

# EXCAVATION INDUCED GROUND MOVEMENTS AND RISK MANAGEMENT STRATEGIES

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## ABSTRACT

As well as the need to maintain stability, retention systems for excavations in urban environments are required to limit ground movements to mitigate adverse impact to adjacent structures. Tolerable ground movements will depend on the circumstances of the site; in particular the proximity and type of adjacent structures and their foundation type, underground services or tunnels and their current structural condition. It is impractical to limit excavation induced movements to zero even though this may be the wish of the adjacent property and infrastructure owners. Therefore, a site specific risk management strategy is essential for the selection, design and construction of an appropriate retention system for a particular set of site circumstances.

This paper provides a brief overview of excavation performance in different types of ground including a review of measured horizontal movements in deep basement excavations in the Sydney region and a literature study of excavations in soils. The paper then discusses prediction methods including analytical, numerical and empirical techniques, impact on adjacent structures and mitigation measures. Two short case studies are then given to illustrate the adopted risk management strategies. The first case study concerns an excavation adjacent to several existing railway tunnels, for which a risk register together with a monitoring system were used to manage the rail authority's concerns on potential damage to the tunnel linings. The second case study describes the application of damage assessment that enabled the client to be informed of the risks and the selection of the appropriate excavation and retention method to reduce risks associated with excavation adjacent to heritage listed buildings.

## 1 INTRODUCTION

The following paragraph was reported by Shirley (1987) who received survey responses from 23 organisations from 230 questionnaires sent out, with data provided on 30 excavation sites:

*Some 17% of excavations reported significant (i.e. greater than 20 mm) movements into the excavation and these movements often caused problems during the construction of the permanent structure. In some cases the movement of the temporary support was very large and resulted in a failure of the excavated face over a significant area; consequently, significant repairs were required to the nearby streets and adjacent building structures.*

The author's experience indicates that not much has changed over the last 25 years, with excavation induced movement remaining a common problem in urban construction. Therefore, the purpose of this paper is to provide an overview of assessment methods, possible mitigation measures, and management of risks associated with excavation induced movements with the aim of improving our practice in this field.

## 2 ASSESSING EXCAVATION INDUCED MOVEMENTS

### 2.1 PRIMARY FACTORS INFLUENCING EXCAVATION INDUCED MOVEMENTS

There are many factors which influence the magnitude of excavation induced movements. The primary factors are as follows:

- Type of ground.
- Type of support.
- Depth of excavation and geometry of the excavation.
- Factor of safety against excavation instability and bottom heave.
- Groundwater conditions and impact of dewatering.
- Stiffness of the supporting system.
- Preloading of anchors or struts.
- Construction sequence and workmanship.

Figure 1 shows two examples of gross deformation which occurred to properties adjacent to basement excavations. In both cases, the "near miss" situations did not cause injury to persons but the consequence of time delay, cost of

remedial works, and litigation were significant. Clearly, when excavation supports are designed with low factors of safety, the risk of excessive deformation increases which it is generally not desirable even for temporary works if workers and/or the public are at risk, or if the consequence of damage to adjacent properties is unacceptably high. In this paper, the main concern is on stable excavations designed to acceptable factors of safety.



1(a) Example 1: Excavation is to the right of the photo

1(b) Example 2: Failure of shotcrete wall

Figure 1: Examples of Excavation Induced Movements

## 2.2 ANALYTICAL AND NUMERICAL TECHNIQUES

Bolton et al (2008) described an analytical approach using the Mobilised Strength Design (MSD) method with the aim of satisfying both safety and serviceability requirements in a single calculation procedure for strutted excavations in clay. Non-linear soil stiffness may be captured in the MSD approach using a strain dependent relationship based on simple shear testing or previously established relationship. They calibrated the results against finite element analyses (FEA) to enable users to adopt this method without having to conduct project-specific FEA. The readers are encouraged to refer to their paper for this interesting analytical method of predicting excavation support performance, and also the 52<sup>nd</sup> Rankine Lecture by Bolton (2012) for the application of the MSD principle in geotechnical design.

Nowadays, versatile and powerful computer programs are readily available commercially for the assessment of excavation induced movements, and numerical analyses have become routine in the design of excavation support. Designers have ready access to commercial programs which range in ability to include partial or full soil-structure interaction analysis. Methods for deformation assessment in commercial software range from analytical correlations from inbuilt database, to deformation analyses using three-dimensional FEA methods. However, adequate determination of the required geotechnical parameters is often lacking in relation to the sophistication of the analysis techniques employed. For example, in situ lateral stresses which have important bearing on excavation induced movements, are not routinely measured, with the at rest earth pressure coefficient,  $K_0$ , often estimated incorrectly without taking into account the effect of over-consolidation.

For excavations in rock, we are fortunate in Sydney that high locked-in horizontal stresses in Sydney Sandstone are well known, as reported by Enever et al (1990) and Enever (1999). For practical design purposes, Pells (2002) recommended the following in situ horizontal stresses in the Sydney area, away from topographic effects:

$$\sigma_{NS} = 1.5 + (1.2 \text{ to } 2.0)\sigma_v$$

$$\sigma_{WE} = (0.5 \text{ to } 0.7)\sigma_v$$

where:

$\sigma_{NS}$  = horizontal stress in the north-south direction

$\sigma_{WE}$  = horizontal stress in the west-east direction

$\sigma_v$  = vertical stress

When adopting these in situ horizontal stresses in numerical assessment of excavation induced movements in Sydney rock, the following points should be borne in mind:

- The choice of appropriate rock mass stiffness needs to take into account anisotropy, stress path (i.e. unloading, small strain values), and the effect of stratification as discussed in Oliveira and Wong (2012). For horizontally bedded rock with widely spaced sub-vertical joints in the Sydney region, the horizontal stiffness can be 1.5 to 2

times the vertical stiffness, and reload modulus (for excavation) can be in the range of 1.5 to 2 times higher than the secant loading stiffness.

- Properties of defects must be carefully considered. Horizontal defects have a strong influence on the lateral movement profile with depth, with the likelihood of shearing concentrated on major bedding planes. Joints will also have an influence, particularly in relation to horizontal strain distribution back from the excavation face.
- The influence of previous excavations in surrounding sites should be taken into consideration, as such excavations may cause stress concentration or stress relief depending on the site geometry.

For the retention of excavations in soil, the understanding of non-linear soil behaviour is an important aspect in the numerical assessment of excavation induced movements, and the use of non-linear analysis may be required. If an elastic-plastic model is adopted, appropriate secant soil stiffness should be selected based on an initial estimate of strain levels. Bentler (1998) provides a good summary of lessons learnt from various researchers regarding the use of numerical analyses in assessing retaining wall performance.

### 2.3 PUBLISHED DATA AND EMPIRICAL PROCEDURES

Prior to launching into numerical analysis to assess excavation induced movements, it is useful to obtain an idea of the magnitude of movement that may be anticipated by consulting published data and empirical procedures.

