Client: Freehold Properties (Investments) LLP

Project: Two Storey Motel Units - Strengthening Design
86 Wellington Street, Picton

Ref: 3587-9593

Date: 5-Aug-20

CALCULATIONS

BY ANA PETROVIC

BE(Hons),MEngNZ

APPROVED BY

ARTHUR BUDVIETAS

BE (Civil) ,CMEngNZ, CPEng, IntPE(NZ)

CONTENTS

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Client:	**Freehold Properties (Investments) LLP**	Ref:	**3587-9593**
Project:	**Two Storey Motel Units - Strengthening Design**	Date:	**5/8/20**
	86 Wellington Street, Picton	BY:	**AP**
Subject:	**Philosophy**		

Philosophy

Scope

In general terms, the scope of work is as follows:
Strengthening of the motel Aldan Lodge in Picton to 80% NBS.

Means of compliance

The following standards have been used:
- NZS 1170.0:2002
- NZS 1170.1:2002
- NZS 1170.2:2002
- NZS 1170.5:2004
- NZS 3101:2006
- NZS 3602:2003
- NZS 3603:1993
- NZS 3404:1997

THE STRUCTURE

General

Strengthening of the longitudinal, North and South walls, of the two storey building to 80%. North upper floor wall strengthening of bending capacity. South upper floor strengthening around windows. At the ground floor level shear is taken by South wall. South wall strengten with new steel structure and new foundation beam.

Chosen Design Life	50 Years
Chosen Importance Level	2
Annual Probability of exceedance (inverse) Ultimate	500
Annual Probability of exceedance (inverse) Service	25

Gravity structure

Load paths:

Roof → Walls → Concrete slab → Walls → Foundations

Lateral load resisting structure

Across the building

Roof → Dividing walls → Foundations

Along the building

Roof → Walls → Concrete slab → Walls → Foundations

Significant Design Features

SOIL CONDITIONS

Assumed good ground, bearing capacity 300kPa.

DESIGN LOADS

Vertical loads

All Dead loads are listed on the gravity loads sheet at the front of these calculations.

All Live loads are listed on the gravity loads sheet at the front of these calculations.

Lateral Loads
Wind

Site wind speed N/A

Further information on wind speeds, internal pressures etc are on the main wind load sheets contained in these calculations.

Seismic loads
Analysis methodology
The seismic analysis has been completed in accordance with NZS 1170.5:2004. Design Spectra are in accordance with NZS 1170.5:2004 for site sub soil class D. Analysis has been completed using the Equivalent Static Method for bracing.

Across the building

Structural ductility factor (Ultimate)	μ	1.25
Structural Performance factor (Ultimate)	Sp	0.93

Along the building

Structural ductility factor (Ultimate)	μ	1.25
Structural Performance factor (Ultimate)	Sp	0.93

SERVICEBILITY CRITERIA

The following servicebility criteria have been chosen for the project:
Note: These are generally in line with those recommanded in NZS1170.0 Table C1.

Seismic deflections/storey drift	**Criteria**	**Phenomenon controlled**
Maximum allowable deflection (SLS)	spacing/200	Damage to cladding
Maximum allowable storey Drift (ULS)	height/40	Soft storey protection
Wind deflections		
Maximum allowable lateral deflection (SLS)	spacing/200	Damage to cladding
Maximum allowable vertical deflection (SLS)	span/200	Damage to cladding/finishes
Gravity deflections		
Maximum allowable deflection (SLS)	span/500	Visual sag

SOFTWARE

The following computer applications were used for the design:

Analysis type	Software used
Stuctural analysis	Excel 2009
Structural design	Excel 2009

Significant or Special Construction Features

None.

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Project: **Two Storey Motel Units - Strengthening Desigr** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **Gravity Loads**

Sheet No.: **4**

Loads

Roof

Corr/Trimdek CS	0.059
Timber 20.6	0.092
Timber 200.2	0.028
Timber 100.1.2	0.034
	0.212 kPa
0.212 / Cos 13 =	0.218 kPa

Dividing wall

20S 2nd core	3.700
	3.700 kPa

Unispan floor

175Inter (275)	2.700
	2.700 kPa

Upper floor

21mm Particie Bd	0.150
Timber 20.6	0.092
90. Nogs & plates	0.067
Battens 05 1.2	0.011
Rockwool Insu.	0.002
Gib Board 13	0.120
	0.442 kPa

Timber wall

Portland Plaster	0.290
HardieBacker	0.073
Battens 05 1.2	0.011
Timber 10.4	0.069
Rockwool Insu.	0.002
Gib Board 10	0.090
	0.535 kPa

Live loads

A2 other rooms	2.00 kPa

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Project: **Two Storey Motel Units - Strengthening Desigr** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **EQ Static 1170.5**

Seismic Loads to NZS 1170.5 Sheet No.: **6**

Ref:	Design	Output

Design working life **50 Years**

Importance level 2
Annual Probability of exceedance (inverse) Ultimate 500
Annual Probability of exceedance (inverse) Service 25

Element			Area/length	Load Kpa	Total kN
Roof			21.50	0.22	4.68
Timber wall			3.00	0.54	1.61
Dividing wall			4.65	3.70	17.21
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
	1.00	0.30	0.00	0.00	0.00
					23.49 kN

Element			Area/length	Load Kpa	Total kN
Beam second floor			23.60	1.92	45.31
Dividing wall			14.20	3.70	52.54
Timber wall			3.00	0.54	1.61
Roof			21.50	0.22	4.68
Block walls			3.80	3.35	12.73
			0.00	0.00	0.00
A2 other rooms	1.00	0.30	12.90	2.00	25.80
	1.00	0.40	0.00	0.00	0.00
					124.61 kN

Element			Area/length	Load Kpa	Total Kn
Dividing wall			18.70	3.70	69.19
Unispan floor			34.40	2.70	92.88
Balcony			10.75	3.16	34.00
Beam first floor			21.50	2.16	46.44
Block walls			8.80	3.35	29.48
			0.00	0.00	0.00
A2 other rooms	0.75	0.30	45.15	2.00	90.30
	1.00	0.40	0.00	0.00	0.00
					292.21 kN

Live load reduction

Total floor area 45.2

$$.3+\frac{3}{\sqrt{A}} = 0.746$$

But not less than .5

Total building weight
440.30 kN

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86 Wellington Street, Picton BY: **AP**
Subject: **EQ Static 1170.5**

Sheet No.: **7**

Ref:	Design				Output
	Soil type				
	D. Deep or soft soil				
	Across the building				
	Period of building across the building		0.40		
	Does the seismic bracing have ductile capabilities but is designed as nominally ductile				☐
	Structural ductility factor (Ultimate)	μ =	1.25		
	Structural ductility factor (Service SLS1)	μ =	1.25		
	Hazard Factor Picton	Z =	0.3		
	Return period factor	Ru =	1.00		
	Return period factor	Rs =	0.25		
	Structural Performance factor (Ultimate)	Sp =	0.93		
	Structural Performance factor (Service)	Sp =	0.70		
	Spectral Shape Factor (across)	Ch(T) =	3.00		
	Near Fault factor	N(T,D) =	1.0	16	
	Elastic site spectra (Ultimate)	C(T) =	0.90		
	Elastic site spectra (Service)	C(T) =	0.23		
	Ultimate	kμ =	1.14		
	Service	kμ =	1.14		
	Ultimate				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.73		But not less than 0.030Ru
	Ultimate force across the building	Cd(T1) x Wi =	320.73	kN Total	
	Service				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.14		
	Service force across the building	Cd(T1) x Wi =	60.68	kN Total	
	Along the building				
	Period of building along the building		0.40		
	Does the seismic bracing have ductile capabilities but is designed as nominally ductile				☐
	Structural ductility factor (Ultimate)	μ =	1.25		
	Structural ductility factor (Service SLS1)	μ =	1.25		
	Structural Performance factor (Ultimate)	Sp =	0.93		
	Spectral Shape Factor (across)	Ch(T) =	3.00		
	Near Fault factor	N(T,D) =	1.0		
	Elastic site spectra (Ultimate)	C(T) =	0.90		
	Elastic site spectra (Service)	C(T) =	0.23		
	Ultimate	kμ =	1.14		
	Service	kμ =	1.14		
	Ultimate				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.73		But not less than 0.030Ru
	Ultimate force along the building	Cd(T1) x Wi =	320.73	kN Total	
	Service				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.18		
	Service force across the building	Cd(T1) x Wi =	60.68	kN Total	

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Project: **Two Storey Motel Units - Strengthening Desigr** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **EQ Static Forces Along**

