MetroLink

Transport Infrastructure Ireland

Aercap House Enhanced Building Damage Assessment Report

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Document history and status

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P01	16/03/24	First issue	PQ	MB	JK	PB
P02	20/03/24	Second issue – Figure 11 frame added	PQ	MB	JK	PB

Contents

1. Enhanced Building Assessment Methodology

TII have used an enhanced Buildings Assessment Methodology based on RIBA Stages 2 and 3 as set out in Section 4.2 of the CIRIA Guidance document C796 (London, 2021) for selected non masonry buildings along the MetroLink alignment to further explore the settlement impacts on these building types.

The methodology is based on the greenfield settlement displacement profile described in the building damage assessment in Appendix 5.17 of the EIAR. For this location, Aercap House, building volume loss (VL) is 0.5% and the K value is 0.4.

The tunnel at the location of Aercap House is wholly within the limestone rock and the Aercap building is also founded on the limestone rock.

This is an enhanced Phase 2 assessment which considers the specific building stiffness and therefore allows the settlement impact on the Aercap building to be calculated with more refinement. The subsequent Phase 3 assessment will be undertaken incorporating less conservative parameters.

2. Introduction

This document outlines the results of the enhanced Phase 2 assessment, which has been carried out in accordance with the guidance and methodology presented in the CIRIA C796 guide.

Aercap House is a 6 story concrete framed building with two basement levels. There is a secant piled wall around the basement which are toed into the rock 6.5m above the crown of the tunnel. The AerCap building is founded on competent limestone, the excavation was continued until this was encountered and any soft areas excavated were filled with mass concrete infill up to base slab level, the limestone is very stiff.

The results of the Phase 2 greenfield settlement as presented by EIAR Appendix 5.17 at the lower basement level of the Aercap building indicate a max slope of 1 in 648 and a max settlement of 18.5mm.

The greenfield settlement calculated (as presented by EIAR Appendix 5.17) has then been modified (as presented by this document), using a method to take account of the behavior of the Aercap House structure in response to ground movements generated by construction of the MetroLink tunnel.

The guide provides options which are selected for the particular building under consideration in accordance with the methodology set out in the CIRIA C796 guide. In this report, the rationale for the selection of the appropriate methodology and the results are presented.

3. Building Assessment

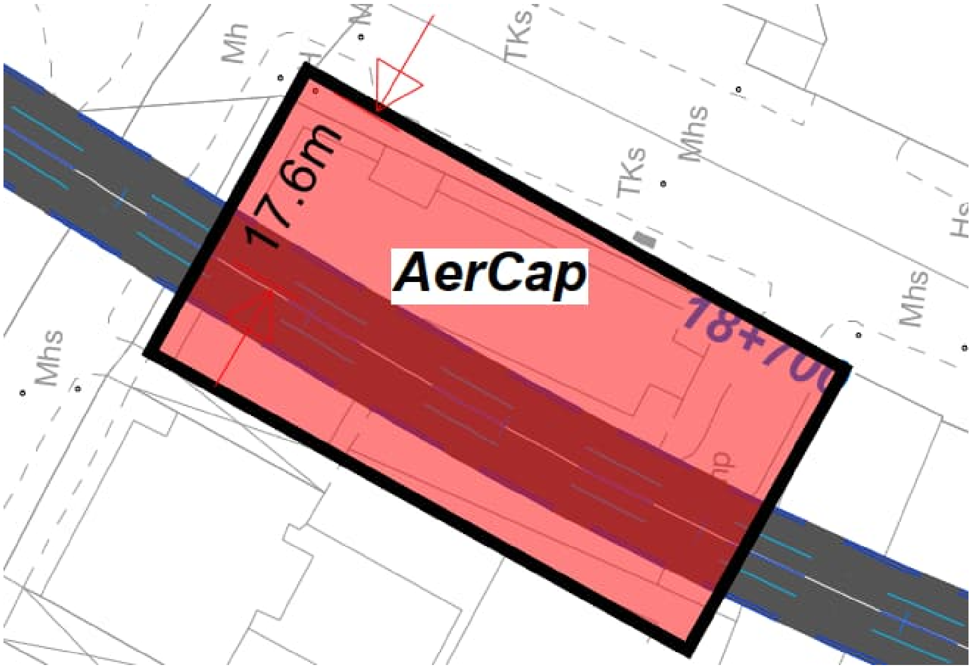

Figure 1 Footprint of Aercap House in relation to the MetroLink horizontal alignment

A typical cross-section of AerCap House was selected for analysis, as height is generally unfavourable for damage assessment, a taller building will experience more tensile strain for the same settlement, therefore a full height cross-section was selected for analysis.

