Estimation of Settlement and Vibration on the Surface Due to the Construction of East-West Metro Tunnels in Kolkata, India

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Abstract. Kolkata, located on the bank of River Ganges in India, is considered to be one of the most densely populated cities in the world. It has an underground metro rail connection cutting across the city running predominantly in the north-south direction. Another underground metro rail connection is being built in the east-west direction connecting the heart of the city to the railway stations and the airport. As a part of the project, twin tunnels of diameter 6.1 m and 15 m apart are being constructed below the Ganges and through the most congested part of the city. The tunnels are located at a depth between 17 m and 24 m below the ground surface. The soil layers in Kolkata essentially consists of soft clays (clayey silt to silty clay) deposited by the river with layers of dense sand at much greater depth. Several old heritage structures have been identified along the route of the E-W metro. These structures are located between 19 m and 33 m from the centerline of the tunnels. Damage to these heritage buildings during the construction of the tunnels due to excessive settlement and/or vibration is a concern. Both static and dynamic finite element analyses have been performed to estimate the settlement and the vibration at those heritage structures due to the tunnel constructions. The results of the analyses are compared with the reported case histories of tunnel constructions worldwide on soft soils and with the empirical methods proposed by Mair et al. and FHWA to estimate the ground settlement due to the proposed tunnel construction.

1 Introduction

Kolkata, the capital of the Indian state of West Bengal, is located on the bank of River Ganges in India. This cosmopolitan city is considered to be one of the most densely populated cities in the world. It has an existing underground metro rail connection cutting across the city running predominantly in the north-south direction. Another underground metro rail connection (E-W metro) is being built in the east-west direction connecting the business district (Brabourne Road) of the city to the main railway station (Howrah Station) across the River Ganges and the international airport. Figure 1 shows the route of the E-W metro. As a part of the project, twin tunnels of 6.1 m in diameter and 15 m apart (centre to centre) are being constructed below the River Ganges and through the most congested part (Brabourne Road) of the city. Figure 2

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shows a ground view of the Brabourne Road, the main business district of Kolkata. The tunnels are located at a depth between 17.6 m and 23.7 m below the ground surface (KMRC 2015). Several old heritage masonry structures have been identified along the route of the E-W metro. These heritage structures are located along the Brabourne Road/JN Road and shown by a circle in Fig. 1. These structures are located between 22 m and 33 m from the centerline of the tunnels. All most all of these old heritage masonry structures are on raft foundation with the bottom of the raft located about 2 m below the ground level. Damage to these heritage buildings during the construction of the tunnels due to excessive settlement and/or vibration is a major concern. The newly constructed high rise concrete reinforced buildings located along the route of the E-W metro are on pile foundations and considered to be less susceptible to damage due to the tunnel construction. A study is undertaken to estimate the ground settlement and the vibration at the ground level during the construction of the tunnels. This paper summarizes the outcome of this investigation.

Fig. 1. E-W metro route & the study area.

Fig. 2. Ground view of braboune road (study area).

2 Subsoil Layers and Their Properties

The subsoil layers in the vicinity of the E-W metro and the heritage structures in Kolkata are obtained from the borehole logs and essentially consist of soft clays (clayey silt to silty clay) deposited by the river with layers of dense sand at much greater depth. As per the borehole logs, the top 2 m is fill material with a permeability (k) of 5×10^{-5} m/s. The ground water table is located at 1 m below the ground surface during rainy season. From 2 m to 15.4 m, the soil is silty clay or clayey silt (Unit 2). From 15.4 m to 19.5 m, the soil is medium silty clay (Unit 3a). Below 19.5 m, the soil is silty clay (Unit 3b) with some sand. The soil strength parameters for the soils are shown in Table 1. The value for Poisson's ratio of the soils (v_s) is assumed to be 0.3. The deformation modulus of a soil is determined from the well established correlation (USACE 1990) between the deformation modulus (Es), the undrained strength of the soil (C_u), overconsolidation ratio (OCR) and plasticity index (PI) of the soil, as shown below.

$$E_s = K_c * C_u \tag{1}$$

Table 1. Soil properties.