Seismic Loads to NZS 1170.5 Sheet No.: **8**

Ref:	Design				Output	
	Seismic weight at level i	Wi	23.49	kN		
	Height at level i	hi	7.0	m		
	Seismic weight at level I	Wi	124.61	kN		
	Height at level I	hi	5.0	m		
	Seismic weight at level I	Wi	292.21	kN		
	Height at level I	hi	2.7	m		
	Seismic weight at level I	Wi	0.00	kN		
	Height at level I	hi	0.0	m		
	Seismic weight at level i	Wi	0.00	kN		
	Height at level i	hi	0.0	m		
	Sum of Wihi		1576.4			
	Base shear ultimate		320.73	kN		
	Base shear service		60.68	kN		
	8% of base shear to be applied at top level		25.66	kN		
	8% of base shear to be applied at top level		4.85	kN		

$$Fi = .92V \frac{Wihi}{\sum(Wihi)}$$

Ultimate			
Equivalent Lateral force at level i (Roof)	56.44		
Equivalent Lateral force at level i	116.62		
Equivalent Lateral force at level i	147.68		
Equivalent Lateral force at level i	0.00		
Equivalent Lateral force at level i	0.00		
Equivalent Lateral force at level i	0.00		
Service	320.73	kN base V	
Equivalent Lateral force at level i (Roof)	10.68		
Equivalent Lateral force at level i	22.06		
Equivalent Lateral force at level i	27.94		
Equivalent Lateral force at level i	0.00		
Equivalent Lateral force at level i	0.00		
Equivalent Lateral force at level i	0.00		
	60.68	kN base V	

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86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Ref:	Design	Output

North

Shear Load on N3

56.4 + 116.6/2 = 115 kN

Gravity N* = 23.5 + 125/2 = 86 kN

Capacity V_{u1prob} = 228 kN

M_{u1prob} = 102 kNm

- insert spreadsheet

M* = 115 x 2 = 230 kNm

NBS = $\frac{M^*}{M_{prob}}$ = 44%

- Strengthen with stripes

M_{u1prob} /2 = 51 kN

Shear to be taken by new steel

0.8x115 - 51 = 41 kN

M_2^* = 41 x 2 = 82 kNm

T_2^* = 82/0.9 = 91 kN

for plate 100 x 6mm

ϕN_t = 130.3 kN

Connect with 5 M24 HiLti Anchors

- Insert HiLti report

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Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design			Output			
N3 Bending					Ts1=	91.82	kN
					Ts2=	17.94	kN
	L=	1000	mm		Ts3=	20.83	kN
	H=	2000	mm		Ts4=	6.71	kN
	d1=	900	mm		Ts5=	3.97	kN
	d2=	700	mm		T2=	141.27	kN
	d3=	500	mm		C=	149.63	kN
	d4=	300	mm		a=	77.21	mm
	d5=	100	mm		c=	90.83	mm
	bw=	190	mm		εs2=	0.0011	
	D1=	19	mm		εs3=	0.0008	
	D2=	10	mm		εs4=	0.0004	
	D3=	13	mm		εs5=	0.0000	
	D4=	10	mm		fs2=	225.85	MPa
	D5=	19	mm		fs3=	151.70	MPa
	As1=	283.385	mm2		fs4=	77.55	MPa
	As2=	78.5	mm2		fs5=	3.40	MPa
	As3=	132.665	mm2				
	As4=	78.5	mm2		Ts1=	91.82	kN
	As5=	283.385	mm2		Ts2=	17.73	kN
	Es=	200	GPa		Ts3=	20.13	kN
	εy=	0.0015			Ts4=	6.09	kN
	f'm=	12	MPa		Ts5=	0.96	kN
	fy=	324	MPa		T2=	136.72	kN
	self weight=	8.36	kN		C=	145.08	kN
					a=	74.86	mm
	T1=	91.82	kN		c=	88.07	mm
	C=	100.18	kN		εs2=	0.0011	
	a=	51.69	mm		εs3=	0.0008	
	c=	60.81	mm		εs4=	0.0004	
	εs2=	0.0011			εs5=	0.0000	
	εs3=	0.0008			fs2=	226.10	MPa
	εs4=	0.0004			fs3=	152.20	MPa
	εs5=	0.0001			fs4=	78.31	MPa
	fs2=	228.50	MPa		fs5=	4.41	MPa
	fs3=	157.00	MPa				
	fs4=	85.51	MPa		Ts1=	91.82	kN
	fs5=	14.01	MPa		Ts2=	17.75	kN
					Ts3=	20.19	kN
					Ts4=	6.15	kN
					Ts5=	1.25	kN
					Mi=	102.0	kNm

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Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design					Output
N3 Shear						
	Lw=	1000	mm			
	H=	2000	mm			
	t=	190	mm			
	bw=	190	mm	fully grouted		
	d=	800	mm			
	Ag=	190000	mm2			
	f'm=	12	MPa			
	vbm=	0.7	MPa			
	vg=	1.5	MPa			
	He/Lw=	2.00				
Basic strength provided by masonry						
	C2=	1.00				
	vm=	0.70	MPa			
Shear stress provided by masonry under axial load						
	N*=	93000	N	<	228000	N
	α=	75	deg			
	vp=	1.20	MPa	<	1.2	MPa
Strength provided by shear reinforcement						
	Av=	0	mm2			
	s=	1400	mm			
	C3=	0.8				
	fy=	324	MPa			
	bw=	190	mm			
	vs=	0.000	MPa			
Maximum total shear stress						
	vn=	1.500	MPa	<	1.5	MPa
	Vn=	228.0	kN			

Company:		Page:	1
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	3/09/2020
E-Mail:			

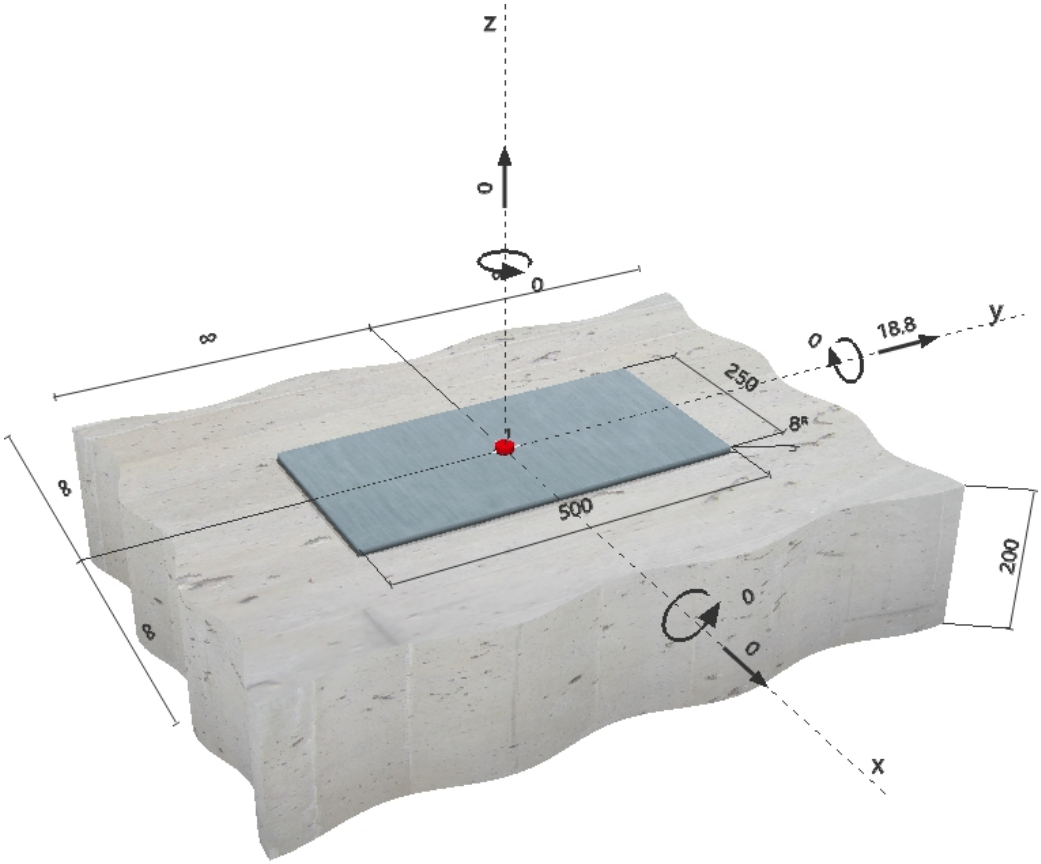
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 + HIT-V (8.8) M24**
Return period (service life in years):	50
Effective embedment depth:	$h_{ef,opti}$ = 96 mm ($h_{ef,limit}$ = 144 mm)
Material:	8.8
Evaluation Service Report:	ETA 12/0084
Issued I Valid:	8/28/2019 | -
Proof:	Design method ETAG BOND (EOTA TR 029) + Seismic (EOTA TR 045)
Seismic performance category:	C1
Seismic proof type:	5.3 a1) Capacity design
Seismic load percentage <=20%:	no
Stand-off installation:	e_b = 0 mm (no stand-off); t = 8 mm
Anchor plate:	l_x x l_y x t = 250 mm x 500 mm x 8 mm; (Recommended plate thickness: not calculated
Profile:	no profile
Base material:	cracked concrete, C20/25, $f_{c,cube}$ = 25.00 N/mm^2; h = 200 mm, Temp. short/long: 0/0 °C
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)
	no longitudinal edge reinforcement

