MetroLink

Transport Infrastructure Ireland

Cadenza Building Enhanced Building Damage Assessment Report

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Project Manager:	Paul Brown
Author:	John Kinnear
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Jacobs Engineering Ireland Limited

Merrion House
Merrion Road
Dublin 4, D04 R2C5
Ireland
T +353 1 269 5666
F +353 1 269 5497
www.jacobs.com

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

The tunnel at the location of the Cadenza 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 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.

The Cadenza building is a seven story commercial building with a two story basement. The structure comprises of in situ concrete basement and in situ concrete columns supporting in situ concrete post-tensioned slabs. The building has an internal reinforced concrete frame and uses post-tensioned reinforced concrete floors for large spans. The building facades are bespoke modern glass and stone façade. There is a secant wall around the building which supports the façade along some aspects, but the south-western edge of the building is stepped back, and the secant wall is along the property boundary, with a low-level garden. The building is supported on columns with pad footing in the basement slab. On some drawings it has been referred to as the Irish Life building due to its owner, however it is the same structure.

The results of the phase 2 greenfield settlement at the lower basement level of the Cadenza building indicate a max slope of 1 in 590 and a max settlement of 19.9mm.

The greenfield settlement prediction has then been modified using a method to take account of the Cadenza building structure and its response to the predicted ground movements that will be generated by MetroLink tunnel construction.

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

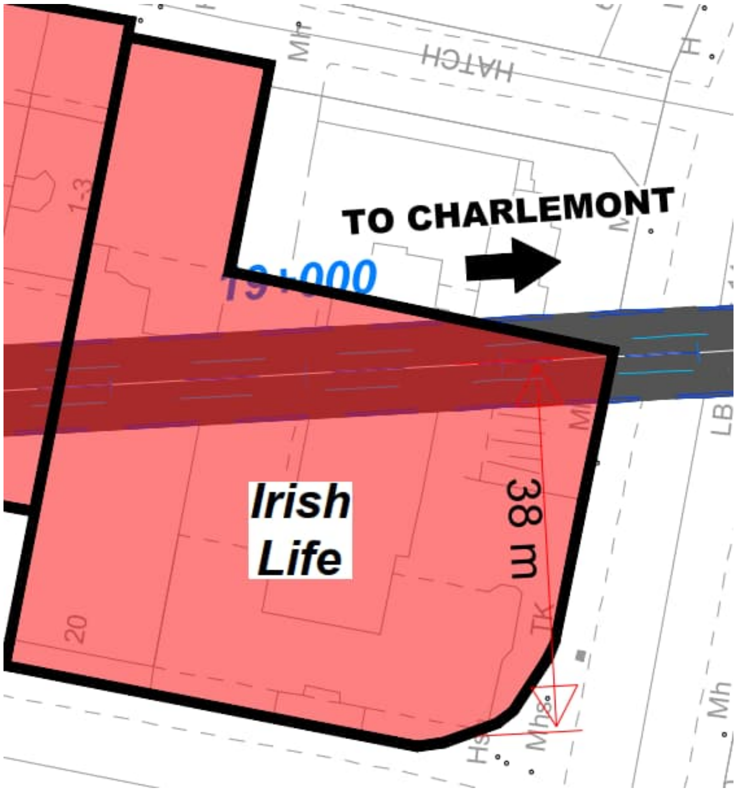

Figure 1 Footprint of the Cadenza building in relation to the MetroLink horizontal alignment labelled as Irish life

The worst case cross-section of the Cadenza Building was selected based on proximity to the tunnel, as indicated in Figure 1. The tunnel axis rises under the building being approximately 1.5m shallower at the end of the building closest to Charlemont, therefore the section was taken at this location. The tunnel axis level was extracted from the 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, spans and element thicknesses were taken from the structural drawings provided by the building owner.

4. Building Stiffness

In accordance with CIRIA 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 Cadenza 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 Cadenza Building.

