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Introduction

The purpose of this note is to demonstrate the type of calculations which have been carried out to assess the robustness of Hartopp and Lannoy Points against disproportionate collapse. The note supplements and should be read in conjunction with our report “*Hartopp Point & Lannoy Point – Structural Assessment*” dated 12 February 2019, and has been written specifically in response to a resident query as requested by London Borough of Hammersmith and Fulham.

Layout and dimensions

Hartopp Point and Lannoy Point have identical superstructures. The tower blocks were constructed using a precast concrete Large Panel System (LPS), where the panels were built in a factory and assembled on site. The floor slabs generally span one-way onto the internal cross-walls and the outer flank walls.

The approximate floor plan of one residential block can be seen in Figure 1. Floor slab panels are coloured according to their span length. There are additional thin concrete partitions supported by the floor slabs at each level which are not considered to be part of the main building structure and have been omitted in Figure 1. There are one and three bedroom flats up to the 4th floor, above which there are two bedroom flats. Floor plans vary slightly between one, two and three bedroom flats.

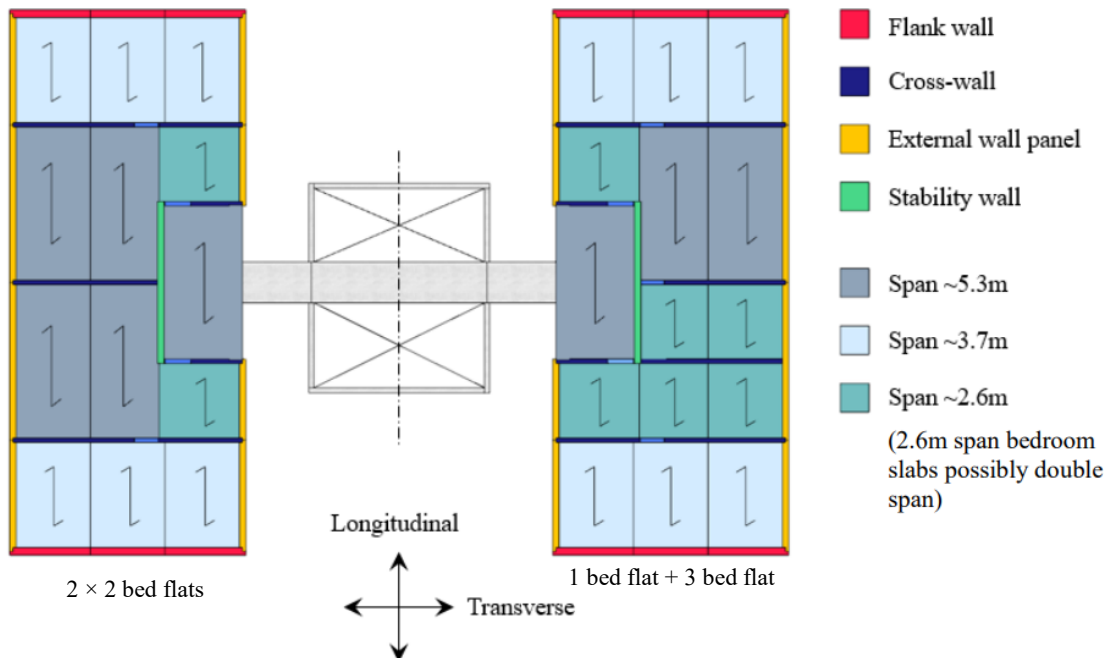


Figure 1 - Approximate floor plan of each block, illustrating the clear span dimensions. For illustrative purposes, a one and three bedroom flat layout is shown on the same floor as a 2 x two bedroom flat layout.

The buildings have 14 storeys and the height between each storey is approximately 2.6m. The floorplan dimensions are shown in Figure 2.

