

Structural Inspection

Client:	Bosworth Caravan Park
Project:	Class Q barn – Bosworth Caravan Park
Project No:	24-21432
Document No:	0-030
Issue:	3
Date:	26 February 2025
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1. Introduction

- 1.1. Further to instructions we visited Bosworth Caravan Park on 4th September 2024.
- 1.2. The property is a single storey portal framed agricultural building that was used as a vehicle and animal store.
- 1.3. The purpose of our visit was to undertake a visual structural inspection of the property prior to carrying out a Class Q Planning Application to convert the building into a dwelling.
- 1.4. Our inspection was limited to elements of the structure which were visible, exposed or to which access was available.
- 1.5. We did not inspect woodwork or any other part of the structure which was not visible or inaccessible and we are unable, therefore, to advise that any such part is free from defect.
- 1.6. This report may not be used or relied upon by any unauthorised third party, or for any other proposed use than that specified above, without the explicit written agreement of DWS Ltd.
- 1.7. This report has been updated and re-issued following receipt of updated Architectural drawing 24-64-PL03A.

2. Observations

2.1. The existing building appears to be a early 2000's steel portal framed agricultural building that is sited on the edge of a field next to a wooded area.

2.1.1. The barn is subject to a Class Q planning application to convert the barn into a dwelling.



2.2. The barn is timber clad to all 4 elevations with a large galvanised steel door to the front elevation. The vertical timber cladding is supporting on horizontal timber rails and is in a hit and miss style, typical of farm buildings of this type.

2.2.1. The cladding will need to be replaced as part of the conversion to form a weathertight envelope.

2.2.2. The cladding extends to half way down the columns on the North and East elevations.

2.3. The roof covering is cementitious corrugated sheeting overlaying timber purlins supported on the top flange of the steel rafters.

- 2.4. The portal framed is formed in a traditional style from 203x102x23 UB rafter sections and 203x133x25 UB column sections.
- 2.5. The frames are overlain with timber purlins at 1.5m centres supporting the corrugated cementitious roof covering over.
- 2.6. The steel frame, roof and walls were in good condition for a building of this age and type. The columns and rafters displayed very little signs of surface corrosion and are in generally excellent condition.
- 2.7. The eaves and ridge line are generally level with no sign of distortion or distress to the structure.
 - 2.7.1. This indicates that there has been minimal foundation movement and it is reasonably assumed that they are adequately sized for the building.
- 2.8. A structural analysis has been carried out on the frames and is contained in Appendix B. This demonstrates that the steel frame is suitable for supporting the new loads.
 - 2.8.1. A dead load applied to the roof of 0.4kN/m^2 has been allowed for which includes a 12.5mm plasterboard ceiling supported from new timber joists, purlins, insulation and a lightweight roof covering such as single ply membrane, metal sheeting or imitation metal tiles.
 - 2.8.2. The new internal loadbearing inner leaf can be constructed from the ground floor slab and will most likely be constructed from blockwork or 100mm timber frame studwork.
- 2.9. The ground floor is currently a compacted stone floor and a concrete slab will need to be installed as part of the works. However, the compacted stone will be suitable to act as a sub-base for the new slab.
- 2.10. The existing openings in the wall are to be mostly retained to act as windows and doors in the new development. Along the rear and side elevation, the ground floor opening will be made watertight by the addition of bi-fold doors

along the full length of the opening. The steel door aperture will be filled timber barn doors more in keeping with the rest of the upgraded façade. These mask a bi-fold door at ground floor level.