#### 2.3.1 Excavations in Sydney Sandstone

Excavations in medium strong or better sandstone in the Sydney region can generally be carried out vertically without systematic support. The relief of lateral stresses due to excavation, however, will induce lateral movements in the rock behind the excavation.

Where high quality measurements of in situ stresses and rock mass stiffness are not available, numerical analyses may be no better than the use of experience in terms of overall magnitude of movements. Figure 2 provides a guide for expected movements at the excavation face measured in deep basements in the Sydney CBD.

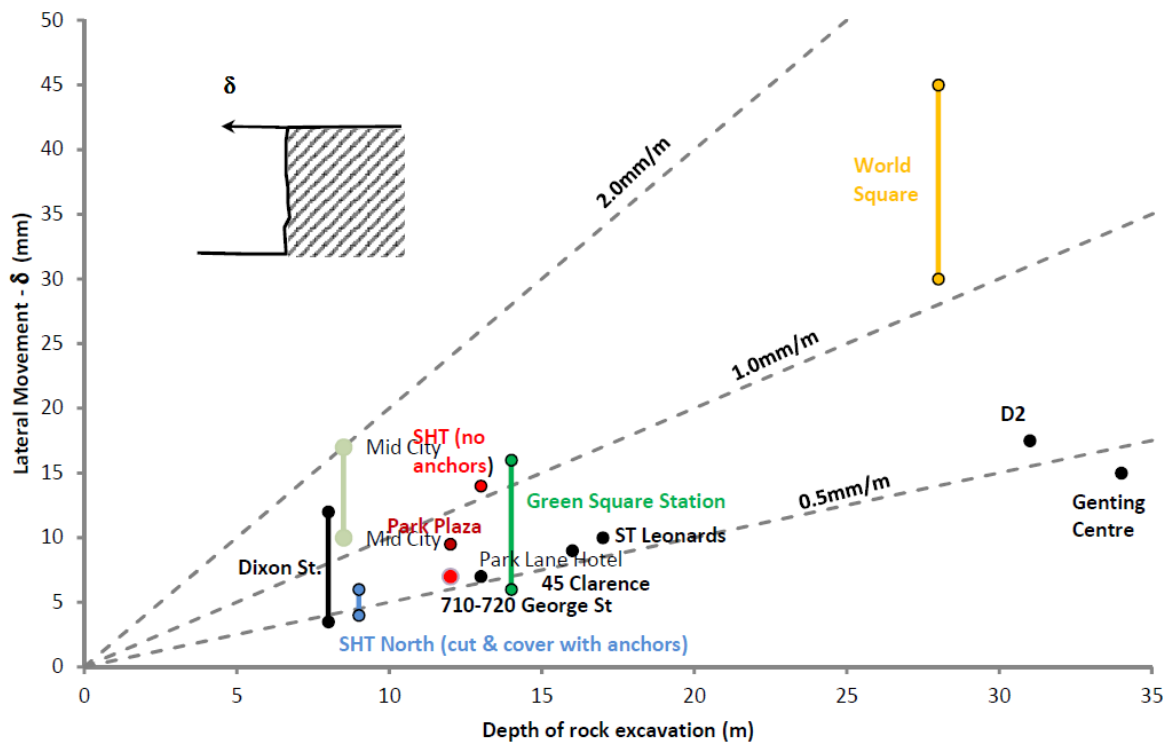


Figure 2: Measured Lateral Movements at the top of Basement Excavations in Sydney Sandstone

Observed lateral movements behind excavations in rock are generally restricted to about 1.5D to 2D from the face, where D is the depth of excavation. However, Braybrooke (2013) has reported movements as far back as 3D from the face north of the Cook and Phillip Park Aquatic and Fitness Centre on College Street, Sydney. When considering the magnitude and extent of ground movements, topographic effects, length of the excavation face, and presence and nature of defects will also need to be considered. The maximum lateral movement usually occurs near the mid-span of the

excavation face. Observed vertical movements at the top of stable excavations in competent rock are generally negligible.

**2.3.2 Excavations in Soils**

Peck (1969) proposed an empirical method for estimating movements for soldier pile or sheet pile walls braced with cross bracing or tie back anchors.

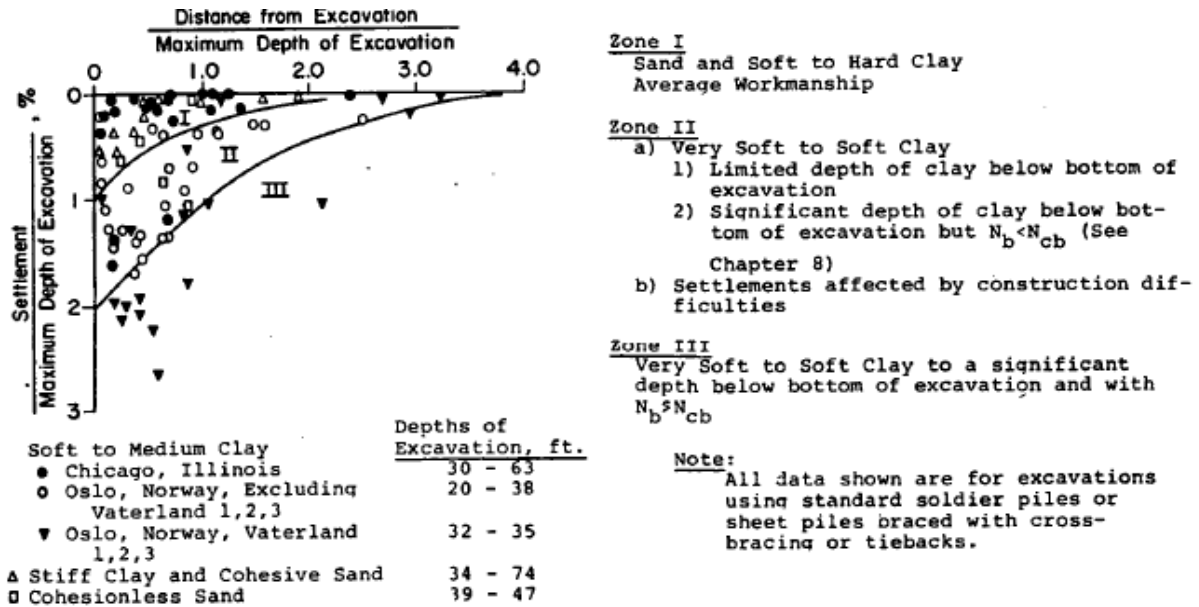


Figure 3: Empirical Procedure for estimating Movements based on Field Measurements (after Peck, 1969)

For soft and medium clays, Clough and O'Rourke (1989) presented design curves for estimating excavation induced lateral movements including the effect of basal heave and support system stiffness as shown in Figure 4.

