Challenges of Tunnelling in East - West Metro Kolkata Tunnelling Below Colvin Court Building

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1. Introduction :

Tunneling for East-West Metro Line faces technological challenges in undertaking heavy underground construction in congested city with narrow Right Of Way. Underground tunneling in soft clayey soil imposes considerable technical concerns. Proper control of ground settlement and management of distress in buildings are demanded in such situations. Proper coordination between TBM operation & monitoring of distress/settlement above ground is key to success.

The West Bound Tunnel Boring Machine hadto pass 11 m below Colvin Court Building which is more than 91 years old 4 storied brick masonary building situated in Howrah Maidan Church Road area. Colvin court is being used for quarters of Railway Officers. This Technical paper covers the aspects of prediction of Ground Movement and Structure Impact Assessment for Colvin Court building due to Tunnel induced ground

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movements and the comparison of actual ground movements. The paper also covers various mitigation measures taken to ensure safety during tunneling operation.

2. Details of the Tunneling below Colvin Court

Internal Diameter of tunnel = 5.55m

RCC Precast liner thickness = 275mm

External Diameter of tunnel = 6.10m

Tunnel Boring Machine : Earth Pressure Balance Machine of Herrenkonnect A.G. make; Excavated diameter of soil face : 6.35m

Length of tunnel crossing below building : 25m

Average Depth of Tunnel crown below building : 11m.

3. Settlement Analysis

a) Tunnel induced settlement

During the process of tunnel excavation, the supported ground around the tunnel moves inwards as the stress relief is taking place. This movement is due to different factors, such as:

(I) Face loss due to the decompression at the tunnel face: The rotating cutters of the shield remove material from the tunnel face and at the same time a confinement pressure is applied. In spite of the confinement, the ground tends to protrude out of the face from a zone of influence ahead and around the tunnel face. This gives rise to the 'face loss'.

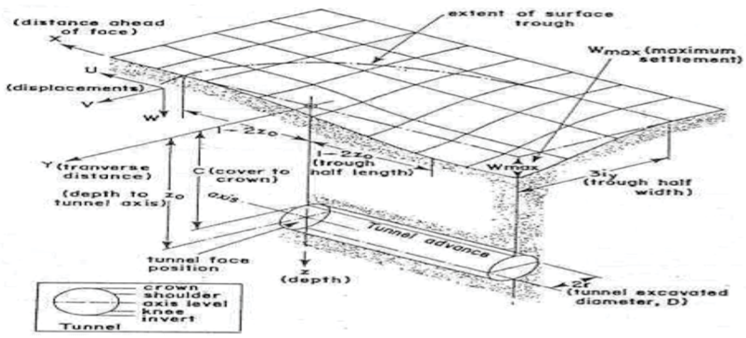
(ii) Shield Loss : Shield loss due to the excavation of a slightly oversized tunnel hole at the front of the shield in order to ease the advance of the shield itself, to reduce the chance of the shield being stuck and to allow steering the shield in curved alignment.

These factors can be increased by an eventual overcutting. Hence, the excavated cavity can converge radially. The amount of convergence depends upon the rate of deformation of the soil with respect to the rate of tunnel advance. Consequently, the excavated perimeter may close partially or completely onto the shield.

(iii) Tail void lossdepends on the pressure, volume and accuracy of the injection filling the tail void around the installed tunnel lining. A further increase of the radial convergence can be observed. Long-term loss due to the lining deformations after the grouting, caused by the transfer of the overburden pressure (unless time-dependent soil behavior is observed, this component is quite negligible). The sum of the two radial displacements is termed 'radial loss'. The sum of the face loss and the radial loss gives the overall 'volume loss', VL, resulting from the excavation of the tunnel.

The volume loss (or ground loss) is hence defined as the volume of soil that has been excavated in excess of the theoretical design volume of tunnel excavation. It is expressed as a percentage of the final tunnel volume (i.e., m^3 per meter of tunnel advance). Due to the volume loss, a settlement trough is developed at the surface while tunneling is proceeding (refer to Figure 1). The shape of the settlement trough is a function of the nature of the soil. In undrained conditions the volume loss represents the volume of the settlements trough at the surface.

