REVIEW OF EVENTS
AND DESIGN REPORT

AT

163 SUMATRA ROAD
WEST HAMPSTEAD

Date: 09 December 2021

Reference: R/20172/2A

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REPORT ON STRUCTURAL CONDITION AT 163 SUMATRA ROAD, WEST HAMPSTEAD

REPORT REFERENCE: R/20172/2

CLIENT: Drawing and Planning Ltd

ENGINEER: Harold James (London) Ltd
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Report Prepared By:

Martin Soper
....
M R Soper BEng (Hons) CEng MIStructE

Date:

09 December 2021

This report has been carried out on behalf of Drawing and Planning Ltd and summarises the condition of the building, and the amendments to the temporary and permanent designs for the building along with a commentary on the reasons for alterations.

The report considers the events known to Harold James (London) Ltd and therefore relates to a timeframe extending from the initial appointment of the practice in August 2020. However, commentary is also given in respect of the emergency works undertaken to the property prior to the appointment of Harold James (London) Ltd.

References to right and left are from a position viewing the front elevation from Sumatra road

General description and condition of building as commencement of Harold James (London) Ltd appointment

In August 2020 Harold James (London) Ltd were appointed to carry out works as summarised below:

- Prepare permanent works design for the refurbishment of the property including a new basement and replacement of all upper floors.
- Prepare temporary works design and guidance on sequencing in order to maintain stability of the existing structure and surrounding properties

It is understood that prior to the appointment of Harold James (London) Ltd works had already been undertaken to underpin the party walls to the left and right hand side, with the underpinning returning for a short distance along the left hand side of the front elevation and also to the right hand side of the in-situ rear elevation.

Localised investigation work and review of progress photos taken at the time show that's the underpinning had been carried out using a simple vertical mass concrete pin, rather than the L shaped pin which would have acted as a retaining wall in accordance with a design by the original structural engineers (Martin Redstone Associates)

During the excavation works instability of the structure to the front leads to a collapse of the basement excavation resulting in the consequential collapse of the right hand portion of the front elevation. That's the rear left hand corner of the building of significant cracking was noted within the party wall suggesting a rearward translation of the short section of rear elevation. The extent of cracking to the wall from the other side was not noted at the time of the initial survey, and it is possible that the cracking was historic; however given the current condition of the building it was considered that any detail that might compromise the robustness of the remaining structure of the need to be dealt with.

As part of the initial emergency response following the collapse the following works had been undertaken:

- Backfill material had been either relocated or delivered to site and placed against the face of the excavation to the front elevation.

- Steel shuttering had been installed to the inside face of the substructure below the remaining section of the front elevation to the left hand side, and below the short section of rear elevation abutting the right hand party wall. It is not known at the time of the report whether the backfill behind the shuttering was installed in order to provide lateral restraint to a previously cast concrete underpin, or whether the material was placed to completely fill the void below the existing foundation before the concrete underpin was able to be cast.

- Three steel beams had been placed at two levels (approximately second floor level and eaves) across the missing section of front elevation in order to provide natural restraint to the internal pier within the front elevation itself

- Strongback props had been installed bearing on to the internal fill material and extending upwards to provide support to the primary note points of the suspended timber floor structure at upper floor levels and roof.

  The

- A scaffold had been installed across the entirety of the front elevation, which had been detailed at its head to provide support to the rafters and ceiling at eaves level

Initial temporary works design

The sequencing of the remedial works was designed in order to overcome the issue of the existing internal timber structure being supported at ground level by the previously installed of emergency Strongback props, but in the new structure would need to be supported by load bearing spine partitions which need to be built off the basement slab; the presence of the Strongback props precluding the necessary excavation required in order to install the basement slab.

Therefore the temporary works entailed:

1. Modifications to the roof framing at the front of the building to allow the roof to be resupported on the as-installed still beam to permit the subsequent removal of the scaffold if necessary

2. Localised vertical support to be provided to the short section of rear elevation abutting the left hand party wall by means of a new underpin footing set at a suitable depth and stand-off distance to permit the adjacent excavation to basement level of a new concrete slab. The underpinning would also provide support to a new temporary spine beam spanning across the core excavation

to initially provide support to the Strongback props and subsequently to a new timber frame as described in item 3 below.

3. Installation of a new steel beam at first floor level within the inset left hand flank wall of the rear extension in order to allow removal of the ground floor section of wall which would interfere with the excavation and formation of the new core.

4. Two new flitch beams installed at first floor level to support the existing joists to the front left hand section and rear right hand section at first floor level, the retained existing timber stonework providing support to the second floor and roof above as per the original construction. The front left hand beam spans between the front elevation pier and the end of the short section or rear elevation abutting the left hand party wall. There rear beam stands between the rear elevation of the annexe on to a new post extending down to the temporary steel beam as described in section one above.

5. The installation of two raking shores extending from a new pad footing within the rear left hand garden up to two points along the in-situ rear elevation of the first and second floor level in order to prevent any further rearward movement of that panel.

6. Installation of new concrete corner ties of across the junction between the left hand party wall and the inset rear elevation in order to reinstate the integrity of the junction between the two.

Site works and implications on design

During the installation of the works as described above, particularly the works to install the temporary steel beam at first floor level within the left hand flank wall of the annexe , the masonry to that flank wall above (which had displayed a significant bow in elevation noted that the time of the initial Harold James (London) Ltd survey) had become destabilised. Apparent movement of that wall noted to result from vibrations from passing trains across the rear of the site gave a strong indication that the condition of the wall, albeit restrained at either end by the left hand rear elevation and the main annexe back wall of was not sufficiently robust to be considered safe in the temporary condition.

The original architectural general arrangement drawings allowed for the retention of this left hand flank wall, to be used as an internal wall within the new layout. The rear elevation and short right hand flank wall of the annexe were to be demolished since they could not be accommodated within the design of the new rear of the building

However, reasonable concerns relating to the strength and stability of this wall, along with significant complications arising from a) the provision of temporary lateral restraint to the wall whilst building the adjacent new core, and b) the necessary work required to strengthen the wall even once the surrounding new structure had been installed, gave rise to the decision that the flank wall of the annexe should be taken down of the and rebuilt as a new internal wall as part of the new permanent works.

Removal of wall and installation of lateral stability frame

The rear annexe of the building provided support of to the rear most sections of remaining timber floor at first and second floor level, as well as to the rear slope of the roof at eaves. Therefore as a consequence of the removal of the annexe the rear of the floor panels needed to be supported by a new timber structure, details of which can be found in the appendix.

As a result of the excavation as part of the works that led to the original collapse, the spine partition of the house had been removed at ground floor level and little spine structure was present throughout each floor level. It is unclear how much removal of the existence by structure was down to the original stripping out of the building and how much may have been compromised by the collapse, the that the cause of the condition was deemed moot. In order to provide lateral restraints to the party walls from each side the new timber frame was designed and detailed two connected to the heads of the previous party wall underpins, thereby allowing a suitably propped and sequenced excavation and underpin to be carried out below the timber frame, thereby permitting the formation of the basement and subsequent shorter action of the spine structure ultimately allowing the temporary timber frame to be removed.

The new temporary rear elevation has been installed in order to enclose the building and prevent rainwater ingress from affecting the temporary and original timber structures.

At the time of writing the timber frame and remedial works have been installed in order to ensure that the building is temporarily stable.

While the previously installed steel beams at high level within the front elevation were not designed by Harold James (London) Ltd, it is apparent that they are of sufficient strength and stability in order to be able to act as props between the leading edge of the right hand party wall and the internal masonry pier. The bearing and embedment details are appropriate and they show no signs of distress. Given the retention of the front scaffold to the front elevation in the temporary condition there is no apparent imposed window loading on the masonry pier, and in any case the aspect ratio of the pier is such that it has sufficient depth to resist any such wind load should it occur.

Lateral restraint will be necessary to the foot of the pier in the temporary condition during the sequenced excavation and formation of the new basement, though this is beyond the scope of this report. Further details relating to the sequenced excavation and formation of the new basement can be found in appendix C of this report. It should be noted that at the time of writing the agreed shape and structural scheme for the rear annexe had not been developed.

Appendix A – Sketches of temporary works

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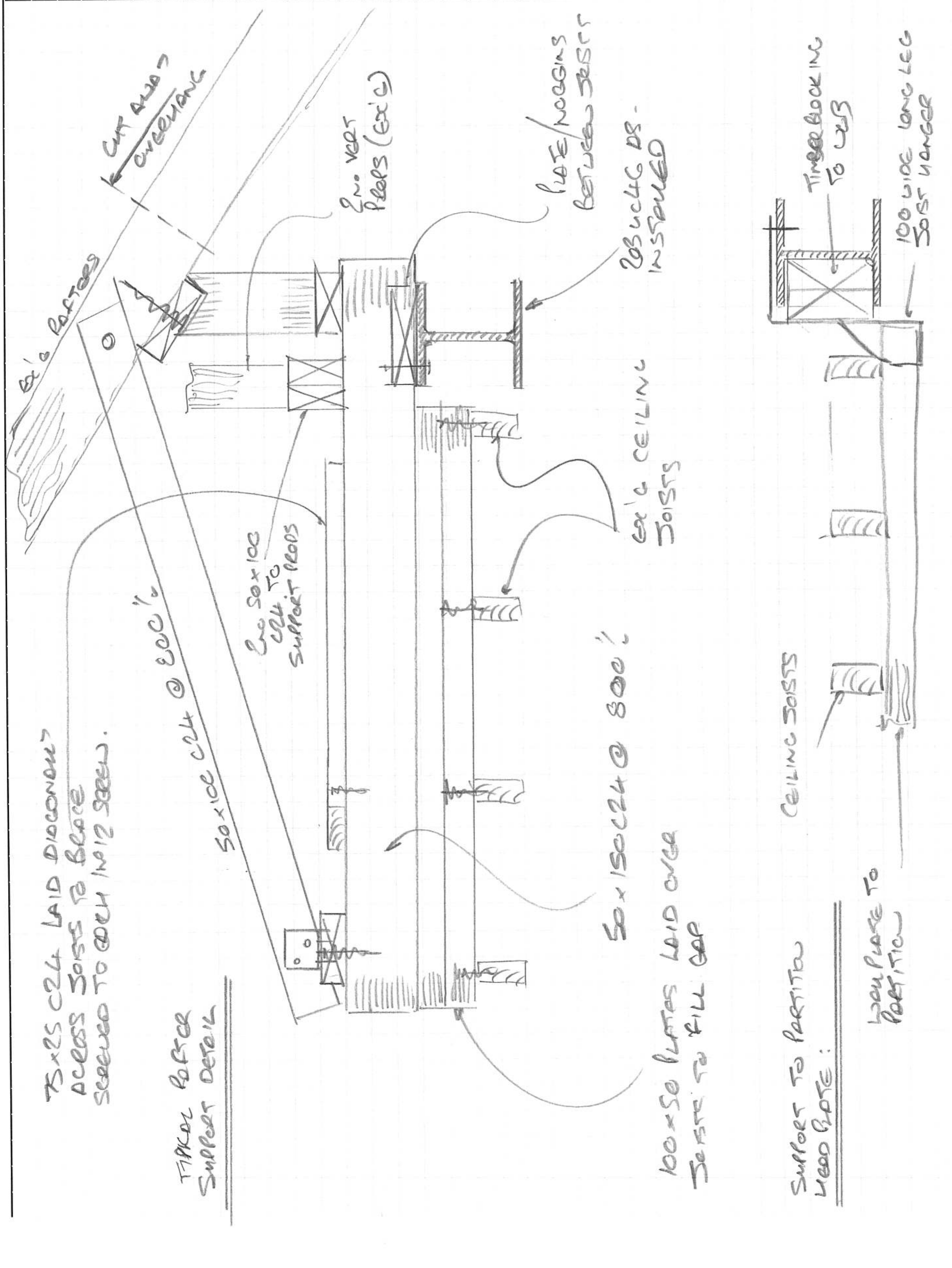
HAROLD JAMES (LONDON) LTD
CONSULTING ENGINEERS

Contract:	Job Ref:
163 SUMATRA RD	20172
Sub-section:	Sheet No: SK100
Prepared by: MS Checked by: Approved by:	Date: 5.10.20



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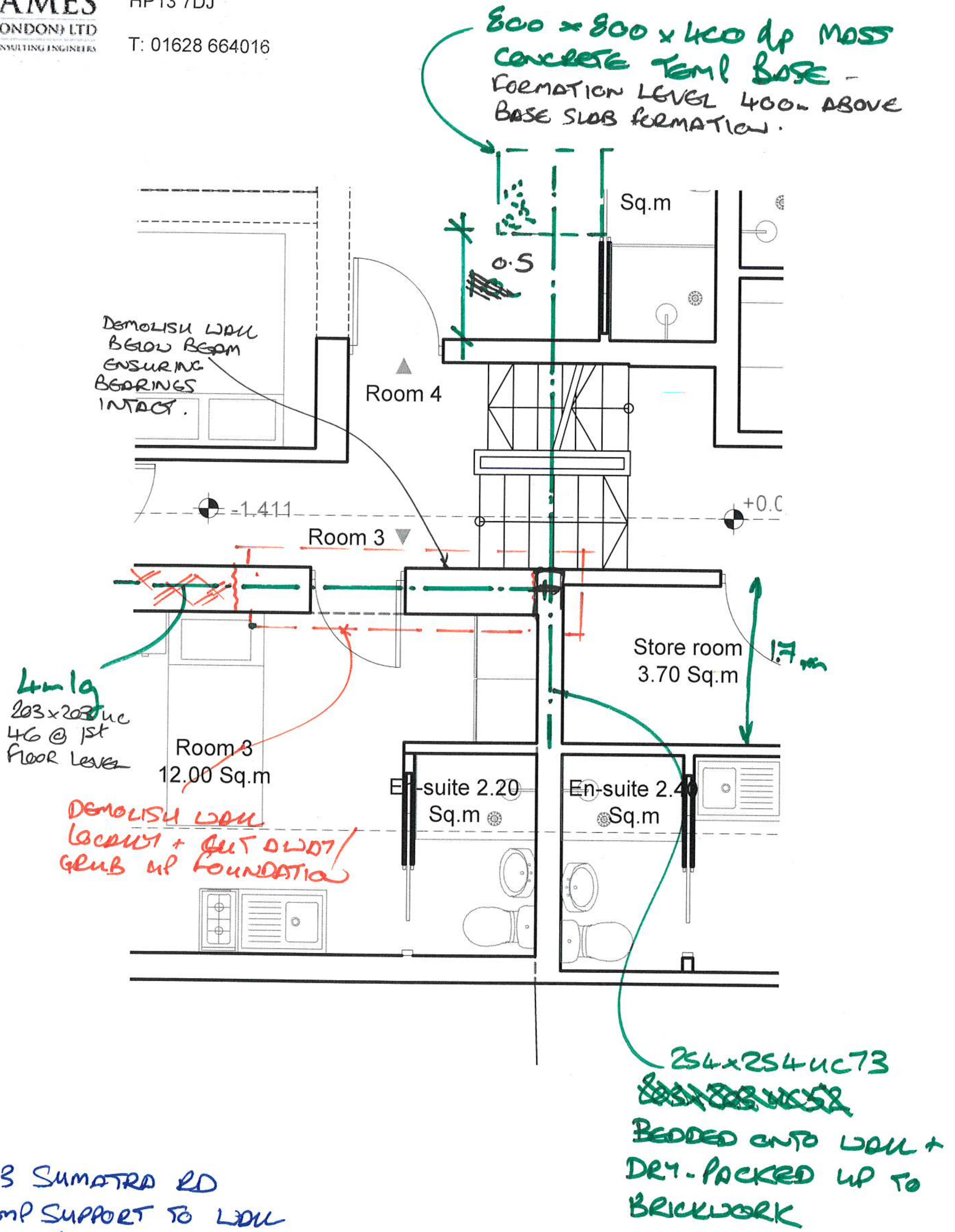

