MetroLink

Transport Infrastructure Ireland

Arthur Cox Enhanced Building Damage Assessment Report

| P01

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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 building, volume loss (VL) is 0.5% and the K value is 0.4.

The tunnel at the location of the Arthur Cox building is wholly within the limestone rock and the buildings are also founded on the limestone rock.

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

2. Introduction

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

The Arthur Cox building has a reinforced concrete framed building consisting of seven floors over a double basement. The superstructure consists of a reinforced concrete frame with reinforced concrete columns supporting flat slab floors at each level. Lateral stability in the frame is achieved via a diaphragm action through the floor plates into three separate structural cores around the building. The perimeter columns are supported on a capping beam, and these loads are transferred into a secant wall and into the limestone rock. The internal columns are transferred onto pad foundations which are integral in the basement ground bearing slab. The basement slab is an inverted suspended slab to accommodate the water pressure.

The results of the phase 2 greenfield settlement at the lower basement level of the Arthur Cox building indicate a max slope of 1 in 666 and a max settlement of 18.9mm.

The greenfield settlement is then modified using a method (as presented by this document), using a method to take account of the behavior of the Arthur Cox building structure in response to ground movements generated by the 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

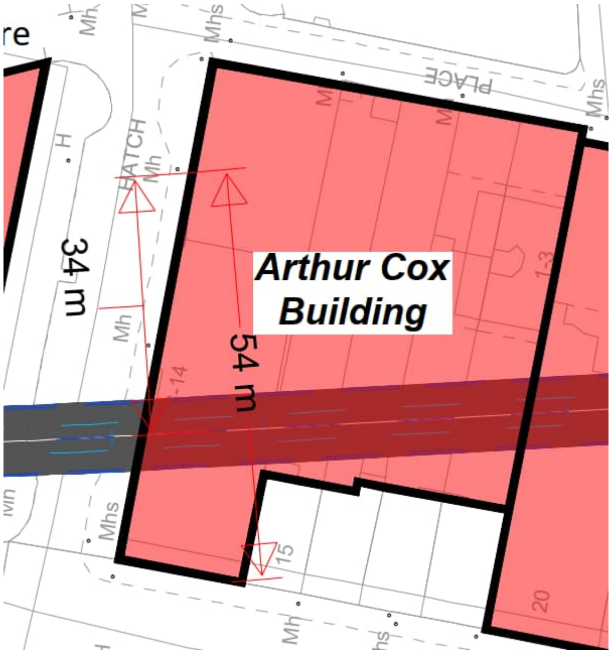

Figure 1 Marked up Footprint of the Arthur Cox Building relative to the proposed MetroLink alignment

A representative cross-section of the Arthur Cox Building was selected, as indicated in Figure 1. This section was selected as it had the full settlement curve developing under the building and so will capture all effects from the full settlement trough. The tunnel axis level was extracted from the most recent alignment drawing and the volume loss and 'k' parameters kept at 0.5% and 0.4 as per the Phase 2 assessment. The Limestone stiffness was taken as 1600MPa.

All building levels and element thicknesses were taken from the Revit model provided by the building owner.

4. Building Stiffness

In accordance with CIRIC C796 guidance, there are four Methods 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 the Arthur Cox building the method proposed by Franza was not considered, as it is applicable 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 Arthur Cox Building.

The method proposed by Franzius includes a building length factor which requires a constant section. Although the Arthur Cox building has elements which are regular this method is not considered appropriate for the Arthur Cox building, particularly given the oblique angle which the tunnel underpasses the building.

Potts & Addenbrooke has been selected over Franzius as it is more appropriate given the structural form of the Arthur Cox 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. In the absence of any soft ground which would alter the movement, a high stiffness approach is closest to expected actual behaviour.