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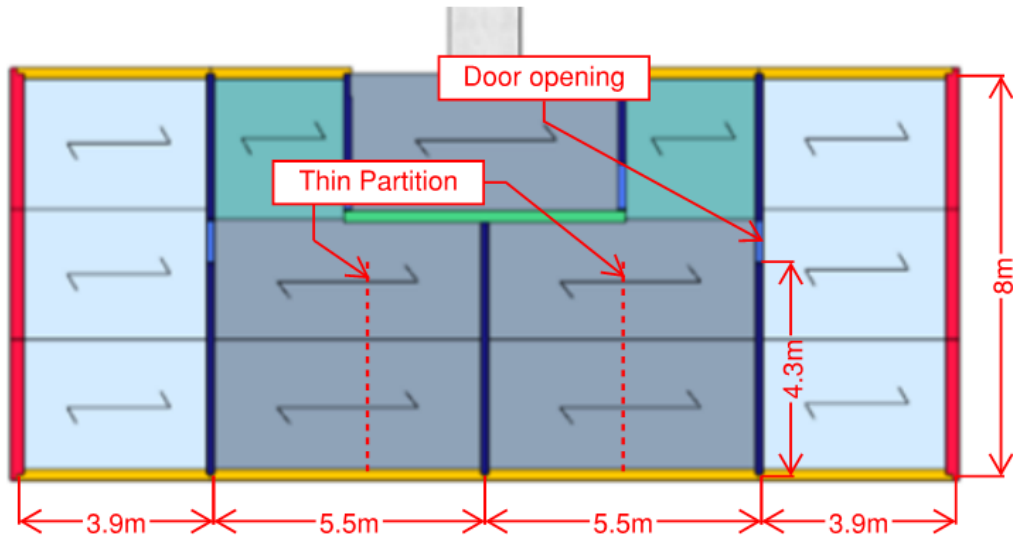


Figure 2 – Floor plan with approximate dimensions of the buildings.

Example wall

For this calculation we will focus on one of the internal cross walls. This has been highlighted in Figure 3.

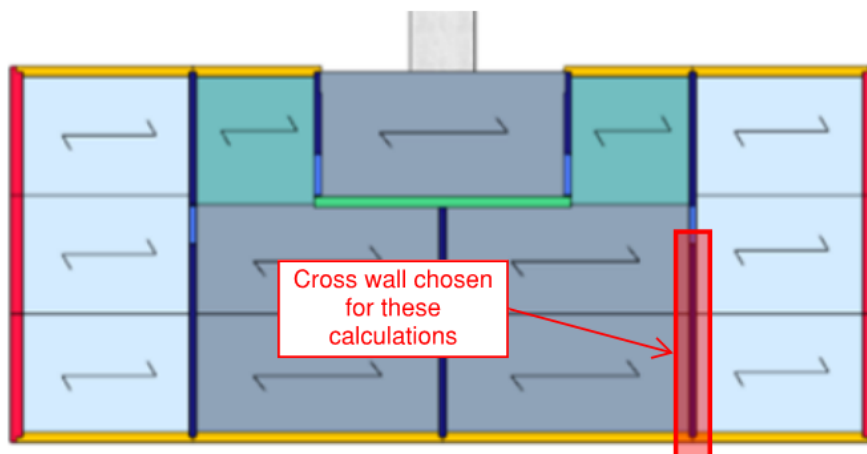


Figure 3 – Cross wall selected for these calculations.

Similar calculations have been carried out for the other elements in the buildings, such as the flank walls and the floor slabs. However, these are more complex and require more technical engineering knowledge. The purpose of this note is to demonstrate the type of calculations which have been carried out.

