Ground Settlement Due to Tunneling in Cohesionless Soil

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Abstract: By the year 2035 it is estimated that Delhi and Mumbai will become two of the most populous cities around the globe. The massive population growth rate has led to the rise of land scarcity, urbanization, and industrialization and developments for rapid transit systems have made accordingly. Modern rapid transit systems comprise Metro rails and subways etc., and increase underground-construction activities. Nowadays, the tunnel-construction process heavily relies on massive machineries such as tunnelling-boring machines (TBM) and operations that produce great hindrance in the soil mass resulting in ground settlement at the surface. This study aimed to address these issues through small-scale laboratory experiments and further amplification to real-valued problems utilizing numerical methods. A cubic box of edge length 1 m made up of mild steel was generated to simulate a tunnelling operation and aluminum-made lining were used to simulate concrete tunnel linings. A finite element-based numerical investigation was done for a 2D elastoplastic numerical tunnel model with dimensions of 42 m \(\times\) 42 m. Analysis was carried out on Optum G2 software. The analyzed variations in lining shapes of lining included circular, horseshoe, arch, elliptical, and square. Results showed that elliptical-shaped linings experienced the least ground settlement and these are recommended for places where surface settlement may cause major damage. It is also recommended that square-shaped linings should not be used in such situations due their higher settlement values.

Keywords: tunnelling; finite-element method; settlement; physical modelling

1. Introduction

Urbanization is directly linked to large migrations of masses to metropolitans resulting in land scarcity issues. It affects the daily commute from one place to another in a highly stringent manner [1]. Thus, for rapid transit, and to ensure smooth traffic flow, utilization of urban ground space became inevitable. This leads to the initiation of underground transportation in cities such as Hong Kong and Delhi [2,3]. In Delhi itself, the metro rail network is increasing exponentially, and underground tunnels are constructed daily [4]. Tunnel construction is a challenging task due to variations in geology and topography. Modern tunnelling methods, such as the new Austrian tunnelling method (NATM) or the Norwegian tunnelling method (NTM), use sophisticated machinery such as the tunnel-boring machine (TBM). The vibrations produced during the process develop stresses in the soil mass which are ultimately released on reaching the ground surface, causing settlement [5]. Ground settlement due to tunnelling, if not appropriately analyzed, could damage the overlying structures, and may develop tensile strains as well [6]. The risk is elevated in cities such as Delhi and Hong Kong, where population density is extremely high. Ground settlement due to tunnelling has been analyzed in the past by various researchers using stochastic cum probabilistic methods [7–10], analytical methods [11–17], and empirical methods [18,19]. Tunnelling is a 3D operation that causes settlement about...
both the axes (i.e., transverse and longitudinal). For simplification, researchers usually focus on steady-state ground settlements occurring transverse to the tunnel axis, though the longitudinal payment has also been analyzed by a few researchers [20,21]. Peck [18], in 1969, through various field observations, suggested that the shape of the ground settlement profile for the transverse direction can be approximated through a standard Gaussian curve, a normal-distribution curve, or a bell curve, as shown in Figure 1. The settlement at any point can be expressed using Equations (1) and (2) providing a relationship between settlement through width and tunnel radius [18].

\[
S(x) = S_{\text{max}} e^{-\frac{x^2}{2\sigma^2}}
\]  

(1)

where, \( S(x) \) is the settlement at distance \( x \) from the tunnel centre line, \( S_{\text{max}} \) is the maximum settlement, \( x \) is the transverse distance from tunnel centreline, and \( i \) is the settlement trough width from centreline up to point of inflection.

\[
\frac{i}{R} = \left( \frac{z_0}{2R} \right)^n
\]  

(2)

where, \( n = 0.1 \) to 0.8, \( R \) is the radius of tunnel, \( z_0 \) is the depth of the tunnel below ground surface, and \( i \) is the settlement trough width from centreline up to point of inflection.

