

# **EFFECTS OF TUNNELING ON SURROUNDING STRUCTURES**

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in Partial Fulfillment of the Requirement  
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Geotechnical Engineering**

**by**

**Jyoti Agarwal**  
(Roll No. 2K20/PHDCE/04)

**Under the Supervision of**

**Prof. Raju Sarkar**  
(Professor, DTU, Delhi)



**To the  
Department of Civil Engineering**

**DELHI TECHNOLOGICAL UNIVERSITY**  
(Formerly Delhi College of Engineering)  
Shahbad Daultapur, Main Bawana Road, Delhi-110042, India

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**Jyoti Agarwal**

**(Roll No. 2K20/PHDCE/04)**



## **DELHI TECHNOLOGICAL UNIVERSITY**

(Formerly Delhi College of Engineering)  
Shahbad Daulatpur, Main Bawana Road, Delhi-42

### **CANDIDATE'S DECLARATION**

I **Jyoti Agarwal** hereby certify that the work which is being presented in the thesis entitled "**Effects of Tunneling on Surrounding Structures**" in partial fulfilment of the requirements for the award of the Degree of Doctor of Philosophy, submitted in the Department of **Civil Engineering**, Delhi Technological University is an authentic record of my own work carried out during the period from **August, 2020** to **October, 2025** under the supervision of **Dr. Raju Sarkar**.

The matter presented in the thesis has not been submitted by me for the award of any other degree of this or any other Institute.

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This is to certify that the student has incorporated all the corrections suggested by the examiners in the thesis and the statement made by the candidate is correct to the best of our knowledge.

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### **CERTIFICATE BY THE SUPERVISOR(s)**

Certified that **Jyoti Agarwal** (2k20/PHDCE/04) has carried out the research work presented in this thesis entitled “**Effects of Tunneling on Surrounding Structures**”, for the award of **Doctor of Philosophy** degree, from the Department of Civil Engineering, Delhi Technological University, Delhi, under my supervision. The thesis embodies results of original work, and studies are carried out by the student herself and the contents of the thesis do not form the basis for the award of any other degree to the candidate or to anybody else from this or any other University/Institution.

#### **Signature**

Dr. Raju Sarkar

Professor, Department of Civil Engineering

Delhi Technological University

Date:

# **Effects of Tunneling on Surrounding Structures**

Jyoti Agarwal

## **ABSTRACT**

Urban tunneling projects in India have expanded rapidly to support metro and regional transit systems; however, tunneling-induced ground deformations continue to pose critical risks to adjacent structures. This thesis develops and applies an integrated analytical–numerical framework for building risk assessment, focusing on the Delhi–Meerut Regional Rapid Transit System (RRTS). Ground deformations were obtained by numerical simulation using PLAXIS 2D under greenfield conditions and validated against established empirical–analytical methods, before being transferred to building’s foundation through the Limiting Tensile Strain Method (LTSM). The research combined systematic numerical modelling, strain-based structural evaluation, and parametric investigations to quantify the effects of tunnel, soil, and structure parameters, with additional emphasis on seismic loading. The baseline numerical model reproduced typical settlement troughs consistent with field behavior, confirming its reliability for Indian soil conditions. Parametric analyses revealed that settlement magnitude and structure response are strongly influenced by tunnel geometry, cover depth, and soil stiffness. Softer ground conditions and larger tunnel dimensions generally resulted in higher ground deformations, while stiffer soils and deeper covers reduced the magnitude of ground movements. Structural characteristics also played a key role in the observed damage outcomes: masonry buildings are more vulnerable to strain concentrations, as compared to framed one. Building damage classification based on LTSM indicated that most structures under baseline conditions fell within slight to moderate categories. However, when seismic loading was introduced, risks increased considerably. Both masonry and framed buildings experienced amplified strains and angular distortions under earthquake conditions, often leading to a shift into higher damage categories. The analysis confirmed that seismic effects represent a critical dimension that is often overlooked in tunneling risk assessments in India.

The integrated methodology demonstrated in this research advances beyond traditional greenfield-based assessments by explicitly linking tunneling-induced ground deformations with structural response measures. Key contributions include: (i) development of validated PLAXIS 2D models for Delhi soil conditions; (ii) systematic understanding of the role of tunnel, soil, and structure parameters in influencing building response; (iii) incorporation of seismic scenarios into tunneling-related building risk assessments; and (iv) practical insights for improving safety margins in urban tunneling projects. The results emphasize that damage risk can be effectively reduced through careful design choices such as optimizing tunnel depth, maintaining adequate construction control (i.e. Volume loss), and considering both geotechnical and structural vulnerability during the planning stage.

Overall, this thesis establishes a qualitative, transferable framework for Building Risk Assessment (BRA) in Indian tunneling projects. It provides engineers and policymakers with a reliable decision-support tool to ensure safer underground construction in densely populated environments. The findings hold direct applicability for metro, RRTS, and other large-scale tunneling projects in Indian urban regions, underscoring the importance of integrated geotechnical–structural evaluation for sustainable infrastructure development.

## TABLE OF CONTENTS

|                                                                        |       |
|------------------------------------------------------------------------|-------|
| ACKNOWLEDGEMENTS .....                                                 | ii    |
| CANDIDATE’S DECLARATION .....                                          | iv    |
| CERTIFICATE BY THE SUPERVISOR(s) .....                                 | v     |
| ABSTRACT .....                                                         | vi    |
| TABLE OF CONTENTS .....                                                | viii  |
| LIST OF TABLES .....                                                   | xii   |
| LIST OF FIGURES .....                                                  | xiv   |
| LIST OF SYMBOLS, ABBREVIATIONS AND NOMENCLATURE .....                  | xviii |
| CHAPTER 1 INTRODUCTION .....                                           | 1     |
| 1.1 Background and Motivation .....                                    | 1     |
| 1.2 Relevance of the study .....                                       | 2     |
| 1.3 Objectives .....                                                   | 3     |
| 1.4 Outline of thesis .....                                            | 4     |
| CHAPTER 2 LITERATURE REVIEW .....                                      | 6     |
| 2.1 Volume Loss .....                                                  | 6     |
| 2.2 Prediction Methods for Tunneling-Induced Ground Deformations ..... | 7     |
| 2.2.1 Empirical Methods .....                                          | 7     |
| 2.2.2 Analytical Methods .....                                         | 12    |
| 2.2.3 Experimental Methods .....                                       | 14    |
| 2.2.4 Numerical Methods .....                                          | 14    |
| 2.3 Building Risk Assessment (BRA) .....                               | 20    |

|                                                                                  |    |
|----------------------------------------------------------------------------------|----|
| 2.4 Research Gaps .....                                                          | 22 |
| CHAPTER 3 RESEARCH METHODOLOGY .....                                             | 24 |
| 3.1 Prediction of Greenfield Ground Movements due to Tunneling.....              | 25 |
| 3.2 Determination of building deformation measures.....                          | 26 |
| 3.2.1 Differential Settlement ( $\delta S_v$ ) .....                             | 26 |
| 3.2.2 Angular distortion ( $\beta$ ) .....                                       | 26 |
| 3.2.3 Relative deflection ( $\Delta$ ) and deflection ratio ( $\Delta/L$ ) ..... | 28 |
| 3.2.4 Horizontal Stain ( $\epsilon h$ ) .....                                    | 28 |
| 3.3 Limiting tensile strain method (LTSM) .....                                  | 29 |
| 3.4 Damage classification.....                                                   | 32 |
| 3.5 Staged Assessment .....                                                      | 36 |
| 3.5.1 Stage I: Preliminary assessments .....                                     | 36 |
| 3.5.2 Stage II: Second stage assessment .....                                    | 36 |
| 3.5.3 Stage III: Detailed Evaluation.....                                        | 37 |
| CHAPTER 4 STUDY AREA AND INPUT DATA.....                                         | 38 |
| 4.1 Geotechnical Data .....                                                      | 40 |
| 4.1.1 Geotechnical Investigations .....                                          | 40 |
| 4.1.2 Subsurface stratigraphy and water table .....                              | 46 |
| 4.2 Design Geotechnical Parameters.....                                          | 47 |
| 4.2.1 Undrained Shear Strength.....                                              | 47 |
| 4.2.2 Modulus of Elasticity of Soil .....                                        | 48 |
| 4.2.3 Poisson's ratio of soil.....                                               | 48 |
| 4.3 Tunnel and Lining Parameters.....                                            | 49 |
| CHAPTER 5 NUMERICAL MODELLING AND VALIDATION .....                               | 51 |
| 5.1 Material Models used in simulation .....                                     | 51 |
| 5.1.1 Linear elastic model.....                                                  | 51 |

|                                                                                               |     |
|-----------------------------------------------------------------------------------------------|-----|
| 5.1.2 Mohr-Coulomb model .....                                                                | 52  |
| 5.2 Numerical modelling .....                                                                 | 53  |
| 5.3 Results and Validation .....                                                              | 55  |
| CHAPTER 6 PARAMETRIC STUDY – RESULTS AND DISCUSSION .....                                     | 60  |
| 6.1 Greenfield Settlements: Parametric Study .....                                            | 60  |
| 6.1.1 Effect of Tunnel Parameters .....                                                       | 61  |
| 6.1.2 Effect of Soil Parameters .....                                                         | 65  |
| 6.2 Analytical Assessment of Existing Structures using LSTM .....                             | 70  |
| 6.3 Limiting tensile strains ( $\epsilon_{lim}$ )- Parametric Study .....                     | 71  |
| 6.3.1 Effect of Tunnel Parameters .....                                                       | 72  |
| 6.3.2 Effect of Soil Parameters .....                                                         | 81  |
| 6.3.3 Effect of Structure Parameters .....                                                    | 87  |
| 6.4 Discussion on Outcomes .....                                                              | 95  |
| CHAPTER 7 SEISMIC ANALYSIS .....                                                              | 97  |
| 7.1 Effect of seismic loading on ground deformations and $\epsilon_{lim}$ .....               | 98  |
| CHAPTER 8 CONCLUSION, FUTURE SCOPE, AND SOCIAL IMPACT .....                                   | 102 |
| 8.1 Conclusion .....                                                                          | 102 |
| 8.2 Recommended Threshold Values for Practical BRA Design in Indian Urban<br>Tunnelling ..... | 104 |
| 8.3 Future Scope .....                                                                        | 105 |
| 8.3.1 Adoption of Advanced Soil Models .....                                                  | 105 |
| 8.3.2 Three-Dimensional Modelling .....                                                       | 105 |
| 8.3.3 Seismic–Soil–Structure Interaction (SSI) .....                                          | 106 |
| 8.3.4 Refined Validation Approaches .....                                                     | 106 |
| 8.3.5 Probabilistic Risk Assessment .....                                                     | 106 |
| 8.3.6 Integration of Machine Learning for Tunnel-Induced Risk Assessment .....                | 106 |

|                                                              |     |
|--------------------------------------------------------------|-----|
| 8.4 Social Impact .....                                      | 107 |
| 8.4.1 Enhanced Public Safety .....                           | 107 |
| 8.4.2 Protection of Heritage and Vulnerable Structures ..... | 107 |
| 8.4.3 Minimization of Social Disruption .....                | 107 |
| 8.4.4 Support for Sustainable Urban Development .....        | 107 |
| 8.4.5 Economic and Livelihood Security.....                  | 108 |
| 8.4.6 Informed Policy and Regulatory Frameworks.....         | 108 |
| 8.4.7 Community Confidence in Infrastructure Projects .....  | 108 |
| REFERENCES.....                                              | 109 |
| ANNEXURE.....                                                | 123 |
| LIST OF PUBLICATIONS AND THEIR PROOFS.....                   | 134 |
| PLAGIARISM VERIFICATION .....                                | 138 |
| PLAGIARISM REPORT .....                                      | 139 |
| CURRICULUM VITAE/ BRIEF PROFILE .....                        | 140 |

## LIST OF TABLES

|                                                                                                                                                                                                                        |    |
|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----|
| Table 2-1 Review of Prior Studies .....                                                                                                                                                                                | 14 |
| Table 3-1 Damage classification based on crack width & $\epsilon_{lim}$ (Boscardin and<br>Cording, 1989; Burland et al., 1977; Gaba et al., 2003; Guglielmetti et al., 2008;<br>Mair et al., 1996; Rankin, 1988) ..... | 33 |
| Table 3-2 Damage classification based on $S_{bmax}$ and $\beta_{max}$ (Guglielmetti et al.,<br>2008; Rankin, 1988) .....                                                                                               | 34 |
| Table 4-1 Laboratory Tests .....                                                                                                                                                                                       | 43 |
| Table 4-2 Classification of Subsurface Soil Layers .....                                                                                                                                                               | 46 |
| Table 4-3 Range of Poisson's ratio values for different types of soils (Bowles, 1996)<br>.....                                                                                                                         | 49 |
| Table 4-4 Soil Properties.....                                                                                                                                                                                         | 49 |
| Table 4-5 Material properties of TBM and lining.....                                                                                                                                                                   | 50 |
| Table 6-1 Range of parameters variation for Greenfield settlements: parametric study<br>.....                                                                                                                          | 61 |
| Table 6-2 Relative change in $S_{max}$ with $D$ .....                                                                                                                                                                  | 62 |
| Table 6-3 Relative change in $S_{max}$ with $z$ .....                                                                                                                                                                  | 64 |
| Table 6-4 Relative change in $S_{max}$ with $V_L$ .....                                                                                                                                                                | 65 |
| Table 6-5 Relative change in $S_{max}$ with $s_u$ under constant $V_L$ .....                                                                                                                                           | 67 |
| Table 6-6 Relative change in $S_{max}$ with $s_u$ under constant GP .....                                                                                                                                              | 68 |
| Table 6-7 Relative change in $S_{max}$ with $E'_{ref}$ under constant GP .....                                                                                                                                         | 70 |
| Table 6-8 Results of the LTSM application.....                                                                                                                                                                         | 71 |
| Table 6-9 Range of parameters variation for $\epsilon_{lim}$ and damage category: parametric<br>study.....                                                                                                             | 72 |
| Table 6-10 Effect of $D$ on building deformation measures and damage categories ..                                                                                                                                     | 74 |
| Table 6-11 Effect of $z$ on building deformation measures and damage categories ...                                                                                                                                    | 76 |
| Table 6-12 Effect of $V_L$ on building deformation measures and damage categories .                                                                                                                                    | 79 |
| Table 6-13 Effect of $s_u$ and $\phi'$ on building deformation measures and damage<br>categories .....                                                                                                                 | 83 |
| Table 6-14 Effect of $E'_{ref}$ on building deformation measures and damage categories<br>.....                                                                                                                        | 86 |

|                                                                                           |     |
|-------------------------------------------------------------------------------------------|-----|
| Table 6-15 Effect of $H$ on building deformation measures and damage categories ..        | 88  |
| Table 6-16 Effect of $L$ on building deformation measures and damage categories...        | 91  |
| Table 6-17 Effect of $X$ on building deformation measures and damage categories...        | 93  |
| Table 7-1 Effect of Seismic Loading .....                                                 | 99  |
| Table 7-2 Relative change in $Sb_{max}$ and $\epsilon_{lim}$ due to Seismic loading ..... | 101 |
| Table 8-1 Recommended Threshold for $V_L$ .....                                           | 104 |
| Table 8-2 Recommended Threshold for $z$ .....                                             | 104 |
| Table 8-3 Recommended Threshold for $X$ .....                                             | 105 |
| Table 8-4 Recommended Threshold for $\epsilon_{lim}$ .....                                | 105 |

## LIST OF FIGURES

|                                                                                                                                                                                                             |    |
|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----|
| Fig. 2.1 Sources of volume loss for a shield driven tunnel (Cording, 1991) .....                                                                                                                            | 7  |
| Fig. 2.2 Transverse surface settlement trough, Gaussian distribution (Agarwal and Sarkar, 2024).....                                                                                                        | 8  |
| Fig. 2.3 Empirical-analytical curves for tunnelling-induced deformations in the transverse direction (Giardina, 2013) .....                                                                                 | 10 |
| Fig. 2.4 Surface and Sub-surface settlement profiles (Mair et al.,1993) .....                                                                                                                               | 10 |
| Fig. 2.5 Deformation of building above tunnelling (Harris and Franzius, 2005) .....                                                                                                                         | 21 |
| Fig. 3.1 Flow Chart- Approach and Methodology .....                                                                                                                                                         | 24 |
| Fig. 3.2 Different positions of buildings in the transverse settlement trough: (i) sagging, (ii) at an inflection point and (iii) hogging .....                                                             | 25 |
| Fig. 3.3 Deformation measures for a building conforming to a ground settlement profile in (a) the sagging zone; (b) both the sagging and hogging zones; and (c) the hogging zone (Dalgic et al., 2017)..... | 27 |
| Fig. 3.4 Cracking of a simple beam in bending and in shear (Burland et al., 1977)..                                                                                                                         | 30 |
| Fig. 3.5 Damage Category- Angular Distortion ( $\beta$ ) and Horizontal Strain ( $\epsilon_h$ ) (Boscardin and Cording, 1989).....                                                                          | 35 |
| Fig. 3.6 Damage Category- Deflection ratio ( $\Delta/L$ ) and Horizontal Strain ( $\epsilon_h$ ) (Burland, 1995 and Burland et al., 2001) .....                                                             | 36 |
| Fig. 4.1 Tunnel Alignment.....                                                                                                                                                                              | 38 |
| Fig. 4.2 Study Area- Google Earth View of RRTS Package - 4.....                                                                                                                                             | 39 |
| Fig. 4.3 Google Earth View of adjacent surface Structures.....                                                                                                                                              | 40 |
| Fig. 4.4 Study Area- Borehole Location .....                                                                                                                                                                | 41 |
| Fig. 4.5 Variation of SPT N with depth .....                                                                                                                                                                | 42 |
| Fig. 4.6 Variation of $N_{60}$ with depth .....                                                                                                                                                             | 42 |
| Fig. 4.7 Variation of dry density with depth .....                                                                                                                                                          | 43 |
| Fig. 4.8 Variation of bulk density with depth .....                                                                                                                                                         | 44 |
| Fig. 4.9 Variation of undrained cohesion with depth.....                                                                                                                                                    | 44 |
| Fig. 4.10 Variation of undrained friction angle with depth.....                                                                                                                                             | 45 |
| Fig. 4.11 Variation of drained friction angle with depth.....                                                                                                                                               | 45 |
| Fig. 4.12 Variation of plasticity index with depth .....                                                                                                                                                    | 46 |

|                                                                                                                                                                                                         |    |
|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----|
| Fig. 4.13 Study Area- Geological Profile.....                                                                                                                                                           | 47 |
| Fig. 4.14 Relationship between $C_u/N$ and Plasticity Index (CIRIA Report 143, 1995)<br>.....                                                                                                           | 48 |
| Fig. 5.1 Schematic representation of the Finite Element Method modelling process                                                                                                                        | 51 |
| Fig. 5.2 Linear elastic model.....                                                                                                                                                                      | 52 |
| Fig. 5.3 Elastic-perfectly plastic model (PLAXIS 2D Material Models Manual, 2013)<br>.....                                                                                                              | 52 |
| Fig. 5.4 Stress circles at yield; one touches Coulomb's envelope. (a) Using effective<br>strength parameters; (b) Using undrained strength parameters (PLAXIS 2D<br>Material Models Manual, 2022) ..... | 53 |
| Fig. 5.5 PLAXIS 2D model of existing ground conditions with tunnel .....                                                                                                                                | 54 |
| Fig. 5.6 PLAXIS 2D model after meshing.....                                                                                                                                                             | 54 |
| Fig. 5.7 PLAXIS Output - Deformed Mesh after Calculations.....                                                                                                                                          | 55 |
| Fig. 5.8 PLAXIS Output – Total deformations.....                                                                                                                                                        | 56 |
| Fig. 5.9 PLAXIS Output –Total Vertical Displacements .....                                                                                                                                              | 56 |
| Fig. 5.10 PLAXIS Output –Total Horizontal Displacements .....                                                                                                                                           | 57 |
| Fig. 5.11 PLAXIS Output –Vertical ground surface settlement .....                                                                                                                                       | 57 |
| Fig. 5.12 PLAXIS Output – Horizontal ground surface displacement .....                                                                                                                                  | 58 |
| Fig. 5.13 Comparison of vertical ground surface settlements obtained from PLAXIS<br>2D and empirical methods .....                                                                                      | 59 |
| Fig. 5.14 Comparison of Horizontal ground surface displacement obtained from<br>PLAXIS 2D and empirical methods .....                                                                                   | 59 |
| Fig. 6.1 Variation of Surface Settlement Profiles with $D$ .....                                                                                                                                        | 62 |
| Fig. 6.2 Variation of <b><math>S_{max}</math></b> with $D$ .....                                                                                                                                        | 62 |
| Fig. 6.3 Variation of Surface Settlement Profiles with $z$ .....                                                                                                                                        | 63 |
| Fig. 6.4 Variation of <b><math>S_{max}</math></b> with $z$ .....                                                                                                                                        | 63 |
| Fig. 6.5 Variation of Surface Settlement Profiles with $V_L$ .....                                                                                                                                      | 64 |
| Fig. 6.6 Variation of <b><math>S_{max}</math></b> with $V_L$ .....                                                                                                                                      | 65 |
| Fig. 6.7 Variation of Surface Settlement Profiles with $s_u$ under constant $V_L$ .....                                                                                                                 | 66 |
| Fig. 6.8 Variation of Surface Settlement Profiles with $s_u$ under constant GP.....                                                                                                                     | 66 |
| Fig. 6.9 Variation of <b><math>S_{max}</math></b> with $s_u$ under constant $V_L$ .....                                                                                                                 | 67 |
| Fig. 6.10 Variation of <b><math>S_{max}</math></b> with $s_u$ under constant GP.....                                                                                                                    | 67 |

|                                                                                                                                     |    |
|-------------------------------------------------------------------------------------------------------------------------------------|----|
| Fig. 6.11 Variation of Surface Settlement Profiles with $E'_{ref}$ under constant $V_L$ .....                                       | 68 |
| Fig. 6.12 Variation of Surface Settlement Profiles with $E'_{ref}$ under constant GP .....                                          | 69 |
| Fig. 6.13 Variation of <b><i>S<sub>max</sub></i></b> with $E'_{ref}$ under constant GP .....                                        | 69 |
| Fig. 6.14 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building .....                    | 70 |
| Fig. 6.15 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building ( $D = 5.5$ m) .....     | 73 |
| Fig. 6.16 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building ( $D = 7.5$ m) .....     | 73 |
| Fig. 6.17 Damage category for various $D$ as per Boscardin and Cording (1989) .....                                                 | 74 |
| Fig. 6.18 Damage category for various $D$ as per as per Burland (1995) and Burland et al. (2001) .....                              | 74 |
| Fig. 6.19 Variation of <b><i>ε<sub>lim</sub></i></b> with $D$ .....                                                                 | 75 |
| Fig. 6.20 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building ( $z = 10$ m) .....      | 76 |
| Fig. 6.21 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building ( $z = 20$ m) .....      | 76 |
| Fig. 6.22 Damage category for various $z$ as per Boscardin and Cording (1989) .....                                                 | 77 |
| Fig. 6.23 Damage category for various $z$ as per as per Burland (1995) and Burland et al. (2001) .....                              | 77 |
| Fig. 6.24 Variation of <b><i>ε<sub>lim</sub></i></b> with $z$ .....                                                                 | 78 |
| Fig. 6.25 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building ( $V_L = 1\%$ ) .....    | 79 |
| Fig. 6.26 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building ( $V_L = 2\%$ ) .....    | 79 |
| Fig. 6.27 Damage category for various $V_L$ as per Boscardin and Cording (1989) .....                                               | 80 |
| Fig. 6.28 Damage category for various $V_L$ as per as per Burland (1995) and Burland et al. (2001) .....                            | 80 |
| Fig. 6.29 Variation of <b><i>ε<sub>lim</sub></i></b> with $V_L$ .....                                                               | 81 |
| Fig. 6.30 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building (Case 1) .....           | 82 |
| Fig. 6.31 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building (Case 3) .....           | 82 |
| Fig. 6.32 Damage category for various $s_u$ and $\phi'$ as per Boscardin and Cording (1989) .....                                   | 83 |
| Fig. 6.33 Damage category for various $s_u$ and $\phi'$ as per as per Burland (1995) and Burland et al. (2001) .....                | 83 |
| Fig. 6.34 Variation of <b><i>ε<sub>lim</sub></i></b> with $s_u$ and $\phi'$ .....                                                   | 84 |
| Fig. 6.35 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building ( $0.5 E'_{ref}$ ) ..... | 85 |
| Fig. 6.36 Ground deformations ( <b><i>S<sub>x</sub></i></b> and <b><i>H<sub>x</sub></i></b> ) and Building ( $1.5 E'_{ref}$ ) ..... | 85 |

|                                                                                                                          |     |
|--------------------------------------------------------------------------------------------------------------------------|-----|
| Fig. 6.37 Damage category for various $E'_{ref}$ as per Boscardin and Cording (1989).                                    | 86  |
| Fig. 6.38 Damage category for various $E'_{ref}$ as per as per Burland (1995) and Burland et al. (2001).....             | 86  |
| Fig. 6.39 Variation of $\epsilon_{lim}$ with $E'_{ref}$ .....                                                            | 87  |
| Fig. 6.40 Ground deformations ( $Sx$ and $Hx$ ) and Building for structure parameters effect .....                       | 87  |
| Fig. 6.41 Damage category for various $H$ as per Boscardin and Cording (1989).....                                       | 89  |
| Fig. 6.42 Damage category for various $H$ as per as per Burland (1995) and Burland et al. (2001).....                    | 89  |
| Fig. 6.43 Variation of $\epsilon_{lim}$ with $H$ .....                                                                   | 90  |
| Fig. 6.44 Damage category for various $L$ as per Boscardin and Cording (1989) .....                                      | 91  |
| Fig. 6.45 Damage category for various $L$ as per as per Burland (1995) and Burland et al. (2001).....                    | 92  |
| Fig. 6.46 Variation of $\epsilon_{lim}$ with $L$ .....                                                                   | 92  |
| Fig. 6.47 Damage category for various $X$ as per Boscardin and Cording (1989) .....                                      | 93  |
| Fig. 6.48 Damage category for various $X$ as per as per Burland (1995) and Burland et al. (2001).....                    | 94  |
| Fig. 6.49 Variation of $\epsilon_{lim}$ with $X$ .....                                                                   | 94  |
| Fig. 7.1 Ground deformations ( $Sx$ and $Hx$ ) and Building (seismic load, $z = 10$ m) .                                 | 98  |
| Fig. 7.2 Ground deformations ( $Sx$ and $Hx$ ) and Building (seismic load, $z = 15$ m) .                                 | 98  |
| Fig. 7.3 Ground deformations ( $Sx$ and $Hx$ ) and Building (after earthquake, $z = 20$ m) .....                         | 99  |
| Fig. 7.4 Damage category for various $z$ and seismic loading as per Boscardin and Cording (1989).....                    | 100 |
| Fig. 7.5 Damage category for various $z$ and seismic loading as per as per Burland (1995) and Burland et al. (2001)..... | 100 |

## List of Symbols, Abbreviations and Nomenclature

|               |                                                                          |
|---------------|--------------------------------------------------------------------------|
| $\beta$       | Angular distortion                                                       |
| $z$           | Axial depth of the tunnel from ground surface                            |
| $EA$          | Axial Stiffness                                                          |
| $EI$          | Bending Stiffness                                                        |
| BCS           | Building Condition Survey                                                |
| BRA           | Building Risk Assessment                                                 |
| $P$           | Central point load on fictitious deep beam                               |
| $c'_{ref}$    | Cohesion of soil used in numerical simulation                            |
| $C_B$         | Correction factor for borehole diameter                                  |
| $C_E$         | Correction factor for hammer energy ratio                                |
| $C_R$         | Correction factor for rod length                                         |
| $C_S$         | Correction factor for sampler type                                       |
| $\Delta/L$    | Deflection ratio                                                         |
| $z_0$         | Depth from the ground surface                                            |
| $t$           | Depth of the neutral axis from the edge of the beam in tension           |
| $S_a/g$       | Design acceleration coefficient                                          |
| $\delta S_h$  | Differential horizontal displacement                                     |
| $\delta S_v$  | Differential settlement                                                  |
| $\psi$        | Dilatancy angle                                                          |
| $b$           | Dimensionless parameter by Celestino et al. (2000)                       |
| $X$           | Distance between the building and tunnel axis                            |
| EPB           | Earth Pressure Balance                                                   |
| $\Psi$        | Empirical parameter by Oteo and Moya (1979) derived from monitoring data |
| $\varepsilon$ | Equivalent undrained ground loss                                         |
| SM            | Fine silty sand                                                          |
| FDM           | Finite Difference Method                                                 |
| FEM           | Finite Element Method                                                    |
| F             | Framed                                                                   |

|                      |                                                                                                                  |
|----------------------|------------------------------------------------------------------------------------------------------------------|
| $\phi'$              | Friction angle of soil used in numerical simulation                                                              |
| $g$                  | Gap parameter                                                                                                    |
| GP                   | Grout pressure                                                                                                   |
| $H$                  | Height of building                                                                                               |
| Hog                  | Hogging                                                                                                          |
| $S_h$                | Horizontal displacement of building point                                                                        |
| $i$                  | Horizontal distance between centreline of the tunnel and the inflection points                                   |
| $x$                  | Horizontal distance from the center line of the tunnel                                                           |
| $H_x$                | Horizontal ground displacement at a distance $x$                                                                 |
| $k_h$                | Horizontal seismic coefficient                                                                                   |
| $\varepsilon_h$      | Horizontal strain                                                                                                |
| $I_S$                | Importance factor                                                                                                |
| $R_{inter}$          | Interface strength reduction factor                                                                              |
| $a$                  | Length dimension by Celestino et al. (2000)                                                                      |
| $L$                  | Length of building                                                                                               |
| $\varepsilon_{lim}$  | Limiting tensile strain                                                                                          |
| LTSM                 | Limiting Tensile Strain Method                                                                                   |
| CL                   | Low-plasticity sandy clayey silt                                                                                 |
| M                    | Masonry                                                                                                          |
| $\beta_{max}$        | Maximum angular distortion                                                                                       |
| $\varepsilon_{bmax}$ | Maximum bending tensile strain                                                                                   |
| $\varepsilon_{dmax}$ | Maximum diagonal tensile strain                                                                                  |
| $S_{bmax}$           | Maximum settlement of building                                                                                   |
| $S_{max}$            | Maximum vertical ground settlement                                                                               |
| Mod.                 | Moderate                                                                                                         |
| $i'$                 | Modified horizontal distance between centreline of the tunnel and the inflection points by Jacobsz et al. (2004) |
| $k'$                 | Modified trough width parameter by Jacobsz et al. (2004)                                                         |
| $E$                  | Modulus of elasticity                                                                                            |
| $E'$                 | Modulus of Elasticity of Soil (drained)                                                                          |
| $E_u$                | Modulus of Elasticity of Soil (undrained)                                                                        |

|                    |                                                                   |
|--------------------|-------------------------------------------------------------------|
| Neg.               | Negligible                                                        |
| $\alpha$           | Parameter by Vorster et al. (2005)                                |
| $\nu$              | Poisson's ratio of material                                       |
| RRTS               | Regional Rapid Transit System                                     |
| $\Delta$           | Relative deflection                                               |
| $E/G$              | Relative stiffness/ stiffness ratio of building                   |
| $R$                | Response reduction factor                                         |
| $\omega$           | Rigid building rotation (tilt)                                    |
| Sag                | Sagging                                                           |
| $\gamma_{sat}$     | Saturated unit weight of soil used in numerical simulation        |
| $I$                | Second moment of the area of fictitious deep beam                 |
| $k$                | Settlement trough width parameter                                 |
| S.                 | Severe                                                            |
| $n$                | Shape function parameter by Vorster et al. (2005)                 |
| $\delta$           | Shape ovalization factor.                                         |
| $G$                | Shear modulus                                                     |
| Sl.                | Slight                                                            |
| $\varphi$          | Slope of the ground settlement profile                            |
| $c$                | Soil cohesion                                                     |
| $\phi$             | Soil friction angle                                               |
| $E_s$              | Soil stiffness                                                    |
| SSI                | Soil–Structure Interaction                                        |
| $N$                | SPT blow count for 30 cm penetration                              |
| $N_{60}$           | SPT blow count for a hammer efficiency of 60%.                    |
| SPT                | Standard Penetration Test                                         |
| $s$                | Subsurface settlement at a depth of $z_0$ from the ground surface |
| 3D                 | Three-dimensional                                                 |
| $\varepsilon_{dt}$ | Total diagonal tensile strain                                     |
| $\varepsilon_{bt}$ | Total tensile strain in bending                                   |
| TBM                | Tunnel Boring Machine                                             |
| $D$                | Tunnel diameter                                                   |
| $r$                | Tunnel radius                                                     |

|                  |                                                                 |
|------------------|-----------------------------------------------------------------|
| 2D               | Two-dimensional                                                 |
| $C_u$            | Undrained shear strength of soil                                |
| $s_u$            | Undrained shear strength of soil used in numerical simulation   |
| $w$              | Unit Weight                                                     |
| $\gamma$         | Unit weight of soil                                             |
| $\gamma_{unsat}$ | Unsaturated unit weight of soil used in numerical simulation    |
| $S_x$            | Vertical ground settlement at a distance $x$                    |
| $k_v$            | Vertical seismic coefficient                                    |
| $S_v$            | Vertical settlement of building point                           |
| V. S.            | Very Severe                                                     |
| V. Sl.           | Very Slight                                                     |
| $V_L$            | Volume loss                                                     |
| $V_T$            | Volume of soil moving into the tunnel cavity (per metre length) |
| $E'_{ref}$       | Young's modulus of soil used in numerical simulation            |
| $Z$              | Zone factor                                                     |

# CHAPTER 1

## INTRODUCTION

### 1.1 Background and Motivation

Tunneling has become an indispensable component of modern infrastructure development, particularly in densely populated urban areas where surface land availability is severely constrained. A tunnel is defined as an artificial underground passage constructed to by-pass surface obstacles and enable the safe transport of passengers, water, sewage, and other utilities without disrupting the overlying terrain (Srinivasan and Bhavsar, 1983). In recent decades, tunnel construction has gained considerable momentum in India, driven by the large need to address increasing traffic congestion and to promote sustainable urban development. Metropolitan cities of India such as Delhi, Mumbai, Chennai, Bangalore, and Kolkata have increasingly embraced underground metro systems as a strategic solution to address limited space availability at the surface and growing transportation demands. Among these, Delhi serves as a prominent example, where extensive underground tunneling networks have been implemented.

As tunneling in urban areas is typically carried out at shallow depths with soil as the primary geo-material, the induced ground deformations can easily propagate up to the ground surface and significantly affect integrity of the existing structures leading to structural distortion and cracks (Schiena et al., 2024). Several techniques such as cut-and-fill, blasting, and mechanized tunneling are employed for tunnel construction, each exerting distinct influences on the surrounding environment (Huat et al., 2024). Among these tunnel boring machines (TBMs) have revolutionized modern tunneling by introducing mechanization and automation that accelerate construction and enhance worker safety (Armaghani, 2015). Although closed-face pressurized TBMs are capable of significantly reducing deformations around the excavation cavity and thereby limiting surface settlements, they cannot entirely eliminate them, as minor over-excavation effects are inevitable (Lee et al., 1992). Thus, while TBMs have made underground construction faster, safer, and less disruptive at the surface, tunneling inevitably causes ground movements that may adversely affect surrounding structures. Consequently, evaluating the impact of tunneling on existing structures has emerged as a critical concern for both engineers and researchers for the successful planning and execution of urban infrastructure projects.

## 1.2 Relevance of the study

The success of tunneling projects heavily relies on precise decisions made during the design, analysis, and construction phases. A key component of this process is the Building Risk Assessment (BRA), which evaluates the likelihood of damage to structures potentially affected by tunneling-induced ground movements. BRA involves estimating possible structural damages based on predicted ground deformations and the vulnerability of nearby buildings (Guglielmetti et al., 2008). Below is the summary of tunneling-related incidents in India, which emphasize the necessity of conducting comprehensive risk assessments before tunneling begins.

- **Agra Metro**

In Agra, tunneling activities for a 2 km underground metro corridor connecting Agra College to Mankameshwar Mandir station commenced in October 2023. Residents in areas such as Moti Katra and Syed Gali have reported cracks in over 150 buildings, along with signs of structural distress observed in a nearly 200-year-old temple, reportedly coinciding with nearby tunneling activities. Precautionary measures have been taken, including the relocation of several families and the installation of iron jacks to stabilize homes (Times of India, 2024).

- **Mumbai Metro Line 7A**

During the construction of Mumbai Metro Line 7A, tunneling activity triggered a significant ground subsidence along Sahar Road at P&T Colony, Andheri East, where a 24 feet deep and 10 feet wide cave-in occurred in August 2024. As a precaution, tunneling was suspended, and stabilization was carried out by grouting and concreting, requiring nearly 20 truckloads of reinforced cement concrete. While no casualties were reported, nine families from nearby buildings were evacuated and temporarily accommodated in a hotel by the contractor. Expert assessments suggested that undetected underground cavities and weak soil conditions were the likely causes, highlighting deficiencies in prior geotechnical investigation and the challenges of ensuring ground stability in densely built urban settings (Times of India, 2024).

- **Bengaluru Metro**

In Bengaluru, tunneling activities for the Namma Metro's underground section between Dairy Circle and Nagawara (14 km) have raised structural concerns in densely populated areas such as Shivajinagar and Bamboo Bazaar. Cracks have appeared in several buildings, and damage to roads has been reported, prompting the temporary relocation of some residents as a precaution. Additionally, a 10-feet sinkhole was reported near the Shivajinagar Metro Station, reportedly linked to tunneling-related disturbances (Times of India, 2020).

- **Delhi Metro**

A 113-year-old heritage building at Kashmere Gate partially collapsed during Delhi Metro's tunneling work for Central Secretariat–Kashmere Gate Line, highlighting the severe risks posed to historic structures in densely built environments. The incident was attributed to foundation weakening from sewer leakage and construction

vibrations (Times of India, 2015; Manchanda and Agarwal, 2022).

- **Chennai Metro**

Chennai Metro tunneling has caused visible damage to over 300 buildings across the city, including 212 in Phase-I along a 24 km underground corridor, with at least 99 affected in North Chennai alone—most of which were already structurally weak and existing cracks widen due to TBM-induced vibrations, especially in areas like Washermenpet, Egmore, Anna Nagar etc. More notably, two historic churches in Broadway CSI Tamil Wesley Church (168 years old) and Arcot Lutheran Church (built in 1892), developed cracks due to metro tunneling (Times of India, 2013, 2017).

These incidents collectively provide quantitative justification for the need for a systematic BRA framework. The reported structural impacts involving more than 150 buildings in Agra and over 300 buildings in Chennai indicate that tunneling-induced ground movements can affect a substantial number of structures within densely developed urban corridors. Furthermore, localized failures, such as the cave-in reported during Mumbai Metro construction and the sinkhole observed in Bengaluru, suggest that inadequate prediction of ground response or excessive ground loss may lead to significant serviceability and stability concerns. From an engineering perspective, these incidents highlight the importance of predicting tunnelling-induced ground deformations and associated structural vulnerability within the tunnel influence zone prior to excavation, thereby enabling the identification and implementation of suitable mitigation measures before tunnel advancement. Accordingly, the present study develops a BRA framework integrating tunnel geometry, subsurface soil conditions, volume loss, and building vulnerability to facilitate quantitative assessment of tunneling-induced risk under Indian urban conditions.

### 1.3 Objectives

The present study is undertaken to advance the understanding of analytical and numerical methodologies for predicting building damage associated with tunneling, which is of critical importance for ensuring the safety and resilience of urban infrastructure in rapidly developing cities. A novel aspect of this research lies in the integration of numerical and analytical methods, which offers a more comprehensive and robust framework for damage prediction compared to conventional approaches. In order to achieve the aim of this study, the research has been structured around the progressive attainment of the specific sub-targets outlined below:

- Estimate the free-field ground (greenfield) movements caused by the tunnel excavation.
- Study and compare the deformations and strains developed over the adjacent structures due to the construction of Tunnel.
- Evaluate the effects of Tunnel parameters such as tunnel diameter, tunnel depth from the earth surface, and location of tunnel axis with respect to structure.
- Evaluate the effects of soil parameters such as stiffness and strength properties.
- Evaluate the effects of structure parameters such as length, height and stiffness

- etc.
- Study the deformations and strains in adjacent structures in both case of occurrence and non-occurrence of an earthquake.

#### 1.4 Outline of thesis

The thesis is presented according to the following chapter headings, the content of which is also briefly described below:

**Chapter 2** focuses on literature review of the extensive research concerning tunneling-induced ground deformations and their impact on surrounding structures. Current engineering practices for predicting tunneling-induced ground deformations including currently available empirical, analytical, experimental and numerical methods are discussed in detail. A substantial review of International and Indian case studies on prediction of tunneling-induced ground deformations and their impact on surrounding structures is summarized. The chapter identifies the key research gaps, emphasizing the need for an integrated analytical–numerical approach for building risk assessment (BRA) and the limited focus on seismic considerations in the Indian context.

**Chapter 3** details the overall approach and methodology adopted in the study. A step-by-step approach is introduced to predict ground deformations and building strains, along with commonly used building damage assessment frameworks such as Burland’s classification and the Limiting Tensile Strain Method (LTSM). A methodology flowchart is also presented to explain the stepwise process followed in this research.

**Chapter 4** presents the specific research problem in the context of the Delhi–Meerut RRTS project, India. It describes the alignment, tunneling details, and surrounding built-up environment. The geological profile of the site is outlined, supported by soil investigation data and properties. Tunnel parameters, TBM specifications, and cover depth are also presented. The chapter establishes the basis for numerical modelling and analysis in subsequent chapters.

**Chapter 5** develops the baseline numerical model for predicting tunneling-induced ground deformations. The model geometry, mesh configuration, boundary conditions, and soil constitutive models are described. The simulation of TBM tunneling stages is explained, and the model is validated against empirical-analytical solutions. This validation builds confidence in the applicability of the integrated methodology to Indian tunneling projects.

**Chapter 6** presents a detailed parametric investigation of tunneling-induced effects on ground deformations and building response. Parameters studied include tunnel depth, diameter, volume loss, soil and structure characteristics. Building strains are computed using LTSM, and structural damage is classified according to

standard frameworks. Results of this study are critically discussed in the context of urban tunneling.

**Chapter 7** evaluates the influence of seismic loading on tunneling–induced ground deformations and building response, an aspect often overlooked in Indian tunneling projects. Numerical simulations are performed to assess the response of tunnels and adjacent structures under earthquake loading. The results highlight the additional risks posed by seismic effects and the implications building safety in seismically active regions.

**Chapter 8** summarizes the key outcomes of the study, emphasizing the contributions of the integrated analytical–numerical methodology and the findings from the parametric and seismic analyses. The practical implications for metro and RRTS tunneling projects in India are highlighted, along with the social impact of safer tunneling practices in densely populated urban environments. Limitations of the study are acknowledged, and directions for future research are proposed.

## CHAPTER 2

### LITERATURE REVIEW

Tunneling invariably leads to ground movements and changes in in-situ stress conditions, which becomes a major concern in urban areas where tunnels are constructed adjacent to the foundations of existing structures (El Naggar et al., 2020; Fattah et al., 2011; Ghahremannejad et al., 2006). The magnitude of ground movements occurring above a tunnel, is primarily characterized by the parameter termed ‘volume loss’. This chapter presents an extensive review of existing methods for predicting tunnelling-induced ground deformations and building response.

#### 2.1 Volume Loss

One of the primary concerns associated with mechanized tunnelling is volume loss ( $V_L$ ), a phenomenon where a portion of the ground surrounding the tunnel cavity moves into the void created during excavation triggered by radial and longitudinal soil deformations resulting from stress release due to delayed installation of tunnel lining (Al-Omari et al., 2019; Fu et al., 2016; Islam and Iskander, 2021). This volume loss leads to settlement of the ground surface, which, in turn, can cause structural damage to nearby buildings and infrastructure. The volume loss ( $V_L$ ) is defined as the ratio of volume (per metre length) of soil moving into the tunnel cavity ( $V_T$ ) to the actual tunnel volume (per metre length) (Attewell and Farmer, 1974; ElMouchi et al., 2018; Franzius et al., 2006; Loganathan and Poulos, 1998; Loganathan et al., 2000; Peck, 1969; Shiau and Sams, 2019). Total volume loss around the tunnel cavity includes: (i) ground loss at the face of the tunnel (caused by movement of soil towards the excavation chamber), (ii) ground loss along the shield (due to progressive decrement of shield diameter from the cutter head to the TBM tail), (iii) ground loss at the tail (Due to the cavity between the tunnel lining and surrounding soil and movement of soil into the gap caused by reduction in grout pressure), (iv) ground loss behind the shield tail (caused by consolidation of grout) and, (v) long term volume loss caused by grout shrinkage and lining deformations (Fig. 2.1) (Al-Omari et al., 2019; Attewell and Farmer, 1974; Chakeri and Ünver, 2014; Devriendt, 2011; Dias and Bezuijen, 2015; Golpasand et al., 2016; Lee et al., 1992; Vu et al., 2016;). In case of clay considering non-dilation and undrained condition volume of surface settlement trough per unit length ( $V_s$ ) should be equal to the volume of soil moving into the tunnel cavity ( $V_T$ ) (Attewell and Farmer, 1974; Franzius et al., 2006; Marshall et al., 2012; Shiau and Sams, 2019). Cohesionless soils like fills, silts, sands, and unsaturated clays provide some drainage which causes actual surface settlement trough to have a different volume than the volume of ground that moves into the tunnel cavity as a result of volume change within the soil (Islam and Iskander, 2021). Consolidation of clays may also cause sublayer subsidence resulting in additional ground movements along with those obtained due to volume loss during tunnelling activity (Wu et al.,

2017). The volume of the surface settlement trough may be less than the amount of ground that moves into the tunnel due to dilatation and swelling brought on by unloading that may cause soil expansion (Möller and Vermeer, 2008).