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

Potts & Addenbrooke has been selected over Franzius as it is more appropriate given the structural form of the Cadenza 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 Cadenza 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 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 0.58 was determined for the Cadenza 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;

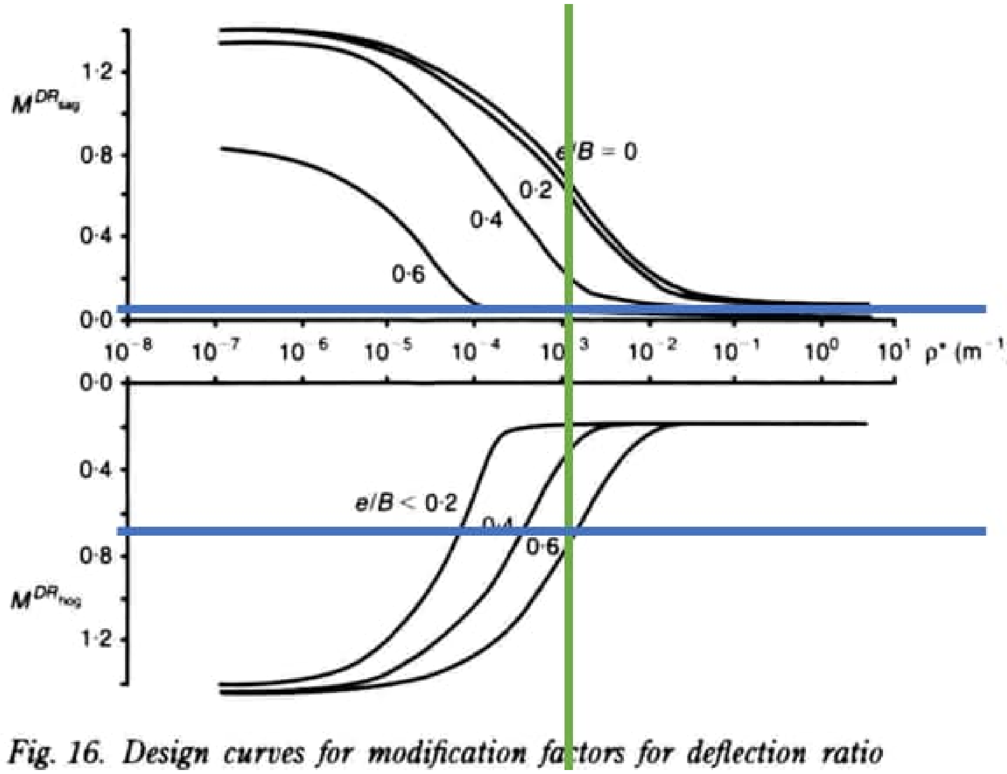

Figure 3 P&A Design Curve with Cadenza building Relative Stiffness marked on

