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671/01

26 November 2018

Inigo Woolf
Treasurer
The Parish Church of St. John-at-Hampstead
Church Row
London
NW3 6UU

Dear Mr. Woolf

St. John-at-Hampstead, London

We note below our structural engineering comments on the proposal to install photovoltaic panels on the roof of the church. During our visit on the 14th of November we accessed the roof space above the nave by staircases under the eastern tower. We had a look at the structure and took some measurements of the timber structure to help in our appraisal. There was no lighting to provide safe access to the roof above the chancel.

St. John's is a Grade I listed church built in the middle of the 18th century. The church was extended eastwards twice in the 19th century. The basic structure comprises solid load bearing masonry walls and columns, and a timber roof. The masonry structure is likely to have brick corbel footings which bear onto the natural subsoil. The British Geological Survey map shows the area to be at the border between Bagshot Formation (sand) and Claygate Member (clay, silt, and sand). The LCC Bomb Damage Maps do not show any damage to the church during the World War II bombing.

The roof structure of the nave appears to be original and is formed by timber trusses spaced at approximately 4 metre centres. The trusses span between the main walls of the church with intermediate support on the internal columns. The trusses support the purlins and rafters which carry the roof finishes and ceiling joists that support the lath and plaster ceiling to the barrel vault above the nave and above the two aisles. The timber rafters are 150mm deep and 75mm wide at approximately 380mm spacing. The maximum span of the rafters is approximately 3.5m. A summary of a typical truss is on the attached drawing SK 1. Based on our limited access there are no obvious signs of any significant issues with the roof structure. We assume that the 19th century roof above the chancel will be a similar form of construction.

The proposal is to install one row of photovoltaic panels on the south-facing pitch of the nave roof, and two rows of photovoltaic panels on the south-facing pitch of the chancel roof. The manufacturer's information shown on two of the drawings you sent us (drawings No. A200 and A400) refers to "Perlight Solar Plus PLM-300-MB60". These panels are 1.84 metres by 0.992 metres on plan and each panel weighs 18 kgs. They are supported on metal rails that span above the roof finishes. According to the supplier, the rails are typically fixed at 1200 mm centres. At St. John's the rafters are approximately at 380mm centres which means the fixings will be to every third rafter.

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The attached calculations for the nave roof estimate the existing loads supported by the roof structure, the additional loads from the proposed panels and the existing and proposed bending moment and deflection in the rafters and purlins. Overall the increase in load is not significant and it is structurally feasible to install the panels above the nave as shown on the drawings. It is likely that there are no structural issues with installing panels above the chancel roof but recommend that lighting be provided so that we can re-visit to look at this area.

To limit the increase in loads onto the rafters we suggest that the fixings for the rails that support upper and lower panels are staggered on plan. It is also important that the rail fixings are into the centre of the rafters. This will require careful site measurements by the installer.

Please let us know if you have questions or comments.

