# CALCULATION SHEET

Project/Task/File No: N-H0060.03 | Sheet No 1 of
Project Description: HNZC KAPITI RHP | Office: PN.
35 KAITAWA CRESCENT | Computed: AV / /
WATER TANK SEISMIC RESTRAINTS- | Check: / /

Tank Seismic Restraint to NZS 4219:2009.

Classify Building & Component

Building is normal residential I.L.2, Table 1.
Tank only represents hazard outside of building Cat P1.

Load Demand.

Floor Height Coefficent $C_H = 1.0$ Ground Level
Seismic Zone $Z = 0.4$ Paraparaumu.
Performance Factor $C_p = 0.85$. Table 4.
Risk Factor $R_c = 1.0$ Table 5.

Seismic Coefficient $C = 2.7 \times 1.0 \times 0.4 \times 0.85 \times 1.0$
$= 0.92$.

Weight of Tank W (6000L)
W = 6000 kg + 130kg tank self wt = 60.2 kN.

Lateral Force on Water Tank = $C \times W$
$= 0.92 \times 60.2$
$= 55.4$ kN (ULS.)

wsp | OPUS

CSF 400 (7/2000)

# CALCULATION SHEET

Project/Task/File No: N-H0060.03.
Project Description: 35 KAITAWA CRESCENT
Sheet No 2 of
Office: PN
Computed: AJ / /
Check: / /

Tank Dim's $1150 \times 3700 \times 1600(H)$

Taking moments about base

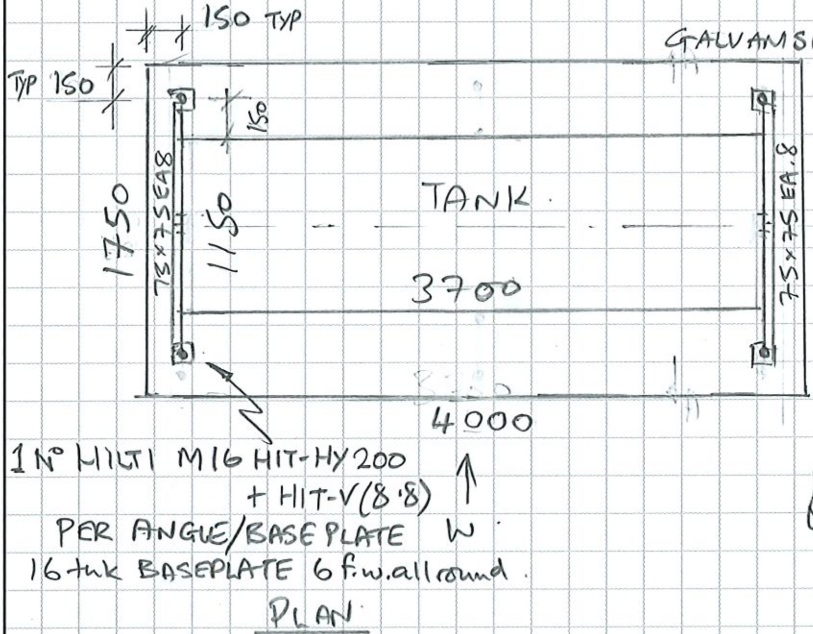
overturning Moment $= 55.4 \times 0.8m = 44.3$ kNm

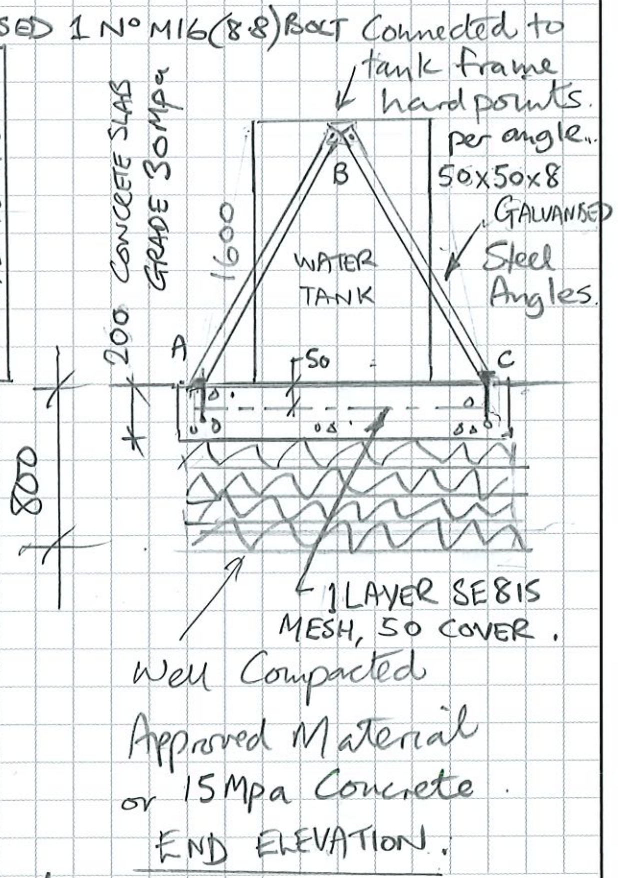
restoring Moment $= 60.2 \times \frac{1.150}{2} = 34.6$ kNm
(from Tank)

restoring Moment $= 1.75 \times 4.3 \times 0.2 \times 24 \times \frac{1.75}{2} = 31.6$ kNm
(Slab = 200 thk)

66.2 kNm

Narrow dimension 1150. direction requires restraint.