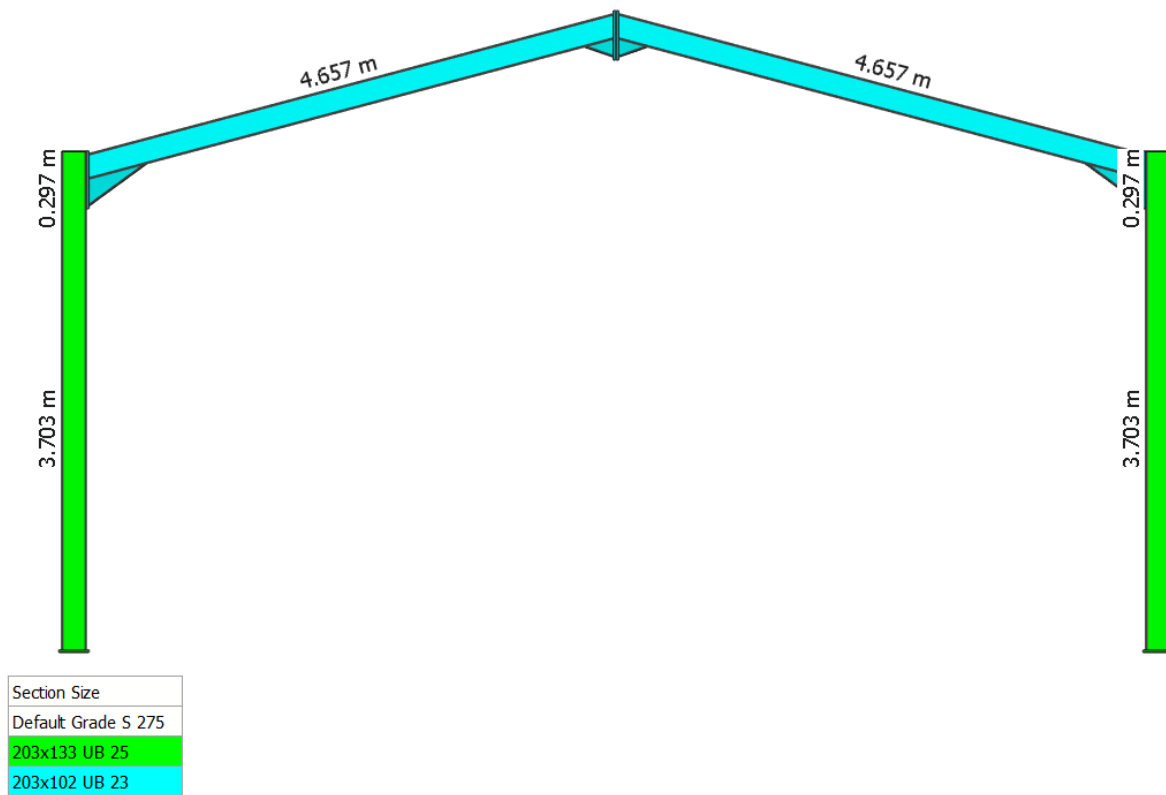
3. Conclusions and Recommendations

- 3.1. It is considered that the building is suitable for conversion into residential accommodation. The steel frame has sufficient structural capacity to resist the vertical and horizontal loads applied to the external envelope.
- 3.1.1. The foundations are displaying no signs of movement and are considered suitable for re-use as part of the conversion with no upgrade works required.
- 3.2. A new ground floor slab is required throughout the building. This will provide support to new internal loadbearing walls that support the new first floor. This would include the necessary insulation and DPM.
- 3.3. It is likely that some repainting to the steel frame will be required.
- 3.4. The upper timber cladded areas will need to be upgraded to provide a weather-tight envelope.
- 3.4.1. The internal leaf of the external walls will need to be constructed from the new groundbearing slab.
- 3.5. The roof will be re-covered in a lightweight covering such as single ply membrane, standing seam metal roof or lightweight metal tiles.
- 3.6. The new slab, inner leaf, first floor and partitions are all internal works that do not effect the external structure and is therefore not considered to be development.
- 3.7. The overall footprint and form of the building is unaffected by the works. The structural works related to the conversion are considered to be minimal.

Appendix A – Photos

	
Photo 1 – Front Elevation	Photo 2 – Internal view

Appendix B – Steel Frame Design



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Steel Design Brief Summary											
Brief No.	Type	Title	Portion	Case	Unity	Pass	Local	Shear	LT buckling	Slenderness	Deflection
1	Elastic Crit.					Pass					
2	Axial+Mom.	Column 1 : Members 3 & 5 (N.1-N.5)	0 to 2	001	0.412	Pass	0.245	0.049	0.537	0.358	0.412
3	Axial+Mom.	Column 1 : Members 3 & 5 (N.1-N.5)	2 to 4	001	0.537	Pass	0.245	0.049	0.537	0.358	0.412
4	App. G	Column 1 : Members 3 & 5 (N.1-N.5)	2 to 4	001	0.687	Pass				0.358	
5	Axial+Mom.	Rafter 1 of Bay 1 : Member 1 (N.5-N.7)	0.1 to 1.5	001	0.354	Pass	0.139	0.047	0.398	0.353	0.302
6	Axial+Mom.	Rafter 1 of Bay 1 : Member 1 (N.5-N.7)	1.5 to 3	005	0.302	Pass	0.139	0.047	0.398	0.353	0.302
7	Axial+Mom.	Rafter 1 of Bay 1 : Member 1 (N.5-N.7)	3 to 4.5	005	0.398	Pass	0.139	0.047	0.398	0.353	0.302