Empirical methods are normally applied to calculate the 'greenfield' settlement trough induced by tunneling (that is settlement induced on a free field where the stiffening effect of buildings and their foundations is neglected).



The settlement trough transversely to the tunnel axis can be approximately expressed by an inverted Gauss distribution curve. Its dimension and shape can be defined mainly through two parameters: the volume loss (VL) and the trough width parameter (K).

Hereafter, the empirical method to determine the vertical displacements induced by the tunnel excavation in transversal sections is summarized.

b) Final settlement in cross section
Vertical displacement

Following the Design Specification report, O'Reilly and New (1982) methodology is proposed to calculate surface settlement. They developed a Gaussian model by making the assumptions that the ground loss could be represented by a radial flow of material toward the tunnel and that the trough could be related to the ground conditions through an empirical "trough width parameter" (K). In this report, Greenfield settlement method, proposed by Attewell et al (1986) and Rankin (1988) which follows the same methodology and formula, was considered as calculation method.

Attewell et al (1986) and Rankin (1988) summarized the widely used empirical approach to the prediction of immediate surface and near surface vertical settlement due to tunneling.

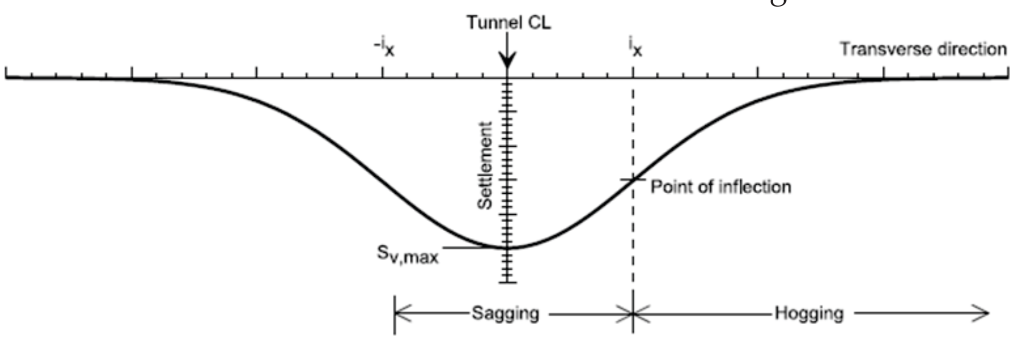


Fig-2 : Transverse settlement trough

The settlements profile at the ground surface in a transverse section well behind the tunnel face, (where the maximum displacement due to tunneling is already achieved) is shown in Figure.

The settlements profile at the ground surface in a transverse section well behind the tunnel face, (where the maximum displacement due to tunnelling is already achieved) is shown in Figure 2 and is expressed for a single tunnel by the following formula:

$$S = S_{\max} \exp\left(\frac{-y^2}{2i^2}\right) = \frac{V_s}{\sqrt{2\pi} \cdot i} \cdot \exp\left(\frac{-y^2}{2i^2}\right) = \frac{V_L \cdot A}{\sqrt{2\pi} \cdot i} \cdot \exp\left(\frac{-y^2}{2i^2}\right)$$

where:

S is the settlement of a generic point at the generic transversal distance 'y' from the tunnel axis;

Smax is the maximum settlement above the tunnel axis;

Vs is volume of the settlement trough per meter of tunnel advance [m^3/m], defined as apercentage VL of the theoretical tunnel section A (=Dexcav2/4) of the tunnel;

'i 'is the trough width parameter expressed as i=k.z0, being 'k' a dimensionless constant, depending on soil type and 'z0' the depth of the tunnel axis below the surface;

'I' represents the position of the inflection point of the settlement curve, where the trough has its maximum slope; it separates the sagging from the hogging zone of the curve;

Horizontal Displacements

A method for predicting horizontal surface displacements induced by O'Reilly and New (1982) assuming that vectors of movements near the ground surface were directed towards the tunnel axis. Sh=y/zo x sv

c) Trough width parameter

The trough width parameter 'i' depends upon the dimensionless constant 'k' which is an empirical factor function of the soil type.