Tanks by Tanksalot N.Z.
have fully engineered internal
Stainless Steel support frame.
This design assumes that the frame design
is sufficient to withstand seismic forces.

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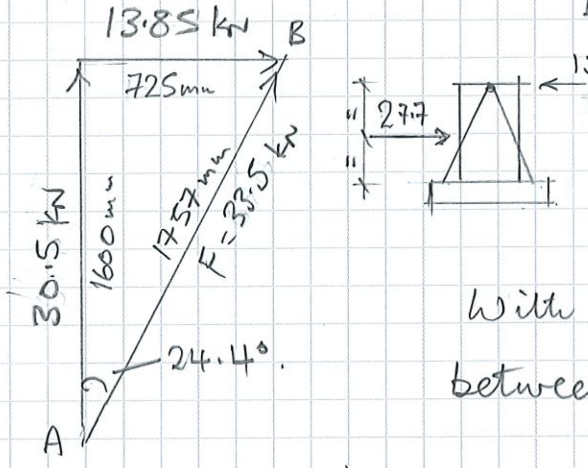
# CALCULATION SHEET

Project/Task/File No: N-M0060.03 | Sheet No 3 of
Project Description: HNZC KAPITI RHP | Office: PN
35 KAITAWA CRES | Computed: AV / /
WATER TANK SEISMIC RESTRAINTS | Check: / /

Consider Angle Braces

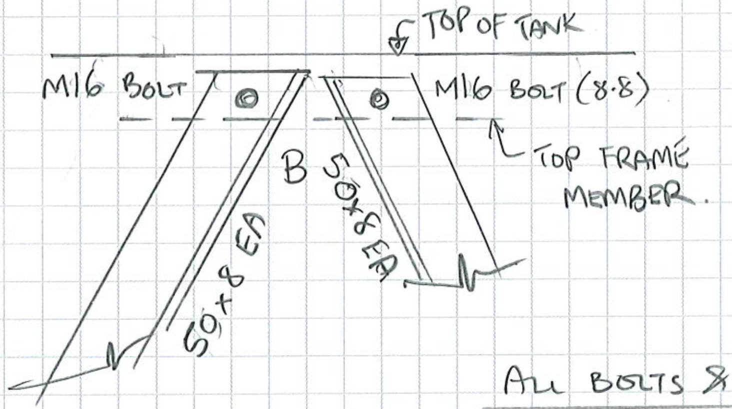
Force in member AB. Half of 55.4kN = 27.7kN taken by brace @ each end of tank.




With Load equally shared between compression/tension angle

Each angle has 16.8 kN C/T. force.

From table 14 50x50x8 EA @1.75m lg compression has >16.8 kN capacity with 1 N° M16 (8.8) Bolt.




ALL BOLTS & 50 ANGLES GALVANISED.

From Table 13. 50x50x8 EA with 1 M16 Bolt has 28 kN tension capacity ∴ O.K.