As the building can be most appropriately represented as shallow pads 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 the provided design curves and modification factors read off. The curves vary with the e/B ratio. Which is a measure of how eccentric the tunnel alignment is relative to the building where ‘e’ is the distance from tunnel axis line to the centre line of the building and B is the building width. A value of 0.25 was determined for the Arthur Cox building based on the plan in Figure 1 and the alignment drawings. The design curves and the selected values are shown below by, Figure 3;

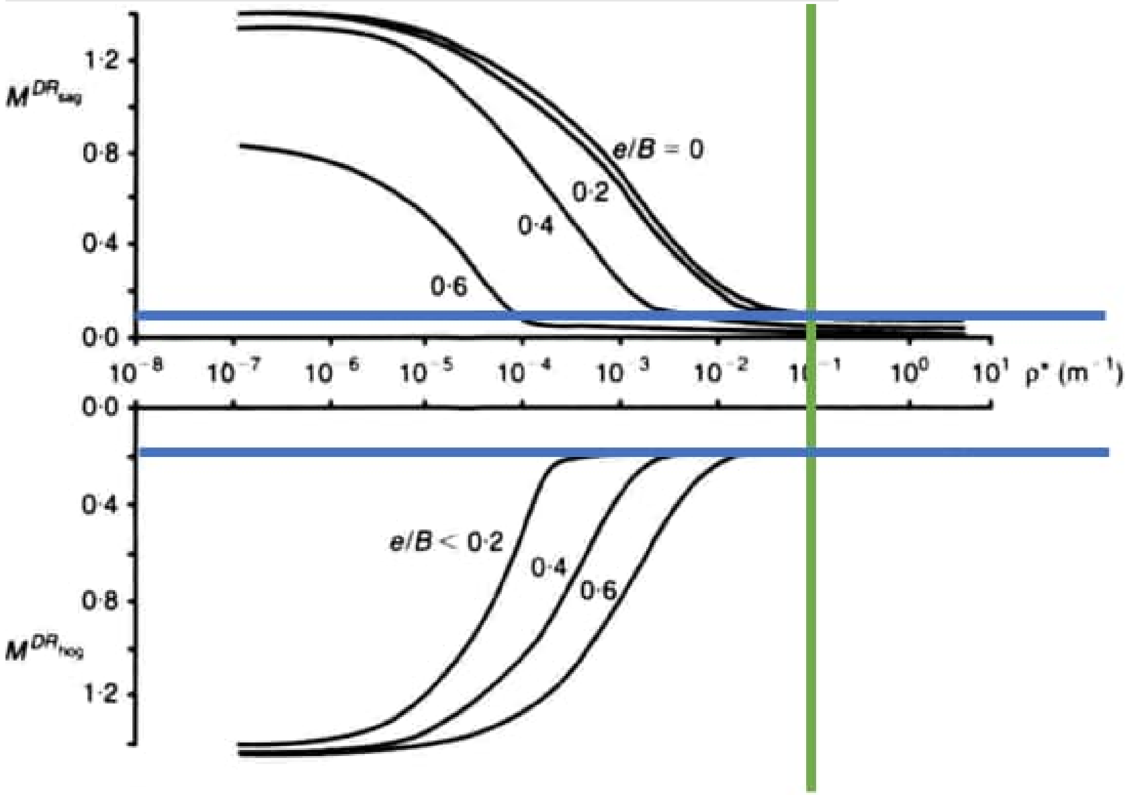

Fig. 16. Design curves for modification factors for deflection ratio

Figure 3 P&A Design Curve with Arthur Cox Relative Stiffness marked on (green line)

