> = 1	A <sub>c.N</sub> [mm <sup>2</sup> ]	32,400	e <sub>c1 N</sub> [mm]	0.0	z [mm]	1,442.4	Group anchor ID 1

Input data and results must be checked for contormity with the existing conditions and for plausbility! PROFIS Engineering ( c.) 2303-2024 Hills AG, PL-9494 Schoon Hill is a registered Trademont of Hill AG, Schoon

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Company:		•
		Page:
Address:		Specifier:
Phone I Fax:		E-Mail:
Design: 09-2693-S.S. Cabinet	S. Cabinet	Date:
Fastening point:		

17/4/2024

	Load [kN]	Capacity [kN]	Capacity [kN] Utilization By [%]	Status
Steel Strength (without lever arm)*	3.519	20.240	18	¥
Steel failure (with lever arm)*	N/A	NIA	N/A	N/A
Pryout Strength**	3.519	31.850	12	ð
Concrete edge faiture in direction x+**	22.236	40,496	55	ð
<ul> <li>highest loaded anchor "anchor group (relevant anchors)</li> </ul>	levant anchors)			

## 4.1 Steel Strength (without lever arm)

		V <sub>Rd,s</sub> [kN] 20.240
		Yms 1.250
EN 1992-4, Table 7.2	EN 1992-4, Eq. (7.35)	V <sub>RKs</sub> [KN] 25.300
EN 199	EN 199	k <sub>7</sub> 1.000
$V_{Ed} \leq V_{Rds} = \frac{V_{Rks}}{7u_s}$	$V_{Rk,s} = k_7 \cdot V_{Rk,s}^0$	V <sup>0</sup> <sub>Rks</sub> [kN] 25.300

V<sub>Ed</sub> [kN] 3.519

### 4.2 Pryout Strength

									t <sub>sori</sub> [N/mm <sup>2</sup> ]	25.00	¥.º.N	1.000			
Table 7.2	. Eq. (7.39a)	. Eq. (7.1)	. Eq. (7.2)	. Eq. (7.3)	. Eq. (7.4)	Eq. (7.6)	, Eq. (7.6)	, Eq. (7.7)	k <sub>8</sub>	2.670	₩ <sup>6,N</sup>	1.000	V <sub>Ed</sub> [kN]	3.519	
EN 1992-4,	EN 1992-4,	EN 1992-4,	EN 1992-4,	EN 1992-4.	EN 1992-4,	EN 1992-4,	EN 1992-4,	EN 1992-4,	s <sub>crn</sub> [mm]	180.0	₩ ec2.N	1.000	V <sub>Rd,cp</sub> [kN]	31,850	
		act.N · V ac2.N · Y M.N							c <sub>erk</sub> [mm]	90.06	e <sub>cz'</sub> v [mm]	0.0	YMCP	1.500	
	v	ν	h.	N,	3 · ≤ 1.00	$\left \frac{\mathbf{e}_{V,1}}{\mathbf{e}_{V,N}}\right  \le 1.00$	$\left(\frac{e_{v2}}{2}\right) \le 1.00$	e.N	A <sub>6N</sub> [mm <sup>2</sup> ]	32,400	W act,N	1.000	N <sup>R</sup> ke [KN]	17.893	
Ed ≤ VRd,cp = VRL.cp	Rucp = K <sub>B</sub> · N <sub>RK</sub>					+ -	+	ii N	A <sub>e N</sub> [mm²]	32,400	e <sub>en v</sub> [mm]	0.0	¥	7.700	Group anchor ID 8
	V <sub>64</sub> ≤ V <sub>164</sub> 00 = <sup>V<sub>664</sub>00 </sup>		אשא, אביא, איניאא, איניאא, איניאא, איניאא, אינאא, אינאא, איניאא, איניאא, איניאא, אינאא, אינאא, אינאא, אינאא, א 	אטין - איזעע איזער איזער איזער איזער איזער איזען אטין - איזעער איזער איזער איזער איזער אטין - איזער	ж 1941 1942 - Маун - Части - Ицези - Чили 1943 - Марикания 1944 - Марикан	е 6 <sup>81</sup> - V. «. V. «	: ====================================	$\sum_{\substack{\alpha,n \\ \alpha,n \\ b_{\alpha}^{(1)}}, \psi_{\alpha,n} \cdot \psi_{\alpha,n} \cdot \psi_{\alpha,2,n} \cdot \psi_{\alpha,2,n} \cdot \psi_{\alpha,2,n} \cdot \psi_{\alpha,n} \cdot \psi_{\alpha,n}$	e	$ \begin{array}{c} {\sf EN \ 1992.4, \ {\sf Table \ 7.2} \\ {\sf EN \ 1992.4, \ {\sf EQ \ 7.39a} \\ {\sf EN \ 1992.4, \ {\sf EQ \ 7.1} \\ {\sf EN \ 1992.4, \ {\sf EQ \ 7.1} \\ {\sf EN \ 1992.4, \ {\sf EQ \ 7.1} \\ {\sf EN \ 1992.4, \ {\sf EQ \ 7.1} \\ {\sf EN \ 1992.4, \ {\sf EQ \ 7.2} \\ {\sf EN \ 1992.4, \ {\sf$	$ \begin{array}{c} \text{EN 1992-4, Table 7.2} \\ \text{EN 1992-4, Eq. (7.39a)} \\ \frac{\text{EN 1992-4, Eq. (7.39a)} \\ \frac{\text{EN 1992-4, Eq. (7.31)} \\ \frac{\text{EN 1992-4, Eq. (7.2)} \\ \frac{\text{EN 1992-4, Eq. (7.3)} \\ \frac{\text{EN 1992-4, Eq. (7.4)} \\ \frac{\text{EN 1992-4, Eq. (7.6)} \\ \frac{\text{EN 1992-4, Eq. (7.6)} \\ \frac{\text{EN 1992-4, Eq. (7.7)} \\ \text{EN 1992-4, Eq. (7$	$ \begin{array}{c} \mbox{EN 1992.4, Table 7.2} \\ \mbox{EN 1992.4, Eq. (7.39a)} \\ \mbox{EN 1992.4, Eq. (7.39a)} \\ \mbox{event} & \mbox{EN 1992.4, Eq. (7.1)} \\ \mbox{event} & \mbox{EN 1992.4, Eq. (7.1)} \\ \mbox{event} & \mbox{EN 1992.4, Eq. (7.2)} \\ \mbox{EN 1992.4, Eq. (7.3)} \\ \mbox{EN 1992.4, Eq. (7.4)} \\ \mbox{EN 1992.4, Eq. (7.4)} \\ \mbox{EN 1992.4, Eq. (7.6)} \\ \mbox{EN 1992.4, Eq. (7.6)} \\ \mbox{EN 1992.4, Eq. (7.6)} \\ \mbox{EN 1992.4, Eq. (7.7)} \\ EN $	$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$	$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$

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$\frac{1}{e^{2}} \sum_{k=1}^{2} (w_{k,k} - w_{k,k} -$	Address:				Specifier:		
$ \begin{array}{c} 09-2693-S.S. \ Cabinet \\ \mbox{illure in circaction x+} \\ $	Phone I Fax:				E-Mail:		
indure in direction x+ $\frac{A_{n,v}^{N} \cdot \psi_{a,v} \cdot \psi_{n,v} \cdot \psi_{a,v} \cdot \psi_{a,v} \cdot \psi_{a,v}}{\sum_{n=1}^{N} \cdot \sqrt{k_n} \cdot c_1^{+5}} EN 1992.4, Tal = \frac{A_{n,v}^{N} \cdot \sqrt{k_n} \cdot c_1^{+5}}{\sum_{n=1}^{N} \cdot \sqrt{k_n} \cdot \psi_{a,v} \cdot \psi_{a,v} \cdot \psi_{a,v}} EN 1992.4, Eq. = \frac{1}{10} \circ (2 + 100) (EN 1992.4, Eq. = 1.5 \cdot c_1 \le 1.00) (EN 1992.4, Eq. = 1.5 \cdot c_1 \le 1.00) (EN 1992.4, Eq. = 1.00) (1.700) (0.063) (1.100) $	Design: Fastening point:	09-2693-5	S.S. Cabinet		Date:		17/4
EN 1992.4. Table is $\left(\frac{-\frac{A_{n,v}}{A_{n,v}}, v_{n,v}, v_{n,v}, v_{n,v}, v_{n,v}, EN 1992.4. Eq\frac{A_{n,v}}{A_{n,v}}, v_{n,v}, v_{n,v}, v_{n,v}, EN 1992.4. Eq\frac{A_{n,v}}{A_{n,v}}, v_{n,v}, v_{n,v}, v_{n,v}, v_{n,v}, EN 1992.4. Eq\frac{A_{n,v}}{A_{n,v}}\right)^{0.3}$ = 1000 EN 1992.4. Eq100 EN 1992.4. Eq	4.3 Concrete edge	failure in direction y	t			an a' sa an a ca a ta bha na ca an	
$ \frac{e^{-\frac{A_{a}^{A_{a}}}{A_{a}^{A_{a}}} \cdot v_{a,a} \cdot v_{a,a} \cdot v_{a,a} \cdot v_{a,a} \cdot v_{a,a}}{\frac{1}{2}^{A_{a}} \cdot v_{a,a}^{A_{a}} \cdot v_{a}^{A_{a}} \cdot v_{a}^{A_{a}$	$V_{Ed} \leq V_{Rd,c} = \frac{V_{Rk,c}}{\gamma_{M,c}}$			EN 1992-4,	Table 7.2		
$ = k_{0} \cdot d_{non}^{2} \cdot \frac{1}{n}^{2} \cdot \frac{1}{n} k_{0}^{2} \cdot c_{1}^{15} = k_{0} \cdot \frac{1}{n} \cdot \frac{1}$	V <sub>Rke</sub> = k <sub>T</sub> · V	$A_{Rkc} \cdot \frac{A_{c.V}}{A_{o.V}} \cdot \psi_{s.V} \cdot \psi_{h.V}$	ለ <sup>መ</sup> ስ , <sup>አንወ</sup> ስ , <sup>አማ</sup> ስ , <sup>1</sup>	ËN 1992-4,	Ëq. (7.40)		
$= 0.1 \cdot \left(\frac{1}{c_{c_{c_{c_{c_{c_{c_{c_{c_{c_{c_{c_{c_{$		am 1 Vick C15		EN 1992-4,	. Eq. (7.41)		
$= 0.1 \cdot \left(\frac{4_{\text{exc}}}{c_{1,1}}\right)^{0.2} = 0.1 \cdot \left(\frac{4_{\text{exc}}}{c_{1,2}}\right)^{0.2} = 4.5 \cdot c_{1,1}^{3}$ $= 4.5 \cdot c_{1,1}^{3} = 0.7 + 0.3 \cdot \frac{c_{2,2}}{1.5 \cdot c_{1,1}^{2}} \leq 1.00 = \text{EN 1992.4, Eq.}$ $= 0.7 + 0.3 \cdot \frac{1}{1.5 \cdot c_{1,1}^{2}} \geq 1.00 = \text{EN 1992.4, Eq.}$ $= \frac{1}{\sqrt{\frac{1}{(1001)} \frac{1}{(15 \cdot c_{1,1}^{2})}} \leq 1.00 = \text{EN 1992.4, Eq.}$ $= \sqrt{\frac{1}{(15 \cdot c_{1,1}^{2})}} \leq 1.00 = \text{EN 1992.4, Eq.}$ $= \sqrt{\frac{1}{(15 \cdot c_{1,1}^{2})}} \leq 1.00 = \text{EN 1992.4, Eq.}$ $= \sqrt{\frac{1}{(15 \cdot c_{1,1}^{2})}} \leq 1.00 = \text{EN 1992.4, Eq.}$ $= \sqrt{\frac{1}{(15 \cdot c_{1,1}^{2})}} \leq 1.00 = \text{EN 1992.4, Eq.}$ $= \sqrt{\frac{1}{(1001)} \frac{1}{(100)} \frac{1}{(100)$		راً۔ 1. ا		EN 1992-4	. Eq. (7,42)		
$= 4.5 \cdot c_{1}^{2}$ $= 0.7 + 0.3 \cdot \frac{c_{2}}{1.5 \cdot c_{1}} \leq 1.00$ $= 0.7 + 0.3 \cdot \frac{c_{2}}{1.5 \cdot c_{1}} \leq 1.00$ $= \left(\frac{1.5 \cdot c_{1}}{1}\right)^{0.2} \geq 1.00$ $= \left(\frac{1.5 \cdot c_{1}}{1}\right)^{0.2} \geq 1.00$ $= \left(\frac{1.5 \cdot c_{1}}{1}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{2 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{2 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{2 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{2 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{2 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} \leq 1.00$ $= 1 + \left(\frac{1.6 \cdot e_{1}}{3 \cdot c_{1}}\right)^{0.2} = 1 + \left(\frac{1.6 \cdot e_{1}$		$\left(\frac{d_{nem}}{c_{i}}\right)^{0.2}$		EN 1992-4	, Eq. (7.43)		
$= 0.7 + 0.3 \cdot \frac{c_2}{1.5 \cdot c_1} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \left(\frac{1.5 \cdot c_1}{5}\right)^{3} \ge 1.00 \qquad \text{EN 1992.4, Eq}$ $= \left(\frac{1.5 \cdot c_1}{5 \cdot c_1}\right)^{5} \ge 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{1.5 \cdot c_1}{5 \cdot c_1}\right)^{5}} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{2 \cdot c_1}{5 \cdot c_1}\right)^{5}} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{1}{5 \cdot c_1}\right)^{5}} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{1}{5 \cdot c_1}\right)^{5}} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{1}{5 \cdot c_1}\right)^{5}} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{1}{5 \cdot c_1}\right)^{5}} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{1}{5 \cdot c_1}\right)^{5}} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{1}{5 \cdot c_1}\right)^{5}} \le 1.00 \qquad \text{EN 1992.4, Eq}$ $= \sqrt{\left(\frac{1}{5 \cdot c_1}\right)^{5}} \ge 1.00 \qquad 0.063 \qquad \text{IO 200} \qquad 1.000 \qquad 1.000 \qquad 0.063 \qquad \text{IO 200} \qquad 1.000 \qquad 0.0003 \qquad \text{IO 200} \qquad 1.000 \qquad 0.0003 \qquad \text{IO 200} \qquad \text{IO 200} \qquad 1.000 \qquad 0.0003 \qquad \text{IO 200} \qquad 1.000 \qquad 0.0003 \qquad \text{IO 200} \qquad 1.000 \qquad \text{IO 200} \qquad 1.000 \qquad 1.000 \qquad \text{IO 200} \qquad 1.000 \qquad \text{IO 200} \qquad 1.000 \qquad \text{IO 200} \qquad 1.000 \qquad 1.000 \qquad 1.000 \qquad \text{IO 200} \qquad 1.000 \qquad 1.00$		~ú		EN 1992-4	. Eq. (7.44)		
$= \left(\frac{1.5 \cdot c_1}{h}\right)^{0.5} \ge 1.00 \qquad \text{EN } 1992.4, \text{Eq.}$ $= \sqrt{\frac{1.5 \cdot c_1}{1 + \left(\frac{3 \cdot c_1}{3 \cdot c_1}\right)} \le 1.00 \qquad \text{EN } 1992.4, \text{Eq.}$ $= \sqrt{\frac{1}{(\cos \alpha_{11})^{2} + (0.5 \cdot \sin \alpha_{11})^{2}} \ge 1.00 \qquad \text{EN } 1992.4, \text{Eq.}$ $\frac{1}{(\text{Imm}]} \qquad \frac{1}{4_{\text{con}} \text{Imm}]} \qquad \frac{k_9}{h_9} \qquad \frac{u}{u}$ $= \frac{1}{160.0} \qquad \frac{10.00}{10.00} \qquad 1.700 \qquad 0.063 \qquad \frac{u}{150.0} \qquad \frac{u}{150.0} \qquad \frac{u}{10.00} \qquad \frac{u}{10.00}$		0.3 - <u>c2</u> ≤ 1.00		EN 1992-4	. Eq. (7.45)		
$= \frac{1}{e^{1} + \left(\frac{2}{3} \cdot \frac{a_{1}}{a_{1}}\right)} \leq 1.00$ $= \sqrt{\frac{2}{(\cos \alpha_{1})^{2} + \left(0.5 \cdot \sin \alpha_{1}\right)^{2}} \geq 1.00$ $= \sqrt{\frac{1}{(\cos \alpha_{1})^{2} + \left(0.5 \cdot \sin \alpha_{1}\right)^{2}} \geq 1.00$ $= \sqrt{1992.4, Eq.}$ $= \sqrt{\frac{1}{(100)} \frac{1}{1000} \frac{1}{1.700} \frac{1}{0.063}$ $= \sqrt{\frac{1}{1000} \frac{1}{1.700} \frac{1}{0.063}}$ $= \sqrt{\frac{1}{1000} \frac{1}{1.700} \frac{1}{0.063}}$ $= \sqrt{\frac{1}{1000} \frac{1}{1.700} \frac{1}{0.063}}$ $= \sqrt{\frac{1}{1000} \frac{1}{1.000} \frac{1}{18.31} \frac{1}{1.039}}$ $= \sqrt{\frac{1}{1000} \frac{1}{1.000} \frac{1}{18.31} \frac{1}{1.039}}$ $= \sqrt{\frac{1}{1000} \frac{1}{1.000} \frac{1}{18.31} \frac{1}{1.039}}$		<mark>. c<sub>1</sub>)<sup>0,5</sup></mark> ≥ 1.00		EN 1992-4	, Eq. (7.46)		
$= \sqrt{\frac{1}{\cos \alpha_{v}} \frac{1}{2 + (0.5 - \sin \alpha_{v})^{2}}} \ge 1.00 \qquad \text{EN 1992-4. Eq.}$ $\frac{1}{1000} \frac{1}{\cos \alpha_{v}} \frac{1}{1000} \frac{1}{1000} \frac{1}{2000} \frac{1}{1000} \frac{1}{$	+			EN 1992-4	, Eq. (7.47)		
diam         ks         u           10.00         1.700         0.063           A <sub>k</sub> (fmm <sup>2</sup> )         A <sup>a</sup> <sub>k</sub> (fmm <sup>2</sup> )         0.063           A <sub>k</sub> (fmm <sup>2</sup> )         A <sup>a</sup> <sub>k</sub> (fmm <sup>2</sup> )         0.053           286.975         101.250         101.250           V <sub>KV</sub> u <sub>V</sub> [?]         V <sub>a</sub> v           1.000         18.31         1.039           kr         N <sub>ac</sub> V <sub>ac</sub> (M)           1.000         18.31         1.039	~	1 os α <sub>v</sub> ) <sup>2</sup> + (0.5 - sin α <sub>v</sub>	≥ 1.00	EN 1992-4	. Eq. (7.48)		
10.00         1.700         0.063 $A_{eV}$ [mm <sup>2</sup> ] $A_{eV}^{o}$ [mm <sup>2</sup> ] $A_{eV}^{o}$ [mm <sup>2</sup> ]           286.875         101.250 $u_{eV}$ $V_{hv}$ $u_{v}$ [] $V_{eV}$ $h_{\tau}$ $N_{ev}$ $V_{eve}$ $h_{\tau}$ $N_{eve}$ $V_{eve}$ $h$ $N_{eve}$ $N_{eve}$	l, [mm]	d <sub>nem</sub> [mm]	κ <sup>8</sup>	ъ	£1	( <sub>eed</sub> [N/mm <sup>2</sup> ]	
A <sub>x</sub> , [mm <sup>2</sup> ]         A <sub>x</sub> <sup>0</sup> , [mm <sup>2</sup> ]           286.875         101.250           v <sub>n</sub> v         u <sub>n</sub> [7]         v <sub>n</sub> v           1.000         18.31         1.039           k <sub>1</sub> N <sub>xx</sub> V <sub>max</sub> [8.39           n         1.000         18.31         1.039	60.0	10.00	1.700	0.063	0.058	25.00	
286,875 101,250 V <sub>n</sub> V, $u_r$ [] V <sub>n</sub> V, 1.000 18.31 1.039 k <sub>T</sub> N <sub>kc</sub> V <sub>Rec</sub> [kN] 1.01 1.500 4.50	c, [mm]	$A_{c_V} [mm^2]$	A <sup>0</sup> <sub>6,v</sub> [mm <sup>2</sup> ]				
Ψ <sub>h</sub> ,v         u <sub>v</sub> ["]         Ψ <sub>u,v</sub> 1.000         18.31         1.039           kr         N <sub>ec</sub> V <sub>ecc</sub> [kN]           n         1.500         16.50	150.0	286,875	101,250				
1.000 18.31 1.039 kr X <sub>4.6</sub> V <sub>86.5</sub> [kN] 4.0 4.6.00	¥ <sub>2</sub>	Wh.V	αν ["]	V. N	e <sub>c v</sub> [mm]	W ac.v	Ψro.v
k <sub>T</sub> γ <sub>M.c</sub> V <sub>R4.0</sub> [KN]           < 0	006.0	1.000	18.31	1.039	0.0	1.000	1.000
1 1 1 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2	V <sup>0</sup> <sub>PKe</sub> [kN]	k⊤	Y <sub>M.c</sub>	V <sub>Rd.c</sub> [kN]	V <sub>Ed</sub> [kN]		
0.1 D.1	22.922	1.0	1.500	40.496	22.236	I	

input data and maute must be checked for contormly with the existing conditions and for parability i PROFIS Engineering ( c ) 2003-2004 Hitl AG, FL-9494 Schaan Hitle is regulative Trademark of Hitl AG, Schoun

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Company: Address:	Page: Specifier:	<b>60</b>	Company: Address: Phone I Fax:		r eye. Specifier: E-Mail:	
Phone I Fax: Design: 09-2693-S.S. Cabinet Fastening point:		17/4/2024	Design: Fastening point:	09-2693-S.S. Cabinet	Date:	17/4/2024
5 Combined tension and shear	5 Combined tension and shear loads (EN 1992-4, Section 7.2.3)			Fastening meets	Fastening meets the design criteria!	
Steel failure						
B <sub>N</sub> B <sub>V</sub>	a Utilization B <sub>NV</sub> [%] Status					
	ļ					
B <sub>x</sub> + B <sub>y</sub> ≤ 1.0						
Concrete failure						
β <sub>N</sub> β <sub>V</sub> 0.523 0.549	α         Utilization β <sub>NV</sub> [%]         Status           1.500         79         0K	1				
$\beta_{N}^{\alpha}+\beta_{N}^{\alpha}\leq1.0$						
6 Displacements (highest loaded anchor)	led anchor)					
Short term loading:						
N <sub>Sk</sub> = 4.328 [kN]	8 <sub>N</sub> = 0.4556 [mm]					
V <sub>st</sub> = 2.607 [kN]	δ <sub>v</sub> = 0.4135 [mm]					
	δ <sub>NV</sub> = 0.6153 [mm]					
	8 = 0.9871 [mm]					
4	H					
	"					
Comments: Tension displacements are valid are valid without friction between the concre- included in this calculation!	Comments. Tension displacements are valid with half of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the anchor plate! The gap due to the drifted hole and dearrance hole tolerances are not included in this catculation:	t for uncracked concrete! Shear displacements not displacements not				
The acceptable anchor displacements depe	The acceptable anchor displacements depend on the fastened construction and must be defined by the designer	d by the designer!				
7 Warnings						
<ul> <li>The archor design methods in PROFIS Engineering require rigid anchor plates EOTA TR029 etc.). This means load re-distribution on the anchors due to elasti anchor plate is assumed to be sufficiently stiff, in order not loa be deformed when calculates the minimum required anchor plate thickness with CBFEM to limit the explained above. The pool if the rigid anchor plate assumption is valid is not ca must be checked for agreement with the existing conditions and for plausibility!</li> </ul>	The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 atc.). This means load re-distribution on the anchors due to elastic defarmations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently with, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with DFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!	pulations (AS 5216:2021, ETAG 001/Annex C, s of the anchor iplate are not considered - the the design loading. PROFIS Engineering anchor plate based on the assumptions ROFIS Engineering. Input data and results				
<ul> <li>Design is only valid if hole is filled to remove the concentration of the concen</li></ul>	Design is only valid if hole is filled to remove clearance, clearance as per EN 1992-4 i apie 0.1 Checkino the transfer of loads into the base material is required in accordance with EN 1992-4. Annex Al	4, Annex Al				
<ul> <li>The design is only valid if the clearance hole in the fixt of the clearance hole see section 6.2.2 of EN 1992-41</li> </ul>	The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 6.1 of EN 1992-4! For larger diameters of the clearance hole see section 6.2.2 of EN 1992-4!	Table 6.1 of EN 1992-4! For larger diameters				
<ul> <li>The accessory list in this report is for the it be followed to ensure a proper installation</li> </ul>	The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.	tions for use provided with the product have to				
- For the determination of the $\psi_{\rm rev}$ (concret cover of the edge reinforcement	For the determination of the w <sub>no</sub> , (concrete edge failure) the minimum concrete cover defined in the design settings is used as the concr cover of the edge relinforcement.	I in the design settings is used as the concrete				L.
The characteristic bond resistances depen	The characteristic bond resistances depend on the return period (service life in years): 50					58
input data and results must be checked for contormity with the extering conditions and for plaustbilly! concerned and second and an act of Lacks Achieven Hills is a motistered fraction of Hills 45, Schein	the existing conditions and for plausibility! have Hills is a reoristened Trademark of Hills AG. Schein		input date and results must be c PROFIS Engineering ( c ) 2003-	input data and results must be checked for conformly with the existing conditions and for plausbilly/ PROFIS Engineering ( c.) 2003-2024 hills AG. PL-9494 Schwan Hill is a registerea Trademark of Hill AG. Schaon	Jausibilityt nark af Hite AG. Schaan	
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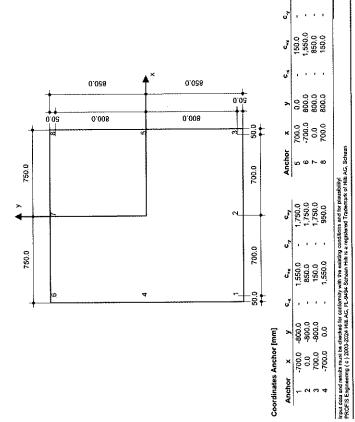
### 8 Installation data

	Charles that and dimension UST2 D M10 had
	HIGHOL MAR alte distributer up 2-14 MITA HEIT
Profile: na profile	Item number: 2105864 HST3-R M10x90 30/10
Hole diameter in the fixture: $d_r \approx -mm$	Maximum installation torque: 45 Nm
Plate thickness (input): 3.0 mm	Hole diameter in the base material: 10.0 mm
Recommended plate thickness: not calculated	Hole depth in the base material: 85.0 mm
Driling method: Hammer drilled	Minimum thickness of the base material: 100.0 mm
Cleaning: No cleaning of the drilled hole is required	

Hitli HST3 stud anchor with 60 mm embedment, M10 hef2, Stainless steel, installation per ETA 98/0001, with annular gaps filled with Hitl Filling set or any suitable gap solutions

### 8.1 Recommended accessories

Setting	<ul> <li>Torque controlled cordless impact tool</li> <li>Torque wrench</li> <li>Hammer</li> </ul>
Cleaning	- No accessory required
Drilling	<ul> <li>Suitable Rotary Hammer</li> <li>Properly sized drill bit</li> </ul>



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	Page: Specifier: E-Mailt: Date:	
	 09-2693-S.S. Cabinet	
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### 9 Drilling and installation

17/4/2024

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energie Saul Fange	TC-CD_TE-YD Second Panel Second Second Second Second
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Input data and results must be checked for conformity with the existing conditions and for plausability PROPIIS Engineering ( c ) 2005-2024 Hala AG, FL-9494 Schean Halls is a registered Trademark at Halla AG. Schaan

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Company:		Page:	10
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	09-2693-S.S. Cabinet	Date:	17/4/2024
Fastening point:			
		MAANA VIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIIII	
10 Alternative fastening	tening		

10 Alternative fastening	
10.1 Alternative fastening data	(real)
Anchor type and diameter:	HST4-R M10
Return period (service life in years):	20
item number:	2329101 HST4-R M10x90 5-40
Filling set or any suitable annular gap filling solution	ap filling solution
Effective embedment depth:	h <sub>stad</sub> = 60.0 mm (h <sub>at</sub> la <sub>mi</sub> = - mm), h <sub>non</sub> = 68.0 mm
Material:	A4 .
Evaluation Service Report:	ETA-21/0878
Issued I Valid:	28/2/2024   -
Proof:	SOFA based on EN 1992-4, Mechanical
Stand-off installation:	$e_b = 0.0 \text{ mm}$ (no stand-off); t = 3.0 mm
Anchor plate <sup>R</sup> :	$l_x x l_y x t = 1.500.0 \text{ mm} \times 1.700.0 \text{ mm} \times 3.0 \text{ mm}$ ; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, C25/30, $f_{cot}$ = 25.00 N/mm <sup>2</sup> ; h =1,000.0 mm, User-defined partial material safety factor $\gamma_c$ = 1.500
installation:	hammer drilled hole, installation condition: Dry
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm ( $\emptyset <= 10$ mm)
	with longitudinal edge reinforcement d >= 12.0 [mm] + close mesh (stirrups, hangers) s <= 100.0 [mm]
-	Max. Utilization with HST4-R M10: 69 % Fastening meets the design criterial

### 10.2 Installation data

Anchor plate, steel: EN S235; E = 205,000.00 N/mm <sup>2</sup> ; $f_{yx}$ = 235.00 N/mm <sup>2</sup>	Anchor type and diameter: HST4-R M10
Profile: no profile	Item number: 2329101 HST4-R M10x90 5-40
Hote diameter in the fixture; $d_f = -mm$	Maximum installation torque: 40 Nm
Plate thickness (input): 3.0 mm	Hole diameter in the base material: 10.0 mm
Recommended plate thickness: not calculated	Hole depth in the base material: 88.0 mm
Drilling method: Hammer drilled	Minimum thickness of the base material: 115.0
Cleaning: No cleaning of the drilled hole is required	

Hill HST4-R stud anchor with 60 mm embedment, M10, Stainless steel, installation per ETA-21/0878, with annular gaps filled with Hill Filling set or any suitable gap solutions

rum thickness of the base material: 115.0 mm

### 10.2.1 Recommended accessories

Setting	<ul> <li>Torque controlled cordiess impact tool</li> <li>Torque wrench</li> <li>Hammer</li> </ul>
Cleaning	No accessory required
Drilling	<ul> <li>Suitable Rotary Hammer</li> <li>Properly sized drill bit</li> </ul>

input data and results must be checked for conformity with the existing conditions and for plaustbility! PROFIS Engineering ( c.) 2003-2024 Hitb AG, PL-9494 Schaan Hitb is a registared Trademark of Hitle AG, Schaan

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Hilti PROFIS Engineering 3.0.93

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Company:		Page:	11
Address:		Specifier:	
Phone 1 Fax:		E-Mail:	
Design:	09-2693-S.S. Cabinet	Date:	17/4/2024
Fastening point:			

## 11 Remarks; Your Cooperation Duties

• Any and all information and data contained in the Software concern solely the use of Hilli products and are based on the principles. formulas and security regulations in accordance with Hillis technical directions and operating, mouning and assembly instructions. Are, that must be strictly complied with by the user. All figures contained therein an average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilli product. The results of the calculations arried out by meases of the Software are based by the operation of the data you use the relevant Hilli product. The results of the calculations arried out by meases of the Software are based assembly to the data you would be relevant Hilli product. The results of the calculation strind out by mease of the Software are taked as to be put in by you. Moreover, you bear soft responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear soft responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear soft responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear soft responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear soft responsibility for the absence of errors, the completeness and the relevance of the paticularity with the patient of the absence of errors, the completeness and the relevance of the results of the absence of errors, the completeness and the relevance of the absence of errors, the concertance and errors are able to be put in by you. Moreover, you bear solt responsibility for the absence of errors, the completeness and the relevance of the absence of errors the absence of errors, the completeness and the relevance of the absence of errors the absence of errors, the completeness and the relevance of the absence of errors are able

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F

JEG	Job BTS site			<b>Job No. Page</b> J8009 52693 511
CALCULATION	Calculations by	Checked by	A. Chan	Date Apr,2024
Design Loading For S	5.5.Cabinet			
qz = 1.35 kPz Cp = 2.00	a ( the building height under	2.5 m)		
Size of S.S.Cabinet	= 0.650 m(W) x 0.300	m(D) x 0.600	m (H) (	50 kg)
= 1.48	KI 1			
$W_{\text{Ley}} = qz \times Cp \times Cp = 0.69$	D.3 x 0.6 x 1.4 kN			
s/w, se = 50 / 100 = 0.70	x 1.4 kN			
Member B : C2 Member C : C3				
	650		<b>▲</b> <u>300</u>	<b>→</b>
		VIEW 'a'		
		,	W <sub>ley</sub>	
		,		
LB V	<u>'IEW 'a'</u>		<u> </u> B	

JEG	Job BTS site		Job No. Page J8009 52693 512
CALCULATION	Calculations by O	Checked by A. Chan	Date Apr,2024
Check Bolt Connectio	on for S.S.Cabinet & Steel Platfor	m	
Size of S.S.Cabinet q <sub>z</sub> = 1.35 kPa C <sub>f</sub> = 2.00	= 0.65 (W) ×	0.30 (D) x 0.60 (H	1)
(row) (cloumii) Try 2 x 2	<b>пов.</b> M8 Grade A1 - 50	$\uparrow$	W - 0.05m
Shear Capacity,P <sub>s</sub>	$= A_t \times P_{sb}$ = 36.6 × 200 / 10 <sup>3</sup> = 7.32 kN	d + 0 + 0 + 0 + + 0 + + + + + + + + + + +	+
Tension Capacity,P <sub>t</sub>	= $A_t \times P_{st}$ = 36.6 × 210 / 10 <sup>3</sup> = 7.69 kN	Total nos. of bolts = Bolt area, $A_t$ = Shear strength of bolt, $p_{sb}$ =	+ 4 nos. 36.6 mm² 200 N/mm²
For wind in x direction		Tension strength of bolt, p <sub>tb</sub> =	210 N/mm <sup>2</sup>
Shear per bolt, $F_{\mathfrak{s}}$	$= q_z \times C_f \times D \times H \times 1.4 / 4 \text{ nos.}$ = 1.35 x 2 x 0.3 x 0.6 x 1.4 / 4 nos. = 0.17 kN < P_s	OK1	
Tension per bolt,F <sub>t</sub>	$= q_z \times C_f \times D \times H^2 / 2 / (W - 0.05)$ = 1.35 × 2 × 0.3 × 0.6 <sup>2</sup> / 2 / (0.69 = 0.17 kN < P_t		
Check Combine Effect			
$F_s/P_s + F_t/P_{nom}$	= 0.17 / 7.32 + 0.17 / 7.69	= 0.05 < 1.4	OK!
For wind in y direction Shear per bolt, F <sub>e</sub>	= q <sub>z</sub> x C <sub>f</sub> x W x H x 1.4 / 4 nos. = 1.35 x 2 x 0.65 x 0.6 x 1.4 / 4no		
Tension per bolt,F <sub>t</sub>	$= 0.37 \text{ kN} < P_{\phi}$ = $q_z \times C_f \times W \times H^2 / 2 / (D - 0.05)$ = $1.35 \times 2 \times 0.65 \times 0.6^2 / 2 / (0.5)$	0K! x 1.4 / 2nos. 3 - 0.05 ) x 1.4 / 2 nos.	
Check Combine Effect F <sub>s</sub> /P <sub>s</sub> + F <sub>t</sub> /P <sub>nom</sub>	$= 0.88 \text{ kN} < P_t$ $= 0.37 / 7.32 + 0.88 / 7.69$	okj = 0.17 < 1.4	OKI

Adopt 4nos, M8 Stainless Steel Bolts

JEG	Job BTS site		Job No. Page J8009 52693 513
CALCULATION	Calculations by O	Checked by A. Chan	Date Apr,2024
Check Member C1	(L = 650 mm)		↓ v <sub>v</sub>
Try EA 50x50x5+		_C	650
Compression on C1,	= WLey / 2 nos = 0.69 / 2 nos = 0.35 kN	c mr	V <sub>x</sub> c
Max Shear on C1,	= $[(WLex /2)^2 + (se /2)^2 + (0.7 /2)^2 + $	) <sup>2</sup> ] <sup>0.5</sup> ) <sup>2</sup> ] <sup>0.5</sup>	
Moment, M <sub>x</sub>	= se / 2 x L / 4 = 0.7 /2 x 0.65 /4 = 0.06 kNm		
Moment, M <sub>y</sub>	= WLex / 2 x L / 4 = 1.48 / 2 x 0.65 / 4 = 0.13 kNm		

For member checking, please refer to next page.

