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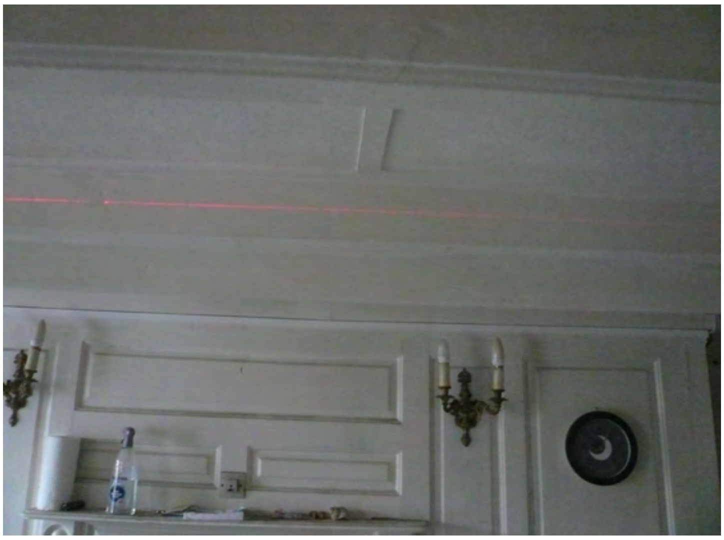
NYMNPA
06/11/2017

STRUCTURAL REPORT.
SHIRLEY HOUSE, SUNNY PLACE, ROBIN HOODS BAY

1.0. REQUIREMENT.
There is a requirement to submit Structural report with this Planning Application because the building concerned is a Listed Building.
The report is also required because there are elements of structure, namely timber floor joists, which require assessment due to obvious deflection which can be easily seen with the naked eye.

2.0. ASSESSMENT.
Photographs 1, 2 and 3 show floor joists at Ground Floor, First Floor and Second Floor levels respectively.

Photograph 1. Shows deflection of GFL joists approx 30mm.



Photograph 2. Shows deflection of FFL joists of approximately 70mm.

Photograph 3. Second Floor joists. Deflection approximately 40mm.

2.1. In the case of GFL joists the deflection represents 1/200 span which is not acceptable in BS 5268:2 2002 Structural use of Timber and in any case, spans over 4.67 metres, the deflection is not to exceed 14mm.

2.2. For the FFL joists deflection amounts to 1/85 of the span which needs no further comment.

2.3. SFL joists are similar in stature to the GFL joists and consequently inadequate.

2.4. A calculation performed in accordance with BS. 5268 is attached in Appendix A to demonstrate the structural capacity of the FFL joists, which are worst case.

2.5. Conclusion of Assessment

Deflection is well beyond what is considered to be satisfactory and this is in the unloaded state. On the introduction of domestic paraphernalia the deflection will increase and without the introduction of additional support, the possible consequences are:

a). The bending stress limit of the timber is surpassed and fail, the fibres tearing apart and cracking.

b). The deflection increases and decreases with movement of loading and due to the excessive amount disturbs construction at the bearing. The walling and mortar joints are disturbed and allows the ingress of water from outside. The structure deteriorates.

c). The deflection becomes so great that the timber falls from its bearing.

3.0 Consideration of Remedial Work.

3.1 Historical Considerations.

It is presumed that the joists are the product of the Georgian era revamp of the building as the lower arrises are moulded. A c17th century fisherman's cottage construction would have been more prosaic than this, quite possibly with unwrot timbers being used, these consequently being replaced. In the absence of any other historic material and in the overall scheme for improvement of the building, it is considered that this Georgian feature is that worthy of retention in this Listed Building.

3.2. Hollington; The Flitch Plate Solution.

Elsewhere in Robin Hoods Bay the addition of a flitch plate and additional joist solution has been presented to address similar problems with over stressed timber joists.
The initial observation is that the span of the joists has had to be broken and at Hollington, this has been achieved with a SHS section fitted beneath the joists.
At Shirley House it is proposed that the ground floor joist span is broken by the introduction of a structural partition at Basement level. A single span beam fitted to the underside of first floor and second floor joists to achieve the same result.