A number of empirical formula has been proposed by many authors in order to define the settlements trough width.

For most purposes, the linear expression shown hereafter can be applied: i=k.Zo

4. Assessment of Building Damage:

Three categories of building damages can be considered that affect:

(i) visual appearance or aesthetics,

(ii) Serviceability or function and

(iii) Stability.

As foundation movements increase, damage to a building will progress successively from (i) through (ii) to (iii). It is only a short step from these three broad categories to the detailed classification given in Table 1. This defines six categories of damage, numbered 0 to 5 in increasing severity. Normally categories 0, 1 and 2 relate to "aesthetic" damage, 3 and 4 relate to "serviceability" damage and 5 represents damage affecting "stability". It was first put forward by Burlandef a1 (1977), who drew on the work of Jennings and Kerrich (1962), the UK National Coal Board (1975) and MacLeod and Littlejohn (1974). Since then it has been adopted with only slight modifications by BRE (1981 and 1990), the Institution of Structural Engineers, London (1978, 1989, 1994 and 2000) and by the Institution of Civil Engineers and BRE again in Freeman et al(1994).

Table1: Classification of visible damage to walls with particular reference to ease of repair of plaster and brickwork or masonry

Category of damage	Normal degree of severity	Description of typical damage (ease of repair is in bold type) *Note: Crack width is only one factor in assessing category of damage and should not be used on its own as a direct measure of it.*
0	Negligible	Hairline cracks less than about 0.1 mm wide
1	Very slight	**Fine cracks that are easily treated during normal decoration.** Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1 mm
2	Slight	**Cracks easily filled. Redecoration probably required. Recurrent cracks can be masked by suitable linings.** 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 5 mm.
3	Moderate	**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.** Doors and windows sticking. Service pipes may fracture. Weather-tightness often impaired. Typical crack widths are 5–15 mm or several > 3 mm.

4	Severe	**Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.** Windows and door frames distorted, floor sloping noticeably[1]. Walls leaning[1] or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15–25 mm, but also depends 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 25 mm, but depends on the number of cracks.

[1] 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.

5. Concept of Limiting Strain

The importance of this development is that (limiting tensile strain) can be used as a serviceability parameter that can be varied to take account of differing materials and serviceability limit states.

Boscardin and Cording (1989) developed this concept of differing levels of tensile strain. Seventeen case records of damage due to excavation-induced subsidence were analyzed. A variety of building types was involved and they showed that the categories of damage given in Table 1 could be broadly related to ranges of limiting tensile strain. These ranges are tabulated in Table 2. This table is important as it provides the link between estimated building deformations and the possible severity of damage.

Table 2: Relationship between category of damage and limiting tensile strain (limiting tensile strain) (after Boscardin and Cording, 1989)

Category of damage	Normal degree of severity	Limiting tensile strain (ε_{lim}) (%)
0	Negligible	0–0.05
1	Very slight	0.05–0.075
2	Slight	0.075–0.15
3	Moderate*	0.15–0.3
4 to 5	Severe to very severe	> 0.3

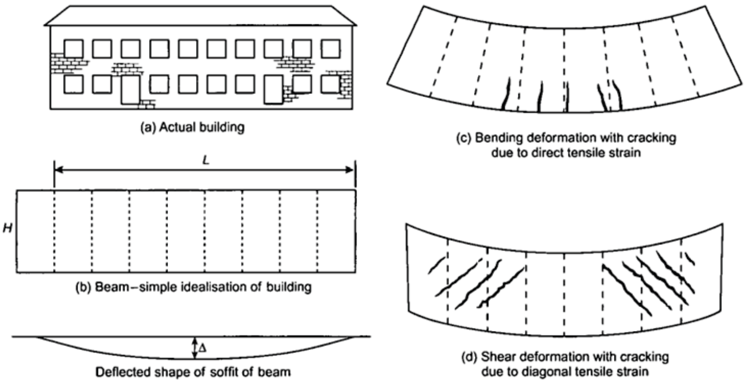
It should be noted that above damage classifications are valid for building in good condition, i.e. without initial defects. While classifying building with initial defects some additional factor of safety should be applied on above tensile strain criteria considering importance of building.