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

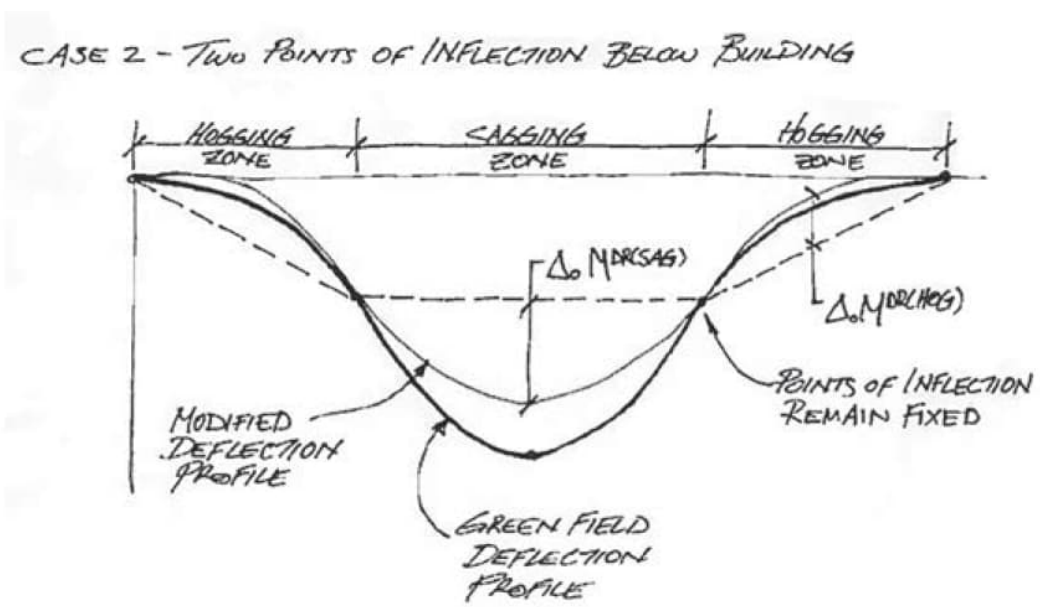

Figure 4 CIRIA Modification Sketch

Using this method a revised damage assessment could be carried out, using the same methods as the standard Phase 2 damage assessment but with updated settlement values. The results are in the Table below.

Method	Vertical Modification	Peak Settlement	Maximum Tensile Strain	Maximum Slope of Ground
Baseline	N	18.9mm	0.040%	1/666
Potts & Addenbrooke	Y	13.0mm	0.018%	1/955

Revised settlement profiles were also produced These are shown in the Figure 5 below, along with Figure 6 showing the modified contour relative to the base slab of the Arthur Cox Building

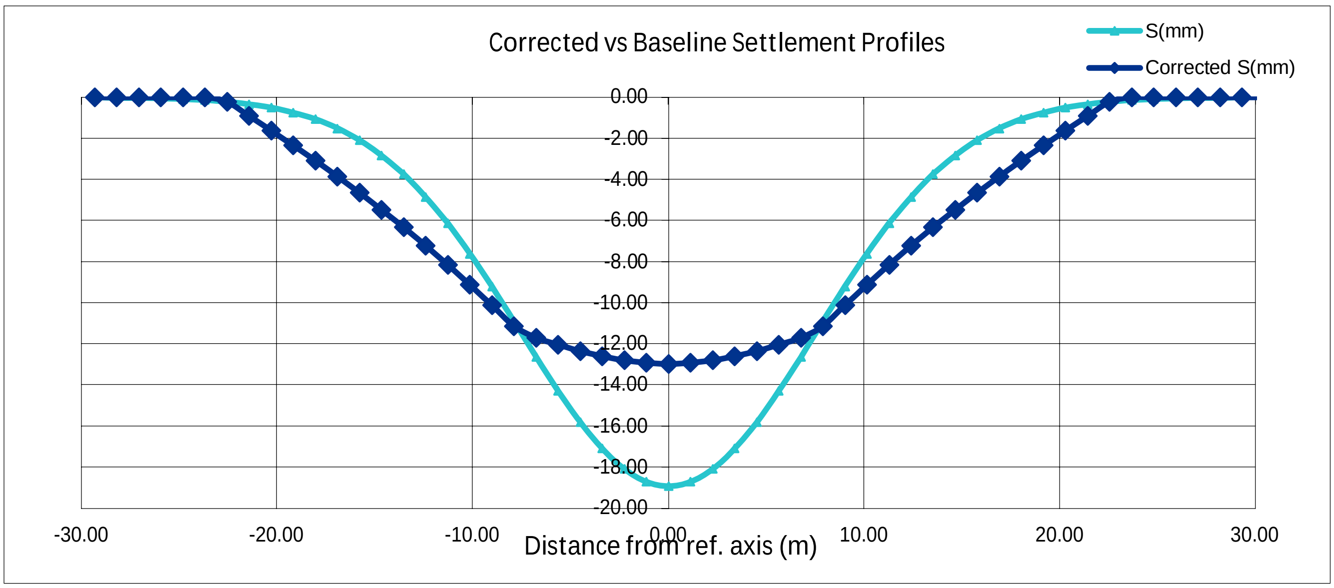

Figure 5 Greenfield Settlement profiles modified by building stiffness for the Arthur Cox Building

