Combroke Parish Council

Consultation for Application 19/00361/FUL

At: The Little House, Combrook, CV35 9HP

Proposed: Enlargement of parking area, erection of retaining wall

Representation

The Parish Council is responding to a further amendment to drawings submitted showing another revision to the design of the retaining wall and calculations based on this design. A response is also made to comments from the Agent made in their email dated 11/07/19.

1. **Observations on "Structural Calculations rev B"**

1.1. These calculations have been assessed on the basis that they represent what the Structural Engineer was told has been built.

1.2. In view of the uncertainty surrounding this wall, the pro-bono opinion of a Chartered Civil Engineer has been obtained. They have visited the site from the public highway and viewed it from neighbouring properties; they have inspected the hillside above the site from the adjacent land; they have examined both versions of the structural calculations submitted; and have had the benefit of seeing photographic evidence[1] documenting the various stages of construction of the retaining wall.

1.3. The conclusions of the Engineer are as follows.

I. The structural calculations ignore the considerable impact of the slope of the hillside being retained by the wall.

II. The geotechnical data assumed for the soil properties is claimed to be "worst case" – it is not.

III. Re-calculation based on allowance for slope and more realistic soil properties indicates there is NO factor of safety.

IV. The calculations do not explore likely failure modes due to the terrain and soil type present at the site.

V. The photographic evidence clearly indicates that the design presented in both the Structural Engineer's sketch and the Architect's drawing is not "as built".

VI. The steel reinforcement has not been incorporated correctly and will not be effective as suggested by the calculations.

1.4. In view of the above, the Case Officer is urged to request further justification for the Structural Engineer's assertion that the design of the wall as built is adequate. It is felt that if the Structural Engineer considers the photographic evidence of construction, they may be less willing to accept the verbal information they have relied upon to date.

1.5. Detailed examination of the calculations presented in "Structural Calculations Rev B" are given in Appendix A and the photographs used are shown in Appendix B.

[1] as can be found submitted with representations published on the E-planning webpage for the application.

2. **Observations on revised drawing 412.42 Rev B**

2.1. This drawing is a revision of 412.42 Rev A which is in turn a revision of 412.42. All of these drawings relate to a wall that had already been built so this is the 3rd version of the same wall.

2.2. The chief difference between this version and those which preceded it is to show that the depth of the reinforced concrete base was in fact a minimum of 700mm deep rather than the 450mm shown in the previous 2 versions.

2.3. This change appears to have been made to be consistent with a lowering of the top surface of the concrete parking area relative to the top of the reinforced concrete base. This was an error that had been noted in the previous 2 versions. However the corresponding increase in the depth of the concrete base requires the base to have been cast in a trench below the excavated level on which the concrete parking area is laid.

2.4. Photographic evidence accumulated during the construction process and presented to the Parish Council simply does not support this. Instead, photographs in Appendix B show the views of timber shuttering, an estimated 450mm high, sitting on the excavated ground level with steel mesh reinforcement at that same level.

2.5. The Parish Council believes that what can be seen in the photographs is self evident and the revised drawing 412.42 RevB does not show the "as built" wall.

2.6. A further observation made from the photographs in Appendix B is that the vertical steel reinforcement is constructed from at least 2 and possibly 3 separate bars. The starter bar is located in the base and some appear to terminate below the level of the top of the base others just above. The final bar is not shown in these photographs but they are clearly seen in the photograph included in the Structural Engineer's letter dated 13th May 2019. The practice of splicing together reinforcing bars is perfectly acceptable but *only* if the length of overlap between 2 bars is a *minimum* of 40 x d where d is the diameter of the bar. In this case the bar is T20 which is 20mm diameter and so the minimum overlap distance must be 800mm. This is simply not achievable. Indeed some bars, as seen in the photographs, project only by the height of a single block, or 225mm, while the photograph in the Structural Engineer's letter of 13th May, shows a projection 2 blocks high which is still only 450mm.

2.7. The steel reinforcement does not therefore provide an adequate structural connection between the stem of the wall and its base. Exactly what bending moment the joint between stem and base is capable of resisting is unquantifiable, and therefore **the risk of the stem toppling from the base must be regarded as significant.**

3. **Observations on the comments made by the Agent in their email 11/07/19**

The Application Site

3.1. The location plan still does not indicate that the road, which is not owned by the applicant, is a public right of way.

