

Project	Pinley Farm, Hatton	Calc.by	R.S.	Proj.No.	R10356
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Introduction –

The building is understood to be 14 years old and has a steel frame construction.

The roof is duo-pitch, covered with insulated, profiled metal sheeting supported on cold rolled purlins spanning between steel portal frames. There are gables to the front and rear.

The walls have an outer leaf of brickwork tied across a cavity to an inner leaf of blockwork.

The floor is concrete, understood to be 140 mm thick, laid over a 1200 gauge polythene slip/damp proof membrane over compacted hardcore.

The building is generally in good structural order and the steel frame is sound and suitable for retention.

Calculations to prove the existing structure follow.

Building Works –

Building works to convert the existing building into residential will include the following:-

- retain the existing steel frames, roof sheeting, purlins, walls and floor slab,
- overlay the existing floor with a damp proof membrane, insulation and screed.

Summary –

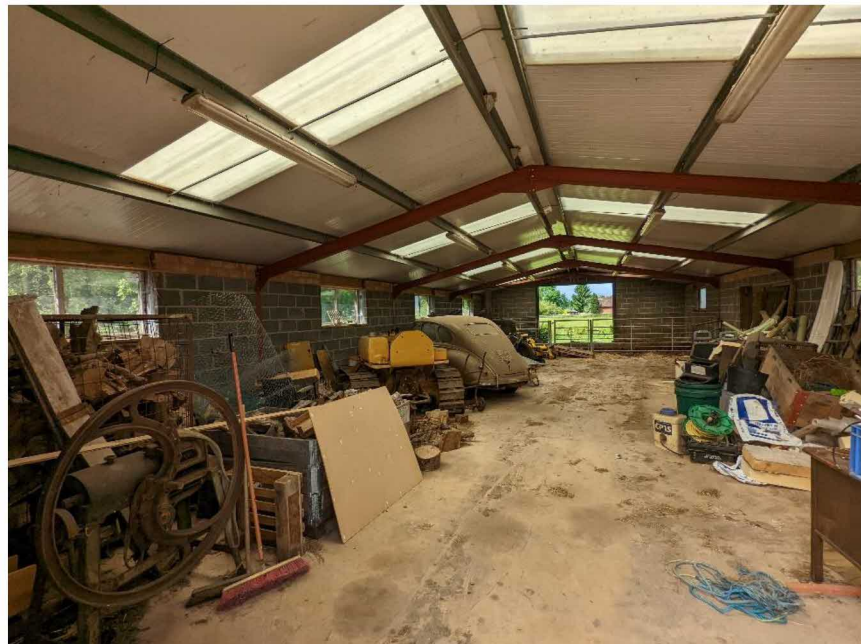
With reference to the plans and large scale details shown on MAB Architecture drawing number 2023-05, I can confirm that the conversion can take place within the fabric of the existing building using the existing structural elements.

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Plate 1 – external view.



Plate 2 – internal view.



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Note – the only purpose of these calculations is to prove the existing steel framed structures in support of an application for change of use and they may not be relied on for any other purpose.

Loading - (kN/m²)

Pitched roof -	<u>Dead</u> -	Composite sheets -	0.15
		Services -	0.10
		Purlins -	0.05
		Rafters -	<u>0.05</u>
			0.35
	<u>Live</u> -		<u>0.60</u>
			0.95

Drifting snow - $S_b = 0.55 = S_o$
 $\mu_1 = 0.8$ & $S_d = 0.8 \times 0.55 = 0.44$

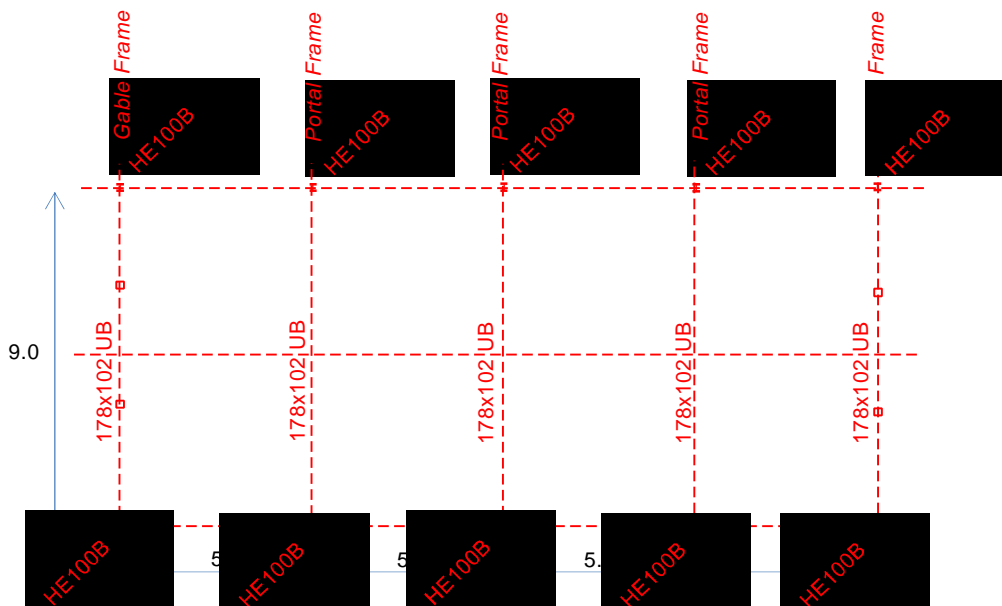
Wind - $S_a = S_d = S_s = S_p = 1.0$
 $V_b = 20.5 = V_s$