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [mm] & Loading [kN, kNm]



Company:
Specifier:
Address:
Phone I Fax: |
E-Mail:

2 Proof I Utilization (Governing Cases)

		Design values [kN]		Utilization	
Loading	**Proof**	**Load**	**Capacity**	**β_N / β_V [%]**	**Status**
Tension	-	-	-	- / -	-
Shear	Pryout Strength	18.800	19.188	- / 98	OK

Loading	**β_N**	**β_V**	**α**	**Utilization $\beta_{N,V}$ [%]**	**Status**
Combined tension and shear loads	-	-	-	-	-

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 Remarks; Your Cooperation Duties

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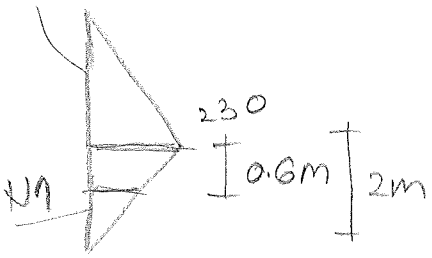
Subject: **901 Blank Calc Sheet**

Ref:	Design	Output

Connect N3 and N1

ϕN_{tf} = 163kN for M20 - Gr 8.8

N3



Strengthening required on 0.6m however anchors are spaced 0.1m ⇒ stripes 2m

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Ref:	Design	Output

South upper floor

2 units

Shear Load on two units : 125 kN
(two halves)

Shear Load on S3 : 125/5 = 25 kN

h = 800 mm

M* = 20 kNm

Gravity Load on two units : 125 kN
on S3: 125/5 = 25 kN

S3 Capacity $V_{u,prob}$ = 91.2 kN
$M_{u,prob}$ = 13 kNm

$NBS = \frac{13}{20} = 65\%$

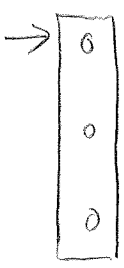
S5 Capacity $V_{u,prob}$ = 320 kN
$M_{u,prob}$ = 682 kNm

M* = 125 x 1.2 = 150 kNm

→ Strengthening around windows

Calcs in spreadsheet

- Insert S.S.
- Insert Hilti report



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Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design			Output
	SG3&SG2			
	L=	400	mm	
	H=	800	mm	
	d1=	300	mm	
	d2=	100	mm	
	bw=	190	mm	
	D=	13	mm	
	As=	132.665	mm2	
	Es=	200	GPa	
	εy=	0.0015		
	f'm=	12	MPa	
	fy=	324	MPa	
	self weight=	1.3376	kN	
	T1=	42.98	kN	
	C=	44.32	kN	
	a=	22.87	mm	
	c=	26.91	mm	
	εs2=	0.0004		
	fs2=	80.30	MPa	
	Ts1=	42.98	kN	
	Ts2=	10.65	kN	
	T2=	53.64	kN	
	C=	54.97	kN	
	a=	28.37	mm	
	c=	33.37	mm	
	εs2=	0.0004		
	fs2=	74.97	MPa	
	Ts1=	42.98	kN	
	Ts2=	9.95	kN	
	T2=	52.93	kN	
	C=	54.27	kN	
	a=	28.00	mm	
	c=	32.94	mm	
	εs2=	0.0004		
	fs2=	75.33	MPa	
	Ts1=	42.98	kN	
	Ts2=	9.99	kN	
	Mi=	13.15	kNm	

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Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design	Output

Strengthening with PFC - SG3

Total shear load

V= 25 kN

	E	I	stiffnes	Shear	M*	Mprob	NBS
	GPa	mm4 x106		kN	kNm	kNm	%
PFC 200	200	19.1	3820	5.02103	4.016824		
Wall	15	1013.33333	15200	19.97897	15.98318	13	81.33553
		Σ	19020				

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Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design	Output

S5 Bending

L=	4600	mm	
H=	1200	mm	
d1=	4500	mm	
d2=	4300	mm	
d3=	3300	mm	
d4=	3100	mm	
d5=	2500	mm	
d6=	2300	mm	
d7=	2100	mm	
d8=	1300	mm	
d9=	1100	mm	
bw=	190	mm	
D=	13	mm	
As=	132.665	mm2	
Es=	200	GPa	
εy=	0.0015		
f'm=	12	MPa	
fy=	324	MPa	
self weight=	23.0736	kN	
T1=	42.98	kN	
C=	66.06	kN	
a=	34.09	mm	
c=	40.10	mm	
εs2=	0.0014		
εs3=	0.0011		
εs4=	0.0010		
εs5=	0.0008		
εs6=	0.0008		
εs7=	0.0007		
εs8=	0.0004		
εs9=	0.0004		
fs2=	286.55	MPa	
fs3=	219.28	MPa	
fs4=	205.83	MPa	
fs5=	165.47	MPa	
fs6=	152.01	MPa	
fs7=	138.56	MPa	
fs8=	84.7485	MPa	
fs9=	71.2953	MPa	
Ts1=	42.98	kN	
Ts2=	38.01	kN	
Ts3=	29.09	kN	
Ts4=	27.31	kN	
Ts5=	21.95	kN	
Ts6=	20.17	kN	
Ts7=	18.38	kN	
Ts8=	11.24	kN	
Ts9=	9.46	kN	
T2=	218.60	kN	
C=	241.67	kN	
a=	124.70	mm	
c=	146.71	mm	

εs2=	0.0014		
εs3=	0.0011		
εs4=	0.0010		
εs5=	0.0008		
εs6=	0.0007		
εs7=	0.0007		
εs8=	0.0004		
εs9=	0.0003		
fs2=	286.22	MPa	
fs3=	217.30	MPa	
fs4=	203.52	MPa	
fs5=	162.17	MPa	
fs6=	148.39	MPa	
fs7=	134.61	MPa	
fs8=	79.48	MPa	
fs9=	65.69	MPa	
Ts1=	42.98	kN	
Ts2=	37.97	kN	
Ts3=	28.83	kN	
Ts4=	27.00	kN	
Ts5=	21.51	kN	
Ts6=	19.69	kN	
Ts7=	17.86	kN	
Ts8=	10.54	kN	
Ts9=	8.72	kN	
T2=	215.10	kN	
C=	238.17	kN	
a=	122.90	mm	
c=	144.58	mm	
εs2=	0.0014		
εs3=	0.0011		
εs4=	0.0010		
εs5=	0.0008		
εs6=	0.0007		
εs7=	0.0007		
εs8=	0.0004		
εs9=	0.0003		
fs2=	286.22	MPa	
fs3=	217.34	MPa	
fs4=	203.57	MPa	
fs5=	162.24	MPa	
fs6=	148.46	MPa	
fs7=	134.69	MPa	
fs8=	79.58	MPa	
fs9=	65.81	MPa	
Ts1=	42.98	kN	
Ts2=	37.97	kN	
Ts3=	28.83	kN	
Ts4=	27.01	kN	
Ts5=	21.52	kN	
Ts6=	19.70	kN	
Ts7=	17.87	kN	
Ts8=	10.56	kN	
Ts9=	8.73	kN	
Mi=	682.32	kNm	

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Structural Concepts

Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design						Output
S5 Shear							
	Lw=	4600	mm				
	H=	1200	mm				
	t=	190	mm				
	bw=	60	mm	not fully grouted			
	d=	3680	mm				
	Ag=	874000	mm2				
	f'm=	12	MPa				
	vbm=	0.7	MPa				
	vg=	1.5	MPa				
	He/Lw=	0.26					
Basic strength provided by masonry							
	C2=	1.49					
	vm=	1.04	MPa				
Shear stress provided by masonry under axial load							
	N*=	136000	N	<	1048800	N	
	α=	27	deg				
	vp=	0.28	MPa	<	1.2	MPa	
Strength provided by shear reinforcement							
	Av=	129	mm2				
	s=	1400	mm				
	C3=	0.8					
	fy=	324	MPa				
	bw=	190	mm				
	vs=	0.126	MPa				
Maximum total shear stress							
	vn=	1.450	MPa	<	1.5	MPa	
	Vn=	320.1	kN				

MemDes Calculations @ 10:52:23 10-09-2020 by Ana Petrovic
==========
Project : 3587-9593
Description : SG3