wsp

CALCULATION SHEET

Project/Task/File No: N- H0060.03

Project Description: 35 KAITAWA CRES

WATER TANK SEISMIC RESTRAINTS

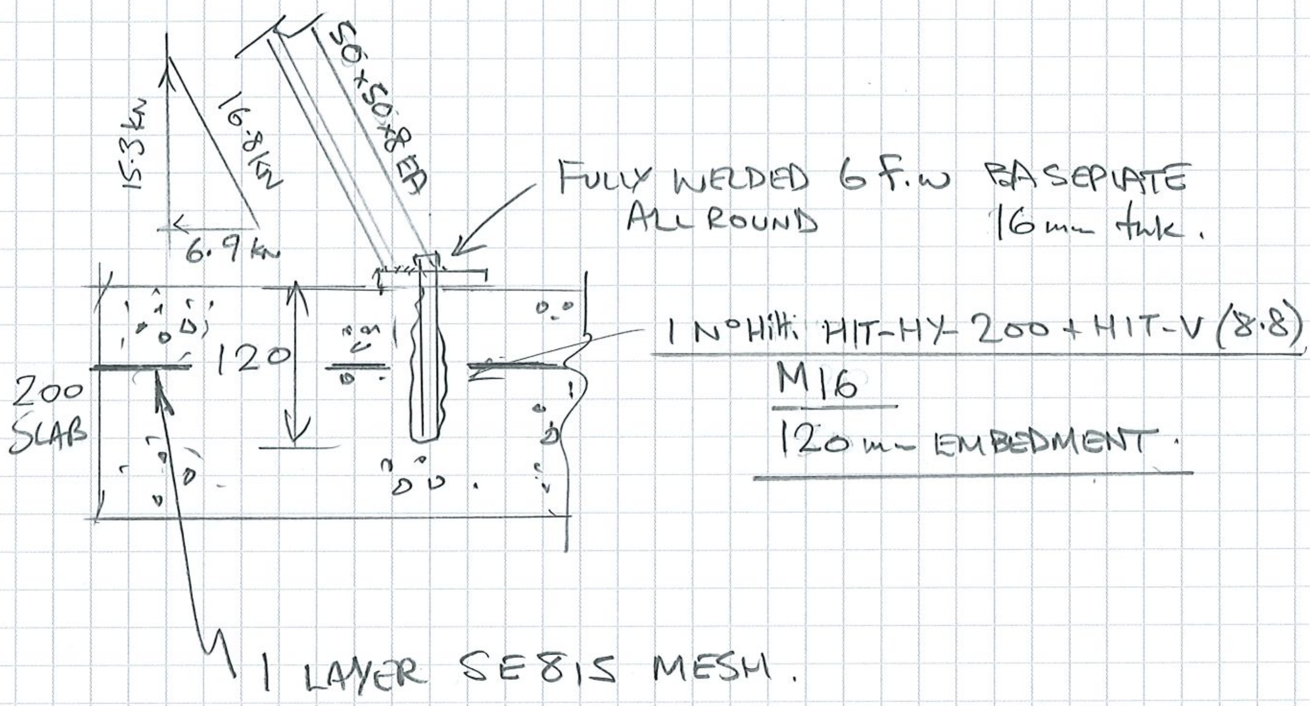
Sheet No 4 of

Office: PN

Computed: AU / /

Check: / /

Bolts @ Locations A & C.



| | | | |
|---|---|---|---|
| Company: | | Page: | 1 |
| Specifier: | | Project: | 35 Kaitawa Crescent |
| Address: | | Sub-Project I Pos. No.: | |
| Phone I Fax: | \| | Date: | 8/10/2020 |
| E-Mail: | | | |