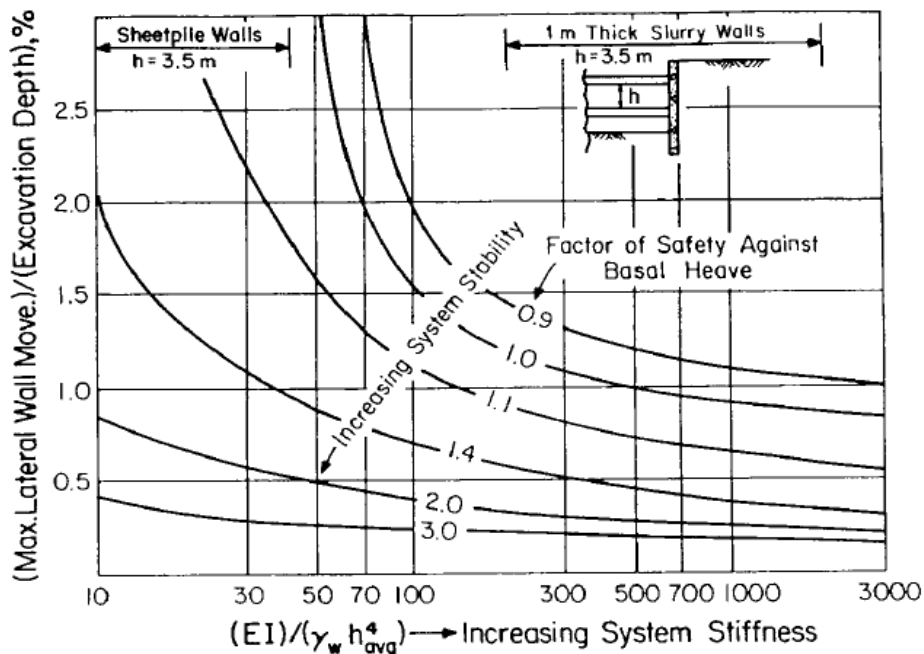


Figure 4: Effect of Basal Heave and Wall System Stiffness on Lateral Movement (from Clough et al 1989)

Recorded settlement magnitudes are generally in the same order as lateral movements. For a cantilevered wall, the maximum settlement is expected to be near the excavation face whereas for an excavation which is braced at the top, the maximum settlement is expected to be some distance back from the wall. Clough and O'Rourke (1990) presented

the settlement profile of retained soil for adequately designed and constructed excavations in different soil types as shown in Figure 5.

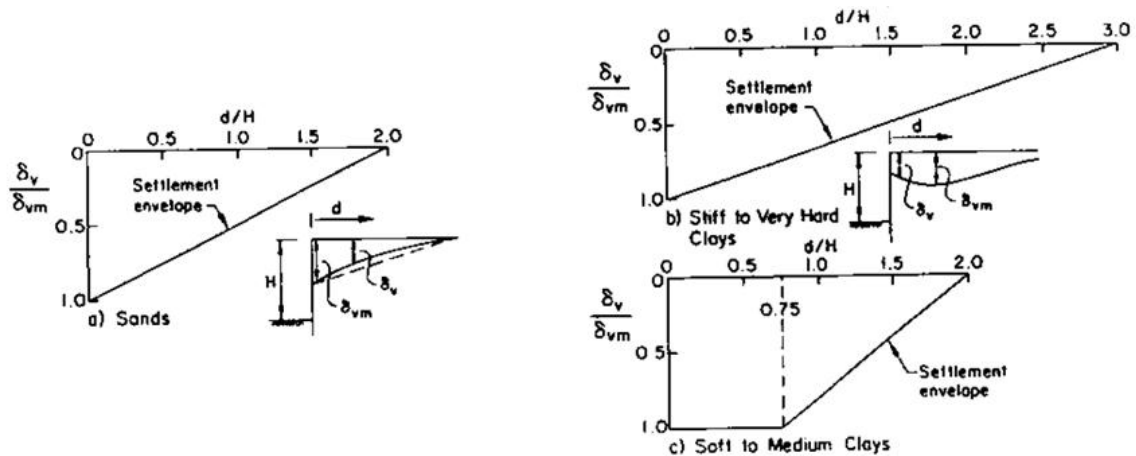


Figure 5: Settlement Profiles adjacent to Excavations in Different Soil Types (from Clough & O'Rourke, 1990)

### 2.3.3 Soil Nail Supported Excavations

Soil nailing is a passive support system and requires some ground movement to mobilise loads in the reinforcement. In its early application, it was thought that soil nailing should not be used where the excavation is adjacent to infrastructure sensitive to movements. Observed results, however, have indicated that, under appropriate ground conditions, and provided the soil nailing system is adequately designed and constructed, small ground movements are required to mobilise the soil nail loads, and excavation performance for soil nailed excavations may be comparable to other forms of excavation support.

The CLOUTERRE Report (1991) and Murray (1993) provided indicative movements for soil nailed excavations in the range of 0.1% to 0.4% of the excavation depth. To limit excavation movements, it is important that soil nails be installed progressively and as promptly as possible following each stage of excavation. Installing soil nails at the completion of substantial excavation will inevitably lead to greater deformation and has the potential to cause overall instability. Large excavation intervals prior to soil nail installation should be avoided unless the risk of short-term instability is assessed to be low and there are no adjacent structures sensitive to excavation induced movements.

### 2.3.4 Other Important Considerations

Other important considerations influencing excavation induced movements are listed below:

- In water charged ground, drawdown of the groundwater level by active dewatering or via seepage into the excavation can cause significant settlement in surrounding areas. The potential for consolidation settlement is particularly important where soft soil layers are present within the soil profile.
- In sands, the adopted wall system should ensure that loss of ground due to migration of material into the excavation does not occur. Installation of tie back anchors in sands has also been known to cause settlement of adjacent structures due to wash out of sand and/or anchor hole collapse during the drilling process.
- Wall installation may also cause adjacent ground settlement prior to excavation, either due to vibrations, stress relaxation or inadequate support during wall excavation (e.g. diaphragm wall or bored secant pile wall).
- Installation of continuous flight auger piles into rock overlain by sandy soils can cause draw-in of materials and ground loss when slow rate of penetration occurs in the rock socket.
- Extra care must be taken if water mains are within close proximity of the excavation as the potential for rupture of such services will inevitably exacerbate ground movements and potentially cause instability. Considerations should also be given to the potential rise in groundwater level due to leakage or rupture of such services from other causes.
- Workmanship and construction sequence play important roles in excavation induced movements; these include factors already mentioned above, as well as timing in the installation of supports (delays tend to cause greater ground movements).
- Pre-stressing tie back anchors or preloading of struts or bracing is effective in reducing wall movements, by removing any slack from connections, and making the soil stiffer in the reloading process.

- Large initial horizontal soil stresses in over-consolidated soils tend to increase excavation induced movements. The effect of high horizontal soil stresses is similar to that discussed in Section 2.3.1 for excavations in rock, but extra care needs to be taken to check the potential for passive failure and basal heave of the soil below the excavation level.

### **3 IMPACT OF GROUND MOVEMENT TO ADJACENT STRUCTURES**

#### **3.1 PREAMBLE**

The impact of ground movements on an adjacent structure is a function of the magnitude and distribution of the movement, and the nature of the adjacent structure (i.e. strength, stiffness and sensitivity to movement and differential movement).