Soil type	Permeability k (m/s)	Unit Wt., γ (kN/m^3)	Undrained strength, Cu (kPa)	Cohesion, c′ (kPa)	Friction Angle, ϕ' (deg.)	Deformation modulus Es (kPa)
Fill	5×10^{-5}	–	–	–	–	–
Silty clay (Unit 2)	10^{-7}	18.5	31.0	1.0	25	18600
Medium silty clay (Unit 3a)	10^{-9}	18.5	70.0	0.0	31	31500
Silty clay (Unit 3b)	10^{-6}	18.5	55.0	0.0	29	27500

The values for the parameter K_c is obtained from Fig. 3 based on the PI and OCR for a soil. The soils are conservatively assumed to be normally consolidated (OCR = 1) in this study. The average plasticity index for the Units 2, 3b and 3b soils are 30, 45 and 35, respectively. Accordingly, the values of K_c for these soils are assumed to be 600, 450 and 500, respectively from the chart. The coefficient of earth pressure at rest is assumed to be 0.5 for all the soils. The average undrained strength of these soils are 31, 70 and 55 kPa, respectively based on the laboratory tests. The strengths (γ = unit weight of the soil, c′ = effective cohesion of the soil and ϕ' = effective friction angle of the soil) of the fill materials at the top is assumed to be same as that of the silty clay/clayey silt (Unit 2) layer except the value of the permeability.

3 Settlement Due to E-W Tunnels Construction

The alignment of the proposed twin tunnels with respect to the three identified heritage structures indicates that one of the heritage structures (Currency Building) is 33 m away from the rightmost tunnel while the other two (Bethel Synagogue and Maghen

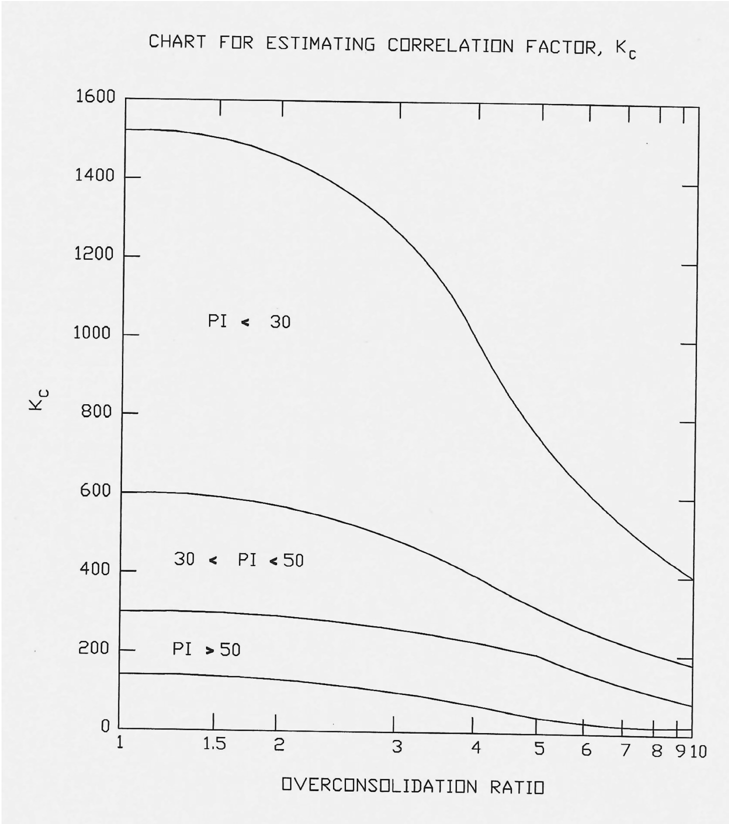


Fig. 3. Estimation of K_c based on OCR and PI of a soil (Ref. USACE 1990).

David Synagogue) are 22 m and 23 m away from the centerline of the second (rightmost) tunnel (KMRC 2015). The twin tunnels are 15 m apart (centerline to centerline). The tunnels are 6.1 m in diameter. The width of the concrete (M40) lining is about 350 mm. The deformation modulus and Poisson's ratio of the concrete are assumed to be E_{conc} = 3.162E + 07 kPa and υ_{conc} = 0.15, respectively. The proposed crown (top) depth and invert (bottom) depth of the tunnels at the study location are 17.6 m and 23.7 m, respectively. The twin tunnels shall be constructed by earth pressure balance shield (EPBS) method. Based on KMRC (2015), it is assumed that the tunnels are not constructed at the same time. The second tunnel shall be constructed when the first tunnel is constructed and advanced by at least 100 m.