163 SUMATRA RD
TEMP SUPPORT TO WALL
20172/SK ~~100~~ 101 'A'
20.10.20

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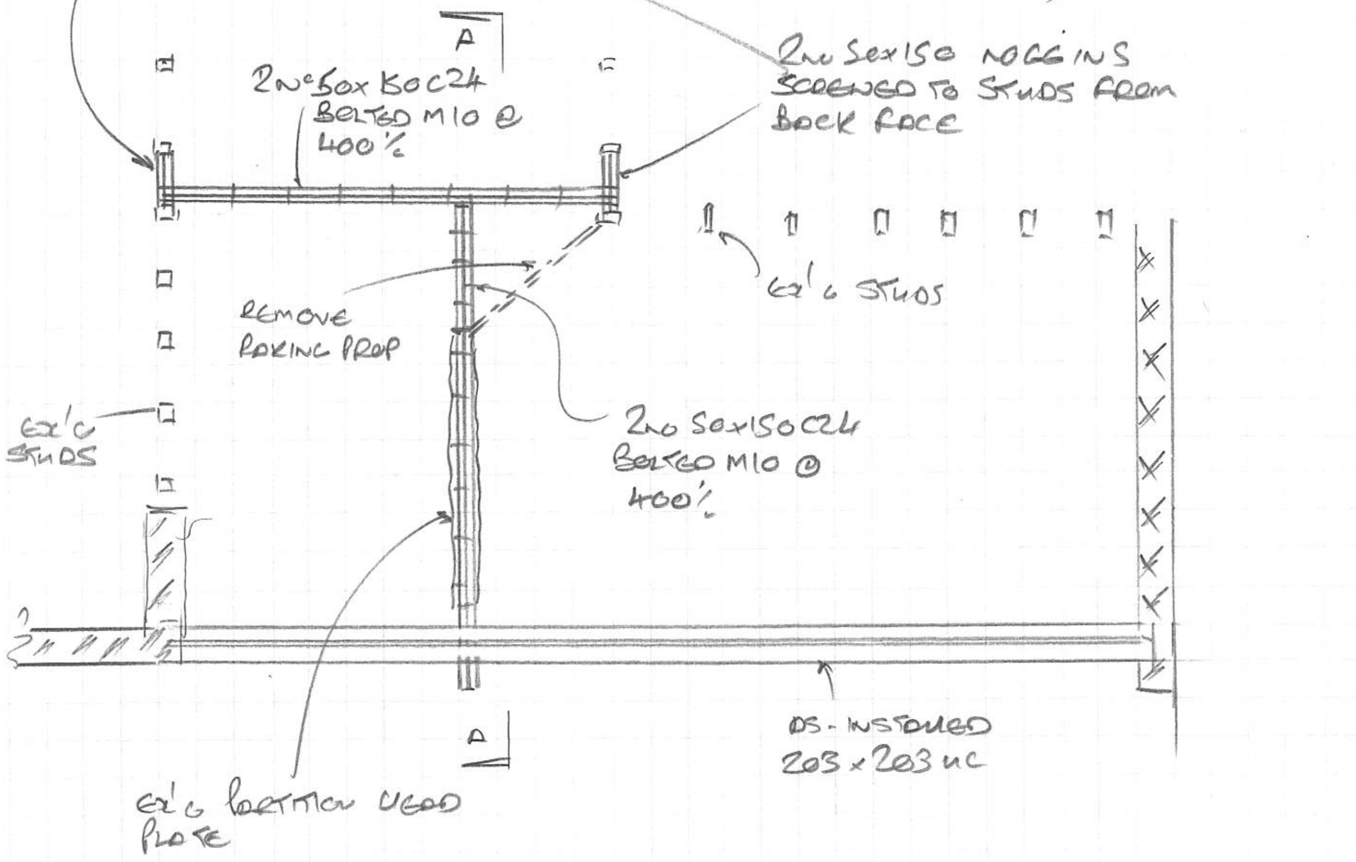
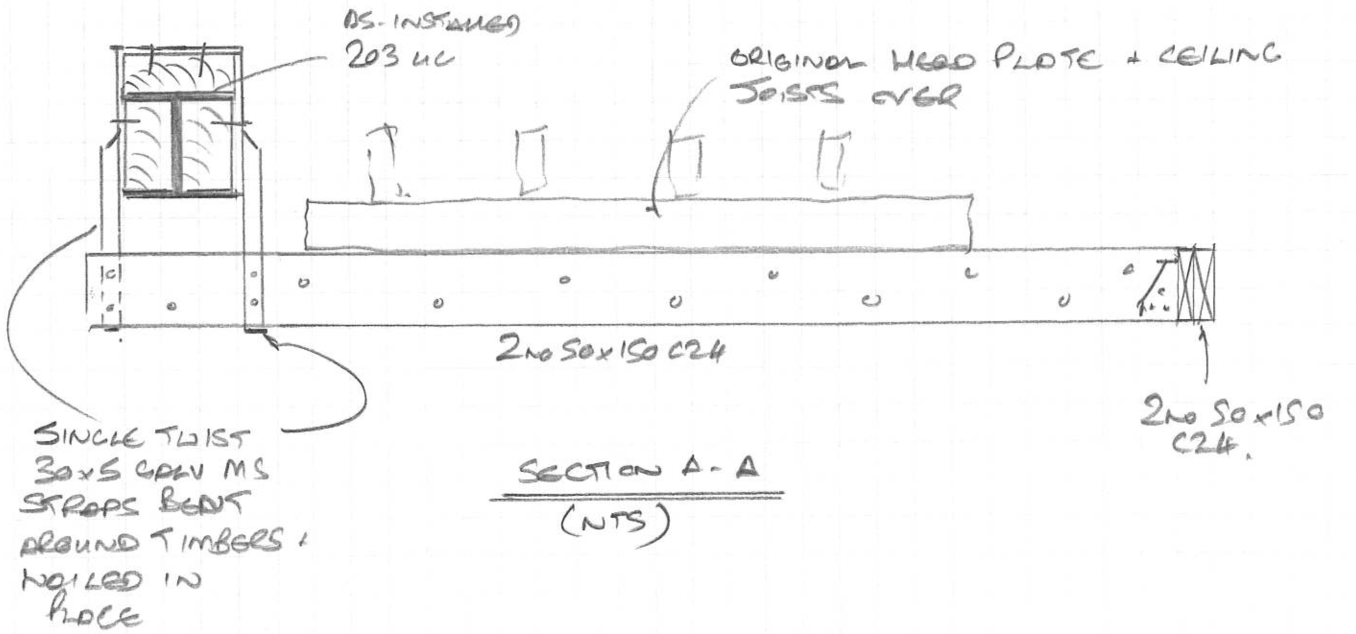
Contract: 163 SUMATRA RD	Job Ref: 20172
Sub-section: SUPPORT TO HEAD PLATE	Sheet No: SK102
Prepared by: MS Checked by: Approved by:	Date: 4.12.20

PART PLAN ON FRONT OF HOUSE
(NTS)





SECTION A - A
(NTS)

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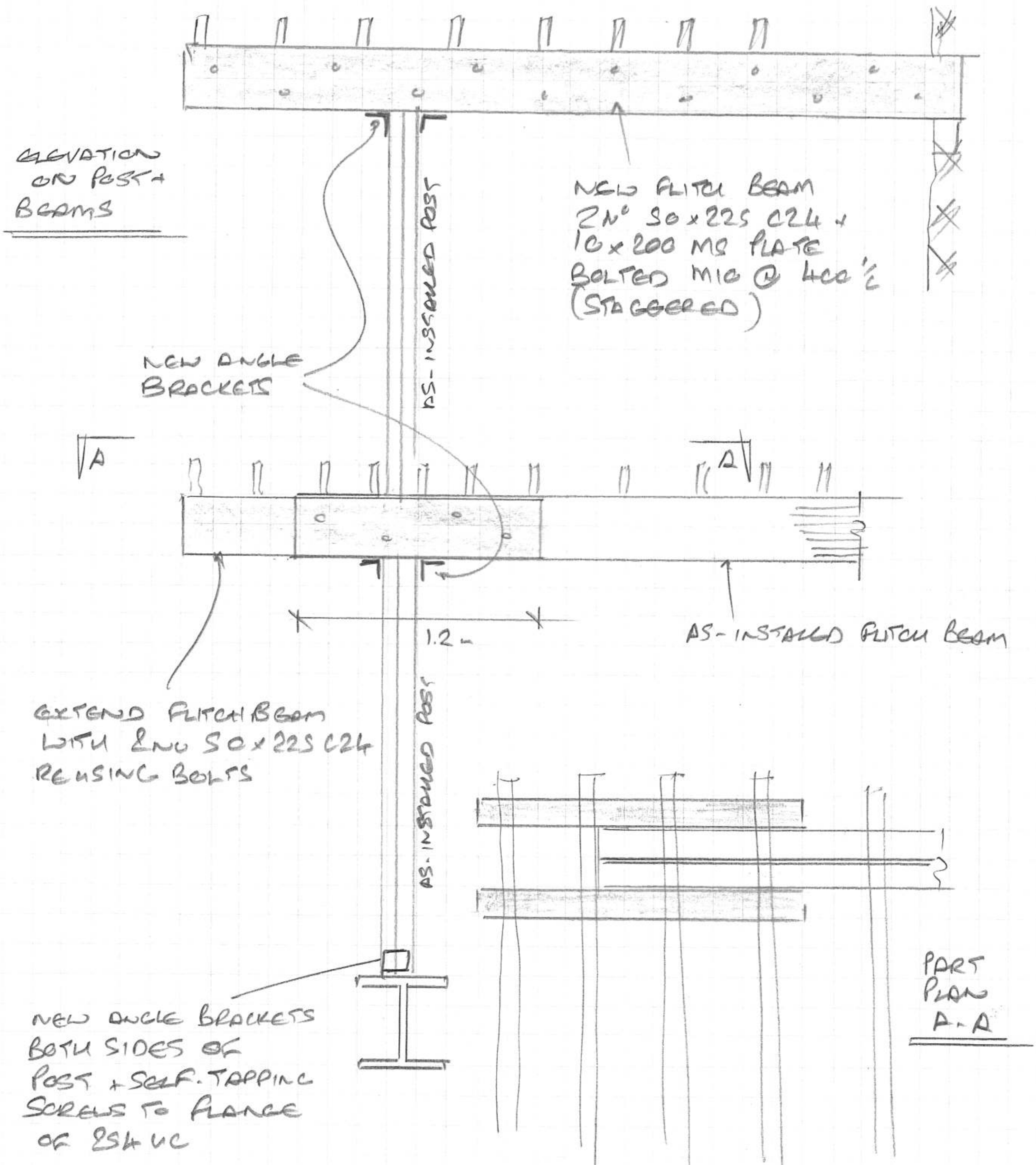
Contract: 163 SUMATRA RD			Job Ref: 20172
Sub-section: FLOOR SUPPORT AMENDMENT.			Sheet No: SK103
Prepared by: MS	Checked by:	Approved by:	Date: 23.2.21



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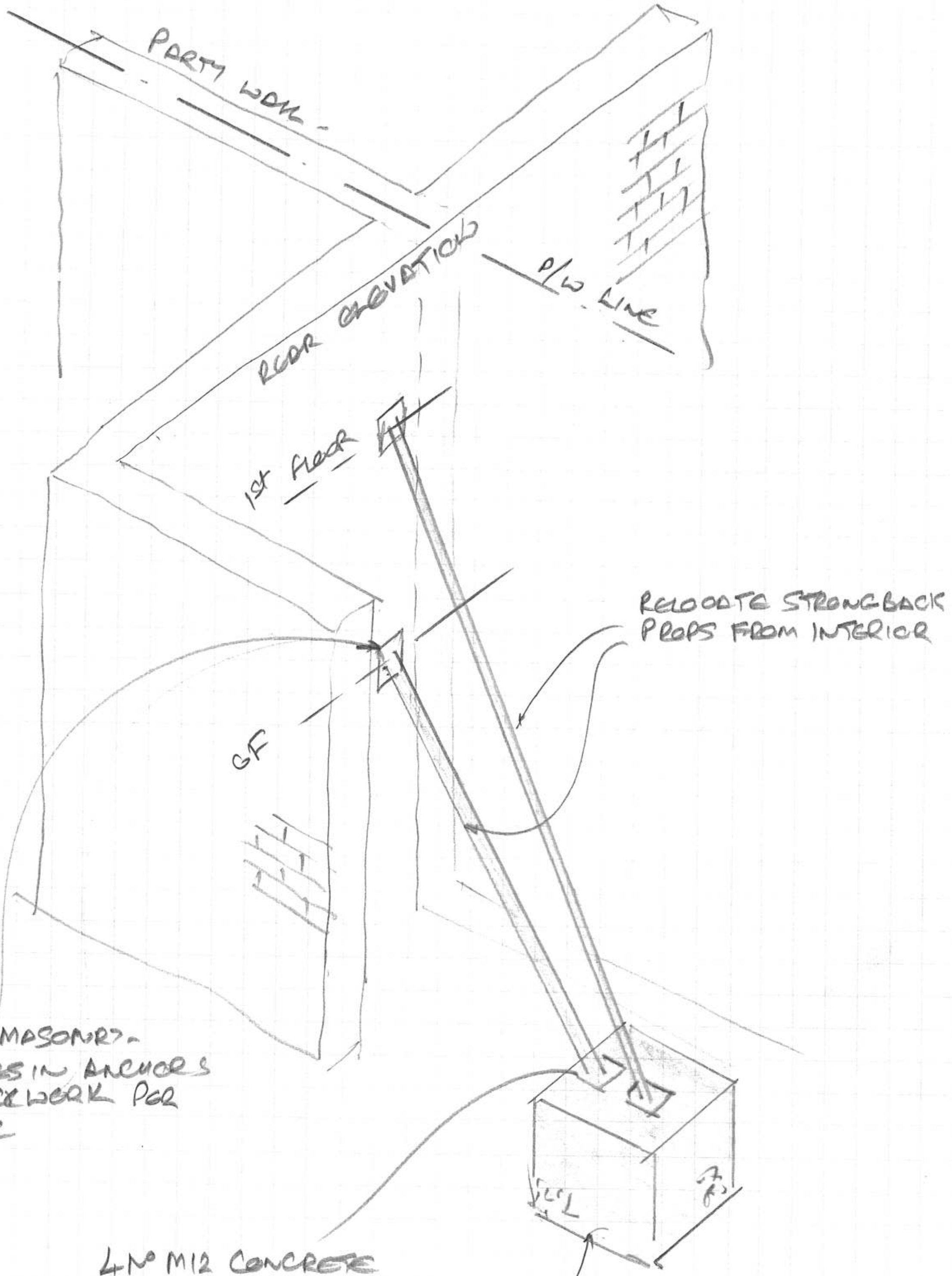
Contract: 163 SUMATRA RD			Job Ref: 20172
Sub-section: TEMP RESTRAINT TO REAR			Sheet No: SK104
Prepared by: MS	Checked by:	Approved by:	Date: 23.2.21



RELOCATE STRONGBACK PROPS FROM INTERIOR

4No M12 MASONRY GRADE RESIN ANCHORS INTO BRICKWORK PER END PLATE

4No M12 CONCRETE GRADE RESIN ANCHORS INTO TOP OF PAD

800 x 800 x 800 GEN 3 MASS CONCRETE PAD CAST INTO BACK GARDEN

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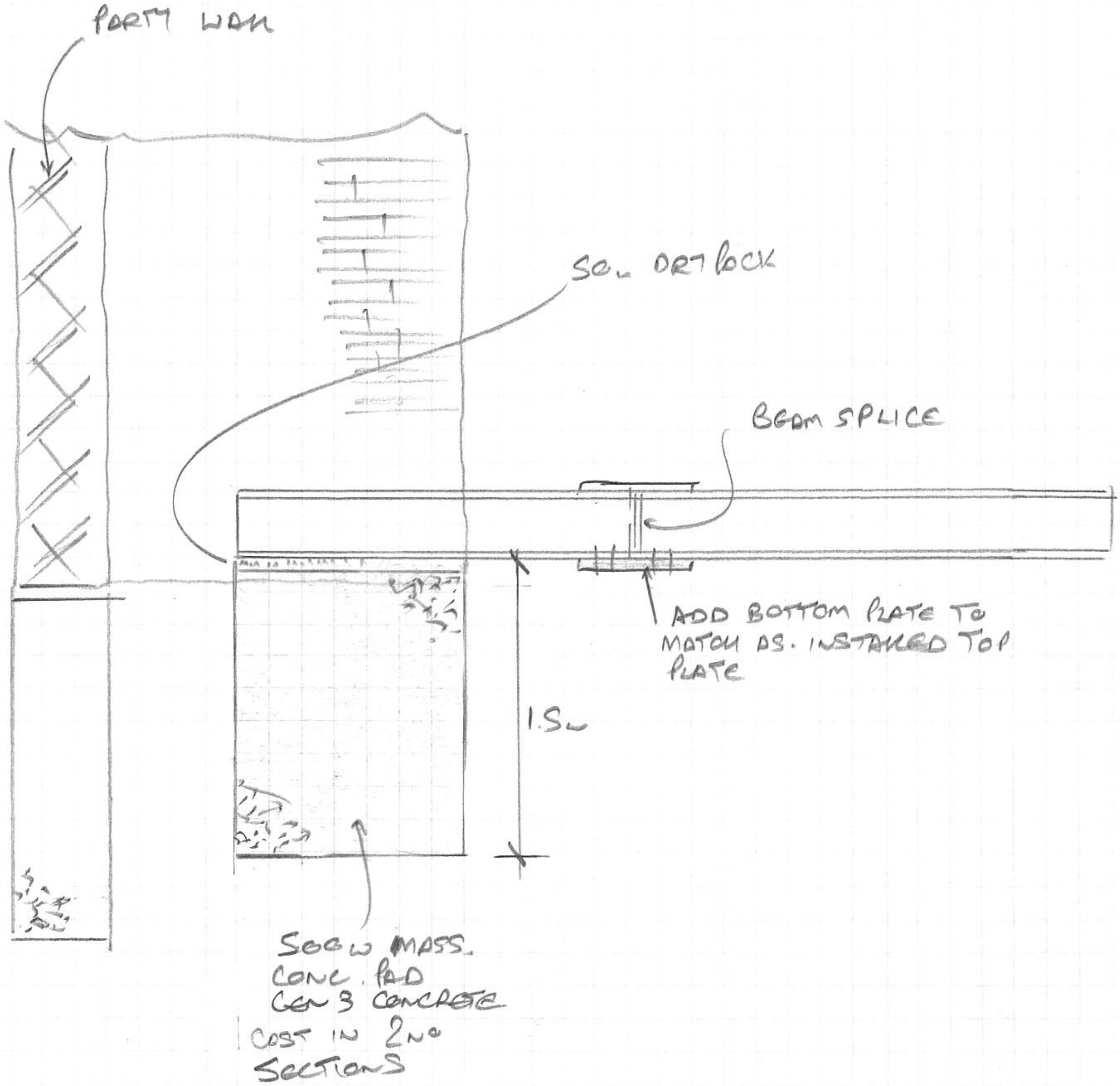
Contract:	Job Ref:
163 SUMATRA RD	20172
Sub-section: SUPPORT TO TEMP BEAM	Sheet No: SK105
Prepared by: MS / Checked by: / Approved by:	Date: 23.2.21



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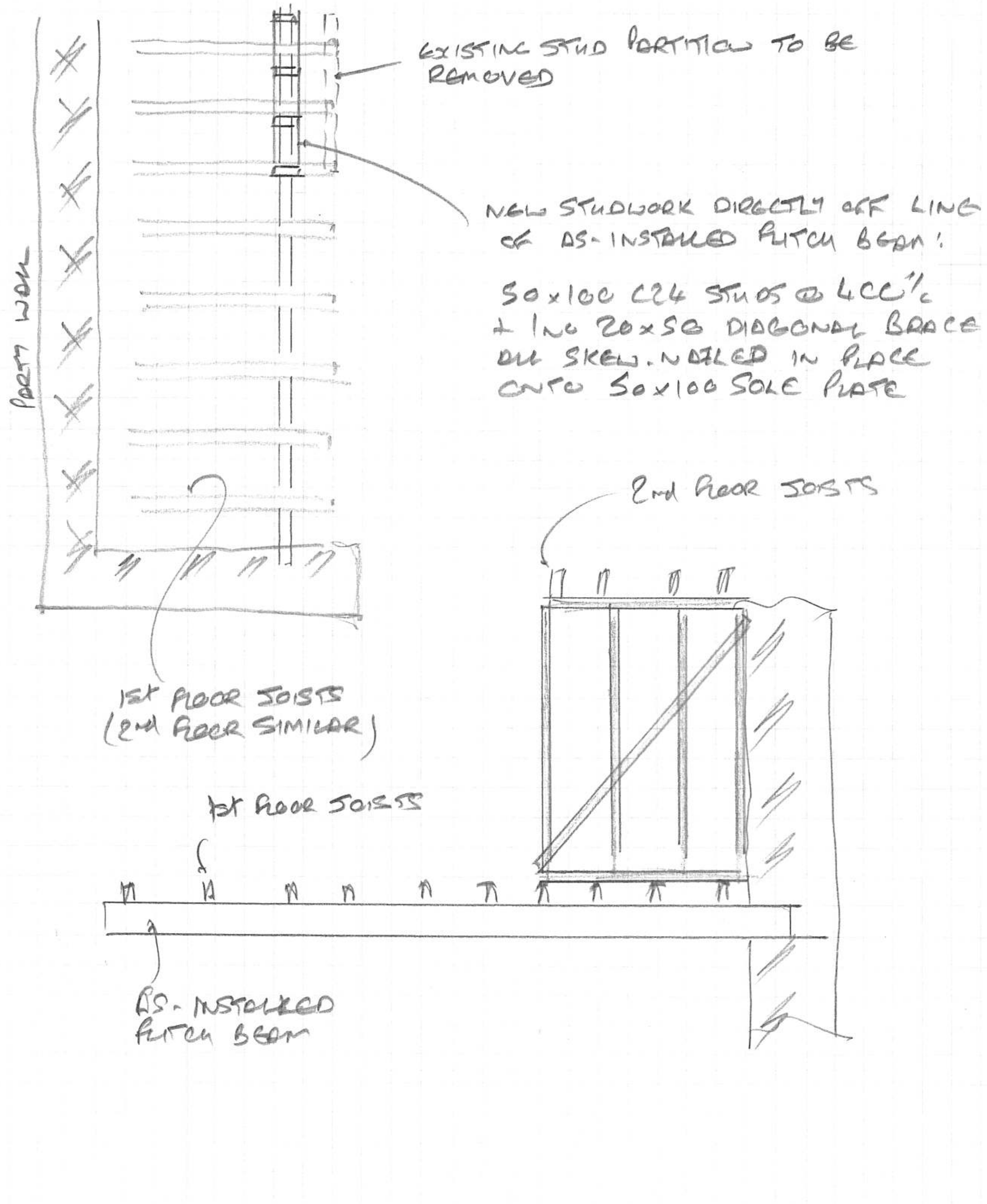
Contract:	Job Ref:
103 SUMATRA RD	20172
Sub-section:	**Sheet No:**
FRONT LH TIMBER SUPPORT	SK106
Prepared by: MS **Checked by:** **Approved by:**	**Date:** 23.2.21