For an adjacent structure that may be at risk from excavation induced movements, it would be useful to classify its existing condition, assess the potential damage caused by additional movements, and reclassify the potential damage following the excavation. Common damage classifications and methods of assessing damage are discussed below.

#### **3.2 CLASSIFICATION OF DAMAGE**

The system of damage category by Burland and Wroth (1975) is widely adopted for the assessment of potential damage to masonry buildings due to tunnel or excavation induced movements. The Australian Standard AS2870-2011 on Residential Slabs and Footings provides a very similar classification system and descriptions of damage, except that it does not have a Category 5 damage classification.

Table 1: Classification of Visible Damage to Walls with Reference to Ease of Repair of Plaster and Brickwork or Masonry (from Burland and Wroth, 1975)

<b>Category of Damage</b>	<b>Normal Degree of Severity</b>	<b>Description of Typical Damage</b> (Ease of repair is underlined) Note: Crack width is only one factor on assessing category of damage and should not be used on its own as a direct measure of it.
0	Negligible	<u>Hairline cracks less than about 0.1mm.</u>
1	Very Slight	<u>Fine cracks which are easily treated during normal decoration.</u> Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1mm.
2	Slight	<u>Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings.</u> Cracks may be visible externally and some repointing may be required to ensure weather-tightness. Doors and windows may stick slightly. Typical crack widths up to 5mm.
3	Moderate	<u>The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced.</u> Doors and windows sticking. Service pipes may fracture. Weather-tightness often impaired. Typical crack widths are 5 to 15mm or several > 3mm
4	Severe	<u>Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.</u> Windows and door frames distorted, floor sloping noticeably <sup>1</sup> . Walls leaning <sup>1</sup> or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15 to 25mm, but also <b>depend</b> on the number of cracks.
5	Very Severe	This requires a major repair job involving partial or complete rebuilding. Beams lose bearing; walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths are greater than 25mm, but depending on the number of cracks.

<sup>1</sup> Note: Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

### 3.3 POTENTIAL DAMAGE ASSESSMENT

The assessment of potential damage to buildings due to ground movements has advanced from simple guides on tolerable settlement and differential settlements such as those given by Skempton and MacDonald (1956), Meyerhof (1956) and Polshin and Tokar (1957), to a more comprehensive assessment by Burland (1995) who considered potential building damage due to bending and shear strains caused by distortions as shown in Figure 6.

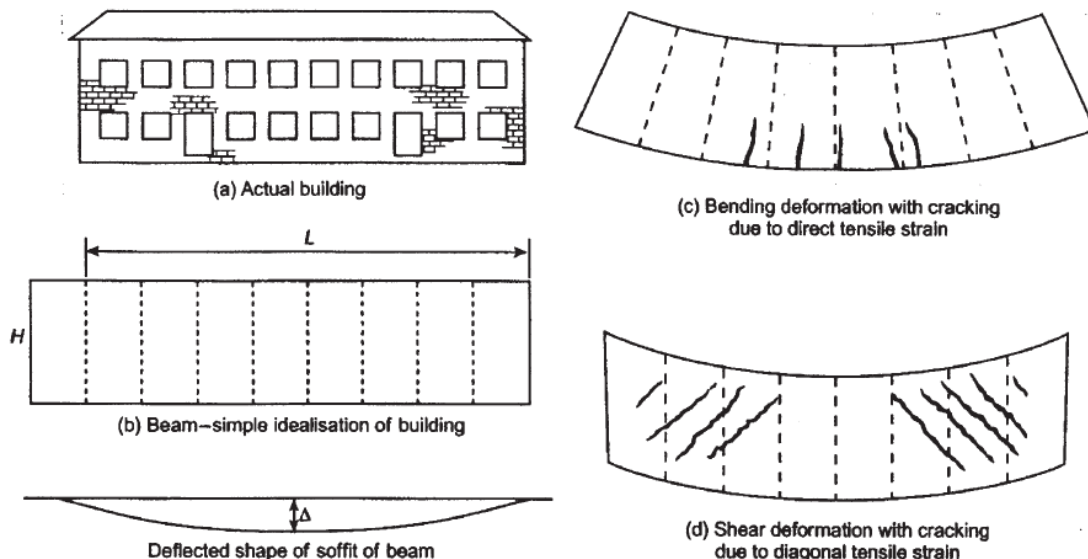


Figure 6: Cracking of an Equivalent Beam in Bending (note: actual building is modelled as an equivalent beam having a height of  $H <$  the building height depending on building stiffness)

Boscardin and Cording (1989) developed an interaction diagram relating potential categories of damage with angular distortion,  $\beta$ , and horizontal strain,  $\epsilon_h$ , for a length to height ratio,  $L/H$  of 1 for the hogging mode of deformation as shown in Figure 7. They concluded that it is possible to assign a range of limiting tensile strains,  $\epsilon_{lim}$ , to the different categories of damage defined by Burland and Wroth (1975). These limiting tensile strains are summarised in Table 2.

Building on the work of Boscardin and Cording (1989), a set of equations was presented by Burland (1995) to assess the maximum tensile strain which may occur in different distortion modes (i.e. bending and diagonal strains) to a structure due to horizontal ground strains and deflection ratios. An interaction diagram for the assessment of potential damage category based on deflection ratio,  $\Delta/L$ , and horizontal strain,  $\epsilon_h$ , was presented by Burland (1995) for  $L/H = 1$  as shown in Figure 8(a), and similar interaction diagrams have been extended by the current author to  $L/H$  ratios of 1.5 and 2 as shown in Figures 8(b) and 8(c) using the procedure by Burland (1995).

The existing condition of the structure is not included in the above method of assessment. Therefore, close interaction between geotechnical and structural engineers is essential in assessing potential damage to adjacent structures due to ground movements.

Table 2: Relationship between Limiting Tensile Strains and Category of Damage from Boscardin and Cording (1989)

Category of Damage	Limiting Tensile Strain, $\epsilon_{lim}$ (%)
0	0 to 0.05
1	0.05 to 0.075
2	0.075 to 0.15
3	0.15 to 0.3
4 to 5	> 0.3

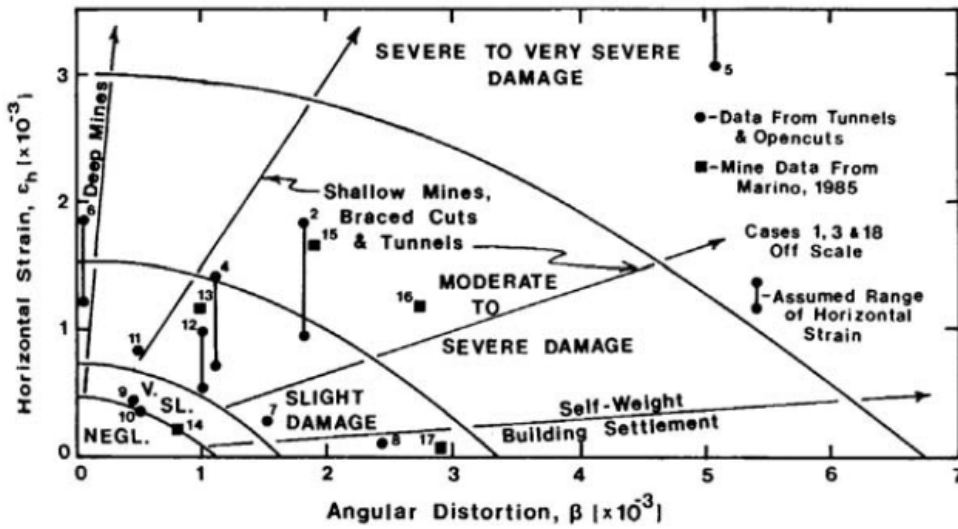


Figure 7: Interaction Diagram of Potential Damage Assessment (from Boscardin and Cording, 1989).