8	App. G	Rafter 1 of Bay 1 : Member 1 (N.5-N.7)	0.1 to 1.621	001	0.437	Pass				0.342	
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AXIAL WITH MOMENTS (MEMBER)

Column 1 : Members 3 & 5 (N.1-N.5)

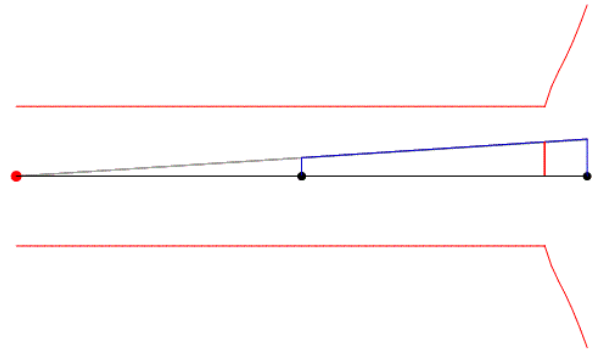
Between 2.000 and 4.000 m, in Load Case 1

Member Loading and Member Forces

Loading Combination : 1 UT + 1.4 D1 + 1.4 D2 + 1.6 L1

UT Spacing 04.500 [Multiply AllLoads]

UT PartFix 20.00 +++ --- (Mt My Mz)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 2						
Member No.	Node End 1 End 2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kNm)	Maximum Moment (kNm)	Maximum Deflection (mm @ m)
-	1	33.685C	-9.454	0.000		4.573
	5	32.290C	-9.454	-37.816		@ 2.370

Classification and Properties (BS 5950: 2000)

Section (25.09 kg/m) 203x133 UB 25 [S 275]
 Class = Fn(b/T,d/t,py,F,Mx,My) 8.54, 30.25, 275, 33.68, 37.82, 0

(Axial: Non-Slender)

Plastic

Auto Design Load Cases

1 & 5

Local Capacity Check

F_{vx}/P_{vx} 9.454 / 191.11 =
 $M_{cx} = p_y.S_{xx} \leq 1.2 p_y.Z_{xx}$ 275 x 257.7 ≤ 1.2 x 275 x 230.42 =
 $P_z = A_g.p_y$ 31.96 x 275 =
 $n = F/P_z$ 33.685 / 878.9 =
 $S_{rx} = F_n(S_{xx})$ 257.7, 0.038
 $M_{rx} = S_{rx}.p_y$ 257.04 x 275
 $(M_x/M_{rx})^{2.1} + (M_y/M_{ry})^{2.2}$ (35.008/70.686)^2 + (0)^2 =

0.049
 70.868 kN.m
 878.9 kN
 0.038
 257.04 cm³
 70.686 kN.m
 0.245

Low Shear

OK

OK

Compression Resistance Pc

$\lambda_x = L_{ex}/r_{xx}$ 100x1x4/8.56 =
 $P_{cx} = A_{area}.p_{cx}$ 31.96x254.257/10 =
 $\lambda_y = L_{ey}/r_{yy}$ 100x1x2/3.1 =
 $P_{cy} = A_{area}.p_{cy}$ 31.96x212.54/10 =

46.7
 812.606 kN
 64.5
 679.262 kN

OK

Table 24 a

OK

Table 24 b

Equivalent Uniform Moment Factors mLT, mx, my and myx

$m_{LT} = 0.2 + (.15M_2 + .5M_3 + .15M_4)/M_{max}$ 0.2 + (.15x24 + .5x28 + .15x33)/38 = 0.44
 $m_y = 0.2 + (.1M_2 + .6M_3 + .1M_4)/M_{max}$ 0.2 + (.1x0 + .6x0 + .1x0)/0 = .8x0/0
 $m_x = 0.2 + (.1M_2 + .6M_3 + .1M_4)/M_{max}$ 0.2 + (.1x-9 + .6x-19 + .1x-28)/38 = .8x28/38
 $m_{yx} = 0.2 + (.1M_2 + .6M_3 + .1M_4)/M_{max}$ 0.2 + (.1x0 + .6x0 + .1x0)/0 = .8x0/0