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Robustness requirements

As discussed in our report dated 12 February 2019, the BRE “*Handbook for the structural appraisal of Large Panel System (LPS) dwelling blocks for accidental loads*” clearly defines three assessment criteria. If the building can be proven to satisfy any one of the three criteria, then it is considered to satisfy requirement A3 of the Building Regulations (which is the requirement to avoid disproportionate collapse) in accordance with Approved Document A. The following is an extract from the BRE assessment digest:

“An LPS dwelling block exceeding four storeys in height (i.e. five storeys or higher) will be considered to satisfy Requirement A3 of the Building Regulations if it meets one of the following criteria:

LPS Criterion 1: There is adequate provision of horizontal and vertical ties to comply with the current requirements for Class 2b buildings as set down in the codes and standards quoted in Approved Document A – Structure as meeting the requirements set down in the Building Regulations.

LPS Criterion 2: An adequate collapse resistance can be demonstrated for the foreseeable accidental loads and actions [which is defined as 34kPa for a block with piped gas or 17kPa for a block without piped gas]

LPS Criterion 3: Alternative paths of support that can be mobilised to carry the load, assuming the removal of a critical section of the load bearing wall in the manner defined for Class 2B buildings in Approved Document A – Structure or alternatively assuming the removal of adjacent floor slabs (taking the floor slabs bearing on one side wall at a time) providing lateral stability to the critical section of the load bearing wall being considered.”

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Structural details

There are many different structural details used throughout the buildings in different locations. In this calculation we present two of them which are required for our assessment of the cross wall – the floor slab to cross wall connection (for assessing the robustness of the cross wall’s connections) and the floor build-up (for calculating the weight of a floor).

Floor slab to cross wall connection

The floor slab to cross wall connection is shown in Figure 4. The lower cross wall was installed first, and the precast hollow-core concrete floor slabs were then placed on top of it. Levelling bolts (not shown) were then used to position the upper cross wall. Once in place, the gap between the upper cross wall and the floor slabs was infilled with what is assumed to be dry pack. Screed was then cast on top of the floor slabs.

One or two longitudinal bars are generally present in the connection, but there is no protruding reinforcement from any of the wall or floor slab units to connect the longitudinal bars to the floor slabs.