The numerical analyses are performed using PLAXIS 2D software (Plaxis bv, 2012). PLAXIS2D is based on two dimentional plane strain finite element method and it is suitable for analyzing any coupled soil-structure interaction problem where large deformation is envisaged. The two extreme side boundaries are located 50 m from the edge of the tunnels and they are considered to be on roller, that is, only vertical movement is allowed at these boundaries. The bottom boundary is located 20 m below

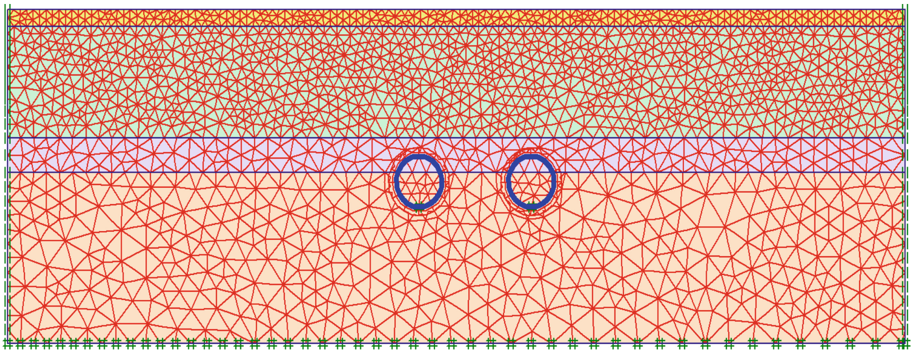
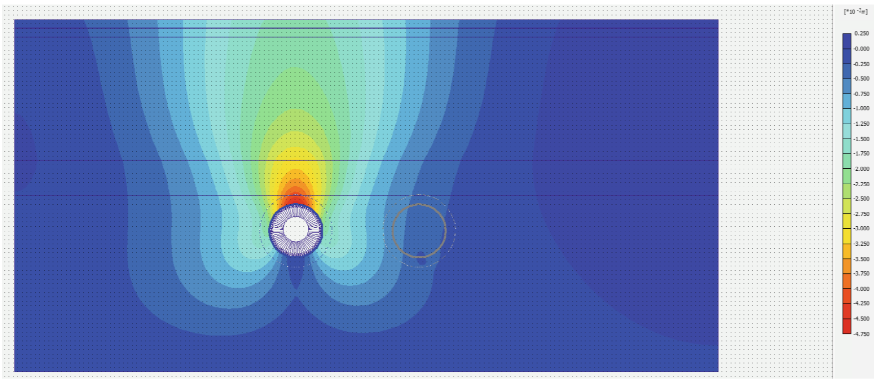
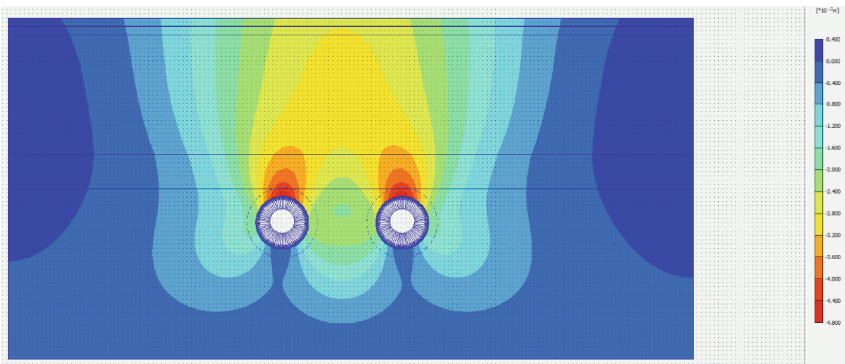


Fig. 4. Discretization of the ground and construction of the tunnels.