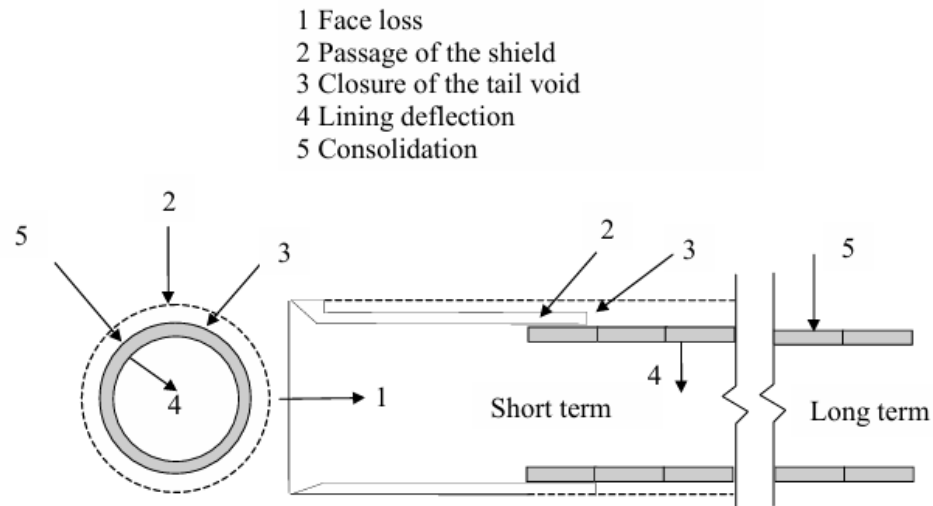


Fig. 2.1 Sources of volume loss for a shield driven tunnel (Cording, 1991)

## 2.2 Prediction Methods for Tunneling-Induced Ground Deformations

Accurate prediction of tunnelling-induced ground deformations is a fundamental requirement for the successful planning and execution of urban infrastructure projects. Numerous researchers have investigated this problem and developed various approaches for estimating the magnitude and distribution of ground deformations due to tunneling. These approaches can broadly be categorized into analytical methods, empirical methods, physical modelling techniques, and numerical simulations. Each method has its own theoretical basis, advantages, and limitations, which are discussed in following sections (Fattah et al., 2011; Fattah et al., 2013; Wang et al., 2016).

### 2.2.1 Empirical Methods

Empirical methods for predicting tunneling-induced ground deformations are primarily derived from field observations and laboratory results case histories. These methods establish correlations between settlement and influencing factors such as ground loss, soil type, and tunnel depth. A major advantage of the empirical method is that ground settlements in a project of similar geological conditions, construction methods, and workmanship can be easily and reasonably predicted (Hsieh and Ou, 1998). Although empirical approaches are straightforward and provide quick estimates, their accuracy is highly dependent on the similarity between the conditions of the current project and the case histories from which they were developed (Loganathan et al., 2000; Shinde et al., 2020). Consequently, while they are invaluable

for preliminary assessments, their applicability is often limited in new or complex ground conditions. The most widely known and classic empirical approach is the Peck (1969) formula, which represents surface settlement troughs using a Gaussian distribution. Firstly, Peck (1969) developed an empirical method to estimate tunnelling-induced settlement using a Gaussian distribution based on field observations, which is a widely adopted approach till now and verified by many field observations in various case studies. This approach has since been modified by many researchers to suit different ground conditions. O'Reilly and New (1982) extended Peck's vertical Gaussian settlement model by introducing a relationship to estimate horizontal displacements. Mair et al. (1993) proposed a method to better predict subsurface settlement profiles above tunnels in clays by modifying the trough width parameter to vary with depth. Celestino et al. (2000) proposed new empirical relation using a family of density curve functions which offers more flexibility and allow for a better fit to actual measured settlement troughs, especially when the observed data shows asymmetry or non-Gaussian behaviour. Jacobsz et al. (2004) proposed a modified empirical settlement equation based on centrifuge model data, improving upon the classical Gaussian distribution used in tunneling induced settlement prediction. Wang et al. (2016) proposed a modified empirical model for tunneling-induced settlements in sands by incorporating the variation of settlement trough volume and width with depth.

The empirical method proposed by Peck (1969) is most widely adopted approach for estimating tunnelling-induced ground settlements, particularly under greenfield conditions (i.e., in the absence of any adjacent structures). The method assumes that the transverse settlement profile across the tunnel axis follows a Gaussian distribution curve, which has been shown to closely approximate both experimental observations and field measurements. The popularity of this approach is attributed to its effectiveness, simplicity, and reliance on a few easy to determine parameters. Using this method, the vertical ground settlement ( $S_x$ ) at a horizontal distance ( $x$ ) from the center line of the tunnel, can be calculated by the Gaussian function, as expressed in Eqn. 2.1 (Fig. 2.2).

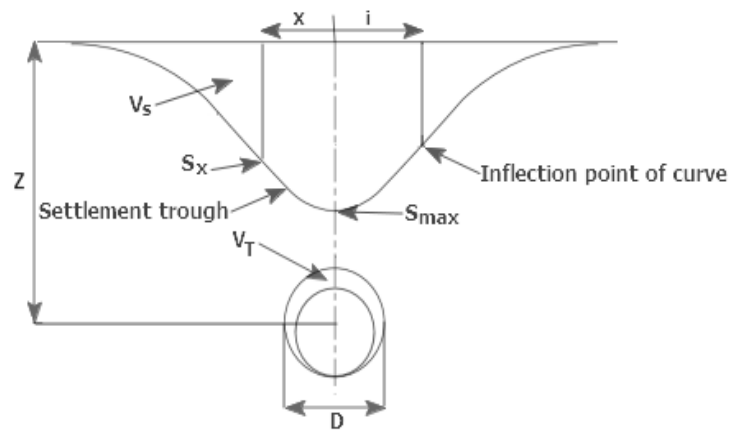


Fig. 2.2 Transverse surface settlement trough, Gaussian distribution (Agarwal and Sarkar, 2024)

$$S_x = S_{max} \exp\left(\frac{-x^2}{2i^2}\right) \quad (2.1)$$

$$S_{max} = 0.313V_L \frac{D^2}{i} \quad (2.2)$$

$$V_L = \frac{V_T}{\pi D^2/4} \quad (2.3)$$

$$i = kz \quad (2.4)$$

Where,  $k$  is the trough width parameter which varies from 0.4 to 0.7 for cohesive soils and 0.2 to 0.3 for cohesionless soils (O'Reilly and New, 1982) and varies from 0.2 to 0.5 for granular soils, from 0.4 to 0.6 for stiff clays and from 0.6 to 0.75 for soft clays (Mair and Taylor, 1997).  $z$  denotes the axial depth of the tunnel,  $S_{max}$  is the maximum vertical ground settlement under greenfield condition (Eqn. 2.2),  $V_T$  is the volume of ground loss around a tunnel cavity per unit length,  $V_L$  is volume of ground loss (Eqn. 2.3) around a tunnel cavity ( $V_T$ ) with respect to the tunnel volume per unit length and  $i$  is the distance between centreline of the tunnel and the inflection points (Eqn. 2.4).

Oteo and Moya (1979) proposed empirical correlations between field/laboratory test results and the geotechnical behaviour of soils during tunneling (Eqn. 2.5). These correlations served as practical guidelines for engineers working on underground projects in Madrid.

$$S_x = \psi \frac{\gamma D^2}{E_s} (0.85 - \nu) \exp\left(-\frac{x^2}{2i^2}\right) \quad (2.5)$$

Eqn. 2.5 represents ground surface settlement expressed as function of soil properties such as Poisson's ratio ( $\nu$ ), unit weight ( $\gamma$ ), stiffness ( $E_s$ ), and an empirical parameter ( $\psi$ ) derived from monitoring data.

O'Reilly and New (1982) provided an expression (Eqn. 2.6) for calculating the horizontal ground displacement ( $H_x$ ) resulting from tunnelling, which they correlated with the vertical ground settlement ( $S_x$ ) at that specific location (Fig. 2.3).

$$H_x = \frac{x}{z} S_x \quad (2.6)$$

Many researchers have observed that tunnelling-induced ground surface settlements generally follow a Gaussian distribution profile, as described by Peck (1969) (Aoyagi, 1995; Attewell and Farmer, 1974; Attewell, 1978; Camós, 2015; Dimmock and Mair, 2008; Grant and Taylor, 2000; Moh et al., 1996; Mroueh and Shahrour, 2003; Rebello et al., 2014). However, the Gaussian curve does not always precisely model ground settlements (Giardina et al. 2012; Hansmire and Cording, 1972; He et al., 2021; Murphy et al., 2010; ElMouchi et al., 2018; Sugiyama et al., 1999), especially for tunnels in sandy soils, and may provide a poor fit for such data

(Marshall et al., 2012). Consequently, many researchers have modified the Gaussian settlement trough equation to more accurately reflect true surface settlements.

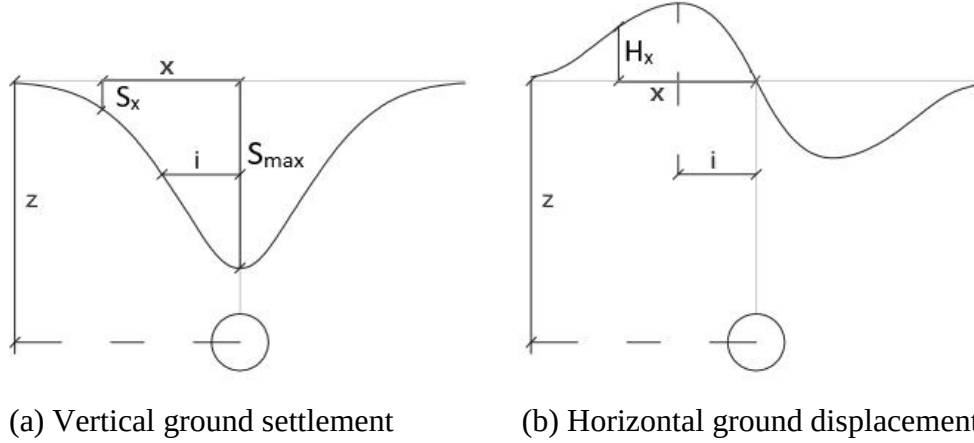


Fig. 2.3 Empirical-analytical curves for tunnelling-induced deformations in the transverse direction (Giardina, 2013)

Mair et al. (1993) proposed a formula (Eqn. 2.7, 2.8) to calculate the subsurface settlement ( $s$ ) at a depth  $z_0$  from the ground surface, in clay considering that the volume of the settlement trough caused by tunneling in clay remains constant along the depth and equal to the volume loss of soil around the tunnel (Fig. 2.4).

$$s = \frac{V_L}{\sqrt{2\pi}k(z-z_0)} \exp\left(-\left[\frac{x}{\sqrt{2\pi}k(z-z_0)}\right]^2\right) \quad (2.7)$$

$$k = \frac{0.175 + 0.325\left(1 - \frac{z_0}{z}\right)}{1 - \frac{z_0}{z}} \quad (2.8)$$

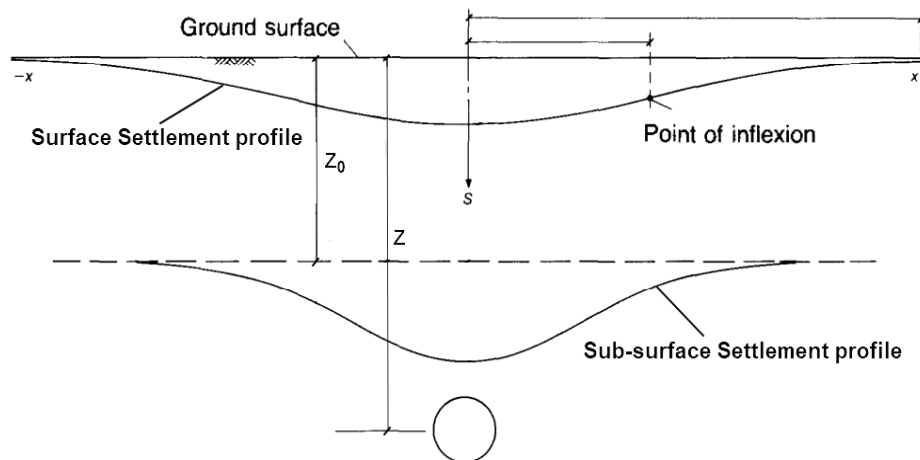


Fig. 2.4 Surface and Sub-surface settlement profiles (Mair et al.,1993)

Celestino et al. (2000) developed a three-parameter curve (Eqn. 2.9) presenting the tunneling-induced settlement trough consisted of three parameters:  $S_x$ ,  $a$  (length dimension), and  $b$  (dimensionless parameter).

$$S_x = \frac{S_{max}}{1 + \left(\frac{|x|}{a}\right)^b} \quad (2.9)$$

$$i = aB \quad (2.10)$$

$$B = \left(\frac{b-1}{b+1}\right)^{\frac{1}{b}} \quad (2.11)$$

Where,  $b = 1 \text{ to } 3$

$$\frac{a}{D} = 0.39(z/D) + 0.31 \quad (\text{For Stiff Clay}) \quad (2.12)$$

$$\frac{a}{D} = 0.46(z/D) + 0.37 \quad (\text{For Porous Clay}) \quad (2.13)$$

Jacobsz et al. (2004) proposed a modified settlement trough (Eqn. 2.14). In this modified equation,  $k'$  serves as the modified trough width parameter, and  $i'$  is the distance from the tunnel's center to the inflection points.

$$S_x = S_{max} \exp\left(-\frac{1}{3}\left(\frac{|x|}{i'}\right)^{1.5}\right) \quad (2.14)$$

$$i' = k'z \quad (2.15)$$

$$k' = 0.85K - 0.12 \quad (2.16)$$

Vorster et al. (2005) proposed a modified Gaussian curve to improve the fit to the observed soil settlement curve obtained from centrifuge experiments in soil. This equation (Eqn. 2.17) features a shape function parameter  $n$  that controls the profile width, and the distance to the inflection point,  $i$  is ensured by the parameter  $\alpha$ .

$$S_x = \frac{nS_{max}}{(n-1) + \exp\left[\alpha\left(\frac{x}{i}\right)^2\right]} \quad (2.17)$$

$$n = e^{\alpha \frac{2\alpha - 1}{2\alpha + 1}} + 1 \quad (2.18)$$

Where,  $n = 0.5, 1, 1.5$

Wang et al. (2016) proposed an empirical formula (Eqn. 2.19) to predict the subsurface settlement ( $s$ ) caused by tunneling in sand considering the variation of settlement trough volume along the depth. This formula was developed based on a lab-scale 1-g physical model test results.

$$s = \frac{V_L \left( \frac{0.5z_0}{z} + 0.5 \right)}{\sqrt{2\pi}k(z - z_0)} \exp \left( - \left[ \frac{x}{\sqrt{2\pi}k(z - z_0)} \right]^2 \right) \quad (2.19)$$

$$k = \frac{0.44 - \frac{0.28z_0}{z}}{1 - \frac{z_0}{z}} \quad (2.20)$$

### 2.2.2 Analytical Methods

Analytical methods offer simplified yet practical approaches for estimating the deformations resulting from tunnel excavation. Analytical solutions are derived from explicit theoretical formulations and usually based on simplified soil–structure interaction theories and assumptions regarding ground behaviour (Fu et al., 2016; Shinde et al., 2020; Whittle et al., 2001). While these methods are straightforward to apply and useful for preliminary assessments, they are limited in their ability to capture the complex nonlinear relationships between ground movements and the governing parameters. Sagaseta (1987) presented an analytical method which provides closed-form solutions to predict tunnelling-induced ground deformations in undrained conditions. Verruijt and Booker (1996) developed an approximate analytical solution for surface settlements caused by tunnel deformation in an elastic half-plane, which accounts for both uniform radial displacement (ground loss) and tunnel ovalization, offering insights into stress and displacement fields using elasticity theory. Loganathan and Poulos (1998) proposed an analytical solution by introducing an equivalent ground loss parameter based on the gap parameter to predict tunnelling-induced ground deformations under undrained conditions, showing reliable results particularly for tunnels in stiff clays. Bobet (2001) presented analytical solutions for shallow tunnels in saturated ground, accounting for the effects of seepage forces and pore water pressure. The study offers valuable insights into tunnel behaviour under coupled hydraulic and mechanical loading conditions. Vorster et al. (2005) presented an analytical method for calculating the maximum bending moment for pipelines that are rigidly joined or continuous and affected by ground movement resulting from tunnelling, which was found conservative as compared with centrifuge tests. Fu et al. (2016) developed an analytical method to predict ground movements caused by a non-uniformly deforming tunnel. Their approach captures complex deformation patterns more accurately than traditional uniform models. Tian et al. (2017) conducted a theoretical analysis to evaluate the impact of tunnel excavation on building strip foundations. Their study offers analytical expressions to assess foundation responses under various tunnel-induced deformation scenarios. Mei and Song (2021) derived analytical solutions for tunneling-induced settlements in

homogeneous structures using Winkler beam theory, modified Peck's formula, and Laplace transform, validating their efficiency through field and numerical data.

Sagaseta (1987) used the mirror source-sink method to model the soil as a linear elastic, homogeneous, isotropic, incompressible half-space. From that framework, he derived closed-form equations for both horizontal displacement ( $H_x$ ) and vertical settlement ( $S_x$ ) at the ground surface in the transverse direction (Eqn. 2.21, 2.22).

$$H_x = -\frac{V_L}{\pi} \left( \frac{x}{x^2+z^2} \right) \quad (2.21)$$

$$S_x = -\frac{V_L}{\pi} \left( \frac{z}{x^2+z^2} \right) \quad (2.22)$$

Verruijt and Booker (1996) presented a generalization of the closed-form method proposed by Sagaseta (1987), which is applicable to the soils with arbitrary values of Poisson's ratio ( $\nu$ ). This approach incorporates the effect of tunnel ovalization, a deformation mechanism previously examined by Wood (1975) for tunnels in an infinite elastic medium. Considering a tunnel within a semi-infinite elastic half-space, Verruijt and Booker (1996) derived expressions for ground surface settlement (Eqn. 2.23).

$$S_x = 4(1-\nu)\varepsilon r^2 \frac{z}{z^2+x^2} - 2\delta r^2 \frac{z(x^2-z^2)}{(z^2+x^2)^2} \quad (2.23)$$

Where  $r$  is the tunnel radius ( $D/2$ ),  $z$  is the depth of tunnel axis,  $\varepsilon$  is the equivalent undrained ground loss and  $\delta$  is shape ovalization factor.

Loganathan and Poulos (1998) developed analytical solutions to predict tunneling-induced ground movements in soft soils by introducing a modified equivalent undrained ground loss parameter, derived from the gap parameter ( $g$ ) of Lee et al. (1992) (Eqn. 2.24). This parameter accounts for nonlinear ground movements caused by the formation of an oval-shaped gap, modeled using an exponential function with appropriate boundary conditions. The modified parameter is then incorporated into the closed-form elastic solutions of Verruijt and Booker (1996), enabling prediction of both surface and subsurface settlements due to tunneling (Eqn. 2.25).

$$\varepsilon = \frac{4gr+g^2}{4r^2} \times 100\% \quad (2.24)$$

$$S_x = 4(1-\nu)\varepsilon r^2 \frac{z}{z^2+x^2} \exp \left[ -\frac{1.38x^2}{(z+r)^2} \right] \quad (2.25)$$

### 2.2.3 Experimental Methods

Experimental or physical modelling has been widely employed to study tunneling-induced ground deformations under controlled laboratory conditions. Centrifuge modelling and 1g small-scale physical tests allow researchers to simulate soil–structure interaction mechanisms and settlement development in a scaled environment (Wang, 2021). These models are particularly useful for visualizing ground movement patterns, validating theoretical assumptions, and providing benchmark data for numerical analyses. However, their accuracy is limited by scaling effects, equipment constraints, and the difficulty of fully replicating in-situ soil behaviour (Ahmed and Iskander, 2011).

### 2.2.4 Numerical Methods

Numerical modelling techniques, particularly the Finite Element Method (FEM) and the Finite Difference Method (FDM), have become powerful tools for analysing tunneling-induced ground deformations. These approaches enable detailed simulation of soil nonlinearity, anisotropy, groundwater effects, and complex construction sequences, which are often beyond the capability of analytical and empirical methods. Software packages such as PLAXIS, FLAC, and MIDAS GTS are widely used for simulating tunneling processes, settlement troughs, and building responses. Although numerical methods require extensive input data and computational effort, they provide high accuracy when validated with field measurements and are increasingly integrated with analytical frameworks to enhance reliability (Schiena et al., 2024). Two-dimensional (2D) methods are widely used for free-field tunneling simulations, while three-dimensional (3D) modelling provides a more accurate representation of induced stress distribution and capture ground deformation mechanisms (El Naggar et al., 2000).

Prior studies employing physical modelling and numerical techniques to assess ground deformations and building response caused by tunnelling are summarized in Table 2-1.

Table 2-1 Review of Prior Studies

| S. No.   | Reference                | Region                                                                                                                                                                              | Analysis Type/<br>Software used     | Validation Method             | Analysis Scenario |
|----------|--------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------|-------------------------------|-------------------|
| 1        | Loganathan et al. (2000) | Cambridge, England                                                                                                                                                                  | Experimental (Centrifuge modelling) | Empirical + Published Results | Tunnel + Pile     |
| Findings |                          | Pile deformations, bending moments and axial forces due to tunneling depend significantly on the tunnel–pile spacing and position relative to the tunnel-induced settlement trough. |                                     |                               |                   |

continued on page 15

Table 2-1 (continued)

| S. No.   | Reference                    | Region                                                                                                                                                                                                                                            | Analysis Type/<br>Software used       | Validation Method     | Analysis Scenario  |
|----------|------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------|-----------------------|--------------------|
| 2        | Burd et al. (2000)           | University of Oxford, (UK)                                                                                                                                                                                                                        | 3D FEM (OXFEM)                        | Field Data            | Tunnel + Structure |
| Findings |                              | Soil–structure interaction reduces predicted settlement damage, especially in sagging modes, but for hogging deformations, a conventional uncoupled approach may still provide reliable results.                                                  |                                       |                       |                    |
| 3        | Jenck and Dias (2003)        | Lisbon, Portugal                                                                                                                                                                                                                                  | 3D FEM (FLAC)                         | Field Data            | Tunnel + Structure |
| Findings |                              | The more affected direction of the building during tunneling is the transverse direction.                                                                                                                                                         |                                       |                       |                    |
| 4        | Jacobz et al. (2004)         | Cambridge, England                                                                                                                                                                                                                                | Experimental (Centrifuge modelling)   | Empirical             | Tunnel + Pile      |
| Findings |                              | Tunneling can cause measurable displacements and bending moments in piles, especially when tunneling occurs close to the pile toe.                                                                                                                |                                       |                       |                    |
| 5        | Ghahremannejad et al. (2006) | Sydney, Australia                                                                                                                                                                                                                                 | Experimental                          | Empirical (Peck 1969) | Tunnel + Pile      |
| Findings |                              | 1g laboratory model tests demonstrated that tunneling-induced ground loss in dense sand increases surface settlements and bending moments in adjacent pile foundations.                                                                           |                                       |                       |                    |
| 6        | Möller and Vermeer (2008)    | Stuttgart, Germany & Rotterdam, Netherlands                                                                                                                                                                                                       | 2D FEM (PLAXIS)                       | Field Data            | Greenfield         |
| Findings |                              | Case studies on the Steinhaldenfeld and Heinenoord Tunnels showed that among various numerical installation techniques, the grout pressure method most accurately predicted settlements and lining forces in shield tunneling.                    |                                       |                       |                    |
| 7        | Ahmed and Iskander (2011)    | New York University, US                                                                                                                                                                                                                           | Experimental (Transparent soil model) | -                     | Greenfield         |
| Findings |                              | Settlement trough width parameter is independent of volume loss but proportional to tunnel depth, with subsurface settlements fitting a normal probability curve and horizontal displacements governed by trough width parameter and volume loss. |                                       |                       |                    |
| 8        | Marshall et al. (2012)       | Cambridge, England                                                                                                                                                                                                                                | Experimental (Centrifuge modelling)   | -                     | Greenfield         |
| Findings |                              | Gaussian curves poorly fit tunnel-induced settlements in sands, which are better represented using modified Gaussian or yield-density curves and significantly affected by tunnel geometry and volume loss.                                       |                                       |                       |                    |
| 9        | Mathew and Lehane (2013)     | Perth, Western Australia                                                                                                                                                                                                                          | 2D & 3D FEM (PLAXIS)                  | Field Data            | Greenfield         |
| Findings |                              | Numerical back-analyses using PLAXIS often overestimate settlement trough widths, with prediction accuracy strongly dependent on soil stratigraphy, model selection, and stiffness assumptions.                                                   |                                       |                       |                    |

continued on page 16

Table 2-1 (continued)

| S. No.   | Reference                | Region                                                                                                                                                                                                                                                                                        | Analysis Type/<br>Software used     | Validation Method                  | Analysis Scenario  |
|----------|--------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------|------------------------------------|--------------------|
| 10       | Farrell et al. (2014)    | Bologna, Italy                                                                                                                                                                                                                                                                                | Experimental (Centrifuge modelling) | Field Data                         | Tunnel + Structure |
| Findings |                          | Building response to tunneling is strongly governed by soil–structure interaction, with building and soil stiffness significantly modifying settlement distortions and risk assessments based solely on greenfield assumptions may be overly conservative.                                    |                                     |                                    |                    |
| 11       | Mathew and Lehané (2014) | Perth, Western Australia                                                                                                                                                                                                                                                                      | 2D & 3D FEM (PLAXIS)                | Field Data                         | Tunnel + Structure |
| Findings |                          | Building stiffness significantly influences tunneling-induced settlement profiles, with stiff structures tilting uniformly and flexible ones showing sagging and hogging, while finite element analyses can qualitatively capture these behaviors and the effects of ground improvement.      |                                     |                                    |                    |
| 12       | Rebello et al. (2014)    | Bangalore, India                                                                                                                                                                                                                                                                              | 3D Discrete element code            | Field Data                         | Tunnel + Structure |
| Findings |                          | The study found that tunneling-induced displacements in granular soils decrease with increasing density, while the presence of buildings further reduced displacements compared to greenfield conditions.                                                                                     |                                     |                                    |                    |
| 13       | Amorosi et al. (2014)    | Rome, Italy                                                                                                                                                                                                                                                                                   | 2D FEM (ABAQUS)                     | Empirical (Peck 1969) + Field data | Tunnel + Structure |
| Findings |                          | Significant differences are observed between uncoupled analysis, where free-field displacements are directly applied at the foundation level, and interaction-based analysis, which shows reduced amount of tensile strain.                                                                   |                                     |                                    |                    |
| 14       | Comodromos et al. (2014) | Thessaloniki, Greece                                                                                                                                                                                                                                                                          | 3D FEM (FLAC)                       | Empirical (Peck 1969) + Field data | Tunnel + Structure |
| Findings |                          | The study concludes that tail gap pressures notably affect the ground movement and presence of stiff building foundations near the trough width, can widen settlement profiles and slightly increase volume loss, though the effects are highly site-specific and not directly generalizable. |                                     |                                    |                    |
| 15       | Camós et al. (2014)      | Barcelona, Spain                                                                                                                                                                                                                                                                              | 2D FEM (DIANA)                      | Empirical (Peck 1969) + Field data | Tunnel + Structure |
| Findings |                          | Equivalent beam method provides reliable predictions of tunneling-induced building damage, closely matching field observations, while non-linear FE models capture the influence of soil–structure interaction.                                                                               |                                     |                                    |                    |
| 16       | Zhang et al. (2014)      | Wuhan, China                                                                                                                                                                                                                                                                                  | 3D FEM (ABAQUS)                     | Field Data                         | Tunnel + Structure |
| Findings |                          | FEM-based safety management approach effectively assesses and mitigates risks to nearby buildings during tunnel construction, ensuring safer and more reliable project execution.                                                                                                             |                                     |                                    |                    |

continued on page 17

Table 2-1 (continued)

| S. No.   | Reference               | Region                                                                                                                                                                                                                                                                                 | Analysis Type/<br>Software used   | Validation Method | Analysis Scenario  |
|----------|-------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------|-------------------|--------------------|
| 17       | Zheng et al. (2015)     | Tianjin, China                                                                                                                                                                                                                                                                         | 3D FEM (ABAQUS)                   | Field Data        | Greenfield         |
| Findings |                         | A practical framework for optimizing earth pressure balance machine performance is established in soft ground by defining correlations between mechanical driving parameters and shield advance speed.                                                                                 |                                   |                   |                    |
| 18       | El-Nahhas et al. (2015) | Cairo, Egypt                                                                                                                                                                                                                                                                           | 3D FEM (PLAXIS)                   | Field Data        | Greenfield         |
| Findings |                         | Advanced 3D numerical modeling can reliably predict tunneling-induced ground deformations for bentonite slurry tunneling in soft ground.                                                                                                                                               |                                   |                   |                    |
| 19       | Zhao et al. (2015)      | Western Scheldt, Netherlands                                                                                                                                                                                                                                                           | 2D & 3D FEM (PLAXIS)              | Field Data        | Greenfield         |
| Findings |                         | 3D numerical modeling with optimized parameters via global sensitivity and back analysis provides reliable predictions of tunneling-induced surface settlements, outperforming 2D models.                                                                                              |                                   |                   |                    |
| 20       | Giardina et al. (2015)  | Cambridge, UK                                                                                                                                                                                                                                                                          | 2D FEM                            | Centrifuge tests  | Tunnel + Structure |
| Findings |                         | The study shows that building response to tunneling depends on both relative stiffness and building weight, highlighting the need to include weight in damage assessment procedures.                                                                                                   |                                   |                   |                    |
| 21       | Son (2015)              | South Korea                                                                                                                                                                                                                                                                            | 2D discrete element method (UDEC) | Field Data        | Tunnel + Structure |
| Findings |                         | Tunneling-induced building damage strongly depends on structure type, ground loss, and tunnel depth-to-diameter ratio, with brick-bearing structures being more vulnerable than brick-infilled frames.                                                                                 |                                   |                   |                    |
| 22       | Son (2016)              | South Korea                                                                                                                                                                                                                                                                            | 2D discrete element method (UDEC) | Field Data        | Tunnel + Structure |
| Findings |                         | Structures in clay soils can tolerate larger ground loss with less distortion than in sandy soils, though building response still strongly depends on tunnel, ground, structure, and construction conditions.                                                                          |                                   |                   |                    |
| 23       | Golpasand et al. (2016) | Tehran, Iran                                                                                                                                                                                                                                                                           | 3D FEM (FLAC)                     | Field Data        | Greenfield         |
| Findings |                         | The study establishes a reliable volume loss of value 0.5 to 0.1% for the Abuzar tunnel excavation, validated by numerical modeling and field measurements, which is lower than semi-empirical estimates.                                                                              |                                   |                   |                    |
| 24       | Shahin et al. (2016)    | Nagoya, Japan                                                                                                                                                                                                                                                                          | 2D Non- linear FEM (FEMtij-2D)    | Model Tests       | Tunnel + Structure |
| Findings |                         | Tunneling significantly influences earth pressure distribution, ground deformation, and structural response, particularly in closely spaced or building-loaded conditions, highlighting the importance of considering tunnel interaction effects for safe and efficient tunnel design. |                                   |                   |                    |

continued on page 18

Table 2-1 (continued)

| S. No.   | Reference                | Region                                                                                                                                                                                                                                                             | Analysis Type/<br>Software used    | Validation Method                   | Analysis Scenario  |
|----------|--------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------|-------------------------------------|--------------------|
| 25       | Cho et al. (2017)        | Seoul, South Korea                                                                                                                                                                                                                                                 | 3D FEM (MIDAS GTS)                 | Field Data + analytical + empirical | Greenfield         |
| Findings |                          | Lining stresses and ground settlements during tunneling are strongly influenced by jack force, soil elastic modulus, and lining thickness.                                                                                                                         |                                    |                                     |                    |
| 26       | Lei-Kilany et al. (2017) | Cairo, Egypt                                                                                                                                                                                                                                                       | 2D & 3D FEM (PLAXIS)               | Field Data                          | Greenfield         |
| Findings |                          | FEM can effectively simulate tunneling and lining phases, enabling cost reduction and better decision-making for future tunnel projects.                                                                                                                           |                                    |                                     |                    |
| 27       | Boldini et al. (2018)    | Milan, Italy                                                                                                                                                                                                                                                       | 2D FEM (ABAQUS)                    | -                                   | Tunnel + Structure |
| Findings |                          | Frame buildings' response to tunneling depends on eccentricity and number of storeys, with sagging deflection decreasing and hogging deflection increasing as building height increase.                                                                            |                                    |                                     |                    |
| 28       | Fu et al. (2018)         | China                                                                                                                                                                                                                                                              | 2D FEM (ABAQUS)                    | Empirical (Peck 1969)               | Tunnel + Structure |
| Findings |                          | Framed building performance under tunneling-induced ground movement is strongly influenced by soil stiffness, infill configuration, and interface properties.                                                                                                      |                                    |                                     |                    |
| 29       | Al-Omari et al. (2019)   | Iraq                                                                                                                                                                                                                                                               | 3D FEM (PLAXIS)                    | -                                   | Tunnel + Pile      |
| Findings |                          | Tunnel advancement near piled foundations can cause pile head settlements up to three times greenfield values, most critical within two times diameter of tunnel horizontally and vertically of the pile.                                                          |                                    |                                     |                    |
| 30       | Malhotra et al. (2019)   | India                                                                                                                                                                                                                                                              | 2D FEM (PLAXIS)                    | Experiment                          | Tunnel + Pile      |
| Findings |                          | Presence of a nearby utility tunnel within 3–5 times pile diameters distance can reduce pile capacity by 5–20%, depending on soil and structural conditions.                                                                                                       |                                    |                                     |                    |
| 31       | Shiau and Sams (2019)    | Queens-land, Australia                                                                                                                                                                                                                                             | 2D finite difference method (FLAC) | Empirical (Peck 1969)               | Greenfield         |
| Findings |                          | Surface settlement trough width increases with volume loss and is primarily influenced by soil strength and depth-to-diameter ratio, while Young's modulus mainly affects the volume loss magnitude.                                                               |                                    |                                     |                    |
| 32       | Khabbaz et al. (2019)    | Sydney, Australia                                                                                                                                                                                                                                                  | 2D FEM (PLAXIS)                    | Field Data                          | Tunnel + Pile      |
| Findings |                          | Tunnel construction significantly increases the bending moments as well as the flexural and axial loads in piles, and when the bending moments induced in an edge pile exceed its original design capacity, the pile may fail due to the nearby tunnel excavation. |                                    |                                     |                    |

continued on page 19

Table 2-1 (continued)

| S. No.   | Reference                           | Region                                                                                                                                                                                                                                                | Analysis Type/<br>Software used     | Validation Method                  | Analysis Scenario  |
|----------|-------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------|------------------------------------|--------------------|
| 33       | Miliziano and Lillis (2019)         | Rome, Italy                                                                                                                                                                                                                                           | 3D FEM (PLAXIS)                     | Field Data                         | Tunnel + Structure |
| Findings |                                     | 3D numerical modeling accurately predicts tunneling settlements, with TBM conicity and cutterhead overcut as key factors.                                                                                                                             |                                     |                                    |                    |
| 34       | Lai et al. (2019)                   | China                                                                                                                                                                                                                                                 | Experimental (Centrifuge modelling) | Empirical                          | Greenfield         |
| Findings |                                     | A new loess-specific method, developed from centrifuge test data, accurately predicts tunnel-induced settlements, showing good agreement with both model and field measurements.                                                                      |                                     |                                    |                    |
| 35       | Zhao et al. (2019)                  | Germany                                                                                                                                                                                                                                               | 2D FEM (PLAXIS)                     | Empirical (Peck 1969)              | Tunnel + Structure |
| Findings |                                     | Numerical methods can effectively predict tunneling-induced surface settlements and deformations, with soil properties, building stiffness, and position relative to the tunnel significantly influencing settlement profiles and structure response. |                                     |                                    |                    |
| 36       | Sarkar et al. (2021)                | Delhi, India                                                                                                                                                                                                                                          | 2D FEM (MIDAS GTX NX, Phase 2)      | Field Data                         | Greenfield         |
| Findings |                                     | Ground settlement decreases with increasing tunnel depth, soil strength and stiffness, with FEM giving reliable predictions.                                                                                                                          |                                     |                                    |                    |
| 37       | Naqvi et al. (2021)                 | Delhi, India                                                                                                                                                                                                                                          | 2D FEM (OptumG2)                    | Empirical (Peck 1969)              | Tunnel + Structure |
| Findings |                                     | Tunnel crown experienced the highest displacement, making it the most critical point for tunnel lining design. Increase in tunnel depth significantly reduces ground displacements.                                                                   |                                     |                                    |                    |
| 38       | Kanagaraju and Krishnamurthy (2020) | Chennai, India                                                                                                                                                                                                                                        | 3D FEM (PLAXIS)                     | Empirical (Peck 1969)              | Greenfield         |
| Findings |                                     | FEM parametric studies define influence zones based on tunnel and soil parameters, enabling prediction of vulnerable zones to guide precautionary measures before tunneling.                                                                          |                                     |                                    |                    |
| 39       | Rezaei and adli (2020)              | Tabriz, Iran                                                                                                                                                                                                                                          | 3D FEM (ABAQUS)                     | Empirical (Peck 1969) + Field data | Greenfield         |
| Findings |                                     | Transverse tunnel-induced settlements can be best predicted using Gaussian methods, validating FE-based settlement analyses while highlighting potential overestimation by MC models.                                                                 |                                     |                                    |                    |
| 40       | Wang and Wen (2021)                 | Yiyang, China                                                                                                                                                                                                                                         | 3D FEM (ABAQUS)                     | Empirical (Peck 1969)              | Tunnel + Pile      |
| Findings |                                     | Shield tunneling near viaduct pile foundations primarily causes transverse (sideways) deformation with S-shaped bending and moderate internal forces near the tunnel axis.                                                                            |                                     |                                    |                    |

continued on page 20

Table 2-1 (continued)

| S. No.   | Reference              | Region                                                                                                                                                           | Analysis Type/<br>Software used   | Validation Method              | Analysis Scenario  |
|----------|------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------|--------------------------------|--------------------|
| 41       | Athar et al. (2022)    | Delhi, India                                                                                                                                                     | 2D FEM (OptumG2)                  | Experiment                     | Greenfield         |
| Findings |                        | Elliptical linings minimize ground settlements, while square linings should be avoided in sensitive areas due to higher settlement.                              |                                   |                                |                    |
| 42       | Zhang et al. (2024)    | Hohhot, China                                                                                                                                                    | 3D FEM (PLAXIS)                   | Field Data                     | Tunnel + Structure |
| Findings |                        | Increased bond strength and grouting pressure reduces strain and deformations in building.                                                                       |                                   |                                |                    |
| 43       | Dutta and Ghosh (2025) | Kolkata, India                                                                                                                                                   | Finite difference method (MATLAB) | Field data + Published Results | Tunnel + Pile      |
| Findings |                        | Incorporating non-linear soil behavior using Pasternak's model provides a more accurate prediction of pile deflection due to tunneling-induced ground movements. |                                   |                                |                    |

Out of the 43 studies reviewed, only seven are directly related to Indian tunneling conditions, with most investigations focusing on greenfield settlements, tunnel–pile interaction, or isolated tunnel–structure response. Limited studies have addressed an integrated Building Risk Assessment (BRA) framework under Indian conditions that simultaneously considers tunnel geometry, volume loss, soil characteristics, and structural vulnerability. Furthermore, the combined evaluation of limiting tensile strain, and seismic influence on different building typologies remains limited in existing Indian literature. Therefore, the present study attempts to bridge this gap through a comprehensive BRA framework for tunneling-induced structural risk assessment under Indian urban conditions.

### 2.3 Building Risk Assessment (BRA)

The assessment of risk of damage for buildings potentially affected by tunnelling-induced deformations involves two primary activities: (a) the Building Condition Survey (BCS), which evaluates the real conditions of buildings before, during, and after tunnel construction, and (b) the Building Risk Assessment (BRA), which estimates the potentially expected damages based on predicted deformations and the inherent vulnerability of the structures (Guglielmetti et al., 2008). BRA is used to analyse the risk involved with people who are located inside buildings on tunnelling site. Limiting Tensile Strain Method (LTSM) is a widely used empirical-analytical approach for assessing and predicting induced damage to buildings, due to ground movements caused by excavations such as tunneling (Netzel, 2009). First formalized by Burland and Wroth (1974) and subsequently refined by later researchers, this method remains a fundamental tool in geotechnical engineering for assessing damage risk in urban environments. LTSM idealizes a building as a weightless, isotropic, elastic beam subjected to tunneling-induced ground deformations. Its fundamental

assumption is that the structure fully follows the settlement profile of the ground, meaning all differential movements are transferred to the building—while neglecting soil–structure interaction, which is a conservative simplification. The maximum tensile strains in the beam, representing the building façade or wall, are then analytically calculated using beam theory, with Timoshenko (1957) beam equations to incorporate both bending and shear effects. Boscardin and Cording (1989) extended this theory to include horizontal strains, caused by lateral ground movements induced by adjacent excavation and tunneling activities. The total calculated strains are then compared to limiting tensile strain thresholds, which are empirically linked to different damage categories (e.g., negligible, very slight, slight, moderate, severe, very severe), characterized by the expected crack width and the ease of repair (Boone, 1996; Torp-Petersen and Black, 2001). These thresholds originate from empirical observations and are intended to predict not just the presence of damage, but also its functional and aesthetic significance (Clarke and Laefer, 2014). For typical buildings, the method distinguishes between zones of sagging and hogging in the settlement profile, each with different strain distributions and damage sensitivity (Fig. 2.5).

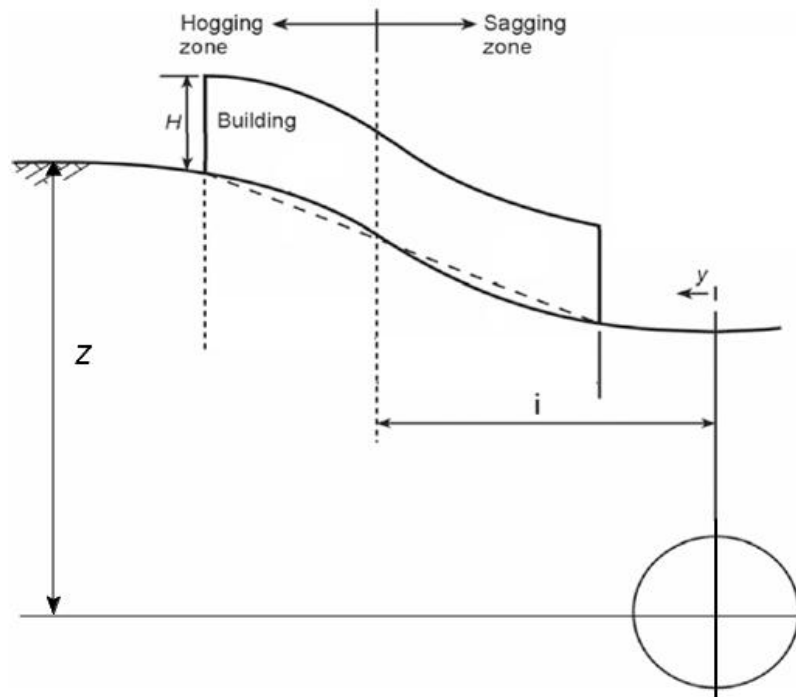


Fig. 2.5 Deformation of building above tunnelling (Harris and Franzius, 2005)

Boone (1996) developed a detailed method to assess building damage from differential ground movements, considering structural geometry, material strain limits, and strain superposition. Load-bearing walls are modelled as beams, while damage in frames and infill walls is linked to column movements and deformations of surrounding members. Potts and Addenbrooke (1997) proposed a modification factor approach, modelling buildings as elastic beams to develop design charts linking building stiffness, geometry, and location with tunneling-induced deformations,

particularly for masonry buildings. Netzel (2005) proposed adjustments to the shear form factor in Timoshenko beam equations to reduce errors in strain estimation by LTSM. Finno et al. (2005) proposed a new simplified laminate model, which accounts for both bending and shear deformations by assuming stiff floor systems that restrain bending while walls (load bearing or infill) resist shear and derived closed-form solutions to estimate deflections, offering a balance between overly simplistic empirical methods and highly complex numerical models. Giardina et al. (2013) performed parametric and numerical studies validating and refining the limiting strain values for wider structural types across various soil conditions. Fok et al. (2012) proposed a refined framework for assessing excavation-induced building damage by integrating desk studies on excavation influence zones and the integration of existing structural conditions from visual inspections with LTSM and provided special considerations for reinforced concrete frames, piled and mixed foundation systems, and structurally weak or heritage buildings. Clarke and Laefer (2014) proposed a vulnerability-based risk assessment method for buildings adjacent to tunneling, integrating damage predictions with cultural and structural criteria, while also accounting for initial damage and existing condition to overcome the limitations of the pure LTSM approach. Goh and Mair (2014) refined modification factors proposed by Potts and Addenbrooke (1997) for framed buildings by incorporating additional building features (weight, soil–foundation interface, tunnel depth, and longitudinal dimensions). Dalgic et al. (2017) reformulated the Burland and Wroth’s (1974) fictitious deep beam equations into a transfer function between tensile strains and effecting factors to improve the applicability of this method.

Thus, the Limiting Tensile Strain Method is a practical and useful tool for assessing structural responses from ground movements. While it provides a rational, empirically grounded framework for building damage assessment, its accuracy is strongly influenced by the correctness of its assumptions (especially regarding soil-structure interaction and structural idealization). Recent research supports its continued use for preliminary risk screening, provided its limitations are understood and supplemented by advanced modelling and case-specific studies where higher precision is required (Camós, 2015; Giardina et al., 2013; Dalgic et al., 2017).