Adpot EA 50x50x5+

JE	G	Job Title						Job No.	Sheet No. 514	Rev.
CALCUL		Design by			Checked by	M.K. V	/ong	Date:		
Member C1		EA 50x50x5+	H	IKSC:2011	I APPROACH	1		L		
Member Forces	F <sub>i</sub> V M <sub>x</sub> M <sub>y</sub>	Axial Compression Axial Tension Shear Force = Major Bending Minor Bending	$\int \frac{1}{V_{x}^{2} + V_{y}^{2}} = \frac{1}{2}$	0.35 0.00 0.82 0.06 0.13	kN kN kN kNm kNm	Load Case 1. Shear Capacity 2. Bending Capacity 3. LTB 4.Compression Cap 5.Tensile Capacity 6. Combined Axial &	acity	0.021 0.160 - 0.004 0.000 0.203	0.K. 0.K. 0.K. 0.K. 0.K. 0.K.	
Section Properti Miller Provide endered Special de estas Mentes	at Alinderstands	enter des constructions a destant	e (tiplet fer polyand)	baren parte						
Dimensions Depth, D Width, B Flange Thk, T Web Thk, t depth, d		mm 50 50 5.0 5.0 50.0	d/t =	10	Length L. L <sub>ex</sub> L <sub>ey</sub>	= = 1.0 L = 1.0 L	<b>mm</b> 1000 1000 1000	max		
width, b Steel	Grade $p_y =$ $p_{yyv} =$ $\epsilon = (275/p_y)^{1/2}$ E = Area $A_g =$	50.0 5275 ▼ 275 275 1.00 205000 491	b/T = N/mm <sup>2</sup> e web = N/mm <sup>2</sup> mm <sup>2</sup>	10 1.00	Radius of Gyra	ltion r <sub>x</sub> = r <sub>y</sub> =	14.8 14.8	mm mm	; Cl. 3.1.2 Table	3.2
Second Moment Section Modulus	l <sub>x</sub> = l <sub>y</sub> =	1.070E+05 1.070E+05	mm⁴ nım⁴							
Elastic:	$Z_x = Z_y =$	2.950E+03 2.950E+03	mm³ mmĭ		Plastic:	S <sub>x</sub> = S <sub>y</sub> =	5.580E+03 5.580E+03	mm³ mmĩ		
Limiting Width-to	o-Thickness R	atios			. <u> </u>			T	; Cl. Table 7.1 a	ind 7.2
	Compression Design Type	Element Compression due to Ben	ding 🍸		Class 1 Plastic	Limiting Value Class 2 Compact	Class 3 Semi-Compact	Ratio	Classification	
	Flange	Compression due to	Bending		8e 8.00	9e 9.00	13e 13.00	b/T 10.00	Semi-Compact	
	Web	Neutral Axis at Mid-	Depth		80e 80.00	100e 100.00	120e 120.00	d/t 10.00	Plastic	
	Web	Generally	(>= 40€ = 40	))	80e/ (1+r1) 79.59	100e / (1+1.5r1) 79.39	120e/ (1+2r2) 119.38	d/t 10.00	Plastic	
=	Classification Fc / d t pyw 0.01 0.01	; -1 < r1 <≖ 1	r <sub>2</sub> = F <sub>c</sub> = <u>0.</u>						; Cl. 7.3 (a) & (c	)
<u>Shear Capacity</u> V <sub>c</sub> =	$P_{y}A_{y}/\sqrt{3}$ Av =		1 mm²						; Cl. 8.2.1	
.∵. V <sub>e</sub> =		kN	Low Shear Condition						> V, OK	
Moment Capacity	l		-					r	; Cl. 7.5.2	
S <sub>x,eff</sub> =	$Z_x + (S_x - Z$	$\int_{a} \left[ \frac{\left(\frac{\beta_{3w}}{d/t}\right) - 1}{\left(\frac{\beta_{3w}}{\beta_{2w}}\right) - 1} \right] \le 2$	$Z_x + (S_x - Z_x) \left[ \frac{1}{2} \right]$	$\frac{\frac{\beta_{M}}{b/T} - 1}{\frac{\beta_{M}}{\beta_{M}} - 1}$		S <sub>y,eff</sub> =	$Z_y + (S_y - Z_y)$	$\left  \frac{\left(\frac{\beta_{M}}{B/T}\right)}{\left(\frac{\beta_{M}}{\beta_{2f}}\right)} - \frac{\beta_{M}}{\beta_{2f}} \right  = \frac{\beta_{M}}{\beta_{2f}}$	1 	
2	26623 26623		34510 m	m <sup>3</sup>		=	34510	mm <sup>3</sup>		
Sv =	2083	mm <sup>3</sup>	$\rho = \left(\frac{2}{2}\right)$	$\frac{2V}{V_c} - 1 \bigg)^2 =$	0.0017					

JEG	Job Title					Jo	ob No. S	Sheet No. S15	Rev.
CALCULATION	Design by			Checked by	M.K. Wong	Di	ate:	]	
1ember C1	EA 50x50x5+		HKSC:2011	APPROACH					
M <sub>c</sub> = p <sub>y</sub> x Z (Seff)					≤ 1.2.py(Zx -p.Sv /1.			Cl. 8.2.2	
$M_{cx} = p_y x \min(Zx, Sx, eff)$	) =	0.81	kNm	.:. M <sub>cx</sub> =	≤ 1.2.py(Zy -p.Sv /1.3 .81 kNm	5) / 1000 = .9		> Mx, OK	
$M_{cy} = p_y \times min(Zy, Sy, eff)$	) =	0.81	kNm 1		.81 kNm		3	> Му, ОК	
$\frac{\text{foment Capacity to Lateral }}{M_{\mu} \geq m_{\text{LT}} M_{x}}$								; Cl. 8.3.5.2	
	= 1.0 = pb Zx or pb Seff							; Table 8.4a ;(8.20 - 8.24)	
$\lambda_{LT}$	= $uv\lambda\sqrt{\beta_w}$						i	; Cl. 8.3.5.3	
λ: ×	= 0.9 = $L_E / r_y = L_{ey} / r_y =$ = $D / T =$ = $\frac{1}{(1 + 0.05 (\lambda / x)^2)}$	$\frac{67.57}{10.00}$	Hot-rolled Section	•					
<i>√</i> β , =			Semi-Compac	t Sections				2.00	
$ \begin{array}{ccc} \vdots & \lambda_{\text{LT}} \\ \vdots & p_{\text{b}} \\ P_{\text{cy}} \ge A_g p_c \end{array} $		N/mm²	Rolled Section	•	-		;	; 8.3.5.2 App. 8.	.1
∴ M <sub>b</sub> = 1.53	kNm						:	> mLT Mx, OK	
Compression Resistance L <sub>ex</sub> = L = 1000 L <sub>ey</sub> = L = 1000	mm mm	۶ <sub>X</sub> : ۲ <sub>y</sub>		mm $\lambda_x = L_{ex}/r_x$ mm $\lambda_y = L_{ey}/r_y$	, =	67.6 67.6	Rolled H	1-Section	
p <sub>ex</sub> p <sub>cy</sub>		N/mm² N/mm²		$P_{cx} = p_{cx} \cdot A = P_{cy} = p_{cy} \cdot A =$	101.47 kN 91.36 kN			; Cl. 8.7.5 ; Table 8.7, App	o. 8.4
.:. P₀ = 91.36	kN							> Fc, OK	
Compression Members unde	r Combined Axial	Force and Mome	<u>ents</u>						
Cross-section Capacity F <sub>c</sub> / Ag P <sub>y</sub> = 0.00 = 0.24	+ M <sub>x</sub> / M <sub>cx</sub> + 0.074	+ M <sub>y</sub> / M <sub>cy</sub> + 0.160	≤I					; Cl. 8.9.1	
< 1.00	~							< 1, OK	
M <sub>cy</sub>	= 1.0 = p <sub>y</sub> Z <sub>y</sub> = 0.81	kNm	<b>.</b>					; Table 8.9 ; Cl. 8.9.2	
F <sub>o</sub> /P <sub>oy</sub> = 0.00			≤ 1					; Cl. 8.9.2	
= 0.20 < 1.00								< 1, OK	
<u>Fension Resistance</u> Ke = 1.0 Ae = K <sub>e</sub> a <sub>n</sub> = 491	≤ a <sub>g</sub> mm²		Hole Area =	0	mm²				
∴ P <sub>t</sub> = py . Ae = 135.03	kN		a1 = a2 =		mm² mm²			> Ft, OK	
oncion Mombara under A	nhinad Avial Earce	and Momente							
ension Members under Cor	numeu Axial Force	and woments							
Ft/Pt = 0.00	+ M <sub>x</sub> / M <sub>cx</sub> + 0.074	+ M <sub>y</sub> /M <sub>cy</sub> + 0.160	≤∣					; Cl. 8.8	

JEG	Job BTS site		Job No. Page J8009 52693 516
CALCULATION	Calculations by O	Checked by A. Chan	Date Apr,2024
Check Member C2	(L = 300 mm)	<b> </b>	300
Try EA 50x50x5+	· ····································	°₅→	V <sub>y</sub> ▼ B
Compression on C2,	= WLex / 2 nos = 1.48 / 2 nos = 0.74 kN	C m	V <sub>x</sub> C
Max Shear on C2,	$= [( 9e / 2 )^{2} + ( WLey / 2)]$ = [( 0.7 / 2 ) <sup>2</sup> + ( 0.69 / 2)] = 0.5 kN	) <sup>2</sup> ] <sup>05</sup> ) <sup>2</sup> ] <sup>05</sup>	
Moment, M <sub>x</sub>	= se/2×L/4 = 0.7/2×0.3/4 = 0.03 kNm		
Moment, M <sub>y</sub>	= WLey / 2 x L / 4 = 0.69 /2 x 0.3 /4 = 0.03 kNm		

For member checking, please refer to next page.

Adpot EA 50x50x5+

		Job Title				*******		Job No.	Sheet No.	Rev.
JE	G								517	
CALCUL	ATION	Design by	······		Checked by	M.K. W	/ong	Date:		
Member C2		EA 50x50x5+	ŀ	IKSC:201	1 APPROACI	1				
Member Forces	F	Axial Compression	=	0.74	kN	Load Case 1. Shear Capacity	1	0.013	0.K.	7
Member roices		Axial Tension	=	0.00	kN	2. Bending Capacity		0.037	0.K.	
		' Shear Force =	$\sqrt{V_{x}^{2} + V_{x}^{2}} =$	0.50	kN	3. LTB		-	О.К.	
		Major Bending	=	0.03 0.03	kNm kNm	4.Compression Capa 5.Tensile Capacity	acity	0.008	О.К. О.К.	
	<sup>174</sup> )	, Minor Bending	-	0.03	NINIG	6. Combined Axial &	Bending	0.065	0.к.	
	et v Spolenner S	and an Andrea and Pricesa An an	er Sharar tel art	Served a D						
lan e a Lanout na bhuil lui	r Docelor di Sone	lege (Section 4 16)								
Dimensions Depth, D		mm 50			Length L	=	<b>mm</b> 1000	max		
Vidth, B		50			њ L <sub>ex</sub>	= 1.0 L	1000	, , , , , , , , , , , , , , , , , , ,		
Flange Thk, T		5.0			Lay	= 1.0 L	1000			
Web Thk, t		5.0								
lepth, d		50.0 50.0	d/t = b/T =	10 10						
width, b		50.0	0/1 -	10						
Steel	Grade	s275 <b>•</b>	····· • · · · ·		Radius of Gyr	ation			; Cl. 3.1.2 Tab	ie 3.2
	p <sub>y</sub> =	275	N/mm <sup>2</sup>			r <sub>x</sub> =	14.8	mm		
	$p_{yw} =$	275	N/mm <sup>2</sup>	1.00		r <sub>y</sub> =	14.8	mm		
	ε =(275/p <sub>y</sub> ) <sup>1/2</sup> Ε =	1.00 205000	e web = N/mm²	1.00						
	t:= Area A <sub>s</sub> =	491	nm²							
	•									
Second Moment	of Area I <sub>x</sub> =	1.070E+05	mmª							
	י <sub>×</sub> == I <sub>v</sub> =	1.070E+05	mm*							
	,									
Section Modulus Elastic:	s Z <sub>x</sub> =	2.950E+03	mm³		Plastic:	S <sub>x</sub> =	5.580E+03	ന്ന		
	$Z_{y} =$	2.950E+03	ന്ന്		1 14410.	S <sub>y</sub> =	5.580E+03	mm		
	o-Thickness	Ratios							; Cl. Table 7.1	and 7.2
	Compression	n Eiement				Limiting Value		Ratio	Classification	ן ק
	Design Type	Compression due to Be	nding 🔻		Class 1	Class 2	Class 3			
					Plastic	Compact	Semi-Compaci		Sami Comag	
	Flange	Compression due t	o Bending		8e 8.00	9e 9.00	13e 13.00	b/T 10.00	Semi-Compac	20
	Web	Neutral Axis at Mid	-Depth		80e	100e	120e	d/t	Plastic	
	Web	Generally	(>= 40∈ = 4	0.)	80.00 80e/ (1+r1)	100.00 100e / (1+1.5r1)	120.00 120e/ (1+2r2)	10.00 d/t	Plastic	-
	1100	Generally	(* 100 (		79.15	78.73	118.70	10.00		- 1.4
Stress Ratios for		; -1 < r1 <= 1	ro = F	c / A <sub>a</sub> p <sub>yw</sub>					; Cl. 7.3 (a) &	(c)
	0.01		= 0							
	0.01								:	
Shear Capacity	n /5								; Cl. 8.2.1	
V <sub>c</sub> ≖	$P_y A_y / \sqrt{3}$ AV :	= ID	1							
		= <u>250</u>	mm²							$\theta = \frac{1}{2} - \frac{1}{2}$
.:, V <sub>c</sub> =	39.69	кN	Low Shear Condition						> V, OK	
Moment Capacit	¥								; Cl. 7.5.2	
		$\left[\left( \rho \right) \right]$	F	$(\beta_{1}, \gamma)$				$\left[ \left( \beta_{3T} \right) \right]$	,]	
S <sub>x,eff</sub> =	$Z_x + (S_x -$	$Z_{x}\left \frac{\left(\frac{\beta_{3w}}{d/t}\right)-1}{\left(\frac{\beta_{3w}}{\beta_{2w}}\right)-1}\right  \leq$	$Z_x + (S_x - Z_x) \bigg $	$\left(\frac{\beta_{3f}}{\beta_{2f}}\right) - 1$ $\left(\frac{\beta_{3f}}{\beta_{2f}}\right) - 1$		S <sub>y,off</sub> =	$z_y + (S_y - Z_y)$	$\int_{T} \left( \frac{\left( \frac{\beta_{3f}}{B/T} \right)^{-1}}{\left( \frac{\beta_{3f}}{\beta_{2f}} \right)^{-1}} \right)$	1	
=		tmm³ ≤ tmm <sup>3</sup>	34510 n	nm³			= 3451(	0 mm <sup>3</sup>		
Sv =	208	3 mm <sup>3</sup>	ρ = (	$\left(\frac{2V}{V_c} - 1\right)^2 =$	= 0.0006					

JEG	Job Title					Job No.	Sheet No. Re S18	ev.
CALCULATION	Design by			Checked by	M.K. Wong	Date:		_
Aember C2	EA 90x90x12		HKSC:2011	APPROACH			_	
$M_c = p_y \times Z$ (Seff)				5			; Cl. 8.2.2	
$M_{cx} = p_y \times \min(Zx, Sx, eff)$	=	0.81	kNm	≦ ∴ M <sub>cx</sub> =	1.2.py(Zy -p.Sv /1.5) / 1000 - .81 kNm	= .97 knm	> Mx, OK	
$M_{cy} = p_y \times min(Zy, Sy, eff)$	-	0.81	kNm 1	.:. M <sub>cy</sub> =	.81 kNm		> My, OK	
Moment Capacity to Lateral To $M_{\mu} \geq m_{LT}M_{x}$							; Cl. 8.3.5.2 ; Table 8.4a	
	pb Zx or pb Seff						;(8.20 - 8.24)	
$\lambda_{LT} =$	$\mu\nu\lambda\sqrt{\beta_w}$						; Cl. 8.3.5.3	
λ = x =	$\frac{40.9}{L_{E} / r_{y} = L_{ey} / r_{y} =}{\frac{1}{(1 + 0.05 (\lambda / x)^{2})^{2}}}$	67.57 10.00	Hot-rolled Section	n <b>T</b> .				
$\sqrt{\beta}$ , =			Semi-Compac	t Sections			2.00	
$  \therefore  \lambda_{LT} = \\  p_b = \\ P > d p $		N/mm <sup>2</sup>	Rolled Section				; 8.3.5.2 App. 8.1	
$P_{cy} \ge A_{q} p_{c}^{p_{0}}$ $\therefore \qquad M_{b} = 1.53$	kNm						> mLT Mx, OK	
<u>Compression Resistance</u> L <sub>ex</sub> = L = 1000 L <sub>ey</sub> = L = 1000	mm mm	r <sub>x</sub> = r <sub>y</sub> =		mm $\lambda_x = L_{ex}/r_x = mm$ $\lambda_y = L_{ey}/r_y = L_{ey}/r_y = L_{ey}/r_y$		Ro	fled H-Section	
p <sub>cx</sub> = p <sub>cy</sub> =		N/mm² N/mm²		$P_{cx} = p_{cx} \cdot A =$ $P_{cy} = p_{cy} \cdot A =$	101.47 kN 91.36 kN		; Cl. 8.7.5 ; Table 8.7, App. 8	8.4
∴ P <sub>c</sub> = 91.36	kN						> Fc, OK	
Compression Members under	Combined Axial	Force and Mome	ints				·	
Cross-section Capacity F <sub>c</sub> / Ag P <sub>y</sub> = 0.01	+ M <sub>x</sub> / M <sub>cx</sub> + 0.037	+ M <sub>y</sub> / M <sub>cy</sub> + 0.037	≤l				; Cl. 8.9.1	
= 0.08 < 1.00							< 1, OK	
M <sub>cy</sub> =	= 1.0 = pyZy = 0.81	kNm					; Table 8.9 ; Cl. 8.9.2	
	+ m <sub>LT</sub> M <sub>x</sub> / M <sub>b</sub> + 0.02 - ·		≤1				; Ci. 8.9.2	
= 0.06 < 1.00							< 1, OK	
<u>Tension Resistance</u> Ke = 1.0 Ae = K <sub>e</sub> a <sub>n</sub> = 491	≤ a <sub>g</sub> mm²		Hole Area =	0 n	nm <sup>z</sup>			
∴ P <sub>i</sub> = py . Ae = 135.03	kN		a1 = a2 =		nm² nm²		> Ft, OK	
Tension Members under Corr	bined Axial Force	e and Moments						
Ft/Pt = 0.00	+ M <sub>x</sub> / M <sub>cx</sub> + 0.037	+ M <sub>y</sub> /M <sub>cy</sub> + 0.037	≤l				; Cl. 8.8	
= 0.07	1 0.007	1 01001					< 1, OK	

-

JEG	Job BTS site		i nu z kalegori i s	Job No. Page J8009 52693 A 9
CALCULATION	Calculations by O	Checked by	A. Chan	Date Apr,2024
Check Member C3	(L= 800 mm)			
Try EA 50x50x5+	♥.		V.	C <sub>c</sub>
Compression on C3,	= se/4nos = 0.18 kN		V <sub>y</sub>	800
Max Shear on C3,	= [( WLex / 4 )2 + (= [( 1.48 / 4 )2 + ( 0= 0.41 kN	WLey / 4 ) <sup>2</sup> ] <sup>0.5</sup> 9.69 / 4 ) <sup>2</sup> ] <sup>0.5</sup>	C 7	
Moment, M <sub>x</sub>	= WLex / 4 x (H / 2 + L) = 1.48 / 4 x (0.6 / 2 + 0.8) = 0.41 kNm			
Moment, M <sub>y</sub>	= WLey / 4 x (H / 2 + L) = 0.69 / 4 x (0.6 / 2 + 0.8) = 0.19 kNm			

For member checking, please refer to next page.

Adpot EA 50x50x5+

JE		Job Title						Job No.	520
		Design by			Checked by			Date:	760
CALCUL	ATION	Design by			Showing by	M.K. We	ong		
ember C3		EA 50x50x5+		HKSC:201	1 APPROACH				
ember Forces	F	Axial Compression	=	0.18	kN	Load Case 1. Shear Capacity	1	0.010	О.К.
	-	Axial Tension	=	0.00	kN	2. Bending Capacity		0.505	0.К.
		Shear Force =	$\sqrt{V_{y}^{(2)} + V_{y}^{(2)}} =$	0.41	kN	3. LTB	oitu	- 0.002	0.K. 0.K.
		Major Bending	=	0.41 0.19	kNm kNm	4.Compression Capa 5.Tensile Capacity	city	0.002	0.K.
	м <sub>у</sub>	Minor Bending	-	0.19	ROUT	6. Combined Axial &	Bending	0.503	О.К.
	n Agerano e	计工作进行 中国经济中国	oo kaalaa too oo	E dona of note					
३३४ केल्ल व संघाष्ट्रम -	O congres pi and				Longth		mm		
mensions		mm 50			Length L	=	1000	max	
epth, D idth, B		50			L <sub>ex</sub>	= 1.0 L	1000		
ange Thk, T		5.0			L <sub>ey</sub>	= 1.0 L	1000		
eb Thk, t		5.0							
epth, d		50.0	d/t =	10					
dth, b		50.0	b/T =	10					
eel	Grade	\$275 🛡			Radius of Gyra	tion			; Cl. 3.1.2 Table 3
	p <sub>y</sub> =	275	N/mm²			r <sub>x</sub> =	14.8	mm	
	p <sub>yw</sub> =	275	N/mm*			r <sub>y</sub> =	14.8	mm	
	$\epsilon = (275/p_y)^{1/2}$	1.00	€ web =	1.00					
	E≖	205000	N/mm <sup>2</sup>						
	Area A <sub>g</sub> =	491	ការាះ						
econd Moment	of Area								
	I <sub>x</sub> =	1.070E+05	mm⁴						
	l <sub>y</sub> =	1.070E+05	mm*						
ection Modulus lastic:	Z <sub>x</sub> =	2.950E+03	mm³		Plastic:	S <sub>x</sub> =	5.580E+03	mm	
	 Ζ <sub>γ</sub> =	2.950E+03	៣៣			S <sub>y</sub> =	5.580E+03	നന്	
miting Width-to	o-Thickness	Ratios							; Cl. Table 7.1 an
miting Width-to	·					imiting Value		Ratio	· ·
miting Width-t	Compression	n <u>Element</u>	ordina 🔻			Limiting Value	Class 2	Ratio	; Cl. Table 7.1 an Classification
imiting Width-ti	·		nding 🔻		Class 1 Plastic	Limiting Value Class 2 Compact	Class 3 Semi-Compact		· ·
imiting Width-to	Compression Design Type	n Element Compression due to 8e	· · · · · · · · · · · · · · · · · · ·			Class 2			· ·
miting Width-to	Compression	n <u>Element</u>	· · · · · · · · · · · · · · · · · · ·		Plastic 8e 8.00	Class 2 Compact 9e 9.00	Semi-Compact 13e 13.00	t b/T 10.00	Classification Semi-Compact
miting Width-t	Compression Design Type	n Element Compression due to 8e	to Bending		Plastic 8e 8.00 80e	Class 2 Compact 9e 9.00 100e	Semi-Compact 13e 13.00 120e	t b/T 10.00 d/t	Classification
miting Width-t	Compression Design Type Flange Web	n Element Compression due to Be Compression due Neutral Axis at Mic	to Bending	40 )	Plastic 8e 8.00	Class 2 Compact 9e 9.00	Semi-Compact 13e 13.00	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact
imiting Width-t	Compression Design Type Flange	n Element Compression due to Be Compression due I	to Bending J-Depth	40 )	Plastic 8e 8.00 80e 80.00	Class 2 Compact 9e 9.00 100e 100.00	Semi-Compact 13e 13.00 120e 120.00	t b/T 10.00 d/t 10.00	Classification Semi-Compact Plastic
Stress Ratios for	Compression Design Type Flange Web Web Classification	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally	to Bending J-Depth (>= 40ε =		Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic
tress Ratios for r1 =	Compression Design Type Flange Web Web Classification	n Element Compression due to 8e Compression due I Neutral Axis at Mic Generally	to Bending 3-Depth $(>= 40\epsilon = r_2 =$	40 ) F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic Plastic
Stress Ratios for r1 =	Compression Design Type Flange Web Web Classification Fc / d t pyw	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally	to Bending 3-Depth $(>= 40\epsilon = r_2 =$	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub>	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic Plastic
Stress Ratios for $r_t =$	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally	to Bending 3-Depth $(>= 40\epsilon = r_2 =$	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub>	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic Plastic
Stress Ratios for r₁ = ∴ = Shear Capacity	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ =	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic Plastic
tress Ratios for r₁ = ∴ =	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_{v}A_{v}/\sqrt{3}$	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally	to Bending 3-Depth (>= 40ε = r <sub>2</sub> = =	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c)
tress Ratios for r₁ = ∴ =	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_r / \sqrt{3}$ Av	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally ; -1 < r1 <= 1	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ =	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c)
tress Ratios for r <sub>1</sub> = :	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_c / \sqrt{3}$ Av	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = 250	to Bending 3-Depth $(>= 40\varepsilon =$ $r_2 =$ = 1 mm <sup>2</sup>	F <sub>c</sub> / A <sub>e</sub> p <sub>yw</sub> 0.00	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c)
tress Ratios for r₁ = ∴ = hear Capacity	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_c / \sqrt{3}$ Av	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally ; -1 < r1 <= 1	to Bending 3-Depth (>= 40ε = r <sub>2</sub> = =	F <sub>c</sub> / A <sub>e</sub> p <sub>yw</sub> 0.00	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1
itress Ratios for $r_1 =$ $\therefore$ = Shear Capacity $V_c =$	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_yA_r/\sqrt{3}$ Av <b>39.69</b>	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = 250	to Bending 3-Depth $(>= 40\varepsilon =$ $r_2 =$ = 1 mm <sup>2</sup>	F <sub>c</sub> / A <sub>e</sub> p <sub>yw</sub> 0.00	Plastic 8e 8.00 80e 80.00 80e/ (1+r1)	Class 2 Compact 9e 9.00 100e 100.00 100e/(1+1.5r1)	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t	Classification Semi-Compact Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1
Stress Ratios for r₁ = ∴ = Shear Capacity V₀ = ∴ V₀ = Moment Capacit	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_c / \sqrt{3}$ Av <b>39.69</b>	n Element Compression due to Be Compression due to Be Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = 250 kN	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ = 1 mm <sup>2</sup> Low Shear Condition	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/(1+r1) 79.79	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2/2) 119.68	t b/T 10.00 d/t 10.00 d/t 10.00	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2
∴ = <u>Shear Capacity</u> V <sub>c</sub> = ∴ V <sub>c</sub> = <u>Moment Capacit</u>	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_c / \sqrt{3}$ Av <b>39.69</b>	n Element Compression due to Be Compression due to Be Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = 250 kN	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ = 1 mm <sup>2</sup> Low Shear Condition	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/(1+r1) 79.79	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2/2) 119.68	t b/T 10.00 d/t 10.00 d/t 10.00	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2
Stress Ratios for r₁ = ∴ = Shear Capacity V₀ = ∴ V₀ = Moment Capacit	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_c / \sqrt{3}$ Av <b>39.69</b>	n Element Compression due to Be Compression due to Be Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = 250 kN	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ = 1 mm <sup>2</sup> Low Shear Condition	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/(1+r1) 79.79	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2/2) 119.68	t b/T 10.00 d/t 10.00 d/t 10.00	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2
Stress Ratios for r₁ = ∴ = Shear Capacity V₀ = ∴ V₀ = Moment Capacit	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_c / \sqrt{3}$ Av <b>39.69</b>	n Element Compression due to Be Compression due to Be Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = 250 kN	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ = 1 mm <sup>2</sup> Low Shear Condition	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/(1+r1) 79.79	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2/2) 119.68	t b/T 10.00 d/t 10.00 d/t 10.00	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2
Stress Ratios for r₁ = ∴ = Shear Capacity V <sub>c</sub> = ∴ V <sub>c</sub> =	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_c / \sqrt{3}$ Av <b>39.69</b>	n Element Compression due to Be Compression due to Be Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = 250 kN	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ = 1 mm <sup>2</sup> Low Shear Condition	F <sub>c</sub> / A <sub>g</sub> p <sub>yw</sub> <u>0.00</u>	Plastic 8e 8.00 80e 80.00 80e/(1+r1) 79.79	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69	Semi-Compact 13e 13.00 120e 120.00 120e/ (1+2r2)	t b/T 10.00 d/t 10.00 d/t 10.00	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2
Stress Ratios for r₁ = ∴ = Shear Capacity V <sub>c</sub> = ∴ V <sub>c</sub> =	Compression Design Type Flange Web Web Classification Fc / d t pyw 0.00 0.00 $P_y A_c / \sqrt{3}$ Av <b>39.69</b>	n Element Compression due to Be Compression due I Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = 250	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ $r_2 =$ 1 mm <sup>2</sup> Low Shear Condition $\leq Z_x + (S_x - Z_x)$	$F_{c} / A_{g} P_{yw}$ $0.00$ $\int \left[ \left( \frac{\beta_{M}}{b/T} \right) - 1 \right] \left[ \left( \frac{\beta_{M}}{\beta_{2f}} \right) - 1 \right]$	Plastic 8e 8.00 80e 80.00 80e/(1+r1) 79.79	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69 S <sub>y.eff</sub> =	Semi-Compact 13e 13.00 120e 120e/(1+2r2) 119.68 = Z <sub>y</sub> + (S <sub>y</sub> - Z	$\int_{y}^{y} \left[ \frac{\beta_{3f}}{\beta_{2f}} \right]$	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2
Stress Ratios for r₁ = ∴ = Shear Capacity V <sub>c</sub> = ∴ V <sub>c</sub> =	Compression Design Type Flange Web Classification Fc / d t pyw 0.00 0.00 $P_y A_v / \sqrt{3}$ Av <b>39.69</b> X $Z_x + (S_x - 2671)$	n Element Compression due to Be Compression due to Be Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = $\frac{250}{kN}$ kN $Z_{r} > \left[ \frac{\left(\frac{\beta_{3w}}{d \ l \ l} - 1\right)}{\left(\frac{\beta_{3w}}{\beta_{2w}} - 1\right)} \right] \le$	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ = 1 mm <sup>2</sup> Low Shear Condition	$F_{c} / A_{g} P_{yw}$ $0.00$ $\int \left[ \left( \frac{\beta_{M}}{b/T} \right) - 1 \right] \left[ \left( \frac{\beta_{M}}{\beta_{2f}} \right) - 1 \right]$	Plastic 8e 8.00 80e 80.00 80e/(1+r1) 79.79	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69 S <sub>y.eff</sub> =	Semi-Compact 13e 13.00 120e 120e/(1+2r2) 119.68 = Z <sub>y</sub> + (S <sub>y</sub> - Z	t b/T 10.00 d/t 10.00 d/t 10.00	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2
Stress Ratios for $r_1 =$ $r_1 =$ $r_2 =$ $r_1 =$ $r_2 =$ $r_1 =$ $r_2 =$	Compression Design Type Flange Web Classification Fc / d t pyw 0.00 0.00 $P_y A_v / \sqrt{3}$ Av <b>39.69</b> X $Z_x + (S_x - 2671)$	The Element Compression due to Be Compression due to Be Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = $\frac{250}{kN}$ kN $Z_{v} > \left[ \frac{\left(\frac{\beta_{3w}}{d t}\right) - 1}{\left(\frac{\beta_{3w}}{\beta_{2w}}\right) - 1} \right] \le 1$	to Bending 3-Depth $(>= 40\epsilon =$ $r_2 =$ $r_2 =$ 1 mm <sup>2</sup> Low Shear Condition $\leq Z_x + (S_x - Z_x)$	$F_{c} / A_{g} P_{yw}$ $0.00$ $\int \left[ \left( \frac{\beta_{M}}{b/T} \right) - 1 \right] \left[ \left( \frac{\beta_{M}}{\beta_{2f}} \right) - 1 \right]$	Plastic 8e 8.00 80e 80.00 80e/(1+r1) 79.79	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69 S <sub>y.eff</sub> =	Semi-Compact 13e 13.00 120e 120e/(1+2r2) 119.68 = Z <sub>y</sub> + (S <sub>y</sub> - Z	$\int_{y}^{y} \left[ \frac{\beta_{3f}}{\beta_{2f}} \right]$	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2
itress Ratios for $r_t =$ $\therefore$ = $\frac{1}{2}$ $r_t =$ $\therefore$ = $\frac{1}{2}$ $r_t =$ $r_t =$	Compression Design Type Flange Web Classification Fc / d t pyw 0.00 0.00 $P_y A_v / \sqrt{3}$ Av <b>39.69</b> X $Z_x + (S_x - 2671)$	n Element Compression due to Be Compression due to Be Neutral Axis at Mic Generally ; -1 < r1 <= 1 = tD = $\frac{250}{kN}$ kN $Z_{r} > \left[ \frac{\left(\frac{\beta_{3w}}{d \ l \ l} - 1\right)}{\left(\frac{\beta_{3w}}{\beta_{2w}} - 1\right)} \right] \le$	to Bending 3-Depth $(>= 40\varepsilon =$ $r_2 =$ $r_2 =$ 1 mm <sup>2</sup> Low Shear Condition $\leq Z_x + (S_x - Z_x)$ $\leq 34510$	$F_{c} / A_{g} P_{yw}$ $0.00$ $\int \left[ \left( \frac{\beta_{M}}{b/T} \right) - 1 \right] \left[ \left( \frac{\beta_{M}}{\beta_{2f}} \right) - 1 \right]$	Plastic 8e 8.00 80e 80e 80e/(1+r1) 79.79 1	Class 2 Compact 9e 9.00 100e 100.00 100e / (1+1.5r1) 79.69 S <sub>y.eff</sub> =	Semi-Compact 13e 13.00 120e 120e/(1+2r2) 119.68 = Z <sub>y</sub> + (S <sub>y</sub> - Z	$\int_{y}^{y} \left[ \frac{\beta_{3f}}{\beta_{2f}} \right]$	Classification Semi-Compact Plastic Plastic ; Cl. 7.3 (a) & (c) ; Cl. 8.2.1 > V, OK ; Cl. 7.5.2