(a)



(b)

Fig. 5. Settlements due to the construction of (a) 1st tunnel and (b) 2nd Tunnel.

the bottom of the tunnels and only horizontal movement is allowed at this boundary. The soil is discretized by 6 noded triangular/8 noded rectangular isoparametric elements. The soils are assumed to be following the nonlinear elasto-plastic behavior satisfying the Mohr-Coulomb yield criteria. The tunnel linings are assumed to be elastic, impervious and seepage loss during tunnel construction is assumed to be minimal. The grouting and grout loss around the tunnels are not considered directly in the numerical analyses. Two cases with respect to the volume loss due to the construction of the tunnels (V_L) are considered. In the first case, V_L is assumed to be 2% as in FHWA (2009). This value corresponds to poor practice during tunnel boring with closed face machine in raveling ground. In the second case a V_L of 0.25% is assumed. This case represents good practice followed with tight control of face pressure within closed face machine in slowly raveling or squeezing ground. This value is found to yield observed settlements in tunnel construction with earth pressure balance (EPB) method in soft marine clay in Singapore (Gouw, 2005). In all the numerical analyses, the bottoms of the tunnels are assumed to be fixed, that is, they are not allowed to readjust their positions during construction. Note that the buildings or their foundations located in the area are not numerically modeled in these analyses. The effect of building weights and their stiffness are not considered in these analyses. However, the differential settlements at those places are obtained to perform more detail structural analyses of the buildings which are not part of this paper. At the initial stage of the numerical analyses, the subsurface without the tunnels is assumed to be in equilibrium condition. After this, the two tunnels are constructed one by one. After the construction of each tunnel, the stresses and deformations are obtained. Figure 4 shows the numerical discretization of the ground and the construction of the two tunnels. Figure 5 shows the settlements due to the construction of the first and second tunnels. Here the vibration due to the TBM operation is neglected. Figure 6 summarizes the ground surface settlement due to the construction of the first and second

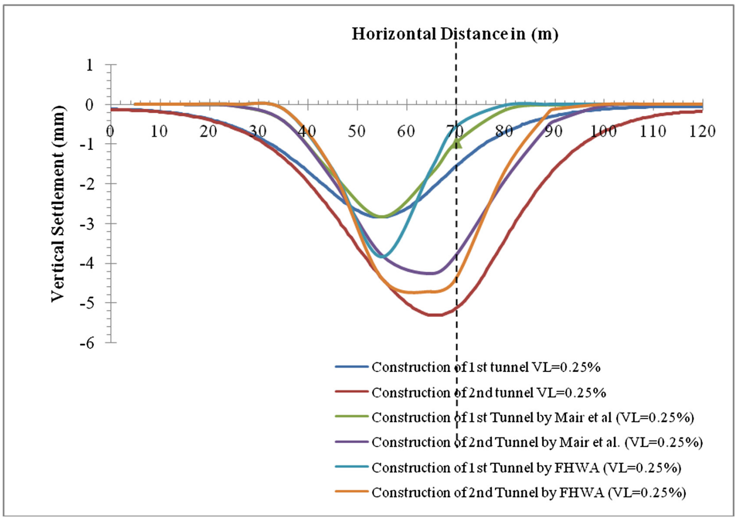


Fig. 6. Ground settlement after construction of both tunnels with $V_L = 0.25\%$.