![Figure 1. Ground-settlement trough.](image)

The normal distribution proposed by Peck was simpler than the previously developed stochastic approaches and has been widely used by researchers since then, confirming its validity to most scenarios [6,22–30]. Modifications to the Gaussian curve for conditions where Peck’s relations do not provide possible results were proposed [31–34]. However, Peck’s hypothesis is still used widely in practical applications due to its simplicity and versatility. Equation (3) was proposed by Attewell and Farmer [26] for UK tunnels based on field observations, Equations (4) and (5) were proposed for loose and dense sands, respectively, based on field observations and laboratory experiments, and Equations (6) and (7) were proposed for cohesive mediums through linear regression technique. Rankine [35] suggested Equation (8) based on case-history data and field observations which can apply to all types of soil media.

\[
\frac{i}{R} = \left( \frac{z_0}{2R} \right)
\]  

(3)

\[
i = 0.25(1.5z_0 + 0.5R)
\]  

(4)

\[
i = 0.25(z_0 + 0.5R)
\]  

(5)

\[
i = 0.43z_0 + 1.1 \text{ for } 3 \leq z_0 \leq 34
\]  

(6)

\[
i = 0.28z_0 - 0.1 \text{ for } 6 \leq z_0 \leq 10
\]  

(7)
\[ i = 0.5z_0 \]  

(8)

All these relationships were derived from Peck’s hypothesis, which does not provide any theoretical background. To find an explanation to this problem, many experimental programs have been carried out involving centrifuge modelling and physical modelling tests [29,32,36–39]. Marshal et al. [38] studied various parameters affecting the ground settlement, such as tunnel depth, tunnel size, and volume loss on greenfield settlement. The authors concluded that Gaussian curves do not provide satisfactory results for sands with higher volume loss. They suggested the use of a modified Gaussian curve for such problems. Mathematical equations were proposed to incorporate the effect of the three parameters (tunnel depth, tunnel size, and volume loss on greenfield settlement). Various researchers carried out further analysis [23,40]. Chakeri [41] analyzed parameters such as tunnel depth and dimensions, overburden pressure, and face pressure [41]. Chapman et al. [37] used small-scale physical modelling to study the effect on ground settlement, of tunnels constructed side-by-side; the authors concluded that the existing Gaussian curve does not provide accurate measurement of ground settlement for tunnels constructed in close proximity. A modified Gaussian curve suggested by Hunt [32] provides better results for the same case as reported by the authors. Adachi et al. [36] performed an axisymmetric trap-door test using centrifuge modelling facilities under 1g speed. Analysis was done with variable depth to diameter ratios. Moreover, displacements and earth pressure around the trap door were recorded. Charles [42] performs centrifuge tests for different geotechnical problems. Numerical analysis was also performed after validation using test results. Marto et al. [39] studied greenfield ground settlement through a physical modelling technique. The analysis was done for variation in cover depth to tunnel diameter ratios and at different relative densities for sand. Results showed a decrease in the surface settlement as the relative density increased and vice-versa. Apart from above-mentioned experimental approaches, numerical techniques were also adopted in some research. Shiau et al. [43] studied a finite-difference model generated through the FLAC package. They compared the results with previously available empirical and analytical methods to propose a set of K values for practical utilization based on tunnel dimensions and soil strengths. Kolivand et al. [44] analyzed a three-dimensional numerical model generated through Plaxis-3D; results obtained showed the slightest deviation from observed field data. Still, there is a lack of experimental and numerical approaches for cohesionless soils. Our study explores the effects of ground settlement due to tunnelling in cohesionless soils.

In our study experimental and numerical approaches were carried out. A physical model of a cubic tunnel box made up of mild steel plates and having dimensions of 1 m was developed. Tunnels with 0.30 m in the centre of the cubic box were used in the experiment. The results of the experimental analysis were back-analyzed to validate our numerical model. We developed a 2D finite-element model to expand the research on a real-life scale. We performed a full-scale 2D elastoplastic study for five tunnel openings: circular, horseshoe, arch, square, and elliptical. The study was aimed at finding proper design recommendations for all these shapes by accurate prediction of ground settlement.