In respect of the flitch plate

- The deflection of first floor joists at Shirley house means that a flitch plate cannot be fixed along the neutral axis of the joist section due to its centenary curve and the disparity in geometry of the steel flitch plate with this.
- The second consideration; the historical aspect. The existing arrangement at Shirley House is that the ceilings are open and the finish is fixed to the underside of the floorboards over. This allows the floor joists to be seen in section and with the moulded arris on display. A secondary joist and metal plate would bulk up this image to become a quite different historical image in the asset.
- Whereas at Hollington the floor joists are to have ceilings below, the headroom to underside of joists throughout the floor levels at Shirley House is limited to; Basement 1.87m, Ground Floor 2.01m and First Floor 1.98m. Ideally it is better for the sake of habitation that the headroom is left uncompromised.

3.3 Joist Intermediate Support.

3.3.1Basement.
The headroom below the floor joists above is 1.87 metres and it is impractical to consider that this could be any less. As a consequence the notion of fitting any kind of structural support beneath the joists is also impractical. It is therefore proposed that a structural timber partition be constructed to break the existing joist span. The actal support required for the joists can generally be contained within the partition structure and allowing the existing floor joists to remain intact. Above the doorways to the staircase Lobby and the Pantry, it will be necessary to notch the existing joists to the door head. Although the removal of material is required it is considered that this is no less than would be involved in attaching flitch plates and equally so reversible by the installation of scarf jointed inserts.
(Flitch plates when removed would leave 12mm diameter holes at 600mm centrelines on every joist.)

3.3.2 Ground Floor.
This is the most difficult of the three floors to address. The headroom to underside of the joists is 2.01 metres. The head of the door between Sitting Room and rear Hall is 1.86 metres leaving a space of only 150mm between the two levels.
At Hollington, a SHS steel section (120 x 120) has been proposed which would ideally suit the situation at Shirley House. However, by comparison the reactions at the supports are 150% higher and so the section fails where the bending moment capacity should be less than 1.2pyZ, which it I not. See Appendix B.
A 140 x 140 SHS is no better, the density/m still compromising the formula.

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The conclusion was to examine the structural properties of a steel column as the sections are, like the SHS, also compact in depth. A calculation carried out to BS. 5950 and to be found in Appendix C, shows that a (serial size) 150 x 150 steel column has the capacity to provide the necessary support for the first floor joists.

The complication here is that of the headroom to the door opening between the Sitting Room and rear Hall as previously mentioned. Drawing 190.18 First & Second Floor Beam details shows the constructional constraints and the proposed installation.

Essentially, the constraints amount to

a). The headroom above the Sitting Room/stair lobby doorway is limited to 140mm from door head to underside of joists. The proposed support beam cannot be fitted within this space and the door head rail needs to be cleared.

b). It would also be preferable if the architrave to the Sitting Room side of the door could be retained and also the Sitting Room side of the adjacent (to the door opening) lath & plaster partition.

The detail drawing demonstrates how this can be effected with the minimum of disturbance to the heritage asset.

It would be intended that the beam is to be installed as one unit in order to avoid bulky connections along the length. This would facilitate the fire protection of the steelwork plus the enclosure of all this with timber panelling with a beaded arris. This would be complimentary to the period of the present floor construction and to the asset overall.

3.3.3 Second Floor installation.

The position of the proposed support beam is controlled by the presence of historic fabric. In this case this is the panelled partition wall which aligns the existing main Bedroom. The detail drawing presents the case for the installation of the support beam towards the rear house side of the panel. This would appear in the First Floor Landing and the adjacent Bathroom and would disrupt the minimum of historic fabric; a section of panel wall to the Landing/Bathroom Lobby.