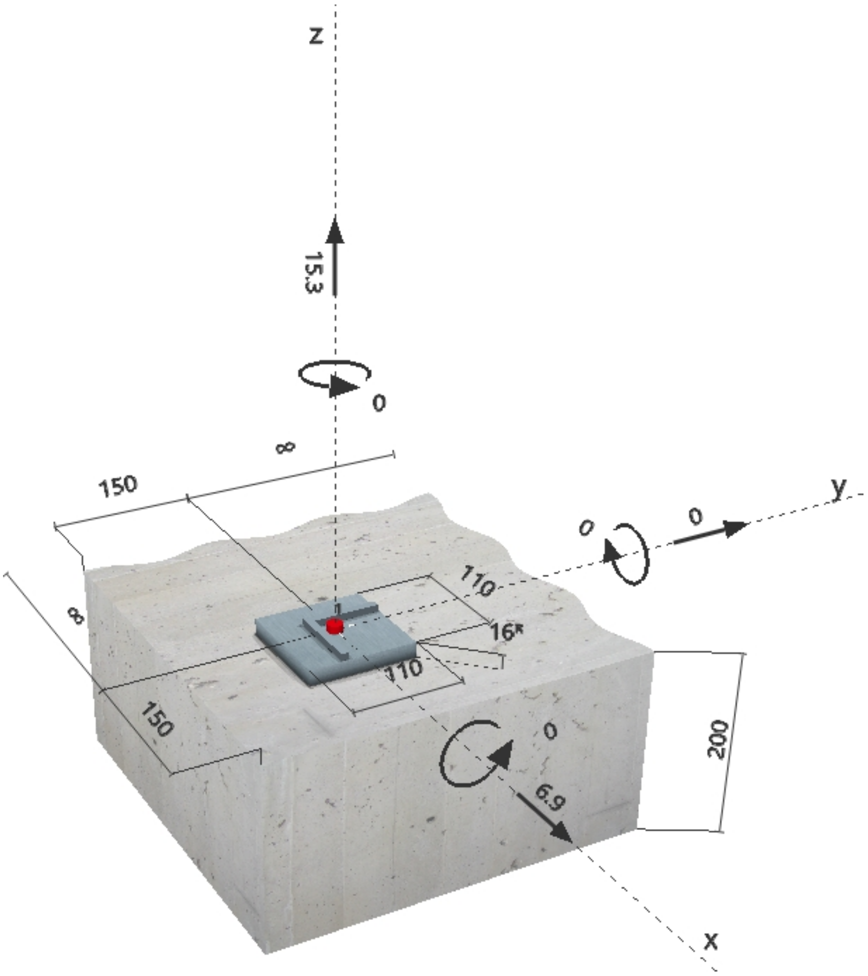
**Specifier's comments:**

## 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: | $h_{ef,act}$ = 120 mm ($h_{ef,limit}$ = - mm) |
| Material: | 8.8 |
| Evaluation Service Report: | ETA 12/0084 |
| Issued I Valid: | 8/28/2019 \| - |
| Proof: | Design method ETAG BOND (EOTA TR 029) |
| Stand-off installation: | $e_b$ = 0 mm (no stand-off); t = 16 mm |
| Anchor plate: | $l_x$ x $l_y$ x t = 110 mm x 110 mm x 16 mm; (Recommended plate thickness: not calculated |
| Profile: | L profile, L 75 x 8; (L x W x T) = 75 mm x 75 mm x 8 mm |
| Base material: | cracked concrete, C25/30, $f_{c,cube}$ = 30.00 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) with longitudinal edge reinforcement d >= 12 |


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

**Geometry [mm] & Loading [kN, kNm]**



Company:
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Address:
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## 2 Load case/Resulting anchor forces

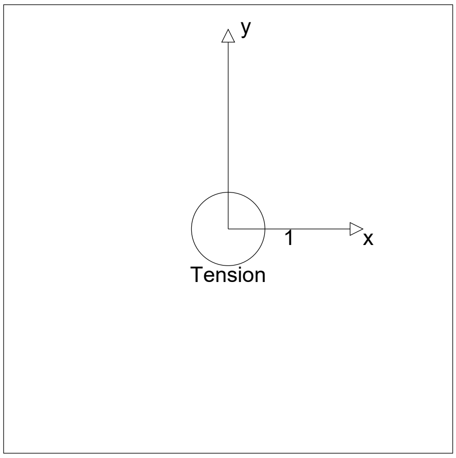
Load case: Design loads

**Anchor reactions [kN]**
Tension force: (+Tension, -Compression)

| Anchor | Tension force | Shear force | Shear force x | Shear force y |
|---|---|---|---|---|
| 1 | 15.300 | 6.900 | 6.900 | 0.000 |

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [N/mm$^2$]
resulting tension force in (x/y)=(0/0): 15.300 [kN]
resulting compression force in (x/y)=(0/0): 0.000 [kN]

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




## 3 Tension load (EOTA TR 029, Section 5.2.2)

| | Load [kN] | Capacity [kN] | Utilization $\beta_N$ [%] | Status |
|---|---|---|---|---|
| Steel Strength* | 15.300 | 83.733 | 19 | OK |
| Combined pullout-concrete cone failure** | 15.300 | 27.838 | 55 | OK |
| Concrete Breakout Strength** | 15.300 | 27.588 | 56 | OK |
| Splitting failure** | 15.300 | 30.229 | 51 | OK |

* anchor having the highest loading **anchor group (anchors in tension)

### 3.1 Steel Strength

| $N_{Rk,s}$ [kN] | $\gamma_{M,s}$ | $N_{Rd,s}$ [kN] | $N_{Sd}$ [kN] |
|---|---|---|---|
| 125.600 | 1.500 | 83.733 | 15.300 |

### 3.2 Combined pullout-concrete cone failure

| $A_{p,N}$ [mm$^2$] | $A^0_{p,N}$ [mm$^2$] | $\tau_{Rk,ucr,25}$ [N/mm$^2$] | $s_{cr,Np}$ [mm] | $c_{cr,Np}$ [mm] | $c_{min}$ [mm] |
|---|---|---|---|---|---|
| 108,900 | 129,600 | 18.00 | 360 | 180 | 150 |

| $\psi_c$ | $\tau_{Rk,cr}$ [N/mm$^2$] | k | $\psi^0_{g,Np}$ | $\psi_{g,Np}$ |
|---|---|---|---|---|
| 1.020 | 8.67 | 2.300 | 1.000 | 1.000 |

| $e_{c1,N}$ [mm] | $\psi_{ec1,Np}$ | $e_{c2,N}$ [mm] | $\psi_{ec2,Np}$ | $\psi_{s,Np}$ | $\psi_{re,Np}$ |
|---|---|---|---|---|---|
| 0 | 1.000 | 0 | 1.000 | 0.950 | 1.000 |

| $N^0_{Rk,p}$ [kN] | $N_{Rk,p}$ [kN] | $\gamma_{M,p}$ | $N_{Rd,p}$ [kN] | $N_{Sd}$ [kN] |
|---|---|---|---|---|
| 52.309 | 41.757 | 1.500 | 27.838 | 15.300 |