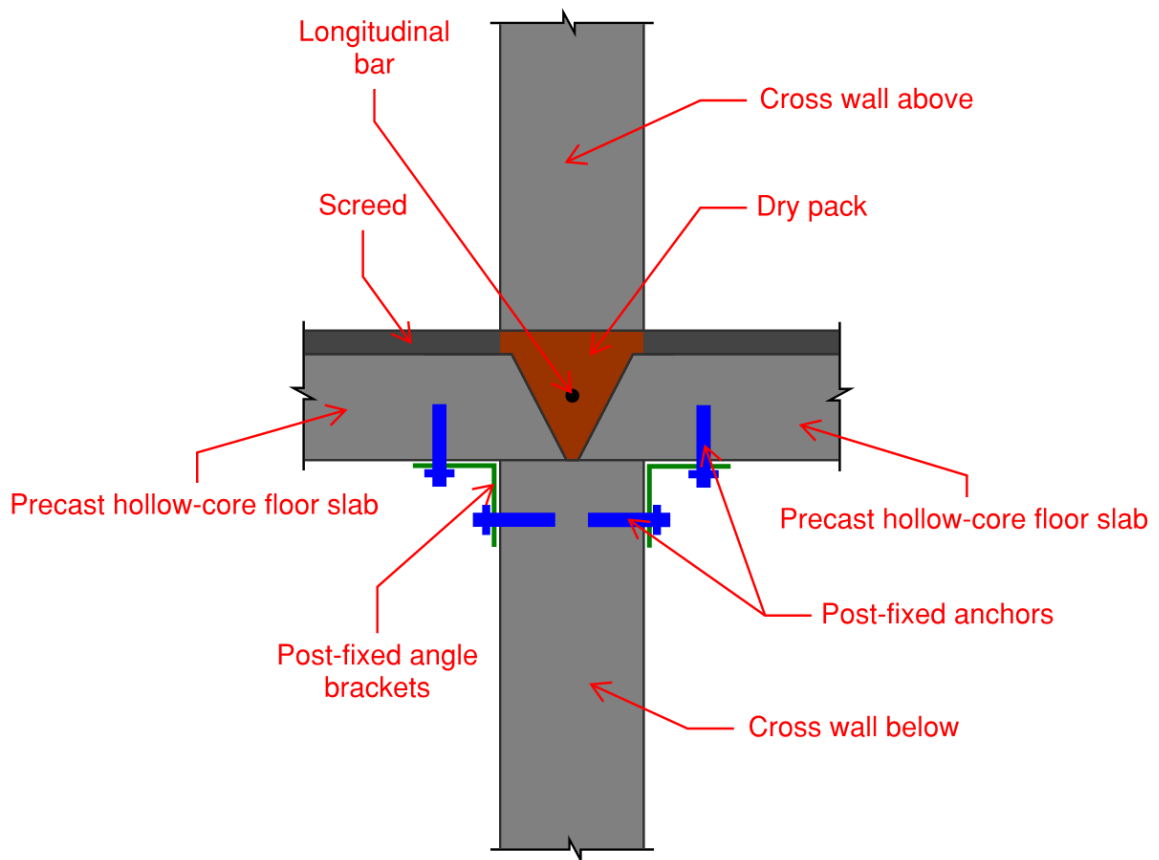


Figure 4 – The floor slab to cross wall connection.

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Following the recommendations after Ronan Point, post-fixed steel strengthening brackets were added as shown in Figure 4, held in place with an early type of wedge anchor (believed to be ¾ inch Phillips Red Head Concrete Anchors). One of these anchors can be seen in Figure 5 (assembled) and Figure 6 (disassembled).

Unlike modern wedge anchors where the end splays out as the anchor is tightened, these were installed by placing the anchor in a hole and hammering on the outer end, the inner end being forced out horizontally by a plug.



Figure 5 – One of the post-fixed anchors (assembled).



Figure 6 – One of the post-fixed anchors (disassembled).

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Floor build-up

The build-up of a typical floor in the buildings is shown in Figure 7. Reinforcement has not been shown.

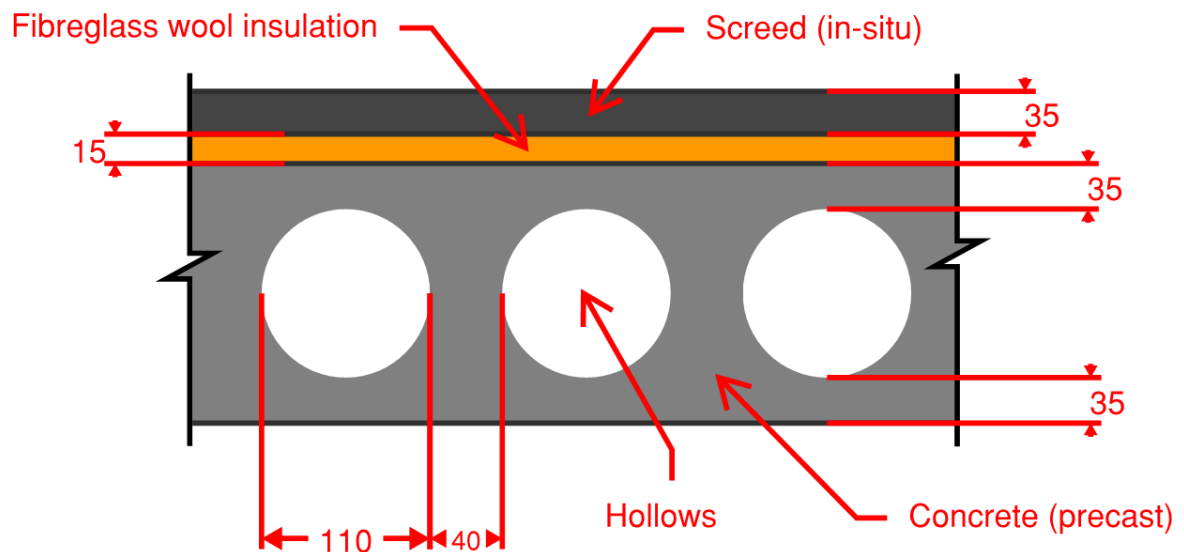


Figure 7 – Cross section of hollow-core floor (all dimensions in mm).