As for the proposed installation beneath first floor joists, the intention would be that the beam is to be installed as one unit in order to avoid bulky connections along the length. This would facilitate the fire protection of the steelwork plus the enclosure of all this with timber panelling with a beaded arris.

<u>4.0. Structure Adjacent Chimney Breast. Basement Floor Level.</u>

The floor joist spanning from the front wall to the chimney breast is structurally compromised.

- The timber section has been reduced to less than half section area; was 150 x 100m, now 70 x 100mm in order to accommodate the fireplace hearth stone.
- A large knot has fallen from the section immediately adjacent to the bearing in the reduced section zone.

The combined effect of these two problems is that the joist is structurally incompetent and the options for remedial work are:

1. Leave the arrangement as it is. It has not fallen down.
2. Removal of the joist and insertion of new.
3. Additional support beneath the joist.

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Discussion of options for remedial work.

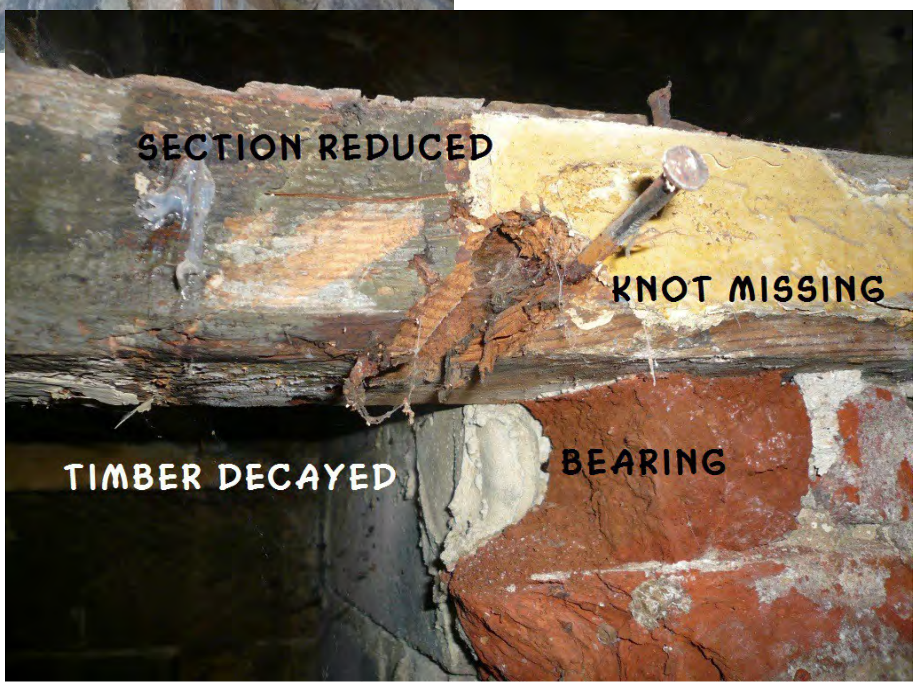
1. If the structure is left to remain the joist will fail as the section is not adequate. If the structure fails, historic fabric and construction will be lost. The most onerous outcome would be injury to persons or fatality. Neither of these two considerations is acceptable.

2. If the joist is replaced then the section still would require to be reduced to accommodate the hearth detail. As the reduction is more than allowed for notching by BS. 5268 (ie h/2) this option is impractical. Additionally, the removal of the joist would require disturbance of historic fabric, which if option 3 is adopted, is bypassed.

3. The addition of a timber post which supports the joist at the full depth of section appears to be a logical conclusion to remedy the structural defecit.

The calculation for this is attached and results in a 100 x 100mm square timber post. This should be of kiln dried Douglas Fir or Pitch Pine which is sympathetic to the timbers used elsewhere in the asset.

Structural Condition addressed in Section 4.



END.

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SHIRLEY HOUSE. ROBIN HOODS BAY.

GROUND FLOOR STRUCTURE.