JEG	Job Title					·	Job No.	Sheet No.	Rev.
CALCULATION	Design by			Checked by	M.K. Wong		Date:	L	
leniber C3	EA 50x50x5+		HKSC:2011	APPROACH					
M <sub>c</sub> ≕ p <sub>y</sub> x Z (Seff)					≤ 1.2.py(Zx -p.Sv /1.			; Cl. 8.2.2	
$M_{cx} = \rho_y \times \min(Zx, Sx, eff)$	=	0.81	kNm	.∴. M <sub>cx</sub> =	≤ 1.2.py(Zy -p.Sv /1. .81 kNm	.5) / 1000 = .	.97 kNm	> Mx, OK	
$M_{cy} = p_y \times min(Zy, Sy, eff)$	=	0.81	kNm 1	∴ M <sub>cy</sub> =	.81 kNm			> My, OK	
coment Capacity to Lateral T $M_h \ge m_{LT} M_x$ $m_{LT} =$			ľ					; Cl. 8.3.5.2 ; Table 8.4a	
	▪ pb Zx or pb Seff							;(8.20 - 8.24)	
$\lambda_{LT} =$	$= uv\lambda\sqrt{\beta_w}$							; Cl. 8.3.5.3	
λ = × =	= 0.9 = $L_E / r_y = L_{ey} / r_y =$ = $D / T =$ $\frac{1}{(1 + 0.05 (\lambda / x)^2)}$	67.57 10.00 $\frac{10}{10.00} = 0.74$	Hot-rolled Section	n <b>v</b>					
√ <i>β</i> . =		)	Semi-Compac	t Sections				2.00	
$ \begin{array}{ccc} \therefore & \lambda_{LT} = \\ \therefore & p_b = \\ P_{cy} \ge A_g p_c \end{array} $		N/mm <sup>z</sup>	Rolled Section	, <b>.</b>	-s.			; 8.3.5.2 App. 8.	.1
∴ M <sub>b</sub> = 1.53	kNm							> mLT Mx, OK	
<u>ompression Resistance</u> L <sub>ex</sub> = L = 1000 L <sub>ev</sub> = L = 1000	mm mm		= 14.8 = 14.8	mm $\lambda_x = L_{ex}/r_{ex$		67.6 67.6	Rollec	I H-Section	•
p <sub>ex</sub> = P <sub>ey</sub> =		N/mm² N/mm²		$P_{cx} = p_{cx} \cdot A = P_{cy} = p_{cy} \cdot A =$	101.47 kN 91.36 kN			; Cl. 8.7.5 ; Table 8.7, App	o. 8.4
∴ P <sub>c</sub> = 91.36	kN							> Fc, OK	
ompression Members unde	r Combined Axial	Force and Mom	ents						
Cross-section Capacity F <sub>c</sub> / Ag P <sub>y</sub> = 0.00	+ M <sub>x</sub> / M <sub>cx</sub> + 0.505	+ M <sub>y</sub> /M <sub>cy</sub> + 0.234	≤1					; Cl. 8.9.1	
= 0.74 < 1.00								< 1, OK	
<i>lember Buckling Resistance</i> m <sub>y</sub> :	a : = 1.0							; Table 8.9	
	= p <sub>y</sub> Z <sub>y</sub> = 0.81	kNm						; Cl. 8.9.2	
F <sub>o</sub> /P <sub>oy</sub> = 0.00	+ m <sub>LT</sub> M <sub>x</sub> / M <sub>b</sub> + 0.27	$+ \frac{m_y \overline{M_y}}{M_z} \frac{M_{ey}}{M_{ey}}$	≤1					; Cl. 8.9.2	
= 0.50 < 1.00								< 1, OK	
F <u>ension Resistance</u> Ke = 1.0 Ae = K <sub>e</sub> a <sub>n</sub> = 491	≤ a <sub>g</sub> mm²		Hole Area =	0	mm <sup>2</sup>				
∴ P <sub>t</sub> ≂ py.Ae = 135.03	kN		a1 a2		៣៣² ៣៣²			> Ft, OK	
Tension Members under Cor	nbined Axial Ford	e and Moments							
Ft/Pt	+ M <sub>x</sub> /M <sub>cx</sub>	+ My/May	≤1					; Cl. 8.8	
≕ 0.00	+ 0.505	+ 0.234							

С

J	EC	7	Job	BTS site	3				_		Job No. J8009 62693	Paq SZ
CALC	ULAT	ION	Calcı	ulations by	' 0		Checked I	by	A. C	han	Dete	pr,202
Design	for We	ld Cor	inectio	n								
Try !	5 mm fi	llet wel	d all rour	nd , p <sub>w</sub>	= 22	0 N/mn	1 <sup>2</sup>	*	<u> </u>	-1		
a = {	5 øin 49	5°										
=	3,54	mm						L				
L = 5	50 - 2 x S	5							1			
4443 1999	40	mm							2		,,,,,	   I ª
Weld Are	ea. Acar	. =	: 2aL					~				
	~→ · ·(weid)	) =		3.54 x 40								
		=	: 28	34 mm²				-				
C1 :	V <sub>A</sub>		0.82	kN								
	C <sub>A</sub>		0.35	kN								
	RA	=	0.9	kΝ								
C2:	Vв	=	0.5	kN								
	С <sub>в</sub>	=	0.74	kN								
	$R_B$	Ξ	0.9	kΝ								
C3 :	Vc	=	0.41	kN								
	Cc				°C3 + Compr							
		= (	0.18 + WI	Lex/2x(H	(2+0.8)/	D + WLey / 2	x(H/2+C	0.8) / W	0.65			
			).18 + 1.4 3.4772		6/2+0.8)	10,5 + 0.68	VX ( 0.0724	-0.0)7	0.00			
	p	<u> </u>	3,51	kN								
	R <sub>C</sub>		0,01	KJN -								
Shear o	n weld	=			x 1000 / 284				OK	1		
		11	12.25	N/mm <sup>2</sup>	<	Pw			ОK	1		
								Ad	lopt 5	mm Fille	t Weid All R	ound
Design	for Ar	ichor	Bolts									
Shear, V	/x	= 1	NLex / 2	/ 1.4	= 1.48 /	2/1.4	=	0.53	kN			
Shear, V			NLey / 2		= 0.69	/2/1.4	Ξ	0.25	kΝ			
Moment	t, My	= \	WLex / 1.4	4 x (H / 2 +	0.8)	= 1.48 / 1.4	4 x (0.6 / 2 +	+ 0.8)	-	1.16	kNm	
Max Ter	15ion, T	= e	se / 2 / 1.	.4 + WLey /	1.4 x (H / 2 +	0.8) / W						
					x (0.6 / 2 + 0							
		<u></u>	1.18	kN								
Try	6 r	105.	HGT3 -	R -	MB							
Please i	refer to l	hilti out	tput								- UCTA P	10 hal
								-	Ad	opt ono	6 HST3-R- I	10 201

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Specifier's comments:

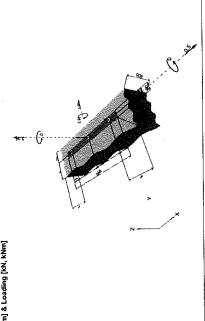
E.
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					h <sub>eem</sub> = 54.0 mm				b bulletin 58, Mechanical	ាងល	$l_x x l_y x t = 650.0 \text{ mm} x 50.0 \text{ mm} x 3.0 \text{ mm}$ ; (Recommended plate thickness: not calculated)		cracked concrete, C25/30, $f_{c,eff}$ = 25.00 N/mm <sup>2</sup> ; h =200,0 mm, User-defined partial material safety factor $r_c$ = 1.500	n condition: Dry	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)	with longitudinal edge reinforcement d >= 12.0 [mm] + close mesh (stimups, hangers) s <≕ 100.0 [mm]
	HST3-R M8 hef2	50	2105896 HST3-R M8x75 -/10	tp filling solution	$h_{slad} = 47.0 \text{ mm} \{h_{slimit} = - \text{ mm}\}, h_{nom} = 54.0 \text{ mm}$	A4	ETA 98/0001	20/7/2023   -	SOFA based on EN 1992-4 and fib bulletin 58, Mechanical	$e_{b}$ = 0.0 mm (no stand-off); t = 3.0 mm	l <sub>x</sub> x l <sub>y</sub> x t = 650.0 mm x 50.0 mm x	no profile	cracked concrete, C25/30, $f_{c,oyi} = 2$ factor $\gamma_c = 1.500$	hammer drilled hole, installation condition: Dry	no reinforcement or reinforcement	with longitudinal edge reinforceme [mm]
1 Input data	Anchor type and diameter:	Return period (service life in years):	Item number:	Filling set or any sultable annular gap filling solution	Effective embedment depth:	Material:	Evaluation Service Report:	Issued I Valid:	Proof.	Stand-off installation:	Anchor piate <sup>®</sup> :	Profile:	Base material:	Installation:	Reinforcement:	

Application also possible with HST4-R M8 under the selected boundary conditions. More information in section Alternative fastening data of this report.

 $^{\rm R}$  - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [mm] & Loading [kN, kNm]



Input data and results must be checked for conformity with the estisting conditions and for plautibility/ PROFIS Engineering (c. ) 2003-2024 trills AG, FL-9494 Schean trild is a registered. Trademark of Hills AG, Schean



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Max. Util. Anchor [%]	68
Fire	2
Seismic	8
Forces [kN] / Moments [kNm]	$N = 2.360; V_x = 0.500; V_y = 1.060; M_x = 0.000;$
-	
Description	Combination 1
Case	-

## 2 Load case/Resulting anchor forces

Anchor Tension force Shear force 1 4.252 0.391				
1 4.252	ce Shear for	ce Shear force x	Shear force x Shear force y	
	0.391	0.167	0.353	
2 2.195	0.391	0.167	0.353	
3 0.139	0.391	0.167	0.353	Tension
max, concrete compressive strain. max, concrete compressive stress: accuting targion farro in (XV)=16.166 10 0).	rain: ress: «(-156 1/0 0)·	0.10 [‰] 2.91 [N/mm²] 6.586 IkNI		

Anchor forces are calculated based on the assumption of a rigid anchor plate.

Input data and reaults must be checked for conformity with the existing conductors and for pluasheley. PROFILS Engineering ( c ) 2003-2024 Hull AG. FL 9494 Schran Hills is in registered Trademark of Hill AG. Schran

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Frione i Fax: Design: Fastening point:	l 09-2693-S.S. Cabinet (1)	C-waii: Date:		17/4/2024
3 Tension load (EN	3 Tension load (EN 1992-4, Section 7.2.1)			
	Load [kN]	Capacity [kN]	Utilitzation Å <sub>s</sub> [%]	Status
Steel Strength*	4.252	12.643	Ř	QĶ
Pullout Strength*	4,252	6.336	68	Ş
Concrete Breakout Failure**	4.252	8.270	52	ð

A/A

۲/N

N/A

A/A

Splitting failure\*\*

### 3.1 Steel Strength

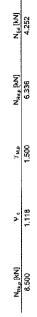
\* highest loaded anchor \*\*anchor group (anchors in tension)

EN 1992-4, Table 7.1 N<sub>Ed</sub> ≤ N<sub>Rd.3</sub> = N<sub>Rk.3</sub> <sup>7</sup>M.5</sup>



### 3.2 Pullout Strength





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									m°]		W <sub>ro.N</sub>		N <sub>Rak</sub> [KN] N <sub>Ed</sub> [KN]	8.270
	EN 1992-4, Table 7.1	EN 1992-4, Eq. (7.1)	EN 1992-4, Eq. (7.2)	EN 1992-4, Eq. (7.3)	EN 1992-4, Eq. (7.4)	EN 1992-4, Eq. (7.6)	EN 1992-4, Eq. (7.6)	EN 1992-4, Eq. (7.7)	s <sub>et N</sub> [mm] [.e. <sub>69</sub> , [N/mm <sup>2</sup> ]	141.0 25.00	W MC2/N W S.N	1.000 1.000	N <sup>D</sup> <sub>RA,C</sub> [KN] Y <sub>M,C</sub>	12.405 1.500
		= Ν <sup>0</sup> - Α <sub>2.</sub> Ψ <sub>2.</sub> Ψ <sub>2.</sub> Ψ <sub>10</sub> Ψ <sub>10</sub> Ψ <sub>10</sub> Ψ <sub>10</sub> .							c <sub>ers</sub> [mm] s <sub>e.</sub>	70.5	e <sub>c2,M</sub> [mm]		k, N <sup>G</sup>	7,700
akout Failure	약 .		≖k, vic. h <sub>u</sub>	= S <sub>er,N</sub> · S <sub>er,N</sub>	$= 0.7 + 0.3 \cdot \frac{c}{c_{crN}} \le 1.00$	$\frac{1}{\left(\frac{2\cdot e_{N,1}}{s_{crN}}\right)} \leq 1.00$	$\frac{1}{\left(\frac{2}{2}\cdot e_{N2}\right)} \leq 1.00$	Sarn J	A <sub>6.N</sub> [mm <sup>2</sup> ]	19,881	W act N	1.000	W M.N	1,000
3.3 Concrete Breakout Failure	$N_{Ed} \le N_{Rd,c} = \frac{N_{Rk,c}}{\gamma_{M,c}}$	N <sub>ak.c</sub> = N <sup>0</sup> <sub>Rk</sub>		A <sub>cN</sub> ≡ s <sub>erN</sub>	₩ <sub>5,N</sub> = 0.7	Ψ <sub>ee1.N</sub> =	ψ <sub>α2,N</sub> = +1	¥ <sub>MN</sub> = 1	A <sub>6.8</sub> [mm <sup>2</sup> ]	19,881	e <sub>c1,N</sub> [mm]	0.0	z {mm]	461.8

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4 Shear load (EN 1992-4, Section 7.2.2)	ection 7.2.2)					4.3 Concrete edge failure in direction y+	llure in direction y+	-				
	Load [kN]	Capa	Capacity [kN]	Utilization A <sub>V</sub> [%]	Status	$V_{ed} \leq V_{edc} = \frac{V_{exc}}{2}$			fib Bulletin 58, Table (10.2-1)	able (10.2-1)		
Steel Strength (without lever arm)*	0.391	-	12.560	4	ý			- M - M -	fib Bulletin 58. Eo. (10.2-5)	ia. (10.2-5)		
Steel failure (with lever arm)*	N/A		N/A	N/A	N/A	<ul> <li>VRkc</li> <li>VRk</li></ul>	$= k_{0} \cdot d_{nn}^{2} + k_{1} \cdot v_{n1}^{2} \cdot v_{n2}^{2} \cdot v_{n2}^{2} \cdot v_{n3}^{2} $	V.84 V.9 F	fib Bulletin 58, Eq. (10.2-5a)	eq. (10.2-5a)		
Pryout Strength**		"	21.668	8	¥ :				fib Bulletin 58. Eo. (10.2-5a.)	ca. (10.2-5a.)		
Concrete edge faiture in direction y+**	1.172	~*	33,440	4	Ś		, <sup>02</sup>					
<ul> <li>highest loaded anchor</li> <li>"anchor group (relevant anchors)</li> </ul>	oup (relevant anchors)						C <sub>1</sub>		fib Bulletin 58, Eq. (10.2-5a2)	Eq. (10.2-5a <sub>2</sub> )		
4 1 Steel Strength (without lever arm)	1					$A_{c,V}^{0} = 4.5 \cdot c_{1}^{2}$			fib Bulletin 58, Eq. (10.2-5b)	Eq. (10.2-5b)		
						$\psi_{A,V} = \frac{A_{c,V}}{\Delta^{D}}$			fib bulietin 58 (C	fib bulletin 58 (07/2011) Section 10.2.5.1.1 b)	25.1.1 b)	
$V_{Ed} \leq V_{Rd,a} = \frac{V_{Rd,a}}{v_{abc}}$ EN 199	EN 1992-4, Table 7.2								fib bulletin 58 (C	fib bulletin 58 (07/2011) Figure 10.2-4	4	
2	EN 1992-4, Eq. (7.35)						$= 0.7 + 0.3 \cdot \frac{c_2}{1.5 \cdot c_1} \le 1.00$		fib Bulletin 58, Eq. (10.2-5d)	Eq. (10.2-5d)		
V <sup>R</sup> RAS [KN] Ky	V <sub>Ris</sub> , [kN]	Y <sub>M.s</sub>	V <sub>Rdis</sub> [kN]	V <sub>Ed</sub> [KN]		$\Psi_{n,v} = \left(\frac{1.5 \cdot 0}{n}\right)$	$\left(\frac{1.5 - c_1}{2}\right)^{0.5} \ge 1.00$		fib Bulletin 58, Eq. (10.2-5c)	Eq. (10.2-5c)		
15.700 1.000	15.700	1.250	12.560	0.391		-	1 / ≤ 1.00 /2 · e,/ ≤ 1.00		fib Bulletin 58, Eq. (10.2-5e)	Eq. (10.2-5e)		
4.2 Pryout Strength		·				+	<u>, c, )</u>					
Ver < Ver = <u>Ver a</u>		EN 1992-4, Table 7.2	Table 7.2			₩ "~ = <mark>\</mark> (00	$\left(\cos \alpha_{v}\right)^{2} + \left(\frac{\sin \alpha_{v}}{w}\right)^{2} \ge 1.00$	1.00	fib Bulletin 58, Eq. (10.2-5f))	Eq. (10.2-5f))		
Vakes = Ka Nake		EN 1992-4, Eq. (7.39a)	Eq. (7.39a)				· >			ţ	ei	
$N_{\text{RK},c} = N_{\text{RK},c}^{0} \cdot \frac{\Lambda_{c,N}}{2} \cdot \Psi_{n,N} \cdot \Psi_{n,N}$	ч. Ч <sub>аст.</sub> м' Ч <sub>ас</sub> г.м' м.м	EN 1992-4, Eq. (7.1)	Eq. (7.1)			[mm]	d <sub>nom</sub> [mm]	4 KV	α 0.060	β 0.060	(cew [N/mm]	
$N_{\text{NK,e}}^{O} = K_1 \cdot \sqrt{K_K} \cdot \frac{1}{N_{01}}$		EN 1992-4, Eq. (7.2)	Eq. (7.2)			47.0 C. [mot]	o.uu Afmm²1	میں A°ر [mm²]	90.00 VA		2	
		EN 1992-4, Eq. (7.3)	Eq. (7.3)			100.0	120,000	45,000	2.667			
$\psi_{s,N} = 0.7 + 0.3 \cdot \frac{1}{c_{s,N}} \le 1.00$		EN 1992-4, Eq. (7.4)	Eq. (7.4)			л° д	$\Psi_{\rm hV}$	ر مر	¥. "	e <sub>cv</sub> [mm]	4' <sub>40,</sub> V	¥
$\Psi_{\text{ectN}} = \frac{1}{2.5 \times 10^{\circ}} \le 1.00$		EN 1992-4, Eq. (7.6)	Eq. (7.6)			1.000	1.000	25.25	1,086	0.0	1.000	1.400
$1 + \left(\frac{s_{\alpha N}}{s_{\alpha N}}\right)$					-	v.06 W	V <sup>0</sup> <sub>Rhi</sub> e [kN]	YMIG	V <sub>Relo</sub> [kN]	V <sub>Ed</sub> [kN]		
$\psi_{0,2,N} = \frac{1}{1 + (2 \cdot e_{V,2})} \le 1.00$		EN 1992-4. Eq. (7.6)	Eq. (7.6)			2.500	12.366	1,500	33.440	1.172		
Y WN = 1 Sen /		EN 1992-4, Eq. (7.7)	Eq. (7.7)									
$A_{c,N}$ [mm <sup>2</sup> ] $A_{c,N}^{0}$ [mm <sup>2</sup> ]	c <sub>er,N</sub> [mm]	s <sub>cr,N</sub> [mm]	k.	f <sub>eod</sub> [Nimm <sup>2</sup> ]								
	70.5	141.0	2.620	25.00								
e <sub>c1 V</sub> [mm] <sup>V</sup> ac1 N	e <sub>czy</sub> [mm] o o	Ψ «c2.N 1.000	Υ <sub>4,N</sub> 1 000	4.000	Y.N.N 1.000							
		Var. IKN	Ves [kN]									
	1.600	21,668	0.391	ł								

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Group anchor ID 3

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# 5 Combined tension and shear loads (EN 1992-4, Section 7.2.3)

Steel failure				
ß	ßv	ಶ	Utilization B <sub>NN</sub> [%]	0
0.336	0.031	2.000	12	

Status

 $\beta_N^{''} + \beta_V^{''} \leq 1.0$ 

Concrete failure



## 6 Displacements (highest loaded anchor)

oading:	
Short term I	

= 0.5250 [mm]	= 0.2309 [mm]	= 0.5735 [mm]		= 0.9624 [mm]	= 0.3479 [mm]	= 1.0234 [mm]	
ç,	ς, γ	å <sub>NV</sub>		õ	δ <sub>V</sub>	ð <sub>NV</sub>	
N_ = 3.150 [kN]	ŀ		Long term loading:	N <sub>sk</sub> = 3.150 [kN]	V <sub>Sk</sub> = 0.289 [kN]		

Comments. Tension displacements are valid with haif of the required installation torque moment for uncracked concrete! Shear displacements are valid without friction between the concrete and the anchor plate! The gap due to the drilled hole and clearance hole tolerances are not included in this calculation!

The acceptable anchor displacements depend on the fastened construction and must be defined by the designer!

#### 7 Warnings

The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Armex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stift, in order not to be deformed when suplected to the design identity. PMOFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to link the stress of the anchor plate are not considered - the calculates the minimum required anchor plate thickness with CBFEM to link the stress of the anchor plate based on the assumptions explained above. The proof if the fright anchor plate assumptions valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!

Design is only valid if hole is filled to remove clearance, clearance as per EN 1992-4 Table 6.1

Checking the transfer of loads into the base material is required in accordance with EN 1992-4, Annex At

The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 6.1 of EN 1992-4! For larger cleaneters
of the clearance hole see section 6.2.2 of EN 1992-4!

. The accessory list in this report is for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.

. For the determination of the  $\psi_{m,v}$  (concrete edge failure) the minimum concrete cover defined in the design settings is used as the concrete cover of the edge reinforcement.

The characteristic bond resistances depend on the return period (service life in years): 50

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Fastening meets the design criterial

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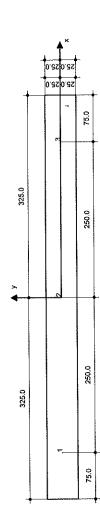
### 8 Installation data

Anchor type and diameter: HST3-R M8 hef2 Item number: 2105896 HST3-R M8x75 -/10	Maximum installation torque: 20 Nm	Hole diameter in the base material: 8.0 mm	Hole depth in the base material: 71.0 mm	Minimum thickness of the base material: 80.0 mm	
Anchor plate, steel: EN S235; E = 205,000,00 N/mm <sup>2</sup> ; f <sub>3</sub> = 235.00 N/mm <sup>2</sup> Profile: no profile	Hole diameter in the fixture: d <sub>f</sub> = - mm	Plate thickness (input): 3.0 mm	Recommended plate thickness: not calculated	Drilling method: Hammer drilled	Cleaning: No cleaning of the drilled hole is required

Hilk HST3 stud anchor with 47 mm embedment, M8 hef2. Stainless steel, installation per ETA 98/0001, with annular gaps filled with Hilt Filling set or any suitable gap solutions

accessories	
Recommended	
8.1	

Setting	<ul> <li>Torque controlied cordiess impact tool</li> </ul>	<ul> <li>Torque wrench</li> </ul>	• Hammer
Cleaning	<ul> <li>No accessory required</li> </ul>		
Drilling	<ul> <li>Suitable Rotary Hammer</li> </ul>	<ul> <li>Property sized drill bit</li> </ul>	•



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### 9 Drilling and installation

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### 10 Alternative fastening

	H\$T4-R M8	50	2329094 HST4-R M8x65 5-20	gap filling solution	$h_{sland} = 47.0 \text{ mm} (h_{sl,tml} = * \text{ mm}), h_{nom} =$	A4	ETA-21/0878	
10.1 Alternative fastening data	Anchor type and diameter:	Return period (service life in years):	Item number:	Filling set or any suitable annular gap filling solution	Effective embedment depth:	Material:	Evaluation Service Report:	

toward Buttle das tanging armune fue to the filler	
Effective embedment depth:	h <sub>atad</sub> = 47.0 mm (h <sub>utumi</sub> ≖ - mm), h <sub>nom</sub> = 53.0 mm
Material:	A4
Evaluation Service Report:	ETA-21/0878
issued I Valid:	28/2/2024   -
Proof.	SOFA based on EN 1992-4 and fib bulletin 58, Mechanical
Stand-off installation:	$e_b = 0.0 \text{ mm}$ (no stand-off); t = 3.0 mm
Anchor plate <sup>R</sup> :	$I_x \times I_y \times t = 650.0 \text{ mm} \times 50.0 \text{ mm} \times 3.0 \text{ mm}$ ; (Recommended plate thickness: not calculated)
Profile:	no profile
Base materiai:	cracked concrete, C2Si30, $f_{\rm Cov}$ = 25.00 N/mm $^2$ ; h =200.0 mm, User-defined partial material safety factor $\gamma_{\rm c}$ = 1.500
Installation:	hammer drilled hole, installation condition: Dry
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any $B$ ) or >= 100 mm ( $B <=$ 10 mm)
	with longitudinal edge reinforcement d >≖ 12.0 [mm] + close mesh (stirrups, hangers) s <= 100.0 [mm]

### Max. Utilization with HST4-R M8: 58 % Fastening meets the design criteria!

### 10.2 Installation data

Anchor plate, steel: EN S235; E = 205,000.00 N/mm <sup>2</sup> ; $f_{yk}$ = 235.00 N/mm <sup>2</sup>	And
Profile: no profile	lten
Hole diameter in the fixture: $d_f \equiv -mm$	Mau
Plate thickness (input): 3.0 mm	Ĥ
Recommended plate thickness: not calculated	F
Drilling method: Hammer drilled	Μ
Cleaning: No cleaning of the drilled hole is required	

chor type and diameter: HST4-R M8 m number: 2329094 HST4-R M8x65 5-20 kimum installation torque: 20 Nm le diameter in the base material: 8.0 mm le depth in the base material: 73.0 mm nimum thickness of the base matéria!: 94.0 mm Hitli HST4-R stud anchor with 47 mm embedment, M8, Stainless steel, installation per ETA-21/0878, with annular gaps filled with Hitli Filling set or any suitable gap solutions

### 10.2.1 Recommended accessories

Setting	<ul> <li>Torque controlled cordiess impact tool</li> <li>Torque wrench</li> <li>Hammer</li> </ul>
Cleaning	No accessory required
Drilling	<ul> <li>Suitable Rotary Hammer</li> <li>Properly sized drill bit</li> </ul>

trour data and results must be checked for conformity with the existing conditions and for phuyshilly. PROFIS Engineering ( c ) 2003-2024 Hilly AG, FL-8494 Schean Hill is a registered Trademark of Hills AG, Scheen



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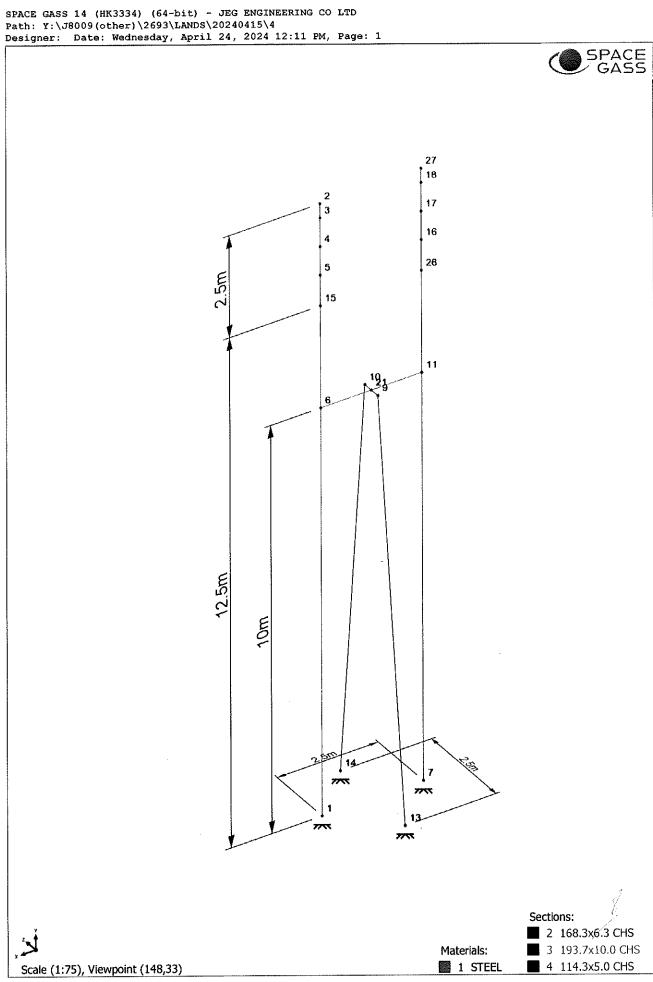
## 11 Remarks; Your Cooperation Duties