## 2.4 Research Gaps

A critical review of the existing literature highlights several limitations in the prediction of tunneling-induced ground deformations and their impact on adjacent structures. While significant progress has been made through analytical, empirical, experimental, and numerical approaches, certain aspects remain underexplored or inadequately addressed. Based on this review, the following research gaps have been identified:

1. Although extensive research has been conducted globally using analytical, empirical, physical modelling, and numerical approaches to assess the effects of tunneling on surrounding structures, very few studies have been carried out in India, and those that exist are largely limited to greenfield conditions (where

there are no existing structures). Consequently, the combined effect of tunneling and structural response in densely built urban areas like Delhi, where tunneling often occurs in close proximity to existing infrastructure remains insufficiently addressed.

2. While studies conducted in India employ numerical tools such as PLAXIS to simulate ground deformations due to tunneling, there is a lack of studies that transfer these deformations to structural foundations to evaluate potential damage. This disconnect between numerical modelling and structural response assessment limits the practical application of existing research.
3. Additionally, methods such as the Limiting Tensile Strain Method (LTSM), which offer a rational strain-based approach for damage prediction, have rarely been applied in conjunction with numerical tunneling outputs in Indian contexts. Integrating LTSM with numerical modelling results for site-specific tunneling assessments remains an unexplored area with strong potential for improving damage prediction in urban infrastructure.

## CHAPTER 3

### RESEARCH METHODOLOGY

To minimize the potential risk-levels associated with the adjacent buildings due to tunneling-induced ground movements, it is necessary to conduct a specific Building Risk Assessment (BRA). The present study introduces a novel hybrid approach for risk assessment of existing structures adjacent to proposed tunnel construction, by integrating the limiting tensile strain method with advanced numerical simulations using PLAXIS 2D (Agarwal and Sarkar, 2025). To provide a clear overview of the adopted approach, the overall research workflow is illustrated in the flow chart shown in Fig. 3.1.

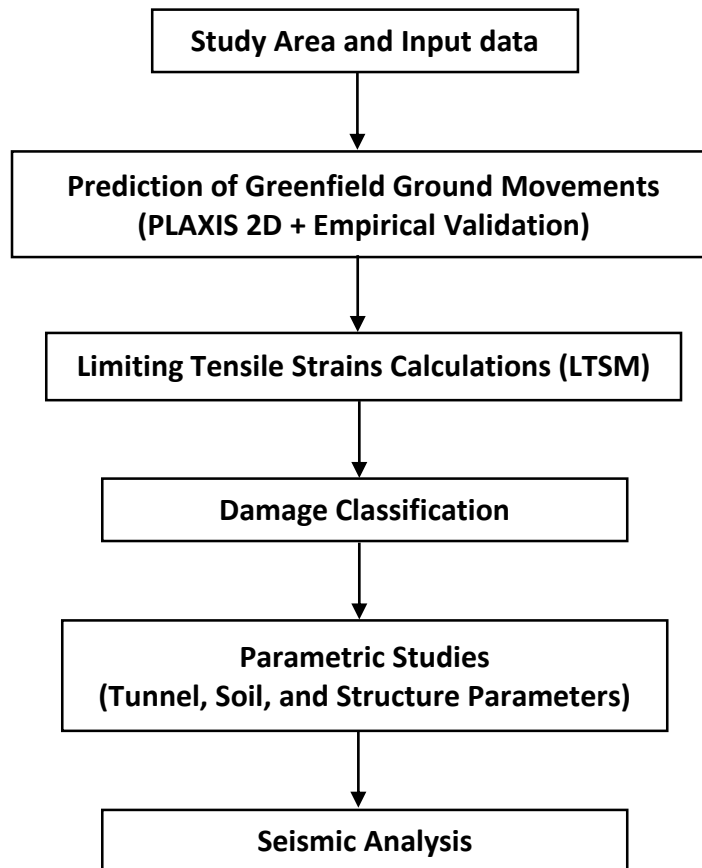


Fig. 3.1 Flow Chart- Approach and Methodology

The BRA of adjacent structures, according to the above mentioned hybrid method, is conducted using a four-step approach (Dalgic et al., 2017; Fok et al., 2012; Netzel, 2009), Step 1: Prediction of greenfield ground movements due to tunneling, Step 2: Projection of greenfield ground movements on the building's foundation and determination of building deformation measures, Step 3: Estimation of induced

building strains using limiting tensile strain method (LTSM), and Step 4: Classification of building damage related to the calculated strain levels. The details of these steps are described in this chapter.

### 3.1 Prediction of Greenfield Ground Movements due to Tunneling

Tunneling operations inevitably induce vertical and horizontal ground movements, within the zone of influence. When the ground surface is free of buildings, the resulting settlement ground displacement curves referred to as the free-field or greenfield ground displacement profiles. Ground settlement profile transverse to the tunnel alignment assumes a concave trough shape, where the downward curvature is termed as sagging and the upward curvature is termed as hogging (Fig. 3.2). Along the longitudinal direction of the tunnel line, advancing settlements may also occur as excavation progresses, accompanied by significant horizontal ground movements in both directions. However, the building is more affected in the transverse direction during tunneling and transverse loads retain significant values in the final state, whereas longitudinal loads return to their initial values (Jenck and Dias, 2003). Several empirical and analytical approaches as described in Chapter 2, have been widely used to estimate greenfield ground displacements profiles.

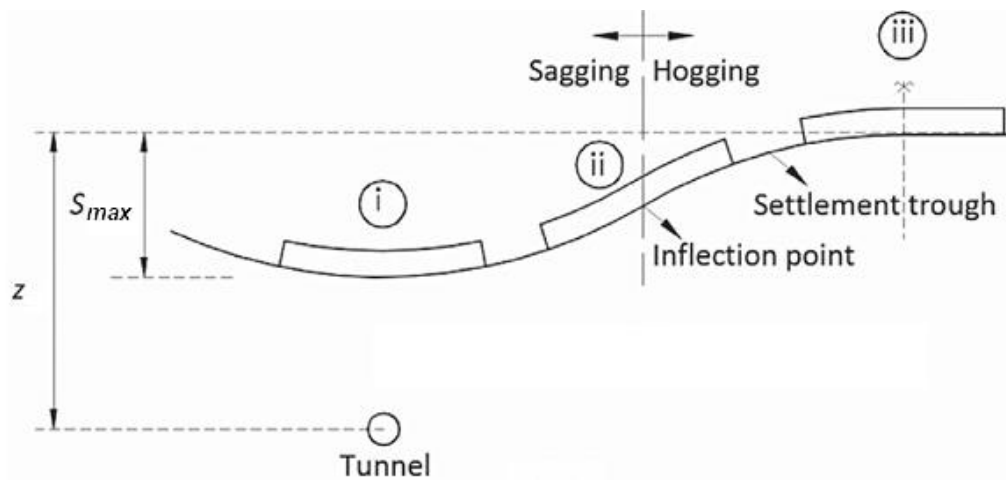


Fig. 3.2 Different positions of buildings in the transverse settlement trough: (i) sagging, (ii) at an inflection point and (iii) hogging

Although tunneling is inherently a three-dimensional (3D) process, two-dimensional (2D) analyses are widely accepted as a practical and reliable tool for predicting greenfield ground movements, particularly transverse settlement troughs. Advanced 3D numerical modelling can capture complex soil–structure interactions and non-linear behaviour with greater detail; however, such analyses are often prohibitively costly, requiring extensive data preparation, parameter calibration, and computational time. Moreover, 3D models are more sensitive to mesh dependency, interface element properties, excavation sequences, and boundary conditions, which may introduce significant uncertainties (Basile, 2014). For preliminary design and

routine engineering applications—where multiple tunnel alignments, depths, and diameters must be assessed—the 2D approach offers an efficient and sufficiently accurate alternative, provided that appropriate soil models and procedures are adopted (Yahya and Abdullah, 2014). By assuming tunnels of considerable length with relatively uniform geometry, 2D simulations can replicate settlement profiles with reasonable accuracy, thereby serving as a robust design tool (Shiau and Sams, 2019). In this context, 3D analyses are better reserved for validation of final designs or benchmarking in major projects, while 2D analyses remain the practical choice for most design stages, effectively bridging the gap between research insights and engineering practice.

### 3.2 Determination of building deformation measures

Building deformation measures are metrics employed to quantify structural alterations in buildings, particularly those influenced by tunneling-induced ground movements. These measures capture observable geometrical distortions in buildings and serve to establish correlations between these distortions and resulting damages. Commonly utilized building deformation measures include differential settlement ( $\delta S_v$ ), angular distortion ( $\beta$ ), deflection ratio ( $\Delta/L$ ), and horizontal strain ( $\epsilon_h$ ). These metrics, as highlighted by Skempton and Macdonald (1956), Polshin and Tokar (1957), Burland et al. (1977), and Dalgic et al. (2017) are pivotal in assessing the impact of tunneling-induced ground movements on structures.

#### 3.2.1 Differential Settlement ( $\delta S_v$ )

Differential Settlement ( $\delta S_v$ ) specifically addresses the displacement of points A and B on a building perpendicular to the tunneling line, resulting in strain within the structure. The calculation of  $\delta S_v$  between points A and B involves determining the vertical movement of these points ( $S_{v,A}$  and  $S_{v,B}$ ) (Fig. 3.3). Uniform settlements (where  $S_{v,A} = S_{v,B}$ ) can also lead to significant serviceability issues, affecting building access and utility line connections. Thus, evaluating  $\delta S_v$  is crucial for understanding the strain within buildings caused by tunneling-induced ground movements. The differential settlement  $\delta S_v$  between points A and B can be calculated using Eqn. 3.1.

$$\delta S_v = S_{v,A} - S_{v,B} \quad (3.1)$$

#### 3.2.2 Angular distortion ( $\beta$ )

The concept of angular distortion as a measure of damage due to settlement was initially introduced by Skempton and Macdonald (1956). They defined angular distortion as the ratio between the differential settlement of two points on a structure

and the distance between these points. Specifically, for points A and B, angular distortion ( $\beta$ ) can be expressed as given in Eqn. 3.2.

$$\beta = \frac{\delta S_v}{L} \quad (3.2)$$

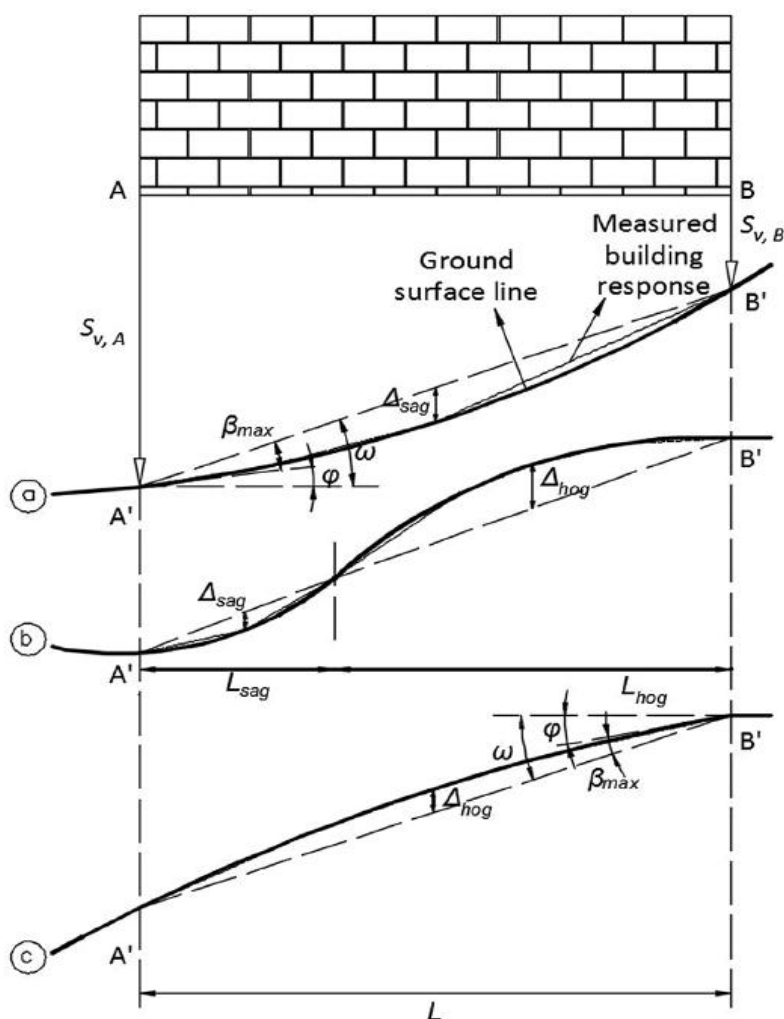


Fig. 3.3 Deformation measures for a building conforming to a ground settlement profile in (a) the sagging zone; (b) both the sagging and hogging zones; and (c) the hogging zone (Dalgic et al., 2017)

Angular distortion,  $\beta$  indicates whether deformation leads to either a 'sag' (positive) or 'hog' (negative) effect, resulting in upward or downward concavity, respectively. This metric proves particularly valuable for predicting crack widths in buildings, especially in areas where movements occur at existing cracks or lines of inherent weakness. Boscardin (1980), redefined the definition of  $\beta$  by excluding the contribution of rigid building rotation ( $\omega$ ), sometimes referred to as tilt. This adjustment was made because  $\omega$  does not contribute to internal straining within the

structure. In contemporary literature,  $\beta$  is calculated using an absolute-value expression as modified by Boscardin (1980) given in Eqn. 3.3. The calculation of  $\beta$  requires another geometrical quantity, the slope of the ground settlement profile ( $\varphi$ ). It is important to consider the maximum value of  $\beta$  ( $\beta_{max}$ ). Fig. 3.3 (a)–(c) schematically illustrate  $\beta_{max}$ ,  $\omega$ , and  $\varphi$  on deformed building foundation lines.

$$\beta = |\omega - \varphi| \quad (3.3)$$

### 3.2.3 Relative deflection ( $\Delta$ ) and deflection ratio ( $\Delta/L$ )

Polshin and Tokar (1957) introduced the deflection ratio ( $\Delta/L$ ), which stands as another significant metric in the literature for assessing damage. In this context,  $\Delta$  signifies the maximum vertical differential distance between the deflected foundation line and the straight-line connecting points A and B as depicted in Fig. 3.3 (a)–(c).  $\Delta/L$  is then calculated by dividing  $\Delta$  by the corresponding span length, denoted as  $L$ . When a building spans the inflection point,  $\Delta/L$  can be calculated separately for both the sagging and hogging sections of the structure. This approach allows for the independent consideration of each part, as noted by Mair et al. (1996). For such buildings, the deflection ratio is represented as  $\Delta_{sag}/L_{sag}$  for sagging and  $\Delta_{hog}/L_{hog}$  for hogging. Here,  $L_{sag}$  and  $L_{hog}$  represent the lengths of the sagging and hogging sections of the building, respectively.

### 3.2.4 Horizontal Stain ( $\varepsilon_h$ )

Tunneling-induced ground movements involve considerable horizontal displacement components. Boscardin and Cording (1989) reported that, horizontal strains transferred from the ground to the structure can remarkably decrease the settlement tolerances of buildings together with bending and diagonal tensile strains. Since the fictitious deep beam approach by Burland and Wroth (1974) does not include the effect of horizontal strain, Boscardin and Cording (1989) considered this effect by combining the average horizontal strain ( $\varepsilon_h$ ) with maximum bending tensile strain ( $\varepsilon_{bmax}$ ) and maximum diagonal tensile strain ( $\varepsilon_{dmax}$ ) values.

Although it is assumed that horizontal strain distributes evenly throughout the height of the structure, in reality, it tends to be significantly higher in the lower sections, especially those close to the foundation and in the upper portions of the building, lateral strains may reduce due to the stiffness and restraint provided by the upper floors (Cording et al., 2010; Son and Cording, 2005). If  $S_{h,A}$  and  $S_{h,B}$  represent the horizontal displacements at sections A and B of a wall,  $\varepsilon_h$  can be determined using Eqn. 3.4. Where  $\delta S_h$  indicates the differential horizontal movement as shown in Eqn. 3.5, while  $L$  denotes the distance between the sections A & B. Horizontal strain is directly involved into the deep beam equations

$$\varepsilon_h = \frac{\delta S_h}{L} \quad (3.4)$$

$$\delta S_h = S_{h,A} - S_{h,B} \quad (3.5)$$

### 3.3 Limiting tensile strain method (LTSM)

Burland and Wroth (1974) extended the research conducted by Polshin and Tokar (1957) and proposed a deep beam analogy applicable to both masonry and framed structures by assuming that visible cracking in a given material is associated with a limiting tensile strain ( $\varepsilon_{lim}$ ). This analogy aims to facilitate the straightforward calculation of  $\varepsilon_{lim}$  values. In the deep beam analogy framework, it is assumed that the deflected shape of the beam is already known, and the criterion for initial cracking deflection is established when the value of  $\varepsilon_{lim}$  is reached at a specified point within the beam. Fig. 3.4 shows the representative deep beam model, which is an approximation of real structures, along with two possible extreme modes of deformation: bending only and shearing only. The midpoint deflection ( $\Delta$ ) is assumed to occur within either the hogging or sagging ground settlement zone. Here,  $L$  represents the length of the beam (which in practice may correspond to the deflected or total length of the structure), and  $H$  denotes the height of the beam (which in real-world scenarios, corresponds to the height of the building from the foundation level excluding the roof from calculations). The fictitious deep beam is modelled as elastic, isotropic, and simply supported. Soil-structure interaction effects are neglected, and only free-field ground settlements are taken into consideration.

Timoshenko (1957) has given the following expression for the central deflection of a centrally loaded beam of unit thickness in both shear and bending.

$$\frac{\Delta}{L} = \frac{PL^2}{48EI} \left[ 1 + \frac{18EI}{L^2HG} \right] \quad (3.6)$$

In this Eqn. 3.6,  $E$  and  $G$  are the modulus of elasticity and the shear modulus of the depicted building, respectively. However, they are considered as a relative stiffness in the form of  $E/G$  in calculations, indicating the longitudinal/shear stiffness ratio of the structure.  $I$ , represents the second moment of the area of the fictitious deep beam, while  $P$  represents a central point load.

Eqn. 3.6 can be rewritten in terms of maximum bending tensile strain ( $\varepsilon_{bmax}$ ) and  $\Delta/L$  can be related to  $\varepsilon_{bmax}$  as shown in equation below (Burland et al., 1977).

$$\frac{\Delta}{L} = \varepsilon_{bmax} \left[ \frac{L}{12t} + \frac{3EI}{2tLHG} \right] \quad (3.7)$$

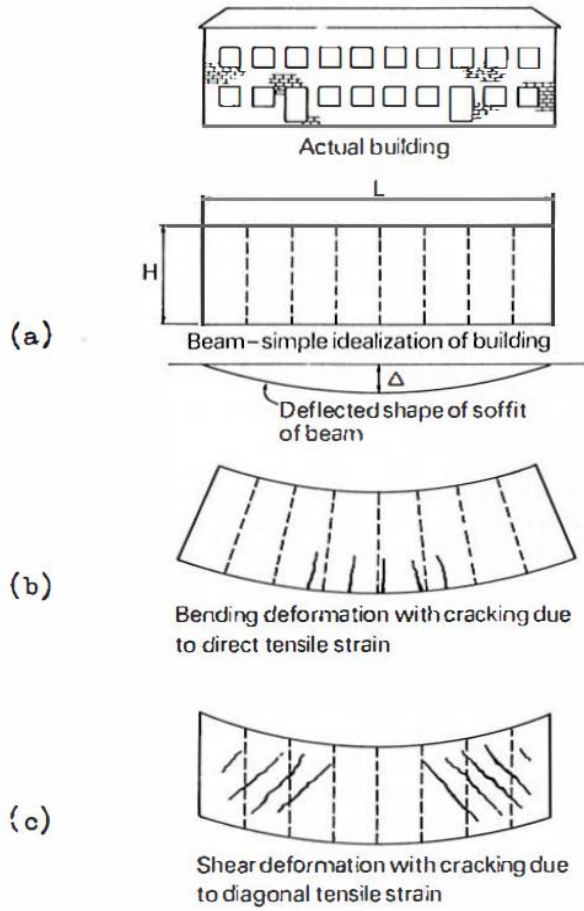


Fig. 3.4 Cracking of a simple beam in bending and in shear (Burland et al., 1977)

Eqn. 3.7 is applicable when the first visible crack is presumed to develop due to bending, caused by either sagging or hogging of the structure.  $\varepsilon_{bmax}$  represents the maximum tensile strain occurring at the outermost fiber of the fictitious model beam once the induced  $\Delta/L$  is fully realized. In Eqn. 3.7,  $t$  denotes the depth of the neutral axis from the edge of the beam in tension.

Similarly, the Eqn. 3.6 can be reformulated in terms of maximum diagonal tensile strain ( $\varepsilon_{dmax}$ ) and the relationship between  $\Delta/L$  and  $\varepsilon_{dmax}$  is shown in Eqn. 3.8.

$$\frac{\Delta}{L} = \varepsilon_{dmax} \left[ 1 + \frac{HL^2G}{18EI} \right] \quad (3.8)$$

Eqn. 3.8 is applicable when the first visible crack is assumed to develop in shear, caused by either sagging or hogging of the structure.  $\varepsilon_{dmax}$  represents the maximum tensile strain occurring in the diagonal direction once the induced  $\Delta/L$  is fully realized.

The above-described equations employ a shear form factor of 1.5, derived from the maximum shear strain at the neutral axis of the beam. Eqn. 3.6 can be rewritten as

$$\frac{\Delta}{L} = \frac{PL^2}{48EI} + \frac{1.5P}{4HG} \quad (3.9)$$

Eqn. 3.9 does not consider the actual variation in shear strains across the height of the beam. This leads to an overestimation of the shear deflection contribution and thus to an underestimation of the tensile strains used in the damage prediction according to the tensile strain method (Netzel, 2005). Netzel (2005) highlighted that Timoshenko and Gere (1972) also present solutions derived from the more exact theory of elasticity and the method of virtual work respectively considering the proper shear strain distribution over the height of the beam while calculating the contributions of the shear deflections to the overall deflection of the beam. For a rectangular beam a shear form factor of 1.2 is derived by Timoshenko & Gere (1972), instead of the simplified shear factor of 1.5 as shown in Eqn. 3.9. With this modified shear form factor, the equations for the deflection ratio caused by a fictitious point load P can be written as Eqn. 3.10.

$$\frac{\Delta}{L} = \frac{PL^2}{48EI} + \frac{1.2P}{4HG} \quad (3.10)$$

The accurate shear form factor (1.2) results in a strain increase of approximately 20% to 25% within practical ranges of  $L/H$  (between 0.7 and 1.5) (Netzel, 2005). Therefore, a shear form factor of 1.2 is used for predicting the building damage using the tensile strain method. Therefore, using Eqn. 3.10, Eqn. 3.7 and Eqn. 3.8 can be rewritten as

$$\frac{\Delta}{L} = \varepsilon_{bmax} \left[ \frac{L}{12t} + \frac{6EI}{5tLHG} \right] \quad (3.11)$$

$$\frac{\Delta}{L} = \varepsilon_{dmax} \left[ 0.8 + \frac{HL^2G}{18EI} \right] \quad (3.12)$$

Boscardin and Cording (1989) emphasized that ground deformations induced by mining, tunneling, and open excavation can exhibit a significant horizontal strain component ( $\varepsilon_h$ ). Incorporating this horizontal strain into the analyses mentioned above will lead to the initiation of damage at smaller levels of differential settlement and angular distortion. Because of this the total tensile strain in bending can be considered to have two components as shown in Eqn. 3.13.

$$\varepsilon_{bt} = \varepsilon_{bmax} + \varepsilon_h \quad (3.13)$$

Similarly, total diagonal tensile strain can be considered to have two components and determined using Mohr's circle of strain (assuming Poisson's ratio as 0.3) as shown in Eqn. 3.14 (Mair et al., 1996).

$$\varepsilon_{dt} = 0.35\varepsilon_h + [(0.65\varepsilon_h)^2 + \varepsilon_{dmax}^2]^{0.5} \quad (3.14)$$

$$\varepsilon_{lim} = \max(\varepsilon_{bt_{hogging}}, \varepsilon_{dt_{hogging}}, \varepsilon_{bt_{sagging}}, \varepsilon_{dt_{sagging}}) \quad (3.15)$$

The parameter  $\varepsilon_{lim}$  obtained from Eqn. 3.15, proves valuable in identifying structure damages, as it remains independent of the loading direction and can be applied in various loading conditions.

Boscardin and Cording (1989) has given the expression for determining the maximum angular distortion ( $\beta_{max}$ ) as depicted in Eqn. 3.16.

$$\beta_{max} = \frac{3\Delta}{L} \left[ \frac{1+4\left(\frac{E}{G}\right)\left(\frac{H^2}{L^2}\right)}{1+6\left(\frac{E}{G}\right)\left(\frac{H^2}{L^2}\right)} \right] \quad (3.16)$$

### 3.4 Damage classification

As per Burland et al. (1977), visual appearance, serviceability and stability are three basic criteria which have to be satisfied when considering limiting movements. Considering this Guglielmetti et al. (2008) defines three types of damage classifications for a building namely Aesthetic damages, Functional damages and structural damages. Aesthetic damages manifest as minor cracks in structures, predominantly impacting internal walls and surface finishes. These damages are easily remedied, often requiring only redecoration to conceal the superficial cracks. Functional damages manifest as the loss of serviceability and functionality or serviceability building components (e.g., doors and windows becoming stuck, pipelines sustaining damage) or sensitive equipment housed within the building being affected; While the structural integrity remains intact, the loss of functionality can result in commercial and economic repercussions for both the building and its occupants. Structural damages resulted from cracks or severe deformations in load-bearing structures, potentially causing partial or complete building collapse. These damages may occasionally be partially hidden by surface finishes and signs like whitewash and plaster serve as reliable indicators of crack propagation. However, Skempton and MacDonald (1956) concluded that for the majority of buildings the allowable settlement is governed more by architectural damage than by overstressing of the structure. With reference to the different damage classification available in literature a consolidated damage classification is given in Table 3-1 and Table 3-2,

based on influencing parameters such as limiting tensile strains ( $\varepsilon_{lim}$ ), maximum settlement of building ( $S_{bmax}$ ), and maximum angular distortion ( $\beta_{max}$ ).

Table 3-1 Damage classification based on crack width &  $\varepsilon_{lim}$  (Boscardin and Cording, 1989; Burland et al., 1977; Gaba et al., 2003; Guglielmetti et al., 2008; Mair et al., 1996; Rankin, 1988)

| Risk Category | Damage Category      | Degree of Damage | Description of damage (ease of repair)                                                                                                                                                                                                     | Crack Width (mm)                      | Tensile strain ( $\varepsilon_{lim}$ , %) | Description of action required                                                                                                                                                                     |
|---------------|----------------------|------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------|-------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 0             | Aesthetic            | Negligible       | Hairline cracks                                                                                                                                                                                                                            | < 0.1                                 | 0–0.05                                    | No action, except for buildings identified as particularly sensitive for which an individual assessment should be made                                                                             |
| 1             | Aesthetic            | Very Slight      | Fine cracks can be easily treated. Damage generally restricted to internal wall finishes. Some cracks in external brickwork or masonry                                                                                                     | < 1.0                                 | 0.05–0.075                                |                                                                                                                                                                                                    |
| 2             | Aesthetic            | Slight           | Cracks easily filled. Redecoration probably required. Recurrent cracks can be masked by suitable linings. Cracks can be visible externally and repointing may be required to ensure water-tightness. Doors and windows may stick slightly. | < 5.0                                 | 0.075–0.15                                | Crack survey and schedule of defects, to fairly assess & compensate any resulting damage. Identify any buildings and pipelines particularly vulnerable to structural damage and assess separately. |
| 3             | Aesthetic/Functional | Moderate         | Cracks require some opening up and patched. Repointing of external brickwork and a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Water-tightness often impaired.                       | 5 – 15 (many cracks with width > 3mm) | 0.15–0.3                                  | Crack survey, a schedule of defects, and a structural assessment. Predict extent of structural damage, assess safety risk, choose whether to accept damage and repair,                             |

continued on page 34

Table 3-1 (continued)

| Risk Category | Damage Category            | Degree of Damage | Description of damage (ease of repair)                                                                                                                                                                               | Crack Width (mm)                         | Tensile strain ( $\epsilon_{lim}$ , %) | Description of action required                                                                                                                                                                                                                                                                                                                                                               |
|---------------|----------------------------|------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------|----------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| 4             | Functional/ serviceability | Severe           | Extensive repair work as breaking-out and replacing wall sections. Windows and door frames distorted, floor sloping and walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. | 15–25 (Depend on the number of cracks)   | >0.3                                   | Crack survey, a schedule of defects, and a structural assessment. Predict extent of structural damage, assess safety risk, choose whether to accept damage and repair, take precautions to control damage or, in extreme cases, demolish. Buried pipelines at risk: identify vulnerable services, and decide whether to repair, replace with a type less likely to suffer damage, or divert. |
| 5             | Structural                 | Very Severe      | Major repair job involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.                                          | >25 (But depend on the number of cracks) |                                        |                                                                                                                                                                                                                                                                                                                                                                                              |

Table 3-2 Damage classification based on  $S_{bmax}$  and  $\beta_{max}$  (Guglielmetti et al., 2008; Rankin, 1988)

| Risk Category | Damage Category | Degree of Damage | Description of damage (ease of repair)                                        | Maximum angular distortion ( $\beta_{max}$ ) | Maximum Settlement of Building ( $S_{bmax}$ , mm) |
|---------------|-----------------|------------------|-------------------------------------------------------------------------------|----------------------------------------------|---------------------------------------------------|
| 1             | Aesthetic       | Negligible       | Superficial damage unlikely.                                                  | < 1/500                                      | < 10                                              |
| 2             | Aesthetic       | Slight           | Possible superficial damage which is unlikely to have structural significance | 1/500 – 1/200                                | 10 -50                                            |

continued on page 35

Table 3-2 (continued)

| Risk Category | Damage Category               | Degree of Damage | Description of damage (ease of repair)                                                                              | Maximum angular distortion ( $\beta_{max}$ ) | Maximum Settlement of Building ( $S_{bmax}$ , mm) |
|---------------|-------------------------------|------------------|---------------------------------------------------------------------------------------------------------------------|----------------------------------------------|---------------------------------------------------|
| 3             | Functional                    | Moderate         | Expected superficial damage and possible structural damage to buildings, possible damage to rigid pipelines.        | 1/200 – 1/50                                 | 50 -75                                            |
| 4             | Serviceability and structural | High             | Expected structural damage to buildings and expected damage to rigid pipelines; possible damage to other pipelines. | >1/50                                        | > 75                                              |

Limiting strain is preferred over strength for damage classification because its value tends to remain relatively consistent across a broad spectrum of common building materials in terms of type and strength. Furthermore, it maintains physical significance even after cracking, unlike strength. For the practical use of the LTSM, some diagrams based on the damage classifications given in Table 3-1 and Table 3-2 were proposed by Boscardin and Cording (1989), Burland (1995) and Burland et al. (2001) shown in Fig. 3.5 and Fig. 3.6.

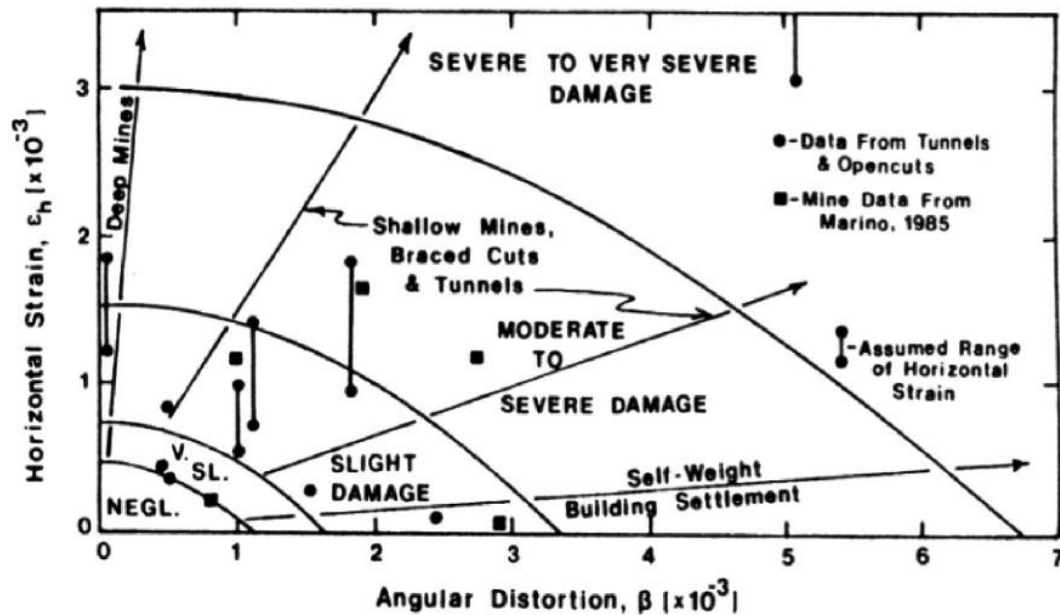


Fig. 3.5 Damage Category- Angular Distortion ( $\beta$ ) and Horizontal Strain ( $\epsilon_h$ ) (Boscardin and Cording, 1989)

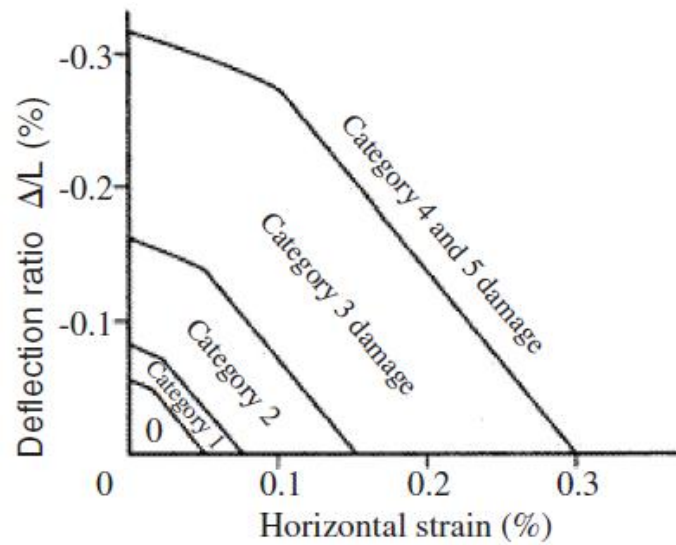


Fig. 3.6 Damage Category- Deflection ratio ( $\Delta/L$ ) and Horizontal Strain ( $\epsilon_h$ ) (Burland, 1995 and Burland et al., 2001)

### 3.5 Staged Assessment

Mair et al. (1996) proposed a three-stage process for Building Risk Assessment (BRA) in bored tunneling. The process consists of three stages, Preliminary Assessment, Second Stage Assessment, and Detailed Evaluation, which has since been widely adopted (Fok et al., 2012; Harris and Franzius, 2005; Torp-Petersen and Black, 2001).

#### 3.5.1 Stage I: Preliminary assessments

In this stage, all structures and utilities located within the influence zone defined by the greenfield settlement troughs are identified. Ground surface settlement contours are prepared, and all buildings predicted to experience settlements of less than 10 mm and slopes smaller than 1/500 are classified in the negligible damage category. Such buildings are therefore eliminated from further analysis, allowing attention to be focused on structures with potentially higher vulnerability.

#### 3.5.2 Stage II: Second stage assessment

In this stage limiting tensile strain ( $\epsilon_{lim}$ ) is calculated for buildings experiencing settlements greater than 10 mm and slopes exceeding 1/500, using limiting tensile strain method discussed earlier in this chapter and damage category is assigned. At this stage, buildings classified as having negligible, very slight, or slight damage are considered low risk and are excluded from further evaluation, while those assigned to moderate or higher damage categories are carried forward to the final stage.

### 3.5.3 Stage III: Detailed Evaluation

This stage is undertaken for buildings that are assigned a damage category of 3 (Moderate) or higher during the Stage II assessment. This stage involves refining the estimates of ground movement and structural distortion by analysing the effects of soil-structure interaction, structural details and excavation methods through advanced computational methods such as finite element or finite difference analysis. These methods account for three-dimensional geometry, non-linear ground response, foundation depth, and building stiffness, thereby providing a comprehensive understanding of soil–structure interaction. Additionally, detailed evaluation considers the potential for additional stresses induced in structural elements and ensures that serviceability limit states are not exceeded.

It should be noted that the Stage-I BRA thresholds of maximum settlement less than 10 mm and slope less than 1/500 are intended as preliminary screening criteria for identifying structures with negligible to very slight damage potential under typical tunnelling conditions. These limits are based on established tunnelling assessment practices. However, old, weak, and heritage structures may exhibit increased sensitivity even at lower deformation levels due to material degradation, pre-existing distress, reduced structural redundancy, or architectural vulnerability. Therefore, such structures should be subjected to individual assessment and engineering judgement, irrespective of whether the Stage-I screening thresholds are satisfied. Also, Neglecting the “cantilever effect” of a structure proceeding beyond the settlement influence area (ie. Stage I) can lead to a significant underestimation of the tensile strains. For longer structures this effect has therefore to be considered carefully in assessing damage risks (Netzel, 2005). Also, the LTSM approach used in Stage II is very conservative as soil-structure interaction is ignored. In reality, presence of the building tends to reduce both the horizontal displacements and vertical settlements, thereby also reducing the induced tensile strains. (Amorosi et al., 2014; Camós et al., 2014; Giardina et al. 2012; Mirhabibi and Soroush, 2012; Potts and Addenbrooke, 1997). The reduction in tunneling-induced strain due to soil–structure interaction is highly dependent on the stiffness and geometry of the structure, as well as the stiffness of the surrounding soil. Literature suggests that strain reduction may vary from negligible to over 90%, with flexible structures tending to approach greenfield behaviour, while stiff structures exhibit significant interaction effects and substantial reduction in deformation (Giardina et al. 2012; Mair and Taylor, 1997; Mirhabibi and Soroush, 2012; Potts and Addenbrooke, 1997). For example, Amorosi et al. (2014) and Camós et al. (2014) reported reductions in building strains of approximately 50%, while Farrell et al. (2014) observed a reduction of around 30%.

Therefore, in order to overcome the shortcoming as described, in the current study Stage II assessment has been done considering the total length of the building. In the present study, ground deformation profiles corresponding to a predefined volume loss ( $V_L$ ), as obtained from PLAXIS 2D analyses, are transferred to the foundation of an existing building located in the vicinity of the tunneling alignment. These induced deformations are subsequently utilized to evaluate the limiting tensile strain within the structure, thereby facilitating the prediction of potential damage levels in accordance with the LTSM.

## CHAPTER 4

### STUDY AREA AND INPUT DATA

The Delhi–Meerut Regional Rapid Transit System (Delhi–Meerut RRTS) is a semi-high-speed rail corridor with a total length of approximately 82.15 km, currently under construction to provide enhanced connectivity between Delhi, Ghaziabad, and Meerut. The alignment consists of both elevated and underground sections. The tunnel alignment for Package 4 is shown in Fig. 4.1. The underground component is divided into two major stretches:

- A 5.6 km section between New Ashok Nagar and Sahibabad (Package 4), which includes an underground station at Anand Vihar.
- A 5.68 km section between Brahampuri DN Ramp and Begumpul UP Ramp (Package 8), incorporating three underground stations at Meerut Central, Bhaisali, and Begumpul.

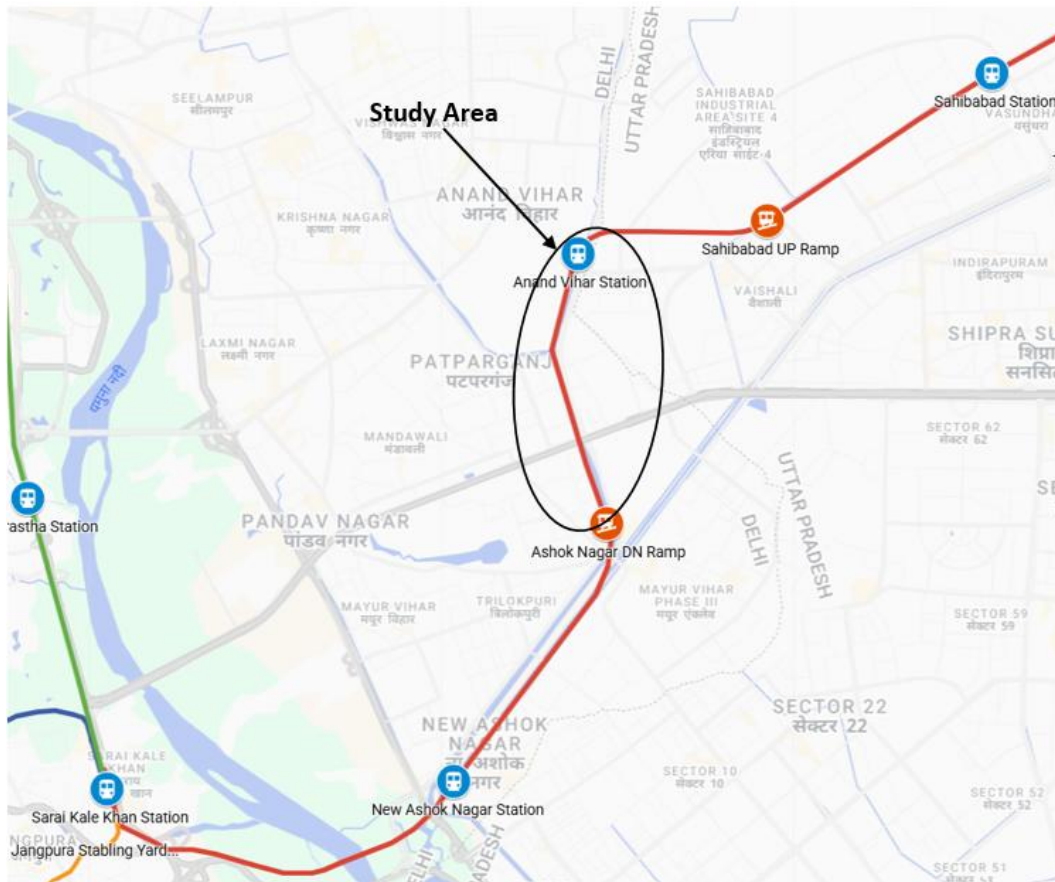


Fig. 4.1 Tunnel Alignment

Excavation will be carried out using four Earth Pressure Balance (EPB) Tunnel Boring Machines (TBMs), all of which are named Sudarshan. The construction plan involves:

- TBM Sudarshan 4.1 and 4.2 excavating approximately 2.8 km of twin tunnels from the Anand Vihar launch shaft to the Khichripur retrieval shaft, situated at the northern end of the Ashok Nagar down-ramp.
- TBM Sudarshan 4.3 and 4.4 constructing 2.09 km of parallel tunnels between the Anand Vihar launch shaft and the Sahibabad up-ramp.

For the purpose of the present study, the tunnel alignment considered extends from the Anand Vihar launch shaft to the Khichripur retrieval shaft (Fig. 4.2). The necessary technical information and alignment details are primarily sourced from open-access tender documents and industry databases (e.g., The Metro Rail Guy, BidAssist).

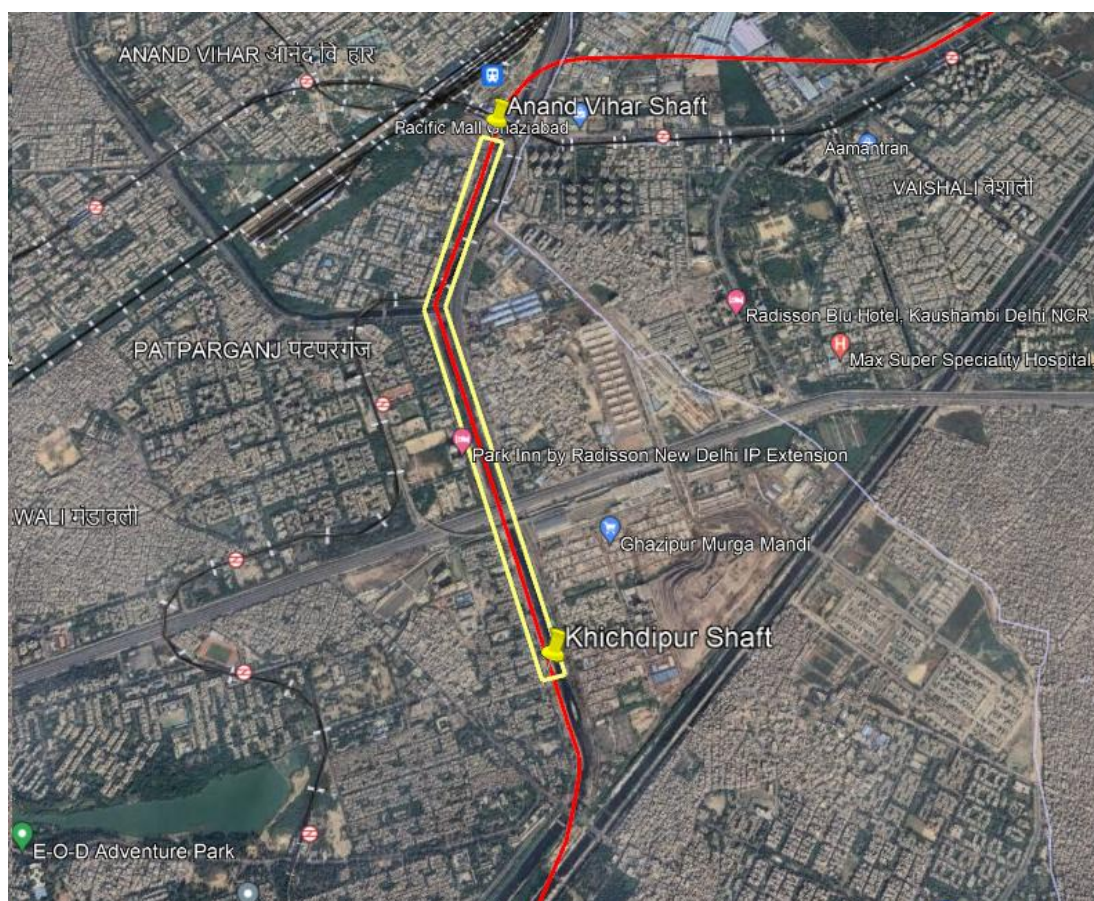


Fig. 4.2 Study Area- Google Earth View of RRTS Package - 4

The selected tunnel stretch passes beneath densely built-up, non-engineered residential zones in the Patparganj and Khichripur areas of East Delhi (Fig. 4.3). These localities consist predominantly of unplanned constructions with limited

adherence to structural codes, making them particularly vulnerable to tunneling-induced ground movements.