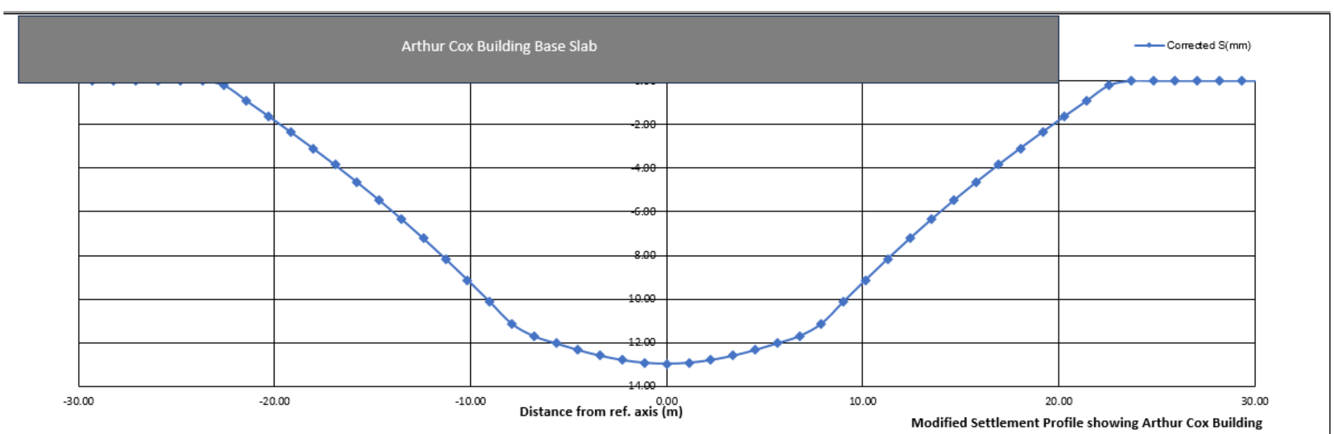

Figure 6 Modified settlement profile showing the location of Arthur Cox building base slab

5. Building Load Weight Effect

The effect of building load on settlement has been investigated by Franzius.[1] It is concluded that the effect of increasing the mean effective stress can increase the Volume Loss and the lateral stress ratio in contrast influences the deformation field of the soil above the tunnel which then affects the response of the building to the tunnel induced subsidence. However, the effect is small compared to the decrease of deflection ratio and horizontal strain with increasing building stiffness and the results lie close to the upper bound curves provided by Potts and Addenbrooke. These curves are referenced in CIRIA C796 and have been used for the enhanced Phase 2 assessment. On that basis the effect has not been considered further in this assessment.

[1] Fransius, J.N, Potts, D.M, Addenbrooke, T.I., Burland, J.B the influence of building weight on tunnelling -induces ground and building deformation, soils and foundations vol 44, No 1 25-38 Feb 2004.

6. Building Form

To consider how the building will respond to the movement, the capacity of the basement slab against the imposed baseline settlement profile has been initially checked, using conservative parameters for the base slab. The minimum base slab thickness of 600mm was assumed for all sections. As the actual properties were not known, conservative assumptions were made and a concrete strength of C32/40, a lower bound long term concrete stiffness of 16.5 GPa, and a minimal slab reinforcement of one layer of A393 mesh top and bottom at 50mm cover, were used.

A profile was developed based on an extract from the Revit model, as shown by Figure 7 below. The settlements at the edge locations and the pad were calculated, using the Potts & Addenbrooke adjustments described above. Adopting a conservative approach, all of the pads were assumed to be at the lower depth as the lowest pads on the section.

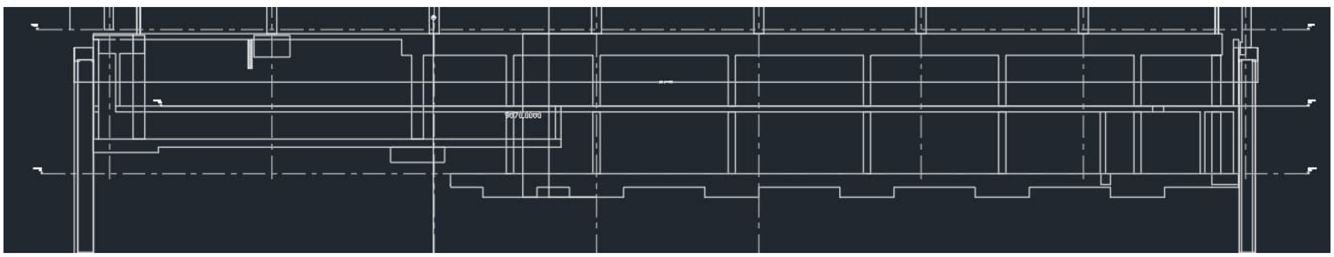

Figure 7 Extract from Arthur Cox Revit Model

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 bent to the required radius of curvature. The worst section for the slab was the one with the largest induced radius of curvature between points; this was determined to occur at the section directly over the tunnel axis location.