The tunnel axis level was extracted from the alignment drawing and the volume loss and 'K' parameters have been kept at 0.5% and 0.4 as per the Phase 2. The Limestone stiffness was taken as 1600MPa.

All building levels, spans and structural element thicknesses were taken from the drawings provided by the building owner.

4. Building Stiffness

In accordance with CIRIA C796 guidance, there are four methods available to assess the effect of building stiffness on settlement and ground movement. As not all are suitable for every structure, the most appropriate method was selected.

Table 4.1 Suitability of various relative stiffness methods for typical foundation types

Method	Suitability for vertical ground movement modification			Suitability for horizontal ground movement modification		
	Shallow footings	*Raft*	*Piles*	*Shallow footings*	*Raft*	*Piles*
Potts and Addenbrooke (1997)	Yes	Yes	No	No	Yes	No
Franzius *et al* (2006)	Yes	Yes	No	No	Yes	No
Goh and Mair (2014)	No	No	No	Yes	No	No
Franza *et al* (2017)	No	No	Yes	No	No	Yes

Figure 2 Table 4.1 from CIRIA C796

For Aercap House the method proposed by Franza was not considered, as it is a method for discrete piles which are not present in the structure.

The Goh and Mair method is specifically for discrete footing elements and is not capable of assessing the impact on vertical ground movement and deflection ratio. This was not considered appropriate for the Aercap House building.

The method proposed by Franzius includes a building length factor which requires a constant section. This is not appropriate for AerCap House as the building section varies along the line of the tunnel, as shown Figure 3.