1
 1
 0.6
 1

Table 18

Table 26

Table 26

Table 26

Lateral Buckling Check Mb

$L_e = 1.00 L$ 1 x 2 =
 $\lambda = L_e/r_{yy}$ 2 / 3.1
 $v = F_n(x, L_e, r_{yy}, \lambda)$ 25.586, 2, 3.1, 64.47
 $\lambda_{LT} = u.v.\lambda.\sqrt{\beta_w}$ 0.877 x 0.933 x 64.47 √ 1
 $p_b = F_n(p_y, \lambda_{LT})$ 275, 52.79
 $M_b = S_{xx}.p_b \leq M_c$ 257.7 x 231.14 ≤ 70.868 =

2 m
 64.47
 0.933
 52.79
 231.14 N/mm²
 59.564 kN.m

OK

Table 19

Table 16

Combined Axial Compression and Bending to Annex I

$r_b = m_{LT} \cdot M_{LT} / M_b$	$1x-35/59.6$	0.588	
$r_c = F_c / P_{cy}$	$33.7/679.3$	0.050	
$\lambda_r = (r_b \lambda_{LT} + r_c \lambda_y) / (r_b + r_c)$	$(0.588 \cdot 52.8 + 0.05 \cdot 64.5) / (0.588 + 0.05)$	53.698	
$\lambda_{r0} = 17.15 \varepsilon (2r_b + r_c) / (r_b + r_c)$	$17.15 \cdot 1 (2 \cdot 0.588 + 0.05) / (0.588 + 0.05)$	32.966	
$M_{ob} = M_b (1 - F_c / P_{cy})$	$59.564 (1 - 33.7/679.3)$	56.610	
$M_{xy} = M_{cx} (1 - F_c / P_{cy})^{1/2}$	$70.868 (1 - 33.7/679.3)^{1/2}$	69.088	
$M_{ox} = M_{cx} (1 - F_c / P_{cx}) / (1 + 0.5 F_c / P_{cx})$	$70.868 (1 - 33.7/812.6) / (1 + 0.5 \cdot 33.7/812.6)$	66.550	
$M_{oy} = M_{cy} (1 - F_c / P_{cy}) / (1 + k_y (F_c / P_{cy}))$	$15.241 (1 - 33.7/679.3) / (1 + 1.0 (33.7/679.3))$	13.801	
$M_{ab} = \text{fn}(\lambda_r, \lambda_{r0}, \varepsilon, M_{xy}, M_{ob})$	53.698, 32.966, 1.000, 69.088, 56.610	65.163	
$M_{ax} = \text{fn}(\lambda_x, \varepsilon, M_{ix}, M_{ox})$	46.745, 1.000, 70.686, 66.550	68.903	
$M_{ay} = \text{fn}(\lambda_y, \varepsilon, M_{iy}, M_{oy})$	64.467, 1.000, 15.241, 13.801	14.249	
$m_x \cdot M_x / M_{ax}$	$0.6x35/68.9$	0.305	OK
$m_{LT} \cdot M_{LT} / M_{ab}$	$1x-35/65.2$	0.537	OK
$m_x \cdot M_x / M_{ax}$	$0.6x35/68.9$	0.305	OK
Compare with Simplified to 4.8.3.3	0.381, 0.637, 0.637	0.637	
Compare with MoreExact to 4.8.3.3	0.344, 0.637, 0.526	0.637	

Deflection Check - Load Case 2

In-span $\delta \leq \text{Span}/360$	$4.57 \leq 4000 / 360$	4.57 mm	OK
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APPENDIX-G STABILITY (MEMBER) : G.2.(A).2

Column 1 : Members 3 & 5 (N.1-N.5)