### 3.3 Concrete Breakout Strength

| $A_{c,N}$ [mm$^2$] | $A^0_{c,N}$ [mm$^2$] | $c_{cr,N}$ [mm] | $s_{cr,N}$ [mm] |
|---|---|---|---|
| 108,900 | 129,600 | 180 | 360 |

| $e_{c1,N}$ [mm] | $\psi_{ec1,N}$ | $e_{c2,N}$ [mm] | $\psi_{ec2,N}$ | $\psi_{s,N}$ | $\psi_{re,N}$ |
|---|---|---|---|---|---|
| 0 | 1.000 | 0 | 1.000 | 0.950 | 1.000 |

| $k_1$ | $N^0_{Rk,c}$ [kN] | $\gamma_{M,c}$ | $N_{Rd,c}$ [kN] | $N_{Sd}$ [kN] |
|---|---|---|---|---|
| 7.200 | 51.840 | 1.500 | 27.588 | 15.300 |

### 3.4 Splitting failure

| $A_{c,N}$ [mm$^2$] | $A^0_{c,N}$ [mm$^2$] | $c_{cr,sp}$ [mm] | $s_{cr,sp}$ [mm] | $\psi_{h,sp}$ |
|---|---|---|---|---|
| 116,964 | 147,456 | 192 | 384 | 1.180 |

| $e_{c1,N}$ [mm] | $\psi_{ec1,N}$ | $e_{c2,N}$ [mm] | $\psi_{ec2,N}$ | $\psi_{s,N}$ | $\psi_{re,N}$ | $k_1$ |
|---|---|---|---|---|---|---|
| 0 | 1.000 | 0 | 1.000 | 0.934 | 1.000 | 7.200 |

| $N^0_{Rk,c}$ [kN] | $\gamma_{M,sp}$ | $N_{Rd,sp}$ [kN] | $N_{Sd}$ [kN] |
|---|---|---|---|
| 51.840 | 1.500 | 30.229 | 15.300 |

# 4 Shear load (EOTA TR 029, Section 5.2.3)

| | Load [kN] | Capacity [kN] | Utilization $\beta_V$ [%] | Status |
|---|---|---|---|---|
| Steel Strength (without lever arm)* | 6.900 | 50.240 | 14 | OK |
| Steel failure (with lever arm)* | N/A | N/A | N/A | N/A |
| Pryout Strength** | 6.900 | 55.176 | 13 | OK |
| Concrete edge failure in direction x+** | 6.900 | 16.839 | 41 | OK |

* anchor having the highest loading **anchor group (relevant anchors)

## 4.1 Steel Strength (without lever arm)

| $V_{Rk,s}$ [kN] | $\gamma_{M,s}$ | $V_{Rd,s}$ [kN] | $V_{Sd}$ [kN] |
|---|---|---|---|
| 62.800 | 1.250 | 50.240 | 6.900 |

## 4.2 Pryout Strength (Concrete Breakout Strength controls)

| $A_{c,N}$ [mm$^2$] | $A^0_{c,N}$ [mm$^2$] | $c_{cr,N}$ [mm] | $s_{cr,N}$ [mm] | k-factor | $k_1$ |
|---|---|---|---|---|---|
| 108,900 | 129,600 | 180 | 360 | 2.000 | 7.200 |

| $e_{c1,V}$ [mm] | $\psi_{ec1,N}$ | $e_{c2,V}$ [mm] | $\psi_{ec2,N}$ | $\psi_{s,N}$ | $\psi_{re,N}$ |
|---|---|---|---|---|---|
| 0 | 1.000 | 0 | 1.000 | 0.950 | 1.000 |

| $N^0_{Rk,c}$ [kN] | $\gamma_{M,c,p}$ | $V_{Rd,cp}$ [kN] | $V_{Sd}$ [kN] |
|---|---|---|---|
| 51.840 | 1.500 | 55.176 | 6.900 |

## 4.3 Concrete edge failure in direction x+

| $h_{ef}$ [mm] | $d_{nom}$ [mm] | $k_1$ | $\alpha$ | $\beta$ |
|---|---|---|---|---|
| 120 | 16.0 | 1.700 | 0.089 | 0.064 |

| $c_1$ [mm] | $A_{c,V}$ [mm$^2$] | $A^0_{c,V}$ [mm$^2$] |
|---|---|---|
| 150 | 75,000 | 101,250 |

| $\psi_{s,V}$ | $\psi_{h,V}$ | $\psi_{\alpha,V}$ | $e_{c,V}$ [mm] | $\psi_{ec,V}$ | $\psi_{re,V}$ |
|---|---|---|---|---|---|
| 0.900 | 1.061 | 1.000 | 0 | 1.000 | 1.200 |

| $V^0_{Rk,c}$ [kN] | $\gamma_{M,c}$ | $V_{Rd,c}$ [kN] | $V_{Sd}$ [kN] |
|---|---|---|---|
| 29.767 | 1.500 | 16.839 | 6.900 |