6. Strains in Rectangular Beams

Burland and Wroth (1974) and Burland et al (1977) used the concept of limiting tensile strain to study the onset of cracking in simple weightless elastic beams undergoing sagging and hogging modes of deformation. This simple approach gives considerable insight into the mechanisms controlling cracking. Moreover, it was shown that the criteria for initial cracking of simple beams are in very good agreement with the case records of damaged and undamaged buildings undergoing settlement. Therefore, in many circumstances, it is both reasonable and instructive to represent the facade of a building by means of a simple rectangular beam.

Sagging and Hogging

Figure at page 2 illustrates the approach adopted by Burland and Wroth (1974), where the building is represented by a rectangular beam of length L and height H. The problem is to calculate the tensile strains in the beam for a given deflected shape of the building foundations and so obtain the sagging or hogging ratio Δ/L at which cracking is initiated. Little can be said about the distribution of strains within the beam unless its mode of deformation is known. Two extreme modes are bending only about a neutral axis at the center (Figure c below) and shearing only (Figure d below). In bending only, the maximum tensile strain occurs in the bottom extreme fiber, which is where cracking will initiate. For shear only,the maximum tensile strains are inclined at 45°, initiating diagonal cracking. In general, both modes of deformation will occur simultaneously and it is necessary to calculate both bending and diagonal tensile strains to ascertain which type is limiting.



Timoshenko (1957) gives the expression for the total mid-span deflection Δ of a centrally loaded beam having both bending and shear stiffness as:

$$\Delta = \frac{PL^3}{48EI}\left(1 + \frac{18EI}{L^2HG}\right)$$

Where E is Young's modulus, G is the shear modulus, I is the second moment of area and P is the point load. The above equation can be re-written in terms of the deflection ratio Δ/L and the maximum extreme fibre strain bmaxas follows:

$$\frac{\Delta}{L} = \left(\frac{L}{12t} + \frac{3I}{2yLH}\cdot\frac{E}{G}\right)\varepsilon_{b\max}$$

where t is the distance of the neutral axis from the edge of the beam in tension. Similarly, for the maximum diagonal strain dmax, The above equation becomes:

$$\frac{\Delta}{L} = \left(1 + \frac{HL^2}{18I}\cdot\frac{G}{E}\right)\varepsilon_{d\max}$$

The influence of horizontal strain

The ground surface movements associated with tunnelling not only involve sagging and hogging profiles but significant horizontal strains as well. Boscardin and Cording (1989) included horizontal tensile strain hin the above analysis using simple superposition, i.e.

it is assumed that the deflected beam is subjected to uniform extension over its full depth. The resultant extreme fibre strain br. is given by: $\varepsilon_{br} = \varepsilon_{bmax} + \varepsilon_h$

Horizontal strain at the surface is obtained by deriving horizontal displacements.

$$\varepsilon_h = \frac{dS_H}{dy} = \frac{S_{max}}{z_0} \cdot \left(1 - \frac{y^2}{i^2}\right) \cdot e^{-\left(\frac{y^2}{2i^2}\right)}$$

In the shearing region, the resultant diagonal tensile strain drcan be evaluated using the Mohr's circle of strain. The value of dris then given by:

$$\varepsilon_{dr} = \varepsilon_h \left(\frac{1-\nu}{2}\right) + \sqrt{\varepsilon_h^2 \left(\frac{1+\nu}{2}\right)^2 + \varepsilon_{dmax}^2}$$

7. Present condition of the Colvin Court Building

A new Building condition survey was conducted to assess the present condition of the Colvin court building. A comparison of the building condition was done to study the amount of deterioration in past 5 years.