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

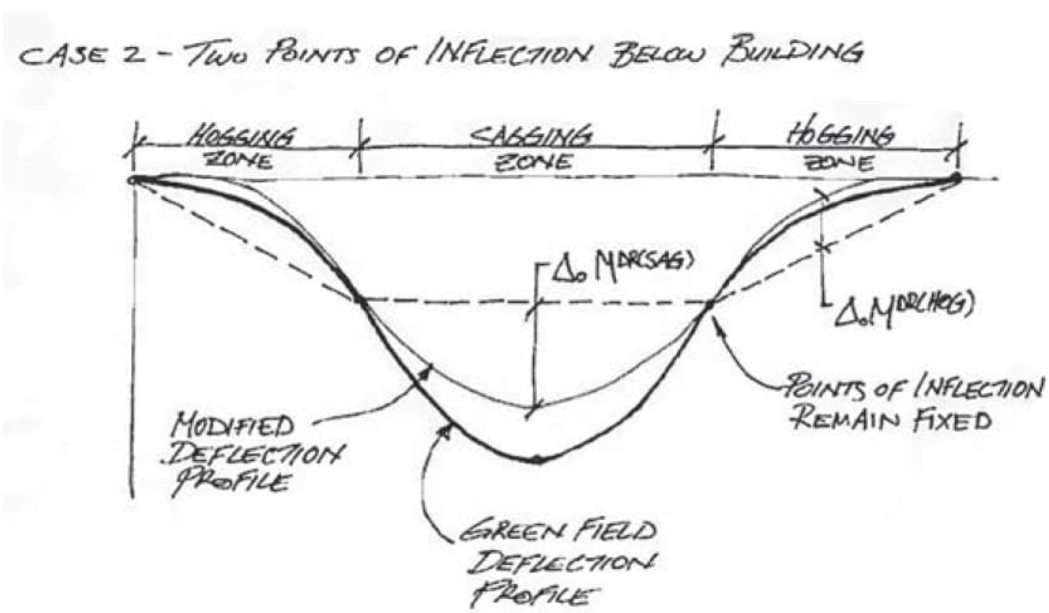

Figure 4 CIRIA C796 Modification Sketch

Using this method a revised damage assessment has been 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	19.9mm	0.0443%	1/590
Potts & Addenbrooke	Y	12.5mm	0.0316%	1/1005

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 Cadenza Building

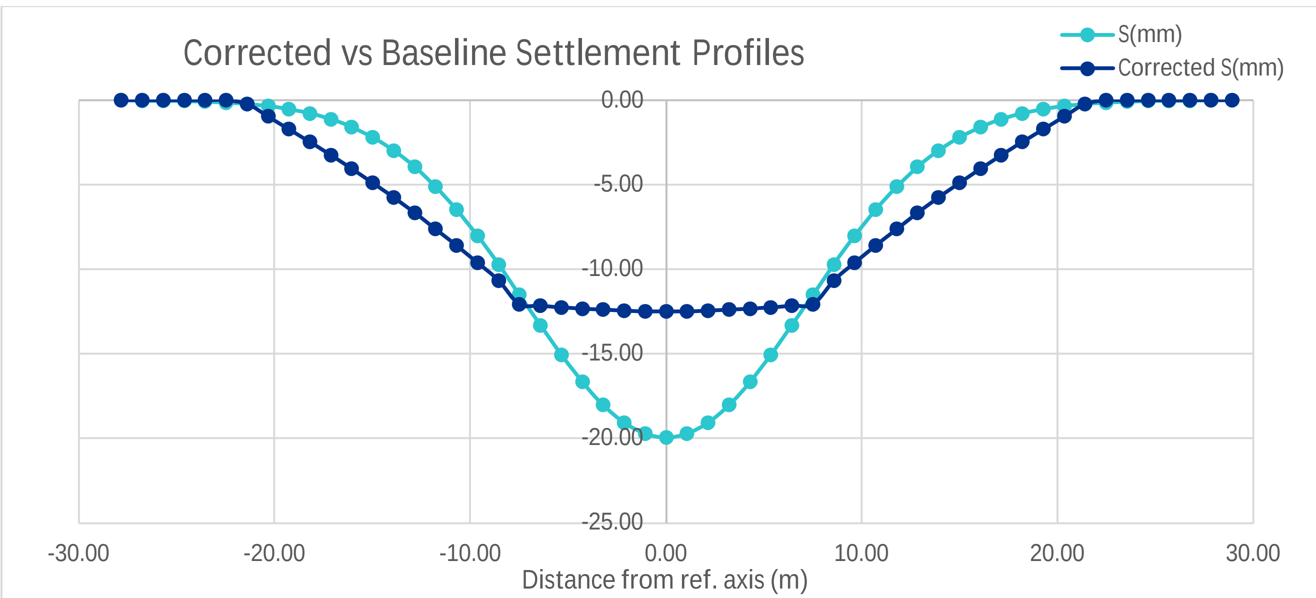

Figure 5 Greenfield settlement profiles modified by building stiffness for the Cadenza Building