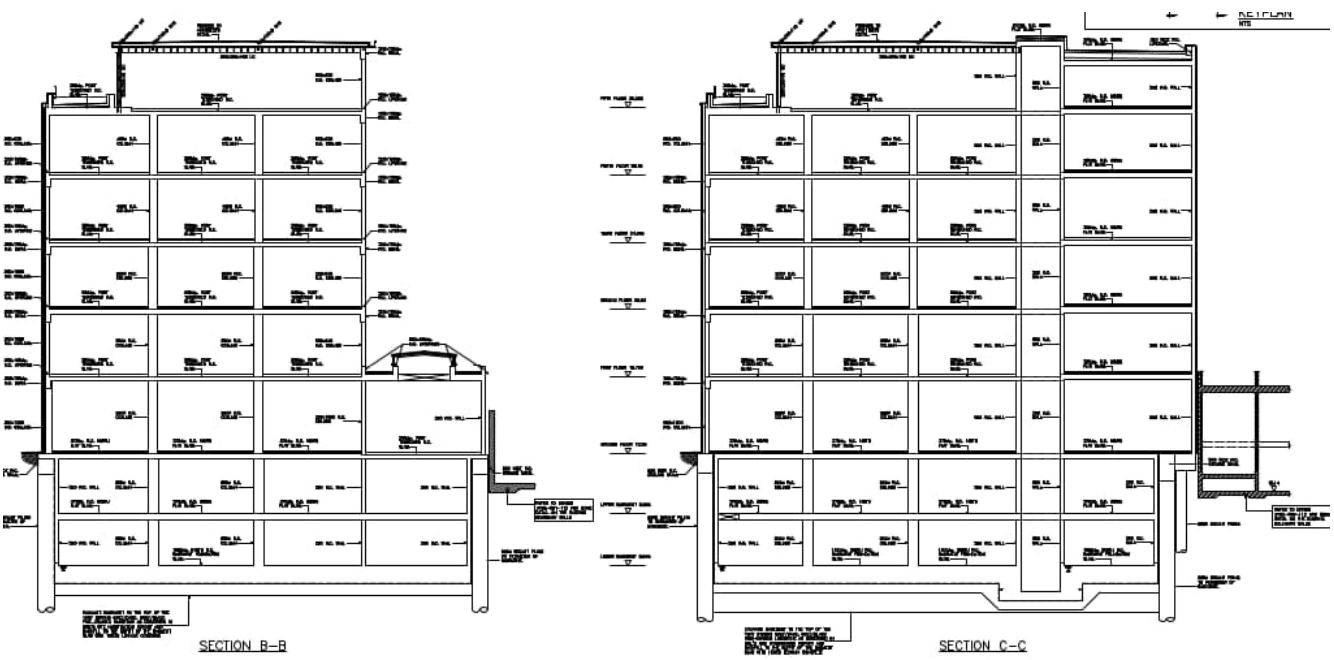

Figure 3 Arecap House sections. (extracted from 3762-C-202-1 Section B-B and C-C through building)

Potts & Addenbrooke has been selected over Franzius as it is more appropriate given the structural form of the AerCap building. The Potts & Addenbrooke methodology does not include a length factor as it accounts for relative stiffness by way of the building width alone.

Method 2 from Table 4.2 was selected. This was considered most appropriate as it is described as the upper bound solution which generates the higher estimate stiffness. This would accurately represent the stiffness of the interaction between the very stiff Limestone rock, mass concrete and the base slab. There is no soft ground which would alter the movement, thus a high stiffness approach is closest to expected actual behaviour.

As the Aercap building can be most appropriately represented as shallow pads foundation the horizontal modification is not used. This in line with CIRIA Table 4.1, which states such modifications are only appropriate for raft/spread footing foundations.

The Potts and Addenbrooke method produces a relative stiffness factor. This is then fed into provided design curves and modification factors are read off. The curves vary with the e/B ratio - a measure of how eccentric the tunnel alignment is relative to the building where 'e' is the distance from the tunnel axis line to the centre line of the building and 'B' is the building width. A value of 0.17 was determined for the AerCap building based on the plan in Figure 1 and the Deeper vertical alignment drawing. The design curves and the selected values are shown below;