Between 2.000 and 4.000 m, in Load Case 1

Member Loading and Member Forces

Loading Combination : 1 UT + 1.4 D1 + 1.4 D2 + 1.6 L1

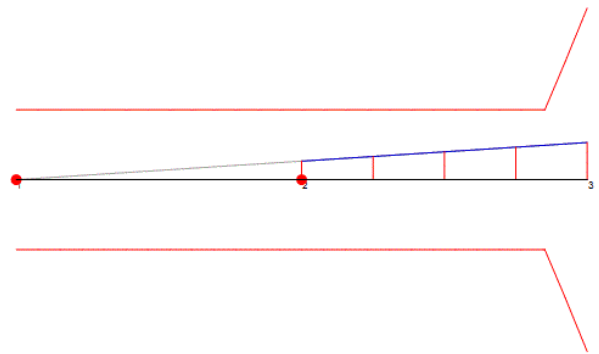
UT Spacing 04.500 [Multiply AllLoads]

UT PartFix 20.00 +++ --- (Mt My Mz)

Lateral and Torsional Restraints

Side rails @ 2 and 4 m

Stays @ 2 and 4 m



Member Forces in Load Case 1						
Member No.	Node End 1 End 2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kNm)	Maximum Moment (kNm)	Maximum Deflection (mm @ m)
-	1	33.685C	-9.454	0.000		0.000
	5	32.290C	-9.454	-37.816		@ 2.370

Classification and Properties (BS 5950: 2000)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = $\text{Fn}(b/T, d/t, p_y, F, M_x, M_y)$	8.54, 30.25, 275, 33.68, 37.82, 0	(Axial: Non-Slender)	Plastic
Haunched Section	Mid flange included in section classification and properties		
Auto Design Load Cases	1 & 5		

Compression Resistance Pc

$\lambda_y = L / r_{yy}$	$100x2/3.1 =$	64.47	OK
$y = \text{Fn}(a, h_s, x, \lambda_y)$	121.92, 195.4, 55.339, 64.467	0.943	G.2.3
$\lambda_{TC} = y \cdot \lambda$	$0.943x64.47 =$	60.8	OK
$P_{cy} = \text{Area} \cdot p_{cy}$	$31.96x219.16/10 =$	700.421 kN	Table 24 b

Slenderness Correction Factor nt

$R_1 M_{x1}(p_y, S_{xx})$	$18.909(.001x 275x257.7) = 0$	0.267	
$R_2 M_{x2}(p_y, S_{xx})$	$23.636(.001x 275x257.7) = 0$	0.334	
$R_3 M_{x3}(p_y, S_{xx})$	$28.363(.001x 275x257.7) = 0$	0.400	
$R_4 M_{x4}(p_y, S_{xx})$	$33.090(.001x 275x257.7) = 0$	0.467	
$R_5 M_{x5}(p_y, S_{xx})$	$37.815(.001x 275x633.6) = 0$	0.217	
$R_5 - R_E$	$0.467 - 0.267 = 0$	0.200	
R_{max}		0.467	
$nt = \text{Fn}(\text{All above})$	0.267, 0.334, 0.400, 0.467, 0.217, 0.200, 0.467	0.934	G.4.3

Lateral Buckling Resistance Moment Mb

$M_p = p_y.S_{xx} \leq 1.2 p_y.Z_{xx}$	$275 \times 257.7 \leq 1.2 \times 275 \times 230.42 =$	70.868 kN.m	
$\lambda_y = L/r_{yy}$	$100 \times 2 / 3.1 =$	64.47	OK
$v_t = F_n(a, x, h_s, \lambda)$	121.92, 55.339, 195.4, 64.467	0.932	G.2.4.2
$c = F_n(R, q, x)$	2, 0.149, 55.339	1.07	G.2.5
$\lambda_{TB} = c.n.t.v_{ty}$	$1.07 \times 0.934 \times 0.932 \times 64.467$	60.005	G.2.4.2
$p_b = F_n(p_y, \lambda_{TB})$	275, 60.01	213.21 N/mm ²	Table 16
$M_b = S_{xx}.p_b \leq M_p \leq p_y.Z_{xx}$	$257.7 \times 213.21 \leq 70.868 \leq 275 \times 230.33 =$	54.803 kN.m	

Elastic Stability of Tapered Members : G.2.2

$F/P_c + M/M_b$	$33.685 / 700.42 + 35.008 / 54.8$	0.687	OK
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AXIAL WITH MOMENTS (MEMBER)

Rafter 1 of Bay 1 : Member 1 (N.5-N.7)