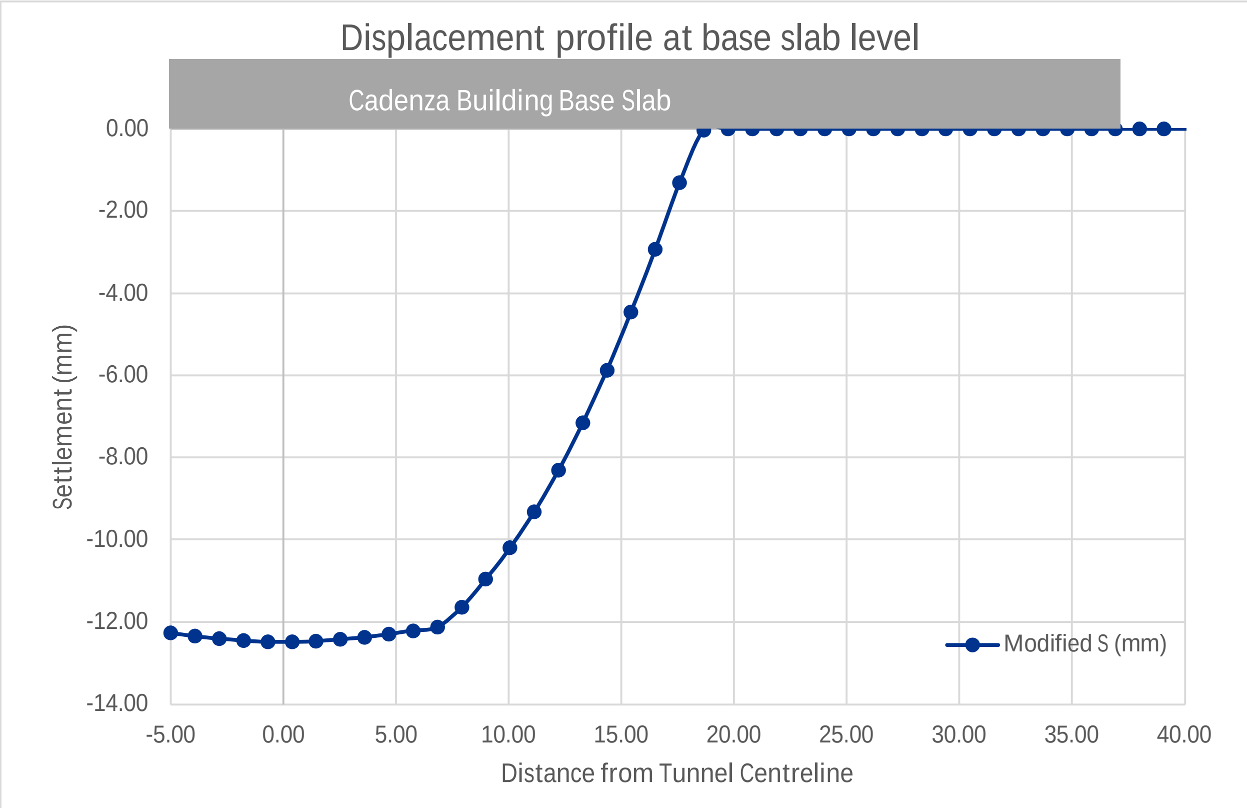

Figure 6 Modified settlement profile at the location of Cadenza building base slab

5. Building Form

To consider how the building will respond to the movement, the capacity of the basement slab against the imposed baseline settlement profile was initially checked, using conservative parameters for the base slab. The base slab thickness of 300mm was taken from the provided drawings. As properties of the 300RC slab were not provided, 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 provided structural drawings, as shown in Figure 7. The settlements at the edge locations and the pads were calculated, using the Potts & Addenbrooke adjustments described above.