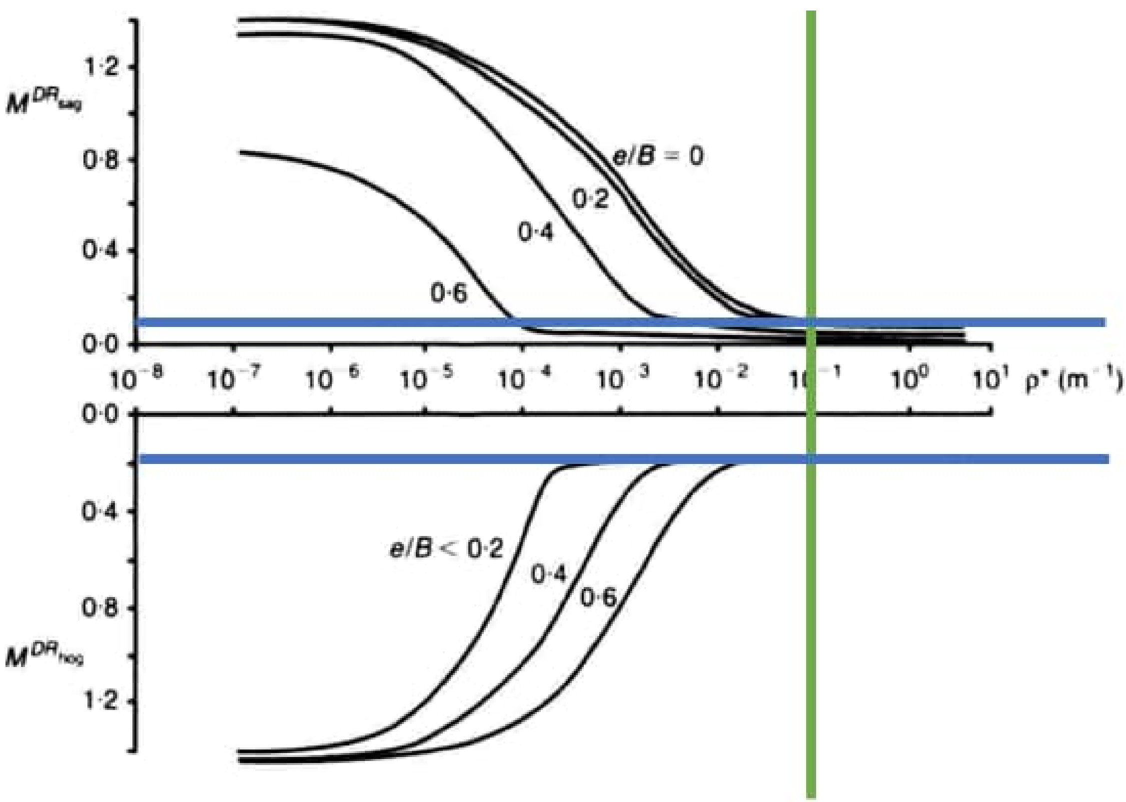

Fig. 16. Design curves for modification factors for deflection ratio

Figure 4 P&A Design Curve with Arecap Relative Stiffness marked on (green line)

The factors were applied as per the CIRIA method outlined in Figure A1.7 reproduced below.

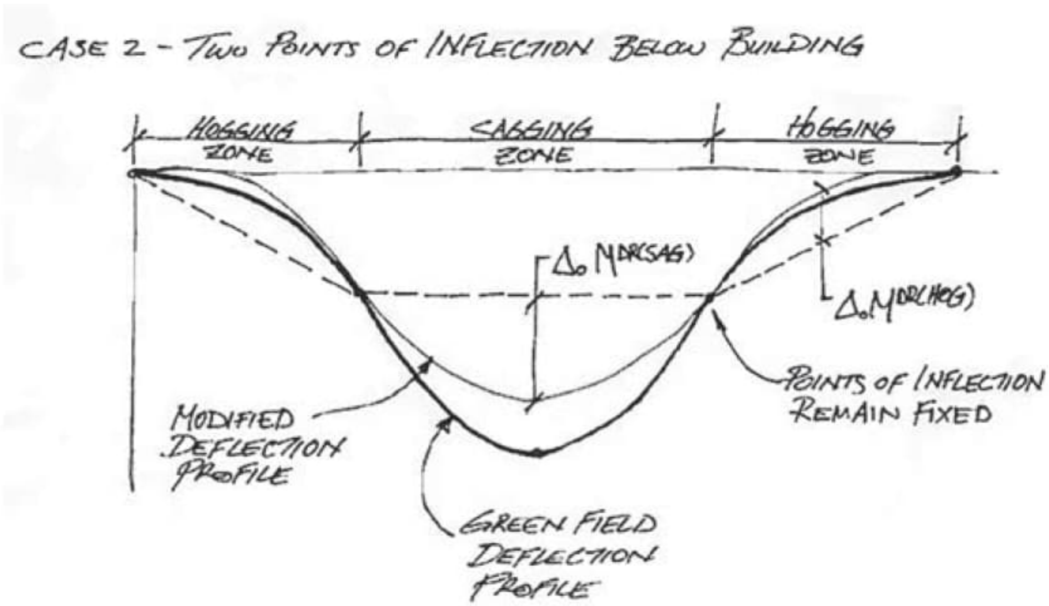

Figure 5 CIRIA Modification Sketch

Using this method a revised damage assessment has been carried out, using the same methods as the Phase 2 damage assessment but with updated settlement values (Enhanced Phase 2). The results are presented in the table below.

Method 2	Vertical Modification	Horizontal Modification	Peak Settlement	Max Tensile Strain	Maximum Slope of Ground
Phase 2 Greenfield as (Baseline)	**N**	**N**	20.2mm	0.0487%	1/538
Enhanced Phase 2 (Potts & Addenbrooke)	**Y**	**N**	11.9mm	0.0038%	1/1060

Revised settlement profiles have also been produced as shown by Figure 6 below. Figure 7 shows the modified contour relative to the base slab of the AerCap House building.