Position relative to Tunnel Axis (m)	-4.8	0	5.125
Baseline Settlement (mm)	15.434	18.9	15.0
Induced Radius of Curvature (m)	33227		
Equivalent imposed Bending Moment	89 kNm		

The induced radius of curvature between these points was calculated and through applying 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 reduce the net deflection (and thus imposed moment) on the slab.

7. 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 Figure 8.

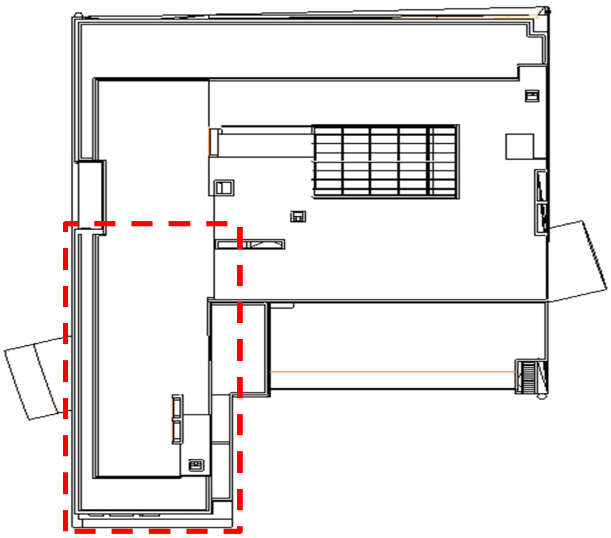

Figure 8 Revit image of the structure showing location of frame analysis

Member sizes and dimensions were taken from the Revit model and the frame in Figure 9. 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 depth of floor carried by the frame being 6.775m.

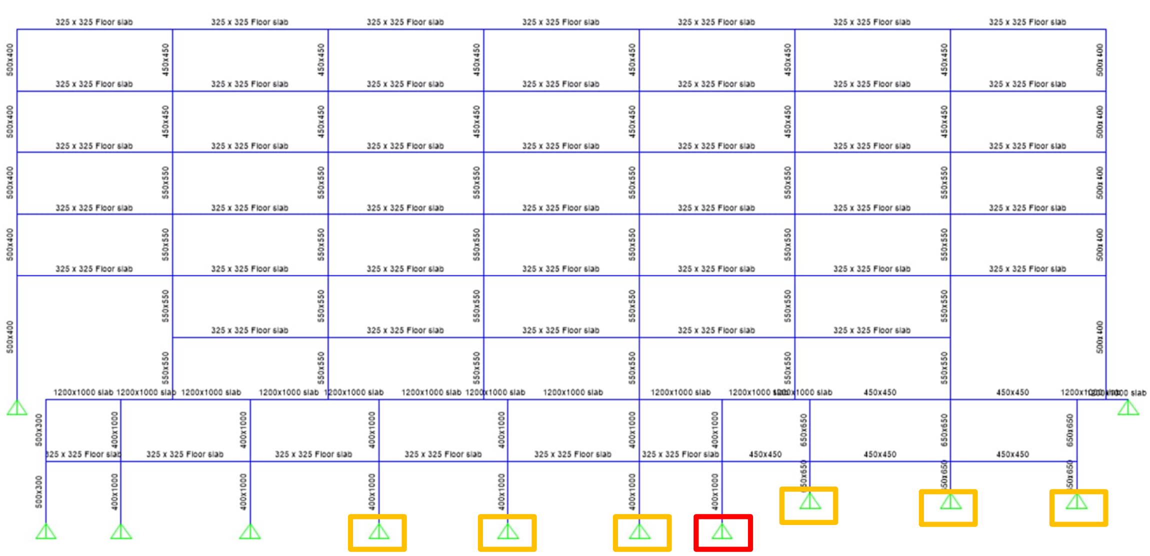

Figure 9 Simplified Frame for imposed deflections analysis

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 highlighted in red on Figure 9, and with decreasing movements radiating out from this point to the supports highlighted in orange. The non-highlighted supports were beyond the settlement profile, and as such, had no displacements applied. The bending moments for the frame for the benchmark + imposed displacement are shown in Figure 10.