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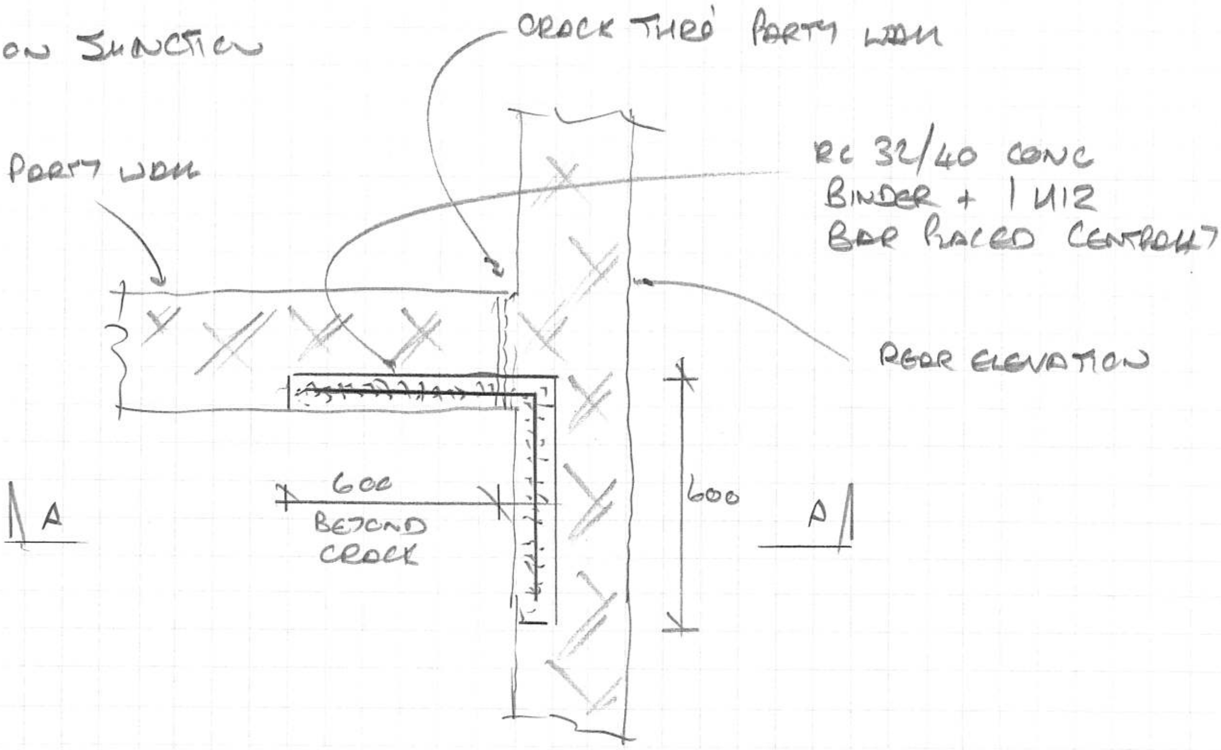
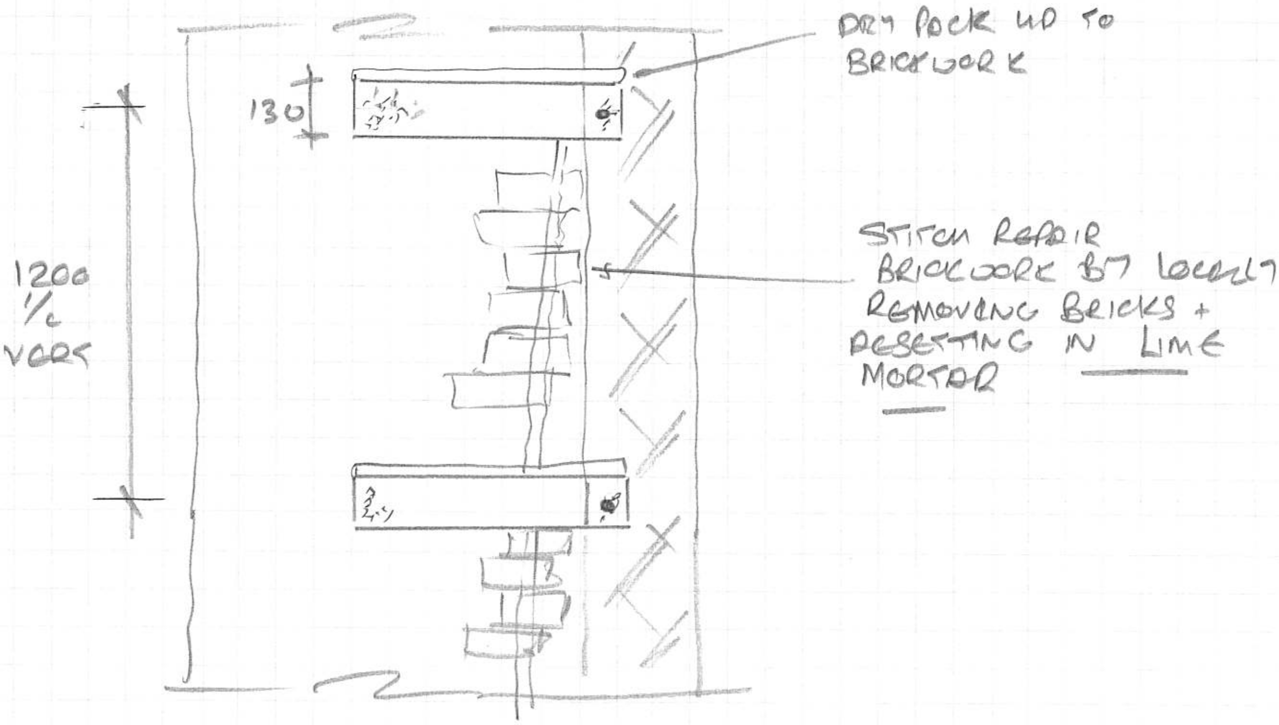
Contract: 165 SUMATRA RD			Job Ref: 20172
Sub-section: RESTRAINT TO REAR WALL			Sheet No: SK107
Prepared by: MS	Checked by:	Approved by:	Date: 24.2.21

PLAN ON JUNCTION





SECTION A-A.

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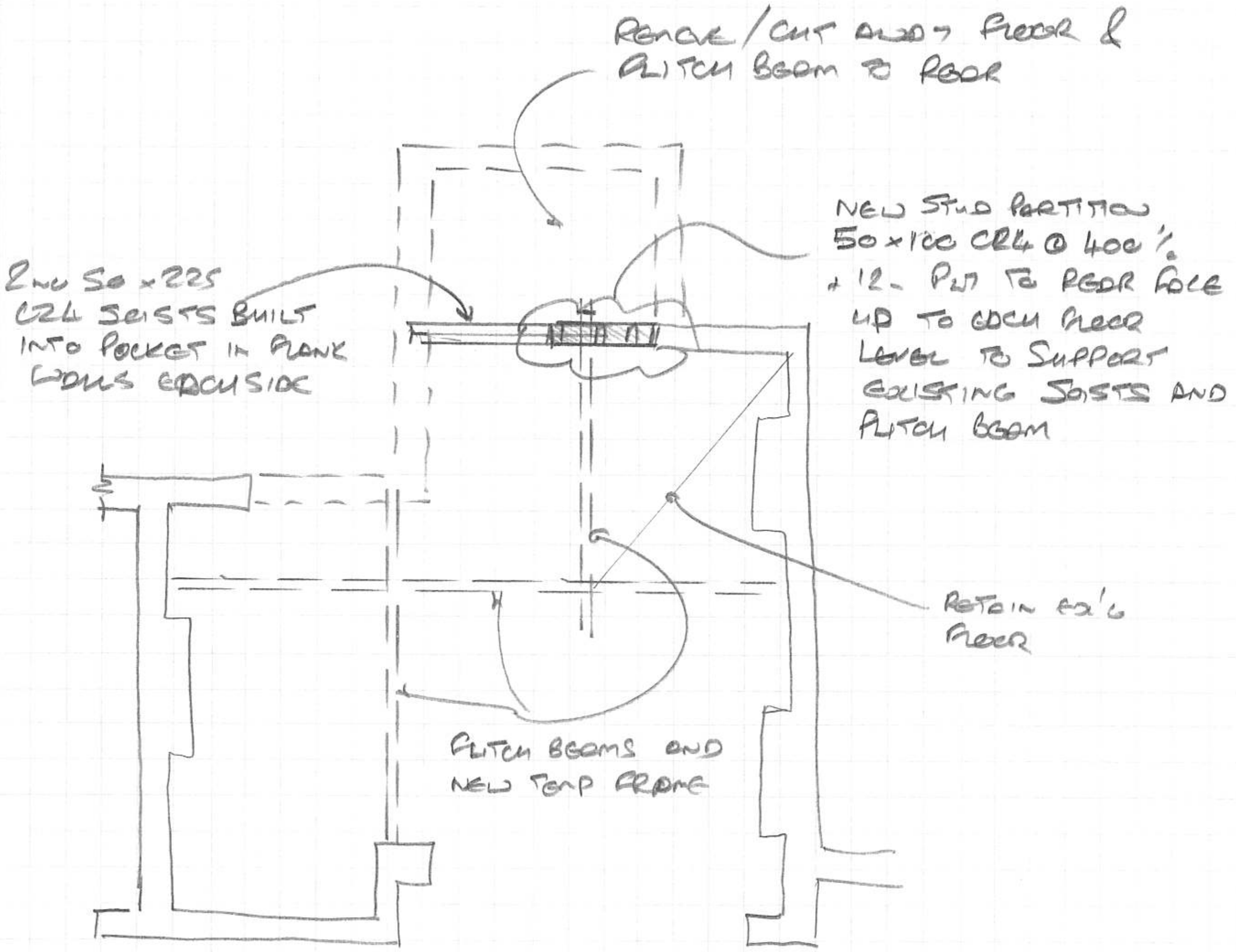
Contract:	Job Ref:
163 SUMATRA RD	20172
Sub-section: TEMP STUD TO REAR	Sheet No: SK112
Prepared by: MS / Checked by: / Approved by:	Date: 12.8.21



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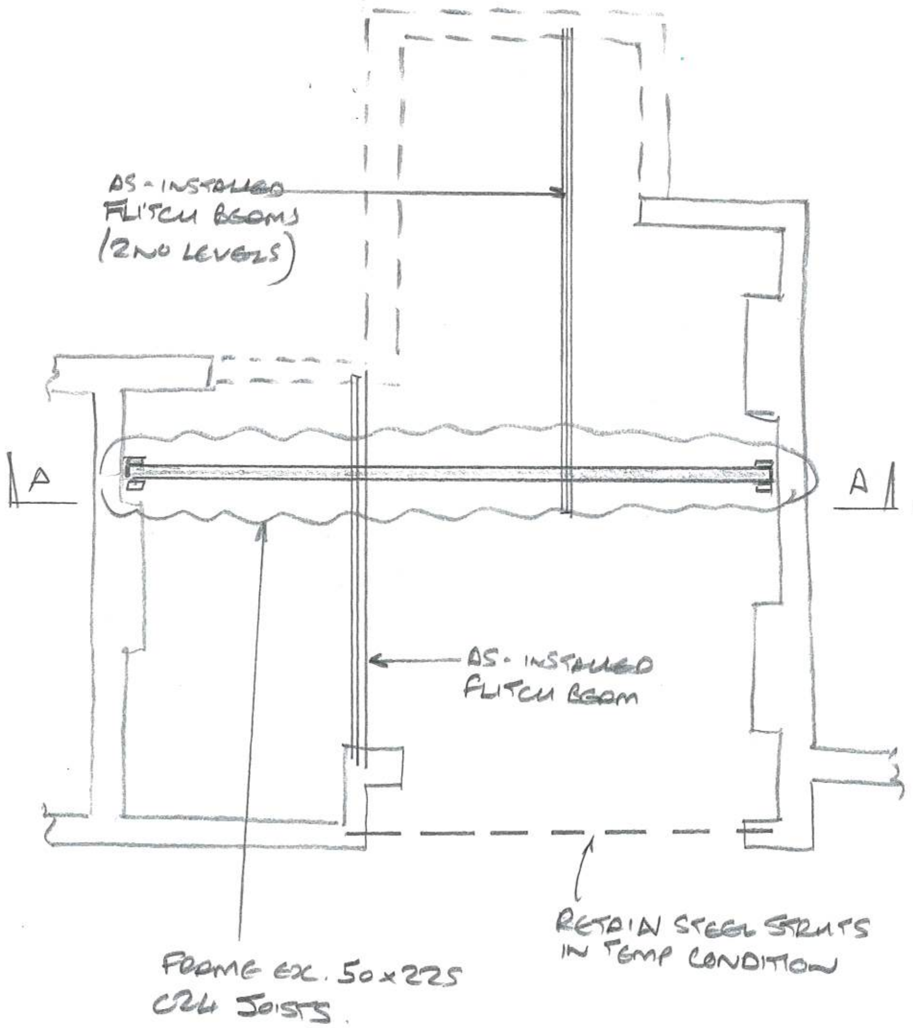
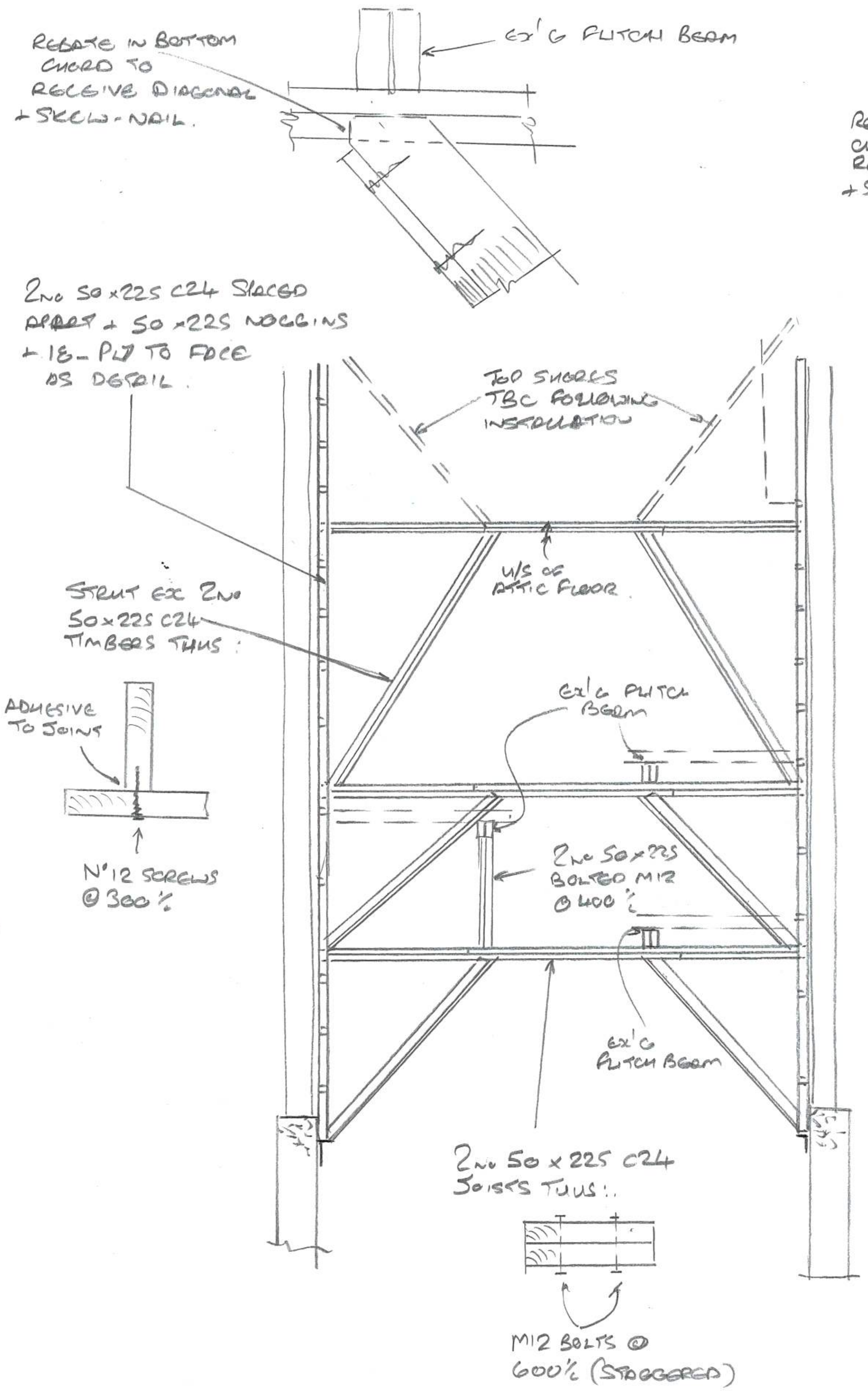
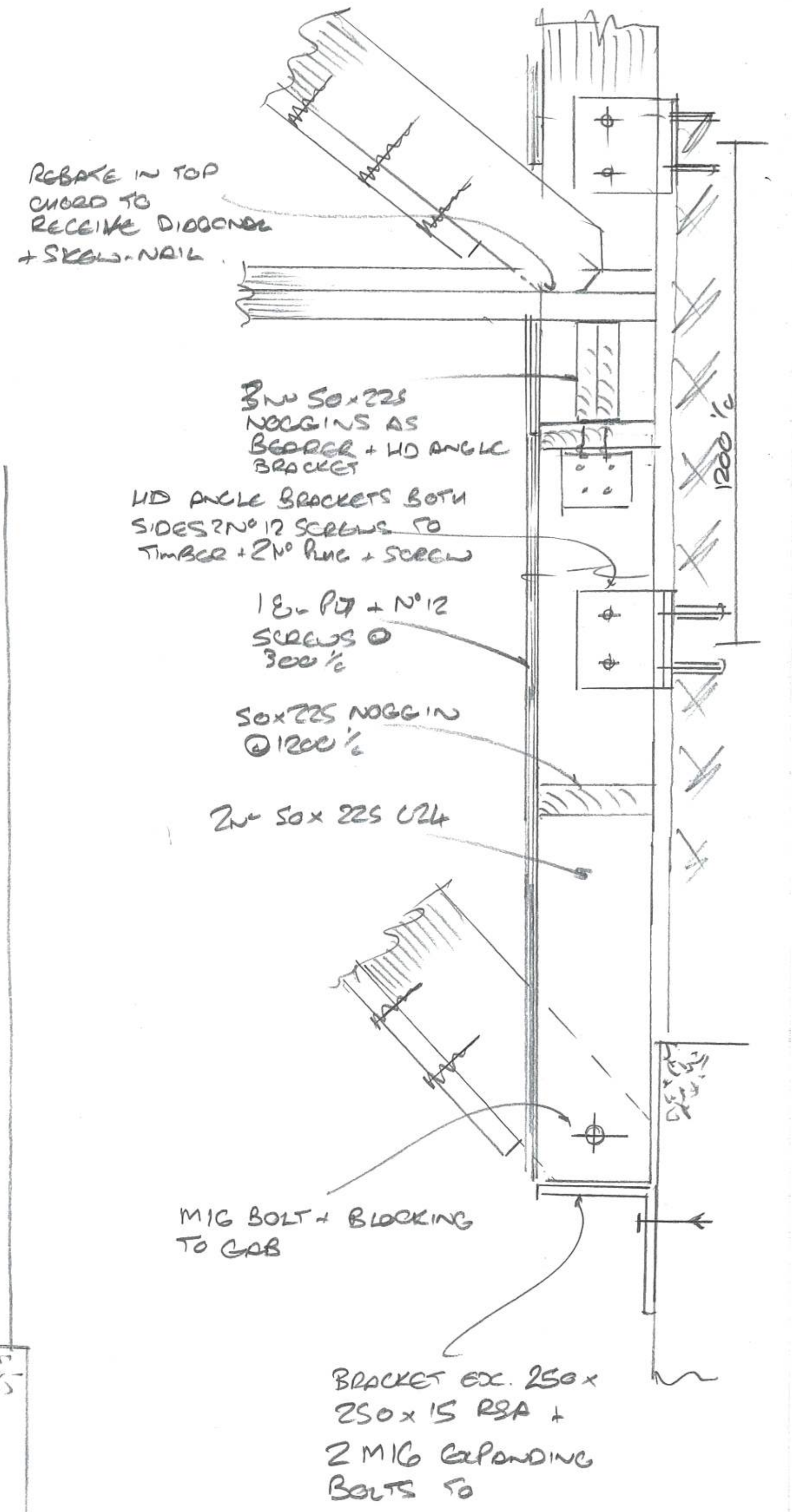


PLAN ON NEW FRAME



SECTION A-A



NOTES - ALL JOINTS TO LONGITUDINAL TIMBERS TO BE STAGGERED TO CREATE 3m OVERLAP.