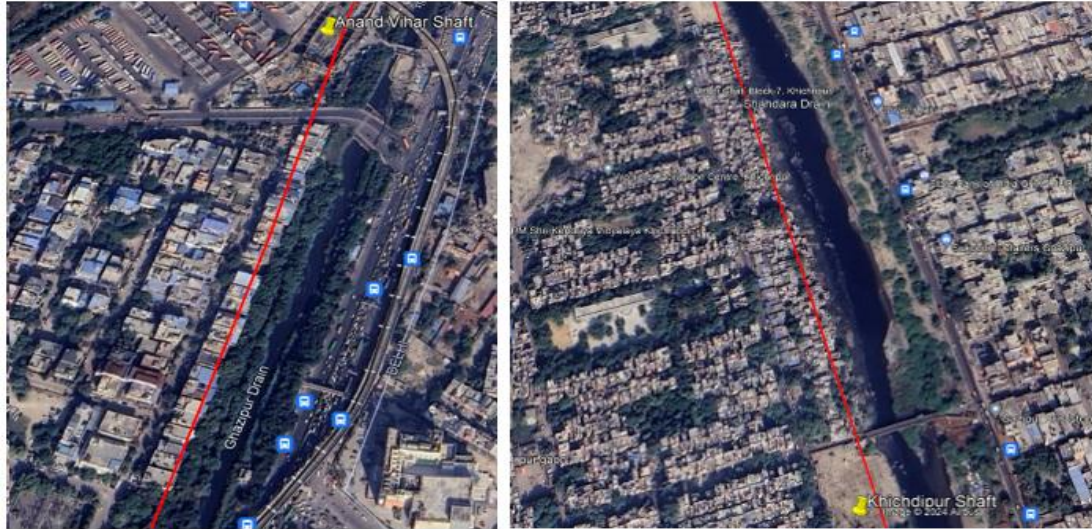


Fig. 4.3 Google Earth View of adjacent surface Structures

#### 4.1 Geotechnical Data

In practice, natural soil deposits are not homogeneous, elastic or isotropic. The stratification of soil deposit may change greatly within a horizontal distance of 15 to 30 m. The geological data for the study area was obtained from borelogs provided in tender documents (BidAssist). Fig. 4.4 shows a Google Earth image showing the locations of the boreholes, drilled down to the stipulated depth of 40.45 m below the existing ground level in the study area.

##### 4.1.1 Geotechnical Investigations

The geotechnical investigations include exploratory drilling, field and laboratory tests, ground water table monitoring, data processing, and interpretation of data for design purpose.

##### 4.1.1.1 Field test- Standard Penetration Test (SPT)

The standard penetration test is an in-situ test that is coming under the category of penetrometer tests. The test measures the resistance of the soil strata to the penetration undergone. The standard penetration tests were carried out in all 150 mm diameter boreholes at 1.5 m intervals and at every change of strata in accordance of IS 2131: 1981. Disturbed and undisturbed soil samples were recovered for the strata identification and testing purposes.

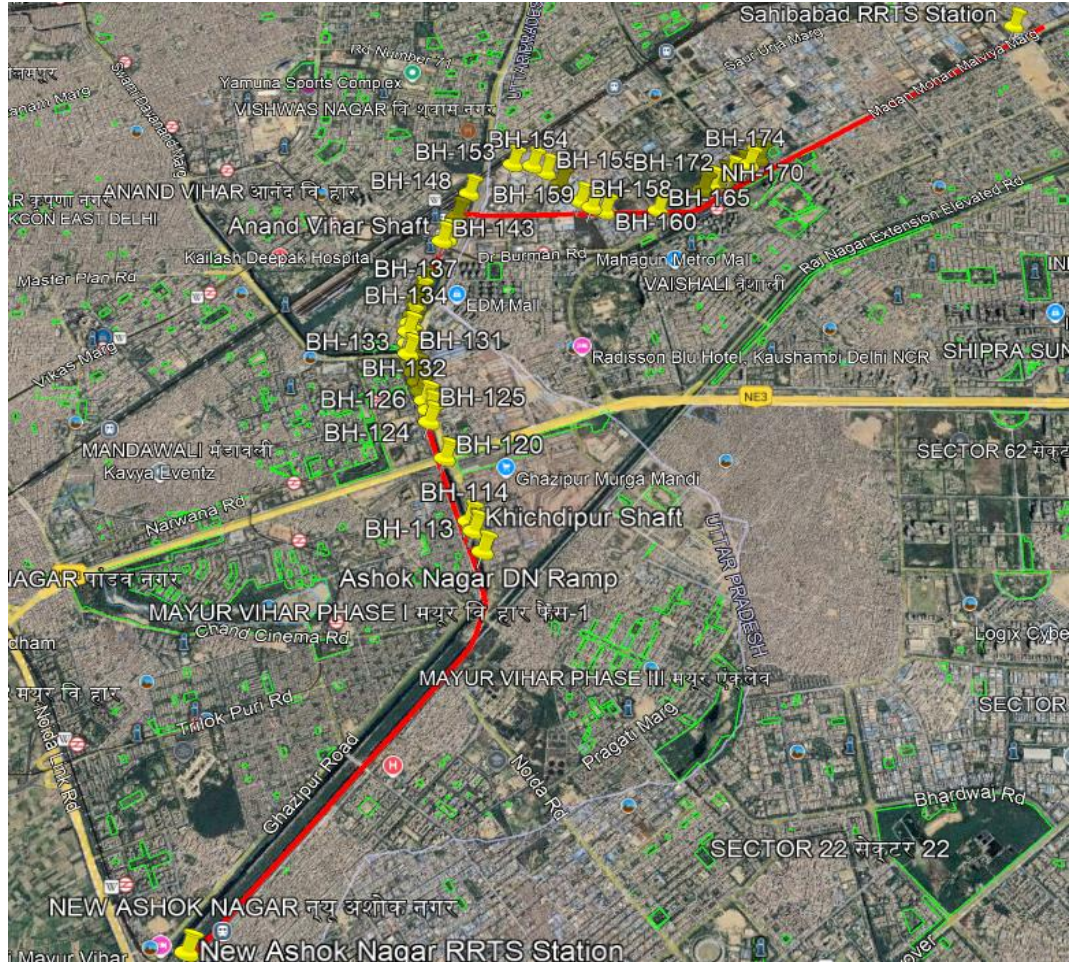


Fig. 4.4 Study Area- Borehole Location

SPT  $N$  was recorded by counting the number of blows for 30 cm penetration after an initial seating drive of 15 cm (for which number of blows were not counted). Corrected SPT  $N_{60}$  is determined using Eqn. 4.1, as given below (IS 1893: 2016). Variation of SPT  $N$  and  $N_{60}$  with depth is shown in Fig. 4.5 and Fig. 4.6.

$$N_{60} = N \times C_E \times C_B \times C_R \times C_S \quad (4.1)$$

$N$  = Observed SPT  $N$

$C_E$  = Correction factor for hammer energy ratio = 0.75

$C_B$  = Correction factor for borehole diameter = 1.05

$C_R$  = Correction factor for rod length = 0.75 (< 3m), 0.8 (3 – 4m), 0.85 (4 – 6m), 0.95 (6 – 10m), 1 (10 – 30)

$C_S$  = Correction factor for sampler type = 1 (Standard sampler)

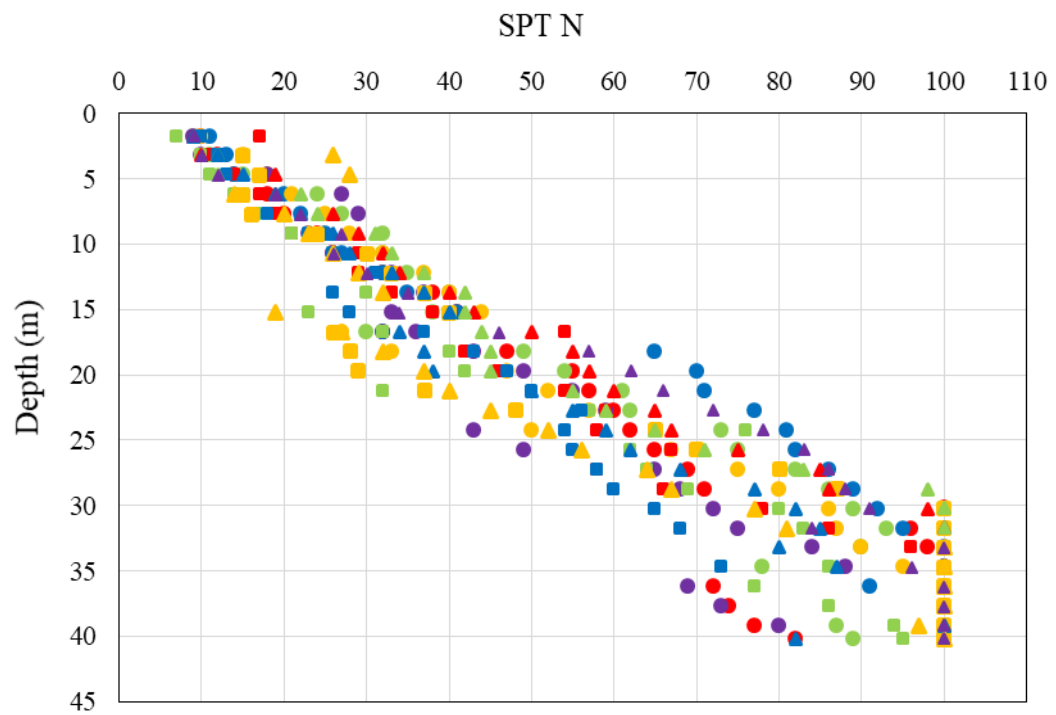


Fig. 4.5 Variation of SPT N with depth

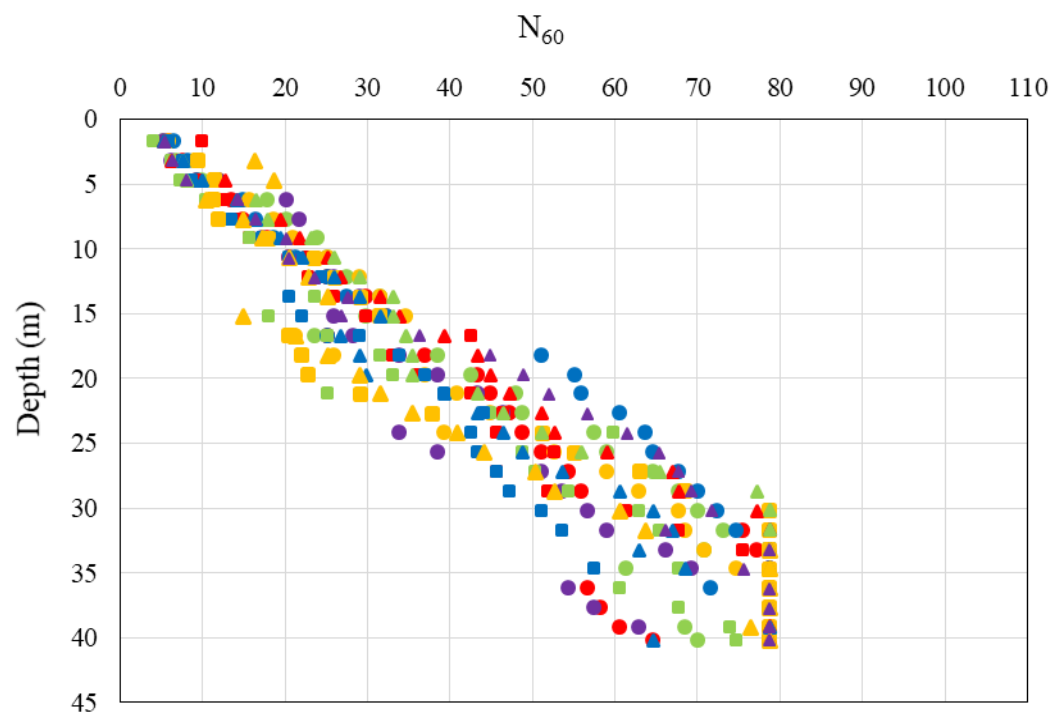


Fig. 4.6 Variation of  $N_{60}$  with depth

#### 4.1.1.2 Laboratory Tests

For the design work, it is essential to obtain the actual soil stratification at the site along with the laboratory test results of soil samples at different depths. The laboratory tests conducted on the soil samples collected from the boreholes are presented in the Table 4-1. Variations of dry density, bulk density, undrained cohesion, undrained friction angle, drained friction angle and plasticity index with depth are shown in Fig. 4.7, Fig. 4.8, Fig. 4.9, Fig. 4.10, Fig. 4.11 and Fig. 4.12 respectively.

Table 4-1 Laboratory Tests

| S. No. | Test                                                                          | Code/ Standard                   |
|--------|-------------------------------------------------------------------------------|----------------------------------|
| 1.     | Determination of water content                                                | IS : 2720 - 1980 (Part 2)        |
| 2.     | Determination of Specific Gravity: Fine Grained Soils.                        | IS : 2720 - 1980 (Part 3/ Sec 1) |
| 3.     | Determination of Specific Gravity: Fine, Medium & Coarse grained soils.       | IS : 2720 - 1980 (Part 3/ Sec 2) |
| 4.     | Grain size analysis                                                           | IS : 2720 - 1985 (Part 4)        |
| 5.     | Determination of water content - dry density relation using heavy compaction. | IS : 2720 (Part 8) -1983         |
| 6.     | Determination of liquid and plastic limit                                     | IS : 2720 - 1985 (Part 5)        |
| 7.     | Determination of shear strength parameters using triaxial apparatus.          | IS 2720-1981 Part 12             |

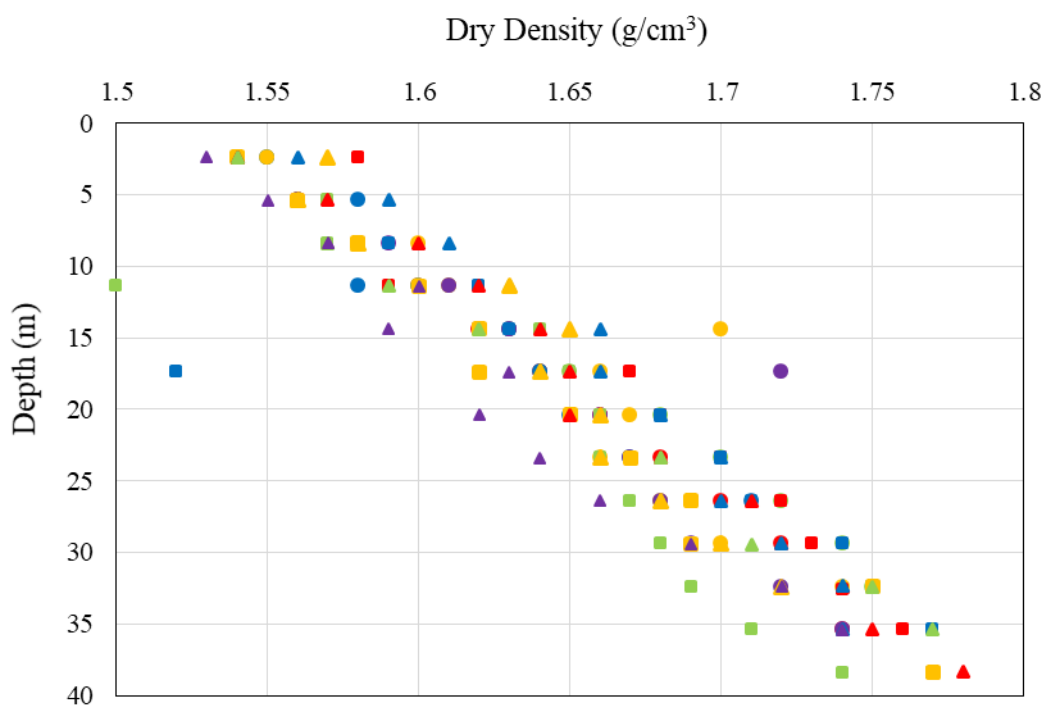


Fig. 4.7 Variation of dry density with depth



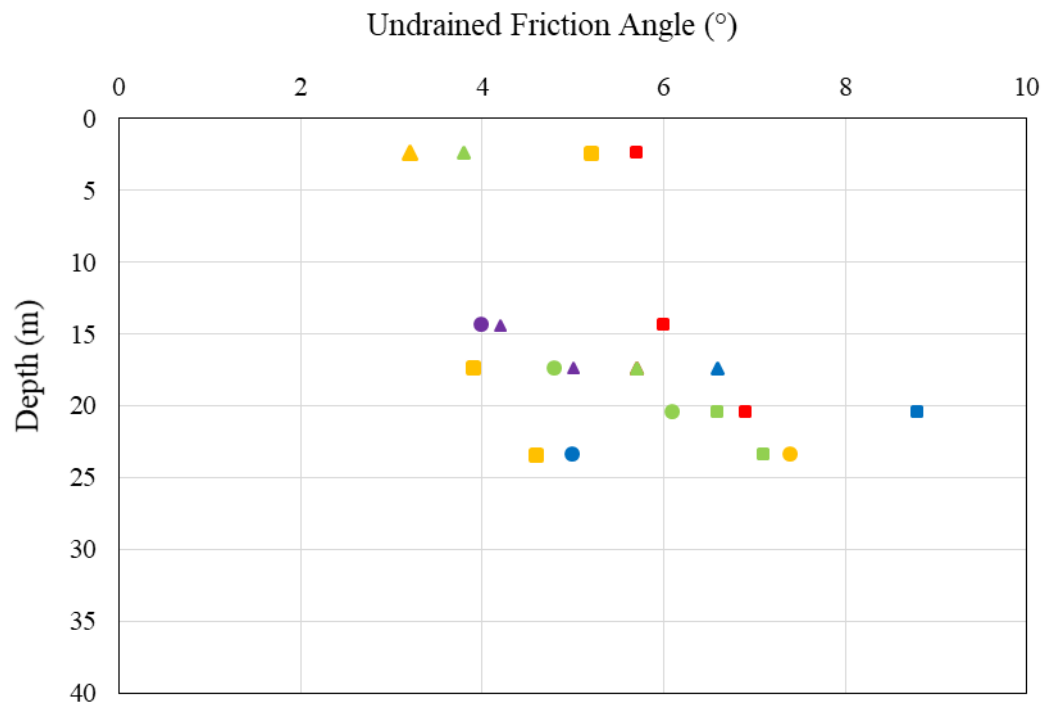


Fig. 4.10 Variation of undrained friction angle with depth

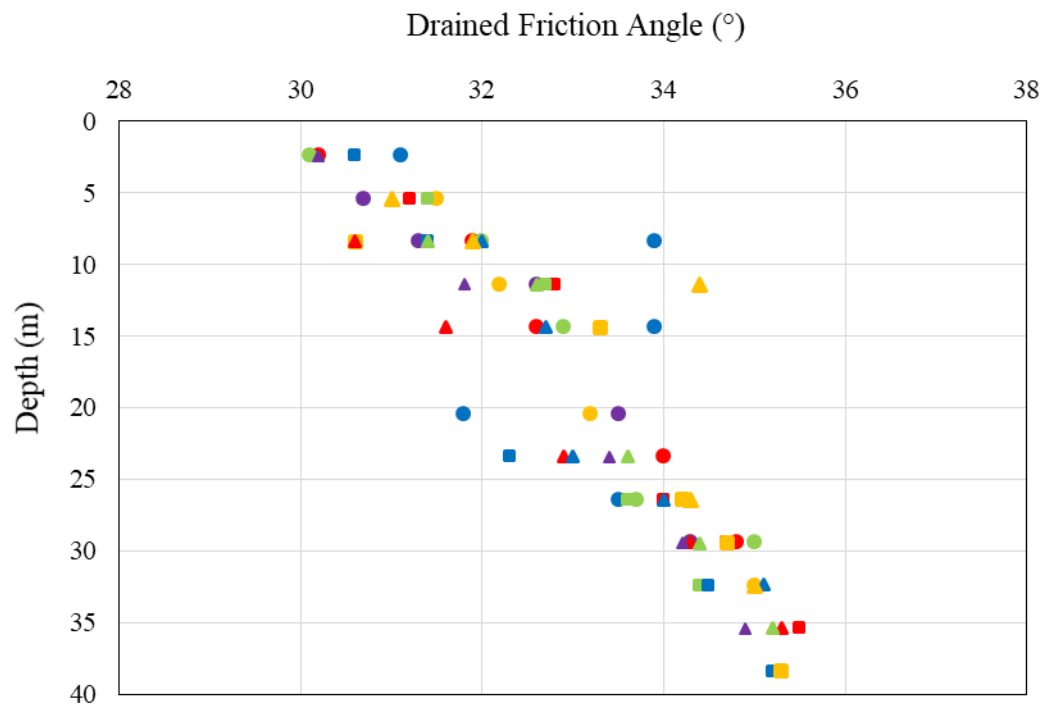


Fig. 4.11 Variation of drained friction angle with depth

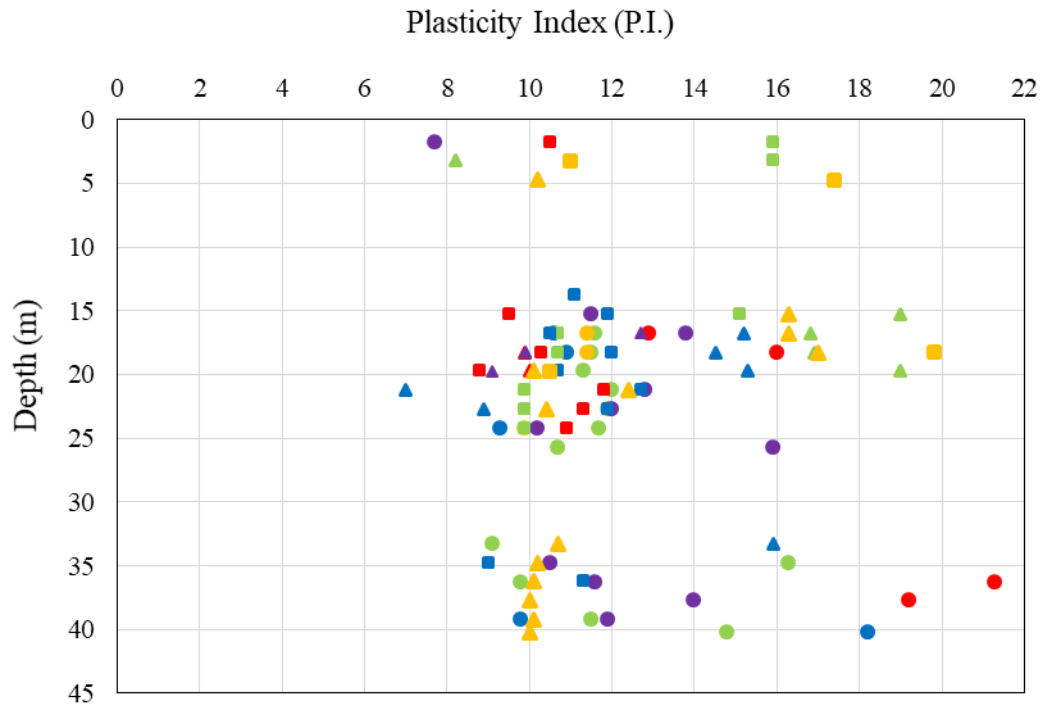


Fig. 4.12 Variation of plasticity index with depth

#### 4.1.2 Subsurface stratigraphy and water table

Based on the soil classification information from borelogs and SPT results, the subsurface profile has been subdivided into eight distinct layers, as summarized in Table 4-2. Accordingly, a geological profile has been developed to interpret the subsurface stratigraphy (Fig. 4.13). The subsurface geology along this alignment primarily comprises two soil types: Cohesionless: fine silty sand (SM) and Cohesive: low-plasticity sandy clayey silt (CL). The maximum groundwater level was encountered at a depth of about 15 meters below ground surface.

Table 4-2 Classification of Subsurface Soil Layers

| Layer No. | Depth (m) | Type | SPT $N$ | $N_{60}$ |
|-----------|-----------|------|---------|----------|
| Layer 1   | 0 – 5     | SM   | 13      | 8        |
| Layer 2   | 5 – 10    | SM   | 16      | 12       |
| Layer 3   | 10 – 15   | SM   | 26      | 20       |
| Layer 4   | 15 – 20   | CL   | 20      | 16       |
| Layer 5   | 20 – 25   | CL   | 29      | 23       |
| Layer 6   | 25 – 30   | SM   | 58      | 45       |
| Layer 7   | 30 – 40   | SM   | 65      | 50       |
| Layer 8   | Below 40  | SM   | 77      | 60       |

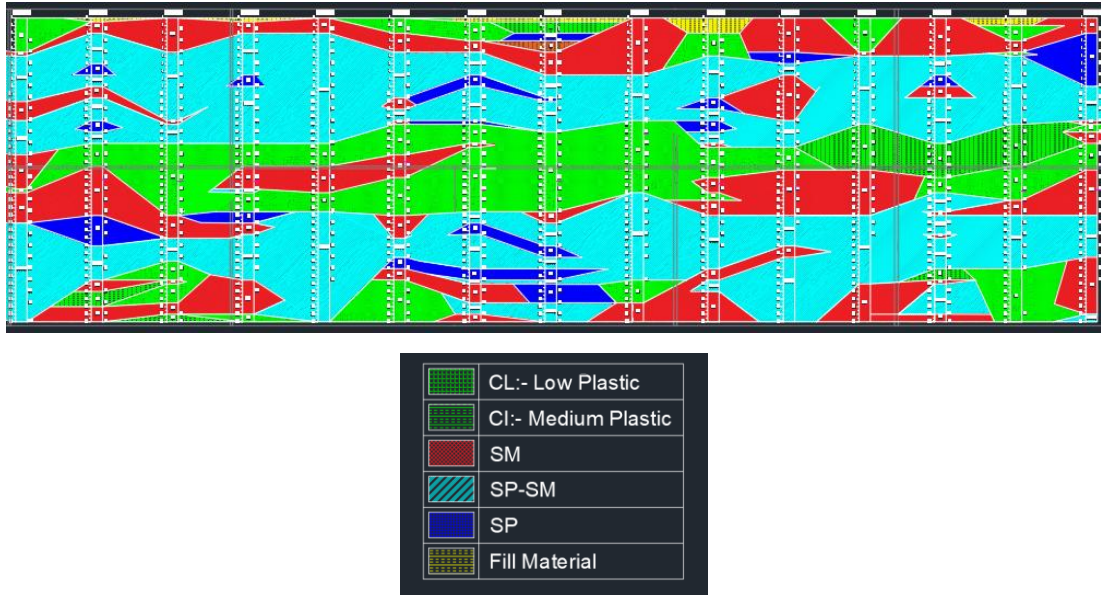


Fig. 4.13 Study Area- Geological Profile

## 4.2 Design Geotechnical Parameters

Atterberg's Limit tests indicate that clayey silts are the border line between silt and clay, but it shows plasticity. Based on the plastic results of the Atterberg limits, it is considered that at least in part, the Alluvial Silt with low plasticity and low clay content may exhibit short term undrained behavior.

### 4.2.1 Undrained Shear Strength

Equivalent values of undrained shear strength are derived from SPT  $N$  values using published relationship with Plasticity Index (CIRIA Report 143, 1995). For the range of plasticity index value obtained from the Atterberg's limit test on cohesive alluvium soil, a multiplication factor between SPT  $N$  and undrained shear strength,  $C_u$ , of between 5 and 7 (Fig. 4.14) is indicated as being appropriate. Therefore, a value of 5 has been adopted to establish a correlation. The correlated  $C_u$  values are found to be reasonably consistent with the undrained shear strength results available in the Geotechnical Investigation Report, thereby supporting the reliability of the adopted parameters. Therefore, the selected undrained shear strength values for the CL layers were based on both available GI test results and established empirical correlations. For the cohesive (CL) layers, the undrained friction angle is conservatively considered as  $0^\circ$ , consistent with undrained behavior assumptions commonly adopted for short-term tunnel stability analyses in low-plasticity cohesive soils.

#### 4.2.2 Modulus of Elasticity of Soil

Field tests were not performed, so modulus of Elasticity of Soil shall be based on SPT N value as per CIRIA Report 143 (1995) shown below:

For Granular Soil

$$\frac{E'}{N_{60}} = 1 \text{ (MPa)} \quad (4.2)$$

For Cohesive Soil

$$\frac{E_u}{N} = 1 \text{ (MPa)} \quad (4.3)$$

$$E' = 0.73E_u \quad (4.4)$$

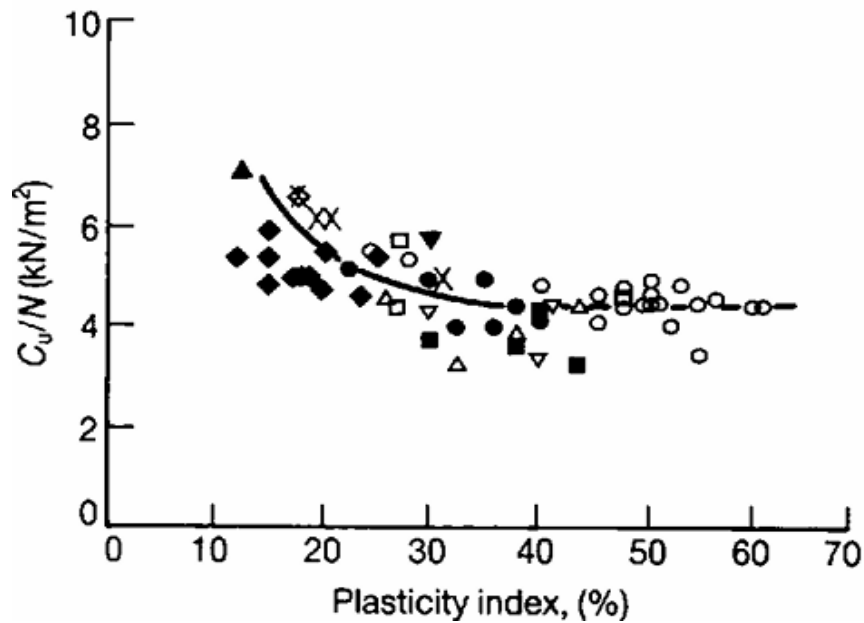


Fig. 4.14 Relationship between  $C_u/N$  and Plasticity Index (CIRIA Report 143, 1995)

#### 4.2.3 Poisson's ratio of soil

Field tests were not performed to obtain the Poisson's Ratio of the soil. In the absence of field test data, the following literature from Bowles (1996) is used to obtain the Poisson's ratio for different soils as given in Table 4-3.

The geotechnical parameters adopted for the analyses in the present study are summarized in Table 4-4.

Table 4-3 Range of Poisson's ratio values for different types of soils (Bowles, 1996)

| Type of soil                      | Poisson's ratio ( $\nu$ )           |
|-----------------------------------|-------------------------------------|
| Clay, Saturated                   | 0.4 – 0.5                           |
| Clay, Unsaturated                 | 0.1 – 0.3                           |
| Sandy clay                        | 0.2 – 0.3                           |
| Silt                              | 0.3 – 0.35                          |
| Sand, gravelly sand commonly used | -0.1 – 1 (0.3 – 0.4)                |
| Rock                              | 0.1 – 0.4 (depends on type of rock) |
| Loess                             | 0.1 – 0.3                           |
| Ice                               | 0.36                                |
| Concrete                          | 0.15                                |
| Steel                             | 0.33                                |

Table 4-4 Soil Properties

| Soil Layer & Parameter                                         | Layer-1 | Layer-2 | Layer-3 | Layer-4     | Layer-5     | Layer-6 | Layer-7 | Layer-8  |
|----------------------------------------------------------------|---------|---------|---------|-------------|-------------|---------|---------|----------|
| Layer type                                                     | SM      | SM      | SM      | CL          | CL          | SM      | SM      | SM       |
| Drainage type                                                  | Drained | Drained | Drained | Undrained B | Undrained B | Drained | Drained | Drained  |
| Depth (m)                                                      | 0-5     | 5-10    | 10-15   | 15-20       | 20-25       | 25-30   | 30-40   | Below 40 |
| Unsaturated unit weight, $\gamma_{unsat}$ (kN/m <sup>3</sup> ) | 17      | 17      | 18      | 19          | 19          | 20      | 20      | 20       |
| Saturated unit weight, $\gamma_{sat}$ (kN/m <sup>3</sup> )     | 18      | 18      | 19      | 20          | 20          | 21      | 21      | 21       |
| Young's modulus, $E'_{ref}$ (kPa)                              | 8000    | 12000   | 20000   | 20000       | 29000       | 45000   | 50000   | 60000    |
| Poisson's ratio, $\nu$ (nu)                                    | 0.38    | 0.38    | 0.38    | 0.35        | 0.35        | 0.38    | 0.38    | 0.38     |
| Friction angle, $\phi'$ (phi)(°)                               | 30      | 30      | 32      | -           | -           | 34      | 35      | 35       |
| Cohesion, $c'_{ref}$ (kPa)                                     | 0       | 0       | 0       | -           | -           | 0       | 0       | 0        |
| Undrained Shear Strength, $s_u$ (kPa)                          | -       | -       | -       | 90          | 130         | -       | -       | -        |

#### 4.3 Tunnel and Lining Parameters

Tunnel is having an internal diameter of 6.5 m which is bigger than the other metro tunnels due to the bigger rolling stock and a high design speed of 180 kmph. Depth of Tunnel axis is varied approximately from a minimum of 8 meters at

the Anand Vihar Station to approximately 25 meters in other sections. Tunnel lining is assumed to be made of 300 mm thick M50 grade concrete segments. Table 4-5 shows material properties of TBM and lining.

Table 4-5 Material properties of TBM and lining

| Material parameters                            | Lining             | TBM               |
|------------------------------------------------|--------------------|-------------------|
| Material Type                                  | Elastic            | Elastic           |
| Unit Weight, $w$ (kN/m <sup>2</sup> )          | 7.5                | 19.5              |
| Axial Stiffness, $EA$ (kN/m)                   | $10.6 \times 10^6$ | $50 \times 10^6$  |
| Bending Stiffness, $EI$ (kN-m <sup>2</sup> /m) | $79.5 \times 10^3$ | $260 \times 10^3$ |
| Poisson's ratio, $\nu$ (nu)                    | 0.15               | 0.3               |

## CHAPTER 5

### NUMERICAL MODELLING AND VALIDATION

The Finite Element Method (FEM) is widely employed in geotechnical studies to investigate soil behaviour. Computer programs based on FEM have been developed and are extensively used as powerful tools for solving complex geotechnical problems (Potts and Zdravković, 2001). FEM is a numerical technique for obtaining approximate solutions to boundary value problems governed by partial differential equations (Zienkiewicz Taylor, 2000). The fundamental concept of FEM is that the numerical model is divided into non-overlapping elements of simple geometry, referred to as finite elements. These elements are interconnected at nodes, with each node possessing a finite number of degrees of freedom (Chen, 2011). Fig. 5.1 presents the fundamental workflow of the finite element modelling process, in which a real-world problem is first idealized into a simplified physical model, subsequently expressed as a mathematical model, and finally discretized into a mesh model suitable for numerical analysis. In this study, numerical simulations have been performed using the finite element program PLAXIS 2D to estimate the tunneling-induced ground deformations.

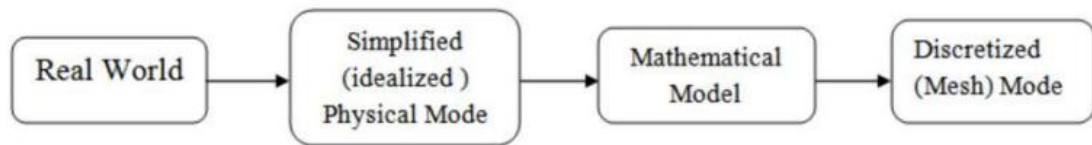


Fig. 5.1 Schematic representation of the Finite Element Method modelling process

#### 5.1 Material Models used in simulation

A material model gives a relationship between stress and strain in the form of mathematical equations.

##### 5.1.1 Linear elastic model

The linear elastic model well represents the engineering materials up to their elastic limit. This model is based on Hook's law of isotropic elasticity. It involves two basic elastic parameters, i.e. Young's modulus  $E$  and Poisson's ratio  $\nu$ . It also represents the complete stress-strain response up to the point of fracture of materials. Linear elastic model is suitable to model stiff volumes like concrete walls. The response of a linear elastic material, where the stress is proportional to the strain, is shown in Fig. 5.2.

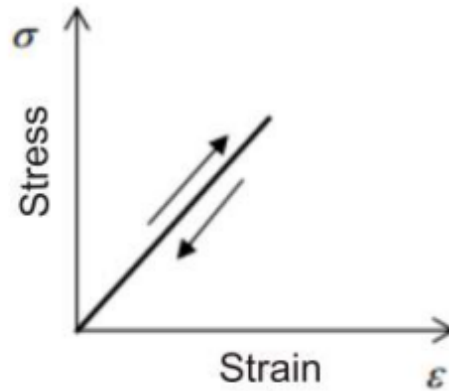


Fig. 5.2 Linear elastic model

### 5.1.2 Mohr-Coulomb model

Mohr-Coulomb model is an elastic-perfectly plastic model which is often used to model soil behaviour in general. The basic principle of elasto-plasticity is that strains and strain rates are decomposed into an elastic part and a plastic part (Fig. 5.3). Stress-strain curve behaves linearly in the elastic range, this behaviour can be defined by two parameters from Hooke's law. These parameters are Young's modulus ( $E$ ) and Poisson's ratio ( $\nu$ ). Parameters which define the failure criteria are the friction angle ( $\phi$ ), cohesion ( $c$ ) and dilatancy angle ( $\psi$ ).

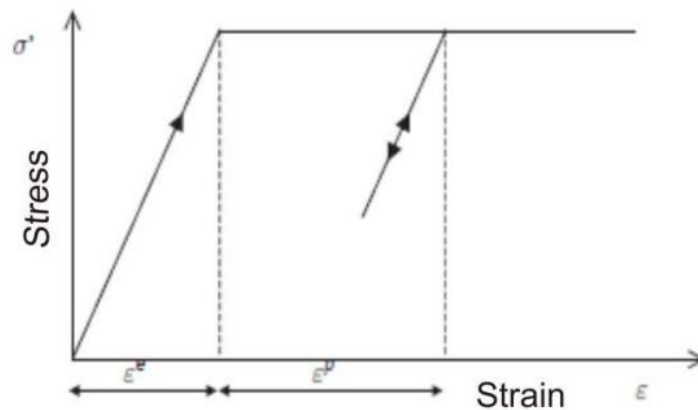


Fig. 5.3 Elastic-perfectly plastic model (PLAXIS 2D Material Models Manual, 2013)

In general, for drained condition the friction angle ( $\phi'$ ) is used to model the effective friction of the soil, in combination with an effective cohesion ( $c'$ ). Alternatively, the soil strength is modelled by setting the cohesion parameter equal to the undrained shear strength of the soil, in combination with  $\phi = 0$ . Fig. 5.4 represents Mohr-Coulomb failure criterion for soils, shown using Mohr's circles.

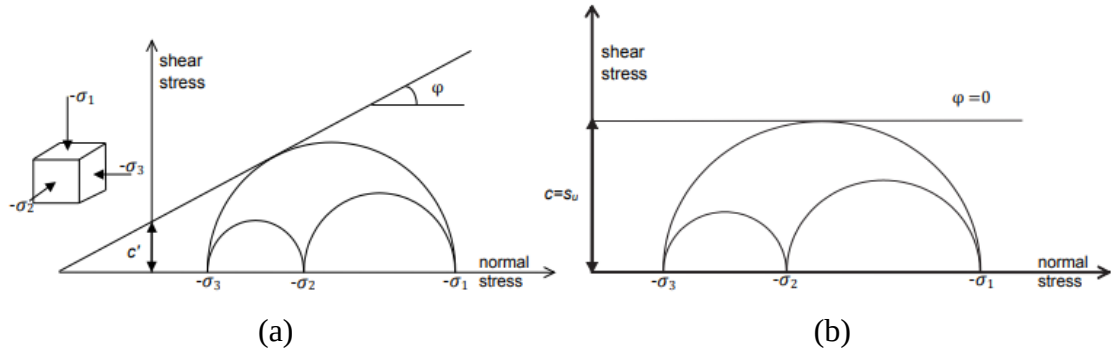


Fig. 5.4 Stress circles at yield; one touches Coulomb's envelope. (a) Using effective strength parameters; (b) Using undrained strength parameters (PLAXIS 2D Material Models Manual, 2022)

## 5.2 Numerical modelling

A 2D plain strain 15-noded element model is created in PLAXIS 2D, with dimensions set at 10 times the diameter of the tunnel in all directions to minimize boundary effects. Previous numerical studies have adopted model boundaries ranging from approximately 4D to 15D horizontally and 3D to 8D vertically, depending on tunnel geometry, tunnel depth, and ground conditions (Aswathy and Mittal, 2022; Golpasand et al., 2016; Möller and Vermeer, 2008; Zheng et al., 2015). The model incorporates standard fixities as per PLAXIS 2D Tutorial and Reference Manuals. In this study, soil layers are modelled using the Elastoplastic soil constitutive Mohr–Coulomb model, with soil properties summarized in Table 4-4. The TBM and tunnel lining are simulated using plate elements with material properties outlined in Table 4-5. Fig. 5.5 and Fig. 5.6 shows the model geometry and meshing respectively for FE analysis with tunnel diameter ( $D$ ) of 6.5 m, axial depth ( $z$ ) of 15 m, and volume loss ( $V_L$ ) of 1.5%. The adopted volume loss ( $V_L = 1.5\%$ ) is selected considering the range commonly reported for EPB-TBM tunnelling in soft ground and alluvial deposits. Previous studies indicate that most commonly reported values of volume loss falling within the range of about 0.2%–2.0% for well-controlled mechanized tunnelling (Mair et al., 1996; Mair and Taylor, 1997; Möller and Vermeer, 2008; O'Reilly and New, 1982; Sarkar et al., 2021). Therefore, a representative intermediate value of 1.5% is adopted to reflect practical EPB-TBM tunnelling conditions in mixed alluvial ground while remaining reasonably conservative for settlement and damage assessment. Furthermore, the adopted value produced realistic surface settlement behaviour consistent with expected tunnelling performance.

The model is discretized into triangular finite elements. A finer mesh is applied in the vicinity of the tunnel to accurately capture the stress and strain variations in this critical zone. Away from the tunnel, a relatively coarser mesh is adopted to minimize computational demand while still accounting for boundary effects. For simulating the interaction between ground and TBM as well as ground and linings interface elements with strength reduction factor ( $R_{\text{inter}} = 0.67$ ) for all the soil layers are provided. The water table is assigned to produce the hydrostatic initial pore water

pressure and phreatic analysis has been done with the assumption of no groundwater flow during excavation and lining is impermeable.

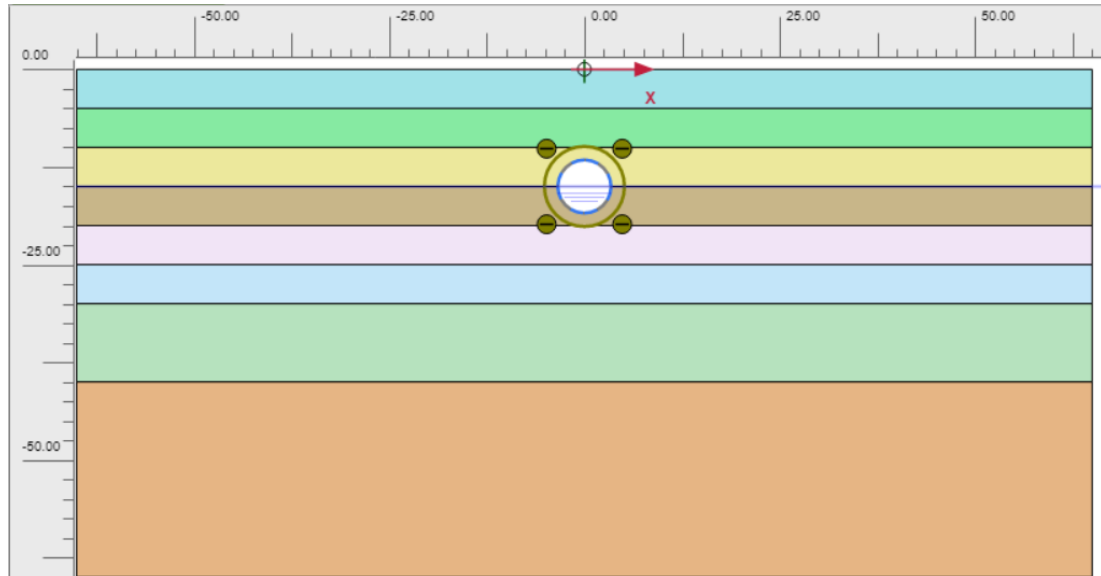


Fig. 5.5 PLAXIS 2D model of existing ground conditions with tunnel

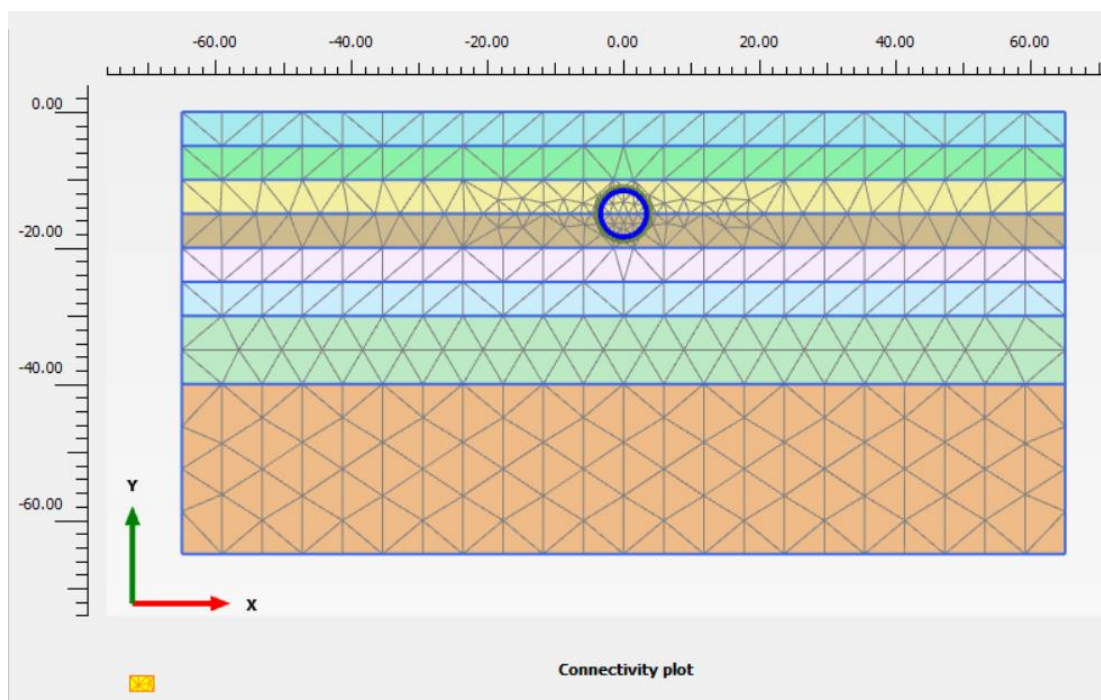


Fig. 5.6 PLAXIS 2D model after meshing

The process of tunnel construction has been modelled through step-by-step method in 5 phases consists of: (i) Initial phase includes setting up of initial conditions (effective stresses and pore pressures) for the model before excavation with K0

procedure calculation type, (ii) TBM phase includes the excavation of soil and activation of TBM, (iii) TBM Conicity phase includes contraction of tunnel lining simulation due to the volume loss resulted from the cone shape of TBM, (iv) Tail void grouting phase includes the simulation of application of grout pressure at the tail of the TBM. The grouting process is simulated by applying a pressure on the surrounding soil, (v) Lining installation phase includes the activation of final lining (Agarwal and Sarkar, 2024).