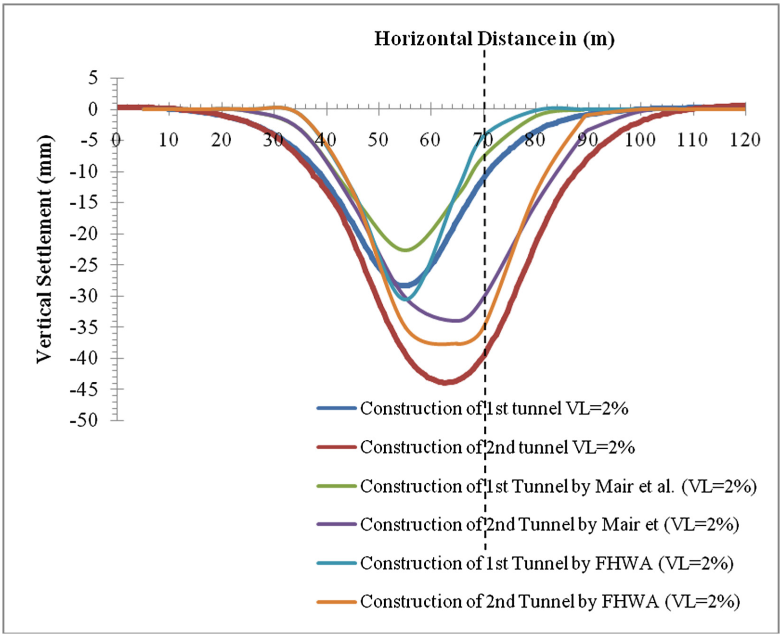


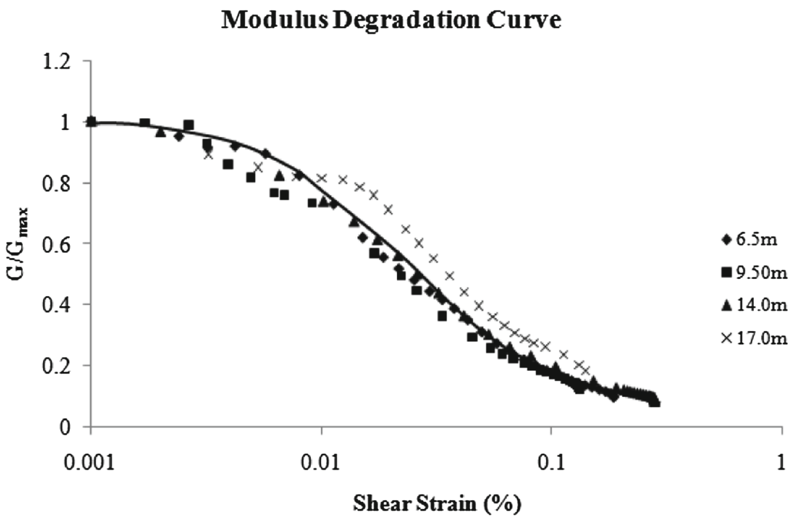
Fig. 7. Ground settlement after construction of both tunnels with V_L = 2%.

tunnel at the Brabourne Road area. In Fig. 6, the volume loss during tunnel construction is assumed to be 0.25% for all the cases. The finite element results are compared with the existing empirical methods given by Mair et al. (1996), and FHWA (2009). Figure 7 shows the estimated ground settlement due to the construction of first and second tunnels with the volume loss during tunnel construction is assumed to be 2% for all the cases. Here also, the finite element results are compared with the existing empirical methods given by Mair et al. (1996), and FHWA (2009). The estimated total settlement and angular distortion at the heritage structures at the end of the second tunnel construction are shown in Table 2. The estimated angular distortions of the buildings are well within

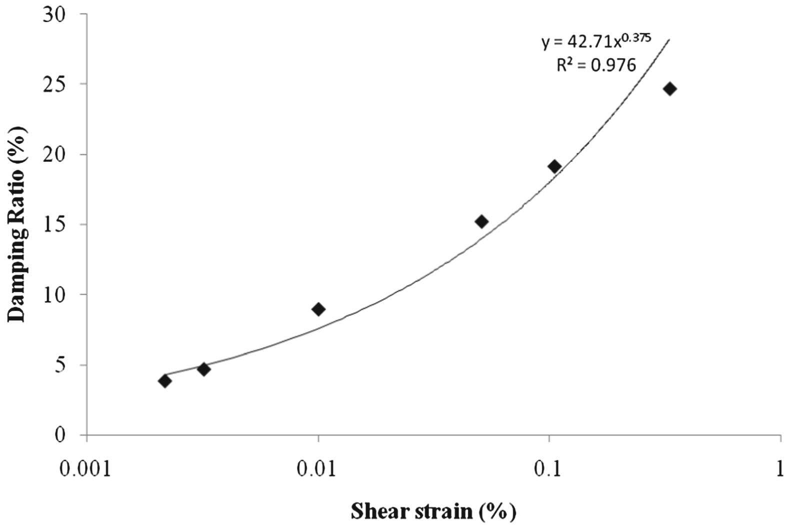
Table 2. Estimated total settlement and angular distortion due to tunnels construction.

Heritage structure	Distance from CL of 2nd tunnel (m)	Assumed volume loss (V_L)	Estimated maximum vertical settlement (mm)	Estimated maximum angular distortion
Currency Building	33 m	0.25	0.6	1/50000
		2.0	1.2	1/6087
Maghen David Synagogue	23 m	0.25	1.3	1/10000
		2.0	5.5	1/1650
Bethel Synagogue	22 m	0.25	1.4	1/10000
		2.0	6.0	1/1600

the permissible limit of 1/750. Note that it makes a significant difference in terms of settlement if you are on the left side of the 1st tunnel or if you are on the right side of the 2nd tunnel. Since it is not known at this time which tunnel will be first constructed, all the structures are conservatively assumed to be located on the right side of the 2nd tunnel.



(a)



(b)

Fig. 8. Dynamic properties of kolkata soils.