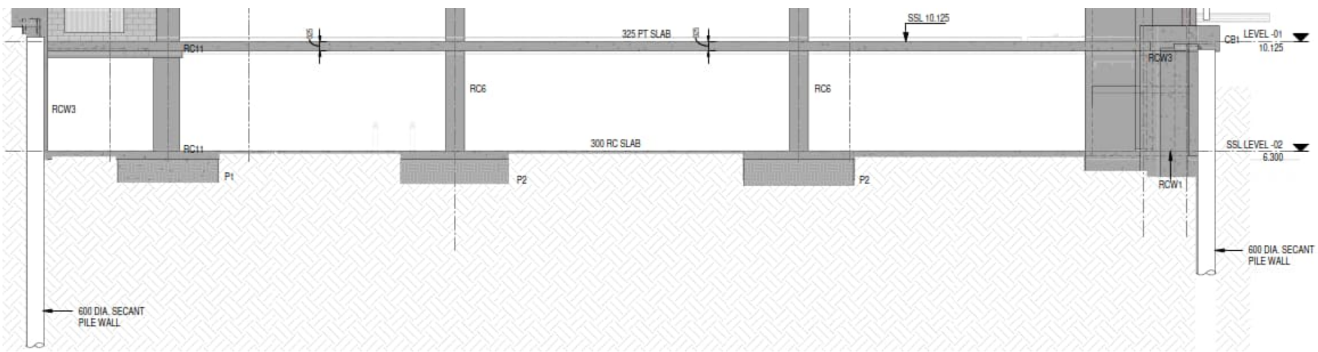

Figure 7 Extract from Cadenza building dwg EARL-WMS-ZZ-ZZ-DR-S-21001 showing selected cross section

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)	-12	0	1.5
Baseline Settlement (mm)	4.861	19.93	19.50
Induced Radius of Curvature (m)	4363		
Equivalent imposed Bending Moment	8.5kNm		

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 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 Figure 8.

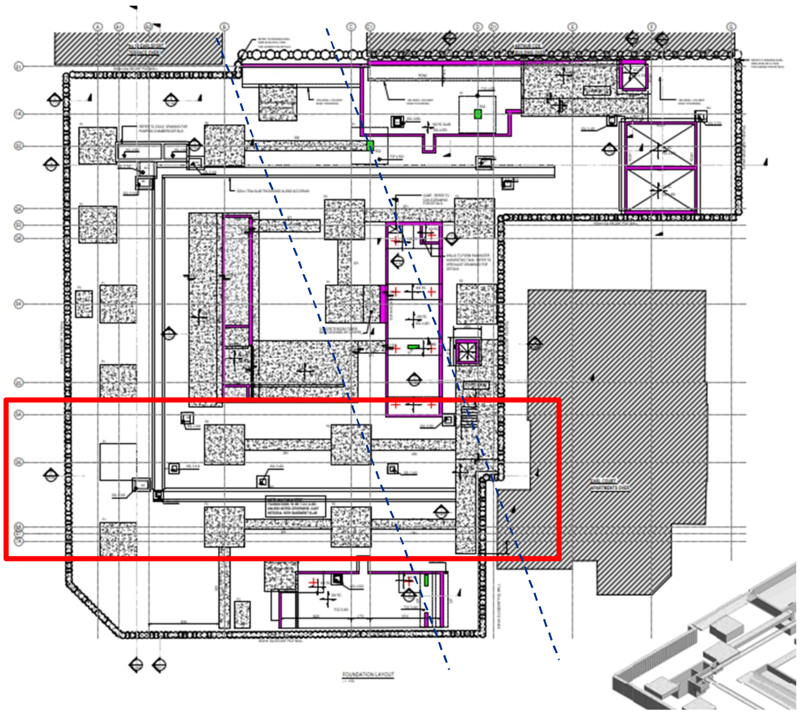

Figure 8 Foundation plan of the structure showing location of frame analysis

Member sizes and dimensions for the frame were taken from drawing EARL-WMS-ZZ-ZZ-DR-S-21002 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 6.625m.

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, 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, the benchmark loading and the benchmark + imposed displacement are shown in Figure 10.