2. Physical Modeling of the Tunnel

Laboratory experiments such as sieve analysis, field density using the core cutter, and pycnometer are carried out to find the physical characteristics of the soil used in our experiments. Figure 2 shows the particle-size distribution curve through which the soil could be classified as sand. The average field density determined using the core cutter was found to be 1.6043 gm/cm^3 and the dry density was 1.53 gm/cm^3. The pycnometer test was performed to find the average specific gravity of sand which turns out to be 2.65. The lab-test data were later used to find an appropriate soil model for numerical analysis.
In the present study, a cubical tunnel box of 1 m dimensions was manufactured with a galvanized iron sheet of 14-gauge thickness. An opening of 30 cm diameter on the vertical walls was provided to insert the lining. A steel frame was attached to the tunnel box for levelling and measurement purposes. The schematic diagram of the laboratory model is shown in Figure 3. Tunnel lining was simulated through an aluminium sheet with 26-gauge thickness.

Figure 2. Particle-size distribution curve.

The experimental study involved various phases as described below.

2.1. Establishment of the Physical Model in the Lab

Loose fine-grained soil was procured from the local site. Various lab tests were performed to test the geotechnical parameters of soil as described above. The physical lab model, of dimensions described above, was manufactured by a skilled ironsmith and installed in the lab as shown in Figure 4. The steel frame was attached after proper levelling of the whole model.

Figure 3. Schematic diagram of lab model.
were lubricated using oil to simulate the frictionless boundary. The sand was then filled in the tunnel box with normal compaction using a standard rammer. Sand was filled in three layers of equal compaction and width. Figure 5 shows the placement of lining inside the tunnel box.

2.2. Placement of Lining and Soil Filling

The lining was filled with soil before the final installation to avoid initial deformation and to simulate the actual ground conditions before excavation. Boundaries of tunnel boxes were lubricated using oil to simulate the frictionless boundary. The sand was then filled in the tunnel box with normal compaction using a standard rammer. Sand was filled in three layers of equal compaction and width. Figure 5 shows the placement of lining inside the tunnel box.

2.3. Levelling of the Top Surface

Initial levelling was done by using the spirit level and the surface was levelled. On this levelled surface, digital Vernier callipers were then used to level the top surface to fix the datum. A steel frame was attached to the tunnel box as shown in Figures 3 and 4, this served as a datum for Vernier calliper measurements for settlement afterward.

2.4. Excavation and Measurement of Ground Settlement

The steel frame had three transverse plates at 25 cm, 50 cm, and 75 cm from advancing faces (Figure 3). The ground settlement was measured along with these three locations. Readings were taken at every 2 cm from the centreline of the tunnel to either side using Vernier callipers. Figure 6 shows the ground-settlement trough/profile for three different locations: (i) at a distance of 25 cm from advancing face, (ii) at a distance of 50 cm from advancing face, and (iii) at a distance of 75 cm from advancing face. The settlement trough was up to 30 cm from either side of the centreline to accommodate the shape of the trough completely. In addition, beyond 30 cm from either side of the centreline, there was little to negligible settlement recorded. A maximum settlement of 1.84 mm was recorded at the second position located at the center of the tunnel. The first and third positions had approximately the same settlement trough with a higher value for the third position.
Figure 6 shows the results of numerical modelling on Optum G2 software (see Section 3). The shape of the settlement trough was in good coherence with the existing trough shape, as discussed in Figure 1.

![Figure 6. Settlement trough generated through lab experiment and numerical modelling on Optum G2.](image)

**2.5. Validation**

Peck [18] proposed Equation (2) for estimating the inflection point or settlement trough width parameter \( i \) through various empirical field observations, Atkinson and Potts [22] then modified this empirical relation for loose sand and proposed Equation (4). The values of \( i \) calculated from these two relations for our problem were found to be 24 cm and 22 cm, respectively. In the present experimental study, the value of \( i \) parameter was approximately 18 cm for all three (i.e., first, second, and third) positions, as described in Section 2.4. The measurement of the experimental study showed 24% and 18% deviations from Peck’s and Atkinson’s relations, respectively. Since both the relations were derived empirically, this much error would be considered. Moreover, maximum ground settlement values obtained by Marto et al. [39] through a physical-modelling test were found to be 1.5 mm for loose sand, which is very near to our maximum settlement value of 1.84 mm. A slightly lower settlement value in the case of Marto et al. [39] can be explained due to the smaller size of their physical model. The above discussion suggests that the results of our analysis are in good coherence with the existing empirical and experimental methods.