$S_b = 1.35, V_e = 27.8, q = 0.47$

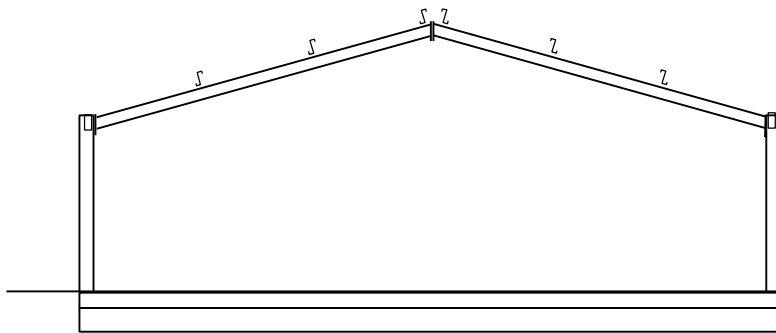
Walls:	Cpe	Cpi	p
Windward	+0.85	-0.3	0.54
Leeward	-0.5	-0.3	-0.09

Roof:	Cpe	Cpi	p
	-0.5	-0.3	-0.09

Risks: Work within scope of routine activities for a competent contractor.



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142 cold rolled purlins
 178 x 102 UB portal rafters
 HE 100 B portal cols
 Frames @ 5 m c/c

Sketch Existing Section

140 tk plain concrete slab

Check ex purlins = 142Z13 @ 1500 c/c –

/ MetSPEC /[®]
DESIGN SUITE

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 Job No.:
 Designer:
 Date: 14/06/2022
Registered Details:-
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Site:

Comment:

PURLIN SELECTION

Z PURLIN SYSTEM TYPE: SLEEVED

METAL CLADDING

Dimensions & Limits

Design Loads in kN/m²

Span: 5.000 m
 Centres: 1.500 m
 Deflection Limit: Span/ 180
 Roof Slope: 12.5 deg.

Dead Load : 0.250
 Service Load : 0.100
 Super Load : 0.600
 Wind Uplift Load : 0.110

SELECTED PURLINS

Section			Pass	Unfactored load for deflection	Ultimate download	Ultimate wind uplift
Required Loads in kN/m ² :				0.950	1.450	0.000
Capacity Loads in kN/m ² :						
Section Reference	Weight in kg/m	Restraint				
142.Z.13	2.84	0	Sags	1.109	1.841	1.119

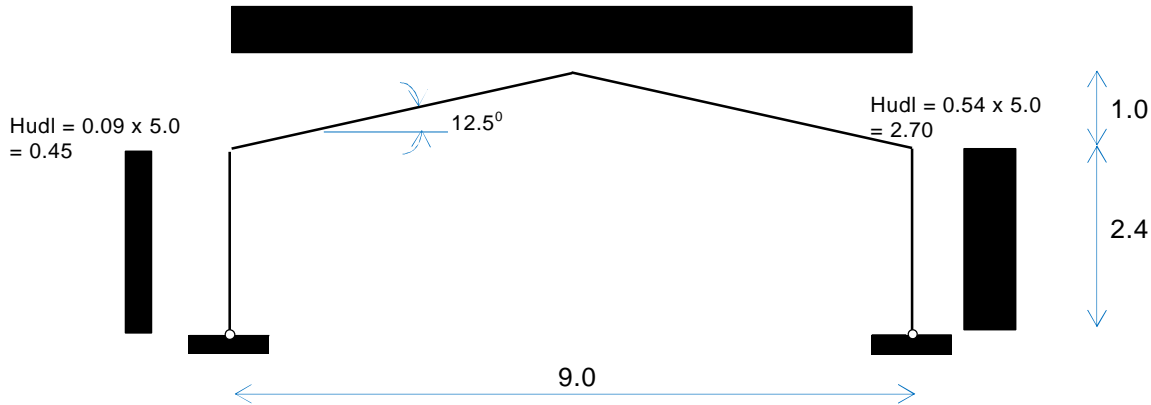
The above values assume that the cladding panel or liner tray is screw fixed to the section(s) at a maximum spacing of 600mm.