### 5.3 Results and Validation

Deformed mesh, total deformations, total vertical displacements and total horizontal displacements after calculations are shown in Fig. 5.7, Fig. 5.8, Fig. 5.9 and Fig. 5.10 respectively. The maximum displacements are observed directly above the tunnel crown, propagating upward towards the ground surface in a funnel-shaped parabolic form, consistent with observations reported by Ahmed and Iskander (2011), Lei-Kilany et al. (2017) and Naqvi et al. (2021). This deformation pattern is characteristic of tunneling-induced settlement troughs, where the soil mass above the tunnel tends to move downward and inward due to stress relief during excavation. In contrast, displacements extending below the tunnel are relatively smaller in magnitude.

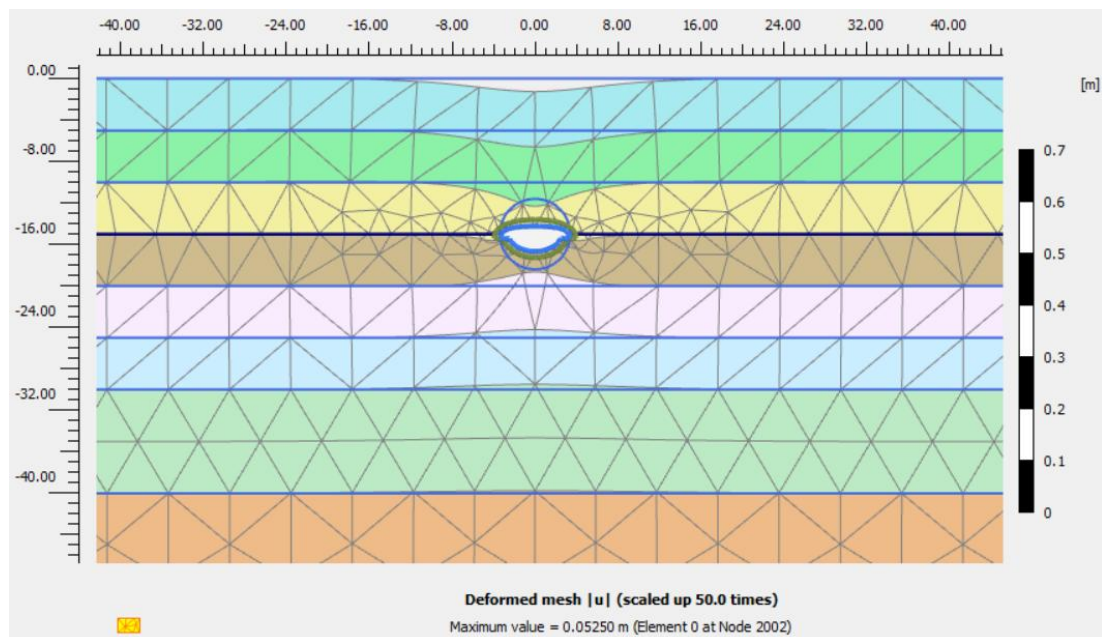


Fig. 5.7 PLAXIS Output - Deformed Mesh after Calculations

In current study numerical model is verified and validated with the help of empirical methods, since numerous researchers have reported that the surface settlement profile induced by tunneling follows a Gaussian curve (Camós et al., 2014; Rebello et al., 2014). This verification and validation are carried out to ensure the

reliability and accuracy of the numerical model, thereby increasing confidence in its ability to realistically simulate tunneling-induced ground deformations. Greenfield ground surface deformations curves obtained by numerical modelling are compared with those obtained from empirical methods available in literature and discussed in Chapter 2. Fig. 5.11 and Fig. 5.12 represent the PLAXIS output for vertical ground surface settlements and horizontal ground surface displacement respectively.

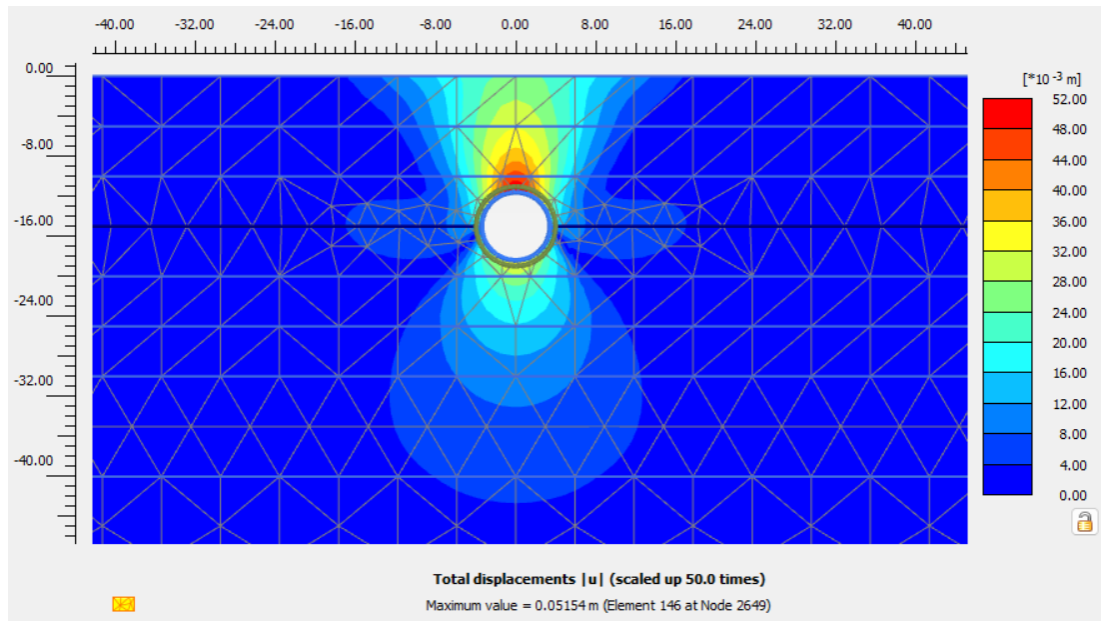


Fig. 5.8 PLAXIS Output – Total deformations

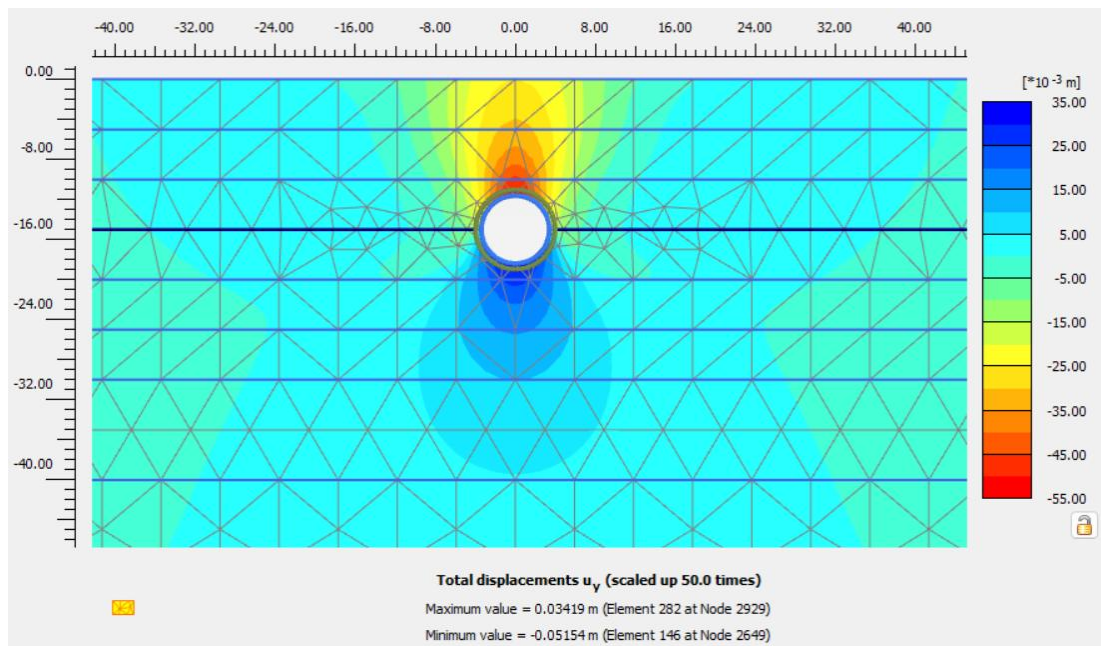


Fig. 5.9 PLAXIS Output –Total Vertical Displacements

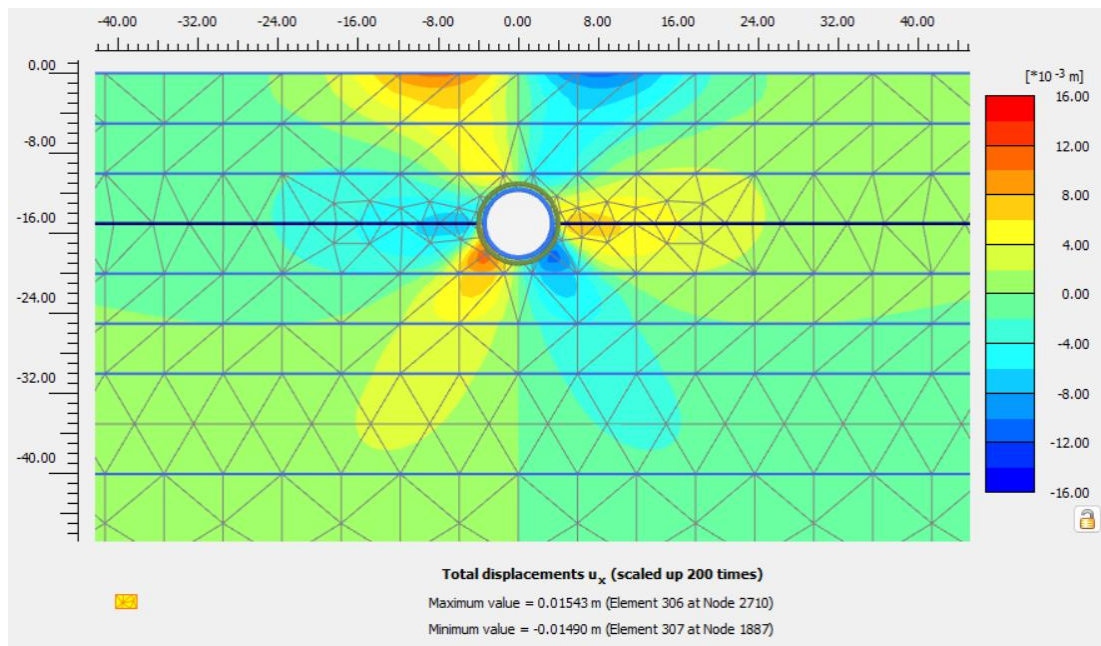


Fig. 5.10 PLAXIS Output –Total Horizontal Displacements

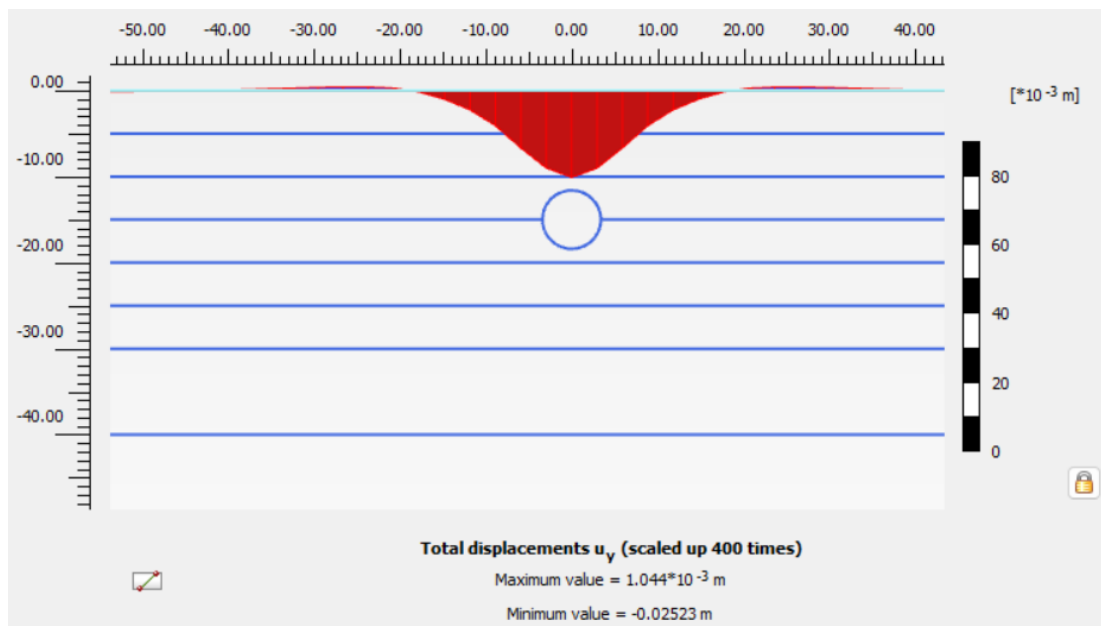


Fig. 5.11 PLAXIS Output –Vertical ground surface settlement

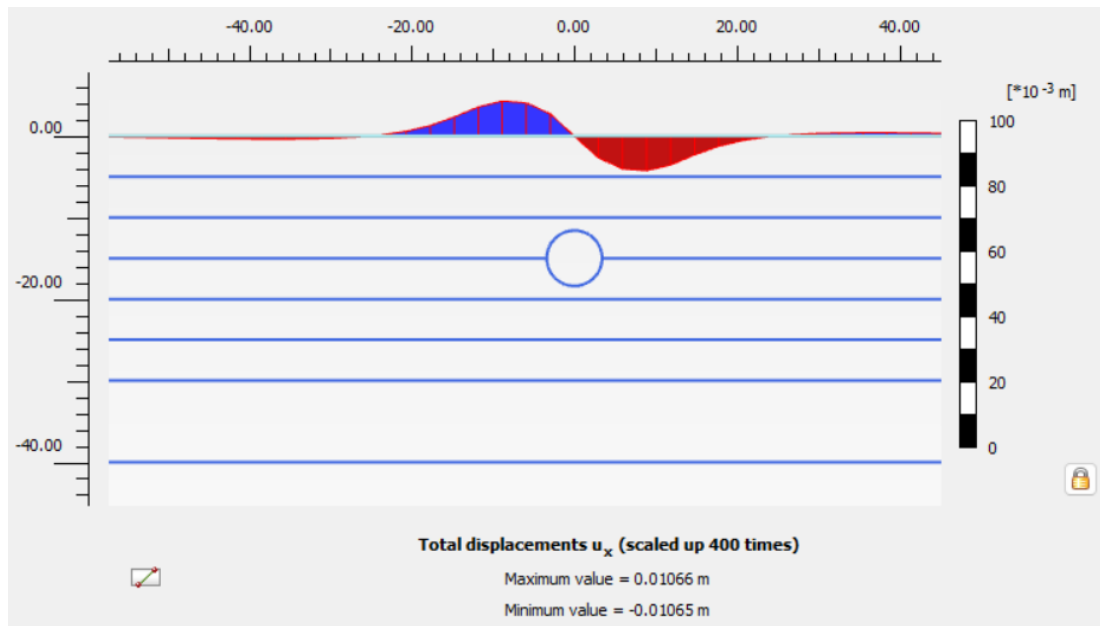


Fig. 5.12 PLAXIS Output – Horizontal ground surface displacement

Fig. 5.13 and Fig. 5.14 represent a comparison between vertical ground surface settlement and horizontal ground surface displacement profiles due to the excavation of the proposed tunnel, analysed using both numerical and empirical methods. By idealizing the settlement trough profiles obtained from PLAXIS 2D as Gaussian curves, the settlement trough parameters were derived, with  $i$  values of 7.92 corresponding to  $k$ -value of 0.528. Since the subsurface profile consists of alternating SM and CL layers, a single soil-type-dependent  $k$ -value could not be selected directly from literature. Therefore, an effective trough width parameter was obtained through back-analysis of the settlement trough derived from PLAXIS 2D by fitting a Gaussian distribution to the computed greenfield settlement profile. The resulting  $k$ -value of 0.528 represents the combined influence of mixed soil strata and lies within the acceptable range reported for tunnelling in transitional ground conditions. Using these parameters, Gaussian and modified distribution curves are superimposed on the ground surface deformation profiles from PLAXIS 2D, enabling a comparison with empirical methods. The results show good agreement in the overall settlement profile shapes; however, the finite element model predicts a narrower settlement trough. This discrepancy can be attributed to the influence of the layered soil profile below the water table (Lee, 2002; Ghahremannejad et al. 2006) as well as the rigidity of the tunnel lining, the initial stress state, and soil stiffness (Mathew and Lehane, 2013; Zhao et al., 2019). The obtained settlement trough volume at the ground surface, approximately 1%, does not correspond exactly to the imposed tunnel volume loss of about 1.5%, a trend also reported by previous researchers (He et al., 2021). The difference between the imposed tunnel volume loss (1.5%) and the resulting surface settlement trough volume is primarily attributed to the layered ground conditions and progressive increase in soil stiffness with depth. The stiffer and denser strata surrounding the tunnel provide greater confinement and redistribute part of the deformation within the subsurface, resulting in attenuation of settlement before it reaches the ground surface. Minor soil heave was observed, likely caused by

dissipation of small negative excess pore pressures in clay layers and the assumption of equal loading and unloading stiffness moduli in the Mohr-Coulomb model (Obrzud, 2010). Furthermore, higher horizontal displacements predicted by PLAXIS 2D compared with empirical methods provide a more conservative basis for tensile strain assessment.

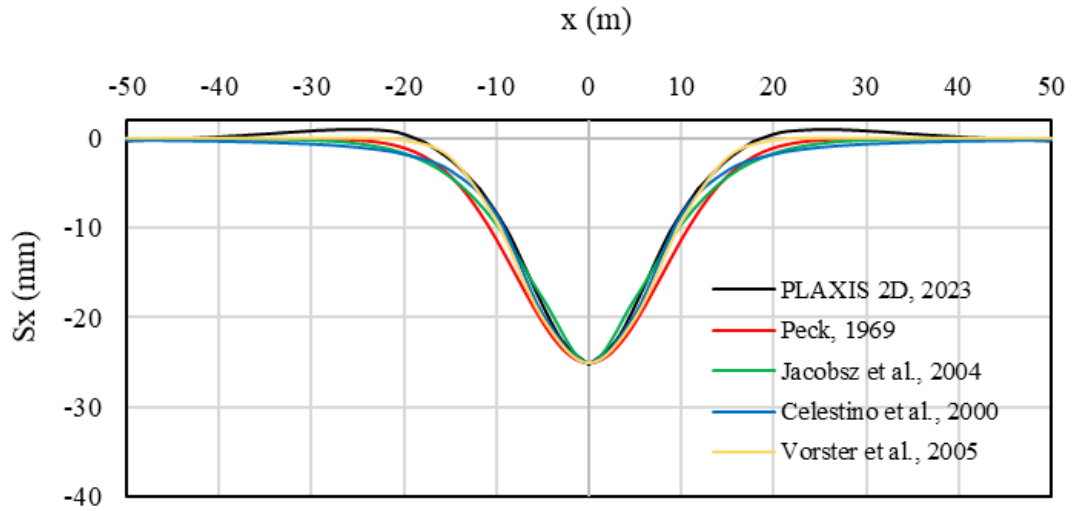


Fig. 5.13 Comparison of vertical ground surface settlements obtained from PLAXIS 2D and empirical methods

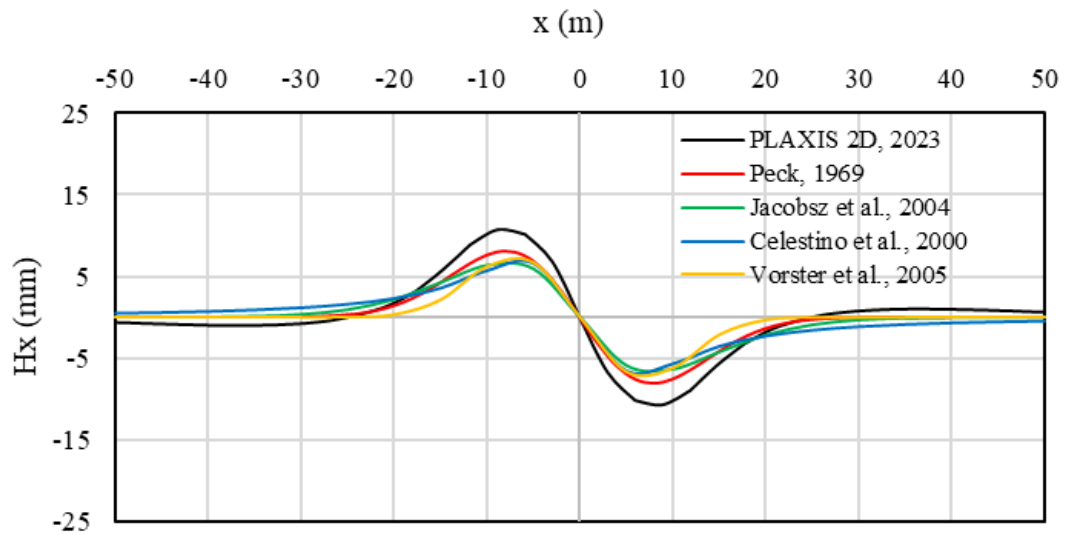


Fig. 5.14 Comparison of Horizontal ground surface displacement obtained from PLAXIS 2D and empirical methods

## CHAPTER 6

### PARAMETRIC STUDY – RESULTS AND DISCUSSION

Parametric studies are essential in tunneling research to systematically evaluate the influence of key parameters on ground deformations and structural responses. Tunneling-induced ground deformations under greenfield conditions are influenced by tunnel parameters, soil properties, and construction effects, with a complex nonlinear relationship existing between these factors (Huat et al., 2024; Leca and New, 2007). In the present study, a series of numerical analyses are conducted to investigate the effects of soil and tunnel parameters on tunneling-induced ground deformations and the subsequent response of surface structures. The chapter is organized into two main parts. The first part focuses on greenfield conditions, where the influence of soil and tunnel parameters on ground surface settlement ( $S_x$ ) is examined in the absence of surface structures. The second part extends the investigation to assess how these parameters influence building deformation measures, induced tensile strains, and potential damage categories. In addition, the effects of structural parameters on these response measures are also analysed. The results are presented in the form of ground surface deformations profiles and structural response measures followed by detailed discussions highlighting the observed trends and their implications for design and risk assessment.

#### 6.1 Greenfield Settlements: Parametric Study

In this study, both tunnel and soil parameters are varied within representative ranges based on practical tunneling conditions, with the aim of assessing the sensitivity of ground surface settlements under greenfield conditions to changes in these influencing parameters. The tunnel parameters considered for analysing their effect on ground surface settlements include tunnel diameter ( $D$ ), tunnel axis depth ( $z$ ), and Volume loss ( $V_L$ ). The soil parameters examined are the undrained shear strength ( $s_u$ ) and the young's modulus of soil ( $E'_{ref}$ ) corresponding to the strata through which the tunnel is excavated. The influence of each of these parameters on tunneling-induced ground surface settlements is evaluated.

Table 6-1 presents the range of parameters investigated in the present study. The rationale behind selecting these ranges was to gain a comprehensive understanding of how variations in these parameters influence the magnitude and distribution of surface settlements due to tunneling operations. While determining the effect of one parameter, the other parameters are kept equal to the base values as given in Table 6-1.

Table 6-1 Range of parameters variation for Greenfield settlements: parametric study

| Parameters                            | Range                           | Increment       | Base Value                                                                     |
|---------------------------------------|---------------------------------|-----------------|--------------------------------------------------------------------------------|
| Tunnel Diameter, $D$ (m)              | 5.5 - 8.5                       | 1               | 6.5                                                                            |
| Tunnel Depth, $z$ (m)                 | 10 - 30                         | 5               | 20                                                                             |
| Volume Loss, $V_L$ (%)                | 0.5 - 2                         | 0.5             | 1.5                                                                            |
| Undrained Shear Strength, $s_u$ (kPa) | $0.75 s_u - 1.75 s_u$           | $0.25 s_u$      | $s_u$ (90 & 130 for layer 4 & 5 respectively)                                  |
| Young's modulus, $E'_{ref}$ (kPa)     | $0.75 E'_{ref} - 1.75 E'_{ref}$ | $0.25 E'_{ref}$ | $E'_{ref}$ ( $20 \times 10^3$ & $29 \times 10^3$ for layer 4 & 5 respectively) |

### 6.1.1 Effect of Tunnel Parameters

While analysing the influence of tunnel parameters, the soil parameters are kept constant at their respective base values, as presented in Table 6-1. The effects of tunnel diameter ( $D$ ) and tunnel axis depth ( $z$ ) are investigated by maintaining a constant volume loss ( $V_L$ ) of 1.5%. Conversely, the effect of volume loss is examined by fixing the tunnel diameter at 6.5 m and the tunnel axis depth at 20 m below the ground surface. A detailed discussion on the influence of individual tunnel parameters is presented in the following sections.

#### 6.1.1.1 Effect of Tunnel Diameter ( $D$ )

One of the most influential factors governing tunneling-induced ground surface settlements is the tunnel size (Aswathy et al., 2022). To investigate the effect of  $D$  on  $S_x$ , four cases are analysed by varying  $D$  from 5.5 m to 8.5 m with an increment of 1 m, while keeping the tunnel depth constant at 20 m below the ground level. The surface settlement troughs presented in Fig. 6.1 indicate that an increase in  $D$  leads to a significant rise in the magnitude of  $S_x$ , with the settlement trough becoming deeper and sharper. This trend is further reflected in Fig. 6.2, which shows that the  $S_{max}$  increases consistently with increasing  $D$ , in agreement with earlier studies (Aswathy et al., 2022; Kanagaraju and Krishnamurthy, 2020; Shiau and Sams, 2019).

Table 6-2 summarises  $S_{max}$  along with its increment rates and percentage increase with  $D$ . The results indicate that while the absolute increment in settlement increases with larger tunnel sizes, the relative percentage increment reduces progressively as the tunnel diameter increases. A total increase of 112.4% in  $S_{max}$  was observed as the  $D$  increased from 5.5 m to 8.5 m. Additionally, the grout pressure (GP) required to maintain the  $V_L$  at 1.5% decreases with larger tunnel diameters (Fig. 6.1). This indicates that less GP is needed to sustain equal  $V_L$  for larger tunnels, which in turn results in higher settlement magnitudes.

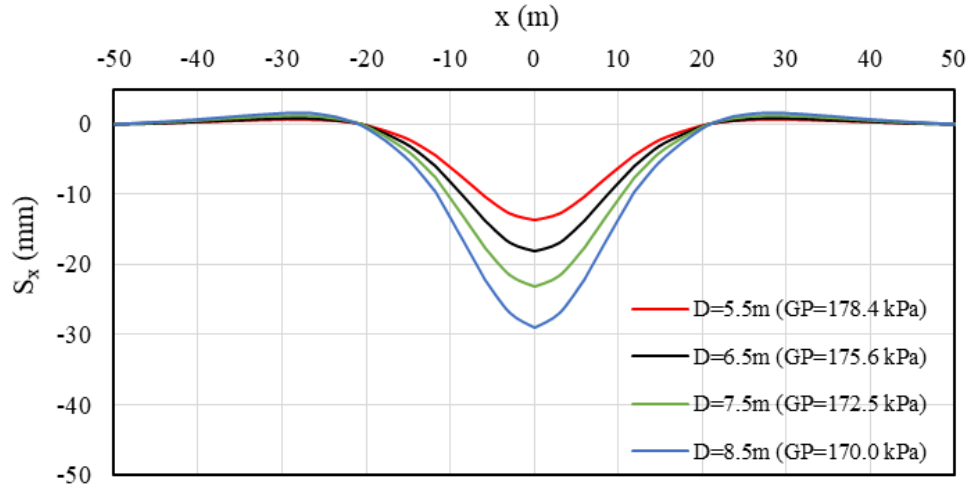


Fig. 6.1 Variation of Surface Settlement Profiles with  $D$

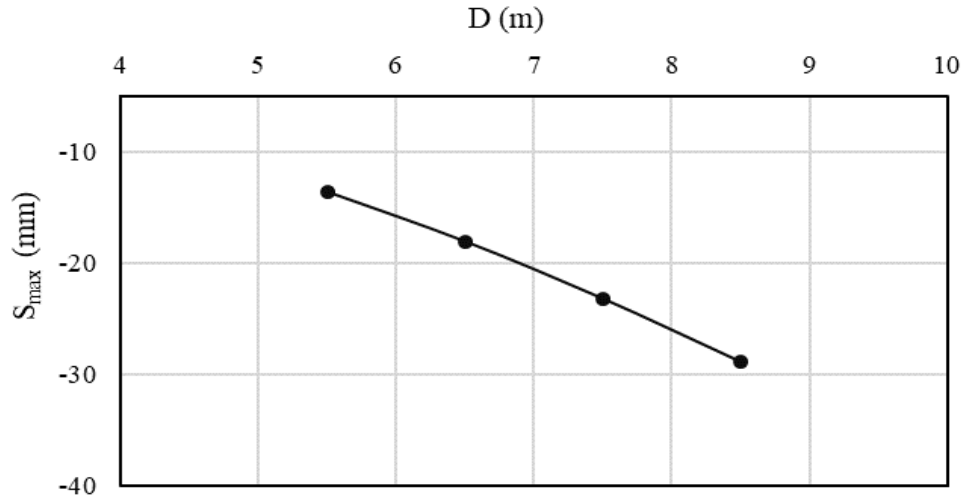


Fig. 6.2 Variation of  $S_{max}$  with  $D$

Table 6-2 Relative change in  $S_{max}$  with  $D$

| Tunnel Diameter<br>( $D$ , m) | Maximum Surface<br>Settlement ( $S_{max}$ , mm) | Rate of increment<br>(mm/m) | % Increase<br>in $S_{max}$ |
|-------------------------------|-------------------------------------------------|-----------------------------|----------------------------|
| 5.5                           | 13.62                                           | -                           | -                          |
| 6.5                           | 18.08                                           | 4.46                        | 32.75%                     |
| 7.5                           | 23.23                                           | 5.15                        | 28.49%                     |
| 8.5                           | 28.93                                           | 5.7                         | 24.53%                     |

#### 6.1.1.2 Effect of Tunnel Axial Depth ( $z$ )

To investigate the effect of  $z$  on  $S_x$ , five cases are analysed by varying  $z$  from 10 m to 30 m with an increment of 5 m, while keeping the tunnel diameter as 6.5

m. The surface settlement troughs presented in Fig. 6.3 indicate that an increase in  $z$  leads to a significant decrease in the magnitude of  $S_x$  near tunnel centreline. This trend is further reflected in Fig. 6.4, which shows that the  $S_{max}$  decreases consistently with increasing  $z$ , in agreement with earlier studies (Ahmed and Iskander, 2011; Golpasand et al., 2016; Mirhabibi and Soroush, 2020; Sarkar et al., 2021; Zhang et al., 2016). However, at locations farther away from the centreline, the opposite trend is observed, with settlements increasing slightly at greater depths. Furthermore, the settlement trough widens with increasing tunnel depth, which is consistent with observations reported in previous studies (Franza et al., 2019; Marshall et al., 2012). The flattening of the settlement trough with depth indicates a more distributed settlement pattern at the ground surface. Table 6-3 summarises  $S_{max}$  along with its decrement rates and percentage decrease with  $z$ .

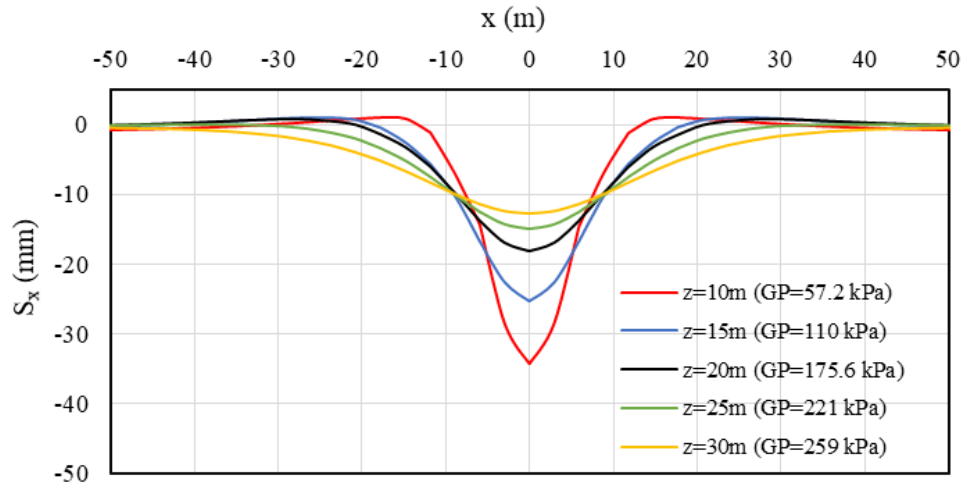


Fig. 6.3 Variation of Surface Settlement Profiles with  $z$

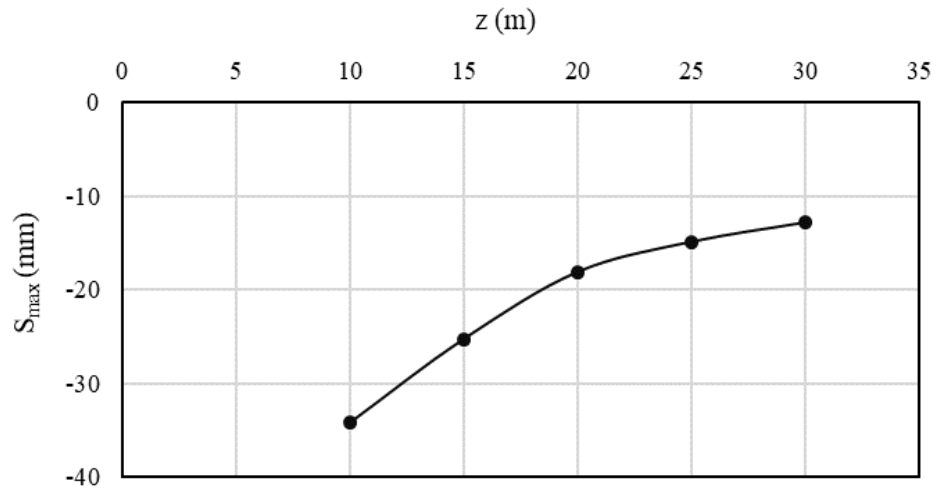


Fig. 6.4 Variation of  $S_{max}$  with  $z$

Table 6-3 Relative change in  $S_{max}$  with  $z$

| Tunnel Depth<br>( $z$ , m) | Maximum Surface<br>Settlement ( $S_{max}$ , mm) | Rate of decrement<br>(mm/m) | % Decrease in<br>$S_{max}$ |
|----------------------------|-------------------------------------------------|-----------------------------|----------------------------|
| 10                         | 34.18                                           | -                           | -                          |
| 15                         | 25.24                                           | 1.8                         | 26.15                      |
| 20                         | 18.08                                           | 1.4                         | 28.38                      |
| 25                         | 14.87                                           | 0.6                         | 17.76                      |
| 30                         | 12.8                                            | 0.4                         | 13.91                      |

The results indicate that both absolute and relative percentage decrement in settlement decreases with increase in depth of tunnel axis. A total decrease of 62.55% in  $S_{max}$  was observed as the  $z$  increased from 10 m to 30 m. Additionally, the GP required to maintain the  $V_L$  at 1.5% increases with larger axial tunnel depth (Fig. 6.3). This indicates that more GP is needed to sustain equal  $V_L$  for tunnels situated at larger depths, which in turn results in lower settlement magnitudes.

#### 6.1.1.3 Effect of Volume Loss ( $V_L$ )

To investigate the effect of  $V_L$  on  $S_x$ , four cases are analysed by varying  $V_L$  from 0.5% to 2% with an increment of 0.5%. The surface settlement troughs presented in Fig. 6.5 indicate that an increase in  $V_L$  leads to a significant increase in the magnitude of  $S_x$ . This trend is further reflected in Fig. 6.6, which shows that the  $S_{max}$  increases consistently with increasing  $V_L$ , in agreement with earlier studies (Ahmed and Iskander, 2011; Golpasand et al., 2016; Kanagaraju and Krishnamurthy, 2020; Lei-Kilany et al., 2017; Miliziano and Lillis, 2019; Shiau and Sams, 2019).

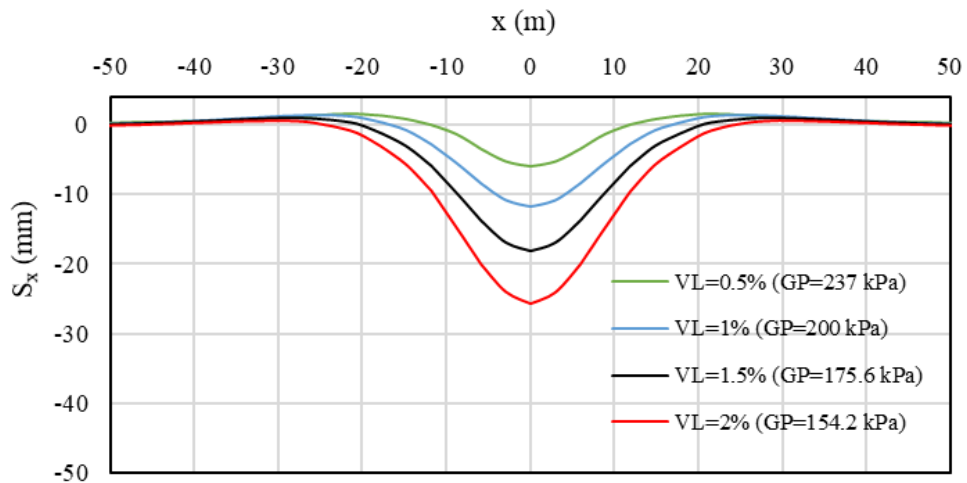


Fig. 6.5 Variation of Surface Settlement Profiles with  $V_L$

Table 6-4 summarises  $S_{max}$  along with its increment rates and percentage increase with  $V_L$ . The results indicate that while the absolute increment in settlement

increases with larger  $V_L$ , the relative percentage increment reduces progressively as the  $V_L$  increases. A total increase of 321% in  $S_{max}$  was observed as the  $V_L$  increased from 0.5 % to 2%. Furthermore, the ground pressure (GP) required to achieve  $V_L$  values of 0.5%, 1.0%, 1.5%, and 2.0% was found to be 237 kPa, 200 kPa, 175.6 kPa, and 154.2 kPa, respectively (Fig. 6.5). This trend indicates that higher grout pressure results in lower volume loss, which is consistent with the observations reported by other researchers (Aswathy et al., 2022; Fagnoli et al., 2013; Möller and Vermeer, 2008).

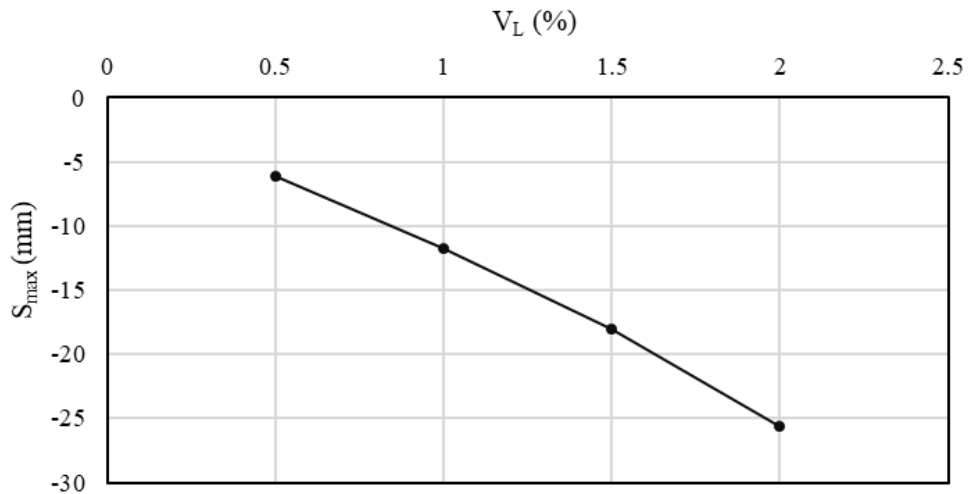


Fig. 6.6 Variation of  $S_{max}$  with  $V_L$

Table 6-4 Relative change in  $S_{max}$  with  $V_L$

| Volume Loss<br>( $V_L$ , %) | Maximum Surface<br>Settlement ( $S_{max}$ , mm) | Rate of increment<br>(mm/0.5%) | % Increase in<br>$S_{max}$ |
|-----------------------------|-------------------------------------------------|--------------------------------|----------------------------|
| 0.5                         | 6.08                                            | -                              | -                          |
| 1                           | 11.77                                           | 5.69                           | 93.58                      |
| 1.5                         | 18.08                                           | 6.31                           | 53.61                      |
| 2                           | 25.6                                            | 7.52                           | 41.59                      |

### 6.1.2 Effect of Soil Parameters

While analysing the influence of soil parameters, the tunnel parameters are kept constant at their respective base values, as presented in Table 6-1. For the parametric study, the properties of the soil layers through which the tunnel passes (i.e., Layers 4 and 5) are varied within a practical range. The effects of soil parameters are examined under two conditions: (i) by maintaining the volume loss ( $V_L$ ) at 1.5%, and (ii) by maintaining the ground pressure (GP) at 175.6 kPa. A detailed discussion on the influence of individual soil parameters under both conditions is presented in the following sections.

### 6.1.2.1 Effect of Undrained Shear Strength of Soil ( $s_u$ )

To investigate the effect of undrained shear strength ( $s_u$ ) on surface settlements, five cases are analysed by varying the  $s_u$  for layers 4 and 5 from  $0.75 s_u$  to  $1.75 s_u$ , with an increment of  $0.25 s_u$ , relative to the base values presented in Table 6-1. The surface settlement troughs presented in Fig. 6.7 and Fig. 6.8 indicate that an increase in  $s_u$  leads to a significant decrease in the magnitude of  $S_x$  under constant  $V_L$  and constant GP conditions respectively. This trend is further reflected in Fig. 6.9 and Fig. 6.10, which shows that the  $S_{max}$  decreases consistently with increasing  $s_u$  under constant  $V_L$  and constant GP conditions respectively. These findings are consistent with the observations reported by other researchers (Kanagaraju and Krishnamurthy, 2020; Rebello et al., 2014; Sarkar et al., 2021; Shiau and Sams, 2019). However, under constant  $V_L$  the reduction in  $S_{max}$  is mild and nearly linear, whereas under constant GP the reduction is much sharper and nonlinear.