Note:  $\beta = 2\Delta/L$  assumed (refer to Figure 6 for definition of  $\Delta$  and  $L$ )

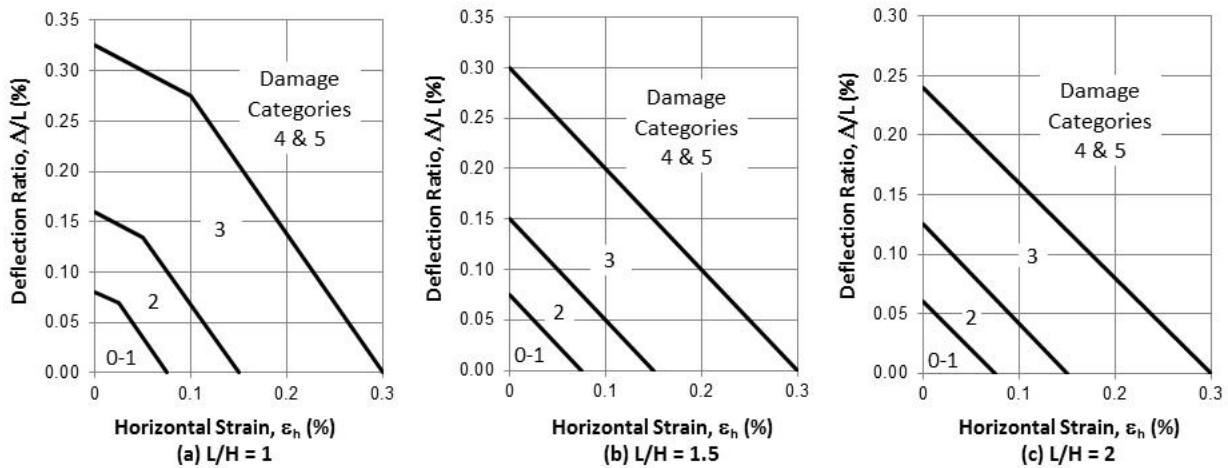


Figure 8: Interaction Diagrams of Potential Damage Assessment (method based on Burland, 1995)

## 4 RISK MITIGATION STRATEGIES

### 4.1 BASIC STEPS AND INVESTIGATIONS

Designing retaining systems to mitigate excavation induced deformations and potential damage to adjacent structures is essentially a performance based design approach. In other words, serviceability usually governs as unacceptable deformations are likely to occur well before collapse of the retaining system is reached. Even in cases where spectacular wall collapse has occurred, sufficiently large deformation should have rung alarm bells such that the collapse should have been avoided.

The basic steps of risk mitigation for excavation induced movements are as follows:

- Understanding the excavation requirements and condition of adjacent structures.
- Understanding the geology, hydrogeology and variability of the site by conducting adequate site investigations.
- Understanding material behaviour and properties associated with the excavation, then formulate appropriate geotechnical models.

- Assessing the potential excavation induced ground movements and potential impact to the adjacent structures.
- Assess the condition of the adjacent structures (dilapidation survey and structural assessment), and assess tolerable deformation limits.
- Design excavation support and mitigation measures to limit displacements to within tolerable limits. Contingency measures should be considered as part of the design.
- Specify appropriate construction methodology to match the design intent.
- Develop and implement an instrumentation and monitoring plan to validate excavation performance, and to provide early warnings system. The monitoring plan should include threshold values for review and the likely response actions. It is important that the various parties (geotechnical designer, structural designer, contractor and project owner) have a clear understanding of their roles and responsibilities in relation to the instrumentation and monitoring plan, and response actions.
- If considered appropriate, the Observational Method (Peck, 1969 & Nicholson et al, 1999) may be adopted as part of the design process which may result in more economic design and/or to reduce risk. The Observational Method (OM) is particularly useful in situations involving many uncertainties and assumptions. Some engineers use the term OM too loosely to mean monitoring performance during construction. It must be stressed that conducting instrumentation and monitoring alone does not constitute the use of the OM. The OM is not a “suck- and-see” approach; it requires rigorous assessment of unfavourable conditions and deviations from the adopted design conditions, sensitivity assessment and development of contingency measures as part of the design, and a preparedness to implement planned contingency measures during construction based on rigorous assessment of the monitoring results and recalibration of the design model as required.

In relation to site investigations and understanding of material properties and behaviours, the author believes that significant improvement should be made to current practice. It is very common to find that sophisticated numerical analyses are performed on deep excavation and retaining wall projects using presumptive shear strength parameters  $c'$ ,  $\phi'$ , stiffness  $E_u$  and  $E'$  (sometimes using strain hardening or more advanced constitutive models), and at rest earth pressure coefficient  $K_0$ , that are based on nothing more than correlations with Standard Penetration Tests (SPT). The assumptions are in fact based more on engineering judgement, as SPT results can only provide, at best, index correlations with undrained shear strength and undrained stiffness. Therefore, it is important that adequate site investigations be carried out, followed by proper formulation of geological, hydrogeological and geotechnical models. Appropriate testing should be carried out for selection of design parameters. For important projects, the use of pressuremeter or flat plate dilatometer testing should be considered in conjunction with laboratory testing such as direct shear or consolidated undrained triaxial testing with pore pressure measurements.

Risk of damage to adjacent structures needs to be considered carefully and tolerable deformation then decided before proceeding to the next step of design. Close interaction between the geotechnical and structural engineers is an essential element in managing risk and development of satisfactory solutions.

#### **4.2 DILAPIDATION SURVEY**

After identifying the adjacent structures that may be affected by the proposed excavation, a dilapidation survey of these structures is essential. This is because their type of construction and existing condition will impact on the assessment of tolerable deformation that may be induced by the excavation. Furthermore, dilapidation survey of adjacent structures is essential to avoid argument with the adjacent property owners regarding existing damage, thus reducing the risk of lawsuits concerning extent of damage caused by the excavation.

The dilapidation survey will form a reference for any claims by the adjacent property owners of damage caused by the excavation (if any), and should as a minimum, cover the area within the likely zone of influence of the excavation, with additional coverage to allow for uncertainties.