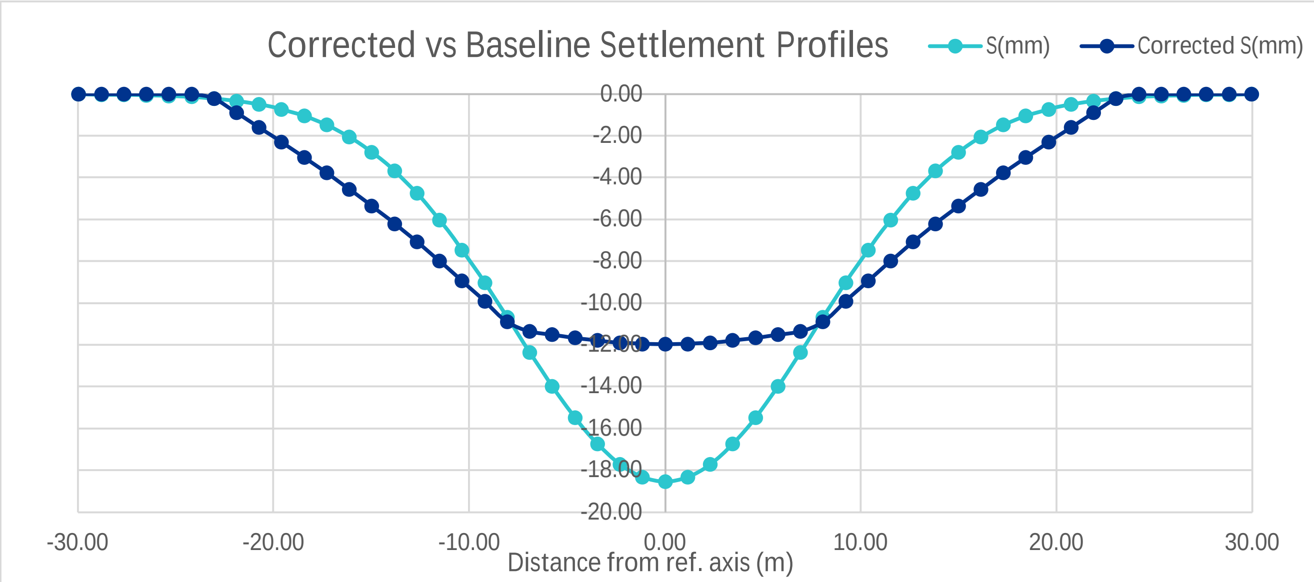

Figure 6 Greenfield settlement profiles modified by building stiffness for the Aercap House Building

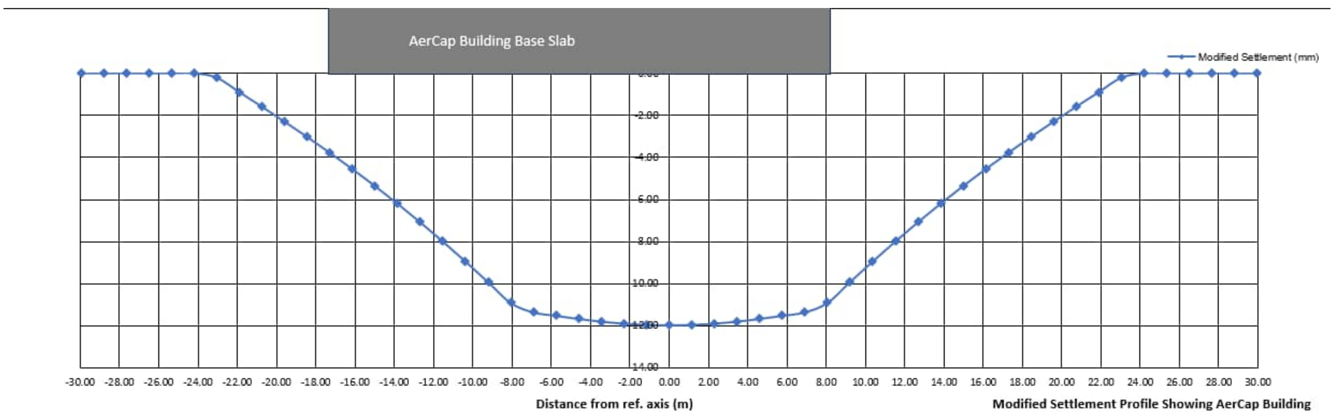

Figure 7 Modified settlement profile showing the location of the AerCap House building base slab