163 SUMATRA RD
SUPPORT FRAME TO RESTRAIN PARTY WALLS
20172/SK11
29.7.21

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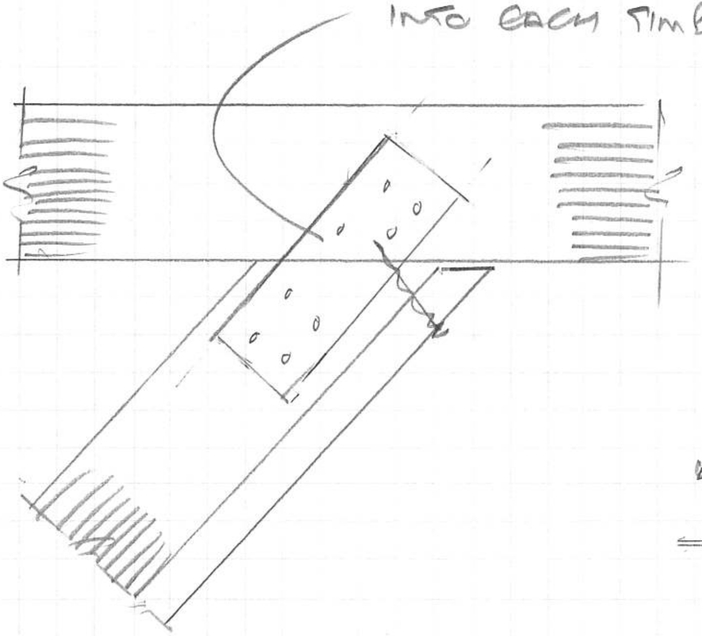
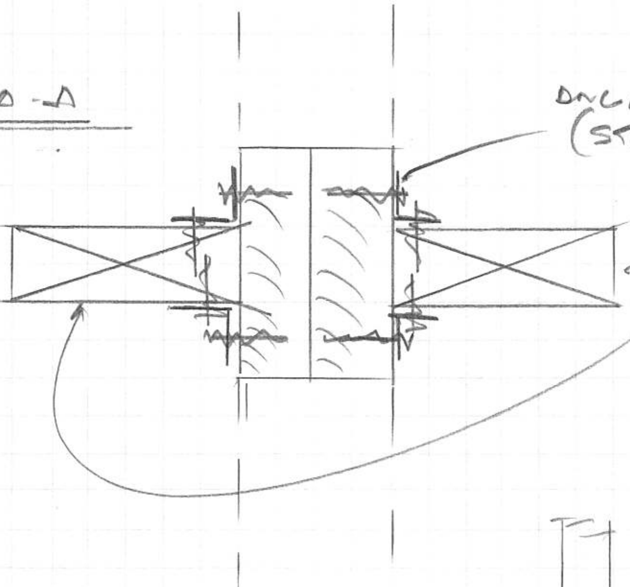
Contract:			Job Ref:
163 SUMATRA RD			20172
Sub-section: FRAME REMEDIAL WKS			Sheet No: SK115
Prepared by: MS	Checked by:	Approved by:	Date: 2.11.21

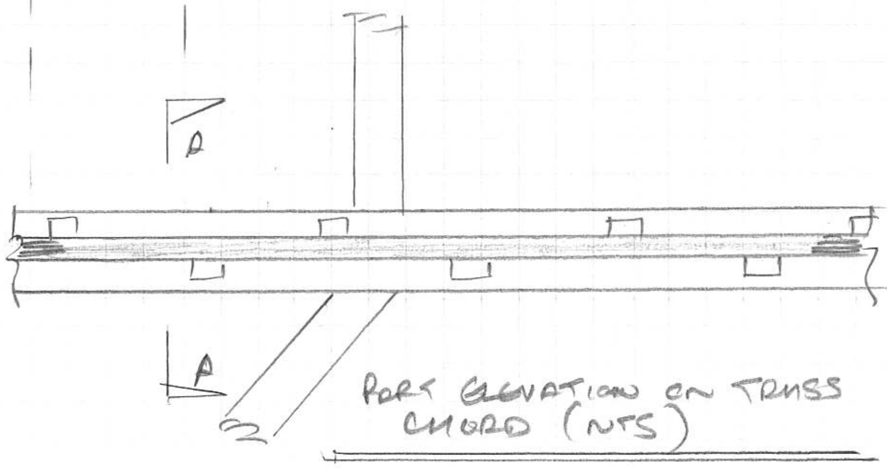


UPGRADE OF FRAME JOINT (NTS)

SECTION A-A



BRACING TO HORIZ. TRUSS CHORD



PART ELEVATION ON TRUSS CHORD (NTS)

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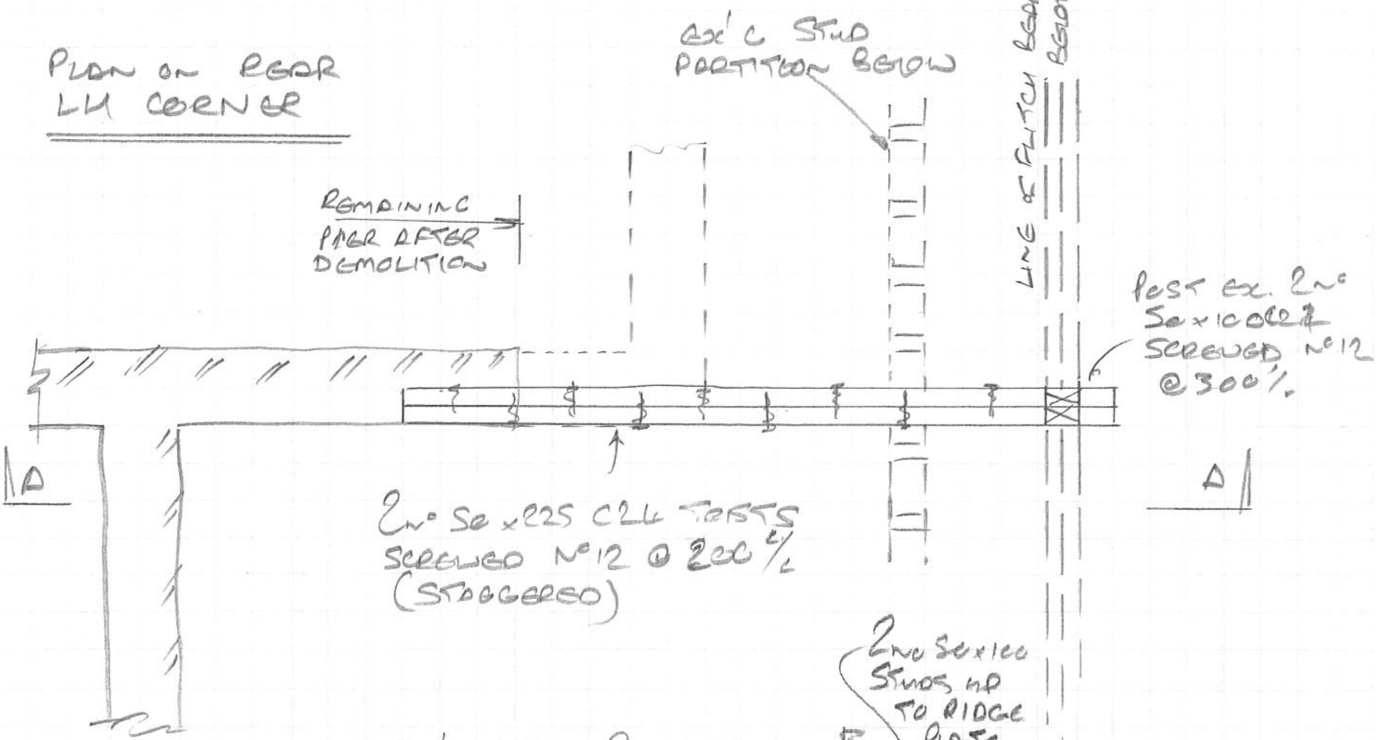
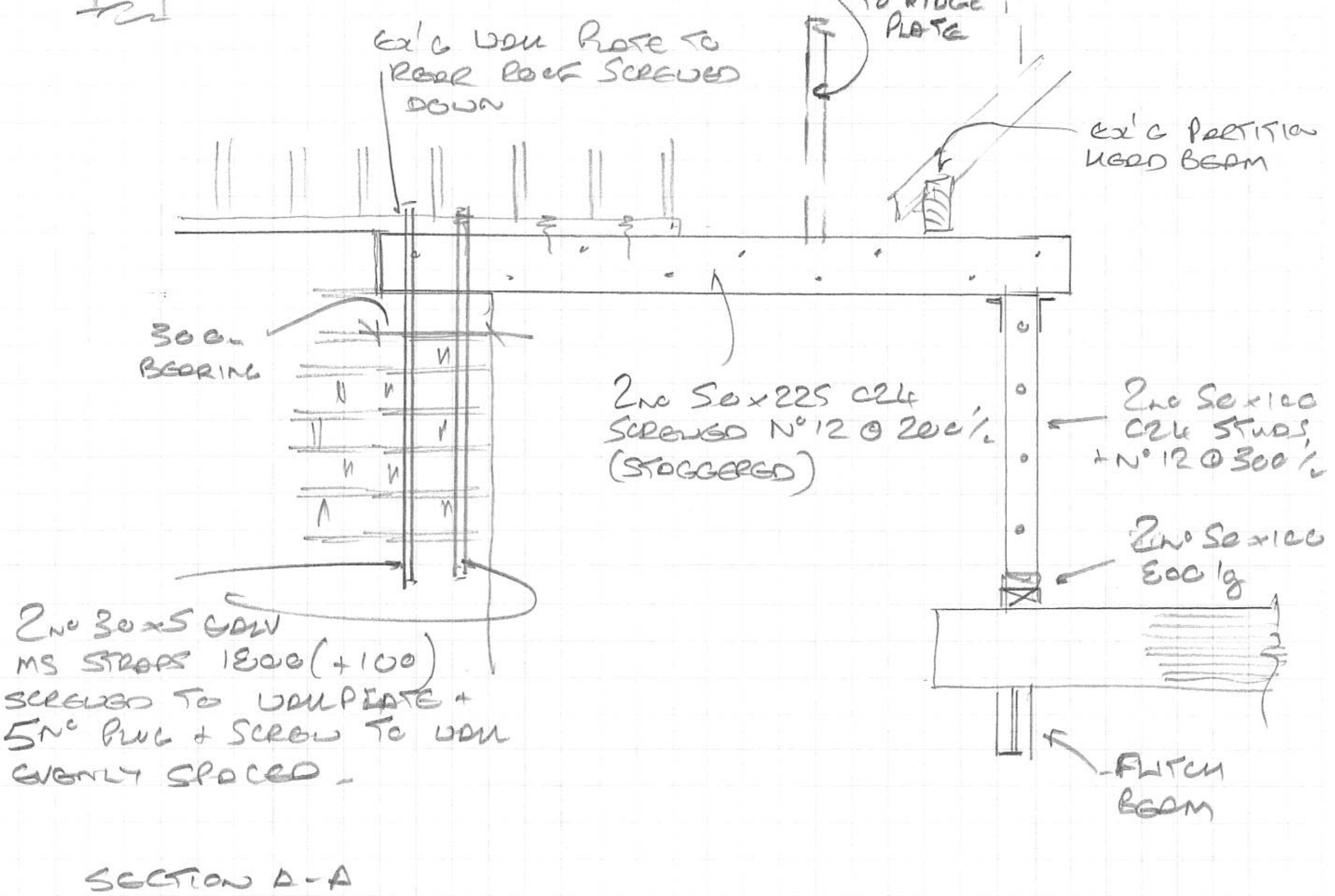
Contract: 163 SUMATRA ROAD	Job Ref: 20172
Sub-section: SUPPORT TO REAR ROOF	Sheet No: SK114'A'
Prepared by: MS / Checked by: / Approved by:	Date: 27.10.21

PLAN ON REAR LH CORNER





SECTION A-A

REV A. 2.11.21. POST UP TO RIDGE ADDED

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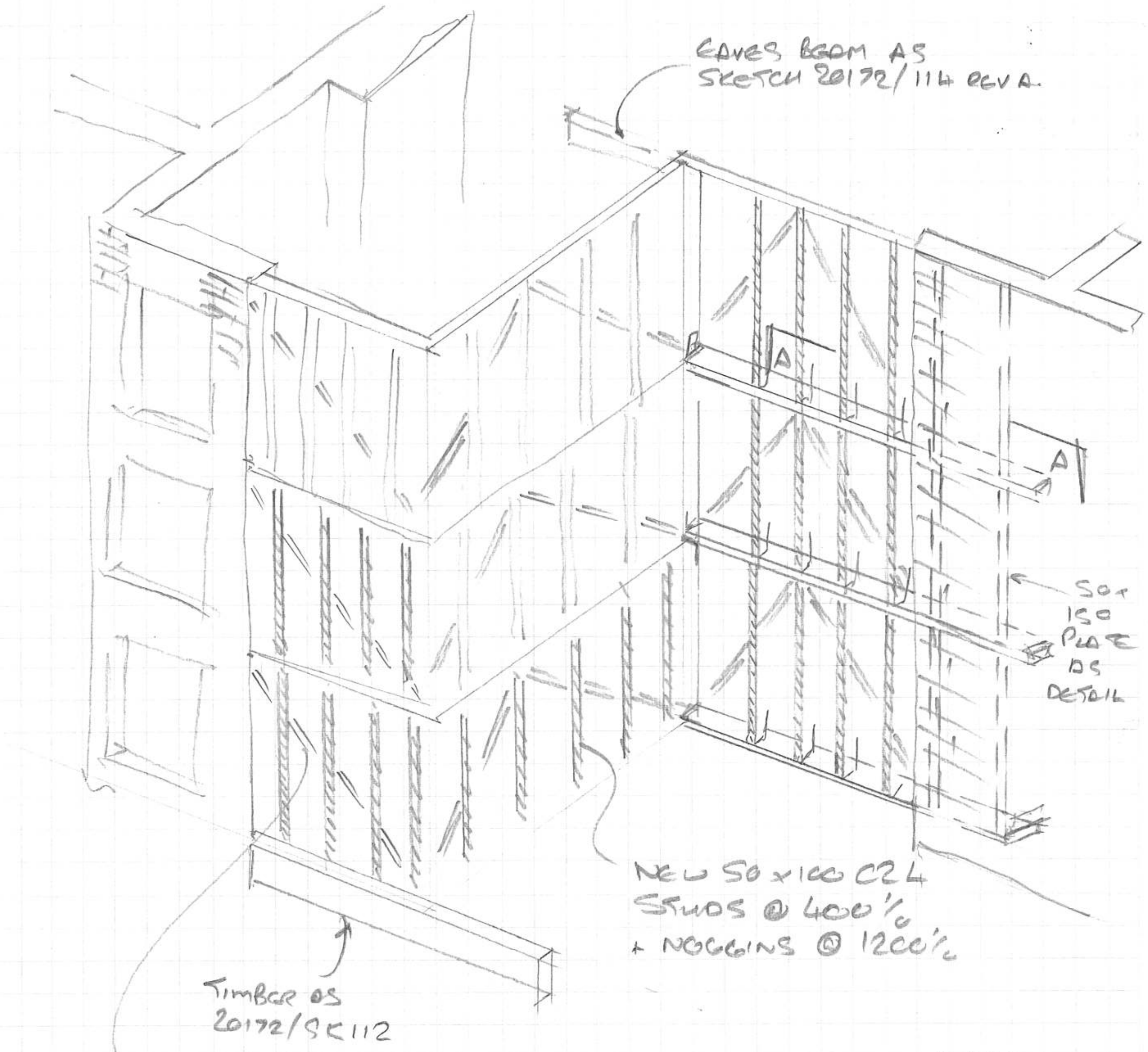
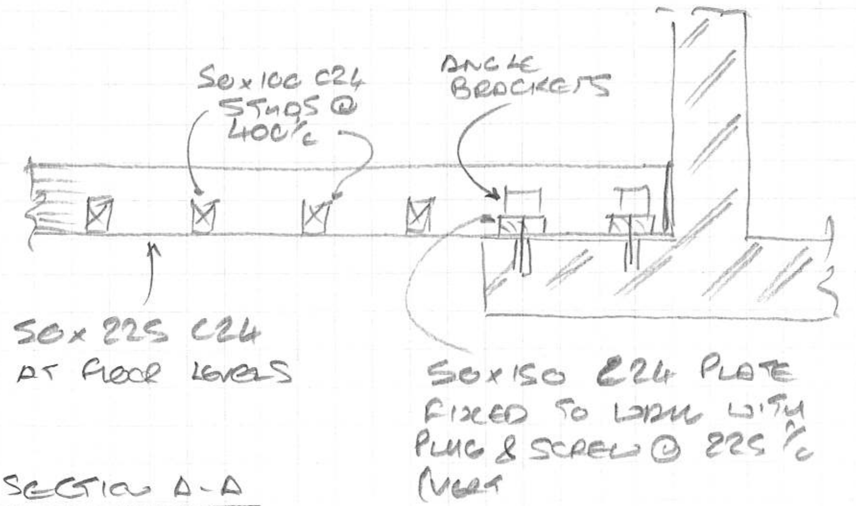
Contract:			Job Ref:
163 SUMATRA RD			20172
Sub-section: ENCLOSURE TO REAR			Sheet No: SK117
Prepared by: MS	Checked by:	Approved by:	Date: 10.11.21