**3. Finite-Element Modelling**

FE modeling was done through geotechnical software Optum G2 available for academic purposes (https://Optumce.Com/Academic/Academic-License-Information/, accessed on 2 January 2022). Finite element analysis was performed in two parts: (1) modelling of the lab model and (2) full-scale models with different shapes of linings. For both the parts, the same software package (Optum G2) was used [45].

**3.1. Lab Model**

A 2D finite-element model was generated on Optum G2 having plan dimensions 1 m \( \times \) 1 m with a circular opening of 30 cm diameter at the center of tunnel box. The main objective of this analysis was to validate our numerical modeling using experimental observations. Tunneling simulation was done through the convergence-confinement method (CCM). CCM has been widely used by various researchers for simulating tunneling problems [46–49]. CCM is based on the concept of the convergence of the ground, the
support pressures, and the resulting confinement losses [46]. The generated numerical model is shown in Figure 7a and the discretized mesh model shown in Figure 7b.

![Figure 7. (a) Numerical model and (b) meshed numerical model.](image)

In the present study, the elastoplastic soil constitutive Mohr–Coulomb model, were used for modelling of soil. Since the laboratory test was simulated through numerical simulation geotechnical parameters for the soil model used needed to be of the same soil used in the laboratory. Table 1 shows soil properties used for numerical analysis [28,50]. The aluminium lining was simulated using a plate feature in Optum G2. The properties of aluminium (Al 6061-T6) are shown in Table 1 [51,52].

| Soil Properties          | Value                  |
|-------------------------|------------------------|
| Unsaturated unit weight | 16 kN/m³               |
| Saturated unit weight   | 19 kN/m³               |
| Young’s modulus         | 1.5 × 10⁴ kPa          |
| Poisson’s ratio         | 0.3                    |
| Cohesion                | 0 kPa                  |
| Friction angle          | 30º                    |
| Dilation angle          | 0º                     |

| Aluminum Lining (Al 6061-T6) | Value                  |
|------------------------------|------------------------|
| Density                      | 26.47                  |
| Young’s modulus              | 6.9 × 10⁴ kPa          |
| Sectional area               | 40 cm²/m               |
| Plastic-section modulus      | 40 cm³/m               |
| Moment of inertia            | 0.534 cm⁴/m            |
| Yield strength               | 275 MPa                |
| Weight                       | 10.6 kg/m/m            |

The model was provided with a fine mesh having many elements in the range of 1000, and six-node Gaussian type elements were used for both soil and aluminium lining. The mesh adaptivity was adopted to provide recommended fines of mesh in the region. The model was provided with roller support at the two vertical edges having displacement possible in vertical directions and fixed in the horizontal direction. The base of the model was provided with fixed support. The above boundary condition is termed standard fixities
in OptumG2. Analysis was done manually using the convergence–confinement method provided in the Optum G2 examples. Analysis was carried out in three stages for better simulation of the tunnelling problem.

Stage 1: This was a greenfield condition having soil modelled on ground conditions. The analysis performed in this stage is known as initial stress analysis. The initial stress state is usually characterized by the earth pressure coefficient Equation (9):

$$K_0 = \frac{\sigma_h}{\sigma_v}$$

where $\sigma_h$ and $\sigma_v$ are the effective horizontal and vertical stresses, respectively. For a purely frictional Mohr–Coulomb material, the bounds on $K_0$ are the well-known active and passive earth pressure coefficients (Equation (10)):

$$\frac{(1 - \sin \varphi)}{(1 + \sin \varphi)} \leq K_0 = \frac{\sigma_h}{\sigma_v} \leq \frac{(1 + \sin \varphi)}{(1 - \sin \varphi)}$$

where, $\varphi$ is the internal friction angle.