# 5 Combined tension and shear loads (EOTA TR 029, Section 5.2.4)

Steel failure

| $\beta_N$ | $\beta_V$ | $\alpha$ | Utilization $\beta_{N,V}$ [%] | Status |
|---|---|---|---|---|
| 0.555 | 0.410 | 1.500 | 68 | OK |

$\beta_N^\alpha + \beta_V^\alpha \leq 1.0$

| | | | |
|---|---|---|---|
| Company: | | Page: | 4 |
| Specifier: | | Project: | 35 Kaitawa Crescent |
| Address: | | Sub-Project I Pos. No.: | |
| Phone I Fax: | \| | Date: | 8/10/2020 |
| E-Mail: | | | |

## 6 Displacements (highest loaded anchor)

Short term loading:

| | | | | | |
|---|---|---|---|---|---|
| $N_{Sk}$ | = | 11.333 [kN] | $\delta_N$ | = | 0.132 [mm] |
| $V_{Sk}$ | = | 5.111 [kN] | $\delta_V$ | = | 0.204 [mm] |
| | | | $\delta_{NV}$ | = | 0.243 [mm] |

Long term loading:

| | | | | | |
|---|---|---|---|---|---|
| $N_{Sk}$ | = | 11.333 [kN] | $\delta_N$ | = | 0.301 [mm] |
| $V_{Sk}$ | = | 5.111 [kN] | $\delta_V$ | = | 0.307 [mm] |
| | | | $\delta_{NV}$ | = | 0.429 [mm] |

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 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 Anchor require rigid anchor plates per current regulations (ETAG 001/Annex 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 stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM 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 Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Checking the transfer of loads into the base material is required in accordance with EOTA TR 029, Section 7!
- The design is only valid if the clearance hole in the fixture is not larger than the value given in Table 4.1 of EOTA TR029! For larger diameters of the clearance hole see Chapter 1.1. of EOTA TR029!
- 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.
- Bore hole cleaning must be performed according to instructions for use (blow twice with oil-free compressed air (min. 6 bar), brush twice, blow twice with oil-free compressed air (min. 6 bar)).
- Characteristic bond resistances depend on short- and long-term temperatures.
- Please contact Hilti to check feasibility of HIT-V rod supply.
- Edge reinforcement is not required to avoid splitting failure
- The characteristic bond resistances depend on the return period (service life in years): 50

# Fastening meets the design criteria!

# APPENDIX A - Site Specific Design Info

## 3.19 Pressurised Stormwater Pipelines

Pressurised stormwater pipelines are required between the spouting system and the rainwater tank. These pipes will remain full of water and it is essential every joint is completely water tight to prevent leaks or the development of unsightly moulds or slimes.

Joins between the downpipe system and in-ground pipework shall be using fabricated PVC joiners with either solvent-weld or rubber ring joint seals.

The complete pressurised system shall be pressure tested prior to being put into use.

Should any leaking joints develop in the system for any reason, the system will need to be drained, dried and the leaking component re-sealed and cured prior to being put back into service.