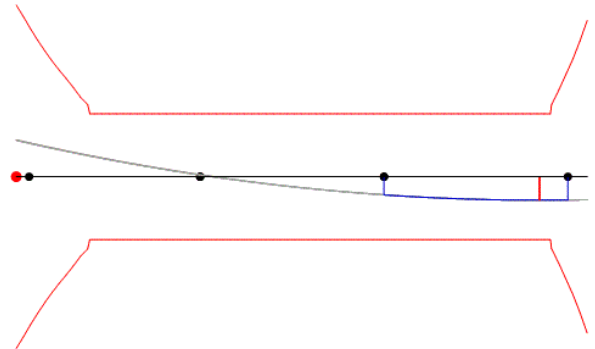
Between 3.000 and 4.500 m, in Load Case 5

Member Loading and Member Forces

Loading Combination : 1 UT + 1.4 D1 + 1.4 D2 + 1.6 L1

UT Spacing 04.500 [Multiply All Loads]

D1 UDLY -000.400	[kN/m]
D2 UDLY -000.000	[kN/m]
L1 UDLY -000.600	[kN/m]



Member Forces in Load Case 5 and Maximum Deflection from Load Case 2						
Member No.	Node End 1 End 2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kNm)	Maximum Moment (kNm)	Maximum Deflection (mm @ m)
1	5	17.336C	28.622	-37.156	23.986	3.907
	7	9.016C	-2.578	23.490	@ 4.285	@ 3.074

Classification and Properties (BS 5950: 2000)

Section (23.07 kg/m)	203x102 UB 23 [S 275]		
Class = $F_n(b/T, d/t, p_y, F, M_x, M_y)$	5.47, 31.37, 275, 17.34, 37.16, 0	(Axial: Non-Slender)	Plastic
Auto Design Load Cases	1 & 5		

Shear Capacity Check

F_{vx}/P_{vx}	$8.524 / 181.051 =$	0.047	OK
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Local Capacity Check

F_{vx}/P_{vx}	$0.02 / 181.051 =$	0	Low Shear
$M_{cx} = p_y.S_{xx} \leq 1.2 p_y.Z_{xx}$	$275 \times 234.1 \leq 1.2 \times 275 \times 207.18 =$	64.378 kN.m	
$P_z = A_g.p_y$	$29.39 \times 275 =$	808.225 kN	
$n = F/P_z$	$17.336 / 808.225 =$	0.021	OK
$S_{rx} = F_n(S_{xx},)$	234.1, 0.021	233.92 cm ³	
$M_{rx} = S_{rx}.p_y$	233.92×275	64.327 kN.m	
$(M_x/M_{rx})^{2.1} + (M_y/M_{ry})^{2.2}$	$(23.985/64.327)^{2.1} + (0)^{2.2} =$	0.139	OK

Compression Resistance Pc

$\lambda_x = L_e/r_{xx}$	$100 \times 1 \times 4.657 / 8.46 =$	55	OK
$P_{cx} = A_{area}.p_{cx}$	$29.39 \times 245.582 / 10 =$	721.767 kN	Table 24 a
$\lambda_y = L_e/r_{yy}$	$100 \times 1 \times 1.5 / 2.36 =$	63.5	OK
$P_{cy} = A_{area}.p_{cy}$	$29.39 \times 214.28 / 10 =$	629.778 kN	Table 24 b

Equivalent Uniform Moment Factors mLT, mx, my and myx

$m_{LT} = 0.2 + (.15M_2 + .5M_3 + .15M_4)/M_{max}$	$0.2 + (.15 \times 21 + .5 \times 23 + .15 \times 24)/24 =$	0.44	1	Table 18
$m_y = 0.2 + (.1M_2 + .6M_3 + .1M_4)/M_{max}$	$0.2 + (.1 \times 0 + .6 \times 0 + .1 \times 0)/0 =$	.8x0/0	1	Table 26
$m_x = 0.2 + (.1M_2 + .6M_3 + .1M_4)/M_{max}$	$0.2 + (.1 \times -8 + .6 \times 11 + .1 \times 22)/37 =$	.8x22/37	0.473	Table 26
$m_{yx} = 0.2 + (.1M_2 + .6M_3 + .1M_4)/M_{max}$	$0.2 + (.1 \times 0 + .6 \times 0 + .1 \times 0)/0 =$	.8x0/0	1	Table 26