Stage 2: An excavation for the tunnel was provided in this stage and an elastoplastic analysis was performed. The tunnel perimeter here was fully supported and a relaxation factor, $\lambda$, was specified and excavation was provided with full support according to the manual of Optum G2. From an initial state, the tunnel was excavated while keeping the perimeter of the tunnel fully supported. This induced no changes to the stress state and hence no deformations. Next, the reactions on the tunnel perimeter were relaxed, i.e., reduced by a factor $\lambda$ where $0 \leq \lambda \leq 1$.

Stage 3: In the third stage the lining was inserted, and all supports around the tunnel perimeter were removed and were replaced by a plate to model the lining. The elastoplastic analysis was then carried out.

4. Full-Scale Model with Different Tunnel Linings

A two-dimensional full-scale numerical model was generated for the actual tunnelling problem. A tunnel box of $42 \text{ m} \times 42 \text{ m}$ (Figure 8) was adopted in our study. Tunnels constructed during the Delhi Metro Phase 3 project are situated at 12–30 m depth from the ground surface. In the present study, depth of the tunnel centreline was kept at 21 m (Figure 8), so that average depth from the Delhi Metro Phase 3 project was achieved. In addition, having the tunnel at the centre of tunnel box enhanced the symmetry of the model. Analysis was carried out for five different shapes of tunnel linings namely (a) circular, (b) horseshoe, (c) square, (d) arch, and (e) elliptical. A schematic diagram of the tunnel box having a circular opening is shown in the Figure 8. Geometric dimensions of different shapes of linings are shown in Figure 9. It can be seen that the diameter or side length of lining in all cases was 5.65 m. This was done to ensure that all edges of tunnel boxes were situated at more than 3d length from lining periphery, where d is the diameter of the tunnel. This approach was incorporated for complete dying out of stress contours, since beyond this length (i.e., 3d) stress becomes trivial [53]. The thickness of all lining shapes was kept at 25 cm. Six-node gaussian type elements were used for meshing of both the lining and soil mass. The mesh adaptivity feature in Optum G2 was also used for the recommended finer mesh in the region of high intensity of displacement.

For the lab model, the soil was modelled using properties given in Table 1. The lining was modelled with concrete properties [54] given in Table 2 using the plate element. Square yield criteria were used for the plate element and the material was considered impermeable. Meshing and boundary conditions were applied as discussed in Section 3.1, and Figure 8 shows a meshed model for tunnel model with a circular opening.
Figure 9. Shapes of lining considered for numerical study (all dimensions are in meters).

Table 2. Properties of concrete lining.

| Property                        | Value                  |
|---------------------------------|------------------------|
| Density                         | $25 \text{ kN/m}^3$    |
| Young’s modulus                 | $3.16 \times 10^7 \text{ kPa}$ |
| Poisson ratio                   | 0.15                   |
| Sectional area                  | $2500 \text{ cm}^2/\text{m}$ |
| Plastic-section modulus         | $15,625 \text{ cm}^3/\text{m}$ |
| Moment of Inertia               | $130,208.33 \text{ cm}^4/\text{m}$ |
| Yield strength                  | $30 \text{ MPa}$       |

5. Results and Discussion

The present study was carried out in two parts—(a) laboratory simulation and its validation and (b) full-scale numerical study.

5.1. Numerical Simulation of Lab Problem and Its Validation

Simulation of the tunnelling procedure was carried out using the convergence–confinement method, CCM, and results from experimental analysis were used to validate the numerical model. Results obtained from experimental and numerical analyses were approximately similar for maximum settlement with an error of 2.7%, hence validating our numerical model. Figure 10 shows vertical displacement in (a) the soil and (b) the lining. In the soil, the maximum vertical displacement of 1.76 mm was recorded at the ground surface, for
lining, the maximum vertical displacement of 1.15 mm was at the crown position. Figure 11 shows the principal stresses in soil along with the horizontal plane and vertical plane, to represent the tensile stresses generated in both cases. Stresses in the horizontal plane were in the range of 12–47 kN/m² with the maximum at the springer and the minimum at the crown. In the vertical plane, stresses were in the range of 28–55 kN/m² showing larger variations than the horizontal plane, having minimum values at the crown and invert, and higher concentrations about the springer position.