## 3.20 Rainwater Tanks

### *3.20.1 Overview of Stormwater System*

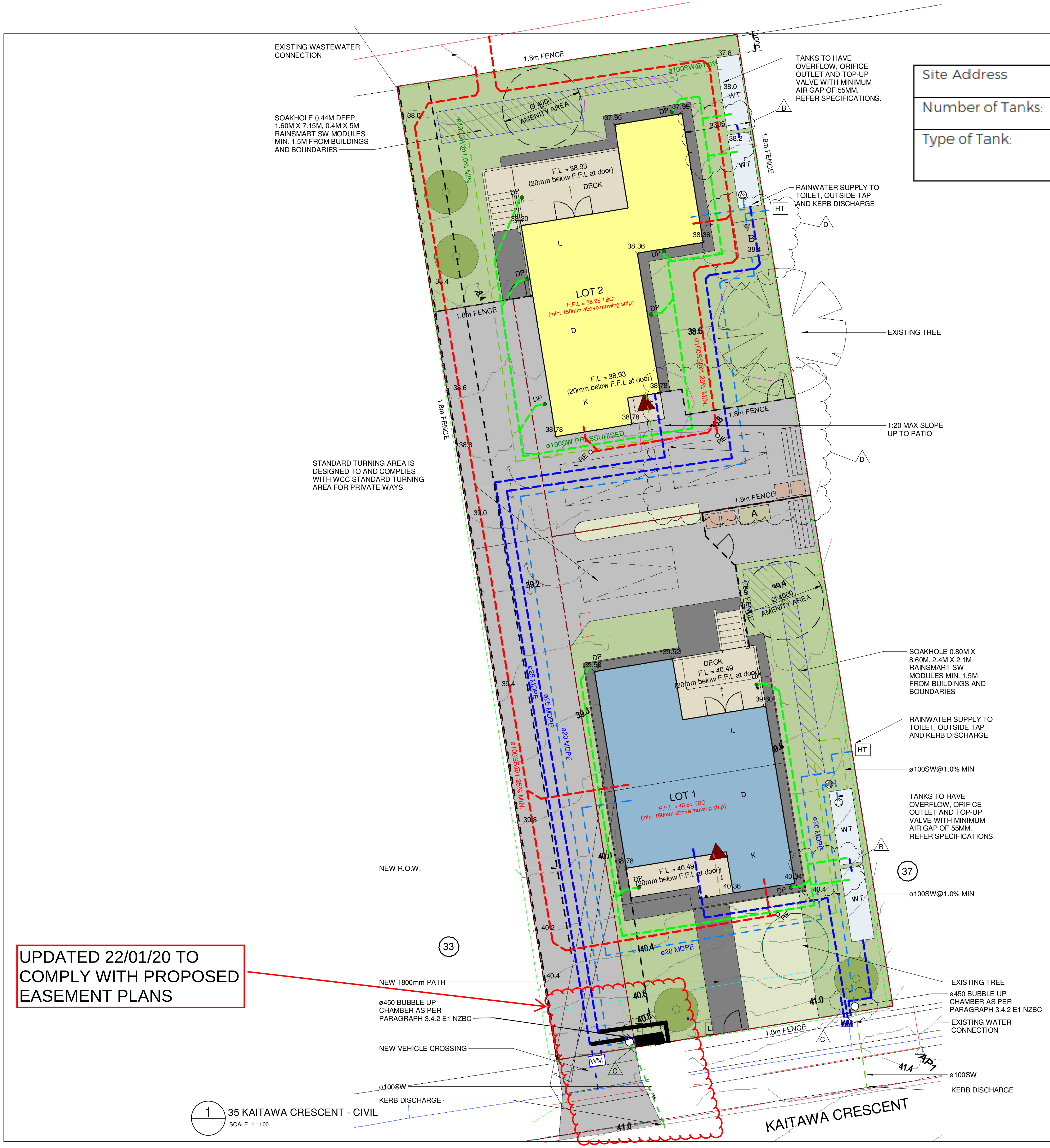
The proposed system includes several features to achieve hydraulic neutrality on a challenging site. These are:

- Rainwater storage and re-use for toilet flushing and garden watering.
- Capture of 90% of average rainfall, followed by pumped discharge to the kerb at less than 0.3 L/s per lot.
- Attenuation up to the 1 in 100 year rainfall event with discharge to on-site soakholes via orifice plates at the tanks.
- Porous paving to reduce site run-off.

### *3.20.2 Tank Model and Fittings*

The tank is to be as specified below, or approved equivalent tank:

| | |
|---|---|
| Site Address | 35 Kaitawa Crescent |
| Number of Tanks: | 4 |
| Type of Tank: | 6000L Slimline Steel Tanks 1150x3700x1600 (WxLxH). Site specific design. |
| Manufacturer: | Tanksalot, Silverdale, Auckland |
| Details 1: | Tank 1 (for Lot1)<br>1. 100mm inlet from stormwater system to 400mm leaf strainer on top of tank.<br>2.Tank top-up valve with minimum air gap of 55mm.<br>3. 50mm connection to Tank2 (at tank base)<br>4. 100mm overflow to Tank2 |




## NOTES:

| | |
|---|---|
| Site Address | 35 Kaitawa Crescent |
| Number of Tanks: | 4 |
| Type of Tank: | 6000L Slimline Steel Tanks 1150x3700x1600 (WxLxH). Site specific design. |

**LEGEND**

- PROPERTY BOUNDARY
- PRESSURE SW
- GRAVITY SW
- GRAVITY SEWER
- WATER
- RE — RODDING EYE
- ORG — OVERFLOW RELIEF GULLEY
- TV — TERMINAL VENT
- DP — DOWNPIPE
- WT — WATER TANK
- HT — HOSE TAP
- PUMP
- SOLENOID VALVE
- SOAKHOLE AREA
- CONCRETE/CONCRETE MOWING STRIP
- PERMEABLE PAVING AS DETAILED IN CIVIL SPECIFICATION. PROVIDE NIBS/KERBS TO PERIMETER EDGES OF PAVING
- FLUSH CONCRETE KERB
- FLUSH CONCRETE NIB

| REVISION | AMENDMENT | APP | DATE |
|---|---|---|---|
| P A | TENDER | | 08.07.2019 |
| P B | COUNCIL RFI | | 18.09.2019 |
| P C | BUBBLE UP CHAMBER LOCATION AMENDED | | 27.09.2019 |
| P D | SITE AMENDMENTS FOR LOT 2 | | 01.11.2019 |