Section : 200PFC Grade 300+

Major Axis Bending
Design Action M^*_x = 6.0 kNm
User provided value for a_m = 1.75
a_s = 0.80
a_m a_s >= 1.0, => Segment Fully Restrained
$M_{bx} = M_{sx}$ = 66.30 kNm
Major axis capacity Ratio = M^*_x / $f M_{bx}$
= 0.10, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V^*_x = 10.0 kN
Nominal Shear Yield capacity V_w = 230.4 kN
a_v = 5.35 >= 1.0 => full web shear capacity
$V_u = V_w$ = 230.4 kN
Shear capacity ratio = V^*_x / $f V_u$
= 0.05, ---- OK ----
========== SUMMARY ==========
**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.10 ---- OK ----

==========
MemDes v 2.1 : Calculations for Duffill Watts Consulting Group
Project : 3587-9593 at 10:52:23 AM on 10/09/2020
Description : SG3 by Ana Petrovic **Powered By Steltech (http://www.steltech.co.nz)**

Company:		Page:	1
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	3/09/2020
E-Mail:			

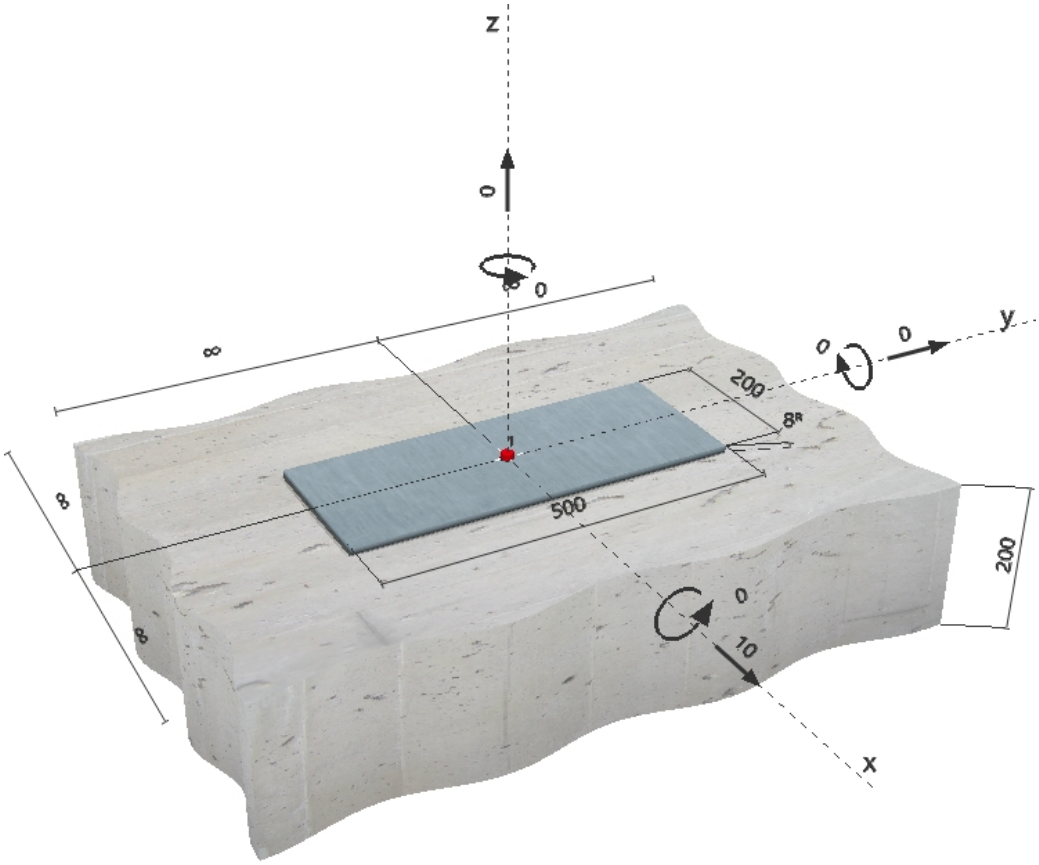
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 + HIT-V (8.8) M16**
Return period (service life in years):	50
Effective embedment depth:	$h_{ef,opti}$ = 80 mm ($h_{ef,limit}$ = 164 mm)
Material:	8.8
Evaluation Service Report:	ETA 12/0084
Issued I Valid:	8/28/2019 | -
Proof:	Design method ETAG BOND (EOTA TR 029) + Seismic (EOTA TR 045)
Seismic performance category:	C1
Seismic proof type:	5.3 a1) Capacity design
Seismic load percentage <=20%:	no
Stand-off installation:	e_b = 0 mm (no stand-off); t = 8 mm
Anchor plate:	l_x x l_y x t = 200 mm x 500 mm x 8 mm; (Recommended plate thickness: not calculated
Profile:	no profile
Base material:	cracked concrete, C20/25, $f_{c,cube}$ = 25.00 N/mm^2; h = 200 mm, Temp. short/long: 0/0 °C
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)
	no longitudinal edge reinforcement

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [mm] & Loading [kN, kNm]



Company:		Page:	2
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	3/09/2020
E-Mail:			

2 Proof I Utilization (Governing Cases)

		Design values [kN]		Utilization	
Loading	**Proof**	**Load**	**Capacity**	**β_N / β_V [%]**	**Status**
Tension	-	-	-	- / -	-
Shear	Pryout Strength	10.000	14.597	- / 69	OK

Loading	**β_N**	**β_V**	**α**	**Utilization $\beta_{N,V}$ [%]**	**Status**
Combined tension and shear loads	-	-	-	-	-

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 Remarks; Your Cooperation Duties

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Structural Concepts

Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Ref:	Design	Output

South ground floor

Shear load on two units total

$320 \times 2 = 640\ kN$

Gravity $2 \times 440.3 = 880.6\ kN$

S2: Shear $640/5 = 128\ kN$

$M^* = 128 \times 0.8 = 102.4\ kNm$

$N^* = 440/5 = 88\ kN$

$M_{prob} = 13\ kNm$ - same as S3

$V_{prob} = 91.2\ kN$

S7: $V^* = 640\ kN$

$M^* = 640 \times 1.2 = 768\ kNm$

$N^* = 440\ kN$

Capacity M_{prob} - same as S5

$M_{prob} = 682\ kNm$

$V_{prob} = 494\ kN$

NBS Shear $= \frac{494}{640} = 77\%$

NBS Bending $= \frac{682}{768} = 88\%$

⇒ - Strengthen around windows
- diagonals in wall

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Structural Concepts

Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design					Output
SG2 Shear						
	Lw=	400	mm			
	H=	800	mm			
	t=	190	mm			
	bw=	190	mm	fully grouted		
	d=	320	mm			
	Ag=	76000	mm2			
	f'm=	12	MPa			
	vbm=	0.7	MPa			
	vg=	1.5	MPa			
	He/Lw=	2.00				
Basic strength provided by masonry						
	C2=	1.00				
	vm=	0.70	MPa			
Shear stress provided by masonry under axial load						
	N*=	88000	N	<	91200	N
	α=	80	deg			
	vp=	1.20	MPa	<	1.2	MPa
Strength provided by shear reinforcement						
	Av=	0	mm2			
	s=	1400	mm			
	C3=	0.8				
	fy=	324	MPa			
	bw=	190	mm			
	vs=	0.000	MPa			
Maximum total shear stress						
	vn=	1.500	MPa	<	1.5	MPa
	Vn=	91.2	kN			

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Structural Concepts

Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design						Output
	SG7 Shear						
		Lw=	4600	mm			
		H=	1200	mm			
		t=	190	mm			
		bw=	60	mm	not fully grouted		
		d=	3680	mm			
		Ag=	874000	mm2			
		f'm=	12	MPa			
		vbm=	0.7	MPa			
		vg=	1.5	MPa			
		He/Lw=	0.26				
	Basic strength provided by masonry						
		C2=	1.49				
		vm=	1.04	MPa			
	Shear stress provided by masonry under axial load						
		N*=	515000	N	<	1048800	N
		α=	27	deg			
		vp=	1.07	MPa	<	1.2	MPa
	Strength provided by shear reinforcement						
		Av=	129	mm2			
		s=	1400	mm			
		C3=	0.8				
		fy=	324	MPa			
		bw=	190	mm			
		vs=	0.126	MPa			
	Maximum total shear stress						
		vn=	2.237	MPa	<	1.5	MPa
		Vn=	493.9	kN			

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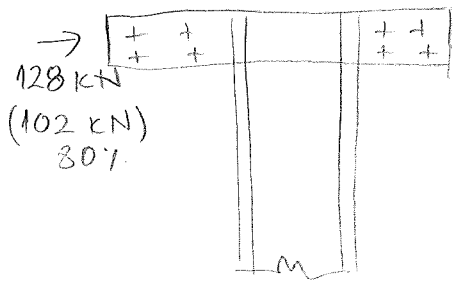
Structural Concepts