![Figure 10. Vertical displacement contours for (a) soil mass and (b) aluminium lining.](image)

![Figure 11. Principal stresses in soil. (a) horizontal plane and (b) vertical plane.](image)

5.2. Full-Scale Numerical Study of Tunnels

A Two-dimensional elastoplastic study for full-scale field simulation of the tunnelling problem was carried out using finite element-based software Optum G2. Analysis was performed for five different lining shapes used in common tunnelling constructions, namely (i) circular, (ii) horseshoe, (iii) arch, (iv) elliptical, and (v) square shapes. The CCM method was adopted for numerical simulation of the problem. The schematic diagram of the model with a circular tunnel is shown in Figure 8. The geometric dimensions for tunnel linings are shown in Figure 9.

The study was to analyze ground settlement and plot the ground-settlement trough for different shapes of the lining. The tunnel constructed with elliptical lining had a minimum value of maximum settlement followed by horseshoe, circular, arch, and square-shaped linings. Figure 12 shows the plotted ground settlement for various shapes of tunnel linings. The square-shaped tunnel had the highest value for maximum settlement with a magnitude of 4 cm and the elliptical-shaped tunnel had the least maximum settlement value among all shapes with magnitude of 1.4 cm. Displacement in the soil can be characterized as horizontal displacement or vertical displacement. Horizontal displacement does not pose any vulnerability to tunnel stability and was therefore not computed. Vertical displacements,
on the contrary, can cause significant damage to the tunnel, so the determination of vertical displacement is thought important and necessary. Vertical displacement contours were plotted for all cases in Figure 13. The maximum settlement was recorded for the square lining and least for the horseshoe lining.

![Figure 12](image-url)  
**Figure 12.** Ground-settlement trough for different shapes of tunnel linings.

![Figure 13](image-url)  
**Figure 13.** Vertical displacement in soil having (a) circular-, (b) arch-, (c) horseshoe-, (d) elliptical-, and (e) square-shaped openings.

The stresses developed in soil are an important parameter to analyze. The model was analyzed to determine the principal stresses in the horizontal plane, \( \sigma_{xx} \) and the
principal stress in the vertical plane, $\sigma_{yy}$, developed in soil media. Tensile stresses are developed throughout the soil mass with the only exception being the area in close vicinity to the ground surface for both the principal plane directions. Results showed that for the horizontal plane, maximum stresses were at invert locations for all shapes within the range 209–252 kN/m$^2$ with a maximum for the elliptical-shaped lining. Vertical-plane stresses were maximum around the springer location within the range 336–422 kN/m$^2$ with a maximum for the circular-shaped lining and stresses around the crown and invert showing large variations. Figure 14 shows a contour plot for horizontal-plane stresses and Figure 15 shows vertical-plane stresses for all shapes of the lining. Stresses at the springer location for the horizontal plane were localized for all shapes except the square opening due to the absence of arch action. There was no effect of the shape of lining at the bottom of soil as the value was approximately the same for all cases in both the principal planes. The results showed that lining shapes such as horseshoe and square experience lower stresses at the springer location than other shapes due to the vertical plate boundaries.