4 Effect of Vibrations Due to TBM Operation

A dynamic analysis is followed after the construction of each tunnel to estimate the effect of vibration on the ground and foundation of the nearby structures. Here again, the structures on the ground are not modeled. Absorbing boundary conditions are assumed at all the boundaries to prevent reflection/refraction of waves from the boundaries. The dynamic properties of the soils are obtained from the cyclic triaxial tests performed on the Kolkata subsoils. The dynamic properties of the soils (for all the units) are given by one single shear modulus degradation (G/G_{max}) curve and one single damping ratio (β/β_{crit}) curve as shown in Fig. 8. In absence of any data on the vibrations generated by TBM in Kolkata subsoil and with reference to the motions given by Mooney et al. (2014), a white Gaussian vibrational motion is generated using random number. Figure 9 shows the generated vibrational motion which is utilized to model the ground vibration to be generated by the TBM in the Kolkata subsoils. This noise is applied radially to the tunnel linings during their construction and their effects on the ground surface are obtained. Note that vibrations are not applied simultaneously to both the tunnels as they are constructed one after another with a gap in between. Figure 10 shows the responses (motions) coming to one of the heritage buildings called Currency Building (located 33 m from tunnel) due to the construction of the E-W tunnels. The maximum amplitude of vibration is 0.002 m/s^2 at the Currency Building and 0.003 m/s^2 at the other two heritage buildings, Bethel Synagogue and Meghen David Synagogue. Figure 11 shows the vertical settlement profiles due to the TBM vibrations during the construction of the first and the second tunnels at the Brabourne Road area. Figure 12 shows the settlement profiles after the tunnel constructions. The vertical settlement at the Currency Building due to the vibration caused by TBM during the construction of the twin tunnels is estimated to be 0.009 mm. The vertical settlement at the Bethel and Meghen David Synagogues due to the vibration caused by TBM during the construction of the twin tunnels is estimated to be 0.01 mm from the above analyses. Both of these numbers are very small for further consideration.

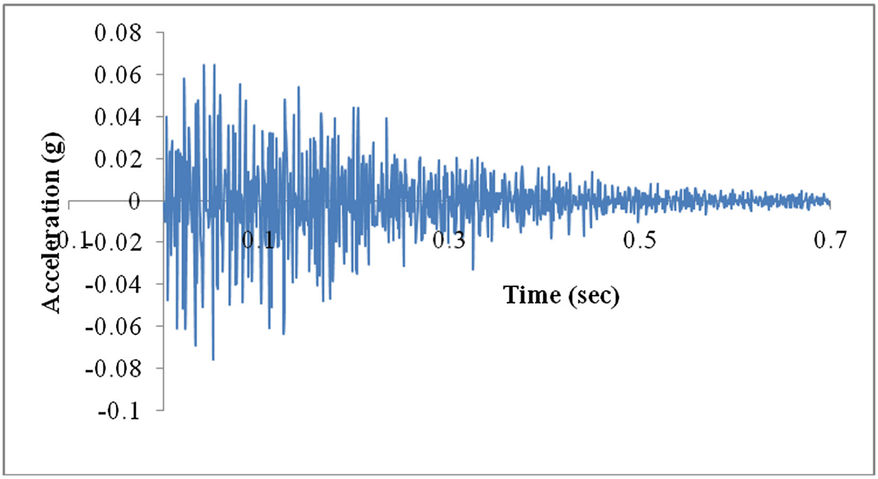


Fig. 9. Generated white gaussian noise to model vibration due to TBM operation in a tunnel.