MODIFICATION FACTORS
$K3 = 1$
$K7 = (300/h)^{0.11} = 1.075$
$K8 = 1.1$

SMALLEST JOIST SECTION 75 x 155 MM.

SECTION PROPERTIES: $Z = \frac{bd^2}{6} = 300312.5\,mm^3$

$I_{xx} \frac{bh^3}{12} = 23.27\,mm^4 \times 10^6$

SPAN 5900 MM. SUPPORT WIDTH AV. 577MM SUPPORT AREA 3.40M²

SUPERIMPOSED LOAD 1.50 KN/M²
FLOOR BOARDS 0.035 THK. 0.206 KN/M²
JOISTS: 0.0116 M³/m² 0.118 KN/M²
INSULATION BOARD 0.032 KN/M²
1.856 KN/M² $W = 1.856 \times 3.40 = 6.31$ KN

MAXIMUM BENDING MOMENT $\frac{WL}{8}$ $\frac{1.856 \times 5.90 \times 0.577 \times 5.90}{8} = 4.66$ KNM

$M = fz \therefore f = \frac{M}{Z}$ $f = \frac{4.66 \times 10^6}{300312.5}$ $f = 15.52\ N/mm^2$ (SEEMS EXCESSIVE!)

IT FOLLOWS THAT THE TIMBER JOISTS (GFL) ARE LIKELY TO BE OF DOUGLAS FIR C 13.1 N/mm² OR PITCH PINE 13.1 N/mm² ∴ E = 11,000 N/mm² (D.FIR)

DEFLECTION $\delta = \frac{5}{384} \frac{WL^3}{EI}$, $\delta = \frac{5 \times 6.30 \times 10^3 \times 5900^3}{384 \times 13500 \times 23.27 \times 10^6} = 53.63$ MM

1/110 SPAN ∴ SECTION FAILS IN DEFLECTION AS MAX ALLOWABLE = 1/200 IE. 30MM.

IMPROVEMENT

LOAD BEARING PARTITION TO PANTRY / STAIRWELL.

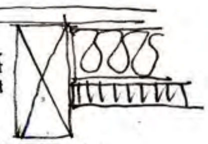
SPAN REDUCED TO 4240 MM. $\therefore W = 4.24 \times 0.577 \times 1.856 = 4.54$ KN

$\delta = \frac{5}{384} \times \frac{4.54 \times 10^3 \times 4240^3}{13500 \times 23.27 \times 10^6} = 14.34$ MM 1/296 APPROACHING ACCEPTABLE

INVESTIGATE ADDITION OF NEW FIRE RESISTANT BOARDS.

LOADING

IMPOSED:		1.50 KN/M²
JOISTS + BOARDS		0.33 KN/M²
PLASTERBOARD		0.12 KN/M²
INSULATION	45kg/M³	0.03 KN/M²
		1.98 KN/M² × 2.45m² = 4.85 KN.



BENDING MOMENT $= \dfrac{4.85 \times 4.24}{8} = 2.57$ KNM

$f = \dfrac{6M}{bd^2}$ $\quad f = \dfrac{6 \times 2.57 \times 10^6}{75 \times 155^2}$ $\quad 8.56\,N/MM^2 \;\therefore$ OK

CHECK DEFLECTION

$\delta = \dfrac{5}{384} \times \dfrac{4.85 \times 10^3 \times 4240^3}{13500 \times 23.27 \times 10^6} = 15.32\,mm = 1/385$ SPAN $\therefore$ OK.

PARTITION @ GLF NOT CONSIDERED AS SUPPORTED BY EXTG STAIRCASE STRUCTURE REMOVED & PANTRY WALL TO BE REMOVED, STRUCTURAL PARTITION TO REPLACE BOTH OF THESE.