The Proposal

3.2. It is noted that the revised proposal is based on a conversation with the Builder who built the wall 9 months ago and who has now provided details of dimensions which differ from those given shortly after the wall was built.

Structural Matters

3.3. These have been dealt with above.

Drainage

3.4. This response does not address any of the concerns raised in the Parish Council's previous representations. There is undoubtedly an additional requirement for surface water disposal arising from the new area of impermeable concrete parking area. There is also the effect of removing approximately 300 tonnes of hillside previously available to soak up water and attenuate flows. Whether the proposed drainage solution will enable the development to "*not result in any additional surface water flooding"* is completely unsupported.

3.5. In particular, it is noted from the proposed drainage layout shown in drawing 412.40F that all of the additional surface water from the development flows into a gulley at the southern end of a new ACO surface water channel drain in front of the house. From this point it appears that it is required either to flow within the channel drain, or it is discharged into a "field drain at the base of retaining wall" in front of the house. Neither of these options is feasible.

3.6. The ACO drain collecting surface water from the road surface must follow the slope of the road to collect the run-off across the road. The arrow on drawing 412.40F pointing north is presumably showing the anticipated direction of flow within the channel. However the topographical data shows a fall of 0.37m in the opposite direction requiring water to flow up hill.

3.7. The alternative route shown on the drawing is to discharge all of the captured water from the development into the field drain shown at the base of the retaining wall in front of the house. Since the front wall of the house blocks the natural percolation route to the stream to the west, the natural topography will allow excess water to filter down the slope to the south towards the adjacent row of listed cottages. This is unacceptable as it adds to the flood risk that is already present for those properties.

Trees

3.8. It is equally "unfortunate" that the applicant did not pursue the proper procedures to seek permission prior to commencement of the work. This would have allowed scrutiny of the implications of the work and amendments to be made to avoid the destruction of two mature trees, the building of an unsafe structure, and potential increase in flood risk to listed buildings. It would also have provided an opportunity to consider whether there are any public benefits in this development which outweigh the requirement, in line with National and Local Policy, to safeguard the adjacent Ancient Woodlands from the harmful impacts of such development.

4. Conclusion

For all of the reasons given above, the Parish Council continues to recommend refusal of this application.

25 July 2019

Appendix 1

Observations on "Structural Calculations Rev B" 10th June 2019

Page 1

The ground behind the wall is shown to be inclined but no angle of slope (α) is indicated.

The sketch shows the retained height to be 2.25m. The overall height of the wall (stem + base) is not shown. This is the height to which the active pressure (P_A) is applied. The height of the driveway above the base of the wall is not shown. This is the height on which the passive pressure acts.

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The soil properties are declared to be "unknown" and therefore parameters for soft clay are said to represent the "worst case" scenario.

However "worst case" parameters have NOT been used.

A more reasonable internal friction angle Ø is 20˚ (ref Geotechdata.info)

A more commonly used saturated density would be 19 kN/m^3 (ref StructX.com)

The coefficient of active pressure K_A is given as a soil property. It is in fact a derived value dependent on soil friction angle Ø and angle of slope α behind the retaining wall. To obtain the given value of 0.33 it was necessary to use the very high value of friction angle of 30˚ and to totally neglect the slope of the hillside behind the wall. See note 1.

From the topographical data provided in this application in drawing 412.39A, the slope immediately behind the retaining wall is approximately 20˚. See note 2.

A recalculation of K_A using Ø = 20˚ and α = 20˚gives a value for K_A of 1 which is **3 times** higher than the value used in the subsequent calculations. See note 3.

Page 3

Using K_A = 1 the total lateral load on the stem is 3 x 8.6 = 25.8 kN/m

This produces a shear force on the wall of 0.17 N/mm^2 **which exceeds 0.1 N/mm^2**

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The force causing sliding must be increased by *both* the increased K_A value *and* the increased value of saturated density for soft clay.