Figure 14. Principal stresses ($\sigma_{xx}$) in soil having (a) circular-, (b) arch-, (c) horseshoe-, (d) elliptical-, and (e) square-shaped openings.
Displacement of tunnel concrete linings can significantly affect the stability of the tunnel. Displacement of even a small magnitude can make the structure unfit for operation. In the same way as for soil movement, the tunnel also experiences horizontal and vertical displacements. However, horizontal displacement can be countered by an intact soil mass and has a negligible effect on the structural integrity of the tunnel lining. Vertical displacement, on the other hand, causes significant damage to the lining in terms of visible cracks, spalling, and, in severe cases, collapse of the lining due to an increase in stresses and strains beyond permissible limits. Results suggested that elliptical-shaped lining are most stable with a maximum displacement of 5 mm. Maximum displacement occurred at an invert location for all shapes. Figure 16 shows the plotted vertical displacement contours for different geometric shapes. The least stable was the square-shaped lining with a maximum displacement of 17 mm. There was a constant displacement along with the springer locations in the horseshoe- and square-shaped linings due to vertical plate boundaries whereas for other shapes there was a variation along the vertical lining due to arching effect. The stability of an elliptical shape can be understood through the concept of arches and the distribution of forces in them. Maximum displacements were observed at three crucial points on tunnel lining—crown, springer, and invert. On the horizontal axis, different shapes of lining are presented. The effect of different shapes of tunnel linings has been analyzed and the normal force, shear force, and bending moment has been reported. Figure 17 shows variations in (a) normal force, (b) shear force, and (c) bending moment for crucial tunnel points i.e., crown, springer, and invert. The horizontal axis represented the lining shape.
the springer locations in the horseshoe- and square-shaped linings due to vertical plate boundaries whereas for other shapes there was a variation along the vertical lining due to arching effect. The stability of an elliptical shape can be understood through the concept of arches and the distribution of forces in them. Maximum displacements were observed at three crucial points on tunnel lining—crown, springer, and invert. On the horizontal axis, different shapes of lining are presented. The effect of different shapes of tunnel linings has been analyzed and the normal force, shear force, and bending moment has been reported. Figure 17 shows variations in (a) normal force, (b) shear force, and (c) bending moment for crucial tunnel points i.e., crown, springer, and invert. The horizontal axis represented the lining shape.

![Figure 16. Vertical displacement at crucial tunnel locations.](image1)

![Figure 17. Variation of (a) normal force, (b) shear force, and (c) bending moment for the crown, springer, and invert locations for different tunnel openings represented on the horizontal axis.](image2)
6. Conclusions

In the present study, ground settlement due to tunneling was analyzed. Experimental and numerical investigations were done to analyze the effects of different shapes of tunnel linings on ground settlement. The main conclusions of the study are summarized as:

- For laboratory simulation, a maximum ground settlement of 1.85 mm was recorded. This was 1.80 mm for the numerical analysis of laboratory simulations, so the error was only 2.7 percent.
- The vertical displacement in the linings was maximum at the crown position with a magnitude of 1.15 mm.
- Numerical analysis showed the least maximum settlement for an elliptical-shaped liner with a magnitude of 1.4 cm. However, the highest maximum settlement of 4.2 cm was recorded for square-shaped linings. The descending order of maximum settlement was elliptical–horseshoe–circular–arch–square.
- The principal stresses in the soil mass were greatest at springer locations and lowest at crown and invert locations along horizontal and vertical planes. The minimum and maximum stresses in the vertical plane were higher than those in horizontal plane.
- The horizontal-plane stresses were greatest around the invert, and the vertical-plane stresses were greatest around the springer. Horizontal-plane stresses were maximum for elliptical-shaped linings and vertical-plane stresses for circular-shaped linings. Because horseshoe and square-shaped linings have vertical boundaries, they experience fewer stresses at springer locations.
- Normal forces in the lining were found to be highest at springer locations for all shapes, and shear forces in the lining were found to be highest at the crown and lowest at springer locations. Circular- and elliptical-shaped linings have a negative bending moment around the springer and a positive bending moment around the crown and invert. Horseshoe- and arch-shaped linings have negative bending moments around the invert.

The results suggest that elliptical-shaped linings are the most suitable for minimizing ground settlement. Square-shaped linings are the least stable and hence it is recommended that they not used where ground settlement might cause severe damage.

Author Contributions: Conceptualization, M.R.S. and M.F.A.; methodology, M.R.S. and M.F.A.; validation, M.R.S. and A.H.A.; formal analysis, M.F.A. and S.A.; investigation, M.R.S.; resources, M.R.S. and A.H.A.; data curation, M.F.A.; writing—original draft preparation, M.F.A.; writing—review and editing, M.R.S., A.H.A., and S.A.; funding acquisition, A.H.A. All authors have read and agreed to the published version of the manuscript.

Funding: The authors would like to acknowledge the support provided by Researchers Supporting Project Number (RSP2022R473), King Saud University, Riyadh, Saudi Arabia.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: Not applicable.

Acknowledgments: The authors are thankful to University Grant Commission, India for providing UGC BSR Startup to Md Rehan Sadique for carrying out this research.

Conflicts of Interest: The authors declare no conflict of interest.

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