SECTION A-A

Appendix B – supporting calculations to temporary works design

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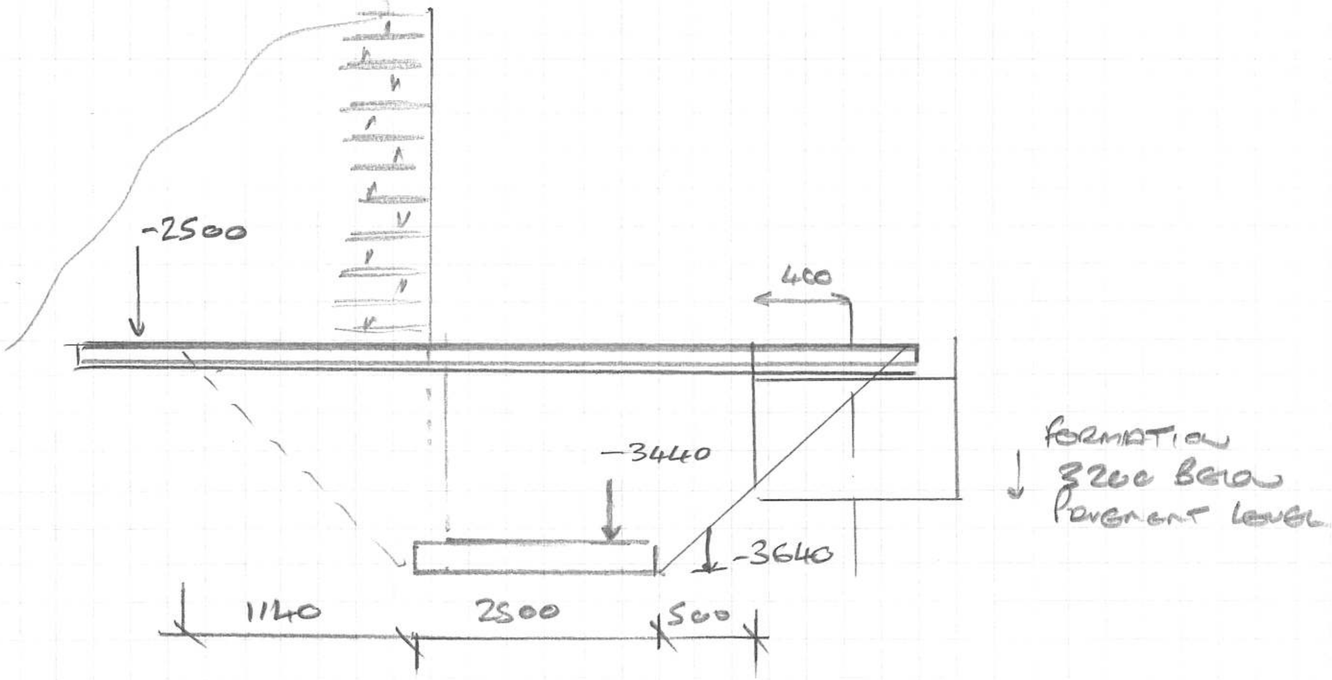
Contract: 163 SUMATRA RD	Job Ref: 20172
Sub-section:	Sheet No:
Prepared by: MS Checked by: Approved by:	Date: 27.11.20

STEELS TO SUPPORT WALL & TIMBERS :-

CONSIDER REQ'D LENGTH TO AVOID UNDERMINING :-



WEIGHT OF WALL - HEIGHT $2.4 \times 4 \times 20 \times 0.215 = 42$ kN/L

LOAD FROM CROSS-WALL $= 42 \times 3.0/2 = 63$ kN

∴ LOAD ON PAD - 807 800 SQ

$R = \dfrac{42 \times 1.14^2/2 + 63 \times 1.14}{4.54} = 22$ kN

⇒ PRESSURE BELOW FOOTING $= 22/0.64 = 34$ kN/m² - OK

$M_u = 22 \times 3.4 = 75$ kN m $\times 1.4 = 104.72$ kNm

USE 254×254 UC73 - CHECK BY SUPERBEAM + INCL. POST LOAD - OK

HAROLD JAMES (London) Ltd. (Consulting Engineers)

Atlantic House, Gomm Road High Wycombe HP13 7DJ

Site: 163 Sumatra Road	**Made by MS**
Job: Temp Works	**Page 4**
Job number: 20172	**File copy**

SuperBeam 4 421711h — **Temp works design 20-11-27.SBW** — **Printed 27 Nov 2020 14:40**

Beam: beam below inset wall — **Span: 4.540 m.**

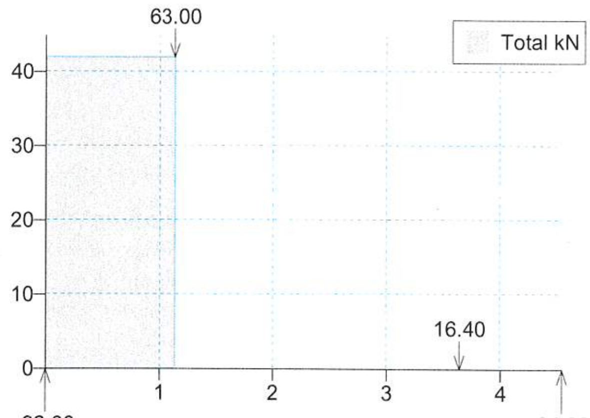
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
R T	Wall	42	0		1.14	41.87	6.01
P T	Wall	63	1.14			47.18	15.82
P T	post	16.4	3.64			3.25	13.15
					Total:	**92.30**	**34.98**

Load types: R:Part UDL P:PL (positions in m. from R1) T: Total

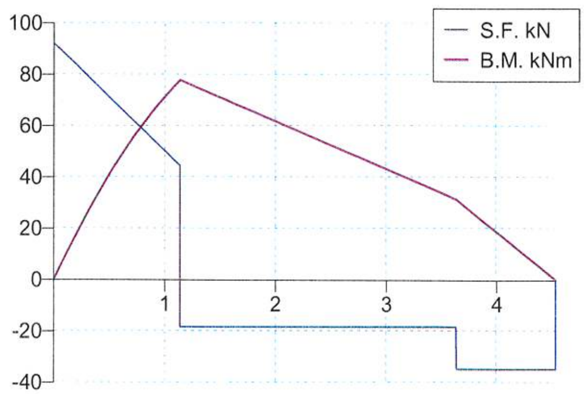
Maximum B.M. = 77.93 kNm at 1.14 m. from R1

Maximum S.F. = 92.30 kN at R1

Total deflection = 137.8 x 1e8/EI at 2.04 m. from R1 (E in N/mm^2, I in cm^4)



Beam loading



Shear force and bending moment

Steel calculation to BS449 Part 2 using Grade 43 steel

SECTION SIZE : 254 x 254 x 73 UC

D=254.1 mm B=254.6 mm t=8.6 mm T=14.2 mm I_x=11,400 cm^4 r_y=6.48 cm Z_x=898 cm^3

L_E/r_y = 4.54x100/6.48 = 70 D/T = 17.9

Permissible bending stress, p_{bc} = 164.0 N/mm^2 (Table 3a)

Actual bending stress, f_{bc} = 77.93 x 1000/898.0 = 86.8 N/mm^2 OK

Maximum shear in web, f_s = 92.30 x 1000/(8.6 x 254.1) = 42.2 N/mm^2 OK

Check on unstiffened web capacity with an applied load of 92.30 kN

Bearing: Required web length at root fillet level = 92.30 x 1000/(8.60 x 210) = 51.1 mm
Required stiff bearing length = 51.1 - (T+r)/tan30 = 4.5 mm

Stiffeners are required if reactions taken on bottom flange and stiff bearing length is less than 4.52 mm

Total deflection = 138 x 1e8/205,000 x 11,400 = 5.9 mm (0.0013 L) OK (L/360=12.6mm)

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Contract: 163 SUMATRA RD			Job Ref: 20172
Sub-section:			Sheet No:
Prepared by: MS	Checked by:	Approved by:	Date: 27.11.20

BEAM OVER INSET FRONT WALL.

SPAN = 3.5m, UDL = 21 kN/m (DL ONLY) - 2NO STOREYS

DESIGN BY SCI. 203 x 203 UC 46 OK. 4mm DEFLECTION

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Site: 163 Sumatra Road	**Made by MS**
Job: Temp Works	**Page 5**
Job number: 20172	**File copy**

SuperBeam 4 421711h — Temp works design 20-11-27.SBW — Printed 27 Nov 2020 14:40

Beam: beam below inset flank wall — **Span: 3.500 m.**

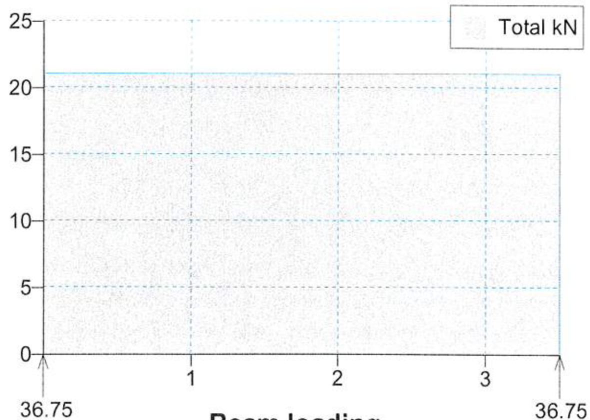
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
U T	Wall	21	0		L	36.75	36.75
					Total:	**36.75**	**36.75**

Load types: U:UDL (positions in m. from R1) T: Total

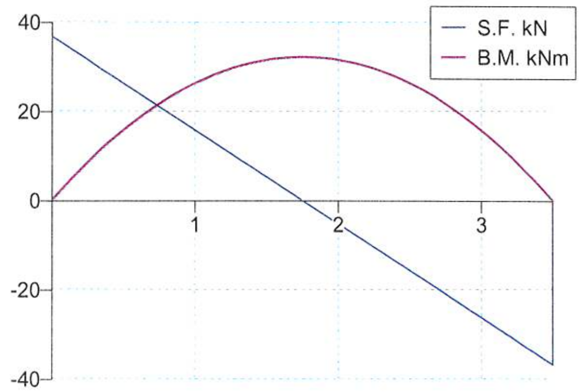
Maximum B.M. = 32.16 kNm at 1.75 m. from R1

Maximum S.F. = 36.75 kN at R1

Total deflection = 41.04 x 1e8/EI at 1.75 m. from R1 (E in N/mm^2, I in cm^4)



Beam loading



Shear force and bending moment

Steel calculation to BS449 Part 2 using Grade 43 steel

SECTION SIZE : 203 x 203 x 46 UC

D=203.2 mm B=203.6 mm t=7.2 mm T=11.0 mm I_x=4,570 cm^4 r_y=5.13 cm Z_x=450 cm^3

L_E/r_y = 3.50x100/5.13 = 68 D/T = 18.5

Permissible bending stress, p_{bc} = 165.3 N/mm^2 (Table 3a)

Actual bending stress, f_{bc} = 32.16 x 1000/450.0 = 71.5 N/mm^2 OK

Maximum shear in web, f_s = 36.75 x 1000/(7.2 x 203.2) = 25.1 N/mm^2 OK

Unstiffened web capacity at bearings (min) = 55.5 kN (web crushing limit with b_1=0)

Total deflection = 41.0 x 1e8/205,000 x 4,570 = 4.4 mm (0.0013 L) OK (L/360=9.72mm)

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Contract: 163 SUMATRA RD	Job Ref: 20172
Sub-section:	Sheet No:
Prepared by: MS Checked by: Approved by:	Date: 20.1.20

SPLICE TO TEMP BEAM :-

ULS MOMENT = 77.93 kNm - ALLOW 100 kNm

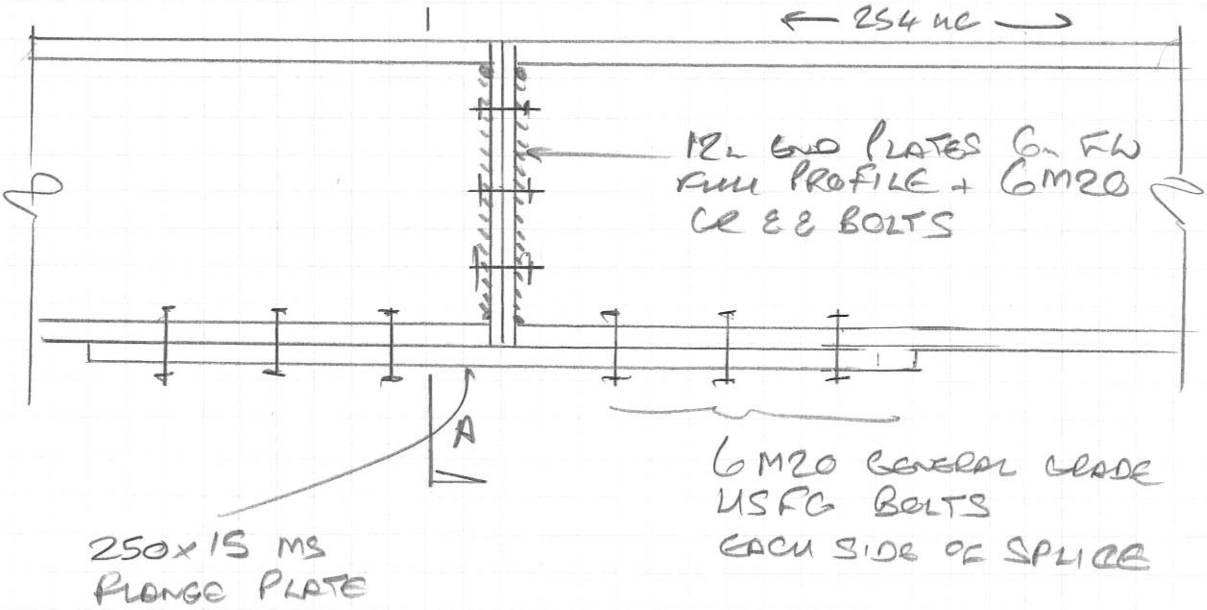
FORCE ON BOLT GROUP = $100/0.256$ = 400 kN

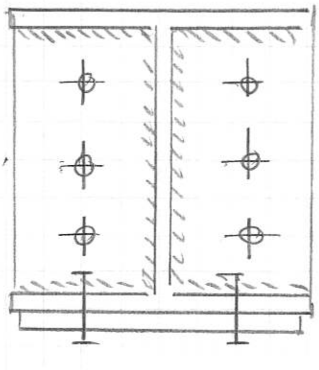
∴ ALLOW 6No M20 HSFG TO BOTTOM.

CAP PLATE 40 kN - SAY 60 kN ULS

∴ 4 M20 OK

PLATE - $400\times10^3/275\times(200-2\times22)$ = 9.32 - ALLOW 15 PLATE





SECTION A-A

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Contract: 163 SUMATRA RD			Job Ref: 20172
Sub-section:			Sheet No:
Prepared by: MS	Checked by:	Approved by:	Date: 27.11.20

TEMP SUPPORT TO REAR INSET ELEVATION + FLOOR - PHASE #2.

FLOOR LOADING :-

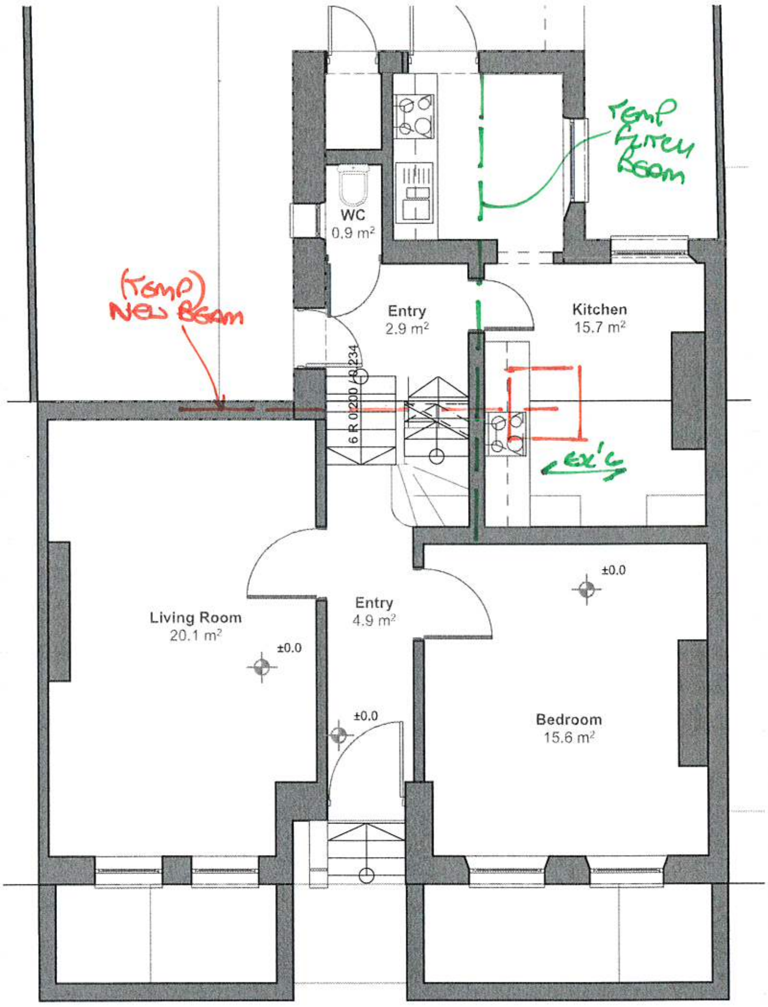
S/W OF JOISTS + BOARDING ONLY - ALLOW 40 kg/m²

UDL ON FLITCH BEAM TO U/S REAR GF :-

$0.4 \times 5.2/2 \times 3\text{No} = 3.12$ kN

$0.2 \times 2.4 \times 2 = 0.96$

S/S 4.1 kN/m (SLS)



FLITCH BEAM - LENGTH 6m (4.5 + 1.5 = CANTILEVER)

ADD TIMBER POSTS TO SUPPORT UPPER FLOORS OFF POST POSITION

∴ 1No FLOOR AS UDL

$= 1.04 + 0.48 = 1.52$ kN/m

DESIGN BY SUPERBEAM :- 2No 50×225 C24 + 10×200 PLATE

REACTION ON POST/STEEL BELOW = 16.4 kN.