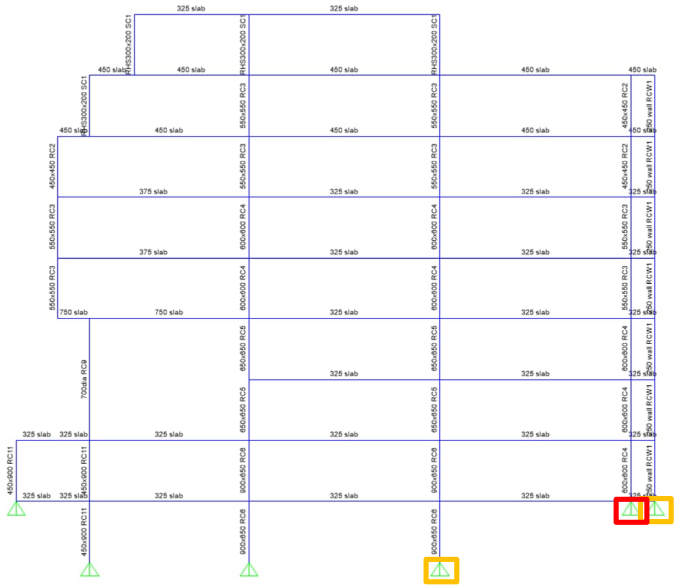

Figure 9 Simplified Frame for imposed deflections analysis

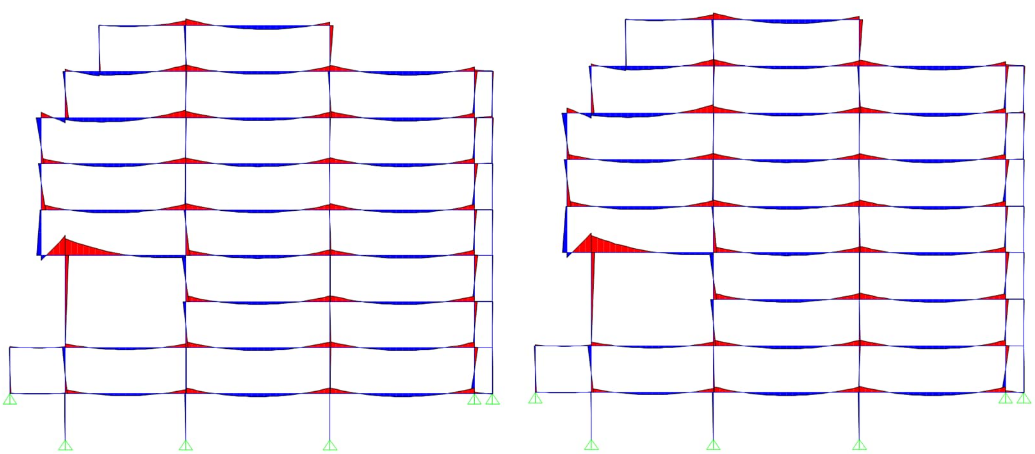

Figure 10 Left: Initial frame bending moment profile, Right: Displacement induced bending moment profile

The maximum moments recorded by element type are shown in Table 1, these increases for all members other than RC5 are less than 4%. The RC5 element recorded moments showing a 48% increase over the benchmark moments. However, although these moments are an increase in % they are still smaller moments than those within the benchmark moment. The RC5 element is also larger than the smaller elements RC2-RC4 which are subject to higher moments therefore is likely to be within the members capacity. By inspection, imposed ground movements are unlikely to have a significant effect on the structural elements of the building.

Table 1 Comparison of worst-case moments by member type

Element **Type**	**Max Benchmark Moment**	**Max** Displacement **Moment**	**Max relative to Benchmark** %
450x450 RC2	1087.38	1072.37	98.62
550x550 RC3	973.80	979.99	100.63
600x600 RC4	791.49	768.35	97.08
650x650 RC5	179.69	265.17	147.57
900x650 RC6	844.91	834.32	98.75
450x900 RC11	679.96	706.25	103.87
325 Slab	1386.37	1397.09	100.77
375 Slab	1273.89	1284.90	100.86
450 Slab	1663.13	1683.03	101.20
750 Slab	4149.68	4117.64	99.23

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 Cadenza 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 Cadenza 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.