The building condition survey of the Colvin court was conducted in the year 2010. Since the survey and assessment was done more than 5 years agoand significant time period has passed, it is evident from the structure monitoring reports of Howrah Maidan Station and Cross Over that the buildings have undergone further settlement even if no construction related activities were taken up during last 5 years. The settlements were predominantly attributed to primary/secondary consolidation of soil, seasonal variation of water table and extraction of ground water by private bore well owners as well as by various authorities. There is sufficient ground to presume, on the similar line, that the Colvin Court should have undergone further settlement during this period of time resulting into increased strain into the building.

It is also worth noting that this building has undergone 3 earthquakes recently. Considering the age and condition of building the impact of earthquake cannot be ignored on the structure and there is likelihood that the impact of Earthquake must have further deteriorated the condition of building.

Considering the above points and latest building condition survey report, it can be concluded that Colvin Court building has undergone further deterioration due to combined effect of multiple reasons enumerated above.

8. Assessment Of Tensile Strains In Structures Due To Ground Movements

Earlier calculations of anticipated surface settlement for 1.5% volume loss were validated and found to be in order.

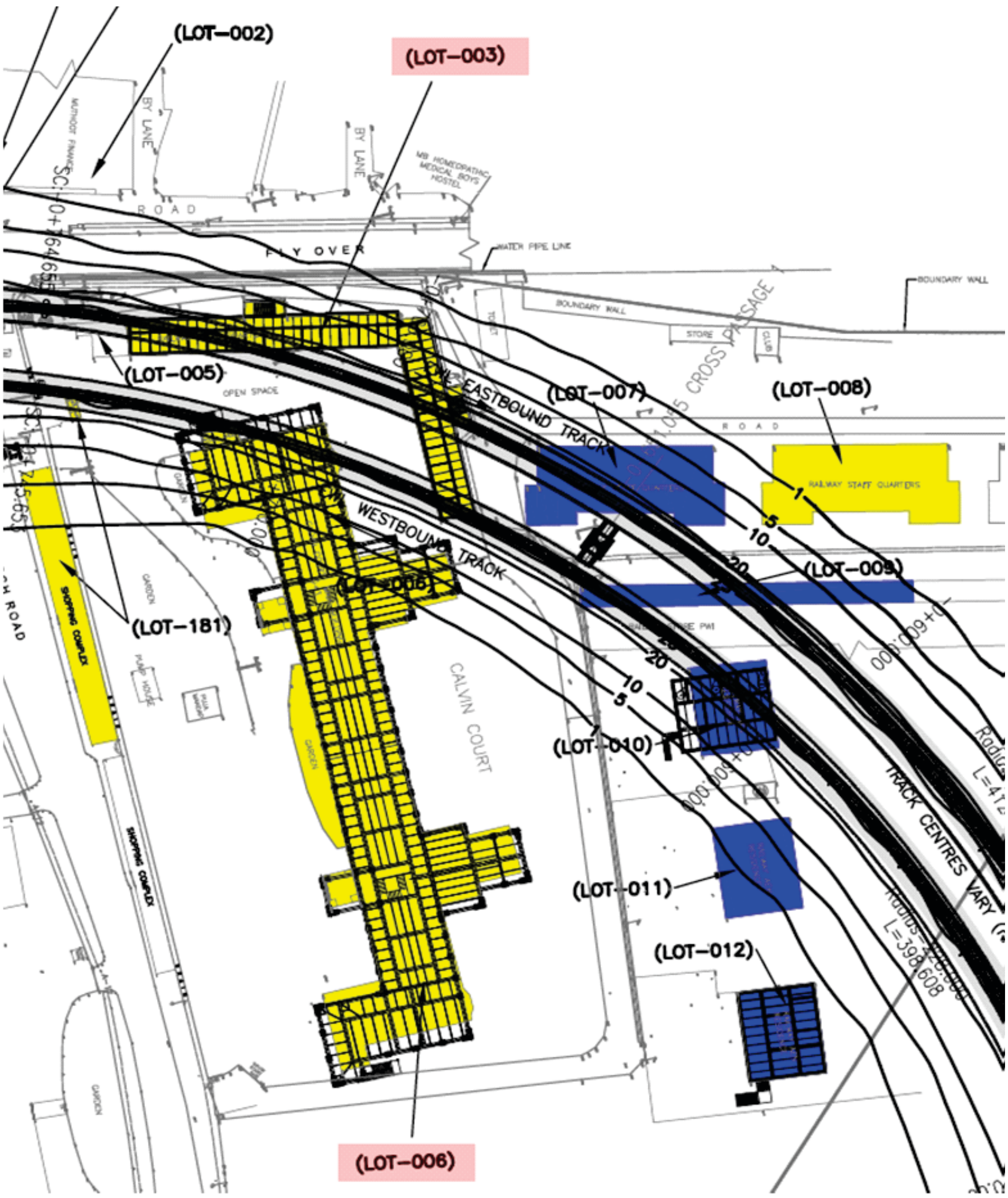
As confirmed by experts, since the initial drive is complete, surface settlements will decrease due to proper grouting methods, less breakdowns of TBM, improved mucking operations and volume loss would not be more than 1.5%. The tensile strains for Colvin Court Building and Railway Staff Quarters has been calculated due to excavation of West-Bound Tunnel only considering a maximum anticipated surface settlement of 25 mm.