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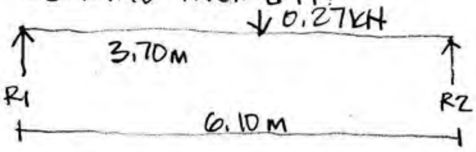
SHIRLEY HOUSE, ROBIN HOODS BAY

INSPECT FIRST FLOOR JOISTS / SECTION AS GROUND FLOOR

ADD BENDING MOMENT FROM PARTITION WALL
1.98M (H) × 4.6M (L) × 0.04 (T) = 0.365m³ @ 600kg/m³ = 2.15 KN TOTAL 0.47KN/M
POINT LOAD ON JOIST 0.27KN
BENDING MOMENT.



6.10R2 = 3.7 × 0.27 R2 = 0.164KN
6.10R1 = 2.4 × 0.27 R1 = 0.106KN
BM. = 2.4 × 0.164 = 0.4KNm

TOTAL BENDING MOMENT ON JOIST 4.66KNm + 0.4KNm = 5.06KNm.
W = 6.3KN + 0.3KN = 6.6KN.

f (actual) $\frac{6 \times 5.06 \times 10^6}{75 \times 155^2}$ = 16.85 N/mm² EXCESSIVE

DEFLECTION. $\delta = \frac{5 \times 6.60 \times 10^3 \times 5900^3}{384 \times 13500 \times 23.27 \times 10^6}$ = 56.2mm = 1/105 SPAN ∴ FAILS.

IMPROVEMENT SINGLE SPAN BEAM:

SPAN: 5.10M SUPPORT WIDTH 3615/2 + 2400/2 = 3.00 M
LOADINGS GK = 0.35 KN/m² QK = 1.50 KN/m²
DESIGN LOADS 1.4GK + 1.6QK 1.4×0.35 + 1.6×1.50 = 3KN/m²
W = 15.3m² × 3KN/m² = 45.90 KN.
PARTITION: 2.15 × 1.4 = 3.00 KN
S/WT. SAY 2.00 KN TOTAL LOADS 51KN

$M = \frac{WL}{8}$ $\frac{51 \times 5.10}{8}$ = 32.52 KNM. R1 & R2 = 25.5KN. (BY INSPECTION)

INITIAL SELECTION ASSUMING py = 275N/mm²

$Sx > \frac{M}{py}$ $\frac{32.52 \times 10^6}{275}$ = 118cm³

TRY 152 × 152 × 37 kg UC Sx = 309cm³

CLASSIFICATION
STRENGTH d/t = 15.5 < 63ε (63) SHEAR BUCKLING NOT CONSIDERED
SHEAR CAPACITY of SECTION PV
PV = 0.6 × 275 × 8 × 161.8 = 213.6 × 10³ N = 214 KN
AS FV = 25.5 KN < 0.6PV = 0.6 × 214 = 128.4 KN LOW SHEAR LOAD.
BENDING MOMENT CAPACITY MC = PY S 275 × 309 × 10³ = 84.9 × 10⁶ Nmm 85KNm
≤ 1.2 py Z 1.2 × 275 × 273 × 10³ = 90.1 × 10⁶ Nmm 91 KNm
∴ OK

MOMENT M DUE TO IMPOSED LOADING = 32.52 KNm
EXTRA MOMENT DUE TO SELF WEIGHT $M_{SW} = 1.4\,(37 \times 9.81/10^3)\,10^2/8$
$\doteq$ 6.35 KNm

TOTAL IMPOSED MOMENT = 32.52 + 6.35 KNm = 38.87 KNm < 85 KNm

∴ SECTION SUITABLE.

DEFLECTION (FOR 152 x 152 x 30)

$$\delta = \frac{5wL^3}{384\,EI} \qquad \delta = \frac{5 \times 51.00 \times 5.1^3}{384 \times 205 \times 10^6 \times 1748 \times 10^{-8}} = 0.025\,\text{M} = 1/204$$

NOT OK FOR BRITTLE FINISH
BUT OK AS $\not<$ SPAN/200

DEFLECTION. (152 x 152 x 37)

$$\delta = \frac{5 \times 51.00 \times 5.1^3}{384 \times 205 \times 10^6 \times 2210 \times 10^{-8}} = 0.019 = 1/262\,\text{SPAN}.$$

$\simeq$ OK.