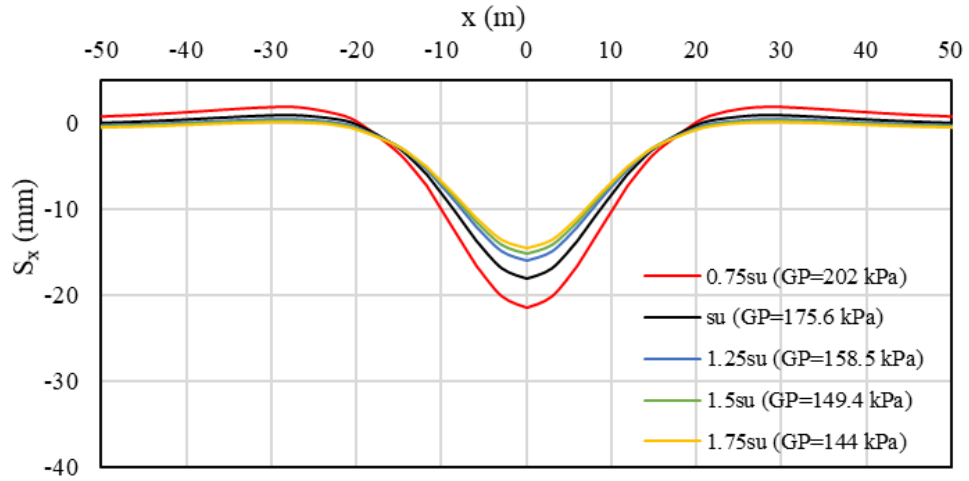


Fig. 6.7 Variation of Surface Settlement Profiles with  $s_u$  under constant  $V_L$

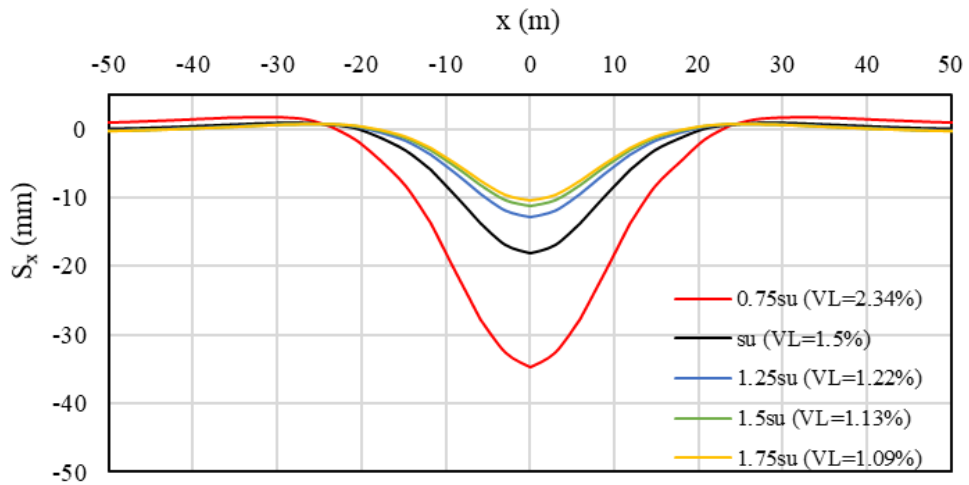


Fig. 6.8 Variation of Surface Settlement Profiles with  $s_u$  under constant GP

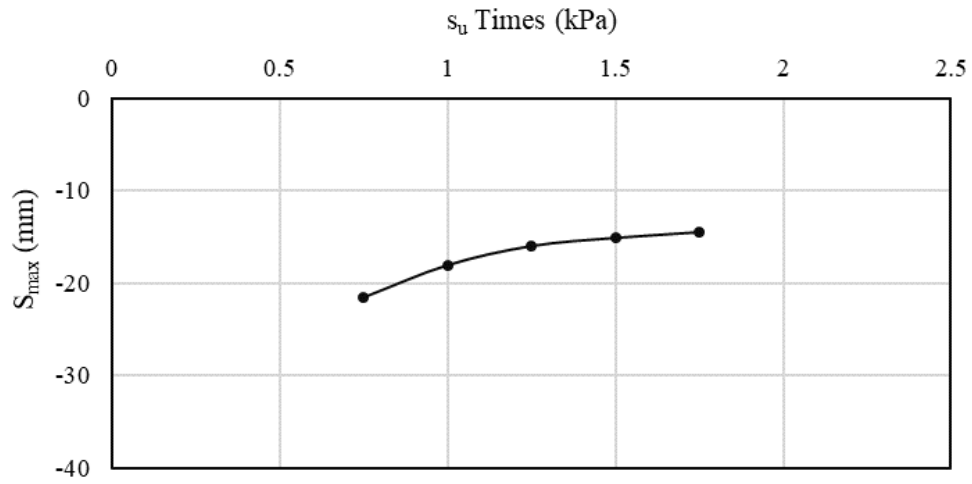


Fig. 6.9 Variation of  $S_{max}$  with  $s_u$  under constant  $V_L$

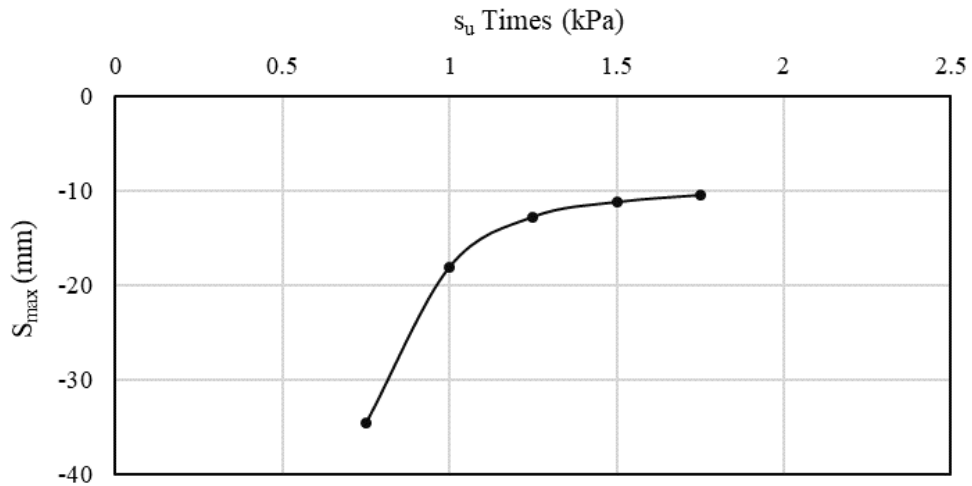


Fig. 6.10 Variation of  $S_{max}$  with  $s_u$  under constant GP

Table 6-5 and Table 6-6 summarises  $S_{max}$  along with its decrement rates and percentage decrease with  $s_u$  under constant  $V_L$  and constant GP conditions respectively.

Table 6-5 Relative change in  $S_{max}$  with  $s_u$  under constant  $V_L$

| Undrained Shear Strength ( $s_u$ , kPa) | Maximum Surface Settlement ( $S_{max}$ , mm,) | Rate of decrement (mm/0.25 times $s_u$ ) | % Decrease in $S_{max}$ |
|-----------------------------------------|-----------------------------------------------|------------------------------------------|-------------------------|
| 0.75 $s_u$                              | 21.53                                         | -                                        | -                       |
| 1 $s_u$                                 | 18.08                                         | 3.45                                     | 16.02                   |
| 1.25 $s_u$                              | 16.03                                         | 2.05                                     | 11.33                   |
| 1.5 $s_u$                               | 15.16                                         | 0.87                                     | 5.42                    |
| 1.75 $s_u$                              | 14.55                                         | 0.61                                     | 4.02                    |

Table 6-6 Relative change in  $S_{max}$  with  $s_u$  under constant GP

| Undrained Shear Strength ( $s_u$ , kPa) | Maximum Surface Settlement ( $S_{max}$ , mm) | Rate of decrement (mm/0.25 times $s_u$ ) | % Decrease in $S_{max}$ |
|-----------------------------------------|----------------------------------------------|------------------------------------------|-------------------------|
| 0.75 $s_u$                              | 34.64                                        | -                                        | -                       |
| 1 $s_u$                                 | 18.08                                        | 16.56                                    | 47.80                   |
| 1.25 $s_u$                              | 12.78                                        | 5.3                                      | 29.31                   |
| 1.5 $s_u$                               | 11.22                                        | 1.56                                     | 12.20                   |
| 1.75 $s_u$                              | 10.49                                        | 0.73                                     | 6.50                    |

The results indicate that both absolute decrement and relative percentage decrement reduces with larger  $s_u$ , under both constant  $V_L$  and constant GP conditions. A total decrease of 32.41% and 69.71% in  $S_{max}$  was observed under constant  $V_L$  and constant GP conditions respectively, as the  $s_u$  increased from 0.75  $s_u$  to 1.75  $s_u$ .

#### 6.1.2.2 Effect of Soil modulus ( $E'_{ref}$ )

To investigate the effect of soil modulus ( $E'_{ref}$ ) on surface settlements, five cases are analysed by varying the  $E'_{ref}$  for layers 4 and 5 from 0.75  $E'_{ref}$  to 1.75  $E'_{ref}$ , with an increment of 0.25  $E'_{ref}$ , relative to the base values presented in Table 6-1. The surface settlement troughs presented in Fig. 6.11 indicate that an increase in  $E'_{ref}$  results in only a marginal change in the settlement profile, as the curves remain almost overlapping, indicating limited sensitivity to stiffness variation under constant  $V_L$  condition. In contrast, Fig. 6.12 indicate that an increase in  $E'_{ref}$  leads to a significant decrease in the magnitude of  $S_x$  under constant GP conditions, with lower stiffness producing deeper and wider troughs and higher stiffness leading to a reduction in settlement magnitude.

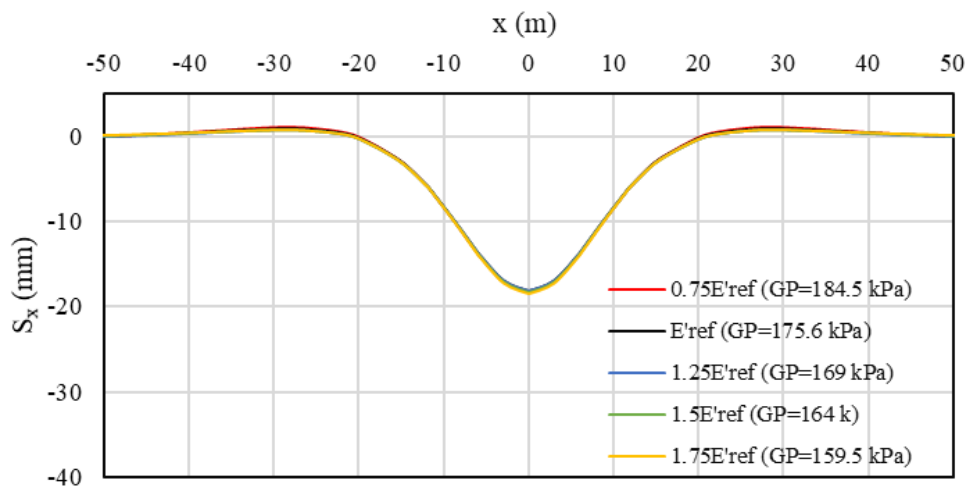


Fig. 6.11 Variation of Surface Settlement Profiles with  $E'_{ref}$  under constant  $V_L$

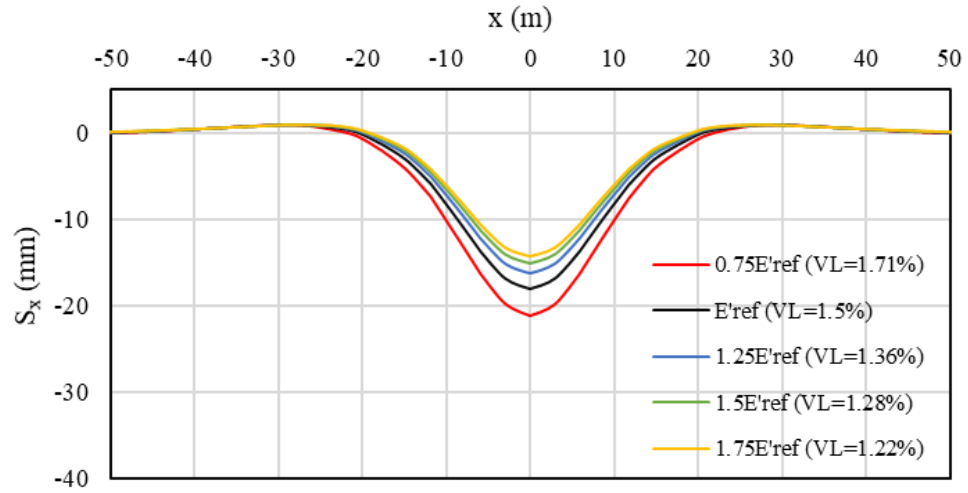


Fig. 6.12 Variation of Surface Settlement Profiles with  $E'_{ref}$  under constant GP

This trend is further reflected in Fig. 6.13, which shows that the  $S_{max}$  decreases consistently with increasing  $E'_{ref}$  under constant GP condition. These findings are consistent with the observations reported by other researchers (Aswathy et al., 2022; Cho et al. 2017; Kanagaraju and Krishnamurthy, 2020; Rebello et al., 2014; Sarkar et al., 2021; Shiau and Sams, 2019).

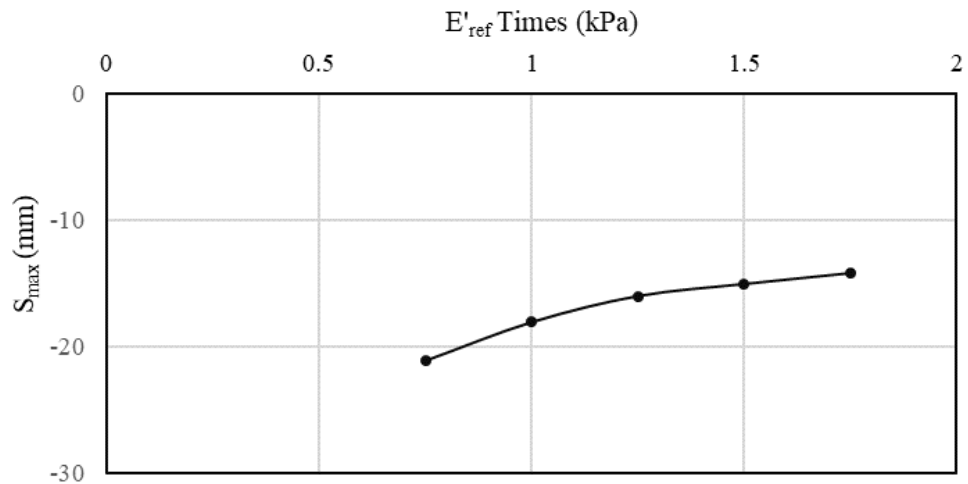


Fig. 6.13 Variation of  $S_{max}$  with  $E'_{ref}$  under constant GP

Table 6-7 summarises  $S_{max}$  along with its decrement rates and percentage decrease with  $E'_{ref}$  under constant GP condition. The results indicate that both absolute decrement and relative percentage decrement reduces with larger  $E'_{ref}$ , under constant GP condition. A total decrease of 32.63% in  $S_{max}$  was observed under constant GP conditions, as the  $E'_{ref}$  increased from  $0.75 E'_{ref}$  to  $1.75 E'_{ref}$ .

Table 6-7 Relative change in  $S_{max}$  with  $E'_{ref}$  under constant GP

| Young's modulus<br>( $E'_{ref}$ , kPa) | Maximum Surface<br>Settlement ( $S_{max}$ , mm) | Rate of decrement<br>(mm/0.25 times $E'_{ref}$ ) | % Decrease in<br>$S_{max}$ |
|----------------------------------------|-------------------------------------------------|--------------------------------------------------|----------------------------|
| $0.75 E'_{ref}$                        | 21.12                                           | -                                                | -                          |
| $1 E'_{ref}$                           | 18.08                                           | 3.04                                             | 14.39                      |
| $1.25 E'_{ref}$                        | 16.03                                           | 2.05                                             | 11.24                      |
| $1.5 E'_{ref}$                         | 15.08                                           | 0.95                                             | 5.93                       |
| $1.75 E'_{ref}$                        | 14.23                                           | 0.85                                             | 5.63                       |

## 6.2 Analytical Assessment of Existing Structures using LSTM

The ground deformations obtained under greenfield conditions using PLAXIS 2D, as presented in Chapter 5, are imposed on a simplified building model following the methodology described in Chapter 3. Fig. 6.14 presents the vertical ground settlements and horizontal displacements obtained from PLAXIS 2D simulations under greenfield conditions, together with the location and dimensions of a representative building. While these deformations profiles form the basis for assessing potential building deformations, it should be acknowledged that the actual magnitudes may be influenced by the presence, geometry, and stiffness of the overlying structures (Comodromos et al., 2009). Depending on the structural type, i.e., masonry or framed, the ratio of longitudinal stiffness to shear stiffness ( $E/G$ ) is assigned to the beam, with values of 2.6 for masonry structures and 12.5 for framed structures, as suggested by Burland and Wroth (1974) and Son and Cording (2007).

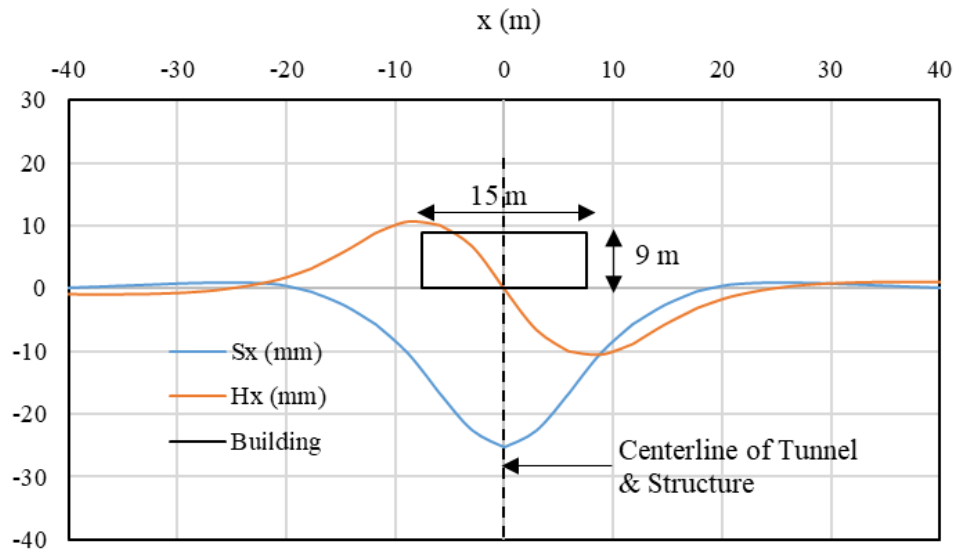


Fig. 6.14 Ground deformations ( $S_x$  and  $H_x$ ) and Building

$\varepsilon_{bmax}$  and  $\varepsilon_{dmax}$  are calculated using Eqn. 3.11 and Eqn. 3.12, respectively. The limiting tensile strain ( $\varepsilon_{lim}$ ) is determined using Eqn. 3.13, Eqn. 3.14

and Eqn. 3.15, while Eqn. 3.16 is employed to calculate  $\beta_{max}$ . Following this damage classification is done on the basis of  $S_{bmax}$ ,  $\beta_{max}$  and  $\varepsilon_{lim}$ . For the study area, buildings with varying dimensions and storeys are considered at different distances from the tunnel axis. A masonry building of 15 m length, 9 m height and located exactly above the tunnel axis is taken as a base reference for parametric study. The same building is analysed as both a masonry and a framed structure, and the corresponding results are summarized in Table 6-8.

Table 6-8 Results of the LTSM application

| Mode Sagging/Hogging                  | Sagging                  | Masonry Building<br>( $E/G = 2.6$ ) | Framed Building<br>( $E/G = 12.5$ ) |
|---------------------------------------|--------------------------|-------------------------------------|-------------------------------------|
| Max Settlement                        | $S_{bmax}$ , mm          | 25.24                               | 25.24                               |
| Slope                                 | $\beta_{max}$            | 0.00161                             | 0.00161                             |
| Maximum horizontal strain             | $\varepsilon_{hmax}$ (%) | 0.1384                              | 0.1384                              |
| Deflection ratio                      | $\Delta/L$ (%)           | 0.0803                              | 0.0803                              |
| Total bending tensile strain          | $\varepsilon_{bt}$ (%)   | 0.2745                              | 0.1836                              |
| Total diagonal tensile strain         | $\varepsilon_{dt}$ (%)   | 0.1529                              | 0.1720                              |
| Limiting Tensile strain               | $\varepsilon_{lim}$ (%)  | 0.2745                              | 0.1836                              |
| Category based on $S_{bmax}$          |                          | 2- Slight                           | 2- Slight                           |
| Category based on $\beta_{max}$       |                          | 1- Negligible                       | 1- Negligible                       |
| Category based on $\varepsilon_{lim}$ |                          | 3- Moderate                         | 3- Moderate                         |

### 6.3 Limiting tensile strains ( $\varepsilon_{lim}$ )- Parametric Study

The limiting tensile strain ( $\varepsilon_{lim}$ ) and the associated damage category due to tunneling-induced ground deformations are influenced by various factors, including tunnel parameters, soil properties, and structural characteristics (Singh et al., 2020). The present study aims to assess the influence of each of these parameters on  $\varepsilon_{lim}$  and the resulting damage classification. Table 6-9 summarises the range of parameters considered in the analysis. While evaluating the effect of a particular parameter, the remaining parameters are maintained at their respective base values as specified in Table 6-9. Each case is analysed considering both masonry (M) and framed (F) structural types. It is observed that, for identical ground movements, the  $\varepsilon_{lim}$  is higher in masonry buildings compared to framed buildings in all the cases studied. This difference arises due to the higher stiffness ratio ( $E/G$ ) associated with framed structures (Netzel, 2009; Mathew and Lehane, 2014; Son, 2015, 2016; Zhao et al., 2019; Mirhabibi and Soroush, 2020). In the comparison between the two structural types, the  $\varepsilon_{bt}$  is greater in masonry buildings, whereas the  $\varepsilon_{dt}$  is higher in framed buildings. Similar findings were also reported by Netzel (2009). Furthermore, the damage categories assigned based on the  $S_{bmax}$  and  $\beta_{max}$ , as proposed by Boscardin and Cording (1989), Burland (1995), and Burland et al. (2001), remain the same for both masonry and framed buildings. This is because these classification approaches do

not explicitly incorporate  $\varepsilon_{bmax}$  and  $\varepsilon_{dmax}$ , which are dependent on the stiffness ratio ( $E/G$ ) of the building. It is also noted that the damage category derived from  $\varepsilon_{lim}$  represents the critical condition in all the cases analyzed.

Table 6-9 Range of parameters variation for  $\varepsilon_{lim}$  and damage category: parametric study

| Parameters                                                   | Range                                              | Increment       | Base Value                                         |
|--------------------------------------------------------------|----------------------------------------------------|-----------------|----------------------------------------------------|
| Tunnel parameters                                            |                                                    |                 |                                                    |
| Tunnel Diameter, $D$ (m)                                     | 5.5-7.5                                            | 1               | 6.5                                                |
| Tunnel Depth, $z$ (m)                                        | 10-20                                              | 5               | 15                                                 |
| Volume Loss, $V_L$ (%)                                       | 1-2                                                | 0.5             | 1.5                                                |
| Soil Parameters                                              |                                                    |                 |                                                    |
| Young's modulus, $E'_{ref}$ (kPa)                            | $0.5 E'_{ref}$ - $1.5 E'_{ref}$                    | $0.5 E'_{ref}$  | $E'_{ref}$ (respective value for each layer)       |
| Shear Strength parameters of soil, $s_u$ (kPa) & $\phi'$ (°) | $0.75 s_u$ - $1.25 s_u$<br>$(\phi'-2) - (\phi'+2)$ | $0.25 s_u$<br>2 | $s_u$ & $\phi'$ (respective values for each layer) |
| Structure Parameters                                         |                                                    |                 |                                                    |
| Building Height, $H$ (m)                                     | 3- 15                                              | 3               | 9                                                  |
| Building Length, $L$ (m)                                     | 6-21                                               | 3               | 15                                                 |
| Building Location, $X$ (m)                                   | 0-24                                               | 6               | 0                                                  |

### 6.3.1 Effect of Tunnel Parameters

While analysing the influence of tunnel parameters such as  $D$ ,  $z$ , and  $V_L$ , the soil and structural parameters are maintained at their base values, as specified in Table 6-9, for both masonry and framed buildings. It is noteworthy that, across the entire range of tunnel parameters considered in this study, the damage categories derived from  $\varepsilon_{lim}$  for framed buildings are consistent with those determined using the criteria proposed by Boscardin and Cording (1989), Burland (1995), and Burland et al. (2001). A detailed discussion of the effects of tunnel parameters is presented in the subsequent paragraphs.

#### 6.3.1.1 Effect of Tunnel Diameter ( $D$ )

The effect of  $D$  on  $\varepsilon_{lim}$  and the associated damage category has been investigated by varying  $D$  from 5.5 m to 7.5 m, with an increment of 1 m. A total of three cases have been analysed, keeping  $z$  and  $V_L$  constant at 15 m and 1.5%, respectively, for both masonry and framed buildings. Fig. 6.15, Fig. 6.14, and Fig. 6.16 present the vertical settlement ( $S_x$ ) and horizontal displacement ( $H_x$ ) profiles, together with the building location and dimensions, for  $D$  equals to 5.5 m, 6.5 m, and 7.5 m, respectively. Table 6-10, together with Fig. 6.17 and Fig. 6.18, presents the

deformation measures and the corresponding damage categories for both masonry and framed buildings. It is evident from Table 6-10 that sagging occurs along the entire length of the buildings for all tunnel sizes, with the  $\varepsilon_{bt}$  consistently emerging as the critical deformation measure when compared to the  $\varepsilon_{dt}$  in both structural types. Furthermore, Table 6-10 and Fig. 6.19 illustrate that, for masonry buildings,  $\varepsilon_{lim}$  increases by approximately 70% as  $D$  increases from 5.5 m to 7.5 m. This increase results in a shift in the damage category from '3 – Moderate' at  $D$  equals to 5.5 m and 6.5 m to '4 to 5 – Severe to Very Severe' at  $D$  equals to 7.5 m.

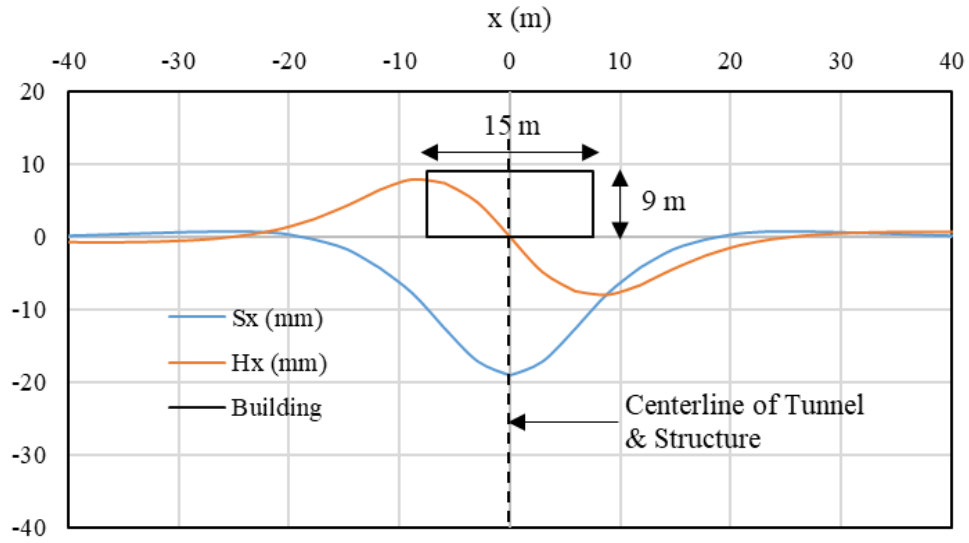


Fig. 6.15 Ground deformations ( $S_x$  and  $H_x$ ) and Building ( $D = 5.5$  m)

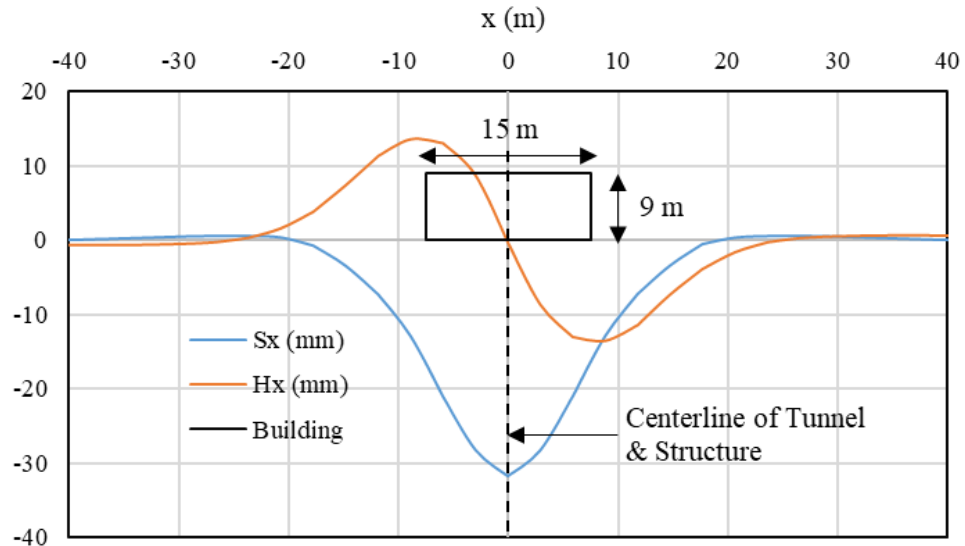


Fig. 6.16 Ground deformations ( $S_x$  and  $H_x$ ) and Building ( $D = 7.5$  m)

A similar trend is observed in framed buildings, where  $\varepsilon_{lim}$  increases by about 71.8% over the same range of  $D$ , leading to a change in the damage category from '2 – Slight' at  $D$  equals to 5.5 m to '3 – Moderate' at  $D$  equals to 6.5 m and 7.5 m.

Table 6-10 Effect of  $D$  on building deformation measures and damage categories

| $D$<br>(m) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\varepsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\varepsilon_{bt}$<br>(%) | $\varepsilon_{dt}$<br>(%) | $\varepsilon_{lim}$<br>(%) | Category based on |               |                      |
|------------|------|------|--------------------|---------------|-----------------------------|-------------------|---------------------------|---------------------------|----------------------------|-------------------|---------------|----------------------|
|            |      |      |                    |               |                             |                   |                           |                           |                            | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$  |
| 5.5        | M    | Sag. | 19.1               | 0.0012        | 0.1024                      | 0.0607            | 0.2053                    | 0.1136                    | 0.2053                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |
|            | F    | Sag. | 19.1               | 0.0012        | 0.1024                      | 0.0607            | 0.1365                    | 0.1282                    | 0.1365                     | 2- Sl.            | 1- Neg.       | 2- Sl.               |
| 6.5        | M    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.2745                    | 0.1529                    | 0.2745                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |
|            | F    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.1836                    | 0.1720                    | 0.1836                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |
| 7.5        | M    | Sag. | 31.78              | 0.0020        | 0.1777                      | 0.1011            | 0.3491                    | 0.1956                    | 0.3491                     | 2- Sl.            | 2- Sl.        | 4 to 5 S.<br>to V.S. |
|            | F    | Sag. | 31.78              | 0.0020        | 0.1777                      | 0.1011            | 0.2346                    | 0.2194                    | 0.2346                     | 2- Sl.            | 2- Sl.        | 3- Mod.              |

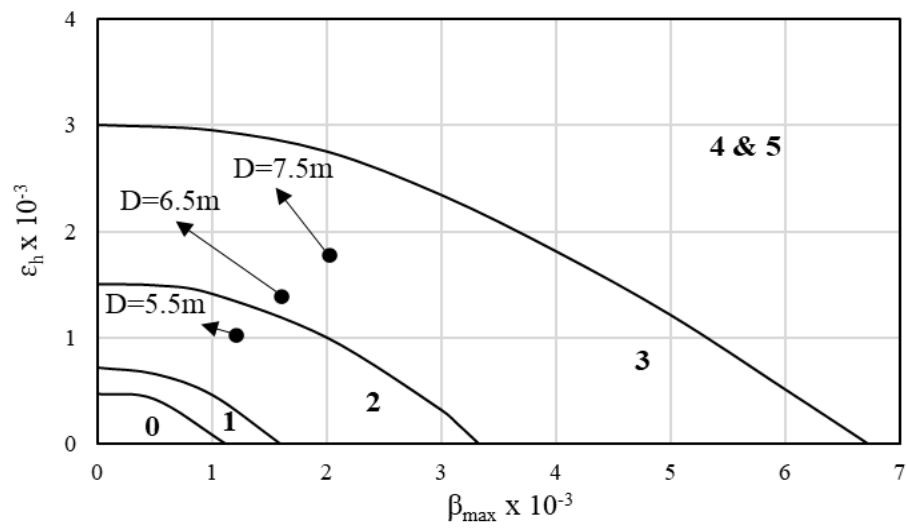


Fig. 6.17 Damage category for various  $D$  as per Boscardin and Cording (1989)

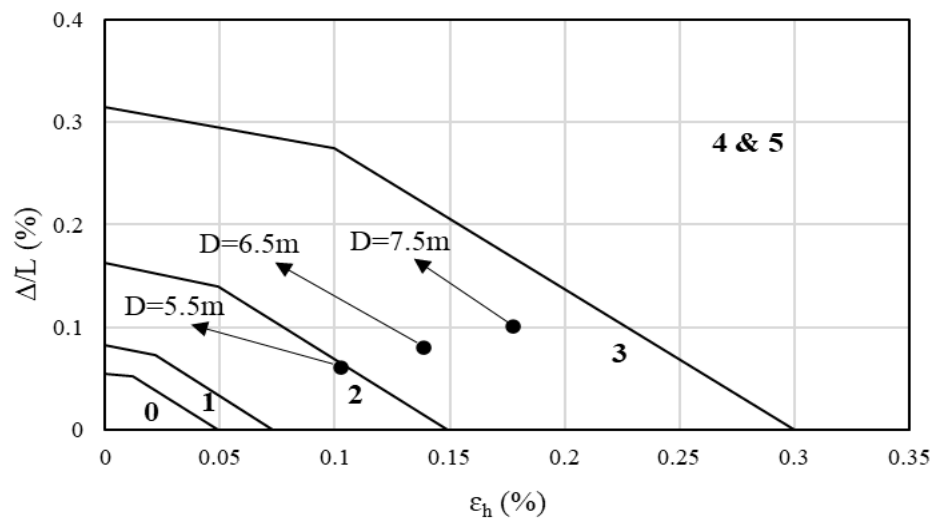


Fig. 6.18 Damage category for various  $D$  as per as per Burland (1995) and Burland et al. (2001)

These results clearly demonstrate that an increase in  $D$  causes a significant rise in ground deformations beneath the building, which in turn amplifies  $\varepsilon_{lim}$  and the associated damage category. Comparable observations have also been reported by other researchers (Mei and Song, 2021; Son, 2015, 2016; Tian et al., 2017).

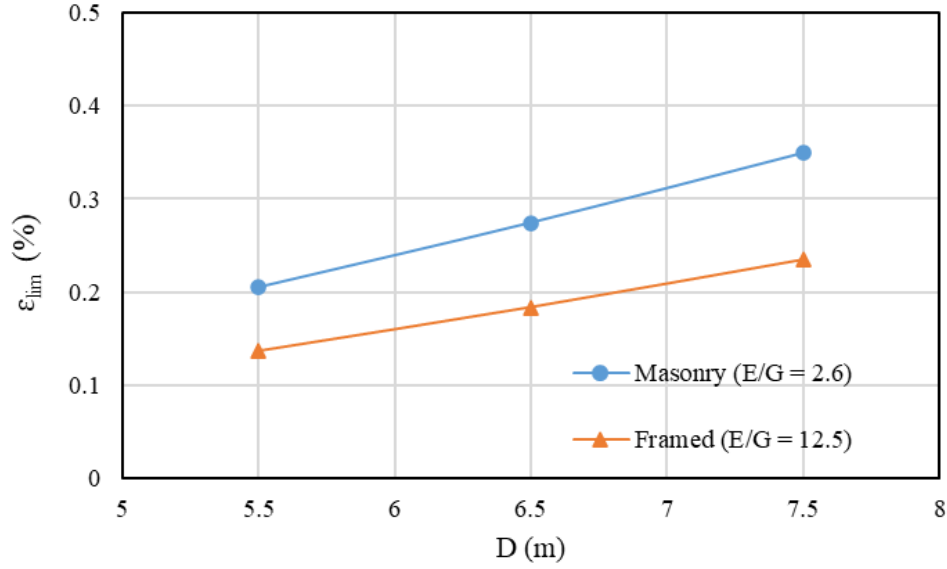


Fig. 6.19 Variation of  $\varepsilon_{lim}$  with  $D$

### 6.3.1.2 Effect of Tunnel Axial Depth ( $z$ )

The effect of  $z$  on  $\varepsilon_{lim}$  and the associated damage category has been investigated by varying  $z$  from 10 m to 20 m, with an increment of 5 m. A total of three cases has been analysed, keeping  $D$  and  $V_L$  constant at 6.5 m and 1.5%, respectively, for both masonry and framed buildings. Fig. 6.20, Fig. 6.14, and Fig. 6.21 present the vertical settlement ( $S_x$ ) and horizontal displacement ( $H_x$ ) profiles, together with the building location and dimensions, for  $z$  equals to 10 m, 15 m, and 20 m, respectively. Table 6-11, along with Fig. 6.22 and Fig. 6.23, summarises the various deformation measures and the corresponding damage categories for both structural types. It is observed from Table 6-11 that, for  $z$  equals to 10 m, the building experiences partial sagging and partial hogging, with sagging identified as the critical mode in terms of deformations. In contrast, sagging occurs along the entire building length for  $z$  equals to 15 m and 20 m. For both masonry and framed buildings,  $\varepsilon_{bt}$  is generally identified as the critical deformation measure when compared to  $\varepsilon_{dt}$ , except in case of frame building at  $z$  equals to 10 m, where  $\varepsilon_{dt}$  exceeds  $\varepsilon_{bt}$ . Furthermore, Table 6-11 and Fig. 6.24 indicate that  $\varepsilon_{lim}$  decreases substantially, by approximately 68.3% as  $z$  increases from 10 m to 20 m for masonry buildings. This reduction results in a shift in the damage category from '4 to 5 – Severe to Very Severe' at  $z$  equals to 10 m to '3 – Moderate' at  $z$  equals to 15 m and 20 m. A similar trend is observed in framed buildings, where  $\varepsilon_{lim}$  decreases by about 65.1% over the same range of  $z$ , leading to a change in damage category from '4 to 5 – Severe to Very Severe' at  $z$  equals to 10 m to '3 – Moderate' at  $z$  equals to 15 m and finally to '2 – Slight' at  $z$  equals to 20 m.

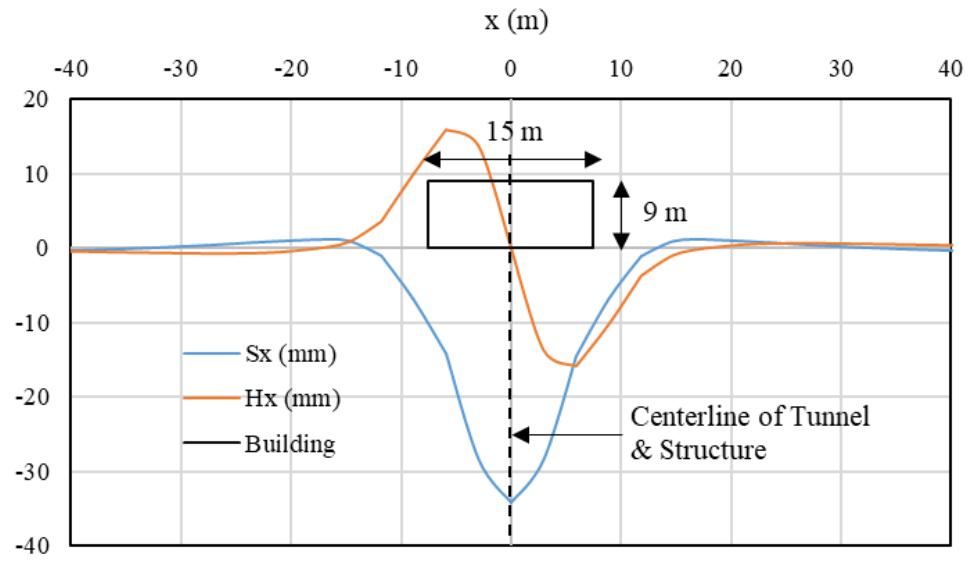


Fig. 6.20 Ground deformations ( $S_x$  and  $H_x$ ) and Building ( $z = 10$  m)

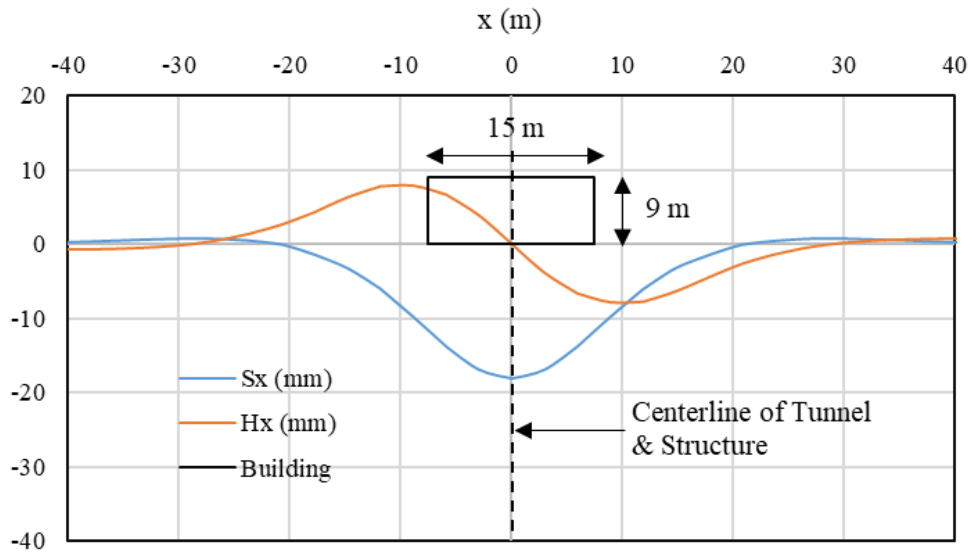


Fig. 6.21 Ground deformations ( $S_x$  and  $H_x$ ) and Building ( $z = 20$  m)

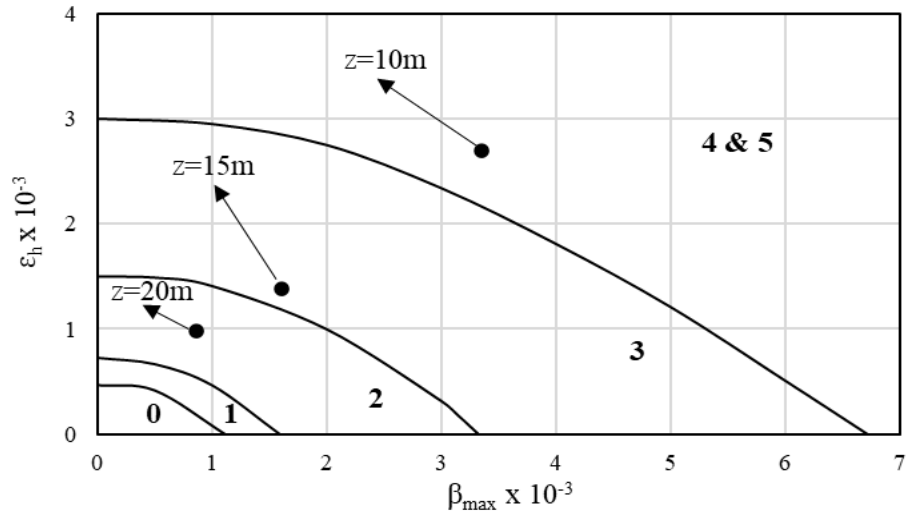
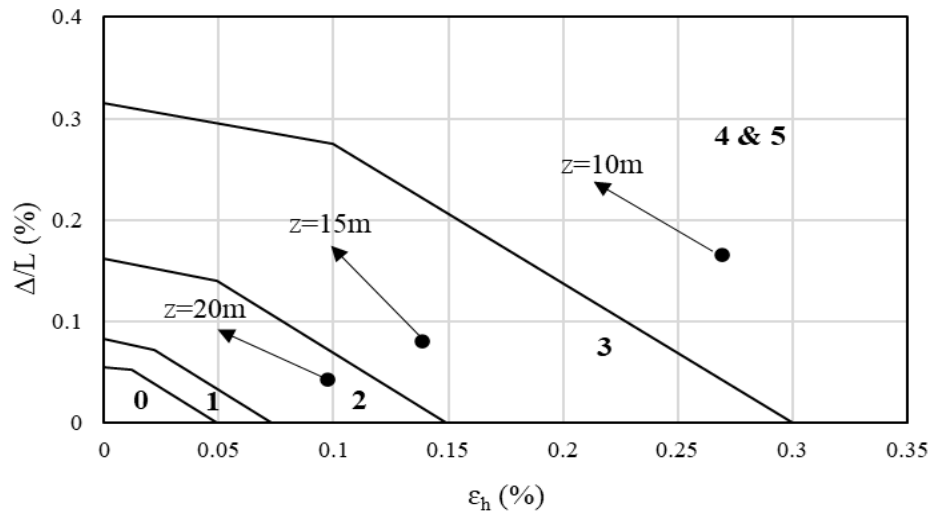
Table 6-11 Effect of  $z$  on building deformation measures and damage categories

| $z$<br>(m) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\epsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\epsilon_{bt}$<br>(%) | $\epsilon_{dt}$<br>(%) | $\epsilon_{lim}$<br>(%) | Category based on |               |                      |
|------------|------|------|--------------------|---------------|--------------------------|-------------------|------------------------|------------------------|-------------------------|-------------------|---------------|----------------------|
|            |      |      |                    |               |                          |                   |                        |                        |                         | $S_{bmax}$        | $\beta_{max}$ | $\epsilon_{lim}$     |
| 10         | M    | Sag. | 34.18              | 0.0034        | 0.2692                   | 0.1656            | 0.5387                 | 0.3141                 | 0.5387                  | 2- Sl.            | 2- Sl.        | 4 to 5 S.<br>to V.S. |
|            |      | Hog. |                    |               | 0.1996                   | 0.0009            | 0.1997                 | 0.1996                 |                         |                   |               |                      |
|            | F    | Sag. | 34.18              | 0.0034        | 0.2692                   | 0.1656            | 0.3473                 | 0.3493                 | 0.3493                  | 2- Sl.            | 2- Sl.        | 4 to 5 S.<br>to V.S. |
|            |      | Hog. |                    |               | 0.1996                   | 0.0009            | 0.1996                 | 0.1996                 |                         |                   |               |                      |

continued on page 77

Table 6.11 (continued)

| z<br>(m) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\varepsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\varepsilon_{bt}$<br>(%) | $\varepsilon_{dt}$<br>(%) | $\varepsilon_{lim}$<br>(%) | Category based on |               |                     |
|----------|------|------|--------------------|---------------|-----------------------------|-------------------|---------------------------|---------------------------|----------------------------|-------------------|---------------|---------------------|
|          |      |      |                    |               |                             |                   |                           |                           |                            | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$ |
| 15       | M    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.2745                    | 0.1529                    | 0.2745                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|          | F    | Hog. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.1836                    | 0.1720                    | 0.1836                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 20       | M    | Sag. | 18.08              | 0.0009        | 0.0975                      | 0.0432            | 0.1708                    | 0.1037                    | 0.1708                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|          | F    | Hog. | 18.08              | 0.0009        | 0.0975                      | 0.0432            | 0.1218                    | 0.1122                    | 0.1218                     | 2- Sl.            | 1- Neg.       | 2- Sl.              |

Fig. 6.22 Damage category for various  $z$  as per Boscardin and Cording (1989)Fig. 6.23 Damage category for various  $z$  as per as per Burland (1995) and Burland et al. (2001)

These findings clearly demonstrate that an increase in tunnel depth significantly reduces ground deformations beneath the building, thereby decreasing

$\varepsilon_{lim}$  and the associated damage category, consistent with observations reported by other researchers (Mei and Song, 2021; Son, 2015, 2016; Tian et al., 2017; Zhao et al., 2019).