## DETAILED DESIGN

Housing New Zealand
Housing New Zealand Corporation

wsp | OPUS


Wellington Office
PO Box 12 003
Wellington 6144
New Zealand
+64 4 471 7000


| | | |
|---|---|---|
| SCALE: 1 : 100 @ A1 | | ORIGINAL SIZE: A1 |
| DRAWN: CC | DESIGNED: TS | APPROVED: JD |
| DRAWING VERIFIED: JD | DESIGN VERIFIED: | APPROVED DATE: 01.11.2019 |

PROJECT
HOUSING NEW ZEALAND
35 KAITAWA CRESCENT, PARAPARAUMU

BUILDING CONSENT

TITLE
CIVIL PLAN

| OPUS PROJECT NO. | | SUITABILITY |
|---|---|---|
| N-H0060.03 | | |
| PROJ-ORIG-VOL-LVL-TYPE | SHEET NO. | REVISION |
| NH0060-OIC-03-XX-DR | A-1400 | RD |

CIVIL

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## 8.7 NZS 3604:2011 "Good Ground"

It is desirable for buildings of light-weight timber frame construction to be founded on "good ground" as defined by NZS3604:2011 cl. 3.1.3. Such foundations do not require specific engineering design of foundations. NZS 3604:2011 defines the criteria for "good ground" as that which has an ultimate geotechnical bearing capacity of at least 300 kPa, and excludes:

- Potentially compressible ground, such as topsoil, soft soils, or fill;
- Expansive soils;
- Ground which has buried services or records of land slips and surface creep.

Topsoil was encountered to a depth of 200 mm.

No laboratory testing has been undertaken to determine if the soils on site are expansive, however, based on the site investigation and observation of the existing structure in the site, the soils at the proposed development site do not appear to fall into expansive soil category.

To adopt the NZS3604:2011 cl. 3.1.3 design criteria for the proposed development the following conditions should be satisfied:

- All top soil and should be completely removed from under proposed building footprint
- Any underground services in the proposed development area should be removed and realigned and the trench should be filled with granular material compacted in layers of 150mm.

Our foundation assessment is based on the Scala test results and has been conducted in accordance with the NZS 3604:2011. We interpret that in order for the site to have 'good ground', the number of blows per 100mm depth of penetration below the underside of the proposed footing at each test site exceeds:

- Five [blows per 100mm] down to a depth equal to the width of the widest footing below the underside of the proposed footing.
- Three [blows per 100mm] at greater depths.

The silt layer encountered at the site to a depth of about 0.8m does not comply with the NZS3604:2011 'good ground' condition. Specific foundation design is required if the depth of the building foundation is above 0.8m.

The Scala test results indicate that the gravel layer underlying the silt from a depth of about 0.8m is compliant with the NZS3604:2011 definition of 'good ground'. If the building foundation was founded on this gravel layer standard foundation details from NZS3604 could be used. This could be achieved by using piles into the gravel layer, or by excavating the overlying silt and replacing with an approved fill.

Alternative a specific foundation design of a suitable foundation system could be undertaken of the building structure founded on the in situ silt layer.

## 8.8 Soakage Test

A soakage test was undertaken in hand auger hole HA-3. The test result is attached in the appendix.

The test revealed a low soakage potential at the site, and it appears that on-site soakage is not appropriate at the site.

# 9 Conclusions and Recommendations

Based on the desk study, ground investigation and geotechnical assessment, the conclusions and recommendations are given as follows:

- Soils underlying 35 Kaitawa Crescent are likely to comprise very stiff silt layer below topsoil underlain by dense to very dense gravel layer with silt matrix;
- Based on the geotechnical investigations, "good ground" is encountered from about 0.8m below the existing ground level;
- The material above this level does not comply with the requirements of "good ground" as defined in NZS3604, and should the building foundation be above the level of 0.8m below existing ground level, specific engineering design will be required.
- A shallow strip / pad foundation or short timber pile foundation is suitable for the proposed building for use at the site.
- The site subsoil class for the proposed development site is considered to be Class D –deep or soft soil site, in terms of the seismic design requirements of NZS 1170.5:2004;
- The likelihood of liquefaction occurring and ground damage in a seismic event at this site is considered low.

# 10 Limitation

We have prepared this report in accordance with the brief provided. The contents of the report are for the sole use of the Client, and no responsibility or liability will be accepted to any third party. Data or opinions contained within the report may not be used in other contexts or for any other purposes without our prior review and agreement.

The recommendations in this report are based on data collected at specific locations and by using suitable investigation techniques. Only a finite amount of information has been collected to meet the specific financial and technical requirements of the Client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it must be appreciated that actual conditions could vary from the assumed model.

Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes. This report is not to be reproduced either wholly or in part without our prior written permission. For further information regarding this geotechnical assessment, please do not hesitate to contact WSP.