WEB BUCKLING (152 x 152 x 37) $P_w = (b_1 + n_1)\,t\,p_c = (100 + 80.9)\,8 \times p_c$

$\lambda = 2.5\,d/t = 2.5 \times 15.5 = 38.75$ ∴ $p_c = 239\,\text{N/mm}^2$ ∴ $180.9 \times 8 \times 239 = \underline{346\,\text{KN}}$

$R_A = 25.5$ KN < 346 KN.

BEARING $P_{crip} = (b_1 + n_2)\,t\,p_{yw} = (100 + 48) \times 8 \times 275 = 325.6$ KN
$(n_2 = 2.5\,(T + r))$.

P_{crip} 325.6 KN $\not<$ 25.5 KN ∴ OK.

BUCKLING
EFFECTIVE LENGTH. BEAM PINNED BOTH ENDS $L_E = 1.0 = 5.10$ m

BUCKLING RESISTANCE $\frac{L_E}{r_y} = \frac{5100}{38.7} = 132$ $n = 0.94$ $\lambda = 0.94 \times 132 = 124$

$x = 13.3$ $p_b = 186$

$M_b = p_b S_x$ $186 \times 309 \times 10^3 = 57.47 \times 10^6$ Nm = 57.47 KNm

IMPOSED MOMENT M = 38.87 KNm < M_b 57.47 ∴ OK.

SECTION 152 x 152 x 37 KG COLD ROLLED UNIVERSAL COLUMN
ACTUAL SIZE FOR DETAILING: D = 161.8 B = 154.4.

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INSPECT ROOF RAFTERS.

SECTION 50 x 75 mm @ 400 ℄
CONSTRUCTION.
CLAY PANTILES
BATTENS WITH LATH & PLASTER.
JOISTS
LATH & PLASTER CEILING.

ALTERATIONS:
REMOVE LATH & PLASTER 20mm o/a. 0.5 KN/M²

ADD.
ROCKWOOL INSULATION. KWA45
45 kg/m³
75mm thk = 3,38 kg/m² 0.033
RW45
100 kg/m³
75 mm thk = 7.50 kg/m² 0.074 KN/M²

STEICO THERM INTERNAL 20mm
D = 160 kg/m³
3.2 kg/m²
0.031 KN/m² 0.031 KN/m²

LIME GREEN SOLO 1 COAT LIME PLASTER
T = 10mm
D = 17 kg/m² 0.170 KN/m²
0.275 KN/m².

RESULT
STRUCTURAL LOADING LIGHTER — PASS

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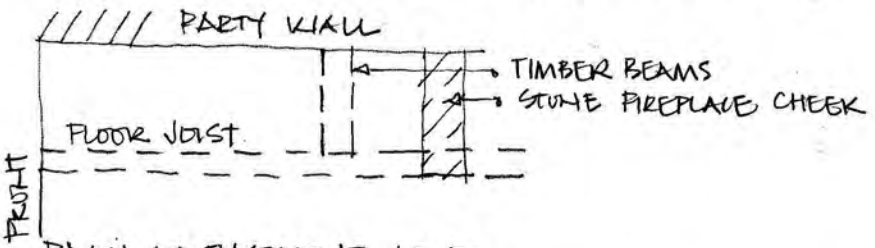
SHIRLEY HOUSE . ROBIN HOODS BAY.
BASEMENT - CHIMNEY BREAST STRUCTURE