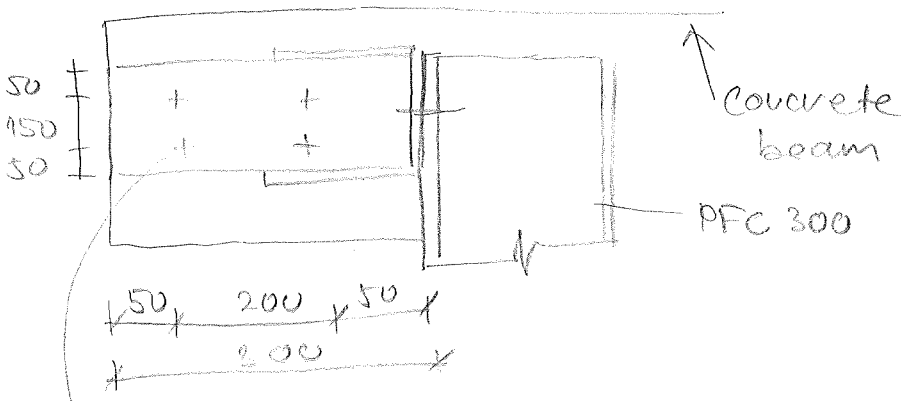
Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Ref:	Design	Output

S2

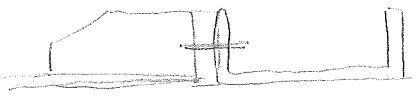


uls Ratio for
PFC 300 · 1.029
⇒ NBS = 97%



4 x M20 HiLti Anchors
HST3 hef 2

- Insert HiLti



for M20 Gr 8.8
ØNtf = 163 kN
> 100% NBS

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Structural Concepts

Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.: -6-

Ref:	Design	Output

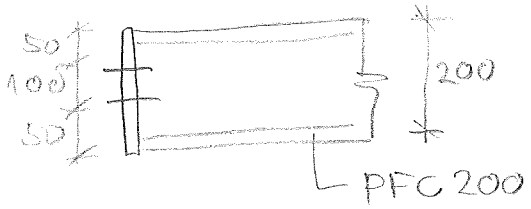
From Space Gass.
PFC Horiz
$N^* = 256$ kN for PFC 200 UC Ratio 0.4

- Insert SG.

Insert Memdes

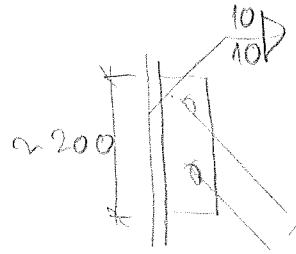
Tension in bolt 254 kN
⇒ 2 M20 Gr 8.8

$\phi N_{tf} = 2 \times 163$ 326 kN



Changed to plate calcs on pg 12

Reid brace
$T_1 = 555$ kN for two D=25 mm capacity
$2 \times 255 = 510$ kN
NBS = 90%.



$T_2 = 440$ kN
for one D=32 mm
capacity 361 kN
NBS = 80%.

Weld $t_w = 10$ mm $L_w = 2 \times 200 = 400$ mm
$\phi V_w = \sim 0.8 \times 0.6 \times 707 \times 400 \times 410 = 557$ kN

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Structural Concepts

Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design	Output

Load on 1 unit

+ 490.3 kN
- 21 kN
419 kN ← Point load

From dividing wall

0
+17.2
+52.5
+69.2
139 kN ← Wall

Other 280 kN ← Point load

UDL 21 + 13.2 = 34.2 / 4.3 = 8 kN/m'

Forwd. exist. 3.375 kN/m'
new 2.6 kN/m'
20 kN/m'

Point ① and ⑥

$\frac{280}{2} + 139 = 279$ kN

② → ⑤ 280 + 139 = 419 kN

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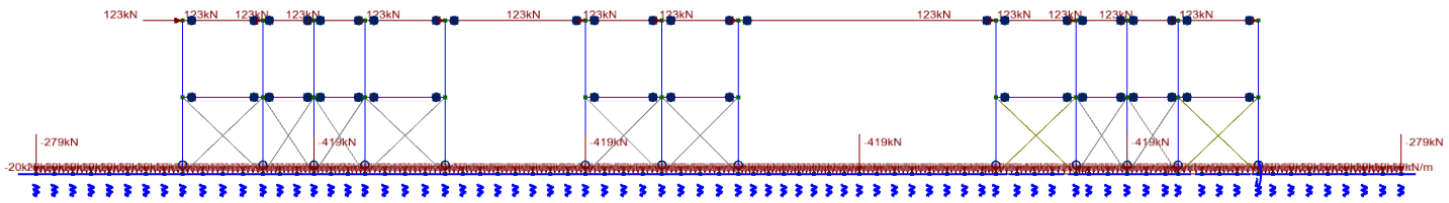
Structural Concepts

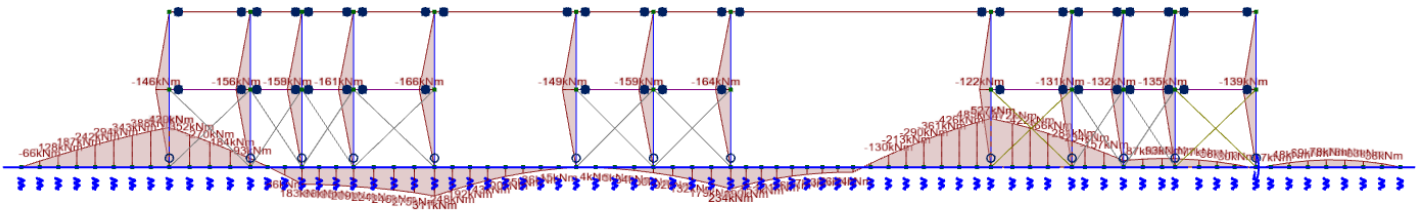
Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design	Output



Space gass model and load S+G



Bending diagram for S+G

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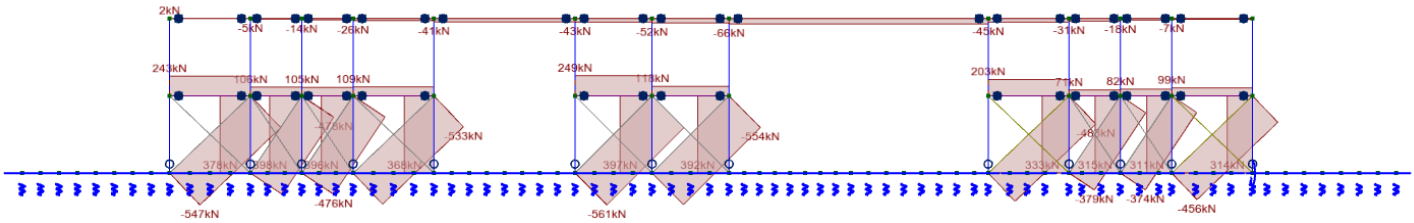
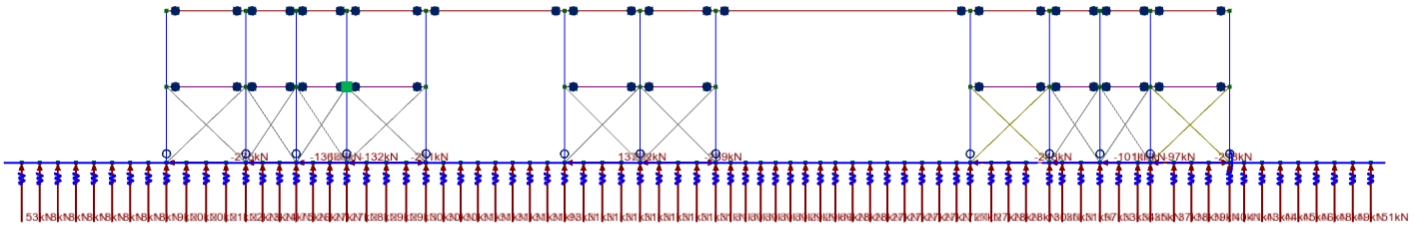
Client: **Freehold Properties (Investments) LLP** Ref: **3587-9593**
Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design	Output



Axial forces diagram for S+G



Reactions diagram for S+G

MemDes Calculations @ 13:32:27 10-09-2020 by Ana Petrovic

==========

Project : 3587-9593
Description : S2

Section : 300PFC Grade 300+

Major Axis Bending

Design Action M^*_x = 154.0 kNm

User provided value for a_m = 1.75

a_s = 0.81

a_m a_s >= 1.0, => Segment Fully Restrained

$M_{bx} = M_{sx}$ = 169.20 kNm

Major axis capacity Ratio = M^*_x / $f M_{bx}$

= 1.01, **** FAIL ****

Shear Calculations (Unstiffened Web)