Yours sincerely

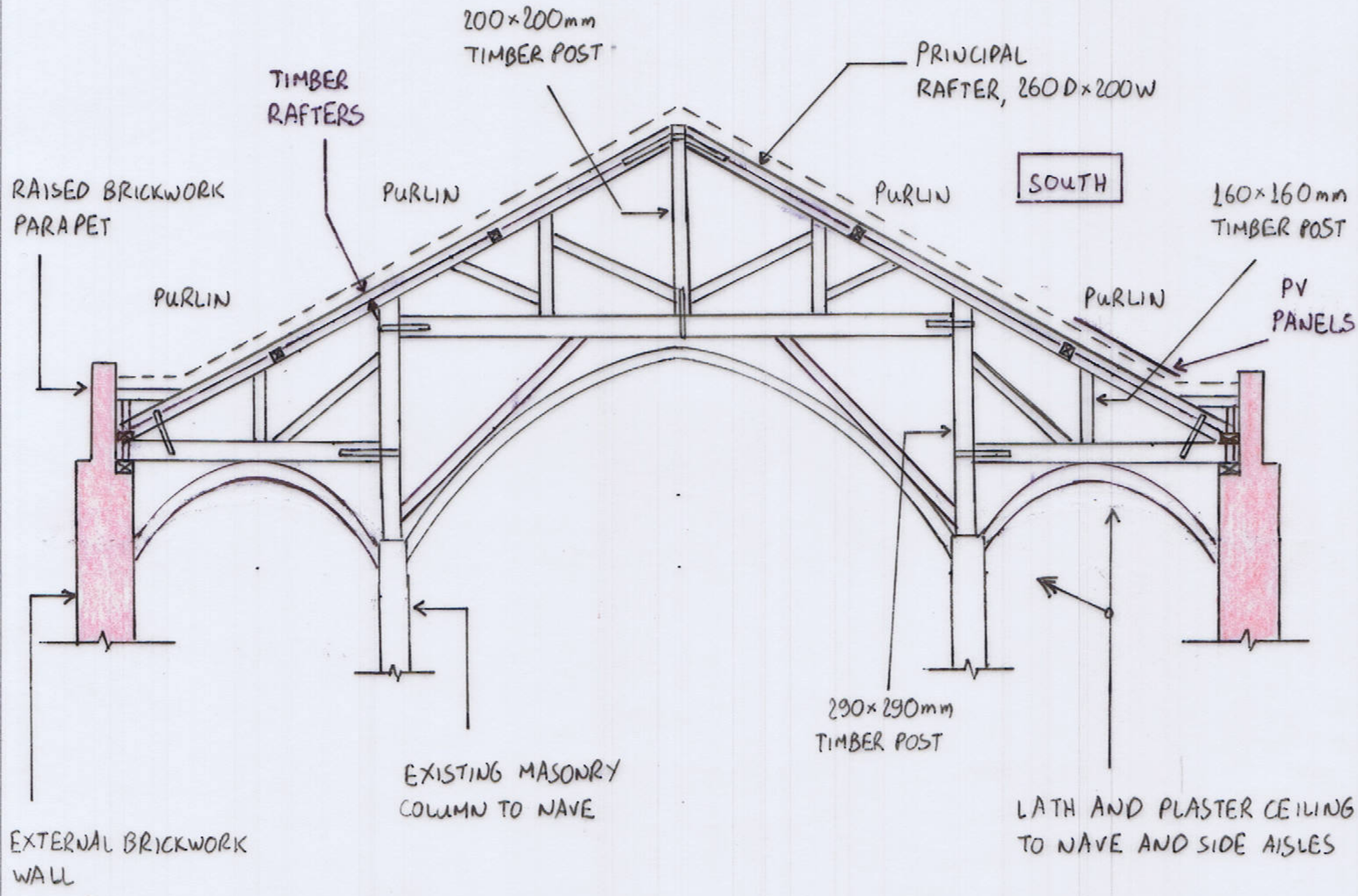
Dimitrios Velesiotis

1. View of church from the east

2. Interior view of church from the east

Stand Consulting Engineers	PROJECT	PROJECT NO.	DRAWING NO.	REV
52 Foundling Court T: 020 7278 6136 The Brunswick Centre W:www.standengineers.eu Marchmont Street London WC1N 1AN	ST. JOHN-AT-HAMPSTEAD	671	SK1	P1

TITLE	SCALE @ A4	DATE	DRAWN	AUTHORISED
NAVE ROOF - EXISTING STRUCTURE	AS NOTED	23.11.18	DV	ST



SECTION THROUGH ROOF STRUCTURE TO NAVE OF CHURCH

SCALE ≃ 1:100

Calculation Sheet

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Existing Roof Structure

Size of timber rafters: 150mm deep × 75mm wide @ 380mm c/c

Size of timber purlins: 160×160mm

Maximum span of rafters : L ≃ 3.5m

Maximum span of purlins : L = 3.0m (conservatively)