LOADINGS:	CHIMNEY STACK	$1.2 \times 0.5 \times 2M$	$= 1.20m^3$	
	FLUES.		$0.24m^3$	$= 1.00m^3$
	ATTIC	$1.45 \times 0.5 \times 2.24$	$= 1.624m^3$	
	FLUES		$0.270m^3$	$= 1.36$
	FIRST FLOOR	$1.92 \times 0.5 \times 2.16$	$= 2.074$	
	FLUES & FIREPLACE		0.800	$= 1.28$
	GROUND FLOOR			$= 1.28$ TOTAL $5M^3$

STONE DENSITY $2000 kg/m^3 \times 5 \times 9.81/1000$ LOAD CHIMNEY $= 96.5 kN$

LOAD DISTRIBUTION AT GPL $96.5 / 2.215m$ $= 44kN/m$
LOAD DISTRIBUTION LHS CHIMNEY BREAST 44×0.6 $= 26.4 kN$

LOAD DISTRIBUTION TO BASEMENT STRUCTURE



PLAN AT BASEMENT LEVEL.

ASSUME TOTAL LOADS RESOLVED BY TIMBER BEAMS THEN REACTIONS AT BEARING $= 26.4/2 = 13.2 kN$

THIS IS THE LOAD REQUIRED TO BE SUPPORTED BY A TIMBER POST:
CALCULATION TO BS 5268.

1. SLENDERNESS RATIO.

$\lambda = \frac{Le}{i}$ FROM TABLES $Le/L = 1$. ENDS RESTRAINED IN POSITION NOT DIRECTION

$Le = 1875mm$

$i = \sqrt{(I/A)} = \sqrt{\frac{75 \times 75^3/12}{75 \times 75}} = 21.65$ $\lambda \ \frac{1875}{21.65} = 86.6 \not> 180$ (Cl. 15.4)

2. GRADE STRESS & MODULUS OF ELASTICITY.

USING GENERAL SOFTWOOD: COMPRESSION ‖ TO GRAIN 6.8 E_{min} 5800

3.0 MODIFICATION FACTOR

$K_3 = 1$ (TABLE 17. BS5268).

K_{12} $\quad \frac{E_{min}}{\sigma_{c,\parallel}} \quad \frac{5800}{6.8 \times 1.0} = 852.9 \quad \lambda = 86.6$

FROM TABLE 22 BS 5268:

	80	86.6	90
800	0.497		0.430
853	0.510	0.466	0.444
900	0.522		0.456

BY INTERPOLATION $\quad K_{12} = 0.466$

4.0 AXIAL CAPACITY

PERMISSIBLE COMPRESSION STRESS PARALLEL TO GRAIN IS
$\sigma_{c,adm,\parallel} = \sigma_{c,g,\parallel} K_3 K_{12} = 6.8 \times 1.0 \times 0.466 = \underline{3.17\ N/mm^2}$

HENCE AXIAL LONG TERM LOAD CAPACITY

$\sigma_{c,adm\parallel} A = 3.17 \times 5625 \times 10^{-3} = 17.83kN. \quad \therefore$ OK $> 13.2kN.$

ADD SACRIFICIAL TIMBER FOR CHARRING FIRE RESISTANCE 12.5mm / FACE

SECTION SIZE 100 x 100mm.

USE BETTER QUALITY TIMBER TO MATCH HISTORIC ASSET.
DOUGLAS FIR OR PITCH PINE KILN DRIED.

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INSPECT FIRST FLOOR JOISTS:
SECTION 75 x 155 @ CENTRES = TO BASEMENT STOREY.
INSPECT SECOND FLOOR JOISTS = DITTO

SINCE JOISTS FAIL IN DEFLECTION PROPOSE INTERMEDIATE SUPPORT.

BEAM SPAN 5.10M SUPPORT WIDTH 3195/2 + 2400/2 = 3.00M

LOADINGS GK = 0.03 kN/m QK = 1.50 kN/m²
DESIGN LOADS 1.4 x 0.03 + 1.6 x 1.5 = 0.042 + 2.4 = 2.442 kN/m²

W = (5.10 x 3.00) x 2.442 = 37.36 kN SAY 38kN
ASSUME BEAM UNIFORMLY LOADED R1 = 19kN R2 = 19kN.