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Site: 163 Sumatra Road	**Made by MS**
Job: Temp Works	**Page 1**
Job number: 20172	**File copy**

SuperBeam 4 421711h — **Temp works design 20-11-27.SBW** — **Printed 27 Nov 2020 14:40**

Beam: flitch beam to rear 1st floor — **Span: 4.500 m.** **+Cantilever: 1.500 m.**

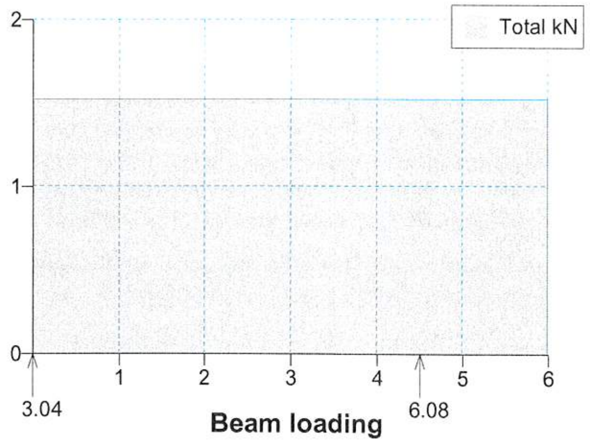
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
R T	Floor	1.52	0		6	3.04	6.08
					Total:	**3.04**	**6.08**

Load types: R:Part UDL (positions in m. from R1) T: Total

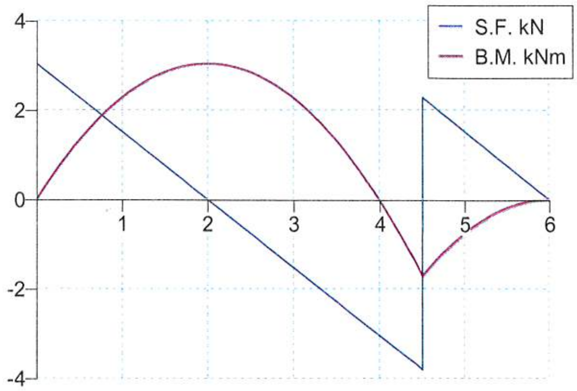
Maximum B.M. (span) = 3.04 kNm at 2.00 m. from R1
(cantilever) = -1.71 kNm

Maximum S.F. (span) = -3.80 kN at R2
(cantilever) = 2.28 kN at R2

Maximum deflection (span) = 5.953 x 1e8/EI at 2.25 m. from R1
(cantilever) = -3.85 x 1e8/EI (E in N/mm^2, I in cm^4)



Beam loading



Shear force and bending moment

Timber beam calculation to BS5268 Part 2 (1996,1997) using C24 timber

Use 2no 50 x 225 C24

z = 843.8 cm^3 I = 9,492 cm^4

Timber grade: C24 2 members acting together: K_8 = 1.1

K_3 (loading duration factor) = 1.50 K_7 (depth factor) = 1.032

E = 7200 x 1.14 = 8208 N/mm^2 (E_{min} x K_9)

Permissible bending stress, $\sigma_{m,adm}$ = 7.500 x 1.50 x 1.1 x 1.032 = 12.77 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 3.040 x 1000/843.8 = 3.60 N/mm^2 OK

Permissible shear stress, τ_{adm} = 0.71 x 1.50 x 1.1 = 1.17 N/mm^2

Applied shear stress, τ_a = 3.800 x 1000 x 3/2 x 100 x 225 = 0.25 N/mm^2 OK

Bending deflection = 5.953 x 1e8/8,208 x 9,492 = 7.64 mm

Mid-span shear deflection = 1.2 x 2.99 x 1e6/(E/16) x 100 x 225 = 0.31 mm

Total deflection = 7.64 + 0.31 = 7.95 mm (0.0018 L) OK (0.003L=13.5mm)

Cantilever deflection = -3.847 x 1e8/8,208 x 9,492 = -4.94 mm (-0.0033 L) OK

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Beam: flitch beam to rear 1st floor

Span: 4.500 m.
+Cantilever: 1.500 m.

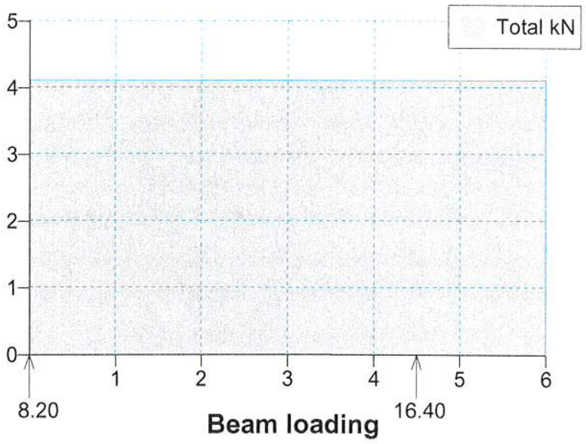
	Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*
R T	Floor	4.1	0		6	8.20	16.40
					Total:	**8.20**	**16.40**

Load types: R:Part UDL (positions in m. from R1) T: Total

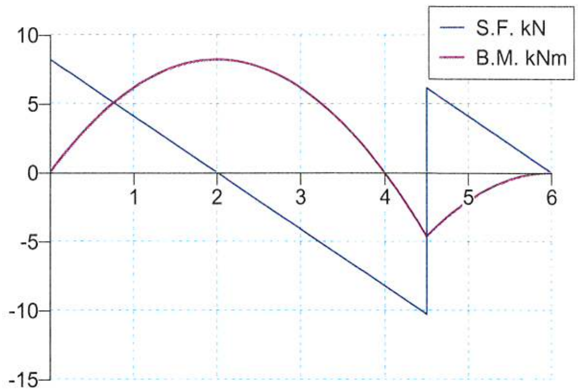
Maximum B.M. (span) = 8.20 kNm at 2.00 m. from R1
(cantilever) = -4.61 kNm

Maximum S.F. (span) = -10.25 kN at R2
(cantilever) = 6.15 kN at R2

Maximum deflection (span) = 16.06 x 1e8/EI at 2.25 m. from R1
(cantilever) = -10.4 x 1e8/EI (E in N/mm^2, I in cm^4)



Beam loading



Shear force and bending moment

Timber beam calculation to BS5268 Part 2 (1996,1997) using C24 timber

Use 2no 50 x 225 C24 + 10 x 200 flitch plate

z = 843.8 cm^3 I = 9,492 cm^4 Flitch plate z = 66.7 cm^3 I = 667 cm^4

Timber grade: C24 2 members acting together: K_8 = 1.1

K_3 (loading duration factor) = 1.50 K_7 (depth factor) = 1.032

Loading will be carried by the timber members and flitch plate in proportion to their EI values. Checks are made using the mean and minimum E-values for timber to produce worst case stresses on timber and steel members respectively.

Flitch plate EI = 205,000 x 667 = 1,367 x 1e9 Nmm^2

Check timber members:

Using E_{mean} Timber EI = 10,800 x 9,492 = 1,025 x 1e9 Nmm^2

Timber carries 1,025/(1,025+1,367) = 0.429 of total load (in worst case)

Permissible bending stress, $\sigma_{m,adm}$ = 7.500 x 1.50 x 1.1 x 1.032 = 12.77 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 0.429 x 8.200 x 1000/843.8 = 4.17 N/mm^2 OK

Permissible shear stress, τ_{adm} = 0.71 x 1.50 x 1.1 = 1.17 N/mm^2

Applied shear stress, τ_a = 0.429 x 10.250 x 1000 x 3/(2 x 100 x 225) = 0.29 N/mm^2 OK

Grade compression stress perpendicular to grain = 2.40 N/mm^2

Minimum bearing lengths: R1: 8.20 x 1000/2.40 = 34mm
(subject to adequate support under bearing) R2: 16.4 x 1000/2.40 = 68mm

Check flitch plate:

Using E_{min} Timber EI = 7,200 x 9,492 = 683 x 1e9 Nmm^2

Flitch plate carries 1,367/(683+1,367) = 0.667 of total load (in worst case)

Flitch plate f_{bc} = 0.667 x 8.20 x 1000/66.7 = 82.00 N/mm^2 OK

Deflection:

Using E_{min} x K_9 (2 members) Timber EI = 8,208 x 9,492 = 779 x 1e9 Nmm^2

Timber carries 779/(779+1,367) = 0.363 of total load (average case)

Bending deflection = 0.363 x 16.057 x 1e8/8,208 x 9,492 = 7.48 mm

Mid-span shear deflection = 0.363 x 1.2 x 8.07 x 1e6/(E/16) x 100 x 225 = 0.30 mm

Total deflection = 7.48 + 0.30 = 7.79 mm (0.0017 L) OK

Cantilever deflection = 0.363 x -10.38 x 1e8/8,208 x 9,492 = -4.84 mm (-0.0032 L) OK

Bolting:

Use M12 4.6 bolts. Bolt numbers are calculated assuming worst case load on flitch plate

Load capacity per bolt in double shear = 5.86kN (BS5268 eq. G.9)
(F_d = 1400; $M_{y,d}$ = 82,944N-mm; p_k = 350kg/m^3; K_{90} = 1.53; $f_{h,1,d}$ = 11.47)

Bearings: R1 (8.20kN): Required number of bolts = 0.667 x 8.20/5.86 = 0.93 i.e. 1 bolt min.
R2 (16.4kN): Required number of bolts = 0.667 x 16.4/5.86 = 1.87 i.e. 2 bolts min.

For load transference a minimum of 3 bolts are also required across the span

To ensure structural integrity consider providing bolts spaced at 600mm max c/s, bolt centres alternately min. 50mm from top and bottom of beam

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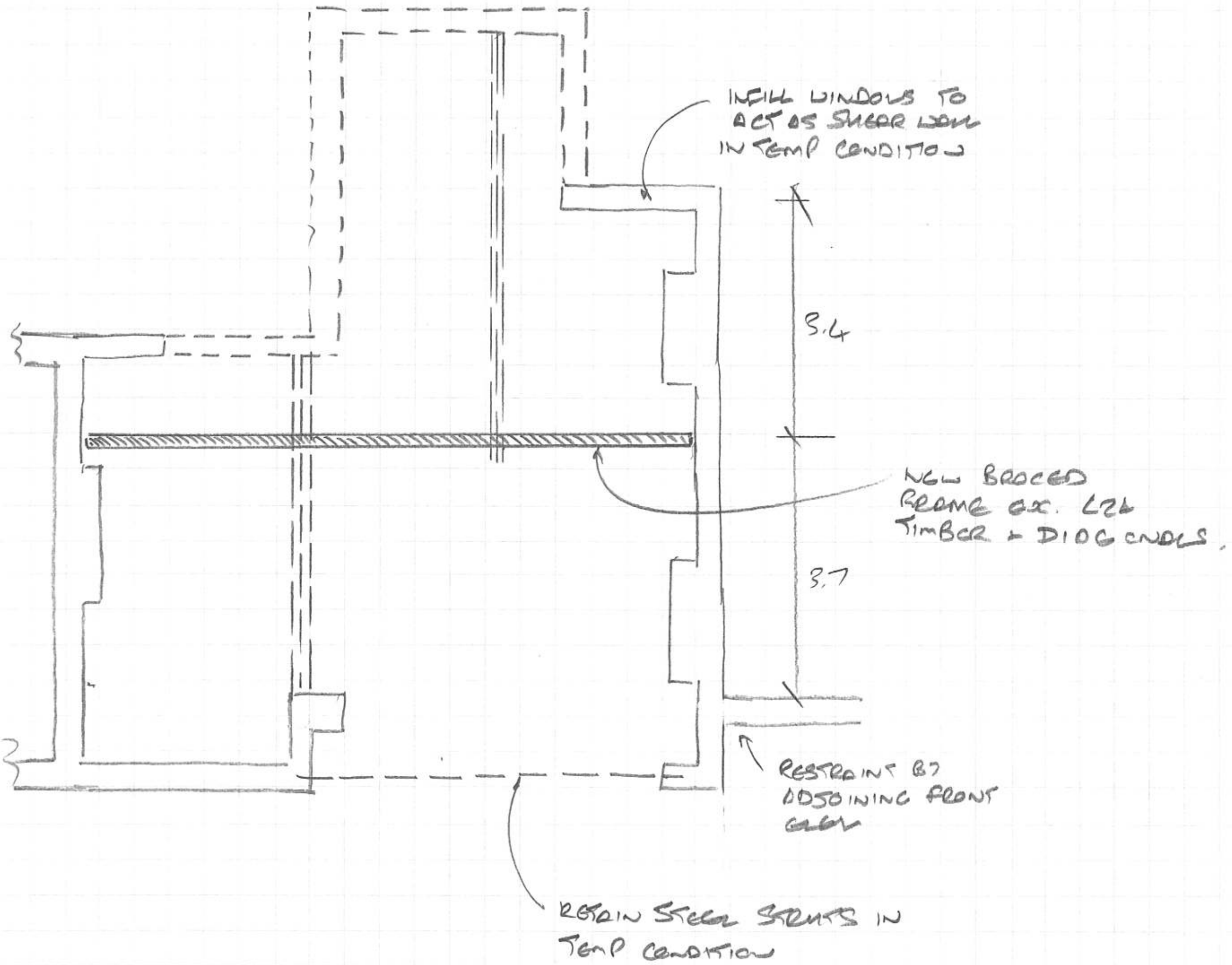
Contract: SUMATRA ROAD	Job Ref: 20172
Sub-section:	Sheet No:
Prepared by: MS Checked by: Approved by:	Date: 29.7.21

TEMP SUPPORT FRAME TO RESTRAIN PARTY WALLS



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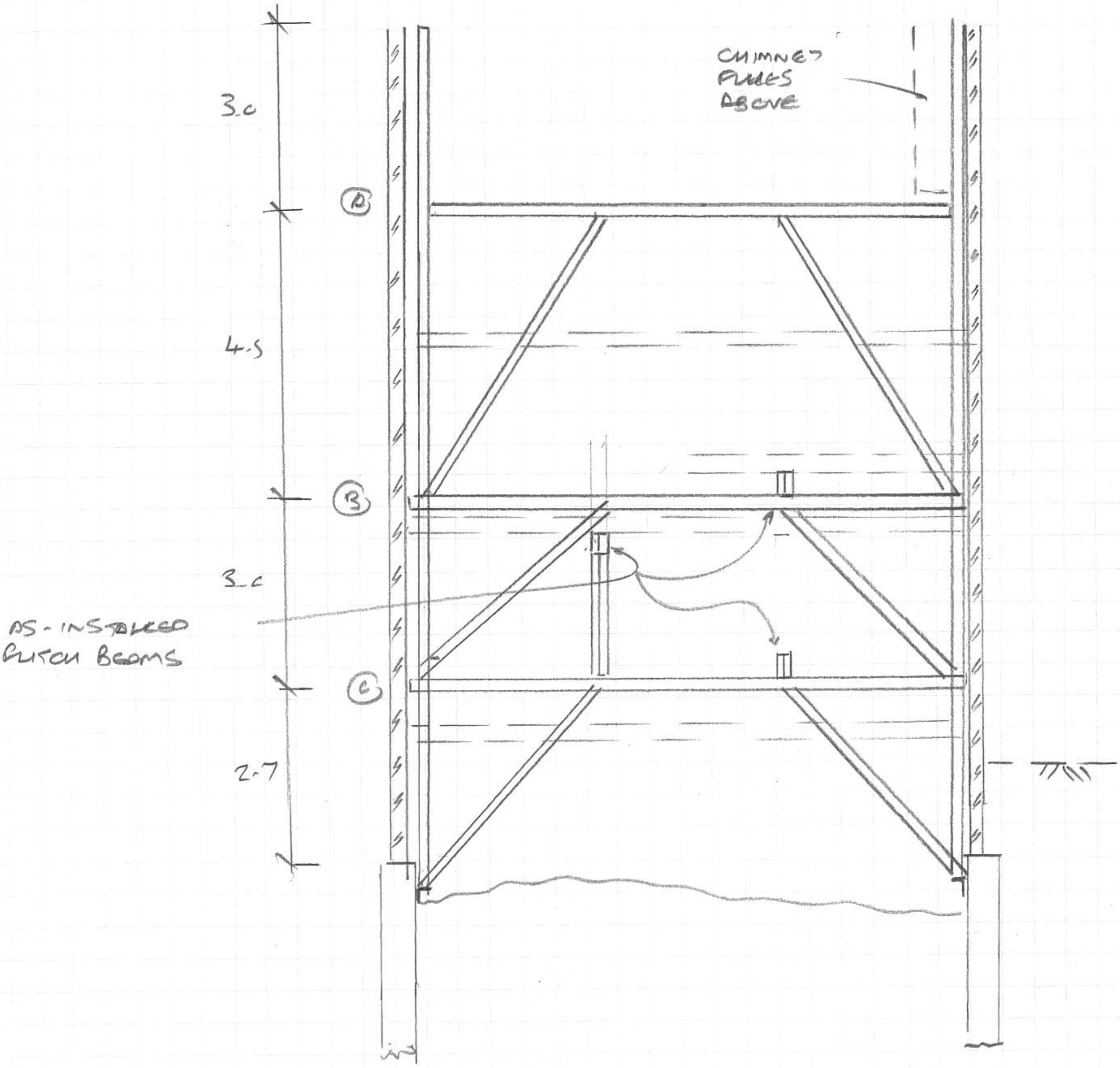
Contract: 163 Sumatra Road	Job Ref: 20172
Sub-section:	Sheet No:
Prepared by: MS Checked by: Approved by:	Date: 29.7.21



WEIGHT OF PARTY WALL - ALLOW 327 BRICKWORK -

$\therefore$ $20 \times 0.327 + 0.2$ (FINISHES) $= 6.74$ kN/m²

CONSIDER RETURNED ELEVATIONS AS ADDITIONAL RESTRAINT

$\therefore$ EFF. WALL WIDTH RESTRAINED $= \dfrac{3.4 + 3.7}{2} = 3.55$m

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	Sub-section:		Sheet No:
	Prepared by: MB	Checked by:	Approved by:	Date: 29.7.21

DIAGONAL STRUTS :-

ASSUME FLOOR LOAD = $\begin{bmatrix} 0.5 \\ 0.5 \end{bmatrix}$ kN/m²

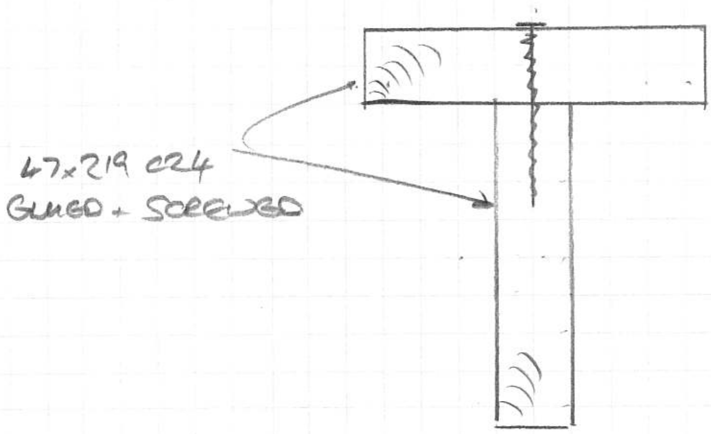
UDL ON L H BEAM = $\begin{bmatrix} 0.5 \\ 0.5 \end{bmatrix} \times \dfrac{3.5^2}{2 \times 3.0} = 2.04$ kN/m

$P_L = 2.04 \times \dfrac{4.8^2}{2 \times 3.5} = 6.7$ kN. AT SUPPORT PT.