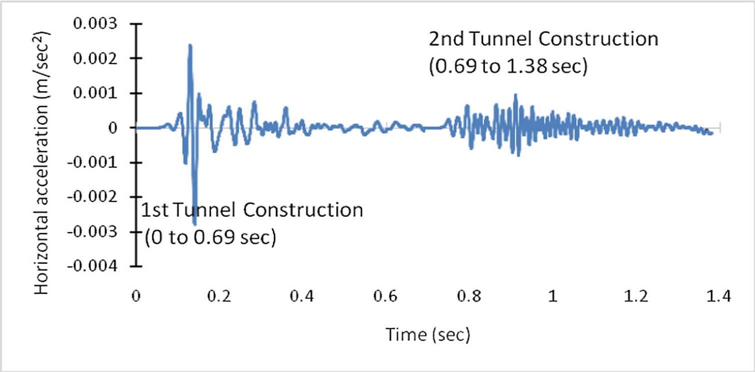
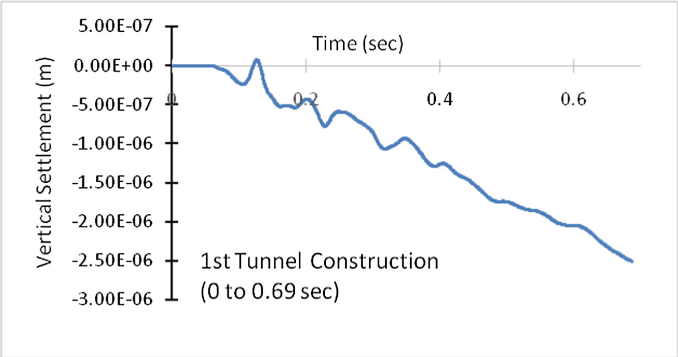
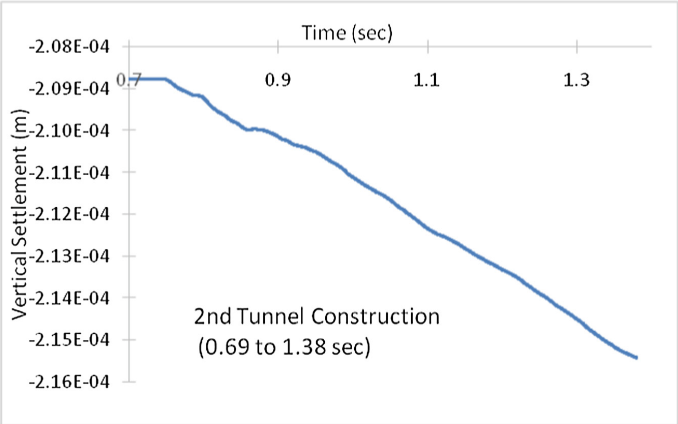


Fig. 10. Horizontal response at currency building due to tunnel construction.



(a)



(b)

Fig. 11. Vertical settlement at currency building due to TBM noise during construction of the tunnels.

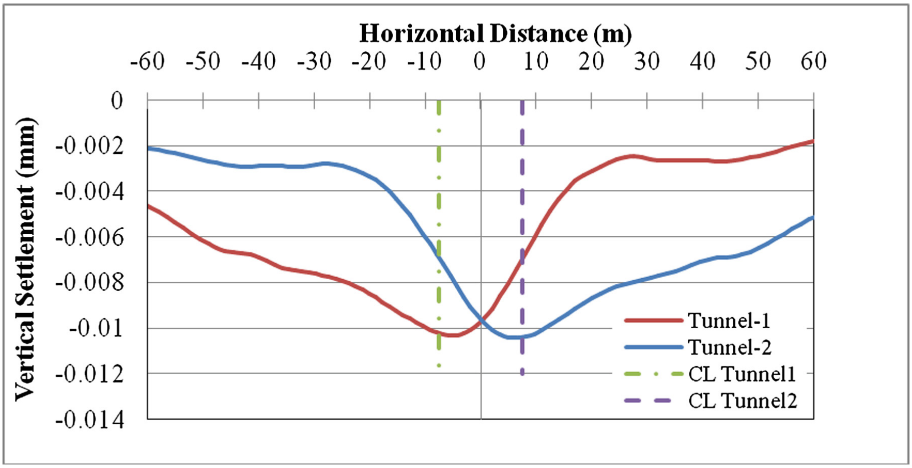


Fig. 12. Ground surface settlement profile after construction of tunnels

5 Conclusions

The vertical settlements due to the vibrations during the TBM operations are found to be negligible from the analyses. However, actual vibrations generated during TBM operation at the Kolkata subsoil needs to be determined to accurately determine their effect on the structures above the ground. The vertical settlement due to the tunnel constructions at the Currency building (located 33 m from the tunnel) is estimated to be between 0.6 to 1.2 mm. The angular distortions are found to be negligible. The vertical settlements at the Bethel and Meghen David Synagogues, located between 22 m and 23 m, are estimated to be between 1.3 and 6 mm. The maximum angular distortion is estimated to be 1 in 1600. This is well within the permissible limit of 1 in 750 for the residential buildings.

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