BENDING MOMENT $\frac{WL}{8}$ $\frac{38 \times 5.10}{8}$ M = 24.23 kNm.

INITIAL SELECTION Sx > $\frac{M}{py}$ ASSUME py = 275 N/mm² $\frac{24.23 \times 10^6}{275}$ = 88.11 cm³

TRY 120 x 120 x 40.9 kg SHS.
CLASSIFICATION T = 12.5 Py = 355 ε $\sqrt{275/355}$ = 0.88
FROM STEEL CONSTN INFO 'BLUE BOOK' SECTION CLASS 1. 'PLASTIC'.

SHEAR STRENGTH d/t = 12.5 x 63ε (= 55.44) SHEAR BUCKLING NOT CONSIDERED.
SHEAR CAPACITY SECTION PV (BLUE BOOK) = 534 kN
AS FV (ACTUAL SHEAR) = 19 kN < 0.6 PV 0.6 x 534 = 320 kN ∴ OK.

BENDING MOMENT CAPACITY OF SECTION
Mc = py S Mc = 355 x 207 x 10³ = 73.485 x 10⁶ Nmm = 73.5 kNm
≤ 1.2 py Z = 1.2 x 355 x 164 x 10³ = 69.864 x 10⁶ Nmm = 69.9 kNm.

120 x 120 x 40.9 SHS FAILS.

120 x 120 x 40.9 HOT FINISHED SQUARE HOLLOW SECTIONS. t = 12.5mm / M = 40.9 kg/m /
A = 52.1 / I = 982 cm⁴ / Rad gy. 4.34 / Z = 164 / Sx = 207

BENDING MOMENT CAPACITY of SECTION

$M_c = p_y S$ $\quad M_c = 355 \times 207 \times 10^3 = 73.485 \times 10^6$ Nmm $\quad$ 73.5 KNM

$\leq 1.2 p_y Z$ $\quad 1.2 \times 355 \times 164 \times 10^3 = 69.864 \times 10^6$ Nmm $\quad$ 69.9 KNM $\quad$ FAILS!

TRY. 140 x 140 x 6.3 $\quad M_c = 58.93$ KNM $\quad \leq 1.2 \times 355 \times 141 = 60.07 \therefore$ OK.

REVIEW SECTION

$t = 6.3$ / $M = 26.1$ kg/m / $A = 33.3 cm^2$ / $I = 984 cm^4$ / $Z = 141 cm$ / $S = 166$

CLASSIFICATION: BLUE BOOK - CLASS 1.

SHEAR STRENGTH $d/t = 19.2 < 63\varepsilon$ (55.44) SHEAR BUCKLING OK.

SHEAR CAPACITY 341 KN.

AS F_v (ACTUAL SHEAR) $= 25.5 < 0.6 p_v = 0.6 \times 341 = 204.8 \therefore$ OK.

BENDING MOMENT CAPACITY $\quad M_c = p_y S$. $\quad 355 \times 166 \times 10^3 = 58.93 \times 10^6$ Nmm
(SEE PROJECTION ABOVE). $\quad = 58.93$ KNM.

$\leq 1.2 \times 355 \times 141 = 60.07$ " $\therefore$ OK

TOTAL IMPOSED MOMENT $\quad 32.52$ KNM ≤ 58.93 KNm $\therefore$ OK.

CHECK DEFLECTION

$$\delta = \frac{5}{384} \times \frac{WL^3}{EI} \qquad \frac{5 \times 51 \times 5.1^3}{384 \times 205 \times 10^6 \times 984 \times 10^{-8}} = 0.044 \text{ m} = 1/116 \text{ SPAN}$$

FAIL

THICKER WEB CONSIDERED FOR SERIAL SIZE
HOWEVER Sx & E DO NOT INCREASE ENOUGH TO SATISFY EQUASIONS.

FAIL