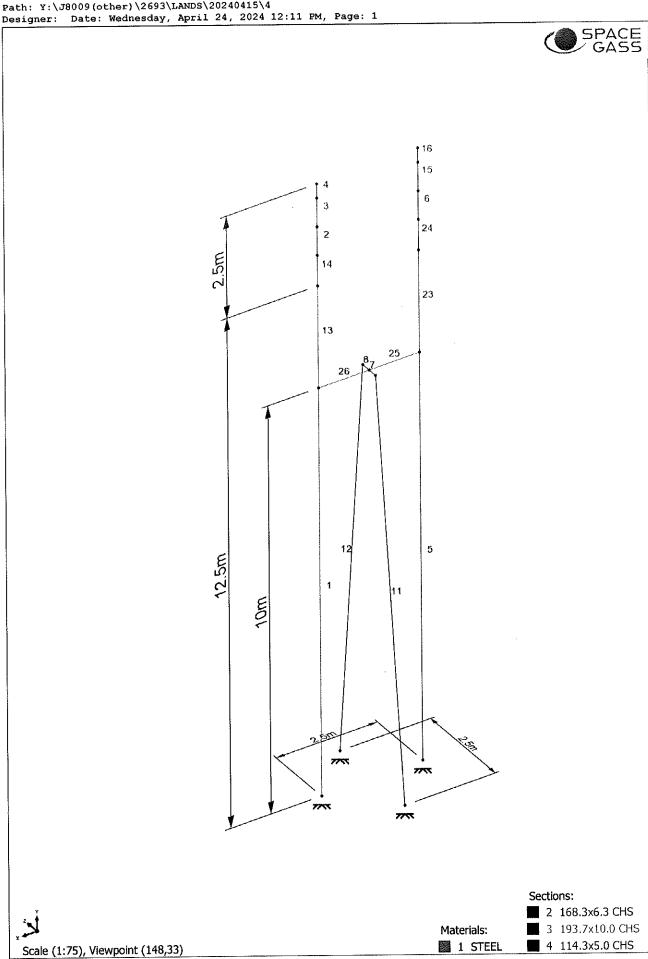
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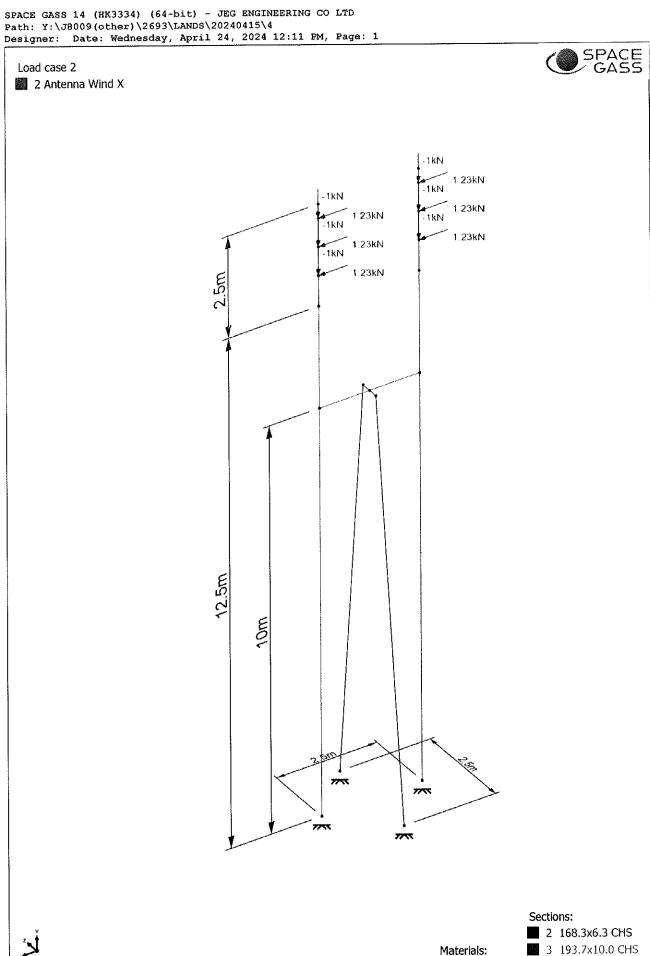
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CALCULATION	Calculations by	Checked by	M.K. Wong		pr 2024		
Design Wind Pressure Calculation under CoP on Wind Effects 2019							
Max. Actual Height	, Z, above ground level	= 16 m					
Wind reference pres	sure, Qo,z	= 3.7 ( 16/ 500	) ^0.16	>	2.13 kPa		
Directionality facto	r on pressure, So	= 0.85 (assu	me cricital valu	e)			
Max. Slope Height	= <i>O</i> m; Max.	Slope Length =	<i>O</i> 'm;				
Upwind slope of top	ographic feature	= 0/0	==)	> #DIV/0]	<0.03		
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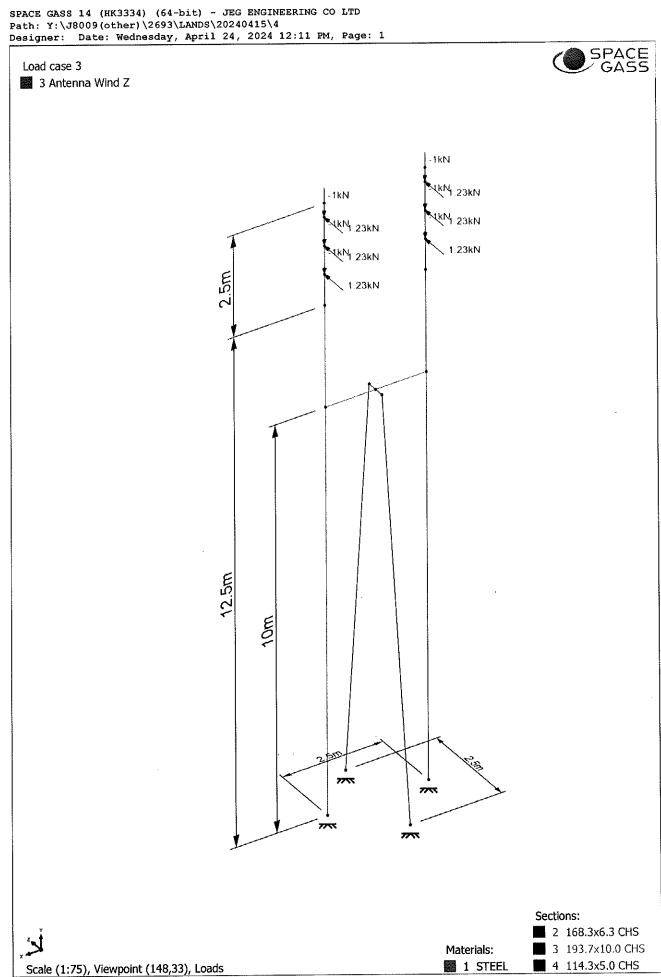
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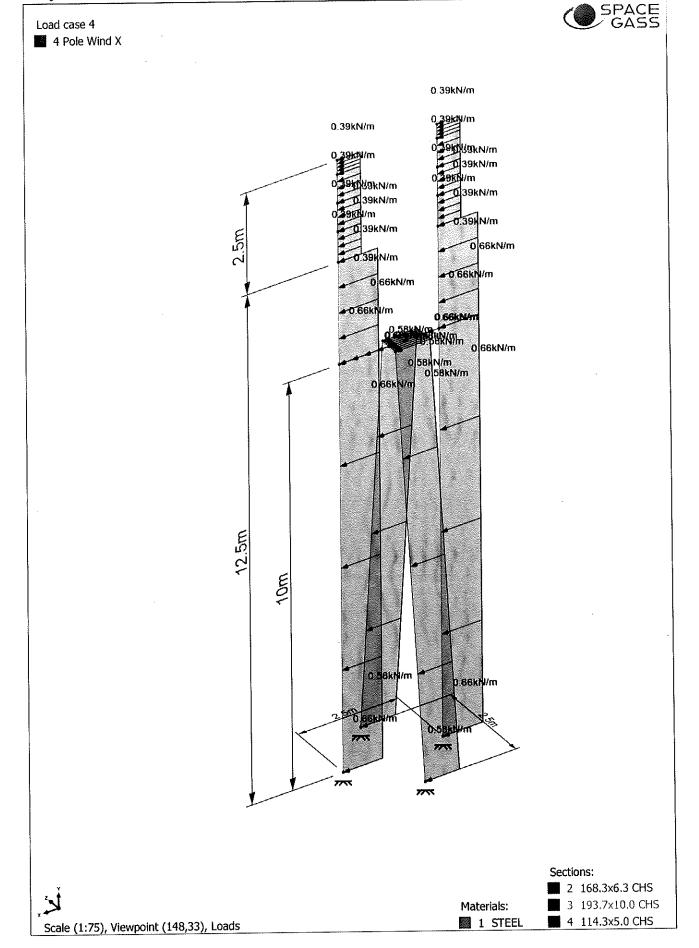
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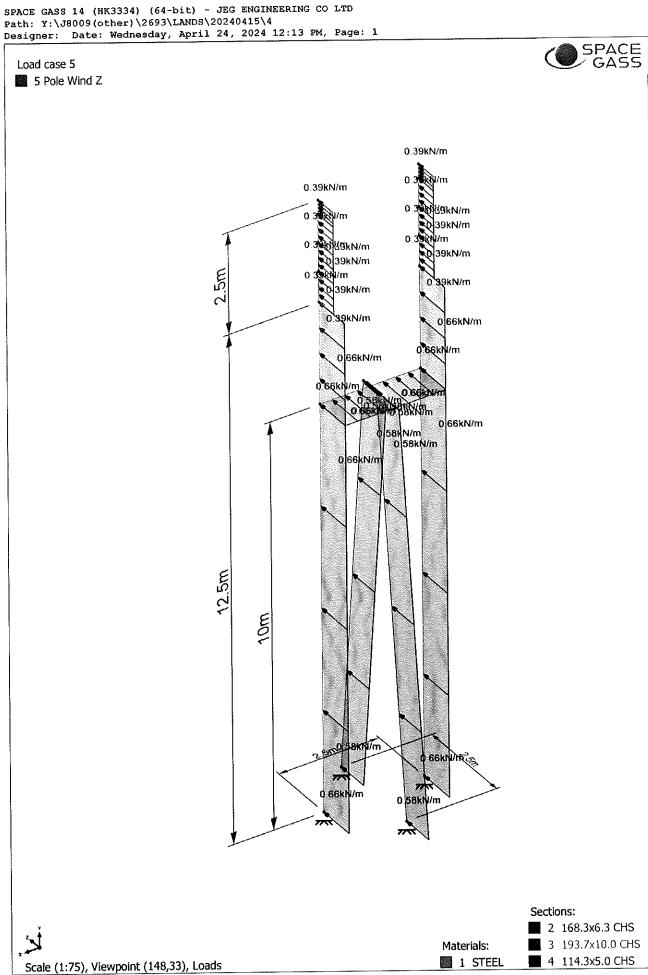


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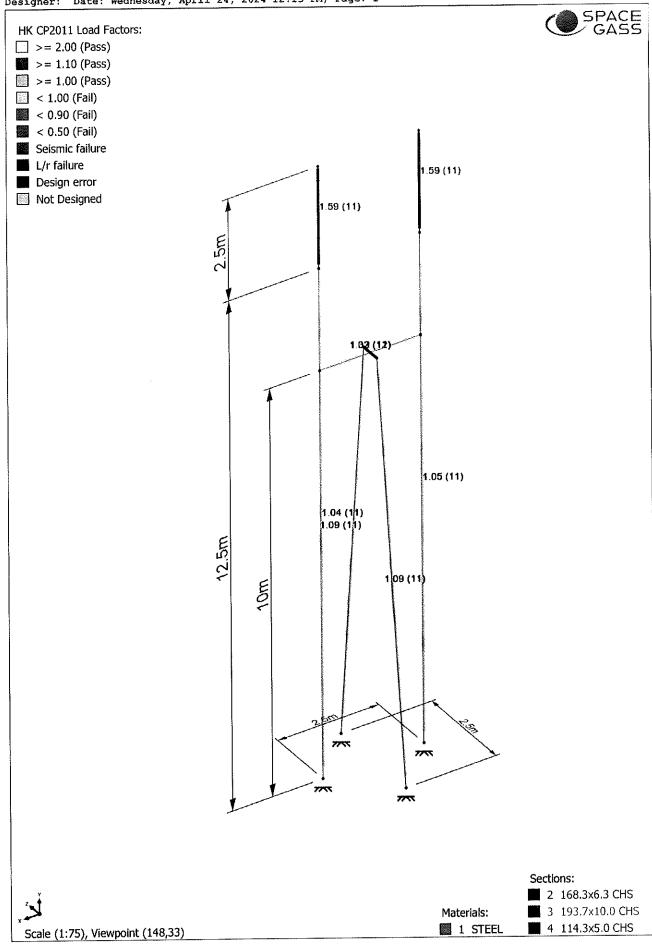
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	X-Akta	V-Acts	Z-Akla	X=Act1 <i>a</i>	Y-A313	
17-1e	fransl'n	Transt'n	fransl'n	Rotation	Rotation	Boganion
17	-0.036	-0.041	43.050	0.0271	0.001	0.000
13	-0.102	-0.043	55.066	0.017	0.001	0.049
21	-D.OOA	-0.004	6.200	0.000	0.005	0.000
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tiode	Transi'n	Tranal 'n	Transi'n	Rotation	Rotation	Rotation
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2	151,759	-0.297	+0.000	+0.400	0.000	-0.006
3	149.596	-0.297	-0.000	-0.300	0,000	-0.006
4	145.303	-0.197	~0.000	~0.00	0,000	-0.006
5	141.239	-0.797	+0.000	-0.000	0.000	~0.006
-	128.550	+0,297	+0.000	-0+000	0.000	-0,003
7	01000	0.000	0,000	0.000	0.000	0.000
э	1201535	-0.0505	0.000	0.000	0.000	-0.001
10	108.535	+0.060	×0+000	-0.000	-0,000	-0.001
11	120-550	0.297	0.0.0	0.000	0.000	-0.003
63	6.000	0.000	0.000	0.000	0.000	0.000
24	0.000	3.000	0.000	0,060 =0.060	0,000	0.000 -0.004
15	1 37 . 5 41	-0.297	-0.000	-0.000	0,500	-0.004
16	141.239	0.197	0.000 0.000	0.060 0.060	0,000	-0.006
17	145,308 149,596	0.297	0.000	0.000	0,000	-0,000
13	128.550	-0.000	0.000	-0.000	0.000	0,000
26	137.041		0.000	9,000	0.000	-0,004
27	151.759	1.297	0.000	0.000	0.000	-0.006
			1,000	0,000	0.00	
ad tase S radise ap	(Sinear): 1991 S-Acto	Pole Wind 3 Y-Axis	Z-Axle	X-Ax13		
ad tase 5	(2.1near): 1941 X-Axta Transl'n	Pole Wind S Y-Axis Trinsl'n	Z-Axie Transl'n	X≁Axia Rotation	T*Asia Rotation	3-Axis Rotation
ad 2005 5 radise do Wode 1	(2.1near): 1993 X-Axia Transl'n 3.000	Pole Wind 3 Y-Axis Trinsl'n 0,000	Z-Acte Transl'n 0.000	X-Axia Rotation 0.000	T*Axis Rotation 0.000	3-Axis Rotation 0.000
ad 2005 5 radise do Wode 1 2	(Linear): 1991 X-Asia Transl'n 0.000 -0.000	Eble Wind S Y-Axis Trinsl'n 0,000 -0,000	Z#8%le Transl'n 0.000 25.921	2+Axis Rotation 0.000 0.002	T*Axis Rotation 0.000 -0.601	S-Axis Rotation 0.000 9.000
ad tase 5 radise do Wode 1 3	(3,1near): 1991 X-Axia Tranglin 0,000 -0,000	<pre>Pole Wind 3     Y=Axis     To mai 'n</pre>	Z#3%le Trans1'n 0.000 25.921 25.395	X-Axia Rotation 0.000 0.002 0.002	T*Axis Rotation 0.000 -0.001 -0.001	S-Axis Rotation Q.000 Q.000 Q.000
až tako 5 radise do Hođe 1 2 3 4	(1,1near): 1991 7/402 7/402 0,000 -0,000 -0,000 -0,000	Pole Wind 3 Y-Axis Toursl'n 0,000 -0,000 -0,000	Z-Acte Transl'n 0.000 25.921 25.395 21.379	2-Ax13 Rotation 0.000 0.000 0.000 0.000	(*Axia Rotation 0.000 -0.001 -0.001 -0.001	3+Axis Rotation 0.000 0.000 0.000 0.000
at 2005 S radise do Node 1 3 4 5	(Elnear): 19-n Tringl'n -0.000 -0.000 -0.000 -0.000	<pre>Pole Wind 3 Y=Axis Tconsl'n 0,000 -0,000 -0,000 -0,000 -0,000</pre>	Z-Acte Transl'n 0.000 25.921 25.395 21.379 23.583	2+Ax15 Rotation 0.000 0.000 0.001 0.001 0.001	7-Axia Rotation -0.000 -0.001 -0.001 -0.001 -0.001	3-Acis Rotation 0,000 0,000 0,000 0,000 0,000
at tase 5 radise ap Wode 1 3 4 5 6	(3.1near): 1991 Transl'n 0,000 -0,000 -0,000 -0,000 -0,000 -0,000	Pole Wind 3 Y-Axis Temp! 'n 0,000 -0,000 -0,000 -0,000 -0,000 -0,000	Z-Axia Teanolin 0.000 25.921 25.935 21.370 23.583 26.683	2+Axia Rotation 0.000 0.002 0.001 0.001 -0.002	Y=Acis Rotation =0.000 =0.001 =0.001 =0.001 =0.001	2-Actis Botation Q.000 Q.000 Q.000 Q.000 D.000 D.000
ad 2005 5 radise do Hode 1 3 4 5 7 7	(%inear): 12er X+Acta Transl*n 0.000 -0.000 -0.000 -0.000 -0.000 0.000	<pre>Pole Wind \$ Y=Axis Tc mai'n 0,000 -0,000 -0,000 -0,000 -0,000 -0,000 -0,000 0,000 0,000</pre>	Z=Axila Transl'n 25,921 25,335 21,379 23,583 26,683 0,000	X-Ax13 Rotation 0.000 0.001 0.001 0.001 0.001 0.003	(*Acis Rotation -0.(01) -0.(01) -0.001 -0.001 -0.001 -0.000	3+Actis Rotation 0,000 0,000 0,000 0,000 0,000 0,000 0,000 0,000
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ad : 146-5 5 radise do 070de 1 2 3 4 5 6 7 7 9 10	(Sinear): 1991 X-Rein Transl'n -0.000 -0.000 -0.000 -0.000 -0.000 -0.000 -0.000 -0.000	<pre>Pole Wind 2 Y=Axis To ms! 'n 0,000 -0,000 -0,000 -0,000 -0,000 0,000 -0,000 0,000 -1,070 0,000 1,570</pre>	Z+Axile Tranol *n 0.000 25.921 25.395 31.370 23.503 26.633 0.000 25.575 25.575	X-Ax13 Rotation 0.000 0.001 0.001 0.001 0.001 0.003	(*Acis Rotation -0.(01) -0.(01) -0.001 -0.001 -0.001 -0.000	2-Axis Rotation 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000 0.000
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Load case 21 (Linear): Ward X Pooting Paradise solver

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ń	1 di 1 7	1,000 2,000	2.410 2.116	-0	-000	ារព័ត្រ កម្មតំពិត	
۲	21	647.11 647.11	-0.200 -0.200	1.7V 1.28	- (1, - 41) - (1, - 41)	-0.311 -0.001	
.5	23 10	0.011	-0.105 -0.105	-1.23" -1.37	5.0 1 1.0 1	0.411	
11	1	01.203 01.203	0.00	1.137	0	-5.310 -5.321	
12	10 14	0.206 0.206	a, 1961 6, 1964	-1.239	4,367	3,110 -d. (21	
13	6 15	3.000 3.000	8.034 8.384	, :1)0 1-,10	,01-1 	-11, (S.A) -11, (S.A)	
14	15	3.000	3, 634	0,000	-0,000 -0,000	- (1, (a)(1)) - (1, (a)(1))	
15	11	1.000	1.2.1	-0.000 -0.000	-0,000 -0,000	აკიტი აკიტი	
1.4	18	0,005 	-0.000 -0.000	-3,000 -3,000	0,000 0,000	0,000 	
: 3	11	21400 51400	3.684 3.604	≁0,000 −0,000	0.000		
24	26 19	3	3.6%) 3.6%)	-0,0-0 -0,0-0	-0,000 -0,000	თ.ები ქ.მოქ	
25	21 11	1.231	25.473		60 60	0.000 0.000	
26		-113	25.233	-0.100	0.000	-0,640	

osi casa 3 (1 Gradice Solva		ntenna Sind	z					
Itarb	12044	Axial Foi:9	7-Axis Shear	2-Actio Sister	X-Agiz Totsion	Y-Ac14 Marent	2-Anis Victori	
1	1	1.735	-0.000	9,642	0.245	-1.475	0.021	
1	ů	3.795	-0.006	0.642	0.246	4.350	-0.043	
2	5	2.000	6.400 6.000	-2,456	0.000 0.000	2.570 0.860	-0.000 -0,000	
	4			-1.008	0.000	0.950	-0,000	
3	4	1.044	0.000	-1.228	0.000	0.000	-0.000	
4	3	0.060	0.000	-0.000	0.000	-0.000	-0.600	
	2	0.000	0,000	-9.079	6.000	- A, ((A)	-0	
5	7 11	2,195	0.006	0.642 0.642	-0.246 -0.246	-1.175 4.956	-0.021 0.043	
					-0.0:0	2.579	0.000	
ű	16	2.000 2.000	-0,000 -0,000	-2,456 -2,456	-01000	0.360	0,000	
7	21	-1,306	41.492	-0.000	0,000	0.000	-9.553	-
	•	-:.306	45+452	-0.00	0.000	0,000	0.810	
a	.1	1.347	-41,862	-0.000 -0.000	0.000	0.000	3.651 -0.814	
	10	4,347	-41.862					
1)	3 13	-41.375 -41.375	-0.160	-0.000 -0.000	-0,000 -0,000	0,000 -0,000	0.810 -0.795	
12	10	42.037	0.163	-0.040	-0.690	0.400	-0,31;	•
	11	42.037	4.160	+0,000	- <b>0</b> +000	-0.000	0.797	
13	9	3,000	0.000	-3.604 -3.634	0.000	14.952	-3.000 -3.000	
	15	3,000	0.000		•		-0,000	
14	15 5	3.000 3.000	0.000	-3.684 -3.634	0.000	5.342 2.579	-0.000	
15	17	1.000	-0.000	-1.28	-0.000	01560	0.000	
	13	1.40	-0.000	-1.226	-0 <b>-0</b> (6)	0.000	0,000	
16	13	0.000	0.000	0.000	0.000	-0.000 0.000	4,400 -4.400	
	.T	0.000	0 <b>,</b> 060	0.006	0.000			
23	11 26	3,000	=0,000 =0,000	-3.64 -3.684	0.000	\$1,552 5,340	0.000	L P
34	2-3	3,000	-0,000	-3.684	40,000	51340	0.000	
N	15	3.000	-0.000	-3.534	+0.000	2.579	Q. (69)	
25	21	-0.006	0.205	4.326	91643 91642	-5.162	-0.213 0.043	
	11	-0.006	0,265	4-3.76			0,043	
2.5	6 21	-1.006 -0.005	-0.205 -0.205	-4.326 -1.326	-9.602 -0.602	0.246 -5.162	-0.213	
Liad tase ( )	Linearl: 1	Pole Mind X						
Paradize ably	ris (							
Merb	Nale	Axial Forse	V-Aris Shear	S=Actia Shéas	X-Arits Toraton	Y-Ax13 Roment	Z-Artis Bort-Di	
		34.314	10.076	0,000	-0.000	-0.000	-40,615	
1	1	34, 514	3,126	0.000	-0.000	0.000	Z?,041	
2	5	0.060	0.683	0,000	-0.000	-0.000	-0.507	
	4	4,000	0,400	0,000	لى تۇرىلەھ	+0,1-00	-0.215	
3	4	12, Q(+) 12, Q(+)	0.109 0.137	ារូវភាព សូមមាត់	-0,000 -0,000	≈0,,000 ≈0,,000	-0.015 -0.024	

	14 (HK3334) (64-bit) - JEG ENGINERRING CO LTD	
Path: Y:\J8	8009 (ather) \2693\LMD\$\20240415\4	
	Date: Mednesday, April 24, 2024 12:45 PM, Page: 12	
Filter: No	filter	

		axi al	(−ēsis	C-Adis	2-Axis	Y-Axis	C-Axi≞
Meriz	Code	Force	Shear	Spage	Torsion	1852-40L	Her ent
4	3	0.000	0.137	0.000	0.000	-0.000	-0.024
•	2	0.000	0.000	0.080	0.000	-9.000	0,000
5	7	-34,314	10,576	-0.000	-0.000	ອະຫຼຸດເບ	-40.615
	11	-34.314	3.455	-0.000	-0.000	+0,000	27.U\$l
6	16	-0.000	0.613	-0.000	+0.000	0,000	-0.507
	17	-0.000	0,400	-0.000	-0+000	0.004	-0.215
7	21	0.000	-0,000	0.002	-10.1.26	-1.031	0.000
	Э	0.000	-0.000	0.146	~10.126	-1.012	~0.000)
a	21	0.000	+0.000	+0,000	10.1.6	1.031	0,000
	15	0.000	-4.000	-0.146	10,126	1.012	-0.000
11	9	0.000	0.000	0.146	=()+0(%)	-10.176	-0,000
	13	0.000	0.000	5.935	-0.000	20.378	0,000
1.2	10	0.000	0,000	-0.146	0.000	10.176	-0,000
	14	0.005	0,000	-5.335	0.000	-20,318	0.000
13	-5	-((,:=)))	2,630	0.060	-0.000	~0.000	-5,725
	15	-0.00	0.975	0.000	-9*600	-0.00	-1-219
14	15	-0,000	0.975	3.000	-0.000	-0.000	-1.219
	5	-0.00	6.693	a.00á	≈0*0 <i>00</i> )	-0, dilə	
15	17	-0.00	0.409	-0.000	-0.000 -0.000	0,000	-0.215 -0.024
	14	-0.000	0.137	+0.000	- <b>0</b> , ma	0.000	
16	13	0.000	0,137 0,000	-0.000 -0.000	0.000	0.050 -0.050	-01024 01060
	27	0.000	0.000	-0.900			
23	11	-0.000 -0.000	2.330	-0.000 -0.000	5.000 6.000	0,000	-5.725
	26						
2.4	26 16	0.000 6.000	0.075	-0.,0.0 +0.,000	=0.000 =0,000	0,000	-1.219
25	21 11	0.002 +0.026	34,314 34,314	-0.000	0,000 0,000	8.000 9.000	-10,126 32,769
						-0.000	-32.766
26	6 21	0.526	34.314 34.314	-0.000	0.000 0.000	-0,400	10.126
oat case 5   uradiae colv		ole Wind 2					
		Axial	Y-Axia	3-Asta	X=Act a	Y-Actis	C-AxIo
Metrix	81: de	Force	Shear	Shear	Toraton	Honoist	Homent
ı	1	0.000	0.000	-5.445	01232	15,178	-14, (A)
L	•	0.000	0,000	1.175	0,232	-5.257	0,000
2	5	0.000	-0,600	-0,583	0,000	0.597	0.000
-	4	0.000	-0,000	-0.109	0.000	0.215	0.000
3	4	6.000	0,000	-0.109	0.000	0.215	Q. 004
-	3	0.000	0,000	-0.137	6,000	0.024	0.000
	,	0.000	-0.000	-0.137	0,060	0.021	0.00
	5	0.000	-0.000	-0.000	0,000	-0.000	-0.000
	т	0.000	-9,000	-5.445	-0.232	15.173	0.00
	11	0,000	-0+(sid)	1.175	-0.232	~6.167	-0,000
G	15	0.000	0.000	-0.683	-0,000	0,597	-0.00
	1,	6,080	0.000	-3,403	-0*000	0.215	<b>→</b> 0,000
7	.21	-1.633	62.341	-0.064 -6,065	0,000 0,000	0.000	-11.89) 3.69)
	9	-1,777	62.341				

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10-1	St. 1 -	5-14-7-4	76-16	.": -: e1	7 mai 1.	11 - 445	315 - 3
3	į.	4.777	12 - 41 - 12 - 44	<ul> <li>- 2 € 1000</li> <li>- 2 € 1000</li> </ul>	ан, салан 19 улаган	1440 (1997) 1996 - 1997 (1997)	11. i • ). i/
11		4 N.S. 17	1, 15)	 	ana (araw) Ana (araw)	$(c_{\pi} < 0)$	), () -1 -, (7
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15	13	10,000	14,00,0	- 4444		6.45	
1.	19	QU 0	Sector 2	-0.13	-0, -00	9,64	
14	13	0.00 Å	1.200.5	-0.151	0. 30	0.0.4	h
	7	1.000	3 g (1)	0+	$(0_{11}, 2_{12}, 2_{12})$	- 11, serve	1,200
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		1.000	den er	- 1, -79		1.200	
23	1.	5,000	A. 48. 18	* 1. a - 3	0	5	-0.
2%	23 13	aya sa Aya sa	= () , ~ ( () + () , () ()	1.633 3.345	L1 8 . 11 5		0. -0.
16		6.000	11, 114	-1, -65	-11.6 1	a, sa	-0,
.0	21	0	G. Say	-4 93	-11.81	-5.042	0
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		Ansas	7-8213	INA:115	Z-A. I.J	c=4xd s	0-501
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3	:	1.595	21295	1,055	-0.000	$= (x_1, (x_2, y_1))$	-1.0
	\$	1,465	1, 40	1,000	-0,.00	#15° (000	,-3
÷	3	6.035 8.000	0.101	4.449 4.640	(), and) (), and)	ان کې د ده. انځه و د	-0,0) -0,0)
	- ,		17,643	-0.000	-0,000	0,0-0	-75,47
5	11	-71.341 -77.530	0.075	-91059 -91029	-0,000 -0,000	-0.0081	53.61
6	16	3.1.23	4.394		-0.000 -0.000	01, QCA 01, QCA	-1.1
	3.2	.1.995	4.017	×07,000			
7	21	0.267	-0.007 -0.051	1.734 1.936	-22.58 -28	-2.252	-0.0 -0.2
8	21	0.267	- Q_ 667	-1,734	121639	2.710	-0.
	10	0.267	-0.14	-1.336	22,682	3,253	-0.23
11	9 13	0.976 4.435	0.171	1.036	-0.010 -0.010	-171801 37,373	-0.1 -0.2
	10	0.176	0.471	-1. 336	0,010	22.301	), 2'
12		-0+740			0.010	-37,578	-0.25
	14	4.135	+es.175	-1040	0.040	-3.,2.2	

SPACE GASS 14	(HX3334) -	(64-61t) - j	TEG ENGINEE	rutika co lato			
Path: Y:\J8009 Designer: Dat Filter: No fil	e: Nednesc	93\LANDS\20 Lay, April 2	240415\4 24, 2024 12	145 PH, Pag	e: 15		
lieri.	∦∂de	Ardial For co	V-Axis Shear	C−Axis Zhear	X+Axia Toracon	Y-Asia Riment	2-Axis Noment
24	26 16	4.663 4.524	-0.000 -0.000	-6.523 -6.113	-0.000 -0.000	$\frac{9.185}{4.146}$	0.000 0.000
25	2) 11	-0.011 -0.011	0,367 0,398	12.543 11.305	30.092 30.092	-14.236 0.659	-0,506 0.073
25	е 21	-0.011 -0.011	-0,600 -0,867	-11.335 -12.543	-34.092 -34.092	0.649	0.073 -3.525
Load caso 21 ( Paradios Jolya		Hind X Epoti	is)				
		AXLA)	Y-Actis	2-A:(1.)	X-Asia	7-6313	2-Axis
Netto	tinse.	for th	Shear	spear.	Torslor	Norent	However,
1	1 6	$68.602 \\ 64.160$	12.515 5.095	0.000 0.000	-0,000 -0,000	-0,000 0,000	-53.858 38.195
2	5	2.232	3.139 2.865	0.000	~0.000 -0.000	-0.000 -0.000	-3.176 -1.075
3	4	1.132	1.633	0.000	-0.000	-0.000	-1.075
,	3	1.046	1.365	0.000	-0.000	-0,000	-3.024
1	3 2	0.046 0.000	0.137 -0.303	0.000 0.000	0.000	0,000 -0.000	-0.024 -0.000
5	, 11	-50.904 -55.406	12.531 5.911	$-\frac{1}{2} \frac{1}{2} 1$	-0.000 -0.000	0.000	53,910 33,297
-i	16 17	2.132	3.139 2.866	-0,000 -0,000	-0,000 -0,000	0.000	-3,176 -1,075
7	21 9	0.191 0.191	-0.620 -0.681	1.230 1.383	-16.207 -16.207	-1.041 -1.614	-0.033 -0.195
0	21 10	0.191 0.191	-0.600 -0.601	-1.239 -1.303	16.207	1.941 1.614	-0,033 -0.105
11	13	0.397 3.169	0.122	1.303	-0.007 -0.007	-16.227 26.633	-0.135 -0.213
12	10 14	0.607 ),165	0.122	-1.363	0.007 0.007	16.267 -26.699	-0,135 -0,213
13	6 15	4.442 3.331	6.314 4.659	0,000	-0.000 -0.000	-0.000 -0.000	-20.277
11	15	3.391	4.659	0.000	-0.000 -0.000	-0,000) -0,000	-6.5-51 -3.176
15	17 10	1.139	1.633	-0_0%) -0_0%)	-0,000 -0,000	0,000 0,000	-1.035 -0.024
16	18 27	0.046	0.137	0,000	0.000	-0.000 0.000	-0.024 0.000
23	11	4.112	6.314	+0.060	0.000	0.000	-20.207
24	26 26	3.331 3.331	4,450	-0.000 -0.000	0,060 -0,060	0.000	-6.561
	19	31232	4.367	-0.000	-0.403	0.000	-3.176
25	21 11	1.231	60.440 59.047	-0.430 -0.430	0.000	0.000 6.000	-14,532 58,574
26		-0.119 -1.347	59.713 59.163	*0,5000 -0,500	0.000 0.000	+0,000 +0,000	-59,470

SPACE GASS 14 Path: Y:\J800 Designer: Dat Filter: No fil	)(other)\2 te: Wednes	693\LAIDS\2	0240415\4		a: 14		AI)
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21)		1,477	22,599 \$2,419	- 2. j. 1999 2. s. j. 1999		na n Na N	-(, 20) -0, 202
15	13	va 35 1990)	143.00 1400 - 1	1740 - 5000 1746 - 5000	0.55 0.550		-02.000 03.000
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1	1	11.947 4.121	011 011	- 1.2 (3 2.4 (5	- 50 - 50	1 - 13) -1,70°	$\frac{\partial_{\mathbf{x}} \phi(f)}{\partial \mathbf{y}} = \frac{1}{2} \frac{\partial_{\mathbf{x}} \phi(f)}{\partial \mathbf{y}}$
1	2. 4	542) 24255	0.000 2.000	-4.314 -4.0120	ar y 10 ki Mayah (ba	1,445	 
	ţ	1.5 %	0.000 10.000	-2.2 () -1,910	ang dinasa ang dinasa	0,000 1,504	$= \frac{1}{2} $
4	1	1120-35 9120-31		-0.100 -0.000	0	0,000) -0,000	-0,0
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ó	16	الدينية. محمد إن	-0,000 -0,000	-413-9 -41012		4, 146 1, 504	:)) A,:5-1
7	ą	-13,379 -13,479	144,320	-ປະຕິດປ ສາເຊີຍານ	Asteri Asteria	nyaan Nyaan	400.173 6.433
	20 10	12.30 33.013	-146.404 -346.551		ನ್ನಿನವನ ಸ್ಮಾತನಗ	ú,tset Q,cest	50.747 -6,530
11	9	-145,167 -142,513	1.975	=0_0000 =0_0000	-0.000 -0.000	1,0(0 -11,000	6, 133 -16, 104
1.2	10 14	147,118	-11635 61083	-0,000 -0,000	-0,000 -0,000	4.000 -0.00	-3,530 35,765
13	-1 15	618 4.069	0.000	-3.+40 -6.523	0,) 0,0	2:	
14	15	4.063 4.063 4.524	4,000 4,000	+6.523	0,000 0,000	3.185 1.445	$= \hat{A}_{+}(\hat{b}, u)$ = $\hat{A}_{+}(u - \hat{b})$
15	17	1.505	-3,000 -3,000	-3.203	-0.:::	1,504	
1.5	15	0.065	0,000	-0.191 -0.000	0.000	0.033	0.000
23	L1 26	0.21÷ 4.66)	-0,000 -0,000 -0,000	-8.340 -6.823	0.000	29.32	0,000 0,000
	26	1.063	w/1000/0	-0.213	1,100	· 107	21,1141

# SPACE GASS 14 (RX3334) (64-bit) - JEG ENGINEERING CO LTD Path: Y\\38059(other)\2653\LANDS\20240415\4 Designer: Date: Hednesday, April 24, 2024 12:45 DH, Page: 16 Filter: No filter