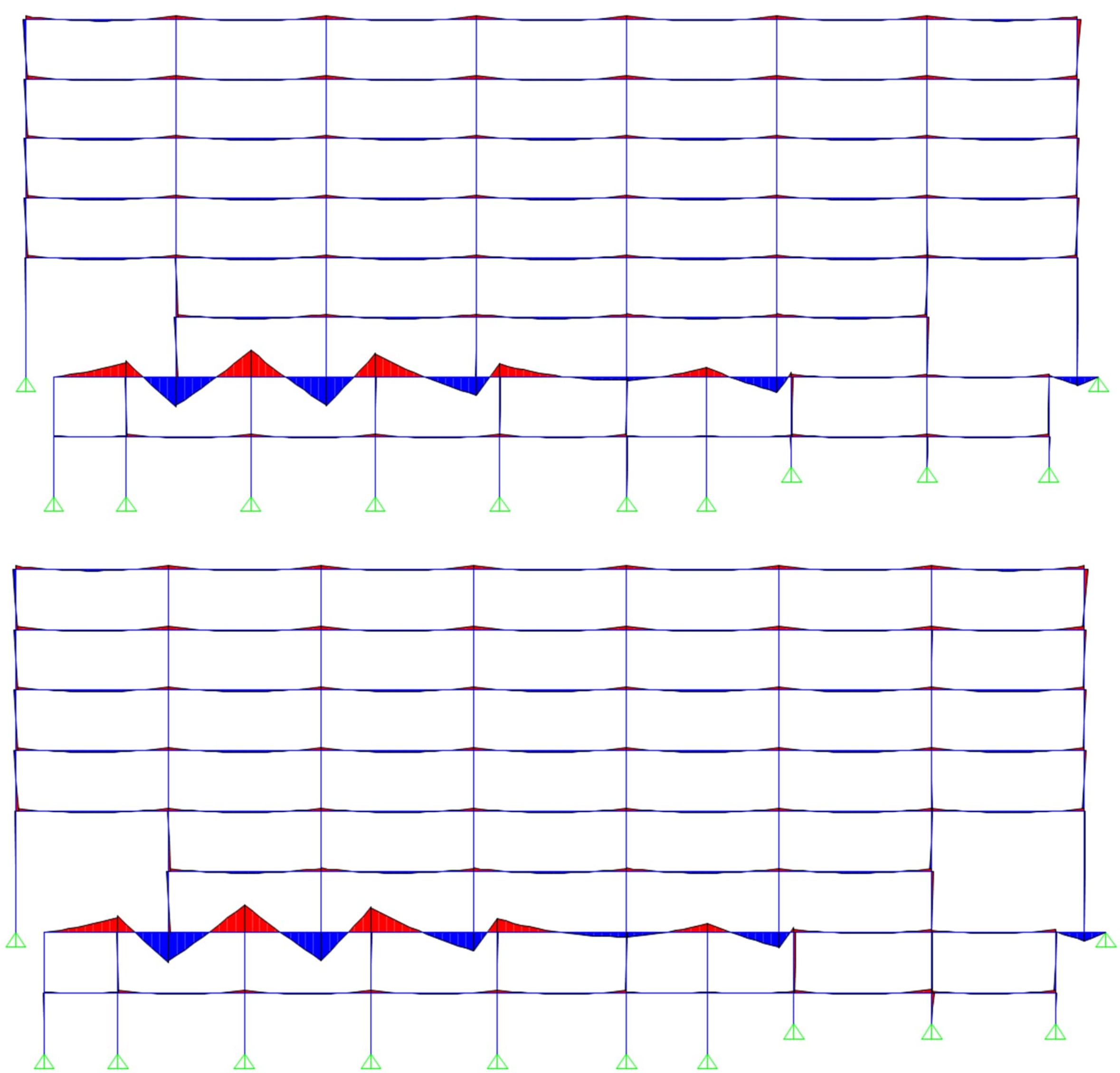

Figure 10 Top: Initial frame bending moment profile, Bottom: Displacement Induced bending moment profile