5. Building Form

To consider how the Aercap House building will respond to the movement predicted, the capacity of the basement slab against the imposed greenfield baseline settlement profile was initially checked using conservative parameters for the base slab. A minimum base slab thickness of 870mm was assumed for all sections, along with a concrete grade of C35/45 and the lower bound long term concrete stiffness of 16.5 GPa. Very conservatively a minimal slab reinforcement of one layer of A393 mesh top and bottom at 50mm cover were used.

The base slab is predominantly 1m thick with the thinner section only in areas which are not subject to traffic around the lift core. Within the basement slab there may be more reinforcement providing a stronger slab.

A model profile was developed based on the provided structural drawings. The settlements at the edge locations and for the two bearing pads were calculated, applying the Potts & Addenbrooke adjustments described above.

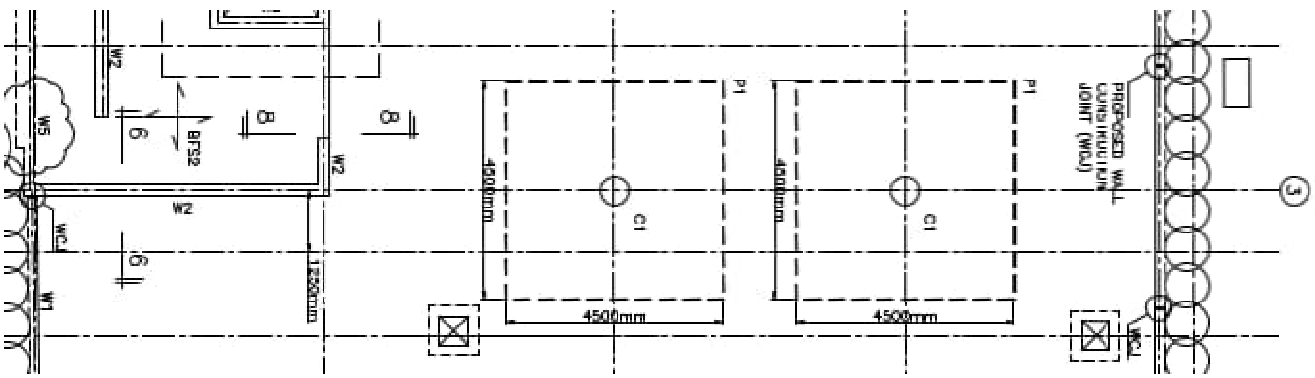

Figure 8 Extract of Arecap House from drawing 3762-C-101-3 Proposed level – 2 Foundation Plan General Arrangement

It was assumed the pads would move with the ground and the slab would span between these points. This would cause the pad to deflect as it bends to the required radius of curvature. The settlement at the centre of the pad was assumed to apply to the entire pad. The worst section for the slab was the one with the largest induced radius of curvature between points. This was determined to occur between the two pads and the slab edge at the right ('right' from the perspective of Figure 8 above).

Position relative to Tunnel Axis (m)	-9.0	-3.0	10.0
Baseline Settlement (mm)	9.33	17.16	7.94
Induced Radius of Curvature (m)	4715		
Equivalent imposed Bending Moment	192 kNm		

The induced radius of curvature between these points was calculated and through the application of elastic-beam theory this curvature was converted into an equivalent imposed bending moment. This moment was input into a standard Eurocode 2 crack check calculation, which confirmed the section was uncracked.

This assessment took no account of any uplift water pressure, which would be beneficial, as it applies a load which will bend the slab upwards and therefore reduce the net deflection (and thus imposed moment) on the slab.

6. Frame displacement analysis

As part of the CIRIA C796 assessment, a frame displacement analysis has been carried out on a simplified frame through the part of the structure outlined in **Error! Reference source not found.**.