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Load case 22 [Linear]: Wind 3 Footing Paradian dolve: Axisi Force V-Acts Shear Z-Accis Shear Z-ROD Torsion V≂Accia Escreto 3-AX13 Stment Nerth Rode +4,502 1,910 0.470 \$3.763 -1.210 3.013 4.377 -0.003 -0.003 01026 1 1 6 0.000 0.000 -3.133 -3.866 0.000 3,176 -0.000 2 5 4 2.232 -1.638 0.000 1.075 0.024 -0.000 -0.000 3 1.133 1.046 -0.000 -0.000 4 3 1 3 2 0.046 0.000 0.000 ~0.1*5*7 -0.000 0.000 0.000  $0.024 \\ 0.000$ -0,000 0,000 13.703 -1.213 -01026 01052 5 8.819 4.377 0.009 0.009 -4.803 1.318 -0.178 -0.178 ) 11 -0.000 -0.000 -0.000 -0.000  $3.176 \\ 1.079$ 0.000 6 2,232 2,139 -3.139 -2.566  $\frac{16}{17}$ 7 -0.000 -0.000 0.000 -21.528 -3.39 -8.13 103,390 103,318 0.000 21 9 6 -104.619 -104.531 -0.000 -0.000 0.0000.000  $\frac{21}{10}$ 9, 50) 9 11 1.412 -0.000 -0.000 -0.0(s) -0.0(s) 0.000 -0,000 4.310 -11.599 9 -103.302 13 -101.797 0.000 -0.000 -4.720 11.203 12 20 34 105.096 108.134 -1.168 4.345 -0.000 -0.000 \*0**±0**00 +0**±0**00 20.277 6.561 -0, án 0 -0, án 0 0.000 0.000 +6.314 -4.659 0.065 13  $\frac{6}{19}$ 4.442 3.331 0.000 61561 -0.000 -0.000 14 9.000 9.000 -41659 -41367 15 5 3.331 -0.000 -0.000 -0.000 -0.000  $\substack{1.075\\0.024}$ 0.000 0.001 15  $\frac{12}{15}$  $1.139 \\ 1.046$ -1.638 16  $\frac{13}{27}$ 0.046 -0.000 -0.000 -0.000 -0.137 0.000 0.000 0.000 0,004 0,000 0.000 6.000 6.000 20,279 61561 23 11 26 4,440 3,331 -0.000 -0,000 -6.314 -4.659 0.000 -0,000 -0.000 -0.000 -0.000 0,000 -4.659 -4.367  $6.561 \\ 3.176$ 24  $\begin{array}{c} 26\\ 16\end{array}$ 3,331 3,230  $^{+1.0}_{-0.473}$ -0.375 0.352 25 23 11 -0.008 -0.008 8.620 0.064 5.960 0.130 21.495 21.495 26 -0.008 -0.068 -0.064 -0.620 -3,132 -3,960 -21.495 -21.495 0.470 -10.205 0.052 -0.375 6 21 NOLE REACTIONS (kH, KNS) Load case I (Linear): 55 Pageniae polyer X-Asis Force Y=Axis Force 3**-Axi**s Force X-Assa Recent Y-Axis Monont S-Acis Micent 10:14 -0.005 0.005 -0.000 -0.000 -0.000 1 7 13 14 300 \*0,000 =0,000 0,170 \*0,170 0,000 -0.000 -0.000 -0.014 0.214 0.000 -0,000 0,000 -0,000 -0,000 0,000 0.001 -0.001 0.000 0.000 0.000 6.024 6.024 2.360 2.960 17.968

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teeb	Perc	Length	Ly	6.5
1	12981454	$\Sigma(0,0,0)$	6.090	6.0%
	91,963	0.7-0		7,430
		1	Not in co	pression
6	91,893	6,706	1.480	7.130
	0.957	0.250	147,137	147.157
	4. 457		147,157	147.157
			46.139	46.793 46.793
	2.463	10,050		13.094
				5,067
	45.941		10.508	10.500
		0.350	Rot In do	operation
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24	137,604	0.750	0.067	6.06
		1,250		29.13:
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	203.731/	0.7.0	1,500	7.59
	4.000	0.350	Mat In of	ามระวงร่อเ
23	263.730	2.590	13,514	13.51
24	263.730	0,750	4.386	1,38
25	-0,563	1.250	NOT LA SE	-pressio
26	-0.563	1.150	Not in to	rpreasie.
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	0.011		74.677	74.57
4 5	-1003-042	10.000	N.t. in Co	
6	43.925	0,700	E0.747	10.74
7	3,750	0.250	74.340	71.34
9	3.750	0.220		74-34
	62+354	10.000	19,232	13.23
			19,232	14,23
	271413 26.004		2004 / 2 4 17 10	23.17 a.79
		0.700	15.643	15.04
15		0.350	74.577	74.57
23	87.418	2.560	23.473	23.17
24	65,061 21,229	0.750	8,796 44,595	8.73 (4.50
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		617.141	0.110	5.423	S. 31 9
	11	~62-4.904	10.000	list in a	niga e paísen
	1.2	2322+423	10.050	5	5.025
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	14	219,397	0,150	1.309	4.8
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	d	6*330	-0.002	0.057
	7	0.000	0.000	4.000
	٦	0.005	-0,005	4,054
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	13	0.000	0.050	0,000
	14	0.000	0,000	0.000
	15	-0.053	-0.001	0.366
	14	0.094	-0,001	0.516
			-0.001	0.092
	3.4	0.131		
	19	0.172	-0,001	0.399
	21	0.400	-0.015	0.054
	26	4,063	-0.06%	0,366
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Iten	Sect	Quy Servio	n Name	Lerath	Length	No.13	151
	3	1 393.74	1010 788	10,000	20.000	0,453	0.5
1	1	4 114 32		0,700		0.049	ò. (
1		2 114.38		0, 150	0.700	0.005	0.0
1	1			0.250	0.500	0.006	a, i
ŝ	4	2 16%. 3x		10.050	20.100	0.253	0.5
	1	2 16%.3x 2 16%.3x	4.3 CH2				
- - - - - - - - - - - - - - - - - - -	1	2 168.3x 2 193.7x	6.3 CH2 10.0 CH2	2,500	5.000	0,113	
- 	1	2 168.3x 2 193.7x 2 111.3x	4.3 CHA 16.0 CHA 5.0 CHA	2,500	1.00	0.010	$\partial \omega 0$
- - - - - - - - - - - - - - - - - - -	1	2 168.3x 2 193.7x	4.3 CHA 16.0 CHA 5.0 CHA	2,500	1,500		0.) 0.
- 	1	2 168.3x 2 193.7x 2 111.3x	4.3 CHA 16.0 CHA 5.0 CHA	2,500	1.00	0.010	$\partial \varphi_{i}$ $\partial \varphi_{i}$
- 	4 2 2 3 4 3	2 168.3x 2 193.7x 2 111.3x 2 193.7g	4.3 CH2 16.0 CH2 5.0 CH3 16.0 CH3	2,500	1.800 2.500 53.100	0.010	0.0 0.3 1.4
- 	4 2 2 3 4 3 2 2 2 2 2 2 2 2 2 2 2 2 2 2	2 168.3x 2 193.7x 2 114.3x 2 193.7z 163.3x	4.3 CH3 16.0 CH3 5.0 CH3 16.0 CH3 6.3 CH3	2,500	1,500	0.010	000 013 114 013
- 	4 2 2 3 4 3	2 168.3x 2 193.7x 2 114.3x 2 193.7z 193.7z 163.3x 103.7z	4.3 CH2 16.0 CH2 5.0 CH3 16.0 CH3	2,500	1.500 2.500 53.100	0.010	0.0 0.0 0.0 1.4 0.0 1.4 0.0 0.0

Total rads = 1.831 Center of gravity = -1.250,6.438,0.000

# SPACE GASS 14 (RC3334) (64-bit) - JZO BNGINEERING OO LTD Path: Y(\0009(other)\26931LNND3\20240415\4 Designer: Date: Wednesday, April 24, 2024 12:45 DM, Page: 25 Filter: No filter

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# SPACE GASS 14 (HK1334) (64-bit) - JKO ENGDINERRING CO LYCO Path: Y:\J9009(other)\2633\JANDB\20240415\4 Designer: Dato: Wednesday, April 24, 2024 12:45 DM, Page: 27 Filter: NG filter

NE CODOLI STEEL MEMBER DESIGN NOTES

- The algn convention used in this design report for prove section also is ahoun below. Note that it is not the same as the sign convention used in the analysis.
- x = rajor geometric axis (or angle pertion short ic)) y = ninor peraktric axis (or angle section iong leg) u = rajor principal axis x = ninor principal axis
- 2. Double angle sections are treated as solid Tee shapes.
- 3. Torsion procents are not sampldered.
- 4. Hence that affect the end connection of reskets such as block shear, bearing, tearing, bolts, weight, utifieners and the like are considered to be part of the connection design rather than the reflect design and, as such are not considered here.
- Cantilevers cannot be automatically detected. ALMAYS check that the bandlay offertive lengths calculated by the program for cantilevered moments are correct.
- 6. Initial frame imperfactions are not automatically allowed for. When applicable, you about apply notional horizontal forces of initial determinions to the anitylar model in accounting with the togetherment of the dealing code.
- 7. A compound, that provides full, partial or lateral restraint to a retrier is not automatically should not see it it is applied of resulting the force graphed to provide such restraint. To check this, the restraint forces should be added to the applied loads.

## HE CP1011 STEEL HERSER LESIGN SUCCARY (MEa, n) (\*=Calloce) (#=Salmist)

Group	Section Note		(tota) Longth		Fatlura Noda	Crit Chie	Logi Factor
1.8	193.7:10.0 CH3	275	12,500	12,500	Section	11	1.01+
54	193.7x10.0 CH2	275	12.500	12.844	Section	11	1,05#
84	168.3x6.) "HS	275	0.500	0.800	Section	12	1.33*
11	163.3x6.3 7H3	375	10.050	10.050	Section	21	1,000
124	163.325.3 CHS	275	10.050	10.058	2+stion	33	1.034
111	124.3x5.0 CHS	275	2.500	2,500	Section	11	1,59#
	111.325.0 CHS	275	2.566	2,500	Section	11	1.598
	199,7:10.0 083	\$75	2.500	2.500	Section	11	1,028

IN IGHTED	AVERAGE	LOAD PACTORS
2386	lass	NALF
1	1.832	64,845
z	1.332	5.845
2	1,330	7.974
4	1.832	2.605
5	1.332	5,643
11	1.332	1.151
12	1.832	2.630
21	1.832	1.610
22	1.332	3.751

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SECCEDENT CALCULATIONS FOR GROUP 1 (\*=Fatlur+)

Critical load case is 11, out of  $1\!-\!23$ 

Section: 193.7a10.0 (MS (Circular tube, Cold formed) (Dess: Seni-corpact

019565	Seni	-conpa	es.						
NARHENGE	Hot a	31 15a	d caaes s	rere analy	rsed non≁1	thearly (	with 2-3		
Failoc≐ Rode			Finish Posto	Actal Force	ltaj∘r Shear	Minor Shear	ttajor Roment	Minor Noment	l,oad Factor
Section	11	0.000		96.04	0.00	17.52	-75.40	0.00	
Cont-or	11	0.000	12,500	96.01			-75,10	6.00	
Shear	11	0,000		36.04	0.00	17.52	-75+40	0.00	
									<ol> <li>(1., (5))</li> </ol>

Load Load Failure Case Factor Node

3		65 \$4:	tion	- Corpre	35104	anti be	ndir	ig (9.9	1 ÷	1 4,78	Į.
4		36 Set	$C_{100}$	- Corpre	53100	and be	nati r	ug (3.9	.1 e-	<b>a</b> a . 70)	į.
5		43 - 84¢	¢1 on	- Bendilr.	g atou	t pino	r a:	ds (3.	2	11	
11	1.	04 - Sec	tios	- Compre	ssion	and the	ndili	sa (3.3	.I e	1 3.78	1
12	2.	):: Sec	tion.	- Corpre	asion	and ive	ndii	ng (8.9		g 3,78;	ļ
21	1.	45 260	ti on	- Corpre	331or.	and be	nd La	53 (8.9	.1 .	a 5.78	J
22	4.	06 dec	tion	- Compre	aston	and be	ndi:	ag (3.)	1 ÷	1 3.79	•
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P5M		275.0				93	-	430.0			
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		96.35				Ceffy					
3eff%						Zeffy Seffy		261.91	10	100 3	((.9)
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Ma:	-	-75.40	k12m	(Plaatic)		Hy	-	0.00	K247	(rias	(10)
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 $\begin{array}{rcl} \nabla u &=& (1,06) \ \forall 1 & (3,4,4) \\ k_{17} &=& (1,0+13) \ (3,7,5) \\ k_{17} &=& (1,06) \ (31 \ (3,7,5)) \\ \forall \gamma y &=& (3,6,75) \ \forall 1 \ (3,2,2,1) \\ f(\gamma) &=& (1,13) \ \text{sigma} \ (3,4,4,2) (1,13) \\ f(\gamma) &=& (3,03) \ \text{sigma} \ (3,4,4,2) (1,13) \\ \end{array}$ Governing mode - Compression and Dending (8.3.1 og (.28) (Pass)

SPACE GASS 14 (HCS114) (64-bit) - JEG SHOJHERETHO GO LTD Path: YiyaGOS(chec)/2653/LANDS/20240415)4 Designe: Date: Hednesday, April 24, 2024 12:45 PH, Page: 32 Filter: Ho filter HE CRIDII CRETULATIONS FOR GEOME 12 (\*=Tailure) Critical load came in 11, out of I-22 Saction: 165.3x6.3 CHS (Circular tute, Cold formed) Class: Seci-compact MAXHING: Not all load mades whre analysed non-linearly with P-3 MAXHING: Minor corpression effective longth was reduced by flange rostraints Failure Crit Start Finish Axial Major Hinor Major Binor Load

117-1e	Case	Pos'n	Pos'n	For te	Shear	Shear	100tent	Montent	Factor	
Section Netber	12	10.050	10.050	4.11 151.3%	-10.04 -10.04	-0.18	-0.32	-37.38 0.00 -37.38	1.93	
Streat	11	101050		4,	-10104	-0.13	-0.5.1	-51.50	(1.00)	

Losi (.)ad. Failure . Case Factor - Mode

C186	factor	15cdo				
1	48.55	Mericr -	Corp and bits	azial b	er, J	ing (8.9.2 eq 3.70)
2	6.51	Section -	Compression	and b⇒	nsili	ng (0.9.1 eq 3.78)
з	10.03	Herber -	Ourp and bis	exial by	201	ing {0.9.2 eg 8.79}
4	2.9.	Section -	Bonding abov	it mino.	r a:	xis (€.2.2.1]
5	3.02	Section -	Compression	and be	ndt.	sy (2.9.1 ∈g 8.78)
5.1	1.00	Section -	Corpression	and be	nd4.	ng (3.3.1 vg 8.78)
12	1 31	Section +	Concession	and he	ndă.	(9 (2.9,1 ÷g 8.79)
21	1 5	Section #	Commentation	and bar	ndi	ng (8.3.1 eg 8.70)
52	2.53	Galfian #	Concession.	and but	ndi.	(3 (3.3.1 ÷1 8.78)
12		occeron -	comprehension	anna is s		.,
		5.0 HPa				
				11.4		4301 MP3
		5.0 HBa				
		.050 m		Lseg	-	10.650 p (FF 2st-2st)
Lo	π Lú,	.050 m (20)				
Lex	= 10.	330 m 40m	pression)	Lay		10.050 m (Corpression)
Lex/ ex	- 31	13.6 150	pression)			175.7 (Bendling)
Art	-	0,0 00012		ALM .		0.0 mm <sup>-1</sup>
Ag	= 321	0.0 m <sup>2</sup>		A-	-	3.10.0 cm <sup>-1</sup> (7.6/9.6)
		0.0 05 2		Avv	-	1926.0 mm12 (8.0.1)
		1.15 m		Cuffy	-	34,15 mm (7.8)

AV.5	20	107010 120 2			1 - 20 - 0 - 121 - 10 - 0 - 0 - 0 - 0
Ceffx	-	34.15 m	CHEEV	-	34,15 mg (7.8)
Zetfy	-	124.78x1013 mm 3	2+ffv		124.78x1013 rm(3 (7.6)
0.45		165.00%1013 ::::3	Zaffy		105.00010 3 cmr3 (7.5)
0110		Identification of the o			
vol		1.00 (3.1.2)	Vp2	-	1.20 (3.1.2)
			6Y		
		0.00 (8.3.5.2)			
	Ŧ				
β. <b>.</b>	=	0,00 (0,015,0789,21		-	
$\lambda_{\rm R}$	-	0.00 (9.7.4)			0.00 (0.7.4)
-10V	=	0.00 (\$Pa (3.1.6)	5. IO	Ŧ	0.00 NPa (0.3.5.27A5.1)
	79	0.00 MPa (3.7.6/A8.4)	6-29	4	0.60 MPA (0.7.6/A3.1)
iter	~	1.20 (3.3.4.4)	Kev	4	1.20 (9.0.4.4)
Ŧ	π	4.14 Md [Jard-Compact]			
Fvx	#	-10.04 80	EVV	-	-0.16 23
DS .		-0.30 Elen (Plastia)			-37.38 kHn (Plastic)
		orro man (resource)	,		
Pt		0.00 88 48.03	Vo	-	0.(6) 1(1 (3.4.6)
		0.00 301 [8.7.5]	P mar	~	0.00 kH {0.7.5}
					0,00 km [5.7.5)
					305.79 KN (8.2.1)
		305.75 KN [8.2.1]			
		41.18 URN (8.2.2.1)			41.10 kHz (8.2.2.1)
195	-	0.00 kBrs (3.3.5.2)	1!p	-	0.00 ENn (8.4.4.20)

Governing node - Completion on i bending (0.3.1 og 0.78) (Pass)

# SPACE GASS 14 (HX3334) (64-bit) - JEO ENGINEERING CO LTD Path: Y:\UWOOS(Cher)\2693\LAND3\20240415\4 Designer: Date: Medheaday, April 24, 2024 12:45 PM, Page: 33 Pilter: No filter H - Spaci: YuhotAH Kit DAR AAPP 11 (1994) and

With rate to throw the  $\Omega_{\rm s}=0.5$  of 1-22  $^\circ$ 

Second and the second of the second s

1224-2011/22017 1	N. N. 46	3.15.3	CONTRACTOR OF A	有一次的最高级的专用 化	n-lisenity	ALCO FAA
260.0114.01	the se-	2.181	-1 <b>3</b> 3 (5.4004)	etta zen diale Pro-	Machine activi	Registance as concerns.

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and the second	1 1545			the second	10100	1. A.1
Perfuse Prix Phart Shidh - Alia 15-4, New State Ports - Con		142	11.0 41	11 112	11 ac. 1	a wa
[15] M. B. M. B.			4.1	- 5.2	2	
Hereit 11 Gree Labor ist	Y			- 4.1	3 3	
Storage 11 Sector 446			26.2	2,011		11.00
End Lot False						
Table Path 6 D. C.						
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2.2 22.18 Contains - Contains	antin	ansi	1.197.5	-4 · 27		
2 - 275 - 192 -						
pyse e (752-) title	0.5	2	1000.00			
ficht a 2.5-5 m	Eller a		2.000.0	FF 8 -t-	€ p+	
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				1 (2.113		
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Selds a \$1,97x1 - 3 -: 3	Secto	-	\$4.91.10	3 25 3 L	7.61	
2efto = 50.30x10000000	20113	-	(0, 67, 20)	0 co 0 (	1,44	
yrst - 1.00 (3.1.1)	¥712		112-4 (3 1910-18			
$r_{22} = - \phi_{a} \partial \theta_{-} (3_{a} \beta_{a} \beta_{a})$ $\sigma_{L2} = - \phi_{a} \partial \theta_{-} (3_{a} \beta_{a} \beta_{a} \beta_{a})$	1.7	2		. 1.27 . 1.5. 17A*	51	
αμ - στου (α. τ. τ. τ. τ. τ. μ - στου (α. τ. τ. τ. τ. βμε - στου (α. τ. τ. τ. τ. λ.: τ. στου (α. τ. τ. τ.)			0.157 (8	5. MAS	11	
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Act + 0.00 (0.7.4)	Ay	4	1,00148	, 2, 42		
μα	NUT V D AY HY		- 04 CO MP	1 (5.3.5.	11.8A	
D 20 0 0000 0000 000 0000 0000 0000	\$ 77	2	0.00.46	£ (527.67. 3.4.4)	A3.41	
Ref = 1.20 19.3.4.40	Rev	~	2.2015	.3.4.4)		
<pre>P = 4.44 (M (Jen)*comput)</pre>						
Fell = 0.000 kN	Evγ	=	5.52.00			
No = -9,10 kNo (Plastle)	347			<ul> <li>(i) aatá</li> </ul>	(7)	
Pt. a. 0.00 (N (3.6)	7.0	-	0.00 kH	(5.4.6)		
Pro - 0.00 LN (3.7.0)	5 - Y	=	0. sei ku	(8.7.5)		
Pridvan = 0.00 kN (3.7.5)	(rtybar	Ĩ.,		(*.7.5) (*.2.1)		
V <sub>11</sub> → 143.35 kH (3.2.1) M <sub>211</sub> = 14.34 kHz (3.2.1)				n (3-0-2-		
Mar a 0.00 [Na (3.3.5.2)	l'o			n (3.4.4.		
the second second second						
Governing rode - Corpusation and by	ading (	0. 9. 1	+q 3.75	) (Paco)		

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no an i	lan adaa w Tiraa dina ad	are or ally restrice to	o La J Zata	arita Andri P	an iy A tubu	sour Source	-a stele	16013	i: ra
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- 1.66 - 0.00	ul (2enietro ul	pact)							
- i.la	Rim (Plasti:			0	6, 200 - R	No (Pl		)	
	kr (3.6) Idt (3.7.5) kr (3.7.5) Kr (3.7.5)	VV  27  27  27  27  27  27  27  27  27  27	y yoar ~	1.39	00 X 0-1 X 0-1 X	ม (3) ป 3) ม (3)	1.6) 1.5) 1.5)		
	Definition         1           Bit of all of the second	<pre>&gt;&gt; Protects Bot all loss factors Bot all loss factors Critics for equivalence Critics for equivalence Critics for all factors factors is not bot all not bot is not bot factors for all f</pre>	$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$\begin{array}{cccccccccccccccccccccccccccccccccccc$	By all loss represents by a sign of the without by Big 2 - represents a set with a logic double burger with loss of the loss of the logic double burger (1) or double burger double loss loss of (1) or double loss of the loss of the loss of (2) of the loss of the loss of the loss of (2) of the loss of the loss of the loss of (2) of the loss of the loss of the loss of (2) of the loss of (2) of the loss of loss of the loss of (2) of the loss of (3) of th	$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$\begin{array}{llllllllllllllllllllllllllllllllllll$	$\begin{array}{llllllllllllllllllllllllllllllllllll$

Governing code = Corpression and bending (3.9.1 eq.3.78) (Fact)

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SPACE GASS 14 (HK3334) (64-bit) Path: Y:\J8009(other)\2693\LANDS Designer: Date: Nednesday, Apri Filter: No filter	\20240415	i\4				
HE CPD011 CALCULATIONS FOR SHOUP		ilure)				
Critical load case is 11, out of	1-22					
Section: 103.7x10.0 CHS (Circul Class: Semi-corpact	at Cuby,	cola f	≤27 € 3 )			
WARNING: Not all load cases were WARNING: Minor compression effect	ansiyse: Mive leng	ln≎n−ì ⊪th vaa	inearly s reduced	rith 2-5 by flange	: seatra	intz
Fallure Crit Start Finish A	eriat i	ator	Hinor	Major	tuner	Load
thole Case Pos'n Pos'n F	orce a	i sa:	20431	Rocent	Homent	Partor
Section 11 2.500	0.86	0.02	43.73	52.52	0.00	1.02
Barbor 11 0.600 2.500	1.72			52.00 32.00	0.00	2.25
Humber 11 0.650 2.500 Shear 11 1.250	1.72	0.00	54,56	-23.21	0.00	5.50 (1.00)
Load Failure						
Case Factor Note						
1 513.21 Service - Bending - 2 5.22 Service - Compusit 1.22 Service - Compusit 1.23 Netter - Compusit 1.24 Netter - Compusit 5 15.44 Netter - Compusit 1.25 Netter - Compusit 2.5.25 Netter - Compusit 2.5.25 Netter - Compusit 2.5.25 Netter - Compusit 2.5.25 Netter - Compusit	ntio and b B blaxist bith and b B blaxial about rej B blaxial sion and b	vending Eending bending bending or ixi bending	( 18.9.1 + (g (8.9.2 ( 13.9.1 ( (g (8.9.2 (a) (9.2.2)) (g (9.9.2)) (g (9.9.1 +	(1.8.79) (3.8.79) (4.3.73) (4.3.73) (4.3.73) (4.3.79) (4.3.79) (4.3.79)		
py . = 275.0 MPa	15 e		437.0 198			
67M + 275.0 MPa Ltot = 2.550 π				(EF Sot-1	Cel .	
La e 2.500 a (Bandida)						
Le « 2.500 m (Bending) Les « 10.500 m (Compredeing	d tey	-	2.500 5	(Complex)	sion)	
<pre>Lex = 10,500 K (compression tex/rx = 192.0 (Compression Act = 0.0 cm2</pre>	n) Le/r;		3314	(Ber, Hnd)		
Act = 0.0 m2	Acv	-	9.0 70	÷		
Ag = 5720.0 cm2	As.		ನಕ್ಕು∂ πರ್ ನಿಕ್ಕೆ ಸಂಗ	2 (7.6/8	-01	
Ava - 3462.0 mm2 Caffe - 96.85 cm	Avy	. = :	96.35 mm	17 22		
		/ =	30.30 ED	1 ( 1 ( 1 )		

Ltot	-	2.500 a	Lse	Ξ	2.500 m (EF Sot-Top)
L3		2.500 m (Banding)			
Lex	=	10.500 m (Compression)	tey	-	2.500 m (Compression)
1.4X/ r	2.4	192.2 (Compression)	Leity	-	33.4 (Bendling)
Act		ບັນປະຕາລີ	Acv	-	9.0 770 2
Ag	-	57 M.0 0702			9770.0 mm 2 (7.6/8.6)
AVA.	-	3462.0 mmh2			3460.0 pm 2 (0.2.1)
Cattr	- A.	96 <b>.85</b> mm	Caffy		96.35 mm (7.3)
Doffx		261.94x1013 (mm1)	Leffy	z	251, 4810 3 mm 3 (7.6)
34f Ex		33310(kt1013 mm13	Zeffy	=	338.00x10 3 mm 3 (2.5)
Vol.		1.00 (3.1.2)			1.20 (3.1.2)
155	-	0.00 [3.3.2]			9.00 (8.9.2)
1-57		0.00 (3.3.5.2)			0,00 (3.3.3.37A3.2)
м <sup></sup>	-	0.00 (0.3.5.3/A3.2)			0.(4) (3,3,5,3/A3.2)
8.7		0.00 (0.3.5.3/A0.2)	3		0.00 (3.3.5.3/85.2)
às -	-	6.35 (3.7.4)	λy	=	ð,ðá (8 <b>.7.1</b> )
-954		0.00 MEs (6.1.6)	ŗb	-	0.00 (Pa 10.3.5.2/A0.1)
p.::3		0.00 MFs (8.1.6/30.4)	j-2¥	5	0.00 MPa (5.7.67A8.4)
Ket	-	1.20 (9.3.4.4)	Eew	=	1,20 (9.3.1.1)
F	-	0.55 kH (Sepimoropact)			
Pro:	÷.	0.00 838	EVY		03.79 kM
118		S2.00 HRR (Ploatfr)	Hy	=	0,00 kMrn (Plaatio)
P.		0.00 334 (3.6)			0,00 231 (0.4.5)
P a s	~	0.00 89 (0.7.5)			0.00 km (8.7.5)
		0.06 (1) (3.7.5)	Payabac	=	0,00 KN (8.7.5)
V-3	7	543.67 80 (0.2.1)	Vey	=	549.67 kW (8.2.1)
11.72	-	49,14 kMm (8.2.2.1)	Кту	$\overline{\tau}$	33.14 kHn (3.2.2.1)
125		0.00 125 (8.3.5.2)			6,65 kHn (8.4.4.2b)

-Foverning node = Hending about major actio (4,2,3,1) (Pass)

.

SPACE GASS 14 (HK3334) (64-bit) - JEG ENGINEERING CO LTD Path: Y:\J8009(other)\2693\LANDS\20240415\4 Designer: Date: Wednesday, April 24, 2024 12:14 PM, Page: 1

Envelope = Load Cases 11,12 and Nodes 1,7,13,14

DISPLACEMENTS (mm, rad) (\*=Maximum, #=Minimum)

Node	Case	Τx	Ту	Τz	Rx	Ry	Rz
1	11	0.00*#	0.00*#	0.00*#	0.00*#	0.00*#	0.00*#
REACTIONS	(kN, kNm)	) (*=Maximum	, #=Minimum)				
Node	Case	Fx	Fy	FZ	Mx	My	Mz
1	12	0.01*	12.35	-6,72	-19.18#	0.67	-0.04#
7	11	-17.54#	-71.35	0,00	0.00	0.00	75.47*
13	11	-10.04	4.43	0.27*	-0.30	-3.73#	37.19
13	12		-141.17#	-20.58	-16.36	0.00	0.00
14	11	-10.04	4.43	-0.27	0.30*	3.73*	37.19
14	12	0.00	150.03*	-21.12#	-15,77	0.00	0.00

-

## SPACE GASS 14 (HK3334) (64-bit) - JEG ENGINEERING CO LTD Path: Y:\J8009(other)\2693\LANDS\20240415\4 Designer: Date: Wednesday, April 24, 2024 12:47 PM, Page: 1

DISPLACEMENTS	AT NODE 8	(mm,rad)

Case	Тx	Ту	Τz	Rx	Ry	Rz
21	0.00	0.00	0.00	0.00	0.00	0.00
22	0.00	0.00	0.00	0.00	0.00	0.00
REACTIONS	AT NODE 8 ()	n, kNm)				
Case	Fx	Fу	Fz	Mx	Му	Mz
21	-39.39	24.48	0.00	0.00	0.00	310.36
22	0.00	24.48	-39.39	-310.36	0.00	0.00

				1010		at Centroid:	Mz kNmm				leld Group:	Mz kNmm		¦pp mm⁴	250,487,468	eld Length:	Force Fzw	¥N	18.698	-9.598	-3.050		90.626	90.626		-77.338	-77.338	A D. A.A.	0.044 6.644		6.644	6.644							-							feld Group	71.350
				DataStore No: 1010		Load Eccentricity Moments at Centroid:	My kNmm				Resultant Moments on the Weld Group:	My kNmm	10,470.0	lvv mm <sup>4</sup>	125,243,734 125,243,734	Force Resultants in Each Weld Length:	Force Fyw	kN																										-		Force Resultants in the Weld Group	
						Load Eccentri	Mx kNmm				Resultant Mor	Mx kNmm	Weld Group Properties about its Centroid (x to that and $= 0.000$	luu mm <sup>1</sup>	125,243,734	Force Resulta	Force Fxw	XN Y	1.119	1.119	1110		1.633	1.633		1.633	1.633	663.4	1 633	202	1.633	1.633														Force Rest	17 540
Page No		Revision				÷	2с тт					on Method:	sort (tx*2 + ty*2 + tz*2) perties about its Centroio	lxy mm*		Node:	Stress fe	kN/mm <sup>2</sup>	0.068	0.010	0100	0.068	0.068	0.188	0.068	0.049	0.170	0.040	0.010	0.010	0.010	0.010	0.010													0.188	0.188
ă,						Load Position from Centroid:	Yc mm					Equivalent Stress Calculation Method:	Properties abo	lyy mm <sup>4</sup>	125,243,734	Stress Components & Resultant at Each Node:	Stress fz	kN/mm <sup>2</sup>	0.068	0.009	240.0	0.068	0.068	0.188	0.068	-0.049	-0.169	0.000		600.0	600.0	0.009	0.009													0.188	Valid Design Maximum Stress
Date		Job No				Load Position	Xc mm					Equivalent St	Weld Group F	ixx mm	125,243,734	onents & Resu	Stress fy	kN/mm <sup>2</sup>																												Maximum of fz & fe:	id Docion May
			ion i	22.01			До тт		"harr galathira a			Mz kNmm	awww.Wayawy. m Oriain:	Zo mm		Stress Comp	Stress fx	kN/mm <sup>2</sup>	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	2000	0.002	0.002	0.002	0.002													Maxi	1~N
Made by	and the second second	Checked	Rending and Torsion		the second s	from Origin:	Yo mm	a state and a state of the	$\setminus$	12023/9307	ents:	My kNmm	Centroid of Weid Group from Origin:	Yo mm			lxy	<b>T</b> EE	-1,136,991	1,136,991	-1,130,591 1 136 001	100100111				q	ዋ				9	q															
			hear Rendi		193.7 Weld	Load Position from Origin:	Хотт		San	A WEARAND	Applied Moments:	Mx kNmm	Centroid of W	Xo mm			lyy	am4	1,515,988	1,515,988	1,515,565	000/010/1	29,794,945	29,794,945		29,794,945	29,794,945																			Group	405 040 704 405 049 794
			letd Groun in Direct Shear	liser Reference:	Weld Description: 193.7 Weld	5:	Mag kN	17.54 ****	March 0 (11)	71.35						igth:	×	₩m	1,515,988	1,515,988	1,515,968	000'010'1							29,794,945	nto'to 1'07	29,794,945	29,794,945														ed Properties for the Weld Group	105 010 701
March 1990			Weld Grow		Welc	Applied Loads	Direction	Fx kN	Fy kN	Fz kN	ck:	gth:	0.220 kN/mm2 ed Max Stress:	0.188 kN/mm2	ence OK	Component Properties for Each Weld Length:	A xo	ee	23,479	23,479	-23,479	01107							-139,3/U	0,0,001-	139,370	139,370														ined Propertie	
sg-ltd.com				Εγ. Υ •	Ŵ		No.	wa values <sup>rx, x</sup>	Sign Convention		Strength Check	Design Strength:	0.220 KN/mm2 Induced Max Stress:	0.188	<allowable; hence="" ok<="" td=""><td>roperties for  </td><td>A yo</td><td>- E</td><td>-23,479</td><td>23,479</td><td>23,479</td><td>01107-</td><td>-139,370</td><td>-139,370</td><td></td><td>139,370</td><td>139,370</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Combin</td><td></td></allowable;>	roperties for	A yo	- E	-23,479	23,479	23,479	01107-	-139,370	-139,370		139,370	139,370																			Combin	
Email. jeg@j						•	Fz. z	*	Sigr							Component F	Area, A	шш <sup>2</sup>	485	485	485		708	708		708	708		80/ avr	001	708	708					-										1 200
Tel: 2117 9500 Fax: 3103 8077 Email: jeg@jeg-ltd.com			ana												Welds		oat, a		100	11110 SUL	17.75 17.75			3.54	3.54		32	100/11 EE4/11	3.5				100	A NUMBER OF STREET			1006 11045	Contraction of the Association			SUSAULAUNAVIANA VIA	SALAN STRUCTURE SALAN				$\left  \right $	
17 9500 Fax			Weld Groun in Y <sub>-</sub> Y Diana						~		$\rangle$			:	× Centroid	input for Weld Group Geometry:	γ	-			80 0.00 19 10 10 10 10 10 10 10 10 10 10 10 10 10					4			0 -96.85 0 -266 ec					Withman (1997)							100MARTING SCHOLEN					$\setminus$	
			and Grai							ľ	r				o Origin	put for We	Part Xo	No	1 -96.85			1 - <b>8</b> .85				_			4 0.00	4 0.00			5 0.00	and the second	1986 M.C.						7,0000,000055	and the second				ľ	
HUCH GSS.	Project	Client: Element:		*						I						<u>ج</u>	1.0	2 2	•••			1 7 7	1				300 j	<u> </u>	<u>س</u> ې	5 4			1	18	19	ଛ	3	ន	ន	27	S S	8 2	8	3 8	30	$\downarrow$	1