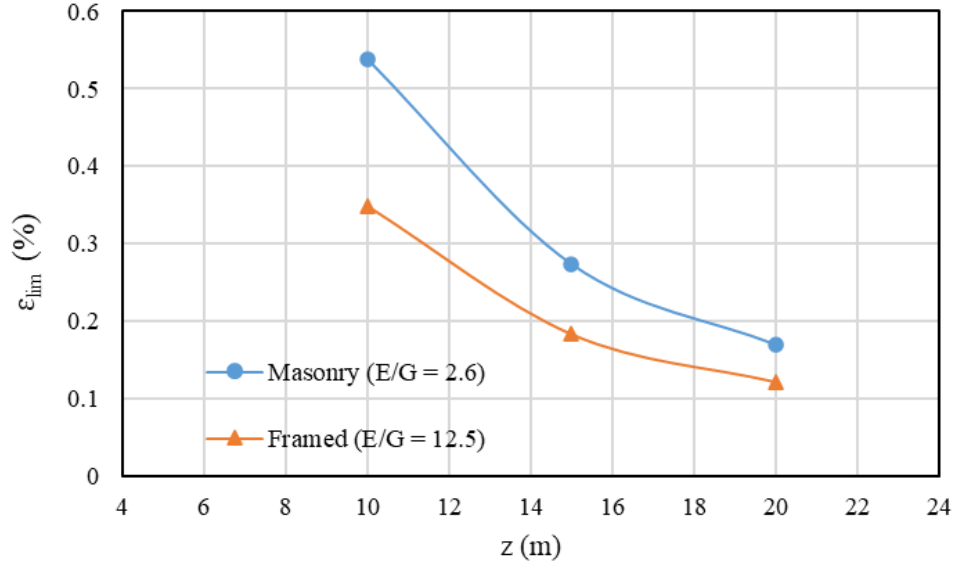


Fig. 6.24 Variation of  $\varepsilon_{lim}$  with  $z$

### 6.3.1.3 Effect of Volume Loss ( $V_L$ )

The effect of  $V_L$  on  $\varepsilon_{lim}$  and the associated damage category has been investigated by varying  $V_L$  from 1% to 2%, with an increment of 0.5%. A total of three cases has been analysed, keeping  $D$  and  $z$  constant at 6.5 m and 15 m, respectively, for both masonry and framed buildings. Fig. 6.25, Fig. 6.14, and Fig. 6.26 present the vertical settlement ( $S_x$ ) and horizontal displacement ( $H_x$ ) profiles, together with the building location and dimensions, for  $V_L$  equals to 1%, 1.5%, and 2%, respectively. Table 6-12, along with Fig. 6.27 and Fig. 6.28, summarises the various deformation measures and the corresponding damage categories for both structural types. It is observed from Table 6-12 that sagging occurs along the entire length of the buildings for all  $V_L$  values, with the  $\varepsilon_{bt}$  identified as the critical deformation measure compared to the  $\varepsilon_{dt}$  in both masonry and framed buildings. Furthermore, Table 6-12 and Fig. 6.29 show that  $\varepsilon_{lim}$  increases substantially, by approximately 66.6% as  $V_L$  rises from 1% to 2% for masonry buildings. This increase causes the damage category to shift from '3 – Moderate' at  $V_L$  equals to 1% and 1.5% to '4 to 5 – Severe to Very Severe' at  $V_L$  equals to 2%. A similar trend is observed in framed buildings, where  $\varepsilon_{lim}$  increases by about 78.8% over the same range of  $V_L$ , resulting in a change in damage category from '2 – Slight' at  $V_L$  equals to 1% to '3 – Moderate' at  $V_L$  equals to 1.5% and 2%. These results demonstrate that an increase in  $V_L$  leads to a significant rise in ground deformations beneath the building, thereby increasing  $\varepsilon_{lim}$  and the associated damage category, consistent with observations reported by other researchers (Boonpichetvong

et al., 2006; Giardina et al., 2015; Jenck and Dias, 2003; Netzel, 2009; Son, 2015, 2016; Tian et al., 2017; Zhang et al, 2014; Zhao et al., 2019).

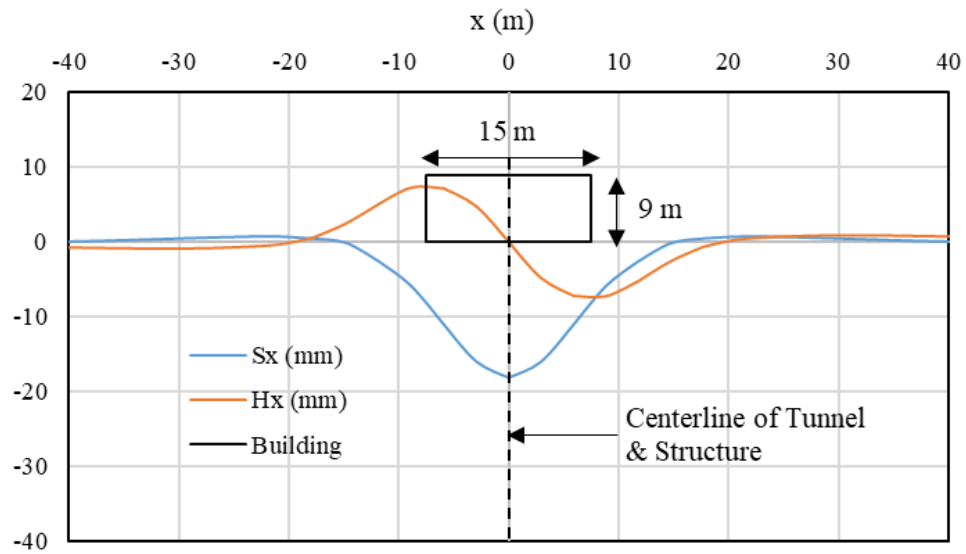


Fig. 6.25 Ground deformations ( $S_x$  and  $H_x$ ) and Building ( $V_L = 1\%$ )

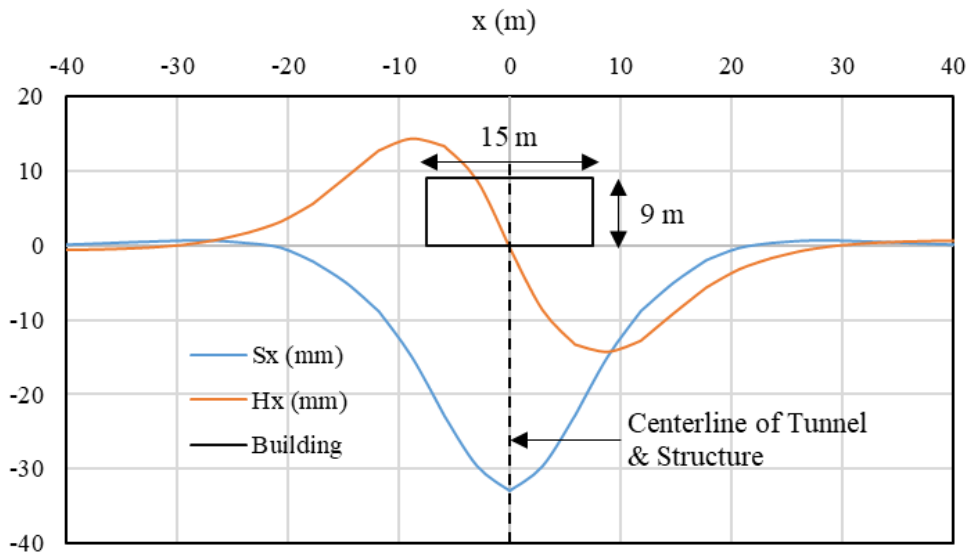


Fig. 6.26 Ground deformations ( $S_x$  and  $H_x$ ) and Building ( $V_L = 2\%$ )

Table 6-12 Effect of  $V_L$  on building deformation measures and damage categories

| $V_L$<br>(%) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\varepsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\varepsilon_{bt}$<br>(%) | $\varepsilon_{dt}$<br>(%) | $\varepsilon_{lim}$<br>(%) | Category based on |               |                     |
|--------------|------|------|--------------------|---------------|-----------------------------|-------------------|---------------------------|---------------------------|----------------------------|-------------------|---------------|---------------------|
|              |      |      |                    |               |                             |                   |                           |                           |                            | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$ |
| 1            | M    | Sag. | 18.15              | 0.0013        | 0.0960                      | 0.0663            | 0.2085                    | 0.1099                    | 0.2085                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|              | F    | Sag. | 18.15              | 0.0013        | 0.0960                      | 0.0663            | 0.1333                    | 0.1273                    | 0.1333                     | 2- Sl.            | 1- Neg.       | 2- Sl.              |

Continued on page 80

Table 6-12 (continued)

| $V_L$<br>(%) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\varepsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\varepsilon_{bt}$<br>(%) | $\varepsilon_{dt}$<br>(%) | $\varepsilon_{lim}$<br>(%) | Category based on |               |                      |
|--------------|------|------|--------------------|---------------|-----------------------------|-------------------|---------------------------|---------------------------|----------------------------|-------------------|---------------|----------------------|
|              |      |      |                    |               |                             |                   |                           |                           |                            | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$  |
| 1.5          | M    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.2745                    | 0.1529                    | 0.2745                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |
|              | F    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.1836                    | 0.1720                    | 0.1836                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |
| 2            | M    | Sag. | 32.88              | 0.0019        | 0.1843                      | 0.0962            | 0.3474                    | 0.2001                    | 0.3474                     | 2- Sl.            | 1- Neg.       | 4 to 5 S.<br>to V.S. |
|              | F    | Sag. | 32.88              | 0.0019        | 0.1843                      | 0.0962            | 0.2384                    | 0.2215                    | 0.2384                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |

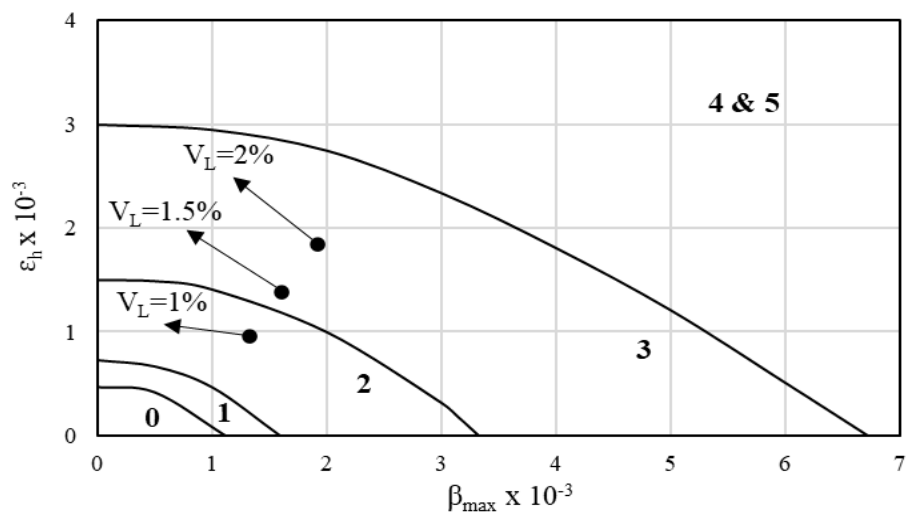


Fig. 6.27 Damage category for various  $V_L$  as per Boscardin and Cording (1989)

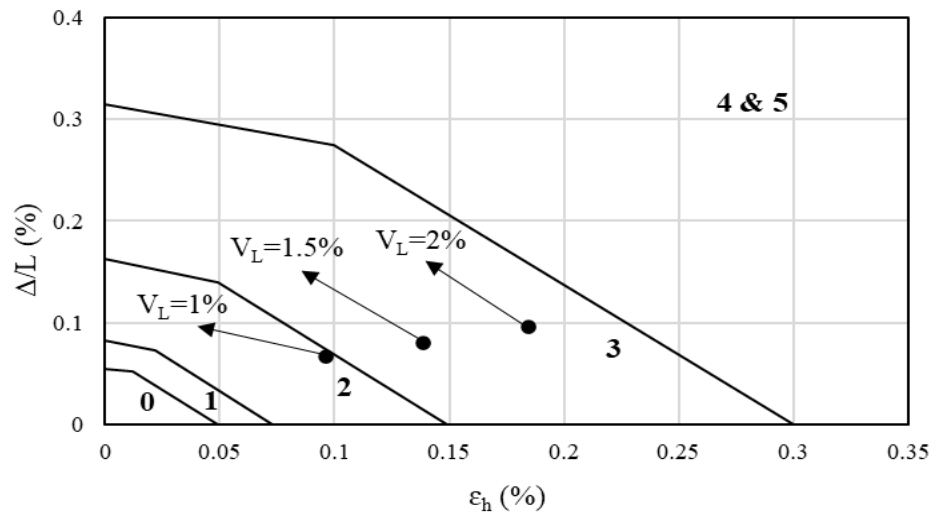


Fig. 6.28 Damage category for various  $V_L$  as per as per Burland (1995) and Burland et al. (2001)

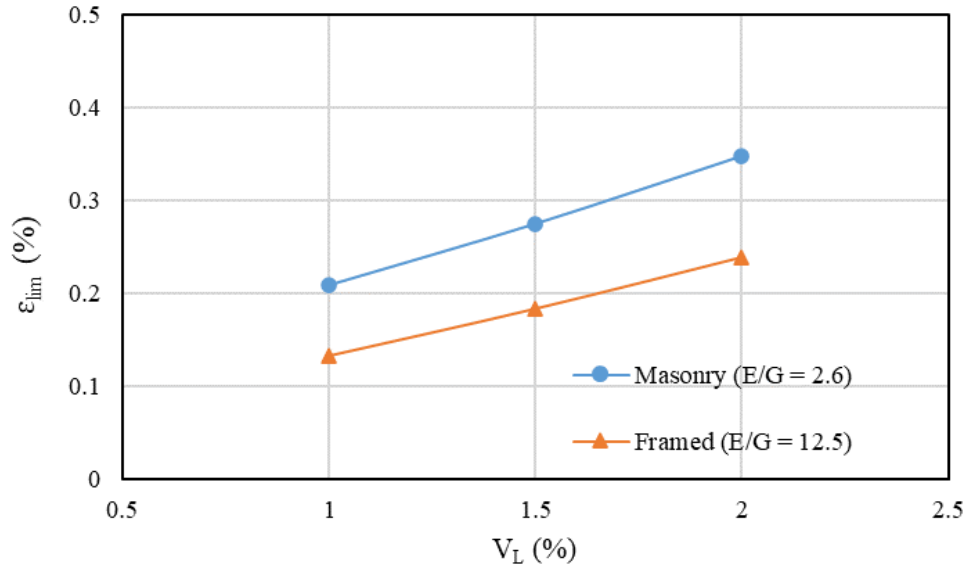


Fig. 6.29 Variation of  $\epsilon_{lim}$  with  $V_L$

### 6.3.2 Effect of Soil Parameters

While analysing the influence of soil parameters such as  $E'_{ref}$ ,  $s_u$  and  $\phi'$ , the tunnel and structure parameters are maintained at their base values, as specified in Table 6-9, for both masonry and framed buildings. A detailed discussion of the effects of these soil parameters is presented in the subsequent paragraphs.

#### 6.3.2.1 Effect of Shear Strength Parameters of Soil ( $s_u$ and $\phi'$ )

The effect of  $s_u$  and  $\phi'$  on  $\epsilon_{lim}$  and the associated damage category has been investigated by varying  $s_u$  from  $0.75 s_u$  to  $1.25 s_u$ , in increments of  $0.25 s_u$  and  $\phi'$  from  $(\phi' - 2)^\circ$  to  $(\phi' + 2)^\circ$  in increments of  $2^\circ$ , across all soil layers. A total of three cases has been analysed while maintaining  $E'_{ref}$  at the base values provided in Table 6-9, for both masonry and framed buildings. The three cases analysed here are defined as follows:

- Case 1:  $s_u = 0.75 s_u$  and  $\phi' = (\phi' - 2)$
- Case 2:  $s_u = s_u$  and  $\phi' = \phi'$  (base case)
- Case 3:  $s_u = 1.25 s_u$  and  $\phi' = (\phi' + 2)$

Fig. 6.30, Fig. 6.14, and Fig. 6.31 present the vertical settlement ( $S_x$ ) and horizontal displacement ( $H_x$ ) profiles, together with the building location and dimensions, for Case 1, Case 2 and Case 3 respectively. Table 6-13, along with Fig. 6.32 and Fig. 6.33, summarises the various deformation measures and the corresponding damage categories for both structural types. It is observed from Table 6-13 that sagging occurs along the entire length of the buildings for all the cases, with

the  $\varepsilon_{bt}$  identified as the critical deformation measure compared to the  $\varepsilon_{dt}$  in both masonry and framed buildings.

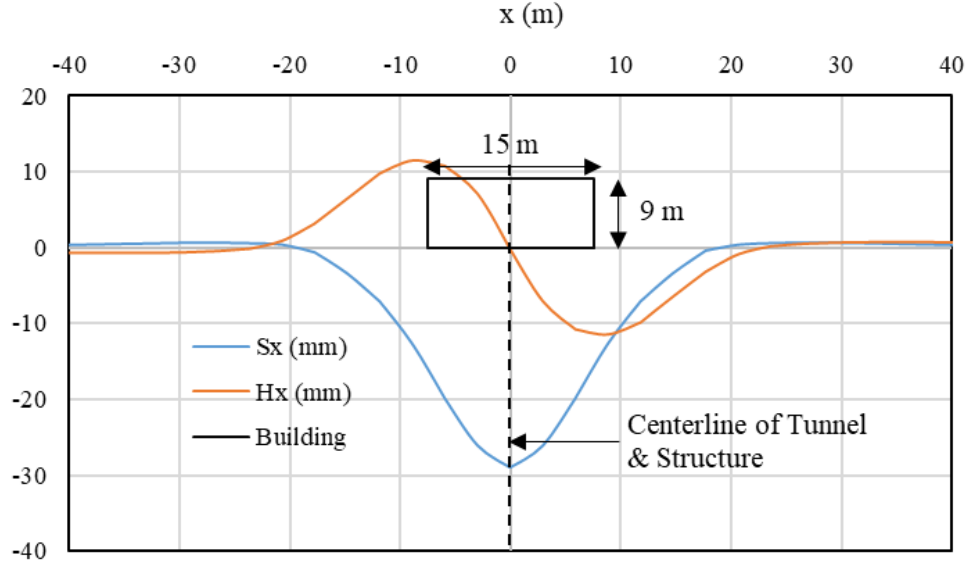


Fig. 6.30 Ground deformations ( $S_x$  and  $H_x$ ) and Building (Case 1)

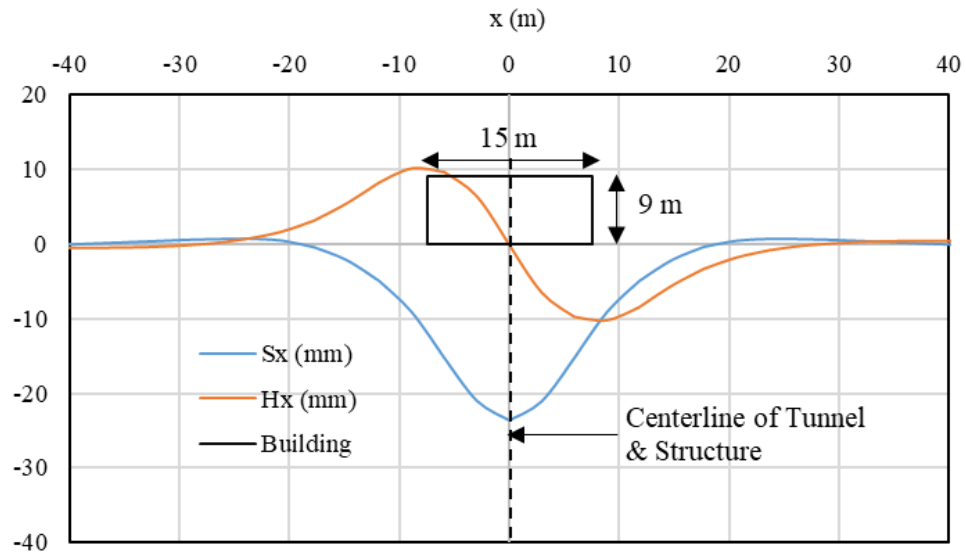


Fig. 6.31 Ground deformations ( $S_x$  and  $H_x$ ) and Building (Case 3)

Furthermore, Table 6-13 and Fig. 6.34 show that  $\varepsilon_{lim}$  decreases by only 10.8% as when  $s_u$  and  $\phi'$  increase from Case 1 to Case 3, for masonry buildings. A similar trend is observed in framed buildings, where  $\varepsilon_{lim}$  decreases by only 11% over the same range of  $s_u$  and  $\phi'$ . Despite this reduction, the damage category remains unchanged at '3 – Moderate' across all cases and both building types.

Table 6-13 Effect of  $s_u$  and  $\phi'$  on building deformation measures and damage categories

| Case No. | Type | Mode | $S_{bmax}$ (mm) | $\beta_{max}$ | $\varepsilon_{hmax}$ (%) | $\Delta/L$ (%) | $\varepsilon_{bt}$ (%) | $\varepsilon_{dt}$ (%) | $\varepsilon_{lim}$ (%) | Category based on |               |                     |
|----------|------|------|-----------------|---------------|--------------------------|----------------|------------------------|------------------------|-------------------------|-------------------|---------------|---------------------|
|          |      |      |                 |               |                          |                |                        |                        |                         | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$ |
| 1        | M    | Sag. | 28.9            | 0.0017        | 0.1491                   | 0.0866         | 0.2960                 | 0.1648                 | 0.2960                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|          | F    | Sag. | 28.9            | 0.0017        | 0.1491                   | 0.0866         | 0.1979                 | 0.1854                 | 0.1979                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 2        | M    | Sag. | 25.24           | 0.0016        | 0.1384                   | 0.0803         | 0.2745                 | 0.1529                 | 0.2745                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|          | F    | Sag. | 25.24           | 0.0016        | 0.1384                   | 0.0803         | 0.1836                 | 0.1720                 | 0.1836                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 3        | M    | Sag. | 23.65           | 0.0016        | 0.1324                   | 0.0775         | 0.2639                 | 0.1465                 | 0.2639                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|          | F    | Sag. | 23.65           | 0.0016        | 0.1324                   | 0.0775         | 0.1760                 | 0.1651                 | 0.1760                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |

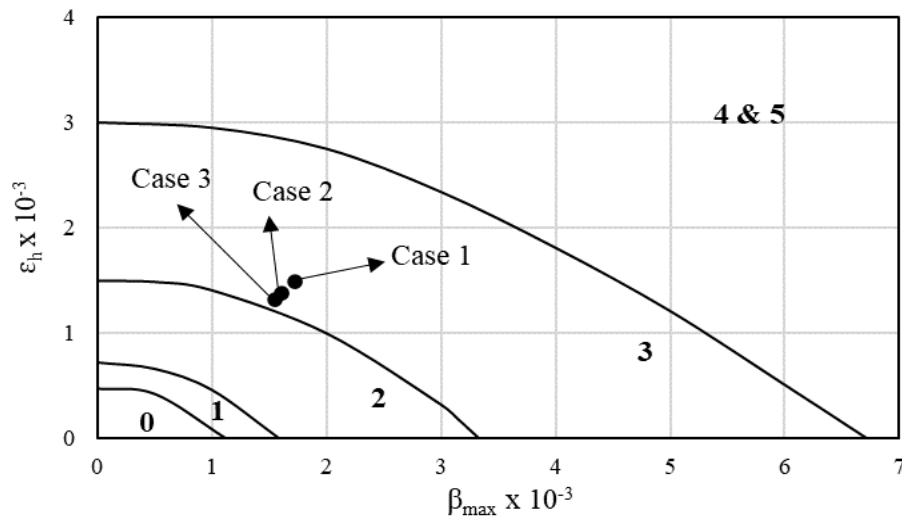


Fig. 6.32 Damage category for various  $s_u$  and  $\phi'$  as per Boscardin and Cording (1989)

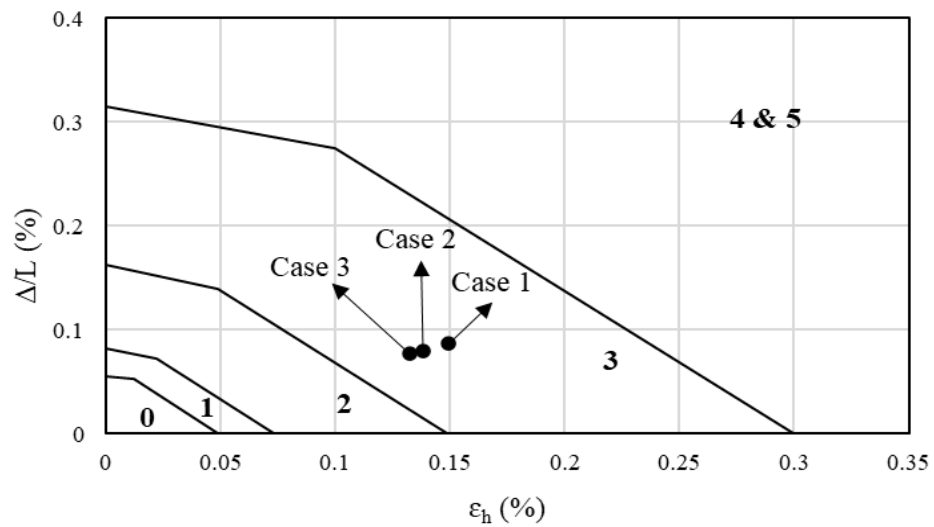


Fig. 6.33 Damage category for various  $s_u$  and  $\phi'$  as per as per Burland (1995) and Burland et al. (2001)

It is further noted that, within the investigated range of  $s_u$  and  $\phi'$ , the damage categories derived from  $\varepsilon_{lim}$  are consistent with those predicted using the criteria proposed by Boscardin and Cording (1989), Burland (1995) and Burland et al. (2001). The observed trend, whereby an increase in  $s_u$  and  $\phi'$  reduces ground deformations beneath the structure, leading to lower  $\varepsilon_{lim}$ , has also been reported by other researchers (Rebello et al., 2014; Son, 2015, 2016).

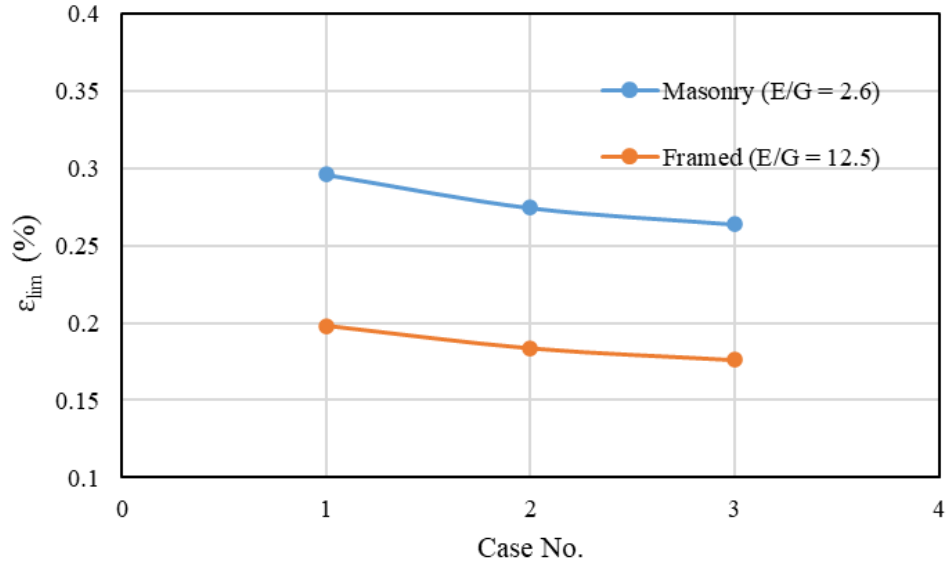


Fig. 6.34 Variation of  $\varepsilon_{lim}$  with  $s_u$  and  $\phi'$

#### 6.3.2.2 Effect of Soil modulus ( $E'_{ref}$ )

The effect of  $E'_{ref}$  on  $\varepsilon_{lim}$  and the associated damage category has been investigated by varying  $E'_{ref}$  from  $0.5 E'_{ref}$  to  $1.5 E'_{ref}$ , in increments of  $0.5 E'_{ref}$ , across all soil layers. A total of three cases has been analysed while maintaining  $s_u$  and  $\phi'$  at the base values provided in Table 6-9, for both masonry and framed buildings. Fig. 6.35, Fig. 6.14, and Fig. 6.36 present the vertical settlement ( $S_x$ ) and horizontal displacement ( $H_x$ ) profiles, together with the building location and dimensions, for  $0.5 E'_{ref}$ ,  $E'_{ref}$  and  $1.5 E'_{ref}$  respectively. Table 6-14, along with Fig. 6.37 and Fig. 6.38, summarises the various deformation measures and the corresponding damage categories for both structural types. For the case of  $0.5 E'_{ref}$ , the building experiences a combination of sagging and hogging, with sagging being identified as the critical deformation mode. By contrast, for  $E'_{ref}$  and  $1.5 E'_{ref}$ , sagging deformation occurs along the full length of the building. In all cases, the  $\varepsilon_{bt}$  is observed to be the critical deformation measure compared to the  $\varepsilon_{dt}$ , for both masonry and framed structures.

As shown in Table 6-14 and Fig. 6.39,  $\varepsilon_{lim}$  decreases by approximately 30.3% as  $E'_{ref}$  increases from  $0.5 E'_{ref}$  to  $1.5 E'_{ref}$  for masonry buildings. This reduction results in a shift in the damage category from '4 to 5 – Severe to Very Severe' at  $0.5 E'_{ref}$  to '3 – Moderate' at  $E'_{ref}$  and  $1.5 E'_{ref}$ . For framed buildings,  $\varepsilon_{lim}$  decreases

by 27% over the same range of  $E'_{ref}$ , although the damage category remains unchanged at '3 – Moderate'. It is further noted that, for the entire range of  $E'_{ref}$  values examined, the damage categories derived from  $\varepsilon_{lim}$  are consistent with those predicted by the criteria proposed by Boscardin and Cording (1989), Burland (1995), and Burland et al. (2001), particularly for framed buildings. The general trend, whereby an increase in  $E'_{ref}$  leads to a reduction in ground deformations beneath the structure and a consequent decrease in  $\varepsilon_{lim}$  and associated damage category, is in agreement with the findings of other researchers (Fu et al., 2018; Rebello et al., 2014; Son, 2015, 2016).

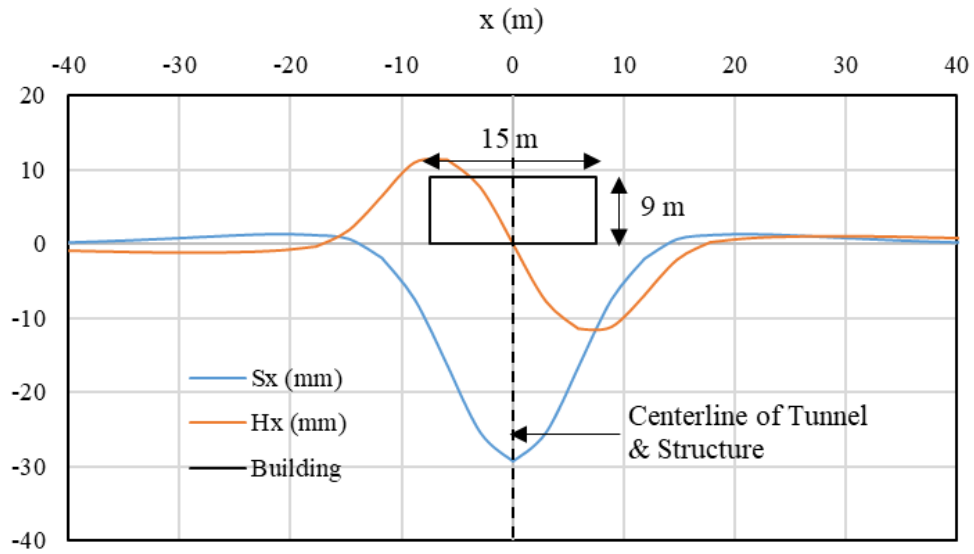


Fig. 6.35 Ground deformations ( $S_x$  and  $H_x$ ) and Building ( $0.5 E'_{ref}$ )

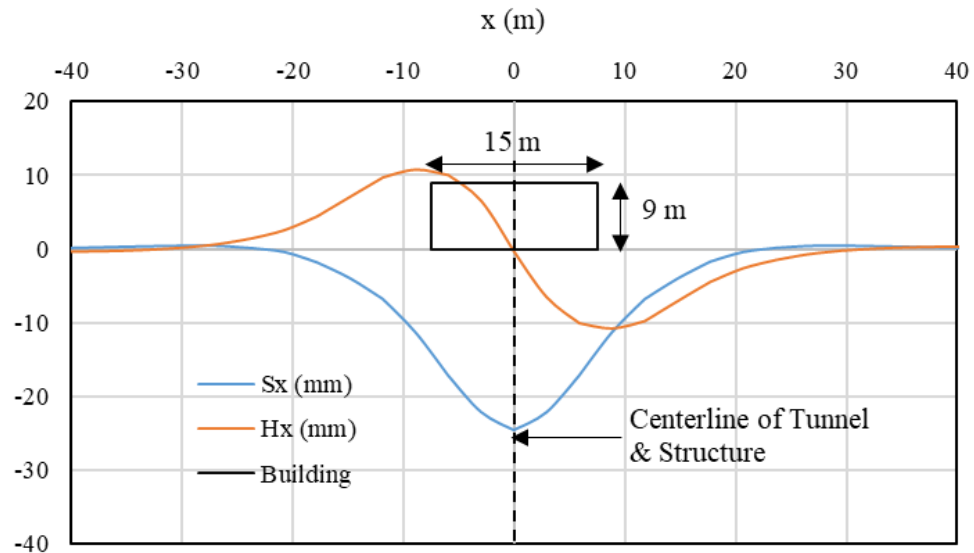


Fig. 6.36 Ground deformations ( $S_x$  and  $H_x$ ) and Building ( $1.5 E'_{ref}$ )

Table 6-14 Effect of  $E'_{ref}$  on building deformation measures and damage categories

| $E'_{ref}$<br>(kPa) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\varepsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\varepsilon_{bt}$<br>(%) | $\varepsilon_{dt}$<br>(%) | $\varepsilon_{lim}$<br>(%) | Category based on |               |                      |
|---------------------|------|------|--------------------|---------------|-----------------------------|-------------------|---------------------------|---------------------------|----------------------------|-------------------|---------------|----------------------|
|                     |      |      |                    |               |                             |                   |                           |                           |                            | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$  |
| $0.5 E'_{ref}$      | M    | Sag. | 29.27              | 0.0022        | 0.1931                      | 0.1095            | 0.3712                    | 0.2209                    | 0.3712                     | 2- Sl.            | 2- Sl.        | 4 to 5 S.<br>to V.S. |
|                     |      | Hog. |                    |               | 0.0108                      | 0.0006            | 0.0109                    | 0.0108                    |                            |                   |               |                      |
|                     | F    | Sag. | 29.27              | 0.0022        | 0.1931                      | 0.1095            | 0.2447                    | 0.2431                    | 0.2447                     | 2- Sl.            | 2- Sl.        | 3- Mod.              |
|                     |      | Hog. |                    |               | 0.0108                      | 0.0006            | 0.0108                    | 0.0108                    |                            |                   |               |                      |
| $E'_{ref}$          | M    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.2745                    | 0.1529                    | 0.2745                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |
|                     | F    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.1836                    | 0.1720                    | 0.1836                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |
| $1.5 E'_{ref}$      | M    | Sag. | 24.63              | 0.0014        | 0.1389                      | 0.0706            | 0.2586                    | 0.1503                    | 0.2586                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |
|                     | F    | Sag. | 24.63              | 0.0014        | 0.1389                      | 0.0706            | 0.1786                    | 0.1657                    | 0.1786                     | 2- Sl.            | 1- Neg.       | 3- Mod.              |

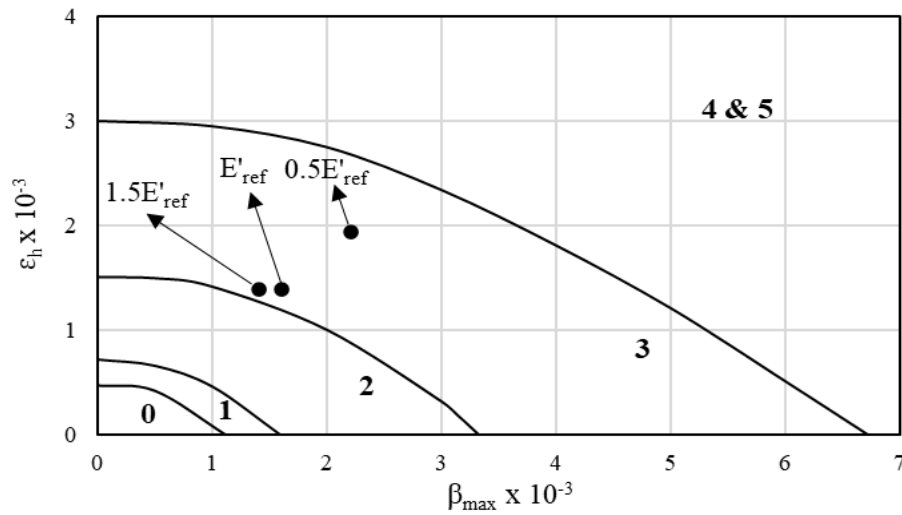


Fig. 6.37 Damage category for various  $E'_{ref}$  as per Boscardin and Cording (1989)

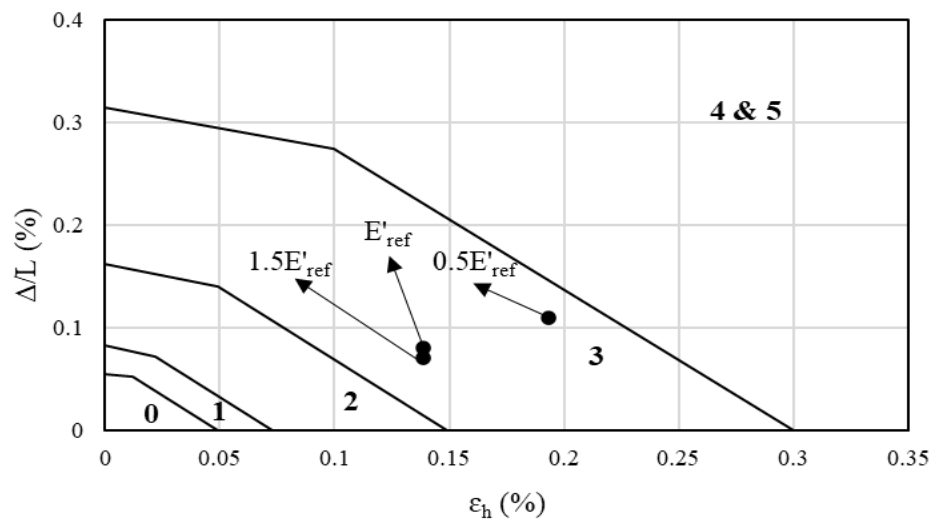


Fig. 6.38 Damage category for various  $E'_{ref}$  as per as per Burland (1995) and Burland et al. (2001)

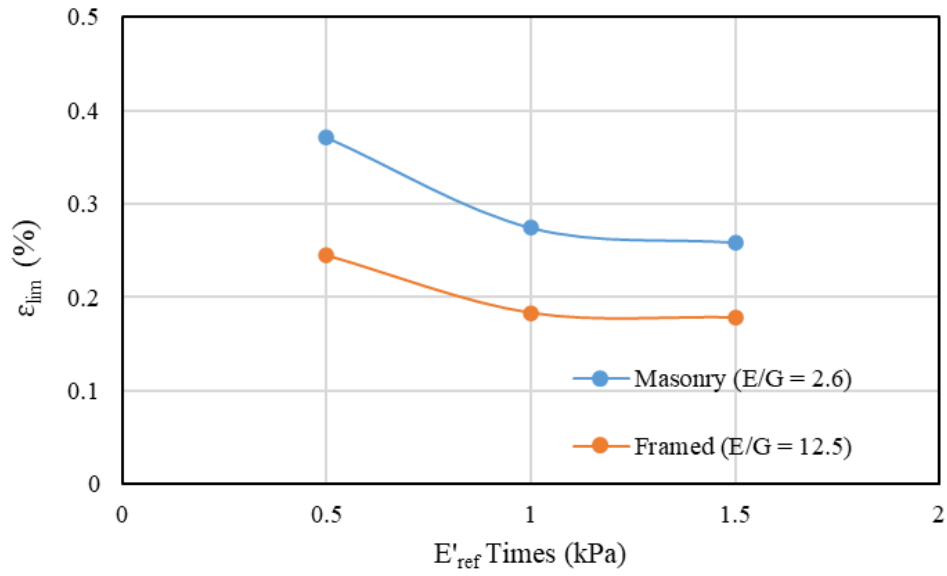


Fig. 6.39 Variation of  $\varepsilon_{lim}$  with  $E'_{ref}$

### 6.3.3 Effect of Structure Parameters

Fig. 6.40 illustrates the vertical settlements and horizontal displacements obtained under greenfield conditions using PLAXIS 2D, together with the location and dimensions of the building. The effects of structural parameters such as building height ( $H$ ), length ( $L$ ), and offset distance ( $X$ ) are investigated for both masonry and framed buildings, while maintaining the tunnel and soil parameters at their base values as specified in Table 6-9.

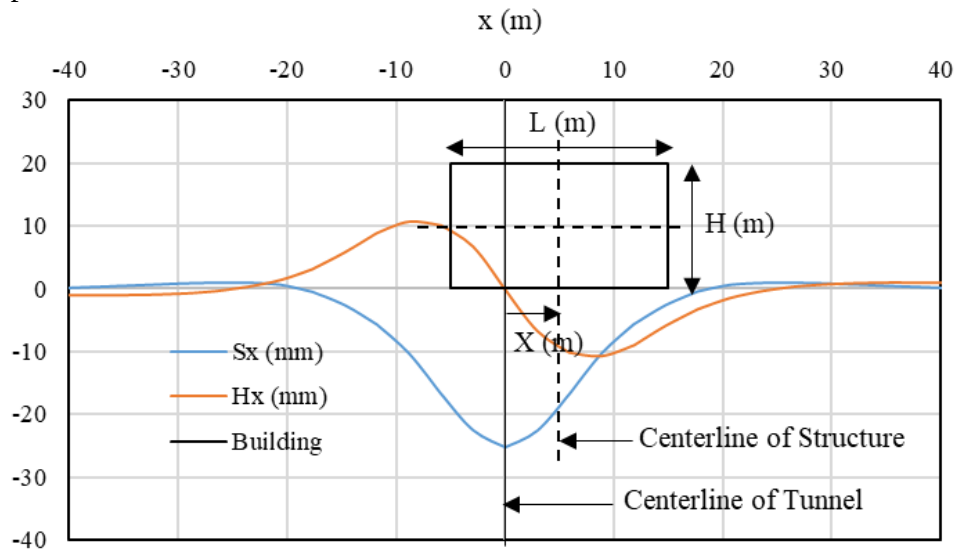


Fig. 6.40 Ground deformations ( $S_x$  and  $H_x$ ) and Building for structure parameters effect

The range of variation of structural parameters has been selected to reflect the typical storeys, dimensions, and setback distances of non-engineered buildings in the Patparganj and Khichdipur areas. A detailed discussion of the effects of these structural parameters is presented in the subsequent paragraphs.

### 6.3.3.1 Effect of Building Height ( $H$ )

The effect of  $H$  on the  $\varepsilon_{lim}$  and the associated damage category has been evaluated by varying  $H$  from 3 m (single-storey building) to 15 m (five-storey building) in increments of 3 m. A total of five cases has been analysed while maintaining  $L$  and  $X$  at 15 m and 0 m, respectively, for both masonry and framed buildings. The various deformation measures and corresponding damage categories for all values of  $H$  are summarised in Table 6-15 and further illustrated in Fig. 6.41 and Fig. 6.42.