The maximum moments recorded by element type are shown in Table 1. The only element that recorded moments outside the benchmark envelope, is the 450x450mm columns at the upper two floors of the structure. Since this increase is 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 %
550x550	368.64	355.33	96
450x450	40.54	43.10	106
325 Slab	6001.34	5969.75	99
1200 Deep Beam	6001.34	5969.75	99

8. Secant Pile Analysis

To assess the response of the secant piles to the imposed movement a settlement curve for the pile toe level was generated. This was then modified using the factors determined from the Potts and Addenbrooke analysis described previously in this report. This modification is considered appropriate as the modified settlement curve occurs due to the soil-structure interaction between the tunnel induced movement and the Arthur Cox building and thus this, is the settlement profile the secant piles will experience. It is noted this is a conservative assumption as the CIRIA C796 guidance notes that secant piles and capping beams will contribute to the building stiffness, but they have not been considered in this assessment.

The baseline and modified profiles at pile toe level are shown in Figure 11.

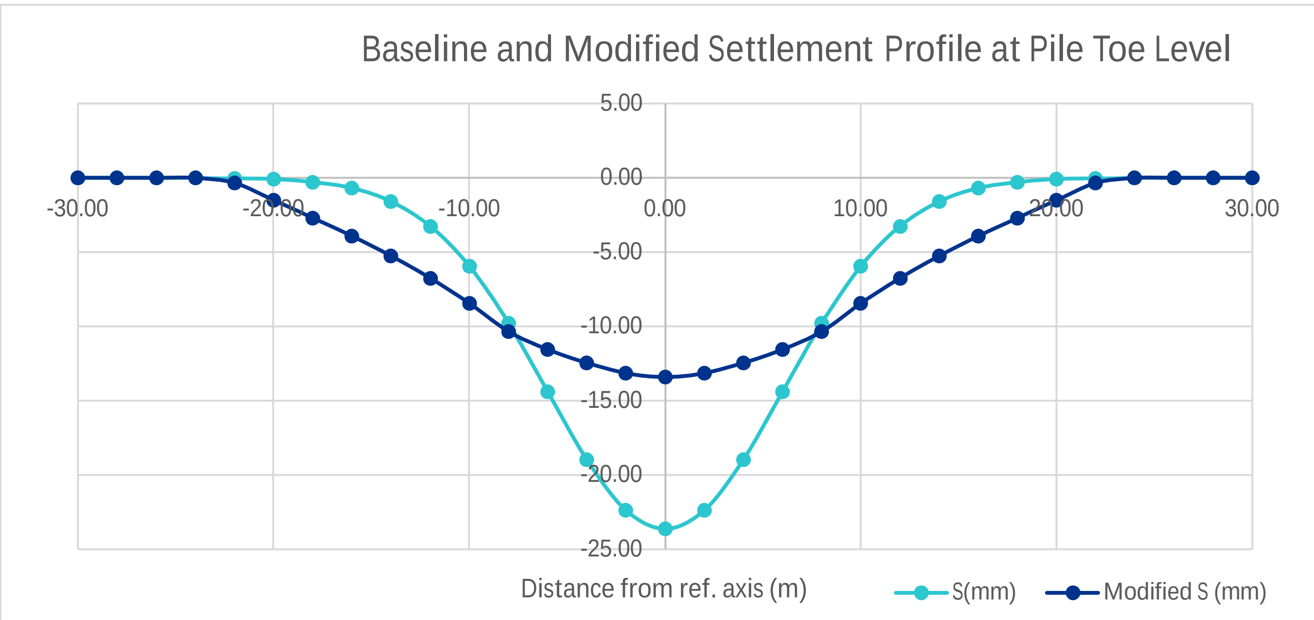

Figure 11 Baseline and modified settlement curves for secant piles

On the conservative assumption that the pile cap will move the same amount as the pile toes, the imposed maximum slope at pile level was calculated as 1/740.

9. 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 Arthur Cox building uses a variety of 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 movement to specific elements in the Arthur Cox 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.