#### **4.3 DESIGN AND CONSTRUCTION METHODS TO LIMIT DEFORMATION**

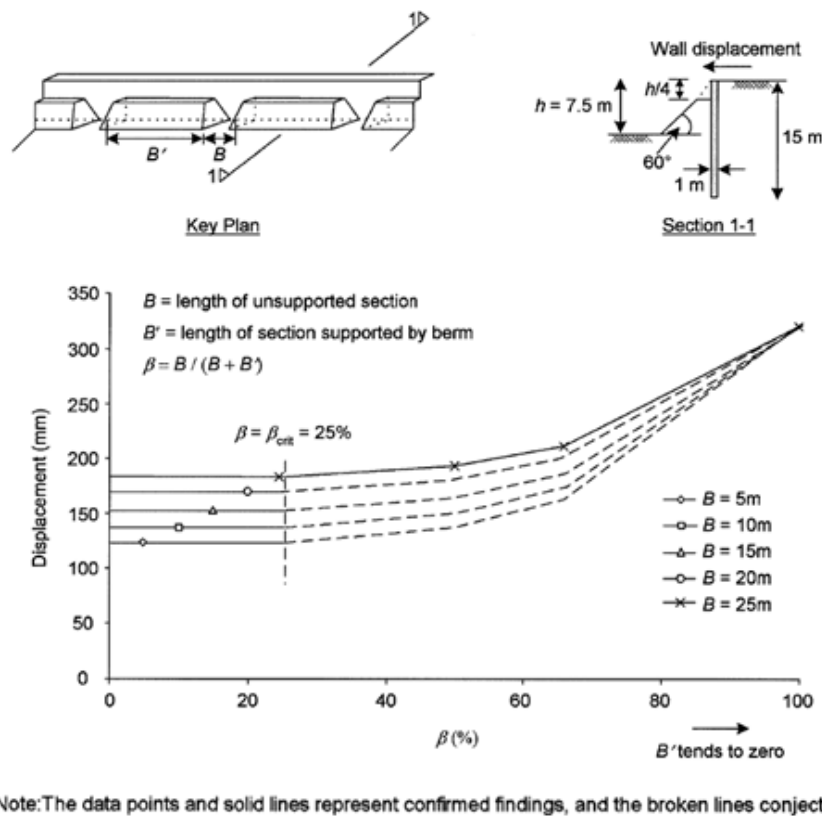
If there is to be an excavation, ground movement cannot be avoided, but precautions can be taken to either strengthen the adjacent structures and their support, or limit excavation movements, or a combination of both. Some of the methods available in limiting excavation induced movements are listed below:

##### **(a) Building Strengthening**

- Underpinning of shallow footings.
- Structural strengthening or reinforcement.
- Ground improvement beneath adjacent structures such as permeation grouting for granular materials, or jet grouting for cohesive materials.

**(b) Controlling Excavation Induced Movements**

- Increasing the excavation support stiffness (refer to Figure 4).
- Adopting top down construction.
- Performing ground improvement (e.g. reducing the active pressure and increasing the stiffness on the adjacent site by grouting, or increasing the passive resistance and stiffness for embedded walls below the excavation). Wong (2004) described the use of lime piling in soft clay for a basement excavation to increase the passive resistance for a tied back embedded wall in Penang, Malaysia.
- Reducing tie back anchor spacing, and/or increasing prestressing or preloading. Additional anchor or strut capacity may be considered to enable restressing or preloading as excavation progresses.
- Use of buttress walls (i.e. walls constructed normal to the excavation boundary) which is more readily achieved in diaphragm wall type construction.
- Use of temporary berms. Gourvenec and Powrie (2000) demonstrated the effectiveness of berms at different spacing by conducting 3D numerical analysis for a cantilevered wall in over-consolidated clay as shown in Figure 9.



Note: The data points and solid lines represent confirmed findings, and the broken lines conjecture

Figure 9: Wall Top Displacement at Centre of Unsupported Section against Degree of Berm Discontinuity, B, for Over-consolidated Clay (from Gourvenec and Powrie, 2000)

Sometimes, the use of tie back anchors may not be feasible due to space constraints or the lack of permission from adjacent property owners. An alternative and rather innovative prestressed wale system may be considered to overcome this problem; the system has been described by Park et al (2009) for projects in Korea (Figure 10), and Fernandes and Xavier (2011) for a project in Portugal (Figure 11).

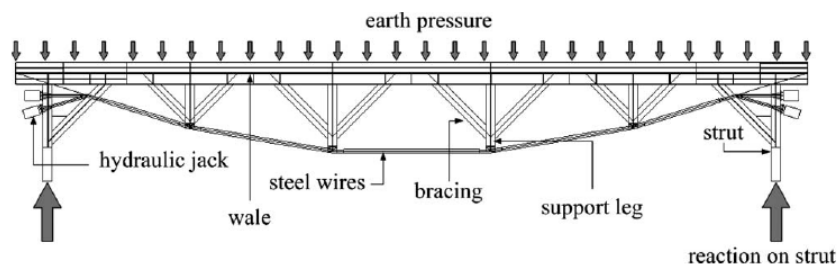


Figure 10: Prestressed Wale System (from Park et al, 2009 adopted for projects in Korea)

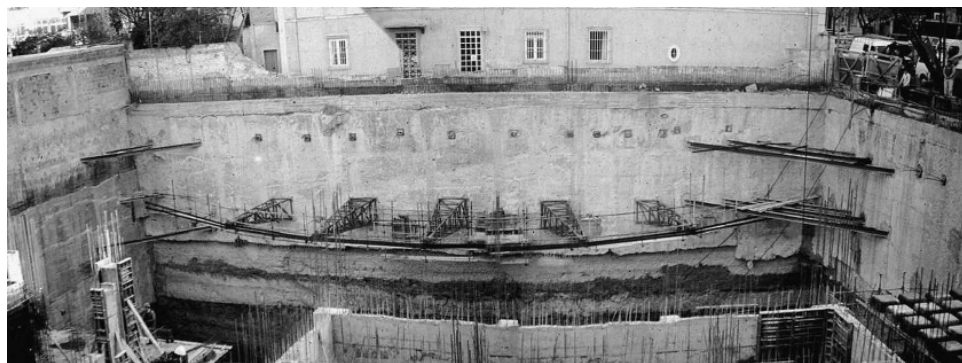


Figure 11: Prestressed Wale System (from Fernandes and Xavier, 2011 for a project in Portugal)

#### 4.4 INSTRUMENTATION AND MONITORING

As mentioned in Section 4.1, instrumentation and monitoring should form an integral part of the Observational Method in managing the risk of excavation induced movements. They are essential to provide an early warning system for the excavation retention and to safeguard adjacent structures and occupants. The commonly used instruments and their functions are summarised in Table 4. Not all types of instrumentation will be required in all circumstances; a combination of instruments should be devised depending on the problem being addressed and the sensitivity of the adjacent structures to movements.