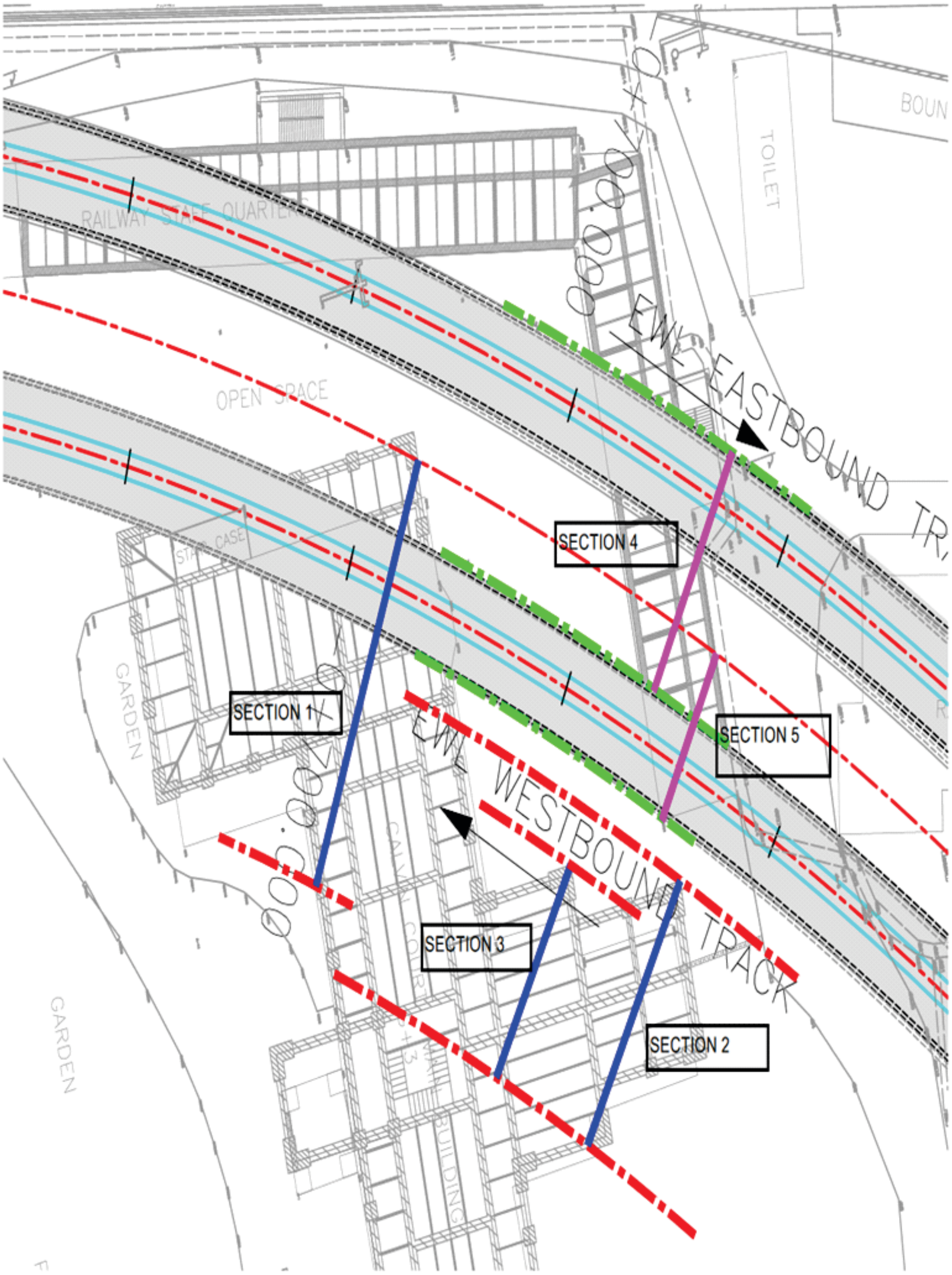
Critical sections were selected for tensile strain calculations. Summary of calculated tensile Strains at critical sections for anticipated surface settlements due to tunneling operation are presented in Table 4.