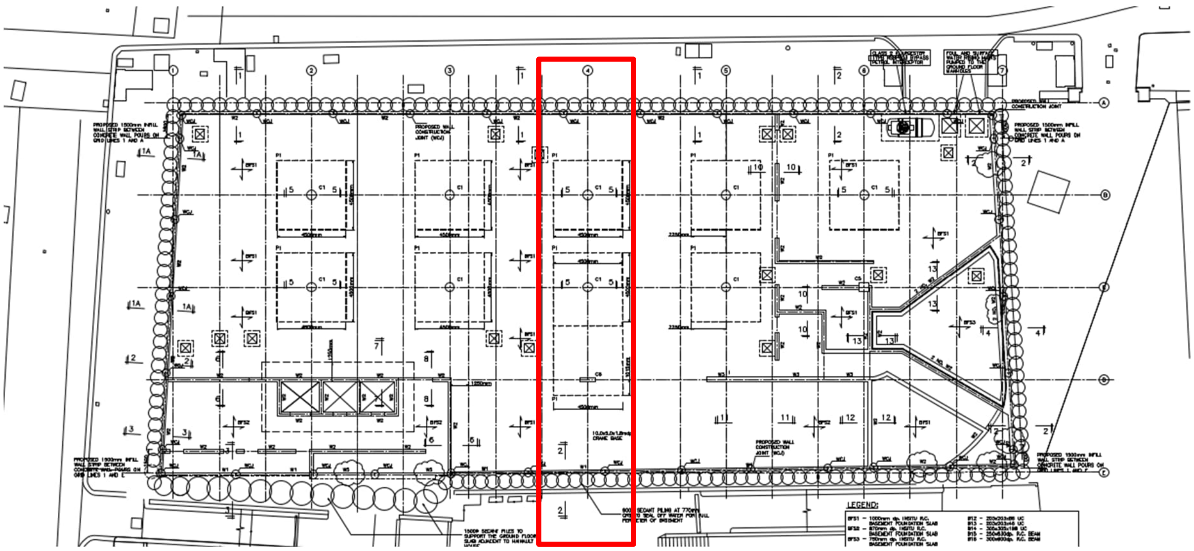

Figure 9 Foundation plan of the structure showing location of frame analysis

Member sizes and dimensions for the frame were taken from Section B-B on drawing 3762-C-202-1 and the frame in Figure 10 was created. An initial frame bending moment profile evaluation was carried out as a benchmark, considering a serviceability design state, loaded with self weight, a 2.5kPa superimposed dead load and a 4.0kPa live load, with the 260/ 325 depth of floor carried by the 9m frame.

To this model the vertical and horizontal displacements produced from the modified settlement profile calculated in accordance with Potts and Addenbrooke (1997) were applied. With the maximum movement being applied to the support in red in Figure 10 with decreasing movements radiating out from this point to the supports highlighted in orange. Due to the width of the structure relative to the settlement trough, every support was subject to an applied displacement. The bending moments for the frame for the benchmark loading and the benchmark + imposed displacement are shown in Figure 11