st. 2117 950	Tel: 2117 9500 Fax: 3103 8077 Email: jeg@jeg-td.com		1999 - 1997 - 1997 - 1998 - 19												
WWW. Control of the second sec		3 J. J. DO O	mail: jeg@jeg	Htd.com				Made by		Date		Page No			
								Checked		Job No		Revision			
								18							
Weld Group in X-Y Plane	X-Y Plane			Ev. v	Weld Group in Direct	in Direct S	hear, Bendi	Shear, Bending and Torsion	ion						
				Â.	Usc	User Reference:	0000 400 0 111-11							Latastore No: 1010	010
			E		weight	Description:	Loo.3 Weld	from Origin		Load Position	Load Position from Centroid		Load Eccentric	Load Eccentricity Moments at Centroid:	t Centroid:
	1		Z			Ma CM	Yo mm	An mm	70 mm	Xemm	Yemm	Zc mm	Mx kNmm	My kNmm	Mz kNmm
	/		Fz, z tve		Direction	CF FC					9			┢	0.0
			Sinn	Sinn Convention	Ev kN	0				0.0	$\left \right $				
•		_	1000		Fz kN	150.03				0.0	0.0		0.0-	0.0-	
8		-	S	Strength Check:			Applied Moments:	ents:					<b>Resultant Mor</b>	Resultant Moments on the Weld Group:	eld Group:
		•	٩	Design Strength:	Ë		Mx kNmm	My kNmm	Mz kNmm	Equivalent Sti	Equivalent Stress Calculation Method:	on Method:	Mx kNmm		Mz kNmm
			<u>'I</u>	0.220 kN/mm2	Vmm2		10 1 10 10 10 10 10 10 10 10 10 10 10 10	augus15770		SRSS	Sqnt [fx^2 +	Sqrt [fx^2 + fy^2 + fz^2]	-0.0	15,770.0	0.0
)			1=	Induced Max Stress:	tress:	-	Centroid of M	Centroid of Weld Group from Origin:		Weld Group F	roperties abo	ut its Centroid	i (x to u-axis ar	Weld Group Properties about its Centroid (x to u-axis angle = -8.472 deg):	;;
		:		0.177 kN/mm2	V/mm2		Xo mm	Yo mm	F	bx mm <sup>4</sup>	lyy mm	*um yx	Iuu mm	lvv mm*	lpp mm <sup>7</sup>
Origin × 0	Centroid	- Weids	<u>v</u>	<allowable: hence="" ok<="" td=""><td>nce OK</td><td></td><td>-0.0</td><td>0.0</td><td></td><td>10,445,440</td><td>10,445,440</td><td>Q</td><td>10,445,440</td><td>10,445,440</td><td>20,890,880</td></allowable:>	nce OK		-0.0	0.0		10,445,440	10,445,440	Q	10,445,440	10,445,440	20,890,880
mut for Weld Group Geometry:	un Geometr		smoonent Pro	operties for E	Component Properties for Each Weld Length:	ath:			Stress Compo	ments & Resu	Stress Components & Resultant at Each Node:	Node:	Force Resulta	Force Resultants in Each Weld Length:	ld Length:
Dout IOI men OI			Area. A	A vo	Axo	X	Å	XX	Stress fx	Stress fy	Stress fz	Stress fe	Force Fxw	Force Fyw Force Fzw	Force Fzw
ļ			2	emm <sup>3</sup>	emm.	1	<b>1</b>	t and	kN/mm <sup>2</sup>	kN/mm²	kN/mm <sup>2</sup>	kN/mm²	kN	kN	kN
				10.065	1 35.4	10 660	R50 794	-111 998	0.007	-0.000	0.177	0.177	0.880	-0.000	21.778
- 10 - 10	0.W	8 9	124	-10,200	3.970	131.011	739.443	-305.984	0.007	-0.000	0.173	0.173	0.880	-0.000	20.720
07-10	12 00	8 8	PC1	-2,000 -8,730	6315	323.876	546.577	417,982	0.007	-0000	0.160	0.160	0.880	0.000	18.676
8 5 9	592	399 989	124	-6.315	8.230	546,577	323,876	417,982	0.007	-0.000	0.140	0.140	0.880	-0.000	15.785
42.08	2.88	5.66	124	-3,970	9,584	739,443	131,011	-305,984	0.007	-0.000	0.114	0.114	0.880	000.0-	12.245
-21.78	81.28	5.66	124	-1,354	10,285	850,794	19,660	-111,998	0.007	-0.000	0.083	0.083	0.880	0.000	8.295
00:0	84.15	5.66	124	1,354	10,285	850,794	19,660	111,998	0.007	0.00	0.050	0.051	0.880	0000	4.207
21.78	81.28	5.66	124	3,970	9,584	739,443	131,011	305,984	0.007	0.000	0.017	0.019	0.880	0000	007.0
42.08	72.88	5.66	124	6,315	8,230	546,577	323,876	417,982	0.007	0.000	-0.013	0.015	0.880	0.000	-5.203
59.50	59.50	5.66	124	8,230	6,315	323,876	546,577	417,982	0.007	0.000	-0.040	0.040	0.880	0.000	-0.1/4
72.88	\$2.08	5.66	124	9,584	3,970	131.011	739,443	305,984	0.007	0.000	-0.060	0.060	0.880	0.000	-8.218
8128	21.78	5.66	124	10,285	1,354	19,660	850,794	111,998	0.007	0.000	-0.072	0.073	0.880	0,000	017.6-
84.15	0.00	5.66	124	10,285	-1,354	19,660	850,794	-111,998	0.007	0.000	120.0-	0.077	0.880	0.000	017.6-
81.28	-21.78	5.66	124	9,584	-3,970	131,011	739,443	-305,984	0.007	0.000	-0.072	0.0/3	U.880	0.000	-0.4.10
72.88	-42.08	5.66	124	8,230	-6,315	323,876	546,577	417,982	0.007	0.000	-0.060	0.060	0.000	0.000	4 0- 0 - 000
59.50	-59.50	5.66	124	6,315	-8,230	546,577	323,876	417,982	0.007	0.000	-0.040	0.040	0.660	0.000	0207.0-
42.08	-72.88	5.66	124	3,970	-9,584	739,443	131,011	-305,984	0.007	0.000	-0.013	610.0	0.880	0000	2007
21.78	-81.28	5.66	124	1,354	-10,285	850,794	19,660	-111,998	0.007	0.000	0.017	610.0	0.860	0000	4.205
0.00	-84.15	5.66	124	-1,354	-10,285	850,794	19,660	111,998	0.007	0.000	0:020	190.0	0.880	0000	12 245
-21.78	-81.28	5.66	124	-3,970	-9,584	739,443	131,011	305,984	0.007	0.000	0.083	0.444	0.000	0000	15 785
-42.08	-72.68	5.66	124	-6,315	-8,230	546,577	323,876	417,982	0.007	000.0-	4114	0.114	0.000	0000	13.676
59.50	-59.50	5.66	124	-8,230	-6,315	323,876	546,577	417,982	0.007	000.0	0,160	0,160	0000		20.20
-72.88	42.08	5.66	124	-9,584	-3,970	131,011	739,443	305,984	0.007	0.00	0.160	0.100		0000	21.72
-81.28	-21.78	5.66	124	-10,285	-1,354	19,660	850,794	111,998	0.007	000.0-	51.0	C/1-0	0.000	000.02	0
-84.15	00.0	5.66							0.007	000070-	0.177	1 0.177			
		((//Withmassier													
An and Andrew															
	JUN PRIMA DE LA CAMPANIA	14104655							-				-		
AND	A STATE AND A STAT	STATISTICS CONTRACTOR											,		
120,000,000							_			-16-05-		0.477	Earca Dec	Force Beentrants in the Weld Group	Ald Groun
				Comb	ined Propertik	Combined Properties for the Weld Group	Group		Maxi	Maximum of iz & le.		_			
												4	101 10		1 50 11 91

File: welding check - 168.3 - base.xls Tab: Welds

3.0.93
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www.hitti.com.hk			
Company:		Page: Snanifiar	0
Address: Phone i Fax:	-	E-Mail:	
Design:	09-2693-139.7 Bracket	Date:	24/4/2024
Fastening point:			

Specifier's comments:

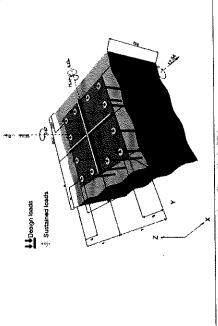
# 1 Input data

	A COMPACT AND A COMPACT A COMPACTA A	ſ
Anchor type and diameter:	HIT-RE 500 V3 + HAS-U 8.8 M16	Ġ,
Return period (service life in years):	20	
ltem number:	2223385 HAS U 8.8 M16x380 (element) / 2123403 HIT-RE 500 V3 (adhesive)	
Filling set or any suitable annular gap filling solution	p filling solution	
Effective embedment depth:	h <sub>ed.act</sub> = 320.0 mm (h <sub>ed.tme</sub> ≖ - mm)	
Material:	8.8	
Evaluation Service Report:	ETA 16/0143	
Issued I Valid:	14/5/2019   -	
Proof:	SOFA based on EN 1992-4, Chemical	
Stand-off installation:	e <sub>6</sub> = 0.0 mm (no stand-off); t = 10.0 mm	
Anchor plate <sup>R</sup> :	$I_x \times I_y \times t = 600.0 \text{ mm} \times 600.0 \text{ mm} \times 10.0 \text{ mm}$ ; (Recommended plate thickness: not calculated)	
Profile:	Cross beam. ; (L × W × T × FT) = 593.7 mm × 593.7 mm × 10.0 mm × 10.0 mm	
Base material:	cracked concrete. C25/30, t <sub>roy</sub> = 25.00 N/mm², h =500.0 mm. Temp. short/long: 40/24 °C, User-defined partial material safety factor r <sub>e</sub> = 1.500	
instaliation:	hammer drilled hole, installation condition: Dry	
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)	

 $^{\rm R}$  - The anchor calculation is based on a rigid anchor plate assumption.

with longitudinal edge reinforcement d >≖ 12.0 [mm] + close mesh (stirrups, hangers) s <= 100.0 [mm]

Geometry [mm] & Loading [kN, kNm]



# Input data and reacht must be checked for conformity with the existing conditions and for plausibility i PRICEIS Engineering ( e ) 2003-2024 Hill AG, FL-9464 Schwan Hill is a ingreater Trodemark of Hill AG. Schwan

# 

Hilti PROFIS Engineering 3.0.93

۱	⊷	54		[%]	
		2414/2024		Fire Max. Util. Anchor [%]	79
				Fire	ĉ
				Seismic	e
	Page: Specifier: F.Mail:	09-2693-139.7 Bracket		Forces [kN] / Moments [kNm]	$N = 71.350$ ; $V_x = 17.540$ ; $V_y = 6.720$ ; $M_x = 19.180$ ; $M_y = 75.470$ ; $M_x = 0.670$ ; $N_{yat} = 0.000$ ; $M_{yate} = 0.000$ ; $M_{yate} = 0.000$ ;
m.hk		-	mbination	Case Description	Combination 1
www.hilti.com.hk	Company: Address: Dhono I Cov.	Design: Fastening point:	1.1 Load combination	Case	-

Shear force x Shear force y 8	1.617 0.405	1.617 0.715 Tension	1.307 0.405	1.307 0.715 1.32	1.400 0.405 Compression	1.307 0.498	1.307 0.622 [] 1 11 10 2		1.524 0.715	1.617 0.622	1.617 0.498	1.524 0.405	
sion) Shear force	1.667	1.768	1.368	1.489	1.457	1.398	1,447	1.572	1.683	1.732	1.692	1.577	
Anchor reactions [KN] Tension force: (+Tension, -Compression) Anchor Tension force Shee	32.654	0.000	38.344	0.757	36.637	27.068	12.033	0.000	0.000	6.344	21.378	34.361	/
Anchor reactions [kN] Tension force: (+Tensio Anchor Tens	1	0	e	4	5	9	7	æ	5	6	: /	4	/

Anchor forces are calculated based on the assumption of a rigid anchor plate.

Input data and results must be checked for conformity with the existing conditions and for pleastbilly. PROFIS Engineering ( c ) 2003-2024 Hills AG. FL-9464 Schnam Hills is a regulated Trademuck of Hill AG. Schnan

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Company:		Page:	0
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	09-2693-139.7 Bracket (1)	Date:	24/4/2024
Fastening point:			

Specifier's comments:

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2223885 HAS-U 8.8 M16x380 (element) / 2123403 HIT-RE 500 V3 (adhesive)
Filling set or any suitable annular gap filling solution
h <sub>atlast</sub> = 320.0 mm (h <sub>el/imt</sub> = - mm)
8.8
ETA 16/0143
N T T N N N

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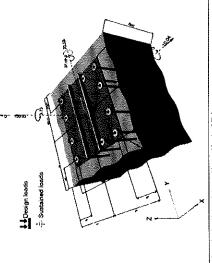
The second s

Effective embedment depth:	$h_{ac,act} = 320.0 \text{ mm} (h_{a(J)met} = - mm)$
Material:	8.8
Evaluation Service Report:	ETA 16/0143
Issued 1 Valid:	14/5/2019   -
Proof:	SOFA based on EN 1992.4, Chemical
Stand-off instailation:	$e_b = 0.0 \text{ mm}$ (no stand-off); t = 10.0 mm
Anchor plate <sup>R</sup> :	$l_x$ x $l_y$ x t = 600.0 mm x 600.0 mm x 10.0 mm; {Recommended plate thickness: not calculated}
Profile:	Double flat bar. ; {L x W x T) = 600.0 mm x 168.3 mm x 8.0 mm
Base material:	cracked concrete, C2630, $t_{cov}$ = 25.00 Nimm <sup>2</sup> ; h =500.0 mm, Temp. shortlong: 40/24 °C. User-defined partial material Safety factor $r_{c}$ = 1.500
Installation:	hammer drilled hole, installation condition: Dry
Reinforcement	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)

<sup>R</sup> - The anchor calculation is based on a rigid anchor plate assumption.

with longitudinal edge reinforcement d >= 12.0 [mm] + close mesh (stimups, hangers) s <= 100.0 [mm]

Geometry [mm] & Loading [kN, kNm]



Input data and results must be checked for contomity with the existing conditions and for plauability( PROFIS Engineering (c.) 2005-2024 Hala AG, FL-9494 Schaan Heli is a registered Trademark of Heli AG, Schaan



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	-			T_AAA	å			
Design: Fastening point:		09-2693-139.7 Bracket (1)	sket (1)	Date:	ź			24/4/2024
1.1 Load combination	oination							
Case	Description		Forces [kN] /	Forces [kN] / Moments [kNm]	Seismic		Fire Max.	Max. Util. Anchor [%]
	Combination 1	~ z°	v = 150.030; V <sub>x</sub> = M <sub>x</sub> = 16.360; M <sub>y</sub> = u <sub>a</sub> = 0.000; M <sub>xaus</sub> =	$ \begin{split} & N = 150.030;  V_{x} = 10.040;  V_{y} = 21.120; \\ & M_{x} = 16.360;  M_{y} = 37.190;  M_{z} = 37.30; \\ & N_{uus} = 0.000;  M_{zuss} = 0.000;  M_{yuus} = 0.000; \end{split} $	ê		0L	74
2 Load cae	2 Load case/Res <u>uting anchor forces</u>	anchor forc	6	,	£.	9	<b>*</b> <i>x</i>	प.
				/				
Anchor reactions [kN] Tension force: (+Tensio	Anchor reactions [kN] Tension force: (+Tension, •Compression)	ession)		/	u			a
Anchor	Tension force	Shear force	Shear force x	Shear force y		(		0
-	23.917	1.922	1.700	1697		•		
2	0.000	3.126	1.700	2.623	۳ ۳	nsion		ž
e	32.762	768.0	-0.027	0.897				
4	6.409	2.624	-0.027	2.623	.12			σn.
S	30.108	1.022	0.491	0.897				
9	24,856	1,415	-0.027	1.415				(
7	14.315	2.106	-0.027	2.105	, ,	÷	Q.	0
8	3.756	2.669	0.491	2.623	-	=	-	
Ð	0.218	2.877	1.182	2.623				
10	5.470	2.706	1.700	2.105				
1	16.011	2.212	1.700	1,415				
12	26.570	1.484	1.182	1890				
max-concrete o	max-concrete compressive strain:		0.18 [%.1	\ \				
max. concreter	max. concrete compressive stress:		5.42 IN/mm <sup>2</sup> 1					

Anchor forces are calculated based on the assumption of a rigid anchor plate.

input data and results must be checked for conformity with the existing conditions and for peurability/ PROFIS Engineering ( c ) 2000-2024 Hug AG, PL-SNM Schnan Hills is a registered Trodemerk of Hill AG, Schnan

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Coloulations by Checked by Date	JEG	Job BTS site					Job No. J8009 52693	Page Az
$N = 38.344 \text{ kN}$ $V_{s} = 3.126 \text{ kN}$ $V_{s} = 3.126 \text{ kN}$ $V_{s} = 3.126 \text{ kN}$ Try MIG Grade 8.8 GMS bolt Bolt area, A = 157 mm <sup>2</sup> Shear strength of bolt, $p_{co} = 375 \text{ N/mm^2}$ Shear capacity, $P_{c} = 560 \text{ N/mm^2}$ Shear Capacity, $P_{c} = 5.88 \text{ kN} = 87.92 \text{ kN}$ Shear per bolt, $P_{a} = A_{a} \times P_{a}$ $= 157 \times 560 / 10^{3}$ $= 58.88 \text{ kN} = 87.92 \text{ kN}$ Shear per bolt, $P_{a} = 4.42 / 58.88 + 38.34 / 87.92 = 0.51 < 1.4 \text{ OK}$ Check Combin Effect $F_{a}/P_{a} + F_{a}/P_{em} = 4.42 / 58.88 + 38.34 / 87.92 = 0.51 < 1.4 \text{ OK}$ Check for anchorage length Tension per bolt $= 326 \text{ kN}$ Check for anchorage length Tension per bolt $= 326 \text{ kN}$ Check for anchorage length Tension per bolt $= 326 \text{ kN}$ Check for anchorage length Tension per bolt $= 326 \text{ kN}$ Check for anchorage bond stress, $p_{a} = 0.55 \text{ For Type 2 : deformed bars}$ $= F_{a}/(\pi \times \varphi \times k)$ Design anchorage bond stress, $p_{a} = 0.55 \text{ KOO}$ $= 1.31 \text{ N/mm^2}$ Utimate anchorage bond stress, $p_{a} = 2.74 \text{ N/mm^2} > f_{b}$ OK	CALCULATION	Calculations by		Checke	d by A. (		)ate	,2024
$N = 38.344 \text{ kN}$ $V_{s} = 3.126 \text{ kN}$ $V_{s} = 3.126 \text{ kN}$ $V_{s} = 3.126 \text{ kN}$ Try MIG Grade 8.8 GMS bolt Bolt area, A = 157 mm <sup>2</sup> Shear strength of bolt, $p_{co} = 375 \text{ N/mm^2}$ Shear capacity, $P_{c} = 560 \text{ N/mm^2}$ Shear Capacity, $P_{c} = 5.88 \text{ kN} = 87.92 \text{ kN}$ Shear per bolt, $P_{a} = A_{a} \times P_{a}$ $= 157 \times 560 / 10^{3}$ $= 58.88 \text{ kN} = 87.92 \text{ kN}$ Shear per bolt, $P_{a} = 4.42 / 58.88 + 38.34 / 87.92 = 0.51 < 1.4 \text{ OK}$ Check Combin Effect $F_{a}/P_{a} + F_{a}/P_{em} = 4.42 / 58.88 + 38.34 / 87.92 = 0.51 < 1.4 \text{ OK}$ Check for anchorage length Tension per bolt $= 326 \text{ kN}$ Check for anchorage length Tension per bolt $= 326 \text{ kN}$ Check for anchorage length Tension per bolt $= 326 \text{ kN}$ Check for anchorage length Tension per bolt $= 326 \text{ kN}$ Check for anchorage bond stress, $p_{a} = 0.55 \text{ For Type 2 : deformed bars}$ $= F_{a}/(\pi \times \varphi \times k)$ Design anchorage bond stress, $p_{a} = 0.55 \text{ KOO}$ $= 1.31 \text{ N/mm^2}$ Utimate anchorage bond stress, $p_{a} = 2.74 \text{ N/mm^2} > f_{b}$ OK	Decian for cast in bo	\ <b>!</b> +						
$V_{1} = 3.126 \text{ kM}$ $V_{2} = 3.126 \text{ kM}$ $V_{3} = 3.126 \text{ kM}$ Try MiG Grade 8.8 GMS bolt Bolt area, A <sub>1</sub> = 157 mm <sup>2</sup> Shear strength of bolt, p <sub>10</sub> = 375 N/mm <sup>2</sup> Ernsion strength of bolt, p <sub>10</sub> = 560 N/mm <sup>2</sup> Shear Capacity, P <sub>1</sub> = A <sub>1</sub> ×P <sub>1</sub> × = A <sub>1</sub> ×P <sub>2</sub> , = A <sub>2</sub> ×P <sub>3</sub> , = A <sub>2</sub> ×P <sub>4</sub> × = A <sub>1</sub> ×P <sub>2</sub> , = A <sub>2</sub> ×P <sub>3</sub> , = A <sub>2</sub> ×P <sub>4</sub> × = N = 157 × 560 / 10 <sup>3</sup> = 58.88 kM = 87.92 kM Shear per bolt, F <sub>4</sub> = $\sqrt{(V_{1}^{4} + V_{2}^{4})}$ = $(V_{1$	-		344 VN					
$\begin{array}{llllllllllllllllllllllllllllllllllll$								
Try MIG Grade 8.8 GMS bolt Bolt area, A <sub>1</sub> = 157 mm <sup>2</sup> Sinear etrength of bolt, p <sub>14</sub> = 375 N/mm <sup>2</sup> Tension strength of bolt, p <sub>14</sub> = 560 N/mm <sup>2</sup> Shear Capacity, F <sub>0</sub> = 675 × 560 / 10 <sup>3</sup> = 58.88 kN = 87.92 kN Shear por bolt, F <sub>1</sub> = $\sqrt{r} (Y_{4}^{2} + V_{4}^{2})$ = $\sqrt{r} (Y_{4}^{2} + V_{4}^{2})$ Check Combin Effect = $\sqrt{r} (Y_{4} + V_{4}^{2})$ = $\sqrt{r} (Y_{4} + V_{4}^{2})$ Check for anchorage length Try anchorage length, = $\sqrt{r} (Y_{4} + V_{4}^{2})$ = $\sqrt{r} (Y_{4} + V_{4$								
Bolt area $A_{t}$ = 157 mm <sup>2</sup> Shear extends of bolt, $p_{ex}$ = 375 N/mm <sup>2</sup> Tension strength of bolt, $p_{ex}$ = 560 N/mm <sup>2</sup> Shear Capacity, $P_{ex}$ = $A_{tx} R_{tx}$ = $A_{tx} R_{tx}$ = $A_{tx} R_{tx}$ = $157 \times 375 / 10^{3}$ = 157 × 560 / 10 <sup>2</sup> = 58.08 kN = 87.92 kN Shear per bolt, $P_{ex}$ = $\sqrt{(X_{t}^{2} + V_{s}^{2})}$ = $\sqrt{(X_{t}^{2} + V_{s}^{2})}$ = $\sqrt{(X_{t}^{2} + V_{s}^{2})}$ = $\sqrt{(X_{t}^{2} + V_{s}^{2})}$ = $4.42 \times kN$ < $P_{ex}$ OKI Tension per bolt, $P_{ex}$ = $38 \times kN$ < $P_{ex}$ OKI Check Combin Effect $F_{u}P_{ex} + F_{u}P_{mon}$ = $4.42 / 58.68 + 36.34 / 87.92 = 0.51 < 1.4$ OK Adopt MIG Grade 8.8 GMS bolt Check for anchorage length Tension per bolt. = 38 kN Concrete cube strength, $f_{max}$ = 30 N/mm <sup>2</sup> Try anchorage length, = 400 mm Coefficient dependent on the bar type, $\beta$ = $0.5$ For Type 2: deformed bars = $F_{u} / (\pi \times 16 \times 400)$ = $1.91 N/mm^{2}$ Ultimate anchorage bond stress, $f_{max}$ = $0.5 \times \sqrt{50}$ = $2.74 N/mm^{2} > f_{max}$	<sup>ү</sup> у	<u> </u>	20 KH					
Shear strength of bolt, $p_{ab}$ = 375 N/mm <sup>2</sup> Tension strength of bolt, $p_{ab}$ = 560 N/mm <sup>2</sup> Shear Capacity, $P_{ab}$ = $A_{a} \times P_{ab}$ = $A_{a} \times P_{ab}$ = $A_{b} \times P_{ab}$ = $57 \times 575 / 10^{4}$ = $157 \times 560 / 10^{3}$ = $58.86 \text{ kN}$ = $87.92 \text{ kN}$ Shear per bolt, $F_{a}$ = $\sqrt{(V_{a}^{2} + V_{a}^{2})}$ = $4.42 \text{ kN}$ < $P_{a}$ OKI Tension per bolt, $F_{b}$ = $N$ = $38 \text{ kN}$ < $P_{a}$ OKI Check Combin Effect $F_{a}/P_{a} + F_{a}/P_{con}$ = $4.42 / 58.86 + 38.34 / 87.92$ = $0.51 < 1.4$ OK Check for anchorage length Tension per bolt Check for anchorage length Tension per bolt Check for anchorage length Tension per bolt $P_{a}/P_{a} + F_{a}/P_{con}$ = $38 \text{ kN}$ Correte cube strength, $f_{ab}$ = $30 \text{ N/mm^{2}}$ Try anchorage length, $h_{a}$ = $400 \text{ mm}$ Coefficient dependent on the bar type, $\beta$ = $0.5 \text{ For Type 2 : deformed bars}$ = $F_{a}/(\pi \times \phi \times h_{b})$ Design anchorage bond stress, $f_{a}$ = $38 \text{ kN} / (\pi \times 16 \times 400)$ = $1.91 \text{ N/mm^{2}}$ Ultimate anchorage bond stress, $f_{ab}$ = $0.5 \times \sqrt{20}$ = $2.74 \text{ N/mm^{2}} \Rightarrow f_{b}$ (Kertice the stress of the stre	Try M16 Grade 8.8	GMS bolt						
Tension strength of bolt, $p_{b}$ = 560 N/mm <sup>2</sup> Shear Capacity, $P_{e}$ Tension Capacity, $P_{e}$ = $\Lambda_{e} \times P_{ab}$ = $\Lambda_{e} \times P_{ab}$ = $\Lambda_{e} \times P_{ab}$ = $157 \times 560 / 10^{3}$ = $58,88 \text{ kN}$ = $87.92 \text{ kN}$ Shear per bolt, $F_{a}$ = $\langle (\chi^{2} + \chi^{2})$ = $\langle (\chi^{2} + \chi^{2})$ = $\langle (\chi^{2} + \chi^{2})$ = $4.42 \text{ kN}$ < $P_{e}$ OKI Tension per bolt, $F_{e}$ = N = $38 \text{ kN}$ < $P_{e}$ OKI Check Combin Effect $F_{a}/P_{a} + F_{a}/P_{non}$ = $4.42 / 58.88 + 38.34 / 87.92 = 0.51 < 1.4 OKI Check for anchorage length Tension per bolt = 38 \text{ kN}Check for anchorage lengthTension per bolt = 30 \text{ N/mm^{2}}Try anchorage length,f_{ab} = 4.00 \text{ mm}Coefficient dependent on the bar type, \beta = 0.5 \text{ For Type 2 : deformed bars}= F_{a} / (\pi \times \varphi \times k_{a})Design anchorage bond stress,f_{ba} = \beta \times \sqrt{f_{ab}}Ultimate anchorage bond stress, f_{ba} = 2.74 \text{ N/mm^{2}} \Rightarrow f_{b}OK$	Bolt area,A <sub>t</sub>	= 15						
Shear Capacity, P <sub>0</sub> Tension Capacity, P <sub>1</sub> = $A_x X_{p_4}^{\mu}$ = $A_x X_{p_4}^{\mu}$ = $A_x X_{p_4}^{\mu}$ = $157 \times 575 / 10^3$ = $157 \times 560 / 10^3$ = $58.88 \text{ kN}$ = $87.92 \text{ kN}$ Shear per bolt, F <sub>0</sub> = $\sqrt{(V_x^2 + V_y^2)}$ = $4.42 \text{ kN}$ < P <sub>0</sub> OKI Tension per bolt, F <sub>1</sub> = $38 \text{ kN}$ < P <sub>0</sub> OKI Check Combin Effect $F_y/F_0 + F_y/P_{rom}$ = $4.42 / 58.88 + 38.34 / 87.92$ = $0.51 < 1.4$ OKI Check for anchorage length Tension per bolt = $38 \text{ kN}$ Adopt MIG Grade 8,8 GMS bolt Check for anchorage length Try anchorage length, = $30 \text{ N/mm^2}$ Try anchorage length, = $30 \text{ N/mm^2}$ $= F_y / (\pi \times \varphi \times h_y)$ Design anchorage bond stress, F <sub>0</sub> = $0.5 \text{ for Type 2 : deformed bars}$ = $F_y / (\pi \times \varphi \times h_y)$ Design anchorage bond stress, F <sub>0</sub> = $0.5 \text{ km}$ Ultimate anchorage bond stress, F <sub>0</sub> = $0.5 \text{ km}$ $= 2.74 \text{ N/mm^2} \Rightarrow f_0$	Shear strength of bolt, p	ob = 3						
$= A_{x} \times P_{x}$ $= 157 \times 375 / 10^{3}$ $= 58.88 \text{ kN}$ $= 87.92 \text{ kN}$ Shear per bolt, $F_{0}$ $= \sqrt{(X^{2} + Y_{x}^{2})}$ $= \sqrt{(3.126^{2} + 3.126^{2})}$ $= 4.42 \text{ kN}$ Check combin Effect $F_{y}/P_{y} + F_{y}/P_{rom}$ $= 4.42 / 58.88 + 38.34 / 87.92 = 0.51 < 1.4 \text{ OK}$ Adopt M16 Grade 8.8 GMS bolt Check for anchorage length Tension per bolt $= 38 \text{ kN}$ Check for anchorage length The solution of the bar type, $\beta$ $= 30 \text{ N/mm^{2}}$ Try anchorage length, $\beta$ $= 4.00 \text{ mm}$ Coefficient dependent on the bar type, $\beta$ $= 0.5 \text{ For Type 2 : deformed bars}$ $= F_{x} / (\pi \times \varphi \times k)$ Design anchorage bond stress, $f_{y}$ $= 2.74 \text{ N/mm^{2}} > f_{0}$ OK	Tension strength of bolt,	р <sub>tb</sub> = 50	60 N/mm²					
$= 157 \times 575 / 10^{3} = 157 \times 560 / 10^{3}$ $= 58.88 \text{ kN} = 87.92 \text{ kN}$ Shear per bolt, F <sub>0</sub> $= \sqrt{(Y_{*}^{2} + V_{y}^{2})}$ $= \sqrt{(Y_{*}^{2} + V_{y}^{2})}$ $= 4.42 \text{ kN} < P_{0} \qquad OKI$ Tension per bolt, F <sub>t</sub> $= N$ $= 38 \text{ kN} < P_{t} \qquad OKI$ Check Combin Effect $F_{y}/P_{0} + F_{y}/P_{num} = 4.42 / 58.88 + 38.34 / 87.92 = 0.51 < 14 \qquad OK$ Adopt M16 Grade 8.8 GMS bolt Check for anchorage length Tonsion per bolt = 38 \text{ kN} Check for anchorage length Tonsion per bolt = 38 \text{ kN} Coefficient dependent on the bar type, $\beta = 0.5 \text{ For Type 2: deformed bars}$ $= F_{0} / (\pi \times \varphi \times I_{0})$ Design anchorage bond stress, f <sub>0</sub> $= 1.91 \text{ N/mm^{2}}$ Ultimate anchorage bond stress, f <sub>0</sub> $= 2.74 \text{ N/mm^{2} > f_{0}}$ OK	,		,					
$ = 58.88 \text{ kN} = 87.92 \text{ kN} $ Shear per bolt, $F_{9} = \sqrt{(Y_{*}^{2} + V_{*}^{2})} = \sqrt{(3.126^{2} + 3.126^{2})} = 4.42 \text{ kN} < P_{9} $ OKI Tension per bolt, $F_{1} = N = 38 \text{ kN} < P_{1} $ (K) Check Combin Effect $F_{9}/P_{9} + F_{9}/P_{som} = 4.42/58.86 + 38.34/87.92 = 0.51 < 14 $ OK Check for anchorage length Tension per bolt = 38 kN Concrete cube strength, $f_{cu} = 30 $ N/mm <sup>2</sup> Try anchorage length, $= 400 $ mm Coefficient dependent on the bar type, $\beta = 0.55 $ For Type 2: deformed bars $= F_{9}/(\pi \times \varphi \times I_{9})$ Design anchorage bond stress, $f_{su} = 0.5 \times \sqrt{3}0$ $= 2.74 $ N/mm <sup>2</sup> > $f_{9}$ (K)								
$= \sqrt{(V_{x}^{2} + V_{y}^{2})}$ $= \sqrt{(3.126^{2} + 3.126^{2})}$ $= 4.42 \text{ kN} < P_{e} $ OKI Tension per bolt, $F_{e}$ $= N$ $= 38 \text{ kN} < P_{e} $ OKI Check Combin Effect $F_{y}P_{g} + F_{y}P_{rom} = 4.42/58.88 + 38.34/87.92 = 0.51 < 1.4 $ OKI Check for anchorage length Tension per bolt = 38 kN Concrete cube strength, $f_{eu}$ = 30 N/mm <sup>2</sup> Try anchorage length, $I_{eu}$ = 30 N/mm <sup>2</sup> Try anchorage length, $I_{eu}$ = 30 N/mm <sup>2</sup> Utilimate anchorage bond stress, $f_{bu}$ = $\beta \times \sqrt{f_{eu}}$ Ultimate anchorage bond stress, $f_{bu}$ = $\beta \times \sqrt{f_{eu}}$ $= 2.74 \text{ N/mm^{2}} > f_{b}$								
$= \sqrt[4]{(3.126^2 + 3.126^2)}$ $= 4.42 \text{ kN} < P_s \qquad OK$ Tension per bolt, $F_t$ $= N$ $= 38 \text{ kN} < P_t \qquad OK$ Check Combin Effect $F_s/P_s + F_t/P_{nom} = 4.42/58.88 + 38.34/87.92 = 0.51 < 1.4 OK$ Adopt MIG Grade 8.8 GMS bolt Check for anchorage length Tension per bolt = 38 kN Concrete cube strength, $f_{cu} = 30 \text{ N/mm}^2$ Try anchorage length, $f_{bu} = 400 \text{ mm}$ Coefficient dependent on the bar type, $\beta = 0.5 \text{ For Type 2 : deformed bars}$ $= F_s/(\pi \times \varphi \times I_b)$ Design anchorage bond stress, $f_{pu} = 38 \times \sqrt{2} f_{cu}$ Uitimate anchorage bond stress, $f_{pu} = 38 \times \sqrt{2} f_{cu}$ $QK$	•							
$= 4.42 \text{ kN} < P_s \qquad \text{OKI}$ Tension per bolt, $F_t$ $= N$ $= 38 \text{ kN} < P_t \qquad \text{OKI}$ Check Combin Effect $F_s/P_s + F_t/P_{nom} = 4.42/58.88 + 38.34/87.92 = 0.51 < 1.4 \qquad \text{OK}$ Check for anchorage length Tension per bolt = 38 kN Concrete cube strength, $f_{cu}$ = 30 N/mm <sup>2</sup> Try anchorage length, $f_{su}$ = 400 mm Coefficient dependent on the bar type, $\beta$ = 0.5 For Type 2: deformed bars $= F_s/(\pi \times \varphi \times I_b)$ Design anchorage bond stress, $f_{tu}$ = $\beta \times \sqrt{f_{cu}}$ Ultimate anchorage bond stress, $f_{tu}$ = $0.5 \times \sqrt{30}$ $= 2.74  \text{N/mm^2} > f_b$								
Tension per bolt, $F_{t}$ = N = 38 kN < P <sub>t</sub> OKI Check Combin Effect $F_{g}/P_{g} + F_{g}/P_{nom}$ = 4.42 / 58.88 + 38.34 / 87.92 = 0.51 < 1.4 OK Adopt M16 Grade 8.8 GMS bolt Check for anchorage length Tension per bolt = 38 kN Concrete cube strength, $f_{cu}$ = 30 N/mm <sup>2</sup> Try anchorage length, $I_{g}$ = 400 mm Coefficient dependent on the bar type, $\beta$ = 0.5 For Type 2: deformed bars = $F_{g}/(\pi \times \varphi \times I_{b})$ Design anchorage bond stress, $f_{b}$ = $\beta \times \sqrt{f_{cu}}$ Ultimate anchorage bond stress, $f_{bu}$ = $0.5 \times \sqrt{30}$ = $2.74 \text{ N/mm^2} > f_{b}$	•			~				OPI
$ = N = 38 \text{ kN} < P_{t} $ OKI Check Combin Effect $ F_{p}/P_{p} + F_{t}/P_{nom} = 4.42/58.88 + 38.34/87.92 = 0.51 < 1.4 \text{ OK} $ Check for anchorage length Tension per bolt = 4.42/58.88 + 38.34/87.92 = 0.51 < 1.4 OK Check for anchorage length Tension per bolt = 38 kN Concrete cube strength, $f_{cu} = 30 \text{ N/mm}^{2}$ Try anchorage length, $l_{p} = 400 \text{ mm}$ Coefficient dependent on the bar type, $\beta = 0.5 \text{ For Type 2: deformed bars} = F_{p}/(\pi \times \varphi \times l_{p})$ Design anchorage bond stress, $f_{p} = 38 \times \sqrt{f_{cu}}$ $= 1.91 \text{ N/mm}^{2}$ Uitimate anchorage bond stress, $f_{pu} = 2.74 \text{ N/mm}^{2} > f_{p}$ OK	= 4.42 kN			< 7 <sub>5</sub>				UN
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OK	Ultimate anchorage bond	d stress,f <sub>bu</sub>						
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					Ada	pt 400mm	anchoraa	e lenati