This produces a multiplying factor of 3 x 19/16 = 3.56 for the force causing sliding. So this increases to 13.4 x 3.56 = 47.7 kN/m

In addition to a force resisting sliding due to friction at the base, the calculations also show a force arising from the passive pressure exerted by concrete and the ground acting through the foundation block supporting the facing stone. No value for the coefficient of passive pressure K_P is given but a figure of 4 appears in the calculation. This figure depends wholly on friction angle Ø, and for the "worst case", condition of 20˚ it is the reciprocal of K_A and so is also equal to 1. The area over which the passive pressure acts is indeterminate from the drawing, but using the same data as that used in the calculations, the force from passive pressure will only be half the value shown.

Re-calculating using K_p = 1 yields a lower force opposing sliding of 25.4 kN/m. See note 4

The factor of safety is therefore 25.4/47.7 = 0.53.

This means, for the design presented, using the method prescribed, that using more realistic data, there is **NO factor of safety and the wall will slide**

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The overturning condition is stated to be determined for the wall and its base.

Moments are said to be taken about the bottom front corner of the base.

But the height used to determine the overturning moment is NOT measured from the bottom of the base. This dimension is not specified. Instead the "retained height" of 2.25m is used which is an **underestimate** of the total height of the wall and its base. It is the total height which determines the area on which the active pressure is applied. Scaling from the drawing the total height is 2.65m. Using this figure to determine the height of the centre of pressure coupled with the corrected applied force, the overturning moment becomes

$$47.7 \times 2.65/3 = 42.1 \text{ kNm/m}$$

Using the given figure for the moment arising from the wall's own mass, the factor of safety is 22.9/42.1 = 0.54

This figure is totally unacceptable. Safety relies entirely on the hope that worst case conditions cannot apply.

Comment.

Due to the steepness of the slope behind the wall, the extent of the undrained catchment area above, the properties of the clay soil, and the history of ground instability in the area, a global stability analysis employing slip circle failure considerations may be indicated.

Note 1 Coefficient of active pressure K_A (Rankine)

$$K_A = \frac{\cos \alpha - \sqrt{(\cos^2 \alpha - \cos^2 \varnothing)}}{\cos \alpha + \sqrt{(\cos^2 \alpha - \cos^2 \varnothing)}}$$

Using $\alpha = 0^\circ$ and $\varnothing = 30^\circ$ yields $K_A = 0.33$ which is the figure used in the Structural Engineer's calculation. This confirms that the angle of the slope used was zero, i.e. the hillside behind the wall was neglected.

Note 2 Angle of slope behind retaining wall from topographic data

Drawing 412.39A of the existing parking area contains topographic data. Steps are shown towards the northern boundary and the base of the steps approximately coincides with the back of the new retaining wall. The height shown at the bottom of the steps is 102.66m and that at the top of the steps is 104.75m, a rise of 2.09m. The horizontal distance between these 2 points is scaled from the drawing and is approximately 4.8m.

$$\alpha = \tan^{-1} 2.09/4.8$$
$$= \tan^{-1} 0.435$$
$$= 23.5^\circ$$

From the geotechnical data, using other pairs of points would suggest this is a maximum. The average slope should be used and this is estimated to be 20˚

Note 3 Calculation of corrected value for K_A using $\alpha = 20^\circ$ and $\varnothing = 20^\circ$

$$K_A = \frac{\cos 20^\circ - \sqrt{(\cos^2 20^\circ - \cos^2 20^\circ)}}{\cos 20^\circ + \sqrt{(\cos^2 20^\circ - \cos^2 20^\circ)}}$$
$$= 1$$

Note 4 Calculation of K_P and force opposing sliding

From Rankine,

$$K_p = \frac{\cos \alpha + \sqrt{(\cos^2 \alpha - \cos^2 \varnothing)}}{\cos \alpha - \sqrt{(\cos^2 \alpha - \cos^2 \varnothing)}}$$

using $\alpha = 20^\circ$ and $\varnothing = 20^\circ$

$$K_P = 1$$

Force resisting sliding from concrete = 1 x 0.1 x 24 = 2.4 kN/m

Force resisting sliding from ground below base = 1 x 0.25 x 16 = 4kN/m

Total force opposing sliding = 19 + 2.4 + 4 = 25.4 kN/m

Appendix 2

Fig 1.

Fig 2.