Section passes – ex purlins OK.

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Check ex Portal Frames –

$$\begin{aligned} \text{RDL} &= 0.35 \times 5.0 = 1.75 \\ \text{RLL} &= 0.6 \times 5.0 = 3.00 \\ \text{RWL} &= -0.09 \times 5.0 = -0.45 \end{aligned}$$



Take nominal support spring stiffness = 20% of $4EI/L$ col stiffness = $0.8EI/L$ (5.1.3.3) – note: use pinned base for section design.

$$\begin{aligned} \text{HE-100-B – major} &= 0.8 \times 205 \text{ kN/mm}^2 \times 450 \text{ cm}^4 / 2.4 \text{ m} \quad (\times 10^{-4}) \\ &= 3.1 \times 10^6 \text{ kNm/rad} \\ I_x &= 450, I_y = 167, Z_x = 89.9, A = 26, \sqrt{I_y/A} = 25.3, D/T = 100/10 = 10. \end{aligned}$$

From analysis following –

Portal Rafters – 178x102 UB's.

$$M_{+max} = 19.5, l_{ef} = 1.5$$

$$I/ry = 1500 / 23.7 = 63 \quad D/T = 22.5 \quad P_{bc} = 166$$

$$f_{bc} = 19.5 \times 10^3 / 153 = 127 \quad \therefore \text{OK}$$

Portal Columns – HE-100-B.

$$M(\text{btm of haunch}) = 13.0, N = 21.4$$

$$I/ry = 2400 / 25.3 = 95 \quad D/T = 10 \quad P_{bc} = 162 \quad P_c = 89$$

$$f_{bc} = 13.0 \times 10^3 / 89.9 = 144 \quad f_c = 21.4 \times 10^3 / 26 \times 10^2 = 8$$

$$\Sigma(f/p) = 144/162 + 8/89 = 0.98 < 1.0 \quad \therefore \text{OK}$$

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Foundations –

Generally 0.6m wide x 1.0 o/all deep below external walls, widened below portal columns to 0.9m x 0.9m –

Length (L) = 0.90 Nmax = 21.4
 Width (B) = 0.90 Hmax = 8.4
 Depth (D) = 0.70
 o/w base = 13.6 W = 35.0
 Depth o/bdn = 0.45
 o/w o/bdn = (LxBxdx20) = 7.3

1/ Vmax: M = 5.9
 e = 0.139 - within middle 3rd

$$Q = \frac{35.0}{0.81} \times \left(1 \pm 6 \times \frac{0.139}{0.9}\right) = 83.3, 3.2 \text{ SGBP} > 83 \text{ reqd}$$

$$RM/OTM = (W \times L/2)/(H \times D) = 15.8 / 5.9 = 2.679$$

2/ Vmin: Nmin = 6.9
 Hmin = 4.6
 Ntotal = 27.8

M = 3.22
 e = 0.116 - within middle 3rd

$$Q = \frac{27.8}{0.81} \times \left(1 \pm 6 \times \frac{0.116}{0.9}\right) = 60.8, 7.8 \text{ SGBP} > 61 \text{ reqd}$$

$$RM/OTM = (N \times L/2)/(H \times D) = 12.5 / 3.2 = 3.885$$

- SGBP > 90 reqd – ex foundations OK.

Gable Frames –

Existing gable rafters and gable columns have the benefit of additional door posts and are therefore, by inspection, OK.

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Members

Mbr ref	Member type	Start joint	Start fixity	End joint	End fixity	Orient (°)	Directional behaviour	Length (m)	P-Delta behaviour	Slope (°)
1	HE 100 B	1	Fixed	2	Fixed	0.0	Normal	2.400	Normal	90.0
2	HE 100 B	4	Fixed	5	Fixed	0.0	Normal	2.400	Normal	90.0
3	178x102 UKB19	2	Fixed	3	Fixed	0.0	Normal	4.610	Normal	12.5
4	178x102 UKB19	5	Fixed	3	Fixed	0.0	Normal	4.610	Normal	12.5