JEG	Job	Characteristic		Job No.	<sup>Раде</sup> AZb
CALCULATION	Calculations by	Checked by A.	Chan	Date	
Shear at Jo	int				
	$f \times \pi \times \left(\frac{0.675}{2}\right)^2 \times 3$	= 7.16	t kN		
Moment at I	Joint				
= J.14 × (	(2.5-0.675-0.2- <u>0.6</u>	$(\frac{75}{2}) = 6.6$	2 KN1	M	

					1010		at Centroid:	Mz kNmm		-0.0		Weld Group:	Mz kNmm	-0.0	lao mm <sup>4</sup>	7,472,750	/eld Length:	Force Fzw		1.034	3.031	4.822	6.284	7.853	7.853	7.317	6.284	4.822	3.031	-1.034	-3.031	4,822	-6.284	-7.317	-7.853	-7.317	-6.284	-4.822	-3.031	-1.034			:		Mald Groun		-0.000
					DataStore No: 1010		Load Eccentricity Moments at Centroid	My kNmm				Resultant Moments on the Weld Group:	My kNmm		ngie = -9.400 (  vv mm <sup>2</sup>	3,736,375	Force Resultants in Each Weld Length:	Force Fyw	Å	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214	-0.214					Force Resultants in the Weld Groun	E 440	0110
							Load Eccentr	Mx kNmm				Resultant Mo	Mx kNmm	6,620.0		3.736.375	Force Results	Force Fxw		0.000	0.000	0.000	0.000	0,000	0.000	0.000	0.000	0.000	0.000	0000	000.0-	-0.000	-0.000	0.000	-0000	-0.000	-0.000	000.0-	0000-0-	-0.000					Erro Poe		0.000
oy	Page No		Revision					Zc mm					n Method:		A Its Centroid	0	lode:	Stress fe	kN/mm²	0.003	0.032	0.062	0.088	0.107	0.124	0.120	0.107	0.088	0.062	260.0	0.032	0.062	0.088	0.107	0.120	0.120	0.107	0.088	0.062	0.032	0.003				0.124	101.0	1.144
Unregistered Cop							Load Position from Centroid	<u> Yc mm</u>	0.0		-0.0		ess Calculatio	Sqrt [tx^2 + ty^2 + tz^2]	hv mm	3,736,375	tant at Each N	Stress fz	kN/mm <sup>2</sup>	0.000	0.032	0.062	0.088	101.0	0.124	0.120	0.107	0.088	0.062	0.000	-0.032	-0.062	-0.088	-0.107	-0.124	-0.120	-0.107	-0.088	-0.062	-0.032	0.000				0 124	12:0	mum arress.
User Registration No: 1	Date		Job No			1	Load Position	Xc mm		0.0	0.0		Equivalent Stress Calculation Method:	SRSS	Weld Group Properties about its Centroid (x to U-axis angle = -3.400 deg): $V_{Y} mm^{4}$ by $mm^{4}$ fxy $mm^{4}$ futum $m^{4}$ by $mm^{4}$ fr	3,736,375	Stress Components & Resultant at Each Node:	Stress fv	kN/mm <sup>2</sup>	-0.003	-0.003	-0.003	-0.003	-0.003	0003	-0.003	-0.003	-0.003	-0.003	500.0- -0.000	-0.003	-0.003	-0.003	-0.003	-0.003	-0.003	-0.003	-0.003	-0.003	-0.003	500.0-				Atovienum of fr E for		d Design Maxi
User R				ion				Zo mm				Г	mm		Γ,	1007	Stress Compo	Stress fx	kN/mm <sup>2</sup>	0:00	0.000	0.000	0.000	0.000	0000	0.000	0:000	0.000	0.000	0.000	-0.000	-0.000	-0.000	-0.000	-0.000	0000-0-	-0.000	-0.000	-0.000	-0.000	0.000				Aborda		VALE
	Made by		Checked	ig and Tors			from Origin:	Yo mm			Ì	nts:	My kNmm	0		0.0		IXV	•mm	-40,062	-109,452	-149,514	-149,514	109,452	40.062	109,452	149.514	149,514	109,452	40,062	-109,452	-149,514	-149,514	-109,452	40.062	109.452	149,514	149,514	109.452	40,062							_
				eld Group in Direct Shear, Bending and Torsion			-oad Position from Origin:	Xo mm				Applied Moments:	Mx kNmm	6620		0.0		Ŵ	, mm	304,332	264,502	195,513	115,852	46,863	7 032	46,863	115,852	195,513	264,502	304,332	264,502	195,513	115,852	46,863	7.032	46.863	115,852	195,513	264,502	304,332						oroup o mo ore	3 7:6.375
				in Direct SI	User Reference:	Weld Description:		Mag kN	0	-5.14	•						-	bx	Ţ	7,032	46,863	115,852	195,513	264,502	304,332	264,502	195,513	115,852	46,863	7 032	46,863	115,852	195,513	264,502	304,332	264.502	195,513	115,852	46,863	7,032		-			Terr At a 1Mala		3 736 375
Sha wan, Kin			ċ۲	Weld Group	Use	Weld	Applied Loads	Direction	Fx kN	Fy kN	Fz kN	k	th.	Vimm2	otress:	nce OK	ach Weld Length:	Axo	200	583	1,711	2,721	3,546	4,130	4,432	4 130	3,546	2,721	1,711	203	-1.711	-2,721	-3,546	4,130	4,432	4 130	-3.546	-2,721	-1.711	-583							c
ig st., Cheung :	g-łtd.com		at joint		Â Î	)e		5ª	1999 - 10 - 10 - 10 - 10 - 10 - 10 - 10	Sign Convention		Strength Check:	Design Strength	0.220 kN/mm2	Induced Max Stress:	<ul> <li>Allowable; Hence OK</li> </ul>	Community Properties for Ea	Avo	e un	-4,432	-4,130	-3,546	-2,721	-1,711	-002	1.711	2,721	3,546	4,130	4,432	4.130	3,546	2.721	1,711	583	-1.711	-2.721	-3,546	-4,130	-4,432						comon comon	q
Bik. E, 1/F, Cheong Fat Factory Bidg., 346 Fuk Wing St., Cheung Si	Email: jeg@jeg-td.com		weld a			L	Ϋ́,	ЕŻ, z	\$	Sign							Comoonent Pr	Area A	2 E E	95	65	65	ន	38	8 8	98	8	65	81	88	3 8	88	65	38	-99 -99 -99	3 12	8 8	33	65	<del>8</del> 8	141.142 T						1 540
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BIK, E, 1/F,	Tel: 2117 9500	-	<u>↔</u>	d Group							8					Origin x	the Wold G	X0		69.85	-67.47	-60.49	49.39	-34.93	80.91-	18.08	34.93	49.39	60,49	67.47 en ee	67.47	60.49	49.39	34.93	18.08	18.08	34.93	49.39	-60,49	-67.47	-69.85						NE: Shariar areas represent user data innut
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File: welding check - 139.7.xls Tab: Welds

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Tel 117 ES0 THS BOLT     Tel 117 ES0 THS BOLT <td>Address:</td> <td></td> <td>Cheono Fat</td> <td>Factory Bldg</td> <td>l., 346 Fuk Wi</td> <td>ng St., Cheung</td> <td>Sha Wan, Kin</td> <td></td> <td></td> <td></td> <td>User Re</td> <td>egistration No: L</td> <td>Inregistered Co</td> <td>ρy</td> <td></td> <td></td> <td></td>	Address:		Cheono Fat	Factory Bldg	l., 346 Fuk Wi	ng St., Cheung	Sha Wan, Kin				User Re	egistration No: L	Inregistered Co	ρy			
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If If I. P. V.         Muld Group In X-P Tane         Muld Group In Viewer Binary         Muld Group In Viewer Binary         Muld Group In Viewer Binary <ul> <li></li></ul>	Project: Client:	411¢	L C PH			ivint				Checkod		op vo		Revision			
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The second in the sec							Applied Loads		Load Position	from Origin:	-	Load Position	from Centroid	÷	Load Eccentric	ity Moments a	t Centroid:
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0 July         X Comm         Comm         Comm         Comm         Comm         Y mm						VIZZU *	Chroce	-	Centroid of We	eld Group fror	1	Weld Group Pi	roperties abo	ut its Centroid	(x to u-axis an	gle = 0 deg):	
0 Oper X Career mode         Control (14)         Contr							Mimm2		Xomm	Тотт	Ē	bx mm	lyy mm <sup>4</sup>	hy mm	լոս mm	w mm	lpp mm <sup>4</sup>
Mart for Weld Charmery:         Component Properties for Each Weld Length.         Arrest for Marce for Each Weld Length.         Arrest for Marce for Each Weld Length.         Arrest for Marce for Each Weld Length.         Form         Arrest for Marce for Each Weld Length.         Arrest for Marce for Marce for Each Weld Length.         Arrest for Marce for		Origin	Centroid	spieM		v. too * <ailowable; h<="" th=""><th>ence OK</th><th></th><th>0.0</th><th>0.0</th><th><math>\left  \right </math></th><th>2,046,444</th><th>2,046,444</th><th>0</th><th>2,046,444</th><th>2,046,444</th><th>4,092,888</th></ailowable;>	ence OK		0.0	0.0	$\left  \right $	2,046,444	2,046,444	0	2,046,444	2,046,444	4,092,888
Minu         Minu <thminu< th="">         Minu         Minu         <th<< td=""><td>1</td><td>1 P(2)4) 7</td><td></td><td></td><td>Commonent</td><td>Pronerties for</td><td>Each Weld Len</td><td>ath:</td><td></td><td></td><td>Stress Compo</td><td>onents &amp; Resu</td><td>tant at Each I</td><td>Node:</td><td>Force Resultar</td><td>its in Each We</td><td>Id Length:</td></th<<></thminu<>	1	1 P(2)4) 7			Commonent	Pronerties for	Each Weld Len	ath:			Stress Compo	onents & Resu	tant at Each I	Node:	Force Resultar	its in Each We	Id Length:
No.         mm         mm*         humm*         hum		put tor weiu		oat a	Area. A	Avo	A xo	1	lyy	x	Stress fx	Stress fy	Stress fz	Stress fe	Force Fxw	Force Fyw	Force Fzw
1         67:16         0.00         3.54         59:1         3.65         1.66.56         5.14.61         0.000         0.004         0.003         0.004         0.003         0.004         0.003         0.004         0.003         0.004         0.003         0.004         0.003         0.004         0.003         0.004         0.003         0.004         0.004         0.003         0.0					mm <sup>2</sup>	, mm	°em	*EE	mm*	mm <sup>4</sup>	kN/mm²	kN/mm <sup>2</sup>	kN/mm <sup>2</sup>	kN/mm <sup>2</sup>	¥N	Ŷ	ĸ
53         2.756         1,44,5         25,667         1,44,870         55,943         0.000         0.004         0.035         0.048	+			354	53	-2.967	391	3,852	166,685	-21,942	0.000	-0.004	0.000	0.004	-0.000	-0.214	1.264
53         -2.374         1.822         63,433         107.064         51.800         -0.000         -0.004         0.032         0.193           53         -1.182         2.374         107.044         53         53.832         -23.942         -0.130         -0.131         -0.135           53         -1.182         2.567         144.870         25.677         53.843         -0.000         -0.014         0.175         0.175           53         13.91         2.967         166.865         3.862         -21.942         -0.000         -0.014         0.175         0.175           53         14.16         2.766         14.810         25.667         14.816         53.943         -0.000         -0.044         0.175         0.135           53         2.374         1.822         25.867         14.810         53.433         107.064         63.453         107.064         0.185         0.035         0.135         <	- ^	+-		3.54	23	-2,765	1,145	25,667	144,870	-59,948	0000-0-	-0.004	0.048	0.048	-0.000	-0.214	3.705
53         -1,822         2,374         1107,044         63,453         -51,800         -0.000         -0.015         -0.135 <td>ر س</td> <td>1</td> <td>28.58</td> <td>3.54</td> <td>53</td> <td>-2,374</td> <td>1,822</td> <td>63,453</td> <td>107,084</td> <td>-81,890</td> <td>-0.000</td> <td>-0.004</td> <td>0.092</td> <td>0.093</td> <td>-0.000</td> <td>0.214</td> <td>0.030 7 680</td>	ر س	1	28.58	3.54	53	-2,374	1,822	63,453	107,084	-81,890	-0.000	-0.004	0.092	0.093	-0.000	0.214	0.030 7 680
53         1/145         2.765         1144.870         2.5647         3.9342         2.0000         0.004         0.175         0.175           53         391         2.967         166.665         3.852         21.942         0.000         0.004         0.175         0.175           53         391         2.967         166.655         3.852         21.942         0.000         0.004         0.175         0.175           53         1145         2.667         164.870         55.9346         0.000         0.004         0.175         0.175           53         2.374         1.822         63.453         107.084         81.890         0.000         0.004         0.165         0.185           53         2.766         1.145         2.5667         144.870         59.948         0.000         0.004         0.179         0.131           53         2.766         1.145         2.5667         144.870         59.948         0.000         0.004         0.046         0.131         0.131           53         2.766         1.145         2.5667         144.870         59.948         0.000         0.004         0.160         0.165           53         2.374 <td>4</td> <td></td> <td>40.41</td> <td>3.54</td> <td>53</td> <td>-1,822</td> <td>2,374</td> <td>107,084</td> <td>63,453</td> <td>-81,890</td> <td>0.000</td> <td>-0.004</td> <td>0.151 0.150</td> <td>0.160</td> <td>0000-0-</td> <td>-0.214</td> <td>8.943</td>	4		40.41	3.54	53	-1,822	2,374	107,084	63,453	-81,890	0.000	-0.004	0.151 0.150	0.160	0000-0-	-0.214	8.943
331         2,967         166,885         3,582         2,1,942         -0.000         -0.004         0.175         0.179           53         1,145         2,766         144,870         25,667         35,345         0,000         0.004         0.175         0.179           53         1,145         2,766         144,870         25,667         144,870         25,667         1000         0.004         0.133         0.131           53         2,766         1,146         55,647         144,870         55,948         -0.000         -0.004         0.135         0.133           53         2,766         1,146         2,784         107,064         63,453         107,064         63,453         107,064         0.000         -0.004         0.131         0.131           53         2,766         1,14,870         59,948         0.000         -0.004         0.136         0.133         0.131           53         2,776         1,145         2,766         1,44,870         25,647         -59,948         0.000         0.004         0.131         0.131           53         2,374         107,084         63,453         31,850         0.0000         0.004         0.166         0.16			49.49	3.54	53	-1,145	2,765	144,870	25,667	-59,948	000.0-	+00.0-	0470	0.170	-0.00	-0.214	9.598
S3         1.351         2.567         59.948         0.000         0.004         0.179         0.179           S3         1.822         2.374         107.004         63.453         81.880         -0.000         0.014         0.179         0.119           S3         1.822         2.374         107.004         63.453         81.880         -0.000         0.014         0.131         0.131           S3         2.766         1.145         2.567         144.870         59.948         -0.000         -0.004         0.032         0.033           S3         2.967         -391         3.862         144.870         59.948         0.000         -0.004         0.032         0.033           S3         2.967         -1,145         2.5667         144.870         59.948         0.000         -0.004         0.032         0.033           S3         2.766         -1,145         2.5667         144.870         55.667         3.852         21.942         0.000         -0.004         0.0179         0.0179           S3         2.766         144.870         25.667         144.870         55.843         91.800         0.000         -0.004         0.015           S3	9	_	-	3.54	ន	-391	2,967	100,000	3,602	24 942	0000	-0.004	0.185	0.185	-0.000	-0.214	9.598
33         1,172         2,374         107,084         63,453         81,800         -0.000         0.160         0.160         0.160         0.131           53         2,374         1,445         25,667         1445         25,667         1445         27,948         0.0193         0.0193           53         2,366         -1,145         25,667         144,870         59,348         -0.000         -0.004         0.093         0.093           53         2,766         -1,145         25,667         144,870         59,348         -0.000         -0.004         0.013         0.013           53         2,766         -1,145         25,667         144,870         59,348         -0.000         -0.004         -0.179         0.013           53         2,766         -1,145         25,667         144,870         59,348         -0.000         -0.004         -0.179         0.179           53         2,917         -1,822         2,343         114,870         55,3483         0.000         -0.004         -0.179         0.179           53         2,917         166,685         3,852         21,942         0.000         -0.014         -0.179         0.179           53 <td></td> <td></td> <td>57.15</td> <td>200</td> <td>88</td> <td>1115</td> <td>2,3UI 2765</td> <td>144.870</td> <td>25.667</td> <td>59.948</td> <td>0.000</td> <td>-0.004</td> <td>0.179</td> <td>0.179</td> <td>0.000</td> <td>-0.214</td> <td>8.943</td>			57.15	200	88	1115	2,3UI 2765	144.870	25.667	59.948	0.000	-0.004	0.179	0.179	0.000	-0.214	8.943
53         2.374         1,822         63,453         107,064         81,800         -0.000         -0.014         0.131         0.131           53         2.766         1,145         25,667         144,870         59,948         -0.000         -0.004         0.083         0.093           53         2.967         331         3,852         166,685         2.13,42         0.004         -0.004         0.003         0.004           53         2.967         331         3,852         166,685         3.14,870         -95,948         0.000         -0.044         0.131         0.131           53         2.374         -1,145         2.567         144,870         -95,948         0.000         -0.044         0.131         0.1	∞ ¢	+	N2.02	5.5	8 8	1.822	2.374	107,084	63,453	81,890	-0.00	-0.004	0.160	0.160	-0.000	-0.214	7.680
53         2.766         1,145         25.667         144,870         58,948         -0.000         -0.004         0.032         0.033           53         2.967         331         3.852         166,685         21,942         0.000         -0.004         0.048         0.048           53         2.967         -1,145         2.867         144,870         55,667         -1,942         0.000         -0.004         0.048         0.048           53         2.374         -1,182         2.3667         -144,870         55,667         -93,943         0.000         -0.004         -0.043         0.043           53         2.374         -1,1822         2.374         107,084         63,453         -1,182         0.003         -0.004         -0.131         0.131         0.131           53         2.374         107,084         63,453         317         2.967         144,870         2.667         29,448         0.004         -0.131         0.131         0.131           53         -1,145         2.3667         144,870         2.6677         29,448         0.000         0.004         -0.139         0.195           53         -1,145         2.3667         144,870         2.6	» ÷	+	40.41	354	8 8	2.374	1,822	63,453	107,084	81,890	-0.000	-0.004	0.131	0.131	-0.000	-0.214	5.893
53         2.967         331         3.852         166.665         21,942         0.000         0.004         0.048         0.048           53         2.765         -11,45         5.567         -5943         0.000         -0.004         0.003         0.004           53         2.374         -1,822         5.567         -5943         0.000         -0.004         -0.032         0.033           53         2.374         17,820         5.567         -5943         0.000         -0.004         -0.131         0.131           53         1.145         2.7765         144,870         25.667         -59,948         0.000         -0.004         -0.165         0.035           53         391         -2.967         166,685         3.852         -21,942         0.000         -0.004         -0.179         0.179           53         -391         -2.967         166,685         3.852         21,942         0.000         -0.004         -0.160         0.165           53         -31,145         2.776         144,870         25,667         53,943         0.000         -0.004         -0.179         0.179           53         -2.2374         107,084         81,890	2.5		28.58	3.54	ន	2,765	1,145	25,667	144,870	59,948	-0.000	-0.004	0.092	0.093	-0000	-0.214	207.5
63         2.967         .391         3.852         166.665         -21.942         0.000         -0.004         0.004         0.004         0.004         0.004         0.004         0.004         0.004         0.004         0.004         0.004         0.004         0.0131         0.131         <	: 2	+	14.79	3.54	ន	2,967	391	3,852	166,685	21,942	-0.000	-0.004	0.048	0.048	0000-	0.214	1 264
53         2.765         -1.145         2.5,667         144,870         -39,946         0.000         -0.004         -0.93         0.03           53         1         1.822         614/3         70,084         -31,890         0.000         -0.004         -0.131         0.131           53         1         1.822         614/370         25,647         -39,980         0.000         -0.004         -0.131         0.131           53         391         2.967         166,685         3.852         -21,942         0.000         -0.004         -0.179         0.179           53         -391         2.967         166,685         3.852         21,942         0.000         -0.004         -0.180         0.179           53         -1,145         -2.765         144,870         25,667         59,948         0.000         -0.004         -0.179         0.179           53         -1,145         -2.765         144,870         25,667         144,870         59,948         0.000         -0.004         -0.160         0.165           53         -2.374         107,084         63,453         14,870         59,948         0.000         0.004         0.166         0.165	13			3.54	53	2,967	-391	3,852	166,685	-21,942	0.000	400 O	-0.048	0.004	0000	-0.214	-3.705
33         1.2214         1.022         0.132         0.131         0	14		-	3.54	8	2,765	-1,145	20,00/	107 084	-04,840 81,800	0000	-0.004	-0.092	0.093	0.000	-0.214	-5.893
23     1142     2.766     144,870     25,667     -59,348     0.000     -0.004     0.179     0.179       53     331     2,967     166,685     3,852     21,942     0.000     -0.004     -0.179     0.179       53     -311     2,967     166,685     3,852     21,942     0.000     -0.004     -0.179     0.179       53     -1,145     -2,765     144,870     25,667     53,948     0.000     -0.004     -0.179     0.179       53     -1,145     -2,776     144,870     25,667     53,948     0.000     -0.004     -0.179     0.119       53     -1,822     2,3453     107,064     63,463     18,800     0.000     -0.004     0.160       53     -2,374     107,064     63,948     0.000     -0.004     0.179     0.131       53     -2,374     14,870     53,948     0.000     -0.004     0.063     0.038       53     -2,374     -1,4870     53,948     0.000     -0.004     0.068     0.048       53     -2,567     -3,822     144,870     53,948     0.000     -0.004     0.048       53     -2,667     -3,822     166,685     21,942     0.000     0.004 <td>15</td> <td>+</td> <td>+</td> <td>3.54</td> <td>38</td> <td>2,3/4</td> <td>-1,022</td> <td>107 084</td> <td>63 453</td> <td>-81,890</td> <td>0,000</td> <td>-0.004</td> <td>-0.131</td> <td>0.131</td> <td>0.000</td> <td>-0.214</td> <td>-7.680</td>	15	+	+	3.54	38	2,3/4	-1,022	107 084	63 453	-81,890	0,000	-0.004	-0.131	0.131	0.000	-0.214	-7.680
53         391         2,967         166,685         3,852         21,942         0.000         -0.004         -0.179         0.179         0.179           53         -391         -2,967         166,685         3,852         21,942         0.000         -0.004         -0.179         0.179         0.179           53         -1,145         -2,765         144,870         25,667         53,948         0.000         -0.004         -0.160         0.160           53         -1,822         -2,374         107,084         63,453         10,1800         -0.004         -0.179         0.131           53         -2,374         107,084         63,453         10,1800         -0.004         -0.160         0.161           53         -2,374         107,084         63,453         10,800         -0.004         -0.181         0.131           53         -2,374         1365         21,942         0.000         -0.004         -0.185           53         -2,367         -3,852         166,685         21,942         0.000         -0.004         0.048           53         -2,367         -3,852         166,685         21,942         0.000         -0.004         0.048      <	16	_	+	5.5	8 8	1 145	-2.765	144,870	25,667	-59,948	0.000	-0.004	-0.160	0.160	0.000	-0.214	-8.943
53     -311     2,967     166,685     3,852     21,942     0.000     -0.004     -0.185     0.185       53     -1,145     -2,765     144,870     25,667     58,453     81,890     0.000     -0.004     -0.179     0.179       53     -1,822     -2,374     -1,822     63,453     107,084     81,890     0.000     -0.004     0.131     0.131       53     -2,374     -1,822     63,453     107,084     81,890     0.000     -0.004     0.131     0.131       53     -2,165     -1,145     2,5667     144,870     59,948     0.000     -0.004     0.033       53     -2,267     -311     3,852     166,685     2,1942     0.000     -0.004     0.033       53     -2,267     -311     3,852     166,685     2,1942     0.000     -0.004     0.033       53     -2,267     -313     3,852     166,685     2,1942     0.000     -0.004     0.033       53     -2,267     -313     3,852     166,685     2,1942     0.000     -0.004     0.034       61     -2,667     -144,870     3,852     166,685     2,1942     0.000     -0.004     0.034       7     -2,667	2 9	+	-	256	3 33	391	-2,967	166,685	3,852	-21,942	0.000	-0.004	-0.179	0.179	0.000	-0.214	-9.598
53     -1,145     -2,765     144,870     25,667     58,948     0.000     -0.004     -0.179     0.179       53     -1,822     -2,374     107,084     63,453     81,890     0.000     -0.004     -0.160     0.160       53     -2,755     -1,145     25,667     144,870     55,948     0.000     -0.004     -0.131     0.131       53     -2,755     -1,145     25,667     144,870     55,948     0.000     -0.004     0.032     0.003       53     -2,155     -1,145     25,667     144,870     55,948     0.000     -0.004     0.033       53     -2,267     -331     3,852     166,685     21,942     0.000     -0.004     0.048       53     -2,867     -381     3,852     166,685     21942     0.000     -0.004     0.034       63     -2,967     -381     3,852     166,685     21942     0.000     -0.004     0.036       7     -2,967     -381     3,852     166,685     21942     0.000     0.004     0.048       6     -2,94     -3,852     166,685     21942     0.000     0.004     0.003       7     -2,967     -3,852     166,685     21942	2 <u>0</u>		+	3.54	8	-391	-2,967	166,685	3,852	21,942	0,000	-0.0 20	-0.185	0.185	0.000	-0.214	5 043
53     -1,822     -2,374     107,084     63,453     81,890     0.000     -0.004     -0.131     0.131       53     -2,374     -1,822     63,453     107,084     81,890     0.000     -0.004     0.131     0.131       53     -2,165     -1,145     25,667     144,870     59,480     0.000     -0.004     0.092     0.003       53     -2,165     -391     3,852     166,685     21,942     0.000     -0.004     0.048     0.044       53     -2,167     -391     3,852     166,685     21,942     0.000     -0.004     0.048       61     -3     -381     3,852     166,685     21,942     0.000     -0.004     0.048       63     -3     -381     3,852     166,685     21,942     0.000     -0.004     0.048       63     -3     -3867     -3862     166,685     21,942     0.000     -0.004     0.048       7     -2,045     -3867     -366,644     2,046,444     0     0.000     0.004       6     -2,046,444     0     2,046,444     0     2,046,444     0     1365	202	1 -14.79	╞	3.54	ន	-1,145	-2,765	144,870	25,667	59,948	0.000	-0.004	-0.179	0.179	0,000	412.0-	-0.94.0 -7 680
53     2.374     -1.822     63.453     107.084     61.830     0.000     -0.004     0.002     0.003       53     -2.765     -1.145     25.667     144.870     59.48     0.000     -0.004     0.002     0.003       53     -2.165     -331     3.852     166.655     21.942     0.000     -0.004     0.004     0.048       53     -2.667     -381     3.852     166.655     21.942     0.000     -0.004     0.048       63     -301     3.852     166.655     21.942     0.000     -0.004     0.004       64     -301     3.852     166.655     21.942     0.000     -0.004     0.004       7     -2.667     -381     3.852     166.655     21.942     0.000     -0.004     0.004       6     -2.064     -301     3.656     7.046.444     0.000     0.004     0.004       7     -2.067     -3.046.444     0     0.000     -0.056     0.185     0.185       7     -0     0     0     0     0.185     0.185     0.185       7     -0     0     0     0     0.185     0.185       7     0     0     0     0     0.185 </td <td>2</td> <td>1 -28.58</td> <td></td> <td>3.54</td> <td>ន</td> <td>-1,822</td> <td>-2,374</td> <td>107,084</td> <td>63,453</td> <td>81.890</td> <td>0.000</td> <td>-0.004</td> <td>-0.160</td> <td>0.131</td> <td></td> <td>-0.214</td> <td>-5.893</td>	2	1 -28.58		3.54	ន	-1,822	-2,374	107,084	63,453	81.890	0.000	-0.004	-0.160	0.131		-0.214	-5.893
53     -2.765     -1.145     25,667     144,870     33,742     0.000     0.004     0.048       53     -2.967     -391     3,852     166,685     21,942     0.000     -0.004     0.004       53     -2.967     -391     3,852     166,685     21,942     0.000     -0.004     0.004       6     -0.004     -0.004     0.000     -0.004     0.000     0.004       7     -0.004     -0.004     0.000     -0.004     0.000     0.004       6     -0.004     -0.004     0.000     -0.004     0.004       7     -0.004     -0.004     0.000     -0.004     0.004       7     -0.004     -0.004     -0.005     -0.004     -0.004       7     -0.004     -0.004     -0.006     -0.004     -0.004       6     -0.004     -0.006     -0.006     -0.004     -0.006       7     -0.004     -0.006     -0.006     -0.185     -0.185       7     -0.004     -0.006     -0.006     -0.185     -0.185       7     -0.004     -0.006     -0.185     -0.185     -0.185	8	1 40.41		3.54	ន	-2,374	-1,822	63,453	107,084	81,890	0.000	*00.0-	2.2	0.093	0000	-0.214	-3.705
b3     -2.16r     -331     5.002     0.000     0.000     0.004       0.001     0.000     -0.004     0.000     0.004       0.002     0.000     -0.004     0.000     0.004       0.001     0.000     -0.004     0.000     0.004       0.002     0.004     -0.004     0.000     0.004       0.002     0.005     -0.004     0.000     0.004       1.265     0     0     2.046.444     2.046.444     0	ន	$\rightarrow$	_	3.54	8	-2,765	-1,145	25,66/	144,8/U 166,685	24,940		-0.004	-0.048	0.048	0.000	-0.214	-1.264
Combined Properties for the Weld Group         Maximum of 12 & fe::         0.185         0.185           1.266         0         2.046,444         0         Valid Design Maximum Stress:         0.185	24	-	+	3.54	23	-2,967	182-	700'0	000'001	74617	0000	-0.004	0000	0.004			
Combined Properties for the Weld Group         Maximum of I2 & fe:         0.185         0.185           1.268         0         2.046,444         2.046,444         0         Valid Design Maximum Stress:         0.185	អ្ន ខ		+	5,0		· · · · · · · · · · · · · · · · · · ·											
Combined Properties for the Weld Group         Maximum of Iz & fe:         0.185         0.185           1,268         0         2,046,444         2,046,444         0         Valid Design Maximum Stress:         0.185	8 6																
Combined Properties for the Weld Group         Maximum of Iz & fe::         0.185         0.185           1,268         0         2.046,444         2.046,444         0         Valid Design Maximum Stress:         0.185	3 8																
Combined Properties for the Weld Group         Maximum of 12 & fe:         0.185         0.185           1,268         0         2.046,444         2.046,444         0         Valid Design Maximum Stress:         0.185	3 8																
Combined Properties for the Weld Group         Maximum of 2.6 feet 0         Maximum of 2.6 feet 0         0.100           1,268         0         2.046,444         2.046,444         0         Valid Design Maximum Stress:         0.185	30													0.185	Eorce Res	Itants in the V	Veld Group
1,268 0 0 2,046,444 2,046,444 0 Valid Design Maxmum Stress: 0.105 0.000 - 0.110	$\Box$	$\backslash$	$\square$	$\left  \right $		Com	bined Properti	es for the Wel	d Group	•	Max		U. 103		000	-5 140	-0.000
	NB: Sh	arted areas re	present user	data input	1,268	0	0	2.046,444	2,046,444	0	Va	id Design May	cimum stress		~~~	21-1-2-	00010-