The exact thickness of the screed was noted to vary throughout the building, but 35mm was found to be a representative average value appropriate for these calculations

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Loading

In accordance with Eurocode (EN1990 eqn 6.11b), the following load combination should be used for accidental cases:

$$\left(\sum_{j \geq 1} G_{k,j} \right) \text{ " + " } A_d \text{ " + " } \psi_{1,1} Q_{k,1} \text{ " + " } \left(\sum_{i > 1} \psi_{2,i} Q_{k,i} \right)$$

Where:

- G_k is the characteristic value of the dead load ($\sum_{j \geq 1} G_{k,j}$ just means “add up all the dead loads”);
- A_d is the accidental action (such as the 17kPa);
- $\psi_{1,1}$ is the relevant ψ_1 factor for the main variable action;
- $Q_{k,1}$ is the characteristic value of the main variable action;
- $\sum_{i > 1} \psi_{2,i} Q_{k,i}$ means “apply the relevant ψ_2 factor to the characteristic value of all the other variable actions and add them up”.

This in effect means:

- Use unfactored dead loads;
- Do not factor the accidental action;
- Use the ψ_1 reduction factor on the live load (no other variable actions considered).

The dead load comes from the building’s concrete walls and floors. This is generally lightly reinforced, and so we will take a concrete density of 24kN/m³. We will also use this value for the screed.

The dead load pressure g_k due to the floor self-weight can be calculated by considering a 150mm wide strip (including one circular hollow) as follows:

$$g_k = \frac{150\text{mm} \times 180\text{mm} - \pi \times (110\text{mm})^2 / 4 + 35\text{mm} \times 150\text{mm}}{150\text{mm}} \times 24\text{kN/m}^3 = 3.6\text{kPa}$$

The live load comes from people and furnishings inside the building. The actual value of this is hard to predict, and so engineers will normally take a conservative estimate. In our case, the live load is favourable in some cases (extra load improves the calculations) and unfavourable in others (extra load makes the calculations worse), and so we will use a realistic live load pressure value of $q_k = 1.0\text{kPa}$. For simplicity, we will keep this value constant throughout. The ψ_1 reduction factor is 0.5.

The values of g_k , q_k and ψ_1 derived above will be used in the following sections in our assessment of the robustness of the cross wall.

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Assessment against LPS Criterion 1 (adequate provision of ties)

As per clause 9.10.1(2) of Eurocode 2, there are three types of tying required in an LPS building:

- *Internal ties* connecting floor panels together and also connecting floor panels to wall panels;
- *Peripheral ties* connecting floor panels to each other around the edge of a whole building floor;
- *Vertical ties* connecting wall panels to wall panels above and below (Eurocode 2 notes that these are particularly important in panel buildings).

We will assess whether the cross wall connection shown in Figure 4 has each of these in turn.

Internal ties

The UK National Annex for Eurocode 2-1-1 states that the required strength for internal ties is:

$$F_{tie,int} = \frac{q_k + g_k}{7.5} \cdot \frac{l_r}{5} \cdot F_t \geq F_t$$

where:

- l_r is the greater of the distances between centres of adjacent walls supporting the adjacent spans in the direction of the tie under consideration in metres (5.5m for our cross wall);
- $F_t = (20 + 4n_0) \leq 60$ (in kN/m)
- n_0 is the number of floors including the ground floor (14 for Hartopp and Lannoy);
- q_k and g_k are the live load and dead load pressures respectively in kPa, with values as calculated in the *loading* section above;
- $F_{tie,int}$ is in kN/m