Loadings Overview

Dead Load (DL)	- Load from timber rafters:	0.14 KN/m²
	- Felt, say 10mm thick:	0.02 KN/m²
	- For thick Welsh slate:	0.48 KN/m²
Imposed Load (LL)	- For slope of appr. 30°:	0.6 KN/m²
	(+)	1.24 KN/m²

For the existing timber elements we consider C27 timber:

$\sigma b// = 10.0\ N/mm^2$ (bending parallel to grain)

$E_{mean} = 12300\ N/mm^2$ (mean modulus of elasticity)

$E_{min} = 8200\ N/mm^2$ (minimum modulus of elasticity)

Check for rafters - Existing case

Bending check (for L=3.5m span)

For 380mm spacing of rafters, UDL: $q = 1.24\ KN/m^2 \times 0.38m = 0.47\ KN/m$

(say, conservatively: $q = 0.5\ KN/m$)

Maximum Bending Moment: $BM_{max} = 0.47 \times \frac{3.5^2}{8} = 0.72\ KNm$

Bending Resistance: $M_R = 10000 \times 1.1 \times \left(\frac{300}{150}\right)^{0.11} \times 0.075 \times \frac{0.15^2}{6} = 3.34\ KNm$

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Therefore $MR > BMmax$ ✓ $\therefore$ OK

Deflection Check

Deflection: $\delta = \left(\frac{5}{384}\right) \times \frac{0.47 \times 3.5^4}{12.3\times10^6 \times 0.075 \times \frac{0.15^3}{12}} = 3.5\times10^{-3}\,m = 3.5mm$

✓

Allowable deflection: $\delta_p = 0.003 \times L = 10.5\,mm > \delta \;\therefore OK$

PV panels (DL)

The proposed PV panels Perlite Solar Plus PLM-300MB-60 Series weigh 18kg each and their size is 1840mm × 992mm (on plan) Therefore, additional UDL on a rafter (for worst case of two rows of PV panels):

$q = \frac{18kg \times 9.81\,m/s^2}{1000} \times \frac{1}{3.5m} = 0.05\,KN/m$

Check for rafters - Proposed Case

Total load: $q' = 0.5 + 0.05 = 0.55\,KN/m$

Bending Check

Maximum Bending Moment: $BM'max = 0.55 \times \frac{3.5^2}{8} = 0.84\,KNm > MR$ ✓ $\therefore OK$

$\frac{BM'max}{BMmax} = 1.167 \simeq 17\%$ bending moment increase

Deflection check

Deflection: $\delta' = \left(\frac{5}{384}\right) \times \frac{0.55 \times 3.5^4}{12.3\times10^6 \times 0.075 \times \frac{0.15^3}{12}} = 4.1\times10^{-3}\,m = 4.1\,mm < \delta_p$ ✓ $\therefore OK$

$\frac{\delta'}{\delta} = 1.16 = 16\%$ deflection increase

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Check for purlins - Existing Case

Span of timber purlins: $L = 3m$

Considering most onerous case for upperpurlin:

loaded width: 4.8m

Total loading: $q = 1.24 KN/m^2 \times 4.8m = 6 KN/m$

Bending Check

Maximum Bending Moment: $BMmax = 6 \times \frac{3^2}{8} = 6.75 KNm$

Bending Resistance: $M_R = 10000 \times 1.1 \times \left(\frac{300}{160}\right)^{0.11} \times 0.16 \times \frac{0.16^2}{6} = 8.0 KNm$

$M_R > BMmax$ ∴ OK, Bending check fulfilled

Check for purlins - Proposed Case

Additional load from PV panels, say 1No. row acting on the purlin (i.e. 3 No. panels over the 3m span):

$q_{PV} = 3 \times \frac{18 \times 9.81}{1000} \times \frac{1}{3} = 0.18 KN/m$

Total loading: $q' = q + q_{PV} = 6.18 KN/m$

Bending Check

Maximum Bending Moment: $BM'max = 6.18 \times \frac{3^2}{8} = 7 KNm < M_R$ ∴ OK

$\frac{BM'max}{BMmax} = 1.037 \simeq 3.7\%$ bending moment increase.