Table 4: Commonly used Instrumentation and Purpose for Monitoring Excavation Induced Movements

Type of Instruments	Purpose
Visual inspections & Face Mapping	Visual inspection for signs of distress, mapping of excavated face to confirm adopted geology and geotechnical conditions. Observe seepage and other ground conditions which may adversely affect the design.
Displacement Markers/Survey Pins	Usually fixed on top and mid-depth of excavation or on adjacent structures close to footing level to measure settlement and lateral deformations.
Inclinometers	Installed behind or within wall to measure lateral deformation profile with depth.
Piezometers	Installed at various distances from the excavation to measure ambient groundwater level and potential drawdown in groundwater conditions as excavation progresses.
Crack meters or tale tells	Installed on existing structures across existing cracks or cracks which develop as a result of excavation to monitor changes in crack width and rate of increase in crack width.
Borehole Extensometers	In soils may be installed beneath the base of excavation to monitor the basal heave profile, or behind excavations in soft soils to monitor the settlement profile with depth. In rock may be installed horizontally from the excavation face to measure strains with distance from the excavation. This is particularly relevant where strain concentration may occur across defects such as sub-vertical joints or sheared zones.
Load Cells	Usually installed at the strut and/or waler against the shoring wall to measure the changes of load with each stage of excavation. This is important for certain situations (esp. soft clay sites) to validate that the shoring system is functioning properly, and to enable additional preloading if required.
Tilt Meters	Usually installed on the walls of adjacent buildings that are sensitive to tilt, or on the exposed face of the wall following partial excavation where there is no room for inclinometers to be installed behind the shoring.
Vibration Monitors	Measures peak particle velocities due to piling or rock excavation activities which may transmit adverse vibration to adjacent structures sensitive to vibration damage.

The frequency for readings and review to be undertaken should be specified in the Instrumentation and Monitoring Plan. Usually, it is important to take readings more frequently during excavation, and if considered appropriate, reduce the frequency following excavation when a trend of reducing deformation rate is established. Monitoring frequency should relate to the rate of excavation, for example, between 1 m to 2 m of excavation depth interval depending on the likely deformation predicted.

Review levels and typical response actions should also be specified as part of the Instrumentation and Monitoring Plan. The selection of review levels is dependent on factors such as accuracy of prediction, mechanism and consequence of deformations, and sensitivity of the adjacent structure to movements and distress. The review levels should also be related to design predictions and tolerable levels which may be different. In most cases, design predictions will be less than tolerable levels, but an exceedance of the design predictions should provide warnings to the designer that the design assumptions may need to be reviewed. Various terminologies are used in the industry, including trigger levels when certain review levels are reached such as Alert, Review, Warning, Limit, Maximum, and Action which are not always consistent. To avoid confusion, the author prefers to adopt a “traffic light” review system of Green, Amber and Red, and considers that appropriate “action” should be carried out at all review levels, even if it simply involves the regular review of movement trends within the Green level. Amber and Red levels are typically used to trigger warning notice or alarms, particularly if automated monitoring systems are employed. Typical review levels and response actions are shown in Table 5, but these must be developed by the designer on a project specific basis.

**Table 5: Typical Review Levels and Response Actions**

<b>Review Level</b>	<b>Typical Levels</b>	<b>Typical Response Actions at Various Trigger Levels</b>
Green 	Within 80% of design predictions.	<ul style="list-style-type: none"> <li>Continue work with on-going monitoring which should include regular review of the trend of deformation and assessment of whether design predictions are likely to be exceeded.</li> </ul>
Amber 	80% of design prediction, or if trend indicates tolerable values likely to be exceeded	<ul style="list-style-type: none"> <li>Increased monitoring frequency.</li> <li>Review design model and geotechnical parameters, and assess impact on design if required.</li> <li>Implement contingency measures to reduce risks if assessment indicates Red Level may be reached imminently.</li> </ul>
Red 	Tolerable values exceeded, or if damage is observed.	<ul style="list-style-type: none"> <li>Stop work immediately.</li> <li>Review impact on shoring and adjacent structures.</li> <li>Implement measures to cease movement.</li> <li>Implement remedial works if required.</li> </ul>

## 5 CASE STUDIES

Two case studies are presented briefly below to illustrate the application of some of the methodology discussed in this paper.

### 5.1 4-10 CAMPBELL STREET AND 710-720 GEORGE STREET, SYDNEY

The two sites are located adjacent to each other and over the Eastern Suburbs Railway (ESR) tunnels as shown in Figure 12.

RailCorp NSW required detailed assessment of excavation induced ground movements for both sites and their potential impact on the ESR tunnels. Only the 4-10 Campbell Street site will be discussed in this paper. The project involved basement excavation to about 15 m below ground level, and at the closest point of the excavation to the ESR tunnels is about 10 m as shown in Figure 13 (near inclinometer position). The proposed excavation adjacent to the ESR tunnels is entirely in rock of at least Class III Sandstone quality based on the rock classification system for Sydney rock by Pells et al (1998).

**EXCAVATION INDUCED GROUND MOVEMENTS AND RISK MANAGEMENT STRATEGIES  
WONG P.K.**



ESR Tunnels

Figure 12: Site Location Plan (4-10 Campbell Street is coloured red)



Figure 13: Plan and Section Closes to ESR Tunnel (4 – 10 Campbell Street, Sydney)

Three dimensional FEA was carried out, and included previous excavation effects on rock stresses associated with the nearby World Square, Roden Cutler building, Regis Tower, Capitol Hotel, and the proposed excavation of the adjacent site at 710-720 George Street. The excavation induced movement at the rock face adjacent to the tunnel was assessed to be 0.5 mm per metre depth of excavation. At the tunnel face, the vertical and horizontal displacements were assessed to be 1 mm and 4 mm respectively, with an induced increase in bending moment in the tunnel lining of about 153 kNm/m. The assessed potential damage was assessed to be negligible to very slight, but because of the importance of the adjacent infrastructure, a detailed risk assessment was carried out in accordance with the RailCorp NSW risk assessment procedure.

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An instrumentation program (comprising installation of a series of 3D survey markers, inclinometers and sub-horizontal borehole extensometers installed in the rock adjacent to the excavation, tunnel mapping and tell-tales across existing cracks in the tunnel) was implemented. The excavation was completed successfully, with measured results very close to design predictions. Eight additional hairline cracks appeared in the ESR tunnel lining at the completion of excavation and three of the 70 existing cracks mapped as part of the dilapidation survey were found to have increased in length. It was not conclusive that the cracks were caused entirely by the excavation, as some of the new cracks were beyond the excavation site and were observed to be connected to holes drilled by others. In any case, the movements and cracks observed did not cause any concern to RailCorp NSW.

The excavation performance was consistent with design predictions, but it was the risk assessment process, including the instrumentation and monitoring program, which gave RailCorp NSW the confidence to allow the developer to proceed with the excavation.

### 5.2 LUXE APARTMENTS, WOOLLOOMOOLOO

This site is situated at 60-72 Sir John Young Crescent, Woolloomooloo, and is adjacent to a heritage building to the north and a row of brick terrace houses to the south as shown in Figure 14. The eastern boundary of the site is adjacent to Crown Street. The neighbouring buildings were investigated to be at least partly supported by high level foundations. These masonry buildings were assessed to be susceptible to damage caused by excavation induced ground movement. The Cross-City Tunnel, which lies to the west of the site below Sir John Young Crescent, was also of major concern in terms of excavation impact.