Design Action V^*_x = 128.0 kN

Nominal Shear Yield capacity V_w = 460.8 kN

a_v = 4.17 >= 1.0 => full web shear capacity

$V_u = V_w$ = 460.8 kN

Shear capacity ratio = V^*_x / $f V_{vm}$

= 0.51, ---- OK ----

========== SUMMARY ==========

**** U.L.S. Capacity Check FAILED, Load Cap. Ratio = 1.01 *** FAIL ***

==========

MemDes v 2.1 : Calculations for Duffill Watts Consulting Group
Project : 3587-9593 at 1:32:27 PM on 10/09/2020
Description : S2 by Ana Petrovic **Powered By Steltech (http://www.steltech.co.nz)**

Company:		Page:	1
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	4/09/2020
E-Mail:			

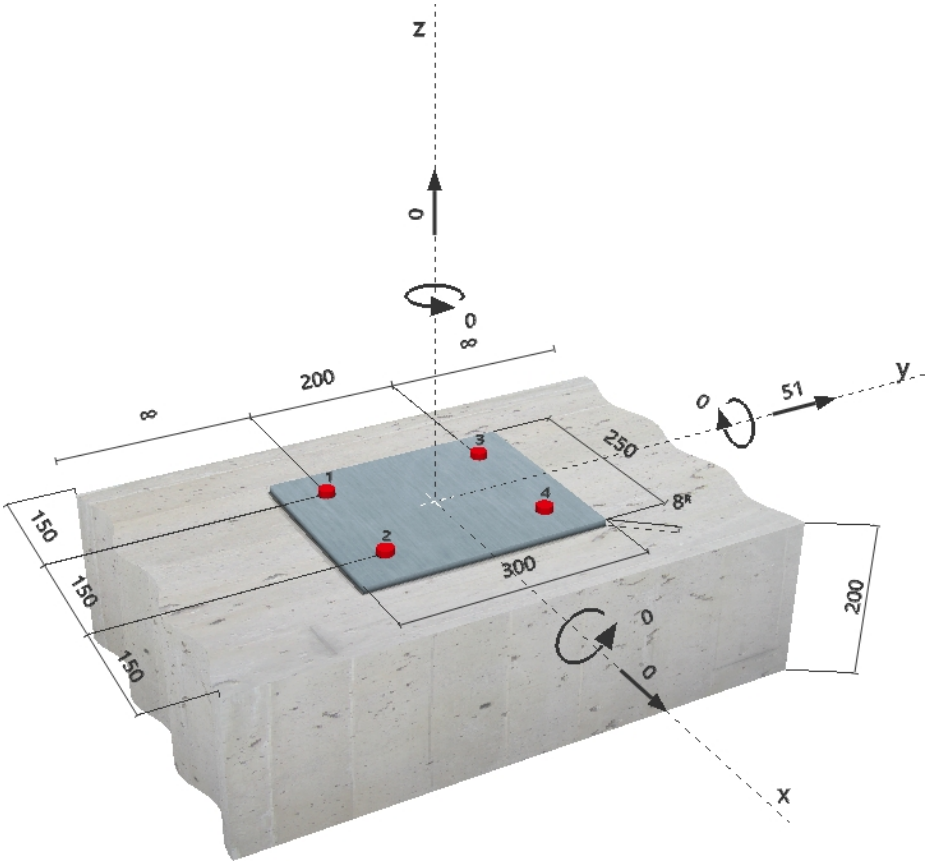
Specifier's comments:

1 Input data

Anchor type and diameter:	**HST3 M20 hef2**
Return period (service life in years):	50
Effective embedment depth:	h_{ef} = 101 mm, h_{nom} = 116 mm
Material:	
Evaluation Service Report:	ETA-98/0001
Issued I Valid:	10/2/2019 | -
Proof:	Design method ETAG (No. 001 Annex C/2010) + Seismic (EOTA TR 045)
Seismic performance category:	C1
Seismic proof type:	5.3 a1) Capacity design
Seismic load percentage <=20%:	no
Stand-off installation:	e_b = 0 mm (no stand-off); t = 8 mm
Anchor plate:	l_x x l_y x t = 250 mm x 300 mm x 8 mm; (Recommended plate thickness: not calculated
Profile:	no profile
Base material:	cracked concrete, C20/25, $f_{c,cube}$ = 25.00 N/mm^2; h = 200 mm
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm)
	no longitudinal edge reinforcement

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

Geometry [mm] & Loading [kN, kNm]



Company:		Page:	2
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	4/09/2020
E-Mail:			

2 Proof I Utilization (Governing Cases)

		Design values [kN]		Utilization	
Loading	**Proof**	**Load**	**Capacity**	**β_N / β_V [%]**	**Status**
Tension	-	-	-	- / -	-
Shear	Concrete edge failure in direction x+	25.500	26.220	- / 98	OK

Loading	**β_N**	**β_V**	**α**	**Utilization $\beta_{N,V}$ [%]**	**Status**
Combined tension and shear loads	-	-	-	-	-

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

4 Remarks; Your Cooperation Duties

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HST3 (-R) subject to:

Anchor size	M8	M10	M12	M16	M20	M24
Hammer drilling* 	TE2(-A) – TE30(-A)				TE40 – TE70	
Diamond core drilling* 	DD-30W, DD-EC1					
Setting tool* 	Setting tool HS-SC				-	
Hollow drill bit drilling* 	-		TE-CD, TE-YD			
Seismic Set/ Filling Set** 	Seismic/Filling Set M8-M20 (Carbon and Stainless Steel A4)					-

***Installation methods provided in ETA-98/0001**

**Seismic set needed to fill the annular gap between anchor and fixture:
No annular gap, double design resistance (agap=1)

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86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

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Foundation

- Weight of two units (half of ground floor)

= 440.3 x 2 = 880.6 kN

- Half weight of the wall S7

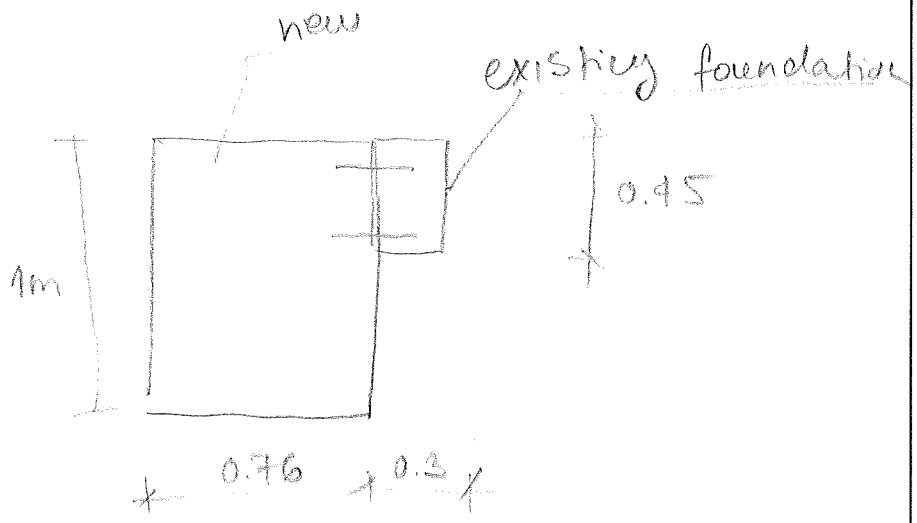
= 7.98/2 x 3.3 = 13.2 kN

Foundation beam L = 8.5m

- existing 0.3 x 0.45 x 8.5 x 24 = 27.5 kN
- new 1 x 0.76 x 8.5 x 24 = 155 kN

Total 1076 kN

g = 1076/8.5 = 126.6 kN



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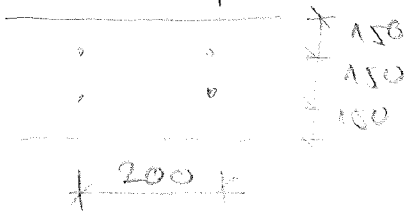
Ref:	Design	Output

Anchors connecting existing foundation with new one.

$V^* = 0.2 \times 126.6 \times 0.8 = 20\,kN$ (80% strength).