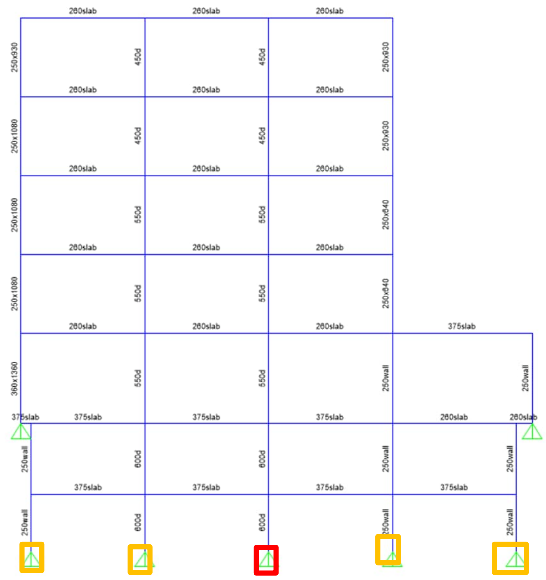

Figure 10 Simplified frame for imposed deflection analysis

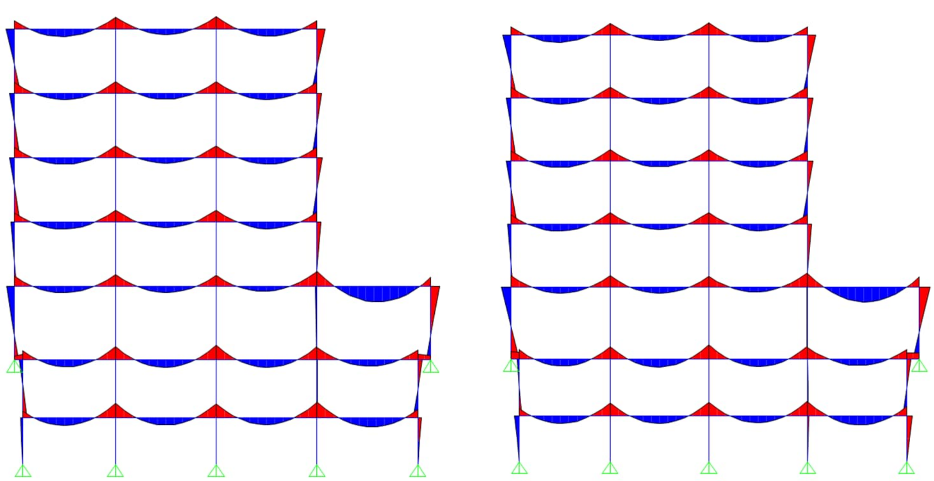

Figure 11 Left : Initial frame bending moment profile, Right: Displacement Induced bending moment profile

The maximum moments recorded by element type are shown in Table 1. The circular columns have been pinned at both ends and such have zero moment in both cases so are not included. A number of the elements show recorded moments outside the benchmark envelope, however these increases are for all members less than 10%. By inspection, imposed ground movements are unlikely to have a significant effect on the structural elements.

Table 1 Comparison of worst-case moments by member type

Element Type	Max Benchmark Moment	Max Displacement Moment	Max relative to Benchmark %
250x1080	163.02	172.69	105.93
250x640	159.06	161.74	101.68
250x930	243.52	251.93	103.45
260 Slab	442.29	434.70	98.28
375 Slab	476.97	450.11	94.37

7. Conclusion

The assessments show that for the building structure using the conservative volume loss of 0.5% is acceptable. The enhanced phase 2 considers the effect of the stiffness of the building and as expected, this reduces, but does not eliminate the predicted settlement.

We have reviewed the structure and the basement waterproofing and the movement will not cause cracking to the basement structure or compromise the frame. This is consistent with the expectations of the Phase 2 assessment.

The façade of the Aercap House Cox building uses different types of fixing details to support the glass and stone cladding system. Each type of fixing detail can accommodate different amounts of movement. There are some types of fixing that will be less tolerant to movement. The specific tolerance for additional movement within each fixing detail over the section of the building where the tunnel may cause movement may vary. There may be elements within the façade fixing details which do not have sufficient additional movement tolerance to accommodate the predicted movements.

Therefore, it is recommended that, due to concerns over movements to specific elements in the AerCap House building façade, “mostly likely” volume loss parameters are used to provide a more refined analysis representing the mostly likely movement at source and to reduce the potential impacts on the more sensitive elements of the fixings. This should be, undertaken in conjunction with building specific mitigation and instrumentation and monitoring as part of a trigger action plan for the building that will provide the appropriate controls for the building and tunnel construction.

A building specific trigger action plan will be put in place which will include the following:

- Detailed assessment of the façade fixing details on the premises, identifying the different elements on each façade.
- Details of instrumentation and monitoring required for the building and façade.
- The agreed actions/ mitigation measure to be undertaken when the tunnelling is with the zone of influence of the building.
- The engagement that will occur with the stakeholder throughout the construction period.

The TAP will be a live document and will be maintained and implemented by the Contractor and overseen by the Independent Monitoring Engineer.