Joints

Joint ref	X pos (m)	Y pos (m)	Z pos (m)	Joint ref	X pos (m)	Y pos (m)	Z pos (m)	Joint ref	X pos (m)	Y pos (m)	Z pos (m)
1	0.000	0.000	0.000	2	0.000	2.400	0.000	3	4.500	3.400	0.000
4	9.000	0.000	0.000	5	9.000	2.400	0.000				

Sections

Reference	Area (cm ²)	Ixx (cm ⁴)	Iyy (cm ⁴)	J (cm ⁴)	Elements (mm)				
					No	Width	Height	Vert. off	Lat. off
178x102 UKB19	24.3	1356	137	4.41					
HE 100 B	26.04	449.5	167.3	9.25					

Member Loads

Load reference	Load type	Start pos'n (m)	Start intensity (kN) & (m)	End pos'n (m)	End intensity (kN) & (m)	Direction	Category
<i>Loads on member 1 (Length 2.400m)</i>							
hl	UL		-0.450			Horiz. (+X)	Other
<i>Loads on member 2 (Length 2.400m)</i>							
hw	UL		-2.700			Horiz. (+X)	Other
<i>Loads on member 3 (Length 4.610m)</i>							
rd	UL		1.750			Vertical (-Y)	Dead
rl	UL		3.000			Vertical (-Y)	Imposed
rw	UL		-0.450			Vertical (-Y)	Other
<i>Loads on member 4 (Length 4.610m)</i>							
rd	UL		1.750			Vertical (-Y)	Dead
rl	UL		3.000			Vertical (-Y)	Imposed
rw	UL		-0.450			Vertical (-Y)	Other

Load Combinations

<u>Load Category</u>		<u>Partial Safety Factors</u>		
No	Name	1	2	3
		d+l	d+l+w	d+w
1	Dead	1.00	1.00	1.00
2	Imposed	1.00	1.00	0.00
3	Wind	0.00	1.00	1.00

Joint Displacements for Combination d+l

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
1	0.00	0.00	0.00	0.000	0.000	0.000
2	-8.45	-0.10	0.00	0.000	0.000	-0.450
3	0.00	-38.82	0.00	0.000	0.000	0.000
4	0.00	0.00	0.00	0.000	0.000	0.000
5	8.45	-0.10	0.00	0.000	0.000	0.450

Joint Displacements for Combination d+l+w

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
1	0.00	0.00	0.00	0.000	0.000	0.000
2	-10.56	-0.09	0.00	0.000	0.000	-0.350
3	-3.19	-33.92	0.00	0.000	0.000	-0.020
4	0.00	0.00	0.00	0.000	0.000	0.000
5	4.18	-0.09	0.00	0.000	0.000	0.429

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Joint Displacements for Combination d+w

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
1	0.00	0.00	0.00	0.000	0.000	0.000
2	-5.23	-0.03	0.00	0.000	0.000	-0.065
3	-3.19	-9.40	0.00	0.000	0.000	-0.020
4	0.00	0.00	0.00	0.000	0.000	0.000
5	-1.16	-0.03	0.00	0.000	0.000	0.144

Support Reactions for Combination d+l

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	8.422	21.375	0.000	0.000	0.000	0.000
4	-8.422	21.375	0.000	0.000	0.000	0.000

Support Reactions for Combination d+l+w

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	9.962	20.358	0.000	0.000	0.000	0.000
4	-2.402	18.342	0.000	0.000	0.000	0.000

Support Reactions for Combination d+w

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	4.643	6.858	0.000	0.000	0.000	0.000
4	2.917	4.842	0.000	0.000	0.000	0.000

Member Effects for Combination d+l

Interval no	Interval pos. (m)	Axial force (kN)	Shear force (kN)		Moment effects (kNm)			
			Normal	Lateral	Torsion	Normal	Lateral	
<i>Member 1</i>								
0	0.000	21.375	-8.422	0.000	0.000	0.000	0.000	0.000
1	0.600	21.375	-8.422	0.000	0.000	0.000	-5.053	0.000
2	1.200	21.375	-8.422	0.000	0.000	0.000	-10.107	0.000
3	1.800	21.375	-8.422	0.000	0.000	0.000	-15.160	0.000
4	2.400	21.375	-8.422	0.000	0.000	0.000	-20.214	0.000
<i>Member 2</i>								
0	0.000	21.375	8.422	0.000	0.000	0.000	0.000	0.000
1	0.600	21.375	8.422	0.000	0.000	0.000	5.053	0.000
2	1.200	21.375	8.422	0.000	0.000	0.000	10.107	0.000
3	1.800	21.375	8.422	0.000	0.000	0.000	15.160	0.000
4	2.400	21.375	8.422	0.000	0.000	0.000	20.214	0.000
<i>Member 3</i>								
0	0.000	12.859	19.039	0.000	0.000	0.000	-20.214	0.000
1	1.152	11.699	13.822	0.000	0.000	0.000	-1.278	0.000
2	2.305	10.540	8.606	0.000	0.000	0.000	11.645	0.000
3	3.457	9.381	3.389	0.000	0.000	0.000	18.557	0.000
4	4.610	8.222	-1.827	0.000	0.000	0.000	19.458	0.000
<i>Member 4</i>								
0	0.000	12.859	19.039	0.000	0.000	0.000	-20.214	0.000
1	1.152	11.699	13.822	0.000	0.000	0.000	-1.278	0.000
2	2.305	10.540	8.606	0.000	0.000	0.000	11.645	0.000
3	3.457	9.381	3.389	0.000	0.000	0.000	18.557	0.000
4	4.610	8.222	-1.827	0.000	0.000	0.000	19.458	0.000