Substituting in our values, we get:

$$F_t = \min \{ 20 + 4 \times 14 ; 60\text{kN/m} \} = \min \{ 76\text{kN/m} ; 60\text{kN/m} \} = 60\text{kN/m}$$

$$F_{tie,int} = \max \left\{ \frac{1\text{kPa} + 3.6\text{kPa}}{7.5} \cdot \frac{5.5\text{m}}{5} \cdot 60\text{kN/m} ; 60\text{kN/m} \right\}$$

$$= \max \{ 40\text{kN/m} ; 60\text{kN/m} \} = 60\text{kN/m}$$

The only possible way of transferring this tensile force between adjacent floor slabs is via the post-fixed steel angle brackets. This would require putting the anchors which connect the brackets to the walls into tension. There is little information about these old wedge anchors, and their capacity is likely to be highly variable. Therefore, to obtain an estimate of their strength, current commercially available wedge anchors were investigated for comparison.

Calculation Sheet

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For example, HILTI's M20 HSA Expansion anchor has a design tensile resistance of 21.9kN when it has an embedment depth of 75mm. The anchors are at approximately 650mm centres, and so this would give a tying resistance of 34kN/m, significantly less than the 60kN/m required. In practice, the anchors installed on Hartopp and Lannoy are likely to have a much lower tensile strength because the anchors are poorly designed and the method of installation is highly workmanship dependent. Therefore, the actual tying resistance is likely to be much lower.

In conclusion, the building does not have sufficient internal ties.

Peripheral ties

No peripheral ties were found in our intrusive investigations.

Vertical ties

No vertical ties were found in our intrusive investigations.

Conclusion

The cross wall does not meet LPS Criterion 1.

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Assessment against LPS Criterion 2 (ability to resist 17kPa pressure)

The cross wall is unreinforced, and so its “moment” or “arching” capacity is directly related to the amount of axial force in it. The full check to be carried out is given in equation 12.2 of Eurocode 2, but a simplified method is to make sure that the axial force remains inside the section under an applied loading.

N.B. In the following calculation we will work with values of moment and axial force per unit length of cross wall (i.e. in kNm/m and kN/m).

For a pressure of 17kPa, a wall height of 2.6m, and assuming that the wall has little rotation capacity at top and bottom (likely, given the lack of continuity reinforcement and poor quality of drypack found in our investigations), global equilibrium gives an effective midheight moment of:

$$M = \frac{pl^2}{8} = \frac{17\text{kPa} \times (2.6\text{m})^2}{8} = 14\text{kNm/m}$$

The cross wall has a thickness $t = 150\text{mm}$, and so the minimum axial force required in the wall for the wall to have a high enough capacity to resist a pressure of 17kPa can be calculated as:

$$N_{min} = \frac{M}{t/2} = \frac{14\text{kNm/m}}{0.15\text{m} / 2} = 190\text{kN/m}$$

(The $t / 2$ assumes that the axial force N is applied at the edge of the wall, giving the maximum possible capacity. For simplicity we will ignore local crushing failure.)

The cross wall supports a 5.5m floor span on one side and a 3.9m floor span on the other side. Half of the weight of each span will end up in the cross wall. Therefore, on each floor, the cross wall receives an axial load of:

$$0.5 \times (5.5\text{m} + 3.9\text{m}) \times (3.6\text{kPa} + 0.5 \times 1\text{kPa}) = 19.3\text{kN/m}$$

There is also dead load building up on each floor due to the dead weight of the cross wall itself. This is 150mm thick and approximately 2.6m high, and so between each storey the dead load in the wall increases by:

$$2.6\text{m} \times 0.15\text{m} \times 24\text{kN/m}^3 = 9.4\text{kN/m}$$

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The total axial load received by the cross wall on each storey is therefore approximately 29kN/m. The number of storeys required to build up sufficient axial force in the wall to resist a pressure of 17kPa can therefore be calculated as follows:

$$\frac{190\text{kN/m}}{29\text{kN/m}} = 6.5 \text{ floors}$$

This means that a cross wall on a particular floor needs the weight of at least 6 storeys above it in order to have sufficient capacity to resist the 17kPa. Therefore, as stated in the report, the cross walls for the top 6 floors (i.e. from level 8 upwards) do not meet LPS Criterion 2.