Lateral Buckling Check Mb

$L_e = 1.00 L$	$1 \times 1.5 =$	1.5 m	
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$$\lambda = L_e / r_{yy} = 1.5 / 2.36$$

$$v = F_n(x, L_e, r_{yy}, \lambda) = 22.431, 1.5, 2.36, 63.52$$

$$\lambda_{LT} = u.v.\lambda.\sqrt{\beta_w} = 0.889 \times 0.919 \times 63.52 \sqrt{1}$$

$$p_b = F_n(p_y, \lambda_{LT}) = 275, 51.89$$

$$M_b = S_{xx}.p_b \leq M_c = 234.1 \times 233.35 \leq 64.378 =$$

$$63.52 \quad \text{OK}$$

$$0.919 \quad \text{Table 19}$$

$$51.89$$

$$233.35 \text{ N/mm}^2 \quad \text{Table 16}$$

$$54.628 \text{ kN.m}$$

Combined Axial Compression and Bending to Annex I

$$r_b = m_{LT}.M_{LT}/M_b = 1 \times 24 / 54.6 = 0.439$$

$$r_c = F_c / P_{cy} = 17.3 / 629.8 = 0.028$$

$$\lambda_r = (r_b \lambda_{LT} + r_c \lambda_y) / (r_b + r_c) = (0.439 \times 51.9 + 0.028 \times 63.5) / (0.439 + 0.028) = 52.575$$

$$\lambda_{r0} = 17.15 \varepsilon (2r_b + r_c) / (r_b + r_c) = 17.15 \times 1 (2 \times 0.439 + 0.028) / (0.439 + 0.028) = 33.288$$

$$M_{ob} = M_b (1 - F_c / P_{cy}) = 54.628 (1 - 17.3 / 629.8) = 53.124$$

$$M_{xy} = M_{ox} (1 - F_c / P_{cy})^{1/2} = 64.378 (1 - 17.3 / 629.8)^{1/2} = 63.485$$

$$M_{ox} = M_{ox} (1 - F_c / P_{cx}) / (1 + 0.5 F_c / P_{cx}) = 64.378 (1 - 17.3 / 721.8) / (1 + 0.5 \times 17.3 / 721.8) = 62.086$$

$$M_{oy} = M_{cy} (1 - F_c / P_{cy}) / (1 + k_y (F_c / P_{cy})) = 10.626 (1 - 17.3 / 629.8) / (1 + 1.0 (17.3 / 629.8)) = 10.057$$

$$M_{ab} = f_n(\lambda_r, \lambda_{r0}, \varepsilon, M_{xy}, M_{ob}) = 52.575, 33.288, 1.000, 63.485, 53.124 = 60.212$$

$$M_{ax} = f_n(\lambda_x, \varepsilon, M_{rx}, M_{ox}) = 55.032, 1.000, 64.327, 62.086 = 63.090$$

$$M_{ay} = f_n(\lambda_y, \varepsilon, M_{ry}, M_{oy}) = 63.519, 1.000, 10.626, 10.057 = 10.242$$

$$m_x.M_x/M_{ax} = 0.473 \times 24 / 63.1 = 0.180 \quad \text{OK}$$

$$m_{LT}.M_{LT}/M_{ab} = 1 \times 24 / 60.2 = 0.398 \quad \text{OK}$$

$$m_x.M_x/M_{ax} = 0.473 \times 24 / 63.1 = 0.180 \quad \text{OK}$$

$$\text{Compare with Simplified to 4.8.3.3} = 0.226, 0.467, 0.467$$

$$\text{Compare with MoreExact to 4.8.3.3} = 0.202, 0.467, 0.386$$

Deflection Check - Load Case 2

$$\text{In-span } \delta \leq \text{Span} / 360 = 3.91 \leq 4657 / 360 = 12.94 \quad \text{OK}$$

APPENDIX-G STABILITY (MEMBER) : G.2.(A).2

Rafter 1 of Bay 1 : Member 1 (N.5-N.7)

Between 0.105 and 1.621 m, in Load Case 1

Member Loading and Member Forces

Loading Combination : 1 UT + 1.4 D1 + 1.4 D2 + 1.6 L1

UT Spacing 04.500 [Multiply AllLoads]

D1 UDLY -000.400 [kN/m]