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Member Effects for Combination d+l+w

Interval no	Interval pos. (m)	Axial force (kN)	Shear force (kN)		Moment effects (kNm)		
			Normal	Lateral	Torsion	Normal	Lateral
<i>Member 1</i>							
0	0.000	20.358	-9.962	0.000	0.000	0.000	0.000
1	0.600	20.358	-9.692	0.000	0.000	-5.896	0.000
2	1.200	20.358	-9.422	0.000	0.000	-11.631	0.000
3	1.800	20.358	-9.152	0.000	0.000	-17.203	0.000
4	2.400	20.358	-8.882	0.000	0.000	-22.614	0.000
<i>Member 2</i>							
0	0.000	18.342	2.402	0.000	0.000	0.000	0.000
1	0.600	18.342	4.022	0.000	0.000	1.927	0.000
2	1.200	18.342	5.642	0.000	0.000	4.827	0.000
3	1.800	18.342	7.262	0.000	0.000	8.698	0.000
4	2.400	18.342	8.882	0.000	0.000	13.542	0.000
<i>Member 3</i>							
0	0.000	13.087	17.946	0.000	0.000	-22.614	0.000
1	1.152	12.038	13.224	0.000	0.000	-4.653	0.000
2	2.305	10.988	8.502	0.000	0.000	7.866	0.000
3	3.457	9.939	3.779	0.000	0.000	14.943	0.000
4	4.610	8.890	-0.943	0.000	0.000	16.577	0.000
<i>Member 4</i>							
0	0.000	12.650	15.978	0.000	0.000	-13.542	0.000
1	1.152	11.600	11.256	0.000	0.000	2.151	0.000
2	2.305	10.551	6.534	0.000	0.000	12.402	0.000
3	3.457	9.502	1.811	0.000	0.000	17.211	0.000
4	4.610	8.452	-2.911	0.000	0.000	16.577	0.000

Member Effects for Combination d+w

Interval no	Interval pos. (m)	Axial force (kN)	Shear force (kN)		Moment effects (kNm)		
			Normal	Lateral	Torsion	Normal	Lateral
<i>Member 1</i>							
0	0.000	6.858	-4.643	0.000	0.000	0.000	0.000
1	0.600	6.858	-4.373	0.000	0.000	-2.705	0.000
2	1.200	6.858	-4.103	0.000	0.000	-5.248	0.000
3	1.800	6.858	-3.833	0.000	0.000	-7.628	0.000
4	2.400	6.858	-3.563	0.000	0.000	-9.847	0.000
<i>Member 2</i>							
0	0.000	4.842	-2.917	0.000	0.000	0.000	0.000
1	0.600	4.842	-1.297	0.000	0.000	-1.264	0.000
2	1.200	4.842	0.323	0.000	0.000	-1.556	0.000
3	1.800	4.842	1.943	0.000	0.000	-0.877	0.000
4	2.400	4.842	3.563	0.000	0.000	0.775	0.000
<i>Member 3</i>							
0	0.000	4.966	5.922	0.000	0.000	-9.847	0.000
1	1.152	4.649	4.494	0.000	0.000	-3.845	0.000
2	2.305	4.331	3.066	0.000	0.000	0.511	0.000
3	3.457	4.014	1.639	0.000	0.000	3.222	0.000
4	4.610	3.697	0.211	0.000	0.000	4.288	0.000

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<i>Member 4</i>							
0	0.000	4.529	3.954	0.000	0.000	-0.775	0.000
1	1.152	4.211	2.526	0.000	0.000	2.959	0.000
2	2.305	3.894	1.098	0.000	0.000	5.047	0.000
3	3.457	3.577	-0.329	0.000	0.000	5.490	0.000
4	4.610	3.260	-1.757	0.000	0.000	4.288	0.000