Date Printed: 4/17/2024

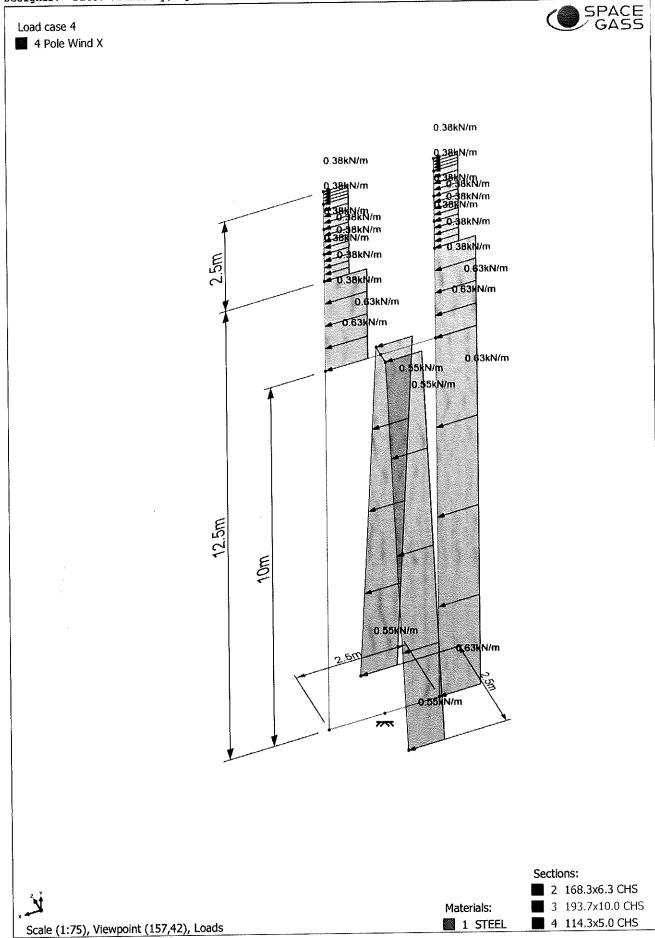
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Weld Group in Direct Shear, Bending & Torsion

File: welding check - 114.3.xls Tab: Welds

Job Job No. Page JEG Porposed Antenna Pole Connection Checking J8009 A 22 52693 Calculations by Checked by Date CALCULATION Apr 2024 A. Chan Section Modulus for Bolt Group at Joint of Antenna Group • for pole above joint Elastic modulus = 45Try CHS 114.3 x 5.0  $cm^3$ Plastic modulus = 45  $cm^3$ Try CHS 193.7 x 10.0  $\checkmark$  for pole below joint Elastic modulus = 252  $cm^3$ Plastic modulus =  $252 \text{ cm}^3$ Distance between outer most bolts about centroid of blot group Diameter of blot, d = 16 mm = 350 -30 x 2 = 290 mm Elastic modulus Ixx for bolt 1, 4 lxx for bolt 2, 3, 5, 6  $= l_c + Ay^2$  $= I_c + Ay^2$  $= \pi d^{2}/64 + \pi d^{2}/4(D/2)^{2}$ = 3217 + 4227327 +  $\pi d^2/4(D/4)^2$  $= \pi d^2/64$ = 3217 + 1056832  $= 4230544 \text{ mm}^4$  $= 1060049 \text{ mm}^4$ Total I<sub>xx</sub> for bolt group = 2 x 4230544 + 4 x 1060049  $= 1.3E+07 \text{ mm}^4$ Elastic modulus of bolt group = 1.3E+07 / (D/2)  $= 87595 \text{ mm}^3$ <u>-</u> 87.60  $cm^3$  > 52.5  $cm^3$  Elastic modulus for CHS 114.3 x 5.0 OK! Plastic modulus Plastic modulus for bolt 1, 4 Plastic modulus for bolt 2, 3, 5, 6  $= A_c y_c + A_T y_T$  $= A_c y_c + A_T y_T$  $(y_c = y_T = D/2)$  $(y_c = y_T = D/4)$  $= \pi d^2 y/4$  $= \pi d^2 y/4 + \pi d^2 y/4$ +  $\pi d^2 y/4$ + 29154 + 14577 = 29154 = 14577 mm<sup>3</sup> = 58308 = 29154  $mm^{3}$ Total  $I_{xx}$  for bolt group + 2 x 29154 = 1 x 58308  $= 116616 \text{ mm}^3$ Plastic modulus of bolt group = 116616  $mm^3$ > 45.0 cm<sup>3</sup> Plastic modulus for CHS 114.3 x 5.0 OK! = 116.6 cm<sup>3</sup> Therefore, section of bolt group is not a critical section





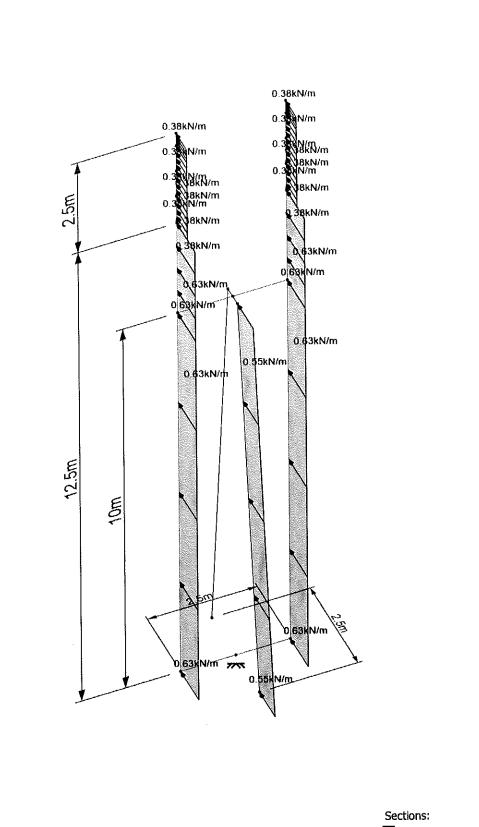
A31

SPACE ⁄ GASS

SPACE GASS 14 (HK3334) (64-bit) - JEG ENGINEERING CO LTD Path: Y:\J8009(other)\2693\LANDS\20240415\5 Designer: Date: Wednesday, April 24, 2024 4:46 PM, Page: 1



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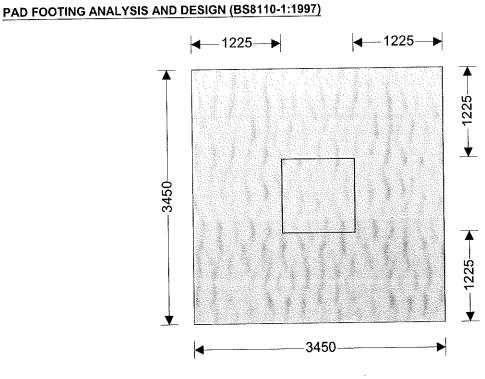


SPACE GASS 14 (HK3334) (64-bit) - JEG ENGINEERING CO LTD Path: Y:\J8009(other)\2693\LANDS\20240415\5 Designer: Date: Wednesday, April 24, 2024 4:51 PM, Page: 1

Case	Тx	Ту	Τz	Rx	Ry	R
21	0.00	0.00	0.00	0.00	0.00	0.0
22	0.00	0,00	0.00	0.00	0.00	0.0
EACTIONS	AT NODE 8 (	kN, kNm)				
EACTIONS Case	AT NODE 8 ( Fx	kN, kNm) Fy	Fz	М×	Му	м
			Fz 0.00	Мж 0.00	Му 0.00	M 251.5

<b>Tekla</b> Tedds	1 10/000				Job no.	RM P
	Calcs for				Start page no./F	Revision
	Calcs by P	Calcs date 4/24/2024	Checked by	Checked date	Approved by	Approved date

Tedds calculation version 2.0.07



Pad footing details Length of pad footing Width of pad footing Area of pad footing Depth of pad footing Depth of soil over pad footing Density of concrete

Column details

Column base length Column base width Column eccentricity in x Column eccentricity in y

### Soil details

Density of soil Design shear strength Design base friction Allowable bearing pressure

# Axial loading on column

Dead axial load on column Imposed axial load on column Wind axial load on column Total axial load on column

# Foundation loads

Dead surcharge load Imposed surcharge load  $\label{eq:2.1} \begin{array}{l} L = 3450 \mbox{ mm} \\ B = 3450 \mbox{ mm} \\ A = L \times B = 11.903 \mbox{ m}^2 \\ h = 800 \mbox{ mm} \\ h_{soil} = 0 \mbox{ mm} \\ \rho_{conc} = 24.5 \mbox{ kN/m}^3 \end{array}$ 

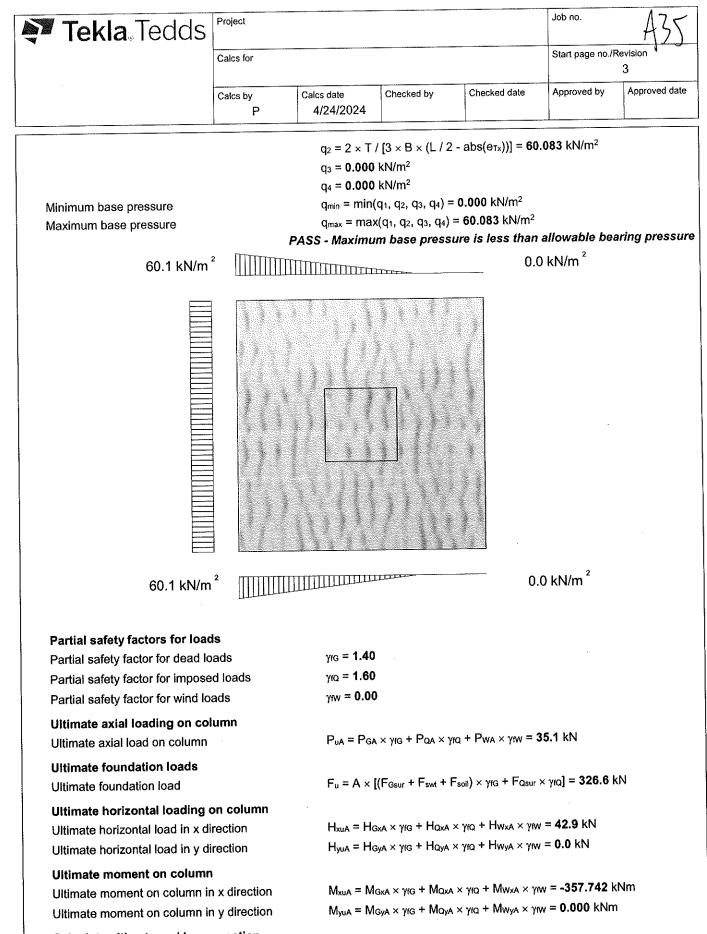
 $I_A = 1000 \text{ mm}$   $b_A = 1000 \text{ mm}$   $e_{PxA} = 0 \text{ mm}$  $e_{PyA} = 0 \text{ mm}$ 

 $\rho_{soil} = 20.0 \text{ kN/m}^3$  $\phi' = 25.0 \text{ deg}$  $\delta = 19.3 \text{ deg}$  $P_{bearing} = 63 \text{ kN/m}^2$ 

 $P_{GA} = 25.1 \text{ kN}$   $P_{QA} = 0.0 \text{ kN}$   $P_{WA} = 0.0 \text{ kN}$  $P_A = 25.1 \text{ kN}$ 

 $F_{Gsur}$  = 0.000 kN/m<sup>2</sup>  $F_{Qsur}$  = 0.000 kN/m<sup>2</sup>

Tekla Tedds ["	oject				Job no.	HJ
C	alcs for				Start page no./	Revision 2
C	alcs by P	Calcs date 4/24/2024	Checked by	Checked date	Approved by	Approved da
Pad footing self weight		F <sub>swt</sub> = h × <sub>f</sub>	Deconc = 19.600	<n m²<="" td=""><td></td><td></td></n>		
Soil self weight		F <sub>soil</sub> = h <sub>soil</sub>	× psoil = 0.000	⟨N/m²		
Total foundation load		$F = A \times (F)$	<sub>Gsur</sub> + F <sub>Qsur</sub> + Fs	<sub>swt</sub> + F <sub>soil</sub> ) = 233.3	kN	
Horizontal loading on column ba	360					
Dead horizontal load in x direction		H <sub>GxA</sub> = <b>30</b> .	7 kN			
Imposed horizontal load in x direction		H <sub>QxA</sub> = 0.0				
Wind horizontal load in x direction		$H_{WXA} = 0.0$				
Total horizontal load in x direction		H <sub>xA</sub> = 30.7				
Dead horizontal load in y direction		H <sub>GyA</sub> = 0.0				
Imposed horizontal load in y direct		H <sub>QyA</sub> = 0.0				
Wind horizontal load in y direction		H <sub>WyA</sub> = 0.0	l kN			
Total horizontal load in y direction		H <sub>yA</sub> = 0.0	٨N			
Moment on column base						
Dead moment on column in x dire	ction	M <sub>GxA</sub> = -25	5.530 kNm			
Imposed moment on column in x of		M <sub>QxA</sub> = 0.0				
Wind moment on column in x dire		MwxA = 0.0	000 kNm			
Total moment on column in x direc		M <sub>xA</sub> = -25	5.530 kNm			
Dead moment on column in y dire	ction	M <sub>GyA</sub> = 0.0	100 kNm			
Imposed moment on column in y of	direction	M <sub>QyA</sub> = 0.0	100 kNm			
Wind moment on column in y dire	ction	MwyA = 0.0				
Total moment on column in y direct	ction	M <sub>yA</sub> = 0.00	00 kNm			
Check stability against sliding						
Resistance to sliding due to base	friction					
				= <sub>Gsur</sub> + F <sub>swt</sub> + F <sub>soil</sub> )	× A], 0 kN) × I	an(δ) <b>= 90</b> .
Passive pressure coefficient		K <sub>p</sub> = (1 + :	sin(փ')) / (1 - siı	n(φ')) = <b>2.464</b>		
Stability against sliding in x dire	ection					
Passive resistance of soil in x dire	ction	$H_{xpas} = 0.5$	$5 \times K_{p} \times (h^{2} + 2)$	$(\times h \times h_{soil}) \times B \times$	<sub>ρsoil</sub> = <b>54.4</b> kN	
Total resistance to sliding in x dire	ection		$_{\rm ction}$ + H <sub>xpas</sub> = 14			
		PASS - Resistar	ice to sliding	is greater than I	orizontal loa	d in x direc
Check stability against overturn	ling in x di			000 004 Lbl		
Total overturning moment		$M_{XOT} = M_X$	$_{A}$ + H <sub>xA</sub> × h = -2	4JV.334 KINIII		
Restoring moment in x direction	n					
Foundation loading				- F <sub>soil</sub> ) × L / 2 = <b>40</b>		
Axial loading on column		•		∋ <sub>PxA</sub> ) = <b>43.263</b> kN	m	
Total restoring moment			sur + M <sub>xaxiai</sub> = 4			AL
	PASS	S - Overturning s	atety factor e	cceeds the minii	num of 1.5 in	ine x direc
Calculate pad base reaction						
Total base reaction			<sub>A</sub> = <b>258.4</b> kN			
Eccentricity of base reaction in x				H <sub>xA</sub> × h) / T = <b>-89</b>		
Eccentricity of base reaction in y		е <sub>ту</sub> = (Р <sub>А</sub> :	х еруа + Муа +	H <sub>yA</sub> × h) / T = <b>0</b> m	m	
Check pad base reaction eccen	tricity					
		abs(e <sub>Tx</sub> ) /	L + abs(e <sub>Ty</sub> ) /			
			Base	reaction acts ou	tside of midd	ie third of l
Calculate pad base pressures						
					0.083 kN/m²	



Calculate ultimate pad base reaction Ultimate base reaction

Eccentricity of ultimate base reaction in x Eccentricity of ultimate base reaction in y 
$$\begin{split} T_{u} &= F_{u} + P_{uA} = \textbf{361.7 kN} \\ e_{Txu} &= (P_{uA} \times e_{PxA} + M_{xuA} + H_{xuA} \times h) \ / \ T_{u} = \textbf{-894 mm} \\ e_{Tyu} &= (P_{uA} \times e_{PyA} + M_{yuA} + H_{yuA} \times h) \ / \ T_{u} = \textbf{0} mm \end{split}$$

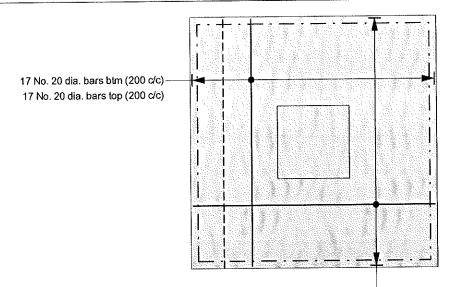
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·	s for				Start page no./	Revision 4
Calc	s by P	Calcs date 4/24/2024	Checked by	Checked date	Approved by	Approved da
Calculate ultimate pad base press	sures					
Outoutato utilitato pau nore press		$q_{1u} = 2 \times T_{u}$	u / [3 × B × (L /	2 - abs(e <sub>Txu</sub> ))] =	84.117 kN/m <sup>2</sup>	
		$q_{2u} = 2 \times T_{u}$		2 - abs(e <sub>Txu</sub> ))] =	84.117 kN/m <sup>2</sup>	
		q <sub>3u</sub> = <b>0.000</b>	) kN/m²			
		$q_{4u} = 0.000$				
Minimum ultimate base pressure				<sub>4u</sub> ) = <b>0.000</b> kN/m		
Maximum ultimate base pressure		q <sub>maxu</sub> = ma	x(q1u, q2u, q3u, 4	q <sub>4u</sub> ) = <b>84.117</b> kN	l/m²	
Calculate rate of change of base	pressure i	in x direction				
Left hand base reaction	- · ·	f <sub>uL</sub> = (q <sub>1u</sub> +	$q_{2u}$ ) × B / 2 = 2	290.202 kN/m		
Right hand base reaction			q <sub>40</sub> ) × B / 2 = 0			
Length of base reaction			/ 2 + e <sub>Txu</sub> ) = <b>2</b> 4			
Rate of change of base pressure			f <sub>uL</sub> ) / L <sub>x</sub> = <b>-116.</b>			
-		(	•			
Calculate pad lengths in x directi	on		⊦ e <sub>PxA</sub> = <b>1725</b> n	nm		
Left hand length		—	· e <sub>PxA</sub> = 1725 m · e <sub>PxA</sub> = 1725 m			
Right hand length			- UPXA - 11460 11			
Calculate ultimate moments in x			210.0	310 5 121	(2 > 1) = 404.2	27 kNm
Ultimate positive moment in x direc				$L^3/6 - F_u \times LL^2/$	(< × L) = 191.3	A FRINTI
Position of maximum negative mon		L <sub>z</sub> = 1725		13/0 5	2//201141	.vh.1.M -
Ultimate negative moment in x direct	ction	•		$< L_L^3 / 6 - F_u \times L_L$	.= / (∠ × ∟) + ⊓xi	∦A,⊼TI∓ WixuA
		-	32.064 kNm			
Calculate rate of change of base	pressure	in y direction				
Top edge base reaction			• q <sub>4u</sub> ) × L / 2 =			
Bottom edge base reaction			+ q <sub>3u</sub> ) × L / 2 =	145.101 kN/m		
Length of base reaction		L <sub>y</sub> = B = 3				
Rate of change of base pressure		$C_y = (f_{uB} -$	f <sub>u</sub> t) / L <sub>y</sub> = <b>0.00</b>	0 kN/m/m		
Calculate pad lengths in y direct	on					
Top length			- e <sub>PyA</sub> = <b>1725</b> r			
Bottom length		$L_{B} = B / 2$	+ e <sub>PyA</sub> = 1725	mm		
Calculate ultimate moments in y	direction					
Ultimate moment in y direction		M <sub>y</sub> = fuт ×	$L_T^2/2 + C_y \times L_z$	$T^3 / 6 - F_u \times L T^2 /$	(2 × B) = <b>75.0</b>	<b>35</b> kNm
-						
Material details Characteristic strength of concrete		f <sub>cu</sub> = <b>30</b> N	/mm²			
Characteristic strength of reinforce		f <sub>v</sub> = 500 N				
Characteristic strength of shear rei		•				
Nominal cover to reinforcement		Cnom = 30				
Moment design in x direction		ф <sub>хВ</sub> = 20 п	nm			
Diameter of tension reinforcement		•	nom - φxB / 2 = <b>7</b>	60 mm		
Depth of tension reinforcement			юіп – фхв / <b>∠ – 1</b>	ya sund		
Design formula for rectangular t	beams (cl					
			(B × d <sub>x</sub> <sup>2</sup> × f <sub>cu</sub> ) =	= 0.003		
		K <sub>x</sub> ' = 0.15		1	rainfarra-	t ie nat raa-
		-		compression		
Lever arm				25 - K <sub>x</sub> / 0.9)], 0		
Area of tension reinforcement requ	drod	Δ	$M_{\odot}$ / (0.87 $\times f_{\odot}$ )	× z <sub>x</sub> ) = 609 mm²		

<b>Tekla</b> Tedds F	oject				Job no.	A5
	alcs for				Start page no./I	Revision 5
C	alcs by P	Calcs date 4/24/2024	Checked by	Checked date	Approved by	Approved d
Minimum area of tension reinforce	ment	$A_{s_x_{min}} = 0$	0.0013 × B × h	= <b>3588</b> mm <sup>2</sup>		
Tension reinforcement provided		17 No. 20	dia. bars bott	om (200 centres	5)	
Area of tension reinforcement prov	vided	As_xB_prov =	$N_{xB}\times\pi\times\phi_{xB}{}^2$	/ 4 = <b>5341</b> mm²		
	PASS -	Tension reinford	ement provid	led exceeds ten	sion reinforce	ment requi
Negative moment design in x di	rection					
Diameter of tension reinforcement		φ <sub>xT</sub> = <b>20</b> m	m			
Depth of tension reinforcement		$d_x = h - c_{no}$	om - φ <sub>xT</sub> / 2 <b>= 76</b>	i0 mm		
besign formula for rectangular	beams (cl	3.4.4.4)				
Design formula for rectangular			$_{a}$ / (B × d <sub>x</sub> <sup>2</sup> × f <sub>cu</sub>	) = 0.002		
		K <sub>x</sub> ' = 0.15				
			K <sub>x</sub> < K <sub>x</sub> '	compression re	einforcement	is not requ
Lever arm		$z_x = d_x \times n$	nin([0.5 + √(0.2	25 - K <sub>x</sub> / 0.9)], 0.9	5) = <b>722</b> mm	
Area of tension reinforcement requ	uired	A <sub>s_x_req</sub> = -	$M_{xneg}$ / (0.87 $ imes$	f <sub>y</sub> × z <sub>x</sub> ) = <b>420</b> mm	1 <sup>2</sup>	
Minimum area of tension reinforce		$A_{s_x_min} = 0$	).0013 × B × h	<b>= 3588</b> mm <sup>2</sup>		
Tension reinforcement provided		17 No. 20	dia. bars top	(200 centres)		
Area of tension reinforcement pro	vided			/ 4 = <b>5341</b> mm <sup>2</sup>		
	PASS	- Tension reinford	cement provid	ded exceeds ten	sion reinforce	ement requ
Moment design in y direction						
Diameter of tension reinforcement	t	ф <sub>уВ</sub> = <b>20</b> п	ım			
Depth of tension reinforcement		$\mathbf{d}_{y} = \mathbf{h} - \mathbf{c}_{n}$	от - фхв - фув / 2	2 = <b>740</b> mm		
Design formula for rectangular	beams (cl	3.4.4.4)				
Doolgii ioitticia ioi iooittigaaa	•		$L \times d_y^2 \times f_{cu}) =$	0.001		
		K <sub>y</sub> ' = 0.15				
			• •	compression r		is not requ
Lever arm		$z_y = d_y \times r$	nin([0.5 + √(0.2	25 - K <sub>y</sub> / 0.9)], 0.9	95) = <b>703</b> mm	
Area of tension reinforcement req	uired	As_y_req =	My / (0.87 × fy >	< z <sub>y</sub> ) = <b>245</b> mm²		
Minimum area of tension reinforce	ement		0.0013 × L × h			
Tension reinforcement provided				ttom (200 centre	es)	
Area of tension reinforcement pro				<sup>2</sup> / 4 = <b>5341</b> mm <sup>2</sup>		
	PASS	- Tension reinfor	cement provi	ded exceeds ter	nsion reinforc	ement requ
Calculate ultimate shear force a	at d from I					
Ultimate pressure for shear		• • • •	-	е <sub>Рха</sub> - I <sub>A</sub> / 2 - d <sub>x</sub> ) /	/ B + q <sub>1u</sub> ) / 2	
			2 <b>71</b> kN/m²			
Area loaded for shear				<sub>«A</sub> - I <sub>A</sub> / 2 - d <sub>x</sub> ), 3 ×	< (L / 2 + e <sub>Tx</sub> )) =	= <b>1.604</b> m²
Ultimate shear force		$V_{su} = A_s $	< (q <sub>su</sub> - F <sub>u</sub> / A) =	= <b>78.338</b> kN		
Shear stresses at d from left fa	ce of colu	ımn (cl 3.5.5.2)				
Design shear stress			$(B \times d_x) = 0.0$	<b>30</b> N/mm²		
From BS 8110:Part 1:1997 - Tal	ble 3.8					
Design concrete shear stress		v <sub>c</sub> = 0.79	N/mm <sup>2</sup> × min(	3, [100 × А <sub>s_x8_pro</sub>	$_{\rm vv}$ / (B × d <sub>x</sub> )] <sup>1/3</sup> )	× max((400
				1 N/mm², 40) / 2		
Allowable design shear stress		v <sub>max</sub> = mi		√(fcu / 1 N/mm²),	5 N/mm <sup>2</sup> ) = 4	. <b>382</b> N/mm <sup>2</sup>

Tekla, Tedds	Project				Job no.	A38
	Calcs for				Start page no./	Revision 6
	Calcs by P	Calcs date 4/24/2024	Checked by	Checked date	Approved by	Approved date
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Calculate ultimate punching shear force at f	ace of column
Ultimate pressure for punching shear	$q_{puA} = q_{1u} + [(L/2 + e_{PxA} - I_A/2) + (I_A)/2] \times C_x/B - [(B/2 + e_{PyA} - b_A/2) + (b_A)/2] \times C_y/L = 0$
	<b>25.910</b> kN/m <sup>2</sup>
Average effective depth of reinforcement	$d = (d_x + d_y) / 2 = 750 mm$
Area loaded for punching shear at column	$A_{pA} = (I_A) \times (b_A) = 1.000 \text{ m}^2$
Length of punching shear perimeter	u <sub>pA</sub> = 2×(I <sub>A</sub> )+2×(b <sub>A</sub> ) = <b>4000</b> mm
Ultimate shear force at shear perimeter	$V_{puA} = P_{uA} + (F_u / A - q_{puA}) \times A_{pA} = 36.642 \text{ kN}$
Effective shear force at shear perimeter	$V_{puAeff} = V_{puA} \times [1+1.5 \times abs(M_{xuA})/(V_{puA} \times (b_A))] = 573.255 \text{ kN}$
Punching shear stresses at face of column	(cl 3.7.7.2)
Design shear stress	$v_{puA} = V_{puAeff} / (u_{pA} \times d) = 0.191 \text{ N/mm}^2$
Allowable design shear stress	$v_{max} = min(0.8N/mm^2 \times \sqrt{(f_{cu} / 1 N/mm^2)}, 5 N/mm^2) = 4.382 N/mm^2$
	PASS - Design shear stress is less than allowable design shear stres
Calculate ultimate punching shear force at	perimeter of 1.5 d from face of column
Ultimate pressure for punching shear	$q_{puA1.5d} = q_{1u} + [(L/2 + e_{PxA} - I_A/2 - 1.5 \times d) + (I_A + 2 \times 1.5 \times d)/2] \times C_x/B - [(B/2 + e_{PyA} - 1.5 \times d)/2] \times$
	ba/2-1.5×d)+(ba+2×1.5×d)/2]×Cy/L = <b>25.910</b> kN/m²
Average effective depth of reinforcement	$d = (d_x + d_y) / 2 = 750 \text{ mm}$
Area loaded for punching shear at column	$A_{pA1.5d} = (I_A + 2 \times 1.5 \times d) \times (b_A + 2 \times 1.5 \times d) = 10.563 \text{ m}^2$
Length of punching shear perimeter	$u_{pA1.5d} = 2 \times (I_A + 2 \times 1.5 \times d) + 2 \times (b_A + 2 \times 1.5 \times d) = 13000 \text{ mm}$
Ultimate shear force at shear perimeter	$V_{puA1.5d} = P_{uA} + (F_u / A - q_{puA1.5d}) \times A_{pA1.5d} = 51.274 \text{ kN}$
Effective shear force at shear perimeter	$V_{puA1.5deff} = V_{puA1.5d} \times [1+1.5 \times abs(M_{xuA})/(V_{puA1.5d} \times (b_A + 2 \times 1.5 \times d))] =$
	216.386 kN
Punching shear stresses at perimeter of 1.5	d from face of column (cl 3.7.7.2)
Design shear stress	$V_{puA1.5d} = V_{puA1.5deff} / (u_{pA1.5d} \times d) = 0.022 \text{ N/mm}^2$
From BS 8110:Part 1:1997 - Table 3.8	
Design concrete shear stress	$v_{c}$ = 0.79 N/mm <sup>2</sup> × min(3, [100 × (A_{s_xB_prov} / (B × d_x) + A_{s_yB_prov} / (L × d_x) + A_{s_yB_prov} / (L × d_x)
	dy)) / 2] <sup>1/3</sup> ) × max((800 mm / (d <sub>x</sub> + d <sub>y</sub> )) <sup>1/4</sup> , 0.67) × (min(f <sub>cu</sub> / 1 N/mm <sup>2</sup> , 4)
	/ 25) <sup>1/3</sup> / 1.25 = <b>0.339</b> N/mm <sup>2</sup>
Allowable design shear stress	v <sub>max</sub> = min(0.8N/mm² × √(f <sub>cu</sub> / 1 N/mm²), 5 N/mm²) = <b>4.382</b> N/mm²
-	PASS - v <sub>puA1.5d</sub> < v <sub>c</sub> - No shear reinforcement require

Tekla Tedds	Project				Job no.	AZJ
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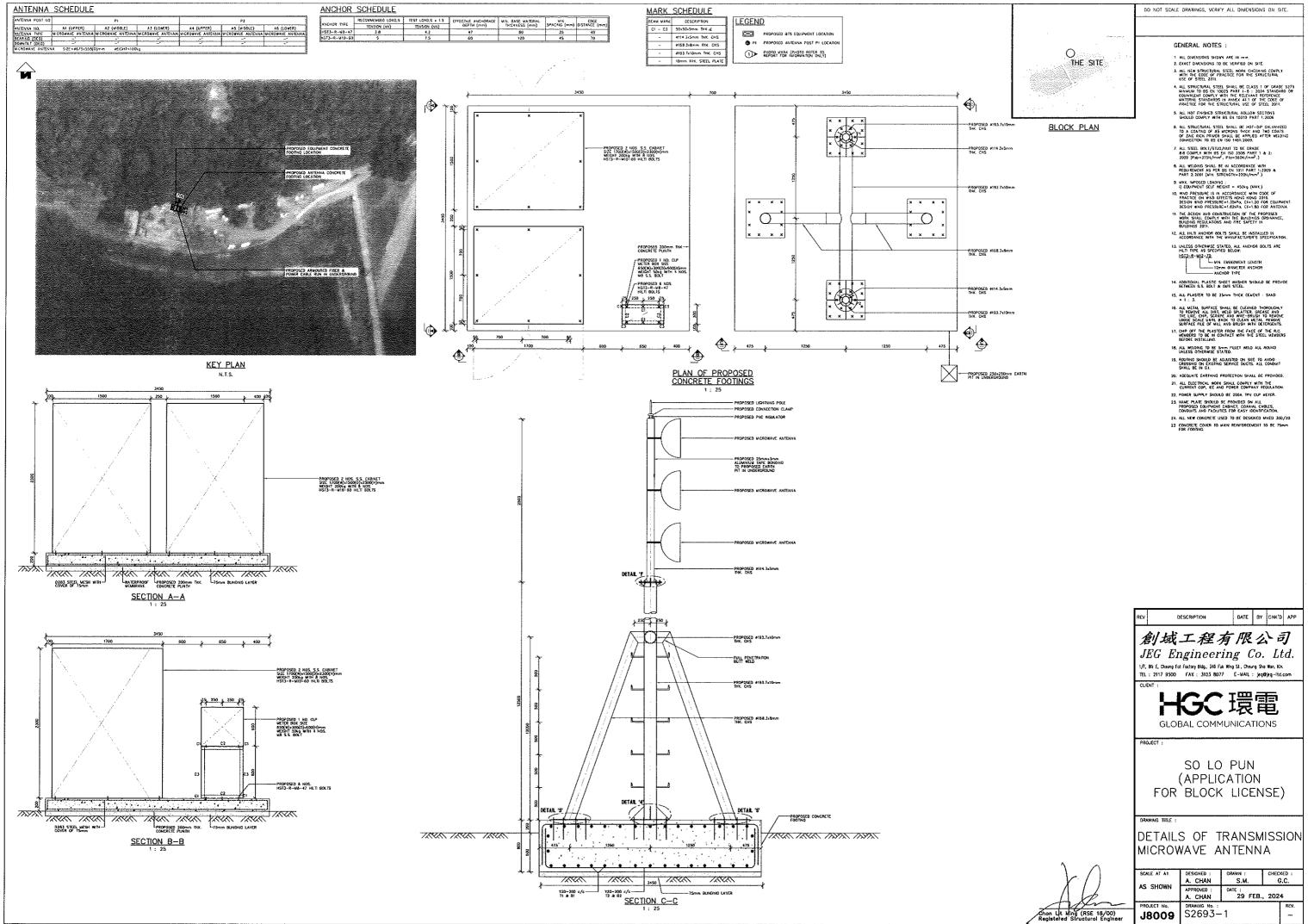
17 No. 20 dia. bars btm (200 c/c), 17 No. 20 dia. bars top (200 c/c) - - Shear at d from column face

# Site Photos

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Photo 1: Showing the proposed antenna & equipment concrete footing location



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