Table 3: Summary of Risk Categories of Analyzed Buildings for Anticipated Surface Settlements

Building	Section analyzed	Tensile Strains (%)	Category
Colvin Court	Section - 1	0.093 **	Slight
	Section - 2	0.103	Slight*
	Section - 3	0.096	Slight
* approximately 0.1% ** As this particular section contains the corner of the building and also skewed to the perpendicular direction of tunnel, therefore it is considered under slight category			

Considering current building condition and age of building, buildings without tunnel operation can also be categorized under slight category. Due to TBM operation an additional surface settlement of the order of 25mm is anticipated. This settlement may create an additional tensile strains of the order of 0.1%. In case surface settlement is more than 25mm, additional tensile strain in building would be more than 0.1 %. Considering initial defects and age of buildings, if additional tensile strains are more than 0.1%, building will require additional contingency measures.





RAILWAY STAFF QUARTERS
OPEN SPACE
STAIR CASE
GARDEN
GARDEN
TOILET
BOUN
EWL EASTBOUND TRA
EWL WESTBOUND TRACK
SECTION 1
SECTION 2
SECTION 3
SECTION 4
SECTION 5

9. Suggestions after Assessment

Considering current building condition and anticipated surface settlement of 25mm for ground volume loss of 1.5% both, impact on Colvin Court building can be categorized under "Slight Category".

Although the estimated additional tensile strains for Colvin Court & Railway Staff Quarter buildings are in order of 0.1%, but present condition of building is not good. Considering the present condition of the building higher tensile strains may increase risk to building damage. Considering that surface settlement may increase, additional mitigation measures were adopted to avoid heavy damage in both the buildings.

Considering condition, age of buildings and volume loss of about 1.5 %, following precautionary measures were recommended to ensure safety of buildings:

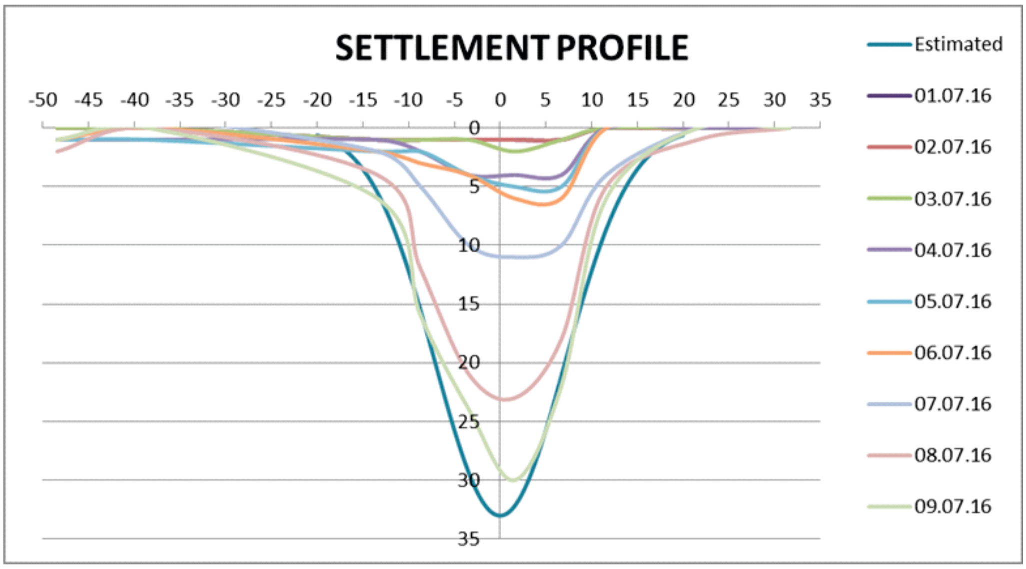
- Vacation of these buildings prior to tunneling.
- Slabs, arches, columns etc. were supported using staging (such as cup lock system, use of tie rods, temporary or permanent propping, wrapping of columns etc.) upto 25 metres on either sides of tunnel.
- Vertical propping at critical load transfer locations as an alternative load path. These supports were provided with jacking facility to ensure a positive support at all times. The protective measures were implemented before the TBM cutter head reached within 25m distance from the building.
- Tunnel boring was done as fast as possible below these critical areas and measures taken to reduce the ground volume loss.
- The operation of TBM shouldn't be stopped while the TBM is below these critical buildings.
- Face pressure for the TBM boring were calibrated in a very interactive way to minimize the ground settlements.

10. Methodology and Protective Measures adopted :

1.0	**Building condition survey and assessment of health of the structure –** - Visual inspection to assess the condition of the building - Detailed high quality digital photographic documentation showing all major distress of the building - Distress mapping of existing building including documentation of distress like cracks , delamination, geometric deformation of structural component and out of plumb etc. - Inspection and assessment of all components of building like structural system including floor , exterior walls, windows, frames, exterior doors and frames, stairs, Balconies, roof coverings, roof drainage etc.
2.0	**Structure Impact Assessment to classify damage category for the buildings due to tunnelling**
3.0	**Pre-emptive measures such as repairing of cracks before start of tunneling under the building**
4.0	**Evacuation of building prior to tunnelling**
5.0	**Widening of foundation of affected portion**
6.0	**Grouting the first layer below the foundation in order to make it stiffer**
7.0	**Vertical propping at critical load transfer locations (doors , windows , staircase etc.) as an alternative load path. These supports were provided with jacking facility to ensure a positive support at all times.** **The protective measures were implemented before the TBM cutter head is within 25m distance from the building.**
8.0	**Application and maintenance of proper face pressure during tunneling operation**
9.0	**Injecting bentonite slurry through the shield to minimize ground relaxation**
10.0	**Reducing the time between ring excavation and ring building (settlements in clay are basically a time dependent phenomenon)**
11.0	**Tunnels bored as fast as possible below these critical areas alongwith above mentioned measures to reduce the ground volume loss. Continuous operation of TBM without any stoppage while the TBM was below these critical buildings**
12.0	**Post tunneling repairs to the building**

11. Execution of Tunneling below Colvin Court Building:

Tunneling below Colvin Court has been executed from 25.06.2016 to 03.07.2016 and the settlement profile is compared with predicted settlement as below :



The maximum predicted settlement is 32mm . The actual observed settlement at Array E1 was 30mm.

12. Conclusion

The tunnelling has been successfully done under the colvin court building . All the shifted residents have been brought back after repairing the minor cracks . This has motivated the Team/KMRCL and Railway Engineers to face the challenges ahead – crossing below the BankimSetu, Howrah Yard & Platforms, office of DRM/HWH – more than 100 years old masonry building and finally the mighty river Hooghly.