This calculation has made a number of simplifications (which are sometimes favourable and sometimes unfavourable). However, accounting for these does not change the result.

Conclusion

The cross walls on the 8th floor and above do not meet LPS Criterion 2.

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Assessment against LPS Criterion 3 (provision of alternative load path)

As shown above, the cross wall is unable to withstand the 17kPa from the 8th floor upwards. We are therefore required to consider what happens if we remove the cross wall on the 8th floor (the panel length is less than the maximum length of critical $2.25 \times \text{storey height}$ specified in Approved Document A of the building regulations).

By inspection, if we remove this wall, there are no alternative loadpaths to mobilise:

- It is not possible to mobilise catenary action because the floor slabs have insufficient tying anchorage at their ends (as shown, in effect, when checking LPS Criterion 1);
- There are no other structural walls joining this one at right angles;
- The façade panels are hung off the cross walls and so cannot support the cross wall;
- The cross wall is separated from the next section of the cross wall by a doorway (refer Figure 2);
- The thin partition walls are only 50mm thick and so by inspection have insufficient capacity.

Therefore, removal of this wall would result in collapse of all the above cross walls and floor slabs as shown in Figure 8. It is likely that it would also pull down the flank wall with it.

This is deemed to be a “disproportionate collapse” because the collapse extends further than the immediately adjacent storeys. It would also produce a significant amount of debris which the 8th floor slab would be unable to support, and so it is likely to cause progressive collapse of all the lower floors too.

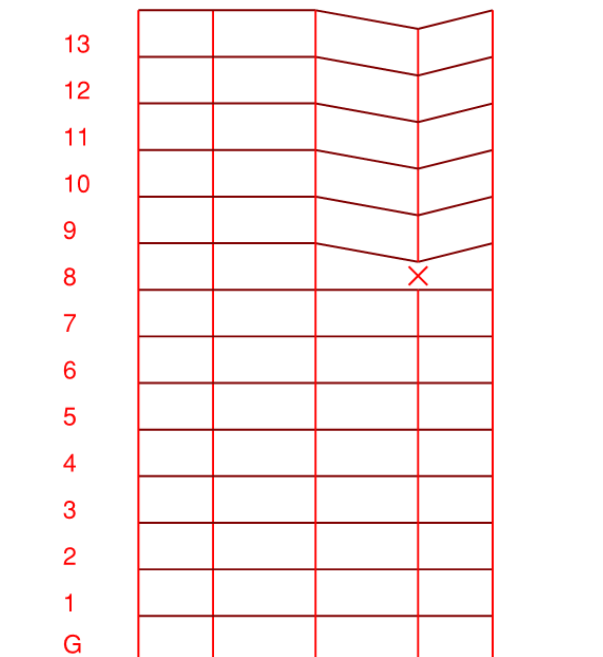


Figure 8 – Collapse mechanism resulting from notional removal of a cross wall on the 8th floor.

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Conclusion

The cross walls on or above the 8th floor do not meet the requirements for robustness against disproportionate collapse. Similar calculations have been carried out on other walls and floor slabs to show that they also have insufficient robustness. The results are summarised in our report.

We therefore conclude that the buildings do not comply with the recommendations for the prevention of “disproportionate collapse” in the 2012 guidance produced by the Building Research Establishment (BRE) and the Ministry of Housing, Communities & Local Government. This means that an accidental extreme event could lead to the collapse of a disproportionately large part of the building. Based on this, we have made a number of recommendations in our report dated 12 February 2019.