CONSIDER AS WORST CASE

DIAGONAL LENGTH = ASSUME $3.0\sqrt{2} = 4.24$ m

FOR COMPOUND TEE-SECTION THUS :-



SECTION PROPERTIES BY TEDDS :-

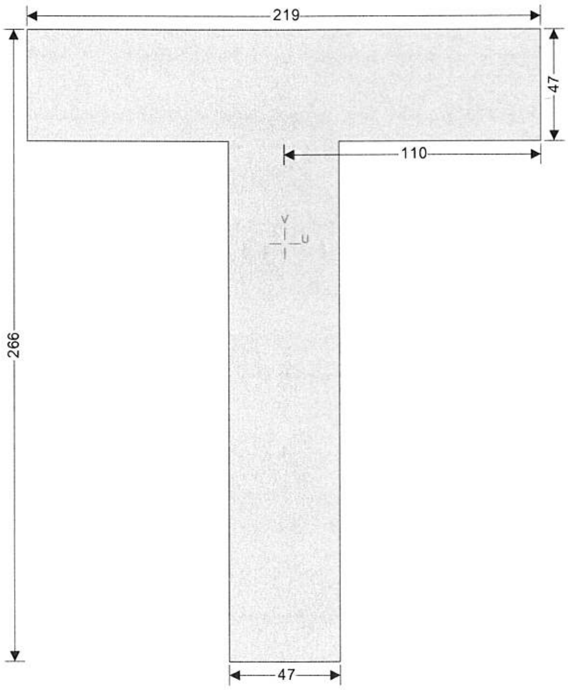
$r_{yy} = 46$ mm $\therefore L_e/r_y = \dfrac{4240}{46} = 92 \quad \therefore K_{12} = 0.44$

$P_c = 7.9 \times 0.44 \times 2 \times 47 \times 219 / 10^3 = 71.5$ kN - BY INSPECTION OK

Harold James (London) Ltd Atlantic House, Gomm Road High Wycombe HP13 7DJ	Project 163 Sumatra Road				Job no. 20172	
	Calcs for Temp Diagonal Strut				Start page no./Revision 1	
	Calcs by MS	Calcs date 29/07/2021	Checked by	Checked date	Approved by	Approved date

CALCULATION OF SECTION PROPERTIES

Tedds calculation version 2.0.07



Area

A = **205.86** cm^2

2nd moment of area

I_{uu} = **13.4×10^3** cm^4 I_{vv} = **4.30×10^3** cm^4 I_{xx} = **13.4×10^3** cm^4 I_{yy} = **4.30×10^3** cm^4

Radius of gyration

r_{uu} = **80.7** mm r_{vv} = **45.7** mm r_{xx} = **8.1** cm r_{yy} = **4.6** cm

Plastic section modulus (only shapes with all rectangles at 90 degs)

S_{xx} = **1.37×10^3** cm^3 S_{yy} = **684.** cm^3

Distance to combined centroid

X_e = **0.0** mm Y_e = **0.0** mm

Distance to equal axis area (only shapes with all rectangles at 90 degs)

X_p = **-0.2** mm Y_p = **43.0** mm

Elastic section modulus

Z_{xx} = **762.** cm^3 Z_{yy} = **392.** cm^3

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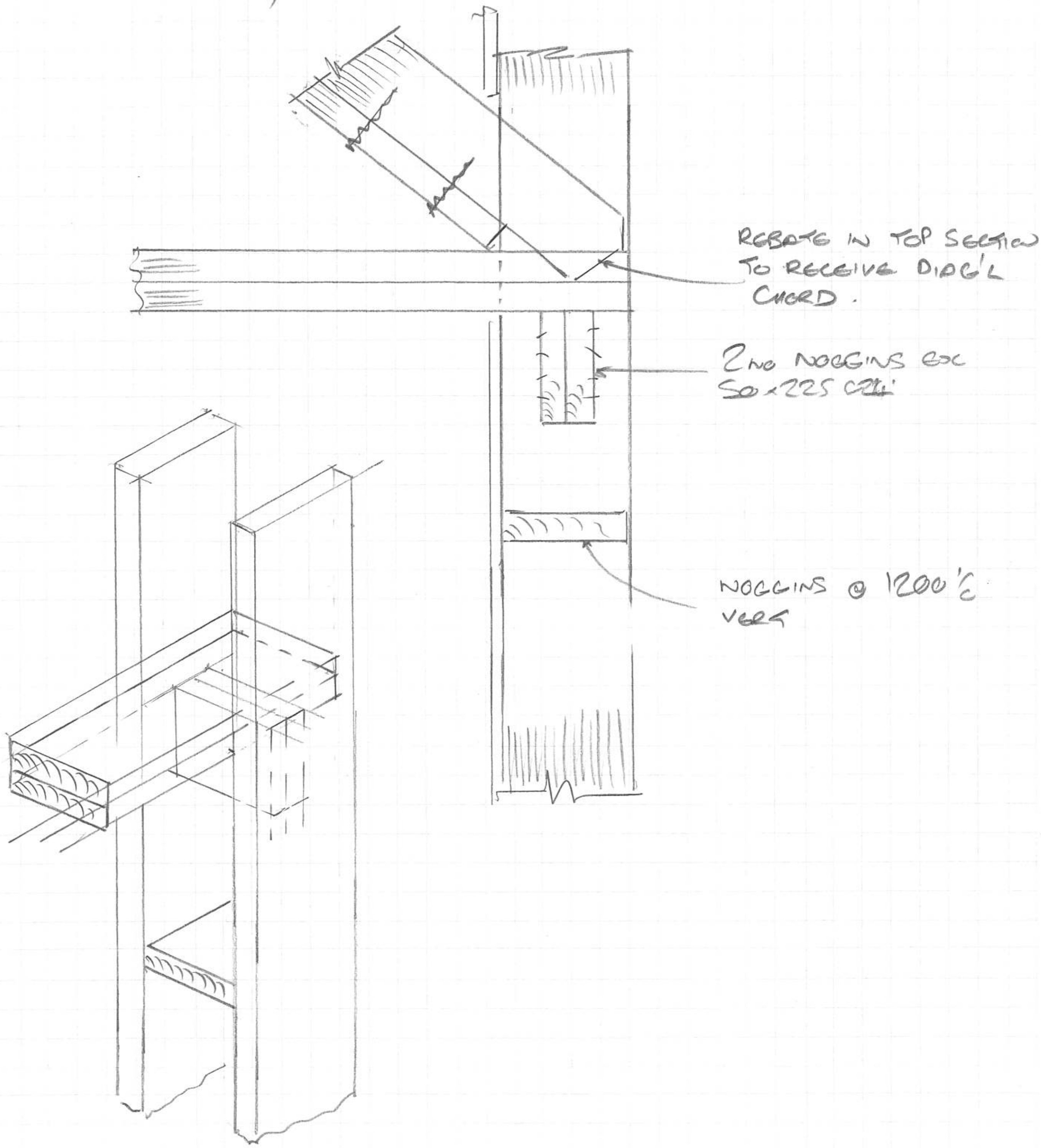
Prepared by: MB Checked by: Approved by:	Date: 29.7.21

SUPPORT TO HORIZ/DIAGONALS AT NODE PTS -



Appendix C – Design for subsequent basement

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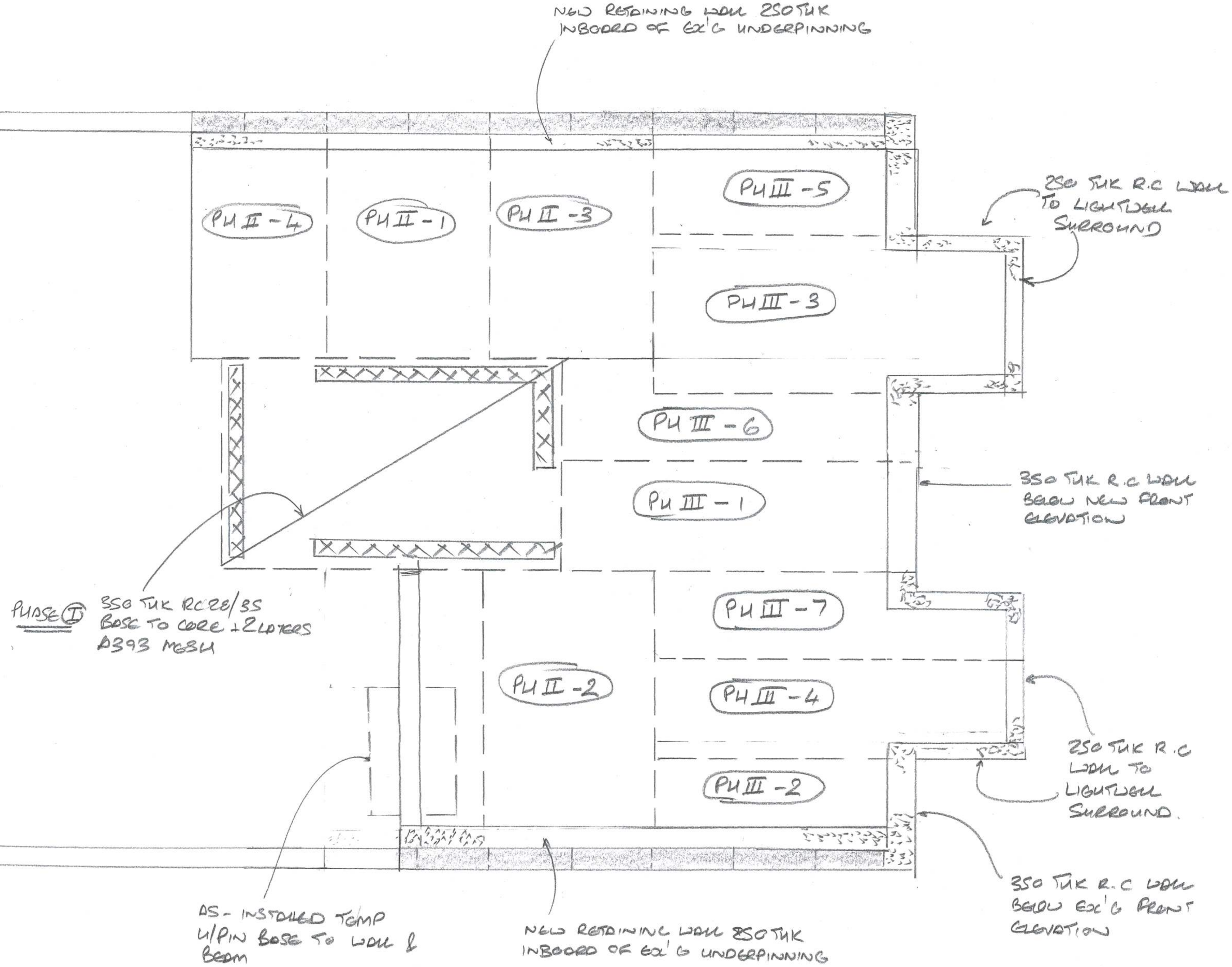

163 SUMATRA RD
PHASES I - III UNDERPIN LAYOUT
20172/SK20
16.3.21

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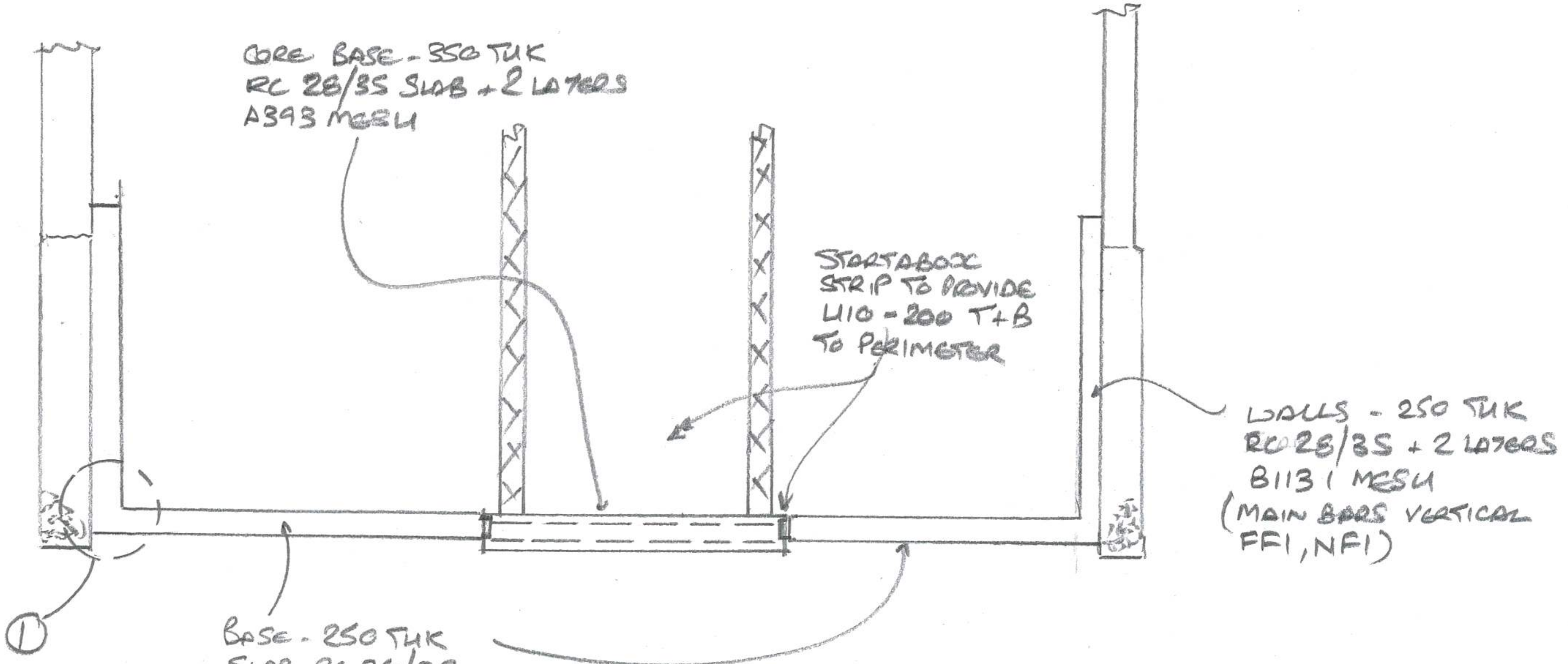
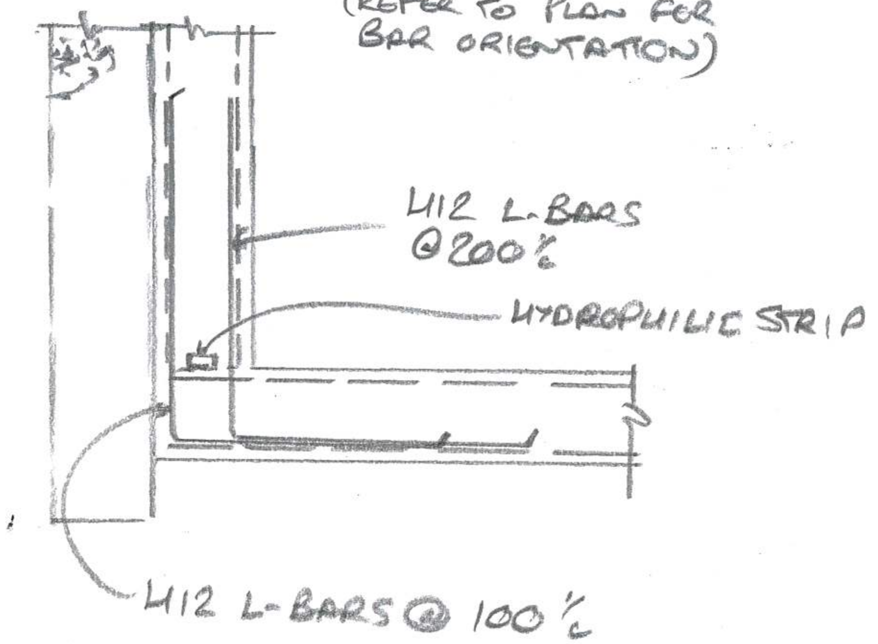


SECTION A-A



DETAIL 1

163 SUMATRA RD
SECTIONS THRO' BASEMENT
20172/SK30
16.3.21

NOTES : CONCRETE GRADE TO BE DESIGNATED MIX RC28/35

: 40. COVER TO ALL REINFORCEMENT

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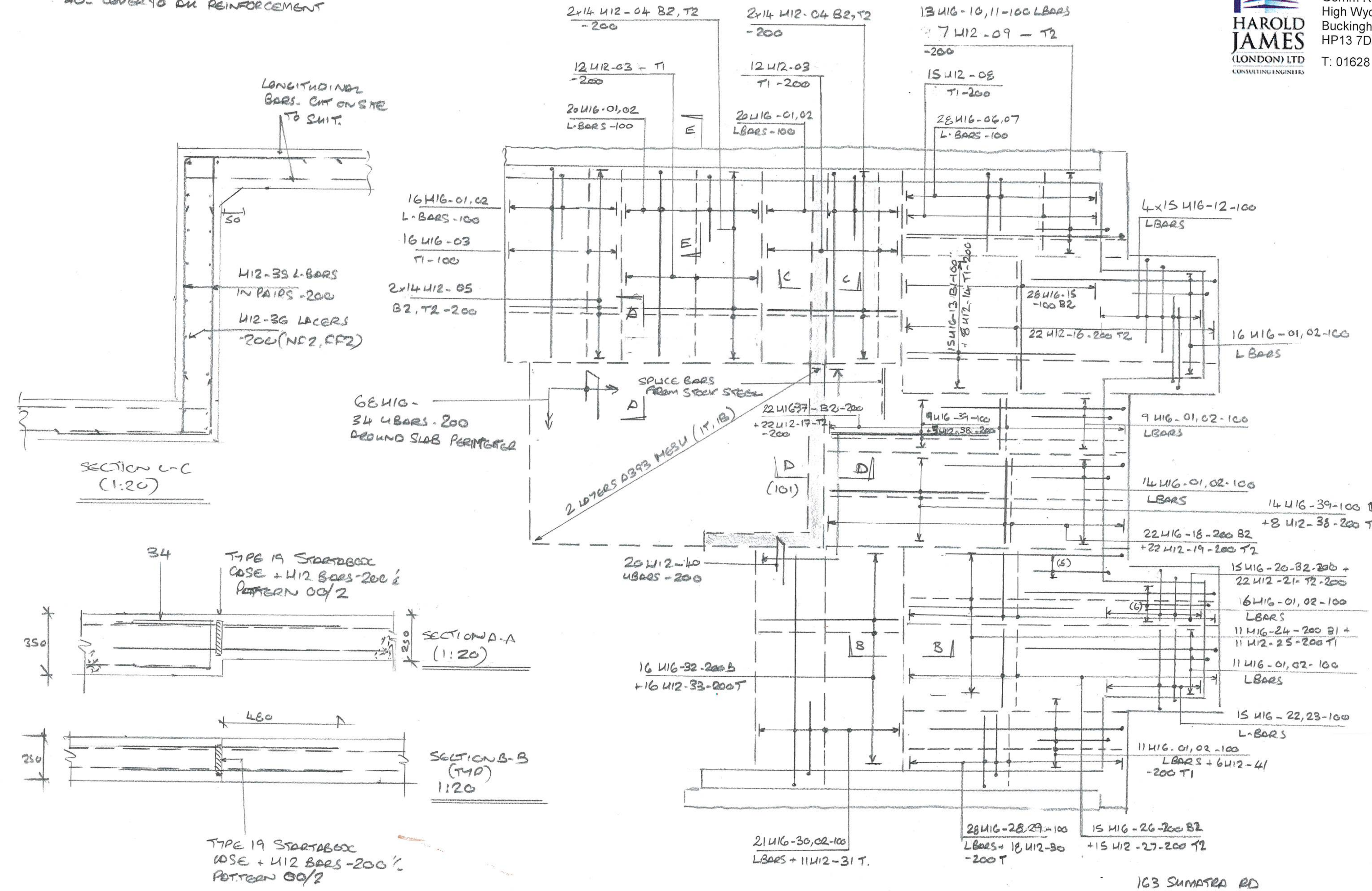