The site is underlain by a palaeochannel infilled with alluvial sand and clay deposits with rock depths ranging from 8 m to 18 m along the site boundaries. The groundwater level at the site is high and the alluvial deposits are susceptible to consolidation settlement due to dewatering.

The proposed development comprised two 9 storey buildings with a combined, split-level three basement, and the site conditions provided major concerns in relation to the excavation support. In addition, there were difficulties in negotiating access with the southern neighbouring owner for the installation of tie back ground anchors to support the proposed secant pile wall shoring system.

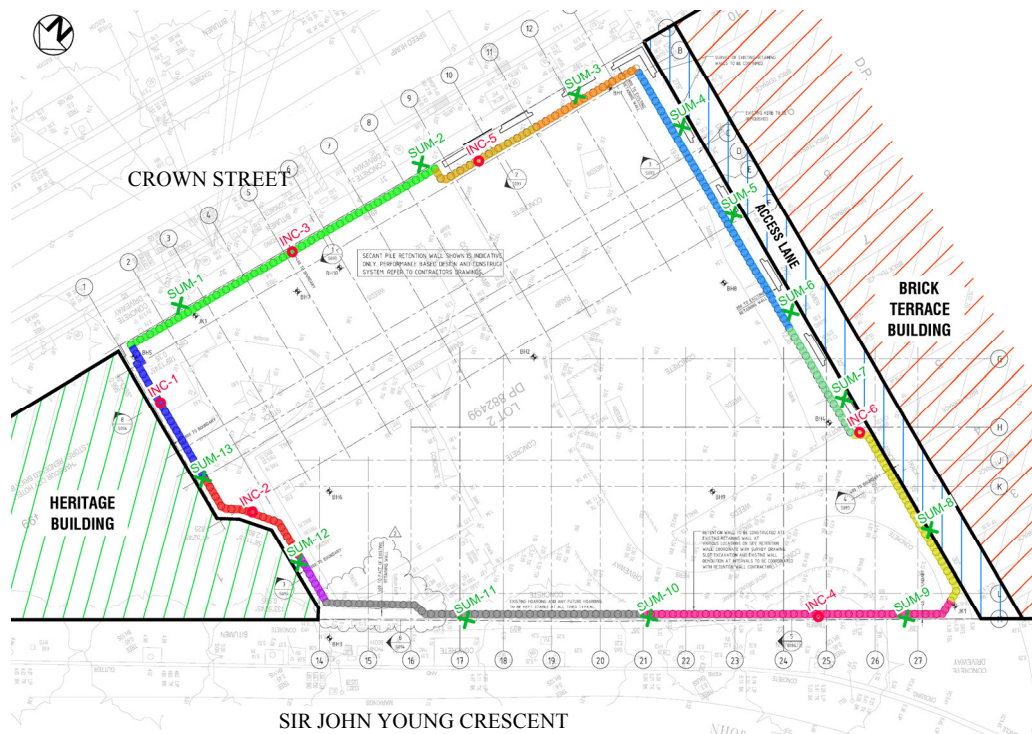


Figure 14: Luxe Apartment Site Plan showing Instrumentations (INC = Inclinometers, SUM = Survey Markers)

In order to assess the risks of excavation induced movements, devise different retention options to reduce risks and communicate the results to the developer, analyses were carried out including the assessment of horizontal strain and deflection ratios for the following excavation support options:

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- Internal bracing system provided by a large steel grillage running around the perimeter of the site, with rock anchors to be installed in the southern and eastern boundary.
- Top-down construction, so that the internal bracing is provided by the permanent floor slabs constructed during excavation.
- Strutting down to a dead man piles was eliminated as a possible shoring system because of the depth of rock and soft ground conditions.

Analysis results were plotted on potential damage category charts (refer to Section 3.3). As an example, the assessed results for the shoring adjacent to the terrace houses are shown in Figure 15, which had computed horizontal strains of 0.14% and 0.05% for the grillage system and top-down construction system, respectively.

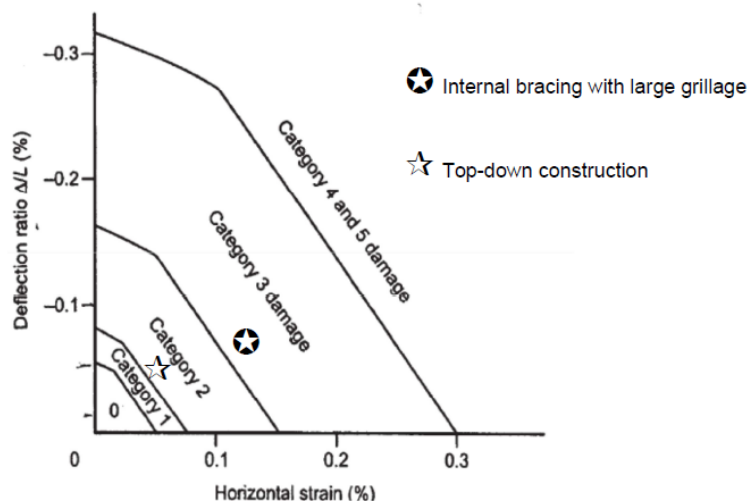


Figure 15: Results of Potential Damage Category Assessment (Terrace Houses to the South)

The assessment enabled the client to make informed decisions in selecting the appropriate excavation support methodology based not only on the cost of the system, but on risks and potential cost of repairs and to avoid the potential for litigation. A top-down construction method was adopted by the construction company. To further reduce the risk of excavation induced settlement of the building on the northern boundary which has high footing loads, compaction grouting was carried out along this boundary. An instrumentation and monitoring program, comprising installation of several inclinometers behind the shoring piles and survey markers (see Figure 14), was implemented. Excavation level B2 was reached in July 2013 with the final basement excavation scheduled for completion in August 2013.

At the time of writing, very little movement has been recorded and the top down shoring system enabled the two buildings each including 9 levels to be constructed above ground level at the same time as the excavation was progressing below.

## 6 CONCLUSIONS

A brief overview of methods for assessing excavation induced movements has been provided in this paper, together with a well-established method for assessing the potential damage category that may be caused to adjacent structures. Possible mitigation measures are discussed and recommendations are given on site investigation, dilapidation survey, instrumentation and monitoring requirements in the context of risk management. The applications of risk management strategy are illustrated with two brief case studies in this paper.

Adequate assessment of the possible extent of movements and potential adverse impact on adjacent structures is essential as it is not possible to eliminate excavation induced movements. Improvements should be made to site investigation practice, to enable better assessment of material strengths, in situ stress states and stiffness parameters. Non-linear stress strain behaviour should be considered, whether by the use of secant modulus at the estimated operating strain levels, or the use of more sophisticated non-linear numerical analysis methods.

Various methods of reducing excavation induced movements are available and these should be considered collaboratively with the structural engineer and the developer. Instrumentation and monitoring, together with a plan of review levels and possible response actions, is an essential part of the risk management strategy for excavation retention works.

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