- HIT - HY 200 + Rebar 16mm

- Insert report

Company:		Page:	1
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	8/09/2020
E-Mail:			

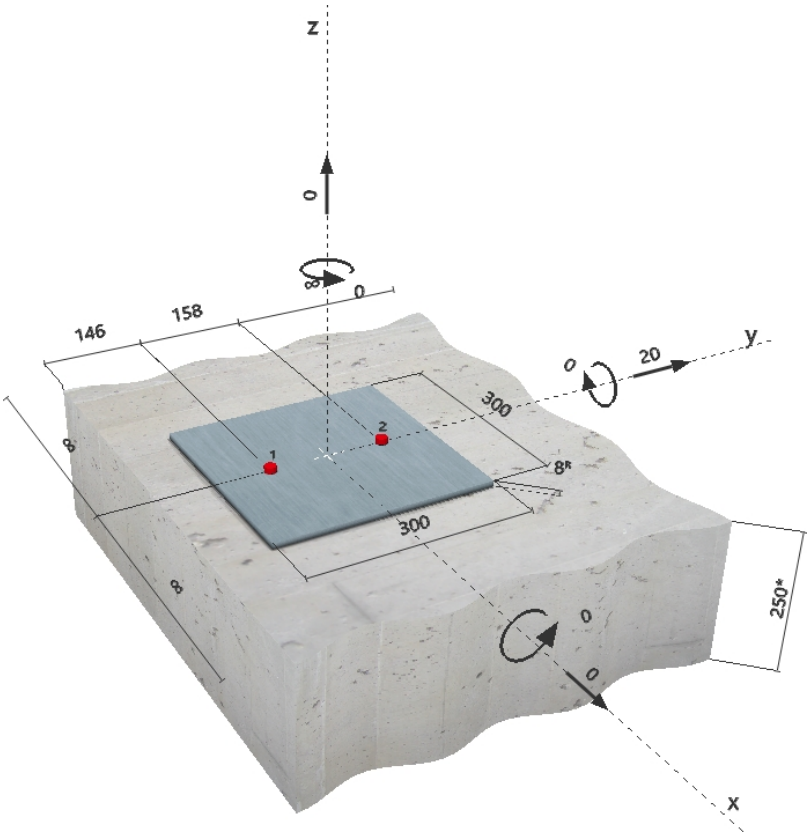
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 + Rebar 16mm**
Return period (service life in years):	50
Effective embedment depth:	$h_{ef,opti}$ = 80 mm ($h_{ef,limit}$ = 210 mm)
Material:	B500B
Evaluation Service Report:	ETA 12/0084
Issued I Valid:	8/28/2019 | -
Proof:	Design method ETAG BOND (EOTA TR 029) + Seismic (EOTA TR 045)
Seismic performance category:	C1
Seismic proof type:	5.3 a1) Capacity design
Seismic load percentage <=20%:	no
Stand-off installation:	e_b = 0 mm (no stand-off); t = 8 mm
Anchor plate:	l_x x l_y x t = 300 mm x 300 mm x 8 mm; (Recommended plate thickness: not calculated
Profile:	no profile
Base material:	cracked concrete, C20/25, $f_{c,cube}$ = 25.00 N/mm^2; h = 250 mm, Temp. short/long: 0/0 °C
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	no reinforcement or reinforcement spacing >= 150 mm (any Ø) or >= 100 mm (Ø <= 10 mm) with longitudinal edge reinforcement d >= 12 Reinforcement to control splitting according to EOTA TR 029, 5.2.2.6 present.

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [mm] & Loading [kN, kNm]



Company:		Page:	2
Specifier:		Project:	
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	|	Date:	8/09/2020
E-Mail:			

2 Proof I Utilization (Governing Cases)

		Design values [kN]		Utilization	
Loading	**Proof**	**Load**	**Capacity**	**β_N / β_V [%]**	**Status**
Tension	-	-	-	- / -	-
Shear	Pryout Strength	20.000	21.359	- / 94	OK

Loading	**β_N**	**β_V**	**α**	**Utilization $\beta_{N,V}$ [%]**	**Status**
Combined tension and shear loads	-	-	-	-	-

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

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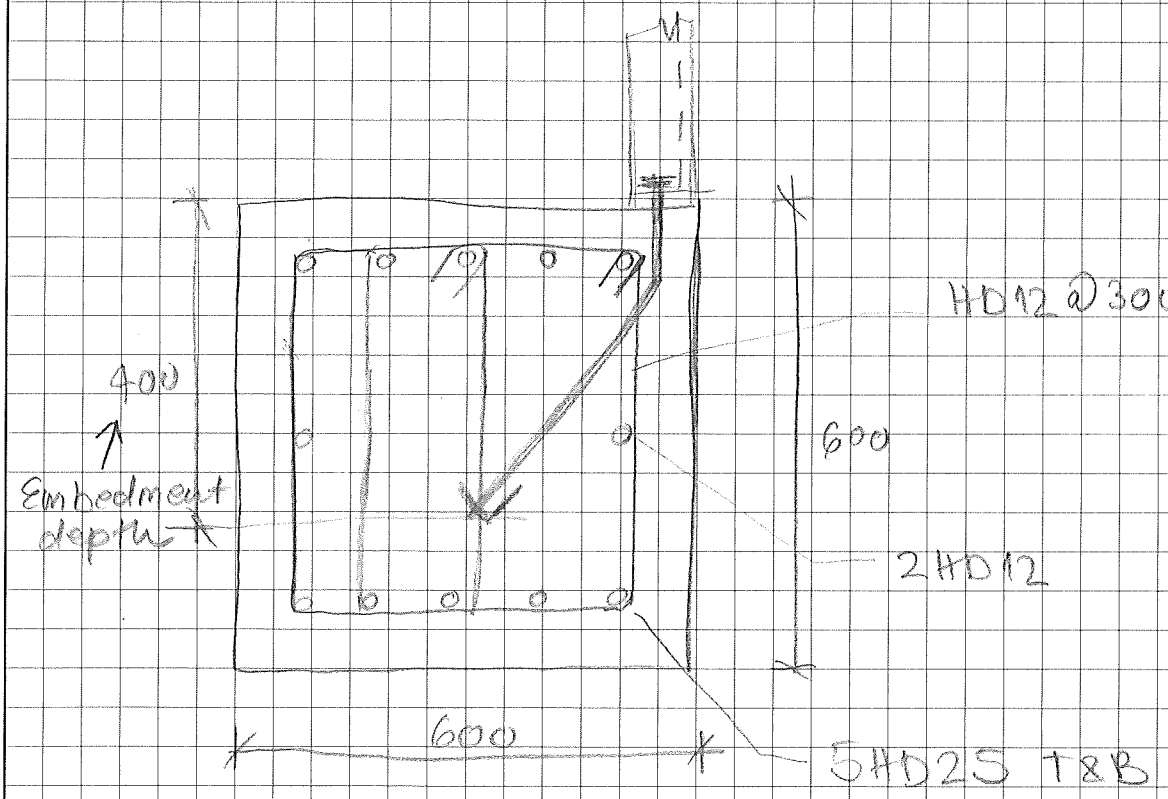
Project:	**Two Storey Motel Units - Strengthening I**	Date:	**5/8/20**
	86 Wellington Street, Picton	BY:	**AP**
Subject:	**900.1 Hand Calcs**		

Design	Sheet No	

NEW FOUNDATION



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Project: **Two Storey Motel Units - Strengthening Design** Date: **5/8/20**
86 Wellington Street, Picton BY: **AP**
Subject: **901 Blank Calc Sheet**

Sheet No.:

Ref:	Design	Output

Foundation beading

From SG

$M^*_{max} = 527$ kNm

For 5HD25

$\phi M_u = 472$ kNm

$NBS = \frac{472}{527} = 90\%$

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Ref:	Design	Output

New foundation

Probable yield strength of reinforcement:

$f_{y,prob}$= 500 MPa

Probable compressive strength:

$f_{c,prob}'$= 25 MPa

α= 0.85 *for fc' < 55MPa*

β= 0.85 *for fc' < 30MPa*

Beam dimensions:

Overall section depth:

h= 600 mm

Width:

b= 600 mm

Protective cover:

c_c= 75 mm

Effective depth:

d= 500.5 mm

Reinforcement:

D= 25 mm

A= 491 mm2

n= 5

Total area of tension steel:

A_s= 2454 mm^2

M_n= 471.9 kNm

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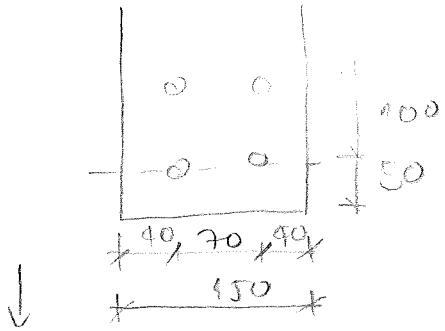
Ref:	Design	Output

Plate braces

$\neq 150 \times 15$

Connect with 4 M20 Gr 8.8



$A_g = 2250\ mm^2$

$A_n = 1590\ mm^2$

Tension in plate

$\phi N_t = \min 0.9 \begin{cases} A_g f_y \\ 0.85 K_{te} A_n f_u \end{cases} = 523\ kN$

$NBS = \frac{523}{555} = 95\%\ NBS$

Bolts capacity:

$\phi V_f = 4 \times 129 = 516\ kN$

$NBS = \frac{516}{555} = 95\%$

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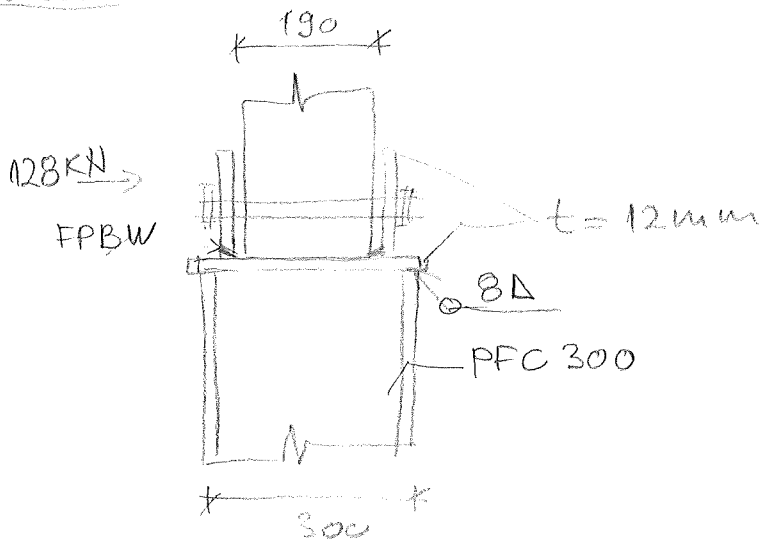
Sheet No.: -13

Ref:	Design	Output

Connection PFC 300 with dividing wall



for M20 Gr 8.8 $\phi N_{tf} = 163$ kN

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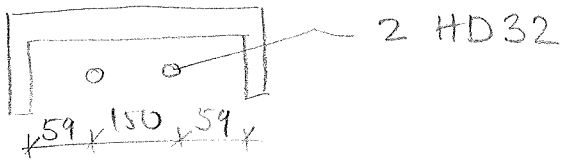
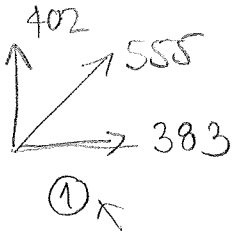
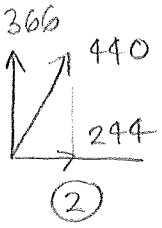
Base plate connection







$N_S = n\, A_{se}\, f_{ut} = 2 \times 804 \times 450 = 724\ kN$

$V_S = N_S = 724\ kN$

$\phi N_S = 0.75 \times 724 = 543\ kN$

$\phi V_S = 0.65 \times 724 = 470.5$

$N^* < \phi N_S$

$V^* < \phi V_S$

$$\frac{N^*}{\phi N_S} + \frac{V^*}{\phi V_S} = 1.24$$

$$NBS = \frac{1.2}{1.24} = 97\%$$

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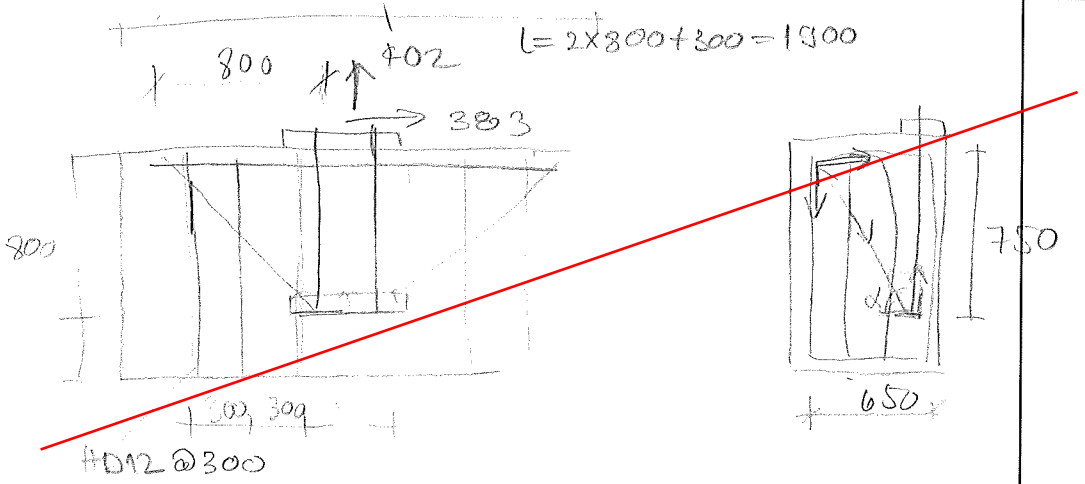

$\phi F_{s,V} = \phi A_s f_y = 0.75 \times 113 \times 500 = 42.4\,kN$ – for HD 12

$\phi F_{s,H} = 0.75 \times 201 \times 500 = 75.4\,kN$ – for HD 16

5HD16 ⇒ $\phi F_{sH} = 377\,kN$

$\frac{402}{sin\alpha} = 532$

$F_{u,V} = 402\,kN$ – 4x6 HD12 = 42.4x24 = 1017 kN

$F_{u,H} = 350\,kN$ – 1x6 HD12 = 42.4x6 = 254 kN

not OK

For 8HD12 $\phi F_{u,H} = 42.4 \times 8 = 340\,kN$

$NBS = \frac{340}{350} = 97\%$

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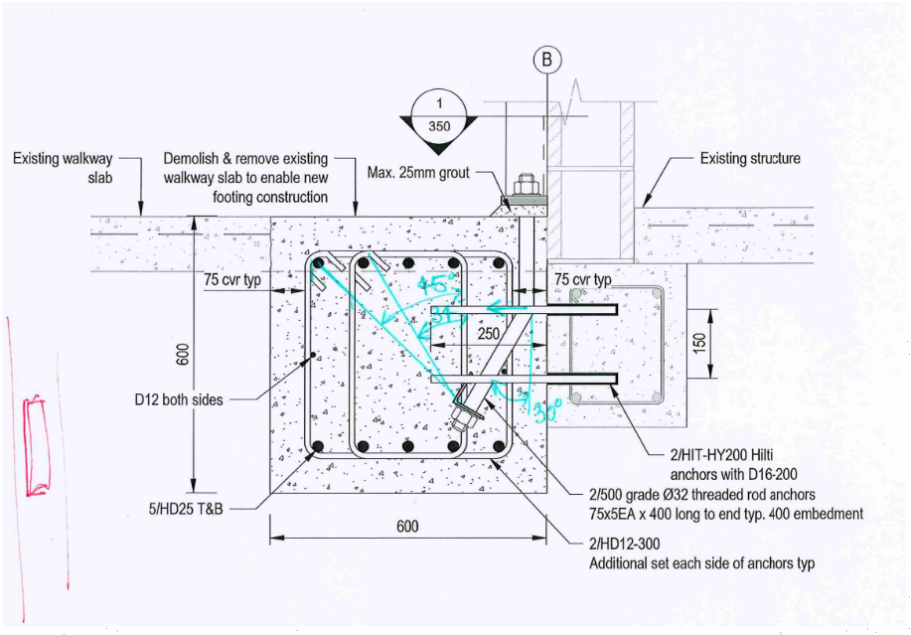

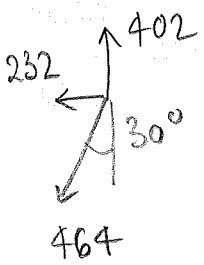

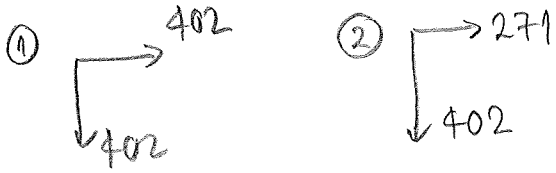

2X6 HD12 = 42.4 X 12 = 508.8 kN

6 HD12 = 254 kN

402/2 = 201 kN ⇒ OK

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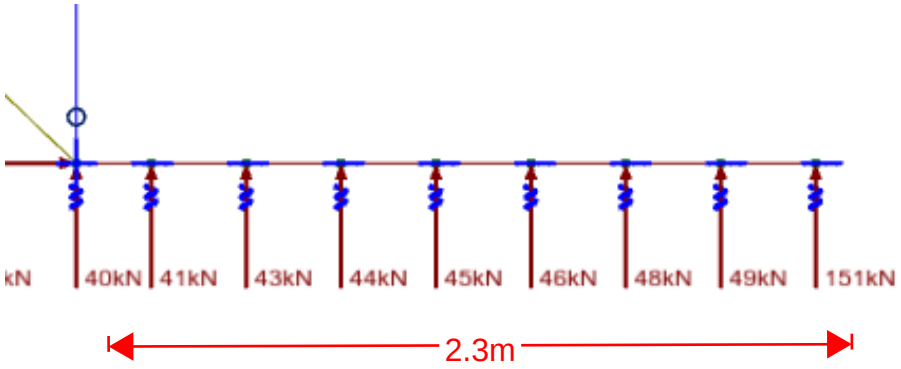

Bearing

R = Σ for 2.3m at the end

= 467kN

$$\sigma = \frac{467}{2.3 \times 0.9} = 225. < 300\ kPa$$