163 SUMATRA RD
REINFORCEMENT GA TO BASEMENT (FRONT SLAB)
20172/SK/RC/100
16.4.21

Harold James (London) Ltd Atlantic House, Gomm Road High Wycombe HP13 7DJ	Project 163 Sumatra Road				Job no. 20172	
	Calcs for SK/RC/100 - For Pricing only				Start page no./Revision 1	
	Calcs by MS	Calcs date 16/04/2021	Checked by	Checked date	Approved by	Approved date

Tedds calculation version 1.0.06

Reinforcement schedule

Member	Bar mark	Type and size	No. of mbrs	No. of bars in each	Total no.	Length of each bar + mm	Shape code	*A** mm	*B** mm	*C** mm	*D** mm	*E/R** mm	Rev letter
Basement Slab	**01**	**H16**	**1**	**133**	**133**	**4375**	**11**	**2600**	**1800**				
	02	**H16**	**1**	**154**	**154**	**2775**	**11**	**1000**	**1800**				
	03	**H12**	**1**	**40**	**40**	**2600**	**00**	**2600**					
	04	**H12**	**1**	**56**	**56**	**1800**	**00**	**1800**					
	05	**H12**	**1**	**28**	**28**	**1500**	**00**	**1500**					
	06	**H16**	**1**	**28**	**28**	**2975**	**11**	**1200**	**1800**				
	07	**H16**	**1**	**1**	**1**	**2675**	**11**	**900**	**1800**				
	08	**H12**	**1**	**15**	**15**	**1100**	**00**	**1100**					
	09	**H12**	**1**	**7**	**7**	**3100**	**00**	**3100**					
	10	**H16**	**1**	**13**	**13**	**4775**	**11**	**3000**	**1800**				
	11	**H16**	**1**	**13**	**13**	**2775**	**11**	**1000**	**1800**				
	12	**H16**	**1**	**60**	**60**	**2975**	**11**	**1200**	**1800**				
	13	**H16**	**1**	**15**	**15**	**2500**	**00**	**2500**					
	14	**H12**	**1**	**8**	**8**	**4300**	**00**	**4300**					
	15	**H16**	**1**	**28**	**28**	**1900**	**00**	**1900**					
	16	**H12**	**1**	**22**	**22**	**1900**	**00**	**1900**					
	17	**H12**	**1**	**22**	**22**	**900**	**00**	**900**					
	18	**H16**	**1**	**22**	**22**	**900**	**00**	**900**					
	19	**H12**	**1**	**22**	**22**	**900**	**00**	**900**					
	20	**H16**	**1**	**15**	**15**	**1000**	**00**	**1000**					

This schedule conforms to BS 8666:2005

* Specified in multiples of 5 mm. + Specified in multiples of 25 mm.

Status: P Preliminary T Tender C Construction;

Tedds calculation version 1.0.06

Reinforcement schedule

Member	Bar mark	Type and size	No. of mbrs	No. of bars in each	Total no.	Length of each bar + mm	Shape code	A* mm	B* mm	C* mm	D* mm	E/R* mm	Rev letter
	21	**H12**	**1**	**22**	**22**	**1000**	**00**	**1000**					
	22	**H16**	**1**	**15**	**15**	**2775**	**11**	**1000**	**1800**				
	23	**H16**	**1**	**15**	**15**	**2575**	**11**	**800**	**1800**				
	24	**H16**	**1**	**11**	**11**	**2500**	**00**	**2500**					
	25	**H12**	**1**	**11**	**11**	**4200**	**00**	**4200**					
	26	**H16**	**1**	**15**	**15**	**1100**	**00**	**1100**					
	27	**H12**	**1**	**15**	**15**	**1100**	**00**	**1100**					
	28	**H16**	**1**	**28**	**28**	**2775**	**11**	**1000**	**1800**				
	29	**H16**	**1**	**28**	**28**	**2575**	**11**	**800**	**1800**				
	30	**H16**	**1**	**21**	**21**	**5075**	**11**	**3300**	**1800**				
	31	**H12**	**1**	**11**	**11**	**3300**	**00**	**3300**					
	32	**H16**	**1**	**16**	**16**	**1900**	**00**	**1900**					
	33	**H12**	**1**	**16**	**16**	**1900**	**00**	**1900**					
	34	**H12**	**1**	**68**	**68**	**1225**	**21**	**500**	**250**	**500**			
	35	**H12**	**1**	**56**	**56**	**1700**	**11**	**1200**	**500**				
	36	**H12**	**1**	**20**	**20**	**2500**	**00**	**2500**					
	37	**H16**	**1**	**22**	**22**	**900**	**00**	**900**					
	38	**H12**	**1**	**13**	**13**	**4100**	**00**	**4100**					
	39	**H16**	**1**	**23**	**23**	**2500**	**00**	**2500**					
	40	**H12**	**1**	**20**	**20**	**1125**	**21**	**500**	**160**	**500**			

This schedule conforms to BS 8666:2005

* Specified in multiples of 5 mm. + Specified in multiples of 25 mm.

Status: P Preliminary T Tender C Construction;

Tedds calculation version 1.0.06

Calculate weight of steel reinforcement

Steel bar weights in tonnes by type and size											
Type	6 mm	8 mm	10 mm	12 mm	16 mm	20 mm	25 mm	32 mm	40 mm	50 mm	Total;
H	**0.000**	**0.000**	**0.000**	**0.760**	**3.134**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**3.894**
Total	**0.000**	**0.000**	**0.000**	**0.760**	**3.134**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**3.894**
Total weight of steel is 3.894 tonnes											

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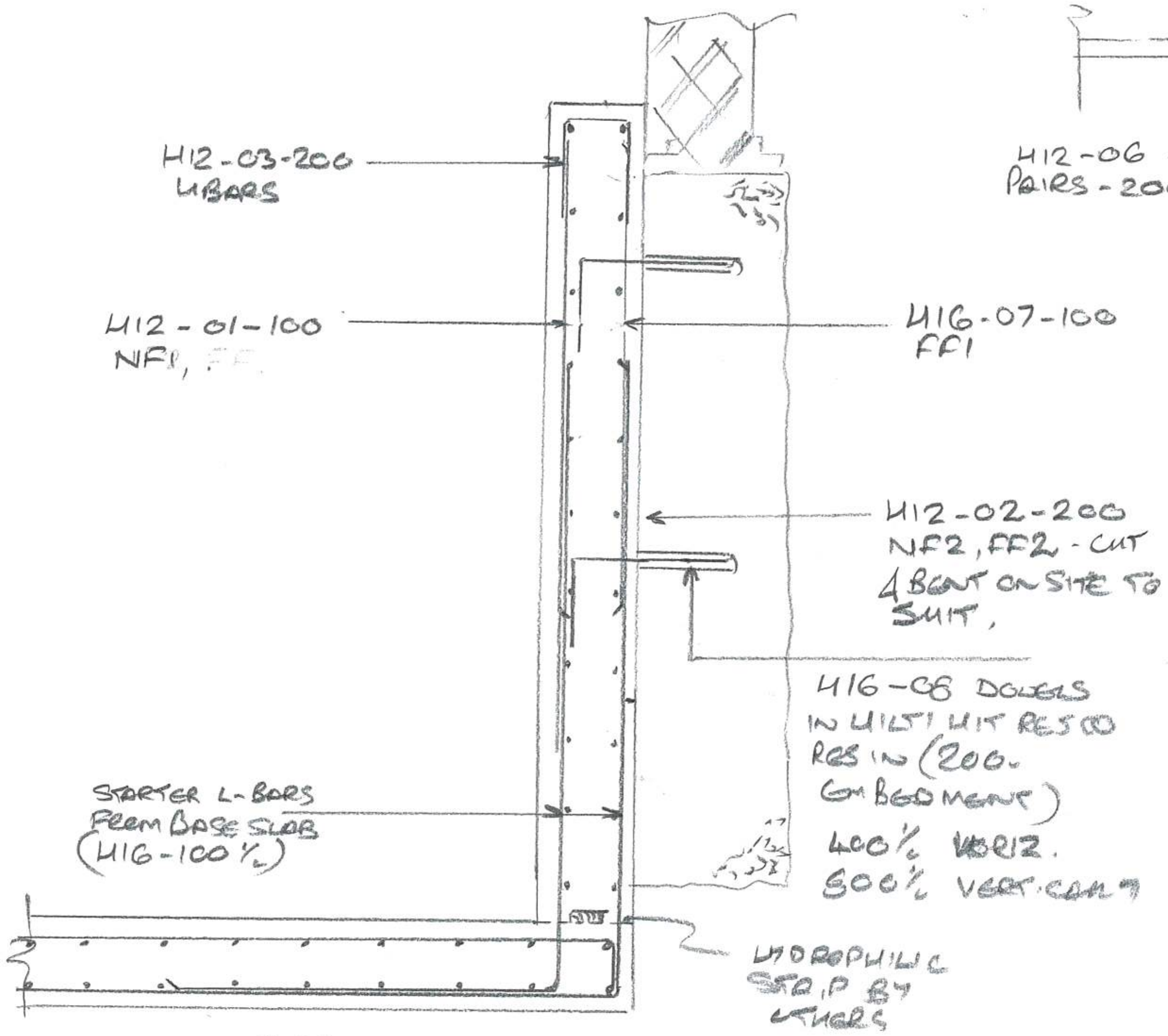
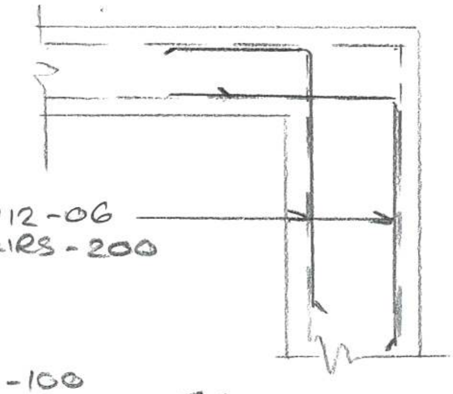
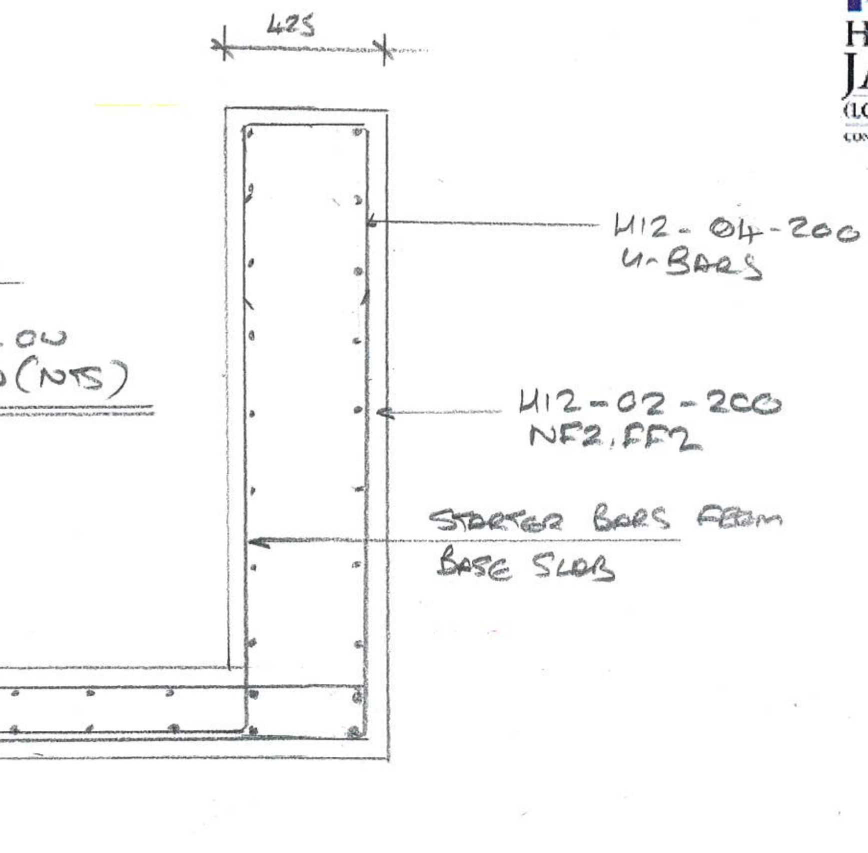


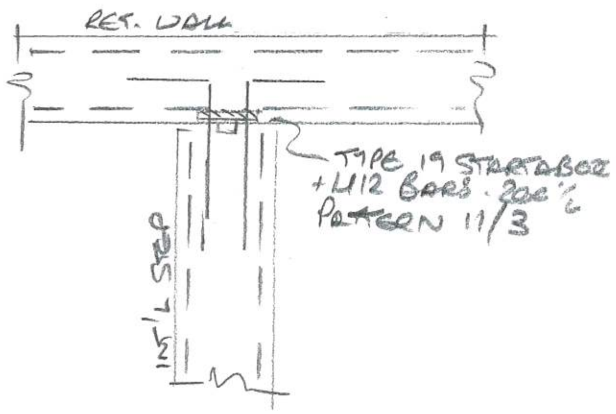
SECTION E-E
(1:20)



TYPICAL DETAIL ON
WALL RETURN (NTS)

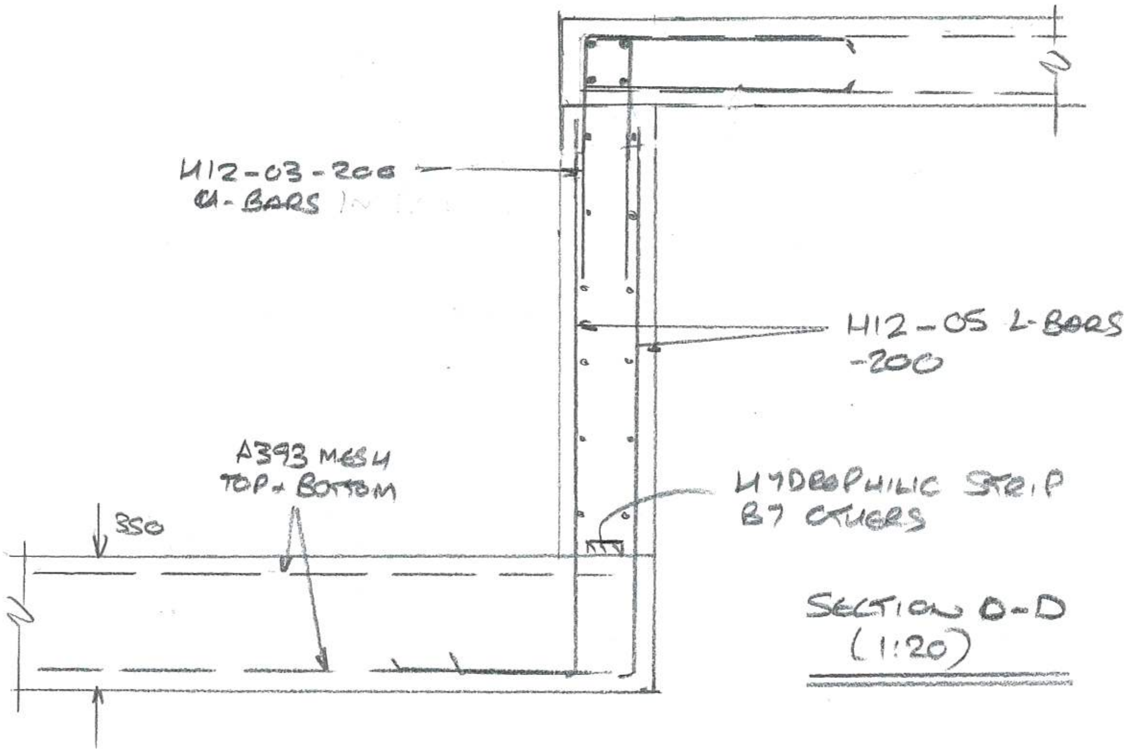


SECTION F-F
(1:20)



JUNCTION OF RET. WALL
& INTERNAL LEVEL CHANGE
(NTS)

WALL RETURN AT
PIN JUNCTION
(TYP)



SECTION D-D
(1:20)

163 SUMATRA RD
REINFORCEMENT SECTIONS
(FRONT SLAB)
20172/SK/RC/101
16.4.21

Tedds calculation version 1.0.06

Reinforcement schedule

Member	Bar mark	Type and size	No. of mbrs	No. of bars in each	Total no.	Length of each bar + mm	Shape code	*A** mm	*B** mm	*C** mm	*D** mm	*E/R** mm	Rev letter
Basement Walls	**01**	**H12**	**1**	**45**	**45**	**2250**	**00**	**2250**					
	02	**H12**	**1**	**175**	**175**	**4500**	**00**	**4500**					
	03	**H12**	**1**	**30**	**30**	**1125**	**21**	**500**	**160**	**500**			
	04	**H12**	**1**	**30**	**30**	**1300**	**21**	**500**	**335**	**500**			
	05	**H12**	**1**	**26**	**26**	**1950**	**11**	**500**	**1450**				
	06	**H12**	**1**	**200**	**200**	**1000**	**11**	**500**	**500**				
	07	**H16**	**1**	**45**	**45**	**2250**	**00**	**2250**					
	08	**H12**	**1**	**120**	**120**	**600**	**11**	**370**	**250**				

This schedule conforms to BS 8666:2005

* Specified in multiples of 5 mm. + Specified in multiples of 25 mm.

Status: P Preliminary T Tender C Construction;

Tedds calculation version 1.0.06

Calculate weight of steel reinforcement

Steel bar weights in tonnes by type and size											
Type	6 mm	8 mm	10 mm	12 mm	16 mm	20 mm	25 mm	32 mm	40 mm	50 mm	Total;
H	**0.000**	**0.000**	**0.000**	**1.140**	**0.160**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**1.300**
Total	**0.000**	**0.000**	**0.000**	**1.140**	**0.160**	**0.000**	**0.000**	**0.000**	**0.000**	**0.000**	**1.300**
Total weight of steel is 1.300 tonnes											