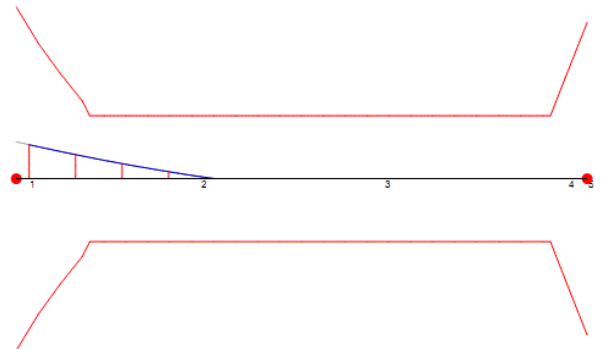
D2 UDLY -000.000 [kN/m]

L1 UDLY -000.600 [kN/m]

Lateral and Torsional Restraints

Purlins @ 1.5, 3, 4.5 and 4.657 m

Stay @ 4.657 m



Member Forces in Load Case 1						
Member No.	Node End 1 End 2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kNm)	Maximum Moment (kNm)	Maximum Deflection (mm @ m)
1	5	17.455C	28.763	-37.816	23.933	0.000
	7	9.135C	-2.436	23.491	@ 4.285	@ 3.074

Classification and Properties (BS 5950: 2000)

Section (23.07 kg/m) 203x102 UB 23 [S 275]

Class = $F_n(b/T, d/t, p_y, F, M_x, M_y)$ 5.47, 31.37, 275, 17.45, 37.82, 0

(Axial: Non-Slender) Plastic

Auto Design Load Cases 1 & 5

Compression Resistance P_c

$$\lambda_y = L / r_{yy} = 100 \times 1.516 / 2.46 = 61.59$$

$$y = F_n(a, h_s, x, \lambda_y) = 121.92, 234.652, 22.431, 61.592$$

$$\lambda_{TC} = y.\lambda = 0.945 \times 61.59 = 58.23$$

$$P_{cy} = \text{Area}.p_{cy} = 29.39 \times 223.59 / 10 = 657.122 \text{ kN} \quad \text{Table 24 b}$$

Slenderness Correction Factor n_t

$$R_1 M_{x1}(p_y.S_{xx}) = 34.831(.001 \times 275 \times 559.1) = 0.227$$

$$R_2 M_{x2}(p_y.S_{xx}) = 24.678(.001 \times 275 \times 327.8) = 0.274$$

$$R_3 M_{x3}(p_y.S_{xx}) = 15.488(.001 \times 275 \times 234.1) = 0.241$$

$R_4 M_{x4}(p_y.S_{xx})$	$7.259(.001 \times 275 \times 234.1) = 0$	0.113	
$R_5 M_{x5}(p_y.S_{xx})$	$0.000(.001 \times 275 \times 234.1) = 0$	0.000	
$R_5 - R_E$	$0.274 - 0.227 = 0$	0.047	
R_{max}		0.274	
$n_t = F_n(\text{All above})$	0.227, 0.274, 0.241, 0.113, 0.000, 0.047, 0.274	0.862	G.4.3

Lateral Buckling Resistance Moment Mb

$M_p = p_y.S_{xx} \leq 1.2 p_y.Z_{xx}$	$275 \times 234.1 \leq 1.2 \times 275 \times 207.26 =$	64.378 kN.m	
$\lambda_y = L/r_{yy}$	$100 \times 1.516 / 2.46 =$	61.59	OK
$v_t = F_n(a, x, h_s, \lambda)$	121.92, 22.431, 234.652, 61.592	0.945	G.2.4.2
$c = F_n(R, q, x)$	1.945, 0.327, 22.431	1.088	G.2.5
$\lambda_{TB} = c.n_t.v_{ty}$	$1.088 \times 0.862 \times 0.945 \times 61.592$	54.625	G.2.4.2
$p_b = F_n(p_y, \lambda_{TB})$	275, 54.63	226.6 N/mm ²	Table 16
$M_b = S_{xx}.p_b \leq M_p \leq p_y.Z_{xx}$	$234.1 \times 226.6 \leq 64.378 \leq 275 \times 207.18 =$	53.005 kN.m	

Elastic Stability of Tapered Members : G.2.2

$F/P_c + M/M_b$	$17.455 / 657.12 + 21.764 / 53.01$	0.437	OK
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Appendix C – Architectural Drawings

The proposed dwelling has been designed by a qualified and experienced Architect and is entirely appropriate in terms of design and external appearance. No more work is proposed than that reasonably required for the barn to perform as a dwelling house.