From Table 6-15, it is observed that sagging deformation develops along the entire building length for all values of  $H$ . For masonry buildings, the  $\varepsilon_{bt}$  is consistently identified as the critical deformation measure compared with the  $\varepsilon_{dt}$ . For framed buildings,  $\varepsilon_{bt}$  governs the response when  $H$  ranges from 3 m to 9 m. However, at  $H$  equals to 12 m and 15 m,  $\varepsilon_{dt}$  becomes the critical deformation measure, exceeding  $\varepsilon_{bt}$ . As indicated in Table 6-15 and Fig. 6.43,  $\varepsilon_{lim}$  for masonry buildings increases by approximately 22.5% as  $H$  increases from 3 m to 9 m, followed by a 7% decrease when  $H$  increases further from 9 m to 15 m. In contrast, for framed buildings,  $\varepsilon_{lim}$  decreases by about 11.2% as  $H$  increases from 3 m to 12 m, with a subsequent marginal increase of 1.3% as  $H$  increases from 12 m to 15 m. Despite these variations, the damage category remains unchanged at '3 – Moderate' across all five cases for both masonry and framed structures.

Table 6-15 Effect of  $H$  on building deformation measures and damage categories

| $H$<br>(m) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\varepsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\varepsilon_{bt}$<br>(%) | $\varepsilon_{dt}$<br>(%) | $\varepsilon_{lim}$<br>(%) | Category based on |               |                     |
|------------|------|------|--------------------|---------------|-----------------------------|-------------------|---------------------------|---------------------------|----------------------------|-------------------|---------------|---------------------|
|            |      |      |                    |               |                             |                   |                           |                           |                            | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$ |
| 3          | M    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.2240                    | 0.1391                    | 0.2240                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.1986                    | 0.1460                    | 0.1986                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 6          | M    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.2669                    | 0.1444                    | 0.2669                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.1951                    | 0.1629                    | 0.1951                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 9          | M    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.2745                    | 0.1529                    | 0.2745                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.1836                    | 0.1720                    | 0.1836                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 12         | M    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.2670                    | 0.1605                    | 0.2670                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.1747                    | 0.1763                    | 0.1763                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 15         | M    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.2553                    | 0.1662                    | 0.2553                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.00161       | 0.1384                      | 0.0803            | 0.1685                    | 0.1786                    | 0.1786                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |

It is further noted that, for the entire range of  $H$  examined in this study, the damage categories derived from  $\varepsilon_{lim}$  are consistent with those predicted using the criteria proposed by Boscardin and Cording (1989), Burland (1995), and Burland et al. (2001). Similar trends in the variation of  $\varepsilon_{lim}$  with increasing building height have also been reported by other researchers for both masonry and framed structures (Boldini et al. 2018; Mirhabibi and Soroush, 2020; Franzius et al., 2006; Tian et al., 2017; Yiu et al., 2017).

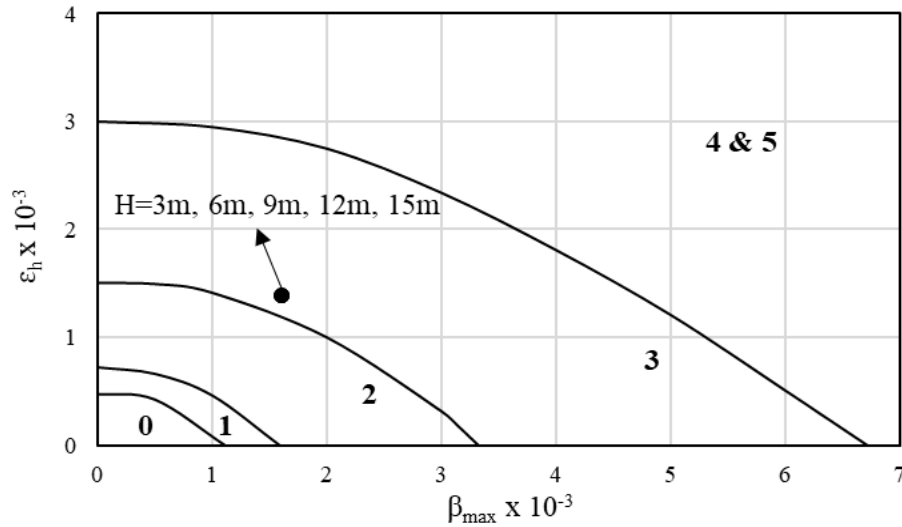


Fig. 6.41 Damage category for various  $H$  as per Boscardin and Cording (1989)

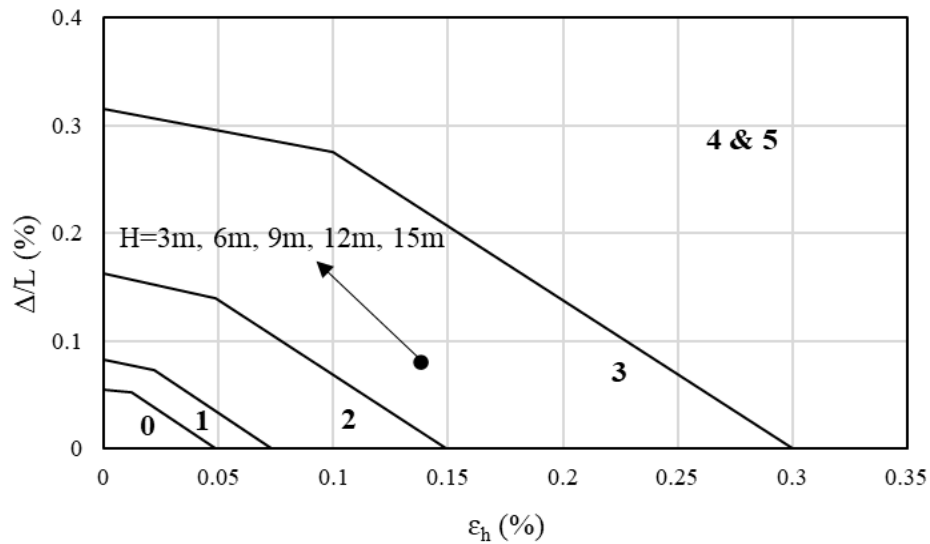


Fig. 6.42 Damage category for various  $H$  as per as per Burland (1995) and Burland et al. (2001)

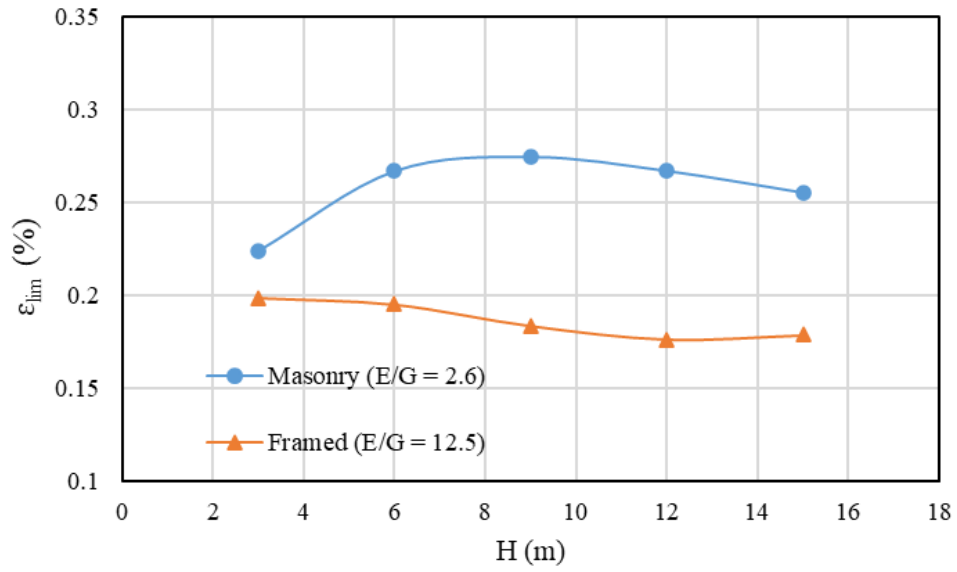


Fig. 6.43 Variation of  $\varepsilon_{lim}$  with  $H$

### 6.3.3.2 Effect of Building Length ( $L$ )

The effect of  $L$  on the  $\varepsilon_{lim}$  and associated damage category has been evaluated by varying  $L$  from 6 m to 21 m in increments of 3 m. A total of six cases has been analysed while maintaining the  $H$  and  $X$  at 9 m and 0 m, respectively, for both masonry and framed buildings. The deformation measures and associated damage categories are presented in Table 6-16 and further illustrated in Fig. 6.44 and Fig. 6.45.

From Table 6-16, it is observed that buildings experience sagging deformation along their entire length when  $L$  ranges from 6 m to 15 m. For longer buildings with  $L$  equals to 18 m and 21 m, a combination of sagging and hogging is observed, with sagging identified as the critical mode of deformation. Across all values of  $L$ , the  $\varepsilon_{bt}$  is found to be the governing deformation measure, exceeding the  $\varepsilon_{dt}$ , for both masonry and framed structures. As shown in Table 6-16 and Fig. 6.46, for masonry buildings,  $\varepsilon_{lim}$  increases by 4.2% as  $L$  increases from 6 m to 12 m, followed by a decrease of 8.5% as  $L$  increases further from 12 m to 18 m. Beyond  $L = 18$  m,  $\varepsilon_{lim}$  stabilises and remains constant. In contrast, for framed buildings,  $\varepsilon_{lim}$  decreases by approximately 26.8% as  $L$  increases from 6 m to 18 m, after which it remains constant. Despite these variations, the damage category remains unchanged at '3 – Moderate' across all six cases for both masonry and framed buildings. It is further noted that, over the entire range of  $L$  examined in this study, the damage categories derived from  $\varepsilon_{lim}$  are consistent with those predicted by the criteria proposed by Boscardin and Cording (1989), Burland (1995), and Burland et al. (2001) for both masonry and framed structures. Similar trends in the variation of  $\varepsilon_{lim}$  with increasing building length have also been reported by other researchers for both masonry and framed structures (Mirhabibi and Soroush, 2020; Franzius et al., 2006).

Table 6-16 Effect of  $L$  on building deformation measures and damage categories

| $L$<br>(m) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\varepsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\varepsilon_{bt}$<br>(%) | $\varepsilon_{dt}$<br>(%) | $\varepsilon_{lim}$<br>(%) | Category based on |               |                     |
|------------|------|------|--------------------|---------------|-----------------------------|-------------------|---------------------------|---------------------------|----------------------------|-------------------|---------------|---------------------|
|            |      |      |                    |               |                             |                   |                           |                           |                            | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$ |
| 6          | M    | Sag. | 25.24              | 0.0009        | 0.2245                      | 0.0459            | 0.2760                    | 0.2329                    | 0.2760                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.0009        | 0.2245                      | 0.0459            | 0.2364                    | 0.2348                    | 0.2364                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 9          | M    | Sag. | 25.24              | 0.0013        | 0.1882                      | 0.0638            | 0.2811                    | 0.2023                    | 0.2811                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.0013        | 0.1882                      | 0.0638            | 0.2122                    | 0.2093                    | 0.2122                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 12         | M    | Sag. | 25.24              | 0.0015        | 0.1684                      | 0.0729            | 0.2876                    | 0.1829                    | 0.2876                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.0015        | 0.1684                      | 0.0729            | 0.2032                    | 0.1955                    | 0.2032                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 15         | M    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.2745                    | 0.1529                    | 0.2745                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.1836                    | 0.1720                    | 0.1836                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 18         | M    | Sag. | 25.24              | 0.0017        | 0.1199                      | 0.0848            | 0.2631                    | 0.1331                    | 0.2631                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            |      | Hog. |                    |               | 0.0675                      | 0.0008            | 0.0675                    | 0.0675                    |                            |                   |               |                     |
|            | F    | Sag. | 25.24              | 0.0017        | 0.1199                      | 0.0848            | 0.1730                    | 0.1567                    | 0.1730                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            |      | Hog. |                    |               | 0.0675                      | 0.0008            | 0.0675                    | 0.0675                    |                            |                   |               |                     |
| 21         | M    | Sag. | 25.24              | 0.0017        | 0.1199                      | 0.0848            | 0.2631                    | 0.1331                    | 0.2631                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            |      | Hog. |                    |               | 0.0588                      | 0.0006            | 0.0589                    | 0.0588                    |                            |                   |               |                     |
|            | F    | Sag. | 25.24              | 0.0017        | 0.1199                      | 0.0848            | 0.1730                    | 0.1567                    | 0.1730                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            |      | Hog. |                    |               | 0.0588                      | 0.0006            | 0.0588                    | 0.0588                    |                            |                   |               |                     |

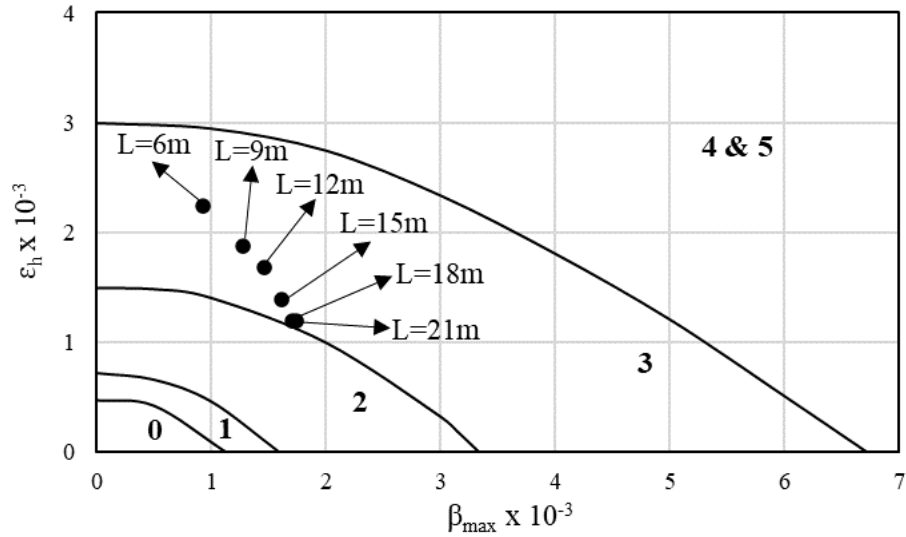


Fig. 6.44 Damage category for various  $L$  as per Boscardin and Cording (1989)

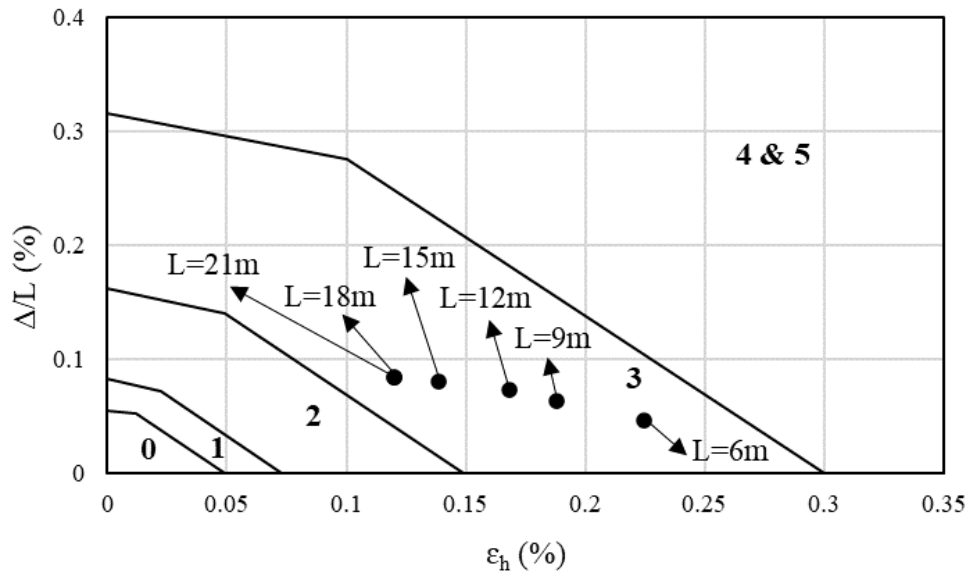


Fig. 6.45 Damage category for various  $L$  as per as per Burland (1995) and Burland et al. (2001)

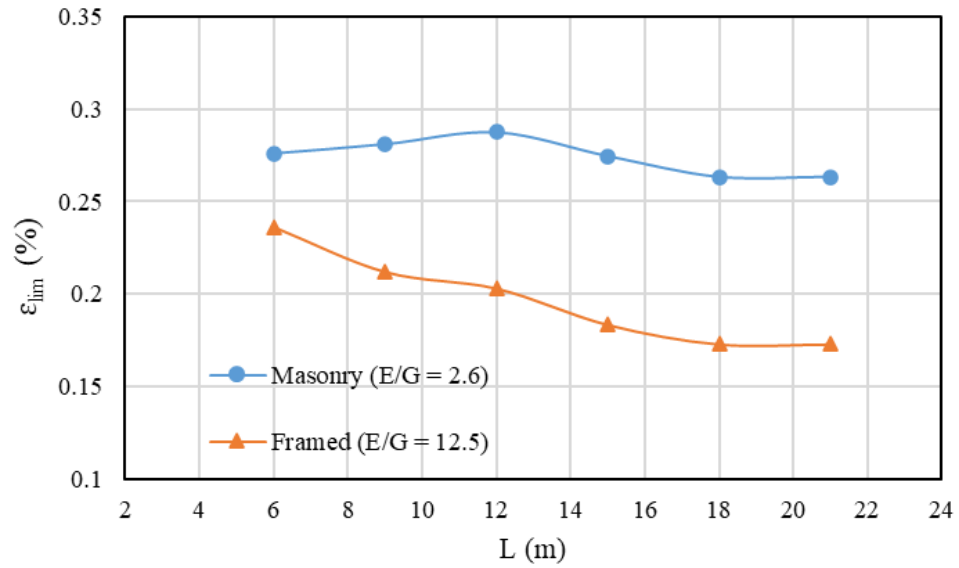


Fig. 6.46 Variation of  $\epsilon_{lim}$  with  $L$

### 6.3.3.3 Effect of Building Location ( $X$ )

The effect of  $X$  on the  $\epsilon_{lim}$  and associated damage category has been investigated by varying  $X$  from 0 m to 24 m in increments of 6 m. A total of five cases has been analysed while maintaining the  $H$  and  $L$  at 9 m and 15 m, respectively, for both masonry and framed buildings. The deformation measures and associated damage categories are summarised in Table 6-17 and further illustrated in Fig. 6.47 and Fig. 6.48.

Table 6-17 Effect of  $X$  on building deformation measures and damage categories

| $X$<br>(m) | Type | Mode | $S_{bmax}$<br>(mm) | $\beta_{max}$ | $\varepsilon_{hmax}$<br>(%) | $\Delta/L$<br>(%) | $\varepsilon_{bt}$<br>(%) | $\varepsilon_{dt}$<br>(%) | $\varepsilon_{lim}$<br>(%) | Category based on |               |                     |
|------------|------|------|--------------------|---------------|-----------------------------|-------------------|---------------------------|---------------------------|----------------------------|-------------------|---------------|---------------------|
|            |      |      |                    |               |                             |                   |                           |                           |                            | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$ |
| 0          | M    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.2745                    | 0.1529                    | 0.2745                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            | F    | Sag. | 25.24              | 0.0016        | 0.1384                      | 0.0803            | 0.1836                    | 0.1720                    | 0.1836                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| 6          | M    | Sag. | 25.24              | 0.0016        | 0.1359                      | 0.0790            | 0.2587                    | 0.1599                    | 0.2587                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            |      | Hog. |                    |               | 0.0757                      | 0.0096            | 0.0804                    | 0.0771                    |                            |                   |               |                     |
|            | F    | Sag. | 25.24              | 0.0016        | 0.1359                      | 0.0790            | 0.1694                    | 0.1742                    | 0.1742                     | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|            |      | Hog. |                    |               | 0.0757                      | 0.0096            | 0.0767                    | 0.0772                    |                            |                   |               |                     |
| 12         | M    | Sag. | 19.5               | 0.0021        | 0.0496                      | 0.0030            | 0.0522                    | 0.0498                    | 0.0999                     | 2- Sl.            | 2- Sl.        | 2- Sl.              |
|            |      | Hog. |                    |               | 0.0801                      | 0.0193            | 0.0999                    | 0.0845                    |                            |                   |               |                     |
|            | F    | Sag. | 19.5               | 0.0021        | 0.0496                      | 0.0030            | 0.0502                    | 0.0498                    | 0.0852                     | 2- Sl.            | 2- Sl.        | 2- Sl.              |
|            |      | Hog. |                    |               | 0.0801                      | 0.0193            | 0.0846                    | 0.0852                    |                            |                   |               |                     |
| 18         | M    | Hog. | 7.664              | 0.0005        | 0.0637                      | 0.0235            | 0.0946                    | 0.0702                    | 0.0946                     | 1- Neg.           | 1- Neg.       | 2- Sl.              |
|            | F    | Hog. | 7.664              | 0.0005        | 0.0637                      | 0.0235            | 0.0712                    | 0.0724                    | 0.0724                     | 1- Neg.           | 1- Neg.       | 1- V. Sl.           |
| 24         | M    | Hog. | 1.23               | 0.0001        | 0.0284                      | 0.0041            | 0.0337                    | 0.0288                    | 0.0337                     | 1- Neg.           | 1- Neg.       | 1- Neg.             |
|            | F    | Hog. | 1.23               | 0.0001        | 0.0284                      | 0.0041            | 0.0297                    | 0.0290                    | 0.0297                     | 1- Neg.           | 1- Neg.       | 1- Neg.             |

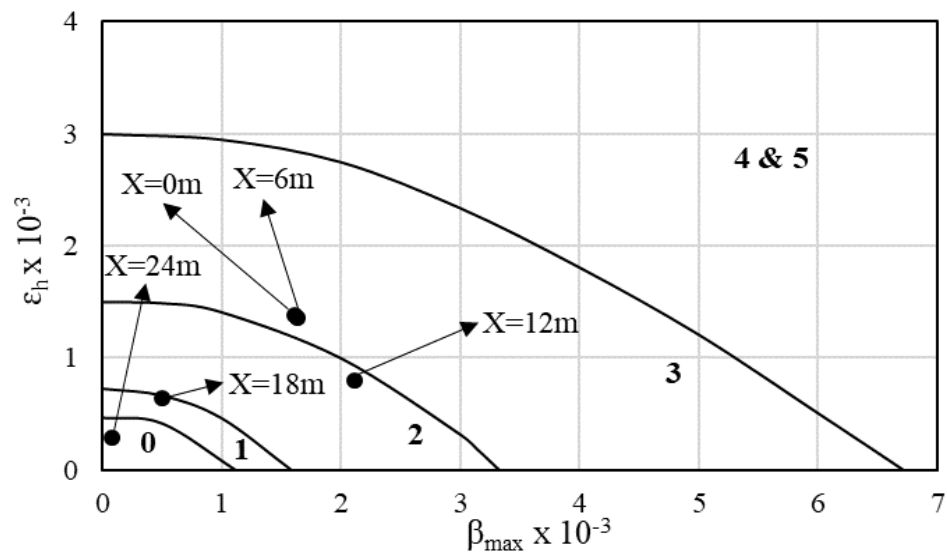


Fig. 6.47 Damage category for various  $X$  as per Boscardin and Cording (1989)

From Table 6-17, it is observed that when  $X$  equals to 0 m, the buildings undergo sagging deformation along their entire length. For  $X$  values 6 m and 12 m, a combination of sagging and hogging is observed, with sagging governing the response for  $X$  equals to 6 m, and hogging governing for  $X$  equals to 12 m. At larger offsets,  $X$  equals to 18 m and 24 m, hogging deformation dominates along the entire building length. For masonry buildings, the  $\varepsilon_{bt}$  is consistently identified as the critical

deformation measure compared with the  $\varepsilon_{dt}$ . For framed buildings,  $\varepsilon_{bt}$  governs at  $X$  equals to 0 m and 24 m, while  $\varepsilon_{dt}$  becomes critical at  $X$  equals to 6 m, 12 m and 18 m. As shown in Table 6-17 and Fig. 6.49,  $\varepsilon_{lim}$  for masonry buildings decreases substantially by approximately 87.7%, as  $X$  increases from 0 m to 24 m, rendering  $\varepsilon_{lim}$  negligible at large offsets. This reduction shifts the damage category from '3 – Moderate' for  $X$  equals to 0 m and 6 m, to '2 – Slight' for  $X$  equals to 12 m and 18 m, and finally '0 – Negligible' for  $X$  equals to 24 m.

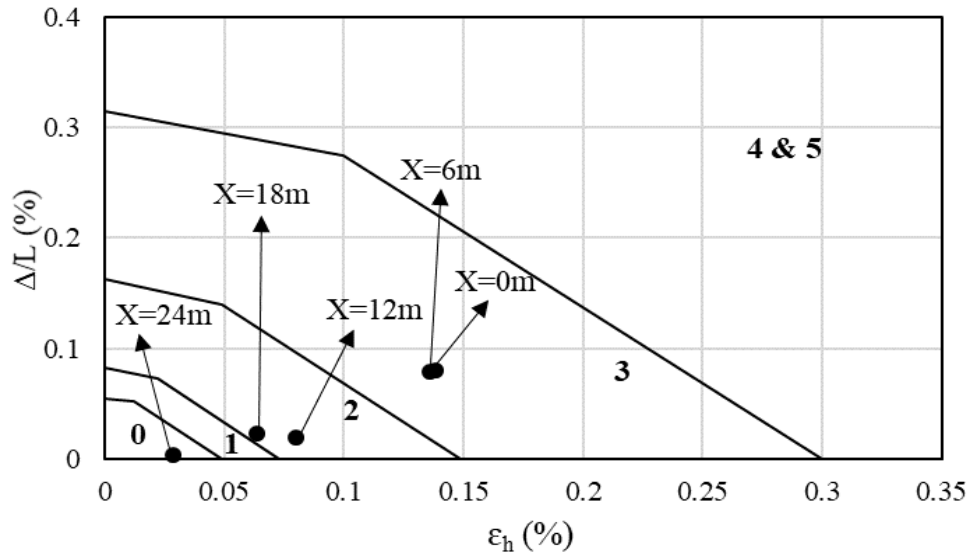


Fig. 6.48 Damage category for various  $X$  as per as per Burland (1995) and Burland et al. (2001)

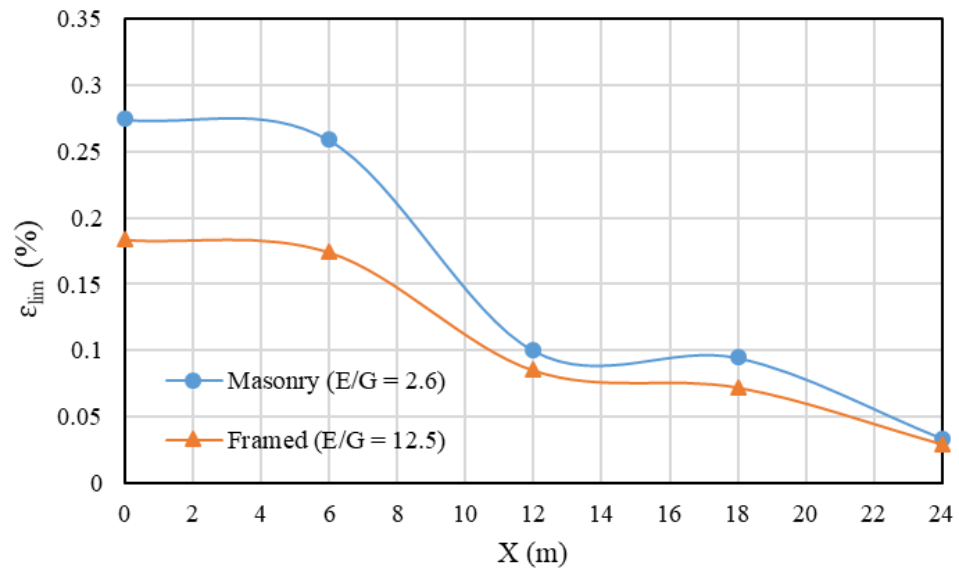


Fig. 6.49 Variation of  $\varepsilon_{lim}$  with  $X$

A similar trend is observed for framed buildings, where  $\varepsilon_{lim}$  decreases by 83.8% as  $X$  increases from 0 m to 24 m. This corresponds to a shift in the damage

category from '3 – Moderate' for  $X$  equals to 0 m and 6 m, to '2 – Slight' for  $X$  equals to 12 m, '1 – Very Slight' for  $X$  equals to 18 m, and finally '0 – Negligible' for  $X$  equals to 24 m. It is further noted that, for the entire range of  $X$  considered, the damage categories derived from  $\varepsilon_{lim}$  are generally consistent with those predicted using the criteria proposed by Boscardin and Cording (1989), Burland (1995), and Burland et al. (2001), with the exception of  $X$  equals to 18 m. At this offset, the damage category determined using Boscardin and Cording (1989) aligns with that obtained for framed buildings, while the damage category determined using Burland (1995) and Burland et al. (2001) corresponds more closely with that of masonry buildings. Overall, increasing  $X$  significantly reduces ground deformations beneath the structure, thereby lowering  $\varepsilon_{lim}$  and the associated damage category, a trend also highlighted by several other researchers (Boldini et al., 2018; Franzius et al., 2006; Giardina et al., 2015; Son, 2015, 2016; Zhao et al., 2019).

#### 6.4 Discussion on Outcomes

1. The increase in tunnel diameter within the studied range significantly increased the  $S_{max}$  by approximately 100% under a constant volume loss of 1.5% imposed on the tunnel cavity. Larger tunnel diameters result in greater excavation-induced ground disturbance and stress redistribution, thereby increasing soil deformation and enlarging the settlement influence zone. This trend can also be verified by Eqn. 2.2, which indicates that the  $S_{max}$  is directly proportional to the square of the tunnel diameter ( $D^2$ ) under a constant percentage of volume loss imposed on the tunnel cavity. Therefore, an increase in tunnel diameter leads to higher ground deformations.
2. The increase in tunnel depth within the studied range significantly reduced the  $S_{max}$ . This reduction can be attributed to greater soil confinement and stress arching effects at larger depths, which dissipate excavation-induced stress redistribution before reaching the ground surface. It is observed that the reduction in settlement was substantial up to a tunnel depth of 20–25 m (Table 6-3). However, beyond this depth, the rate of reduction became comparatively smaller. Therefore, within the studied conditions, a tunnel depth of approximately 20–25 m may be considered an effective axial depth range for reducing settlement-related risks, although the optimum depth remains dependent on site-specific ground conditions, tunnel geometry, and allowable settlement criteria.
3. The increase in volume loss significantly increased the  $S_{max}$ , demonstrating the high sensitivity of ground response to TBM operational control. This substantial increase is attributed to greater excavation-induced ground loss and associated soil deformation around the tunnel cavity. In TBM tunnelling, volume loss is governed by operational factors such as face pressure, shield overcut, tail void grouting, and TBM conicity. Therefore, effective operational control is essential to maintain low volume loss and reduce settlement-induced damage risk to nearby structures.

4. The different settlement responses observed under  $GP$  and  $V_L$  are observed. Under constant  $V_L$  conditions, the percentage of ground loss around the tunnel cavity remains fixed irrespective of the soil stiffness, resulting in only marginal variations in the settlement profile with changing stiffness. In contrast, under constant  $GP$  conditions, the induced ground deformations depend on the stiffness of the surrounding soil. Lower stiffness soils undergo greater deformation under the same grout pressure, resulting in larger settlements, whereas higher stiffness soils exhibit reduced deformations and lower settlement magnitudes. Therefore, increasing  $E'_{ref}$  produces a significantly greater reduction in settlement under constant  $GP$  conditions compared to constant  $V_L$  conditions.

## CHAPTER 7

### SEISMIC ANALYSIS

Tunnels behave differently from surface structures under the effect of seismic forces due to their full enclosure in soil and their significant length (Hashash et al., 2001). Pseudo-static analysis is a simplified method that approximates seismic forces as static loads used to evaluate the effects of seismic loading on tunneling-induced ground deformations. This study presents a two-dimensional (2D) simplified pseudo-static analysis of a soil–tunnel system using PLAXIS 2D, with a tunnel diameter of 6.5 m and at axial depths of 10m, 15 m and 20m. The model, shown in Fig. 5.5 and Fig. 5.6, is used to perform the pseudo-static analysis, and the results, including ground deformations, limiting tensile strains ( $\epsilon_{lim}$ ), and damage categories are compared before and after the application of seismic forces.

In pseudo-static analysis using PLAXIS 2D, horizontal and vertical seismic acceleration coefficients are applied to simulate the effects of earthquake loading. As per IS 1893 (Part 1): 2016, the horizontal seismic coefficient ( $k_h$ ) is determined using the design horizontal seismic acceleration, which is calculated using Eqn. 7.1.

$$k_h = \frac{Z}{2} \times \frac{I_s}{R} \times \frac{S_a}{g} \quad (7.1)$$

Where,  $Z$  is zone factor (based on seismic zone),  $I_s$  is importance factor of the structure,  $R$  is response reduction factor and  $S_a/g$  is design acceleration coefficient (based on soil type and natural period). Study area, Delhi is situated in zone IV as per seismic zonation map given in IS 1893 (Part 1): 2016, therefore  $Z$  can be taken as 0.24.  $I_s$  and  $R$  both can be taken as 1.5. Also, for pseudo-static seismic analysis, a spectral acceleration value of  $S_a/g = 1.0$  can be adopted for rigid or short-period structures (Jain et al., 2025; Wang, 1993), assuming a negligible fundamental period where the response spectrum reaches its maximum. Using this horizontal seismic acceleration ( $k_h$ ) is calculated as  $Z/2$  in lined with as suggested in IRC: 6-2017. IS 1893 (Part 1): 2016 gives the vertical seismic coefficient ( $k_v$ ) as a fraction of  $k_h$  (Eqn. 7.2).

$$k_v = 0.65 \times k_h \quad (7.2)$$

Using Eqn. 7.1 and Eqn. 7.2, values of  $k_h$  and  $k_v$  are calculated as 0.12 and 0.8 respectively. Using these coefficients analysis is done and results are presented here.

## 7.1 Effect of seismic loading on ground deformations and $\varepsilon_{lim}$

Fig. 7.1, Fig. 7.2 and Fig. 7.3 present the vertical settlement ( $S_x$ ) and horizontal displacement ( $H_x$ ) profiles along with the building location and dimensions, following the earthquake for  $z$  equals to 10 m, 15 m and 20 m respectively. It is observed that ground deformations increase significantly in both vertical and horizontal directions after the earthquake, as compared to Fig. 6.20, Fig. 6.14 and Fig. 6.21 for  $z$  equals to 10 m, 15 m and 20 m respectively, indicating greater structural risk due to seismic loading. Table 7-1 along with Fig. 7.4 and Fig. 7.5 summarises the various deformation measures and the corresponding damage categories for all values of  $z$ , for both masonry and framed buildings, under both pre and post-earthquake conditions.

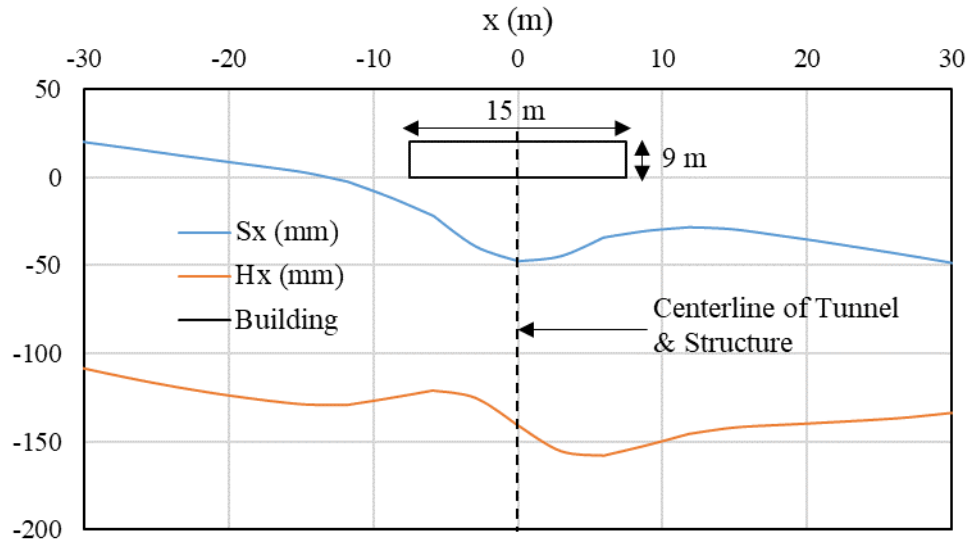


Fig. 7.1 Ground deformations ( $S_x$  and  $H_x$ ) and Building (seismic load,  $z = 10$  m)

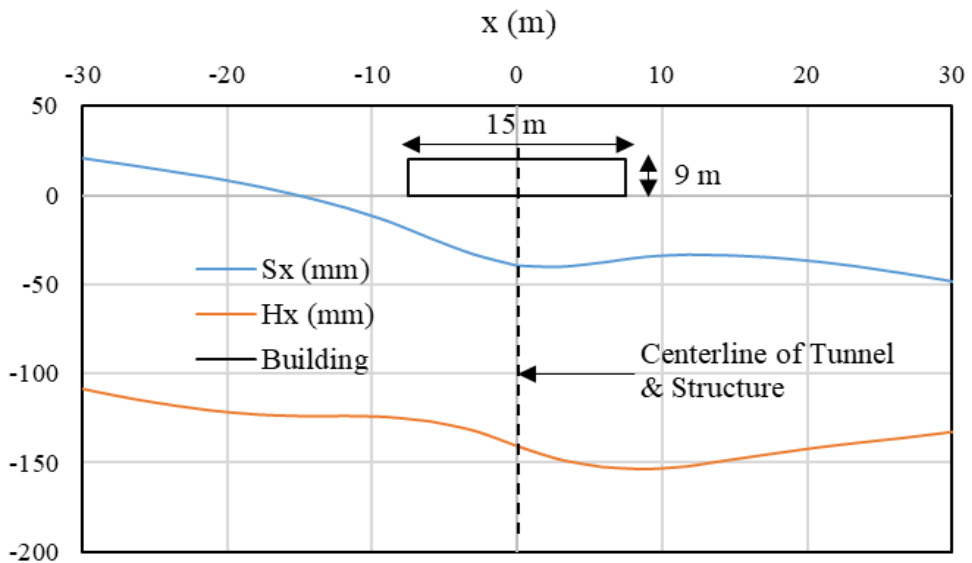


Fig. 7.2 Ground deformations ( $S_x$  and  $H_x$ ) and Building (seismic load,  $z = 15$  m)

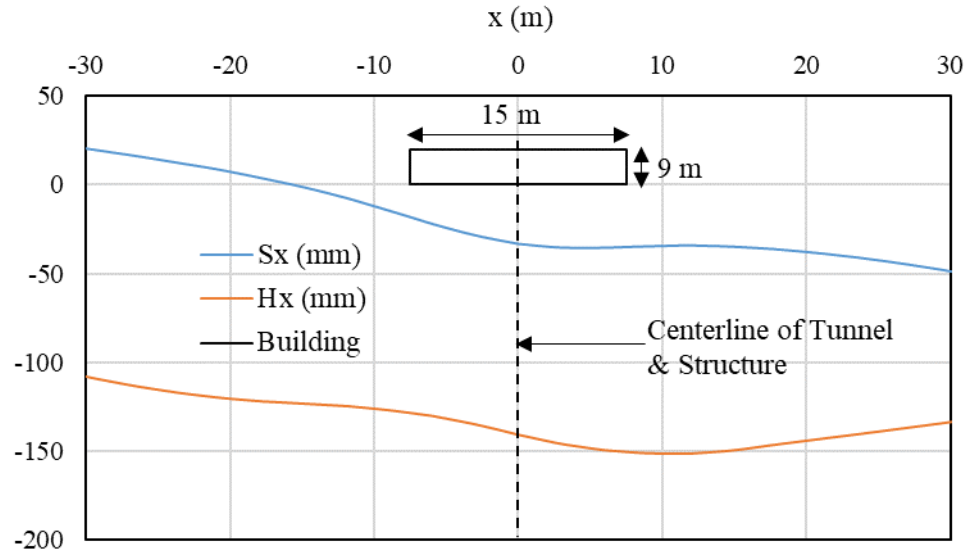


Fig. 7.3 Ground deformations ( $S_x$  and  $H_x$ ) and Building (after earthquake,  $z = 20$  m)

Table 7-1 Effect of Seismic Loading

| Seismic load | Type | Mode | $S_{bmax}$ (mm) | $\beta_{max}$ | $\varepsilon_{hmax}$ (%) | $\Delta/L$ (%) | $\varepsilon_{bt}$ (%) | $\varepsilon_{dt}$ (%) | $\varepsilon_{lim}$ (%) | Category based on |               |                     |
|--------------|------|------|-----------------|---------------|--------------------------|----------------|------------------------|------------------------|-------------------------|-------------------|---------------|---------------------|
|              |      |      |                 |               |                          |                |                        |                        |                         | $S_{bmax}$        | $\beta_{max}$ | $\varepsilon_{lim}$ |
| z = 10 m     |      |      |                 |               |                          |                |                        |                        |                         |                   |               |                     |
| Before       | M    | Sag. | 34.18           | 0.0034        | 0.2692                   | 0.1656         | 0.5387                 | 0.3141                 | 0.5387                  | 2- Sl.            | 2- Sl.        | 4 to 5 S. to V.S.   |
|              |      | Hog. |                 |               | 0.1996                   | 0.0009         | 0.1997                 | 0.1996                 |                         |                   |               |                     |
|              | F    | Sag. | 34.18           | 0.0034        | 0.2692                   | 0.1656         | 0.3473                 | 0.3493                 | 0.3493                  | 2- Sl.            | 2- Sl.        | 4 to 5 S. to V.S    |
|              |      | Hog. |                 |               | 0.1996                   | 0.0009         | 0.1996                 | 0.1996                 |                         |                   |               |                     |
| After        | M    | Sag. | 47.7            | 0.0042        | 0.3118                   | 0.1665         | 0.5826                 | 0.3521                 | 0.5826                  | 2- Sl.            | 2- Sl.        | 4 to 5 S. to V.S    |
|              |      | Hog. |                 |               | 0.1400                   | 0.0027         | 0.1404                 | 0.1400                 |                         |                   |               |                     |
|              | F    | Sag. | 47.7            | 0.0042        | 0.3118                   | 0.1665         | 0.3903                 | 0.3847                 | 0.3903                  | 2- Sl.            | 2- Sl.        | 4 to 5 S. to V.S    |
|              |      | Hog. |                 |               | 0.1400                   | 0.0027         | 0.1404                 | 0.1400                 |                         |                   |               |                     |
| z = 15 m     |      |      |                 |               |                          |                |                        |                        |                         |                   |               |                     |
| Before       | M    | Sag. | 25.24           | 0.0016        | 0.1384                   | 0.0803         | 0.2745                 | 0.1529                 | 0.2745                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|              | F    | Sag. | 25.24           | 0.0016        | 0.1384                   | 0.0803         | 0.1836                 | 0.1720                 | 0.1836                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |
| After        | M    | Sag. | 40              | 0.0027        | 0.1866                   | 0.0795         | 0.3215                 | 0.1976                 | 0.3215                  | 2- Sl.            | 2- Sl.        | 4 to 5 S. to V.S.   |
|              | F    | Sag. | 40              | 0.0027        | 0.1866                   | 0.0795         | 0.2314                 | 0.2128                 | 0.2314                  | 2- Sl.            | 2- Sl.        | 3- Mod.             |
| z = 20 m     |      |      |                 |               |                          |                |                        |                        |                         |                   |               |                     |
| Before       | M    | Sag. | 18.08           | 0.0009        | 0.0975                   | 0.0432         | 0.1708                 | 0.1037                 | 0.1708                  | 2- Sl.            | 1- Neg.       | 3- Mod.             |
|              | F    | Sag. | 18.08           | 0.0009        | 0.0975                   | 0.0432         | 0.1218                 | 0.1122                 | 0.1218                  | 2- Sl.            | 1- Neg.       | 2- Sl.              |
| After        | M    | Sag. | 33              | 0.0020        | 0.1474                   | 0.0440         | 0.2220                 | 0.1517                 | 0.2220                  | 2- Sl.            | 2- Sl.        | 3- Mod.             |
|              | F    | Sag. | 33              | 0.0020        | 0.1474                   | 0.0440         | 0.1721                 | 0.1580                 | 0.1721                  | 2- Sl.            | 2- Sl.        | 3- Mod.             |

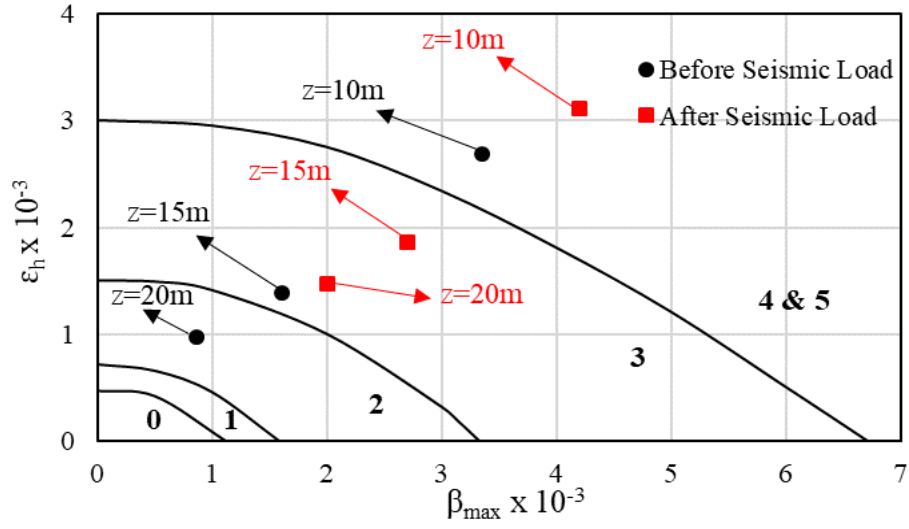


Fig. 7.4 Damage category for various  $z$  and seismic loading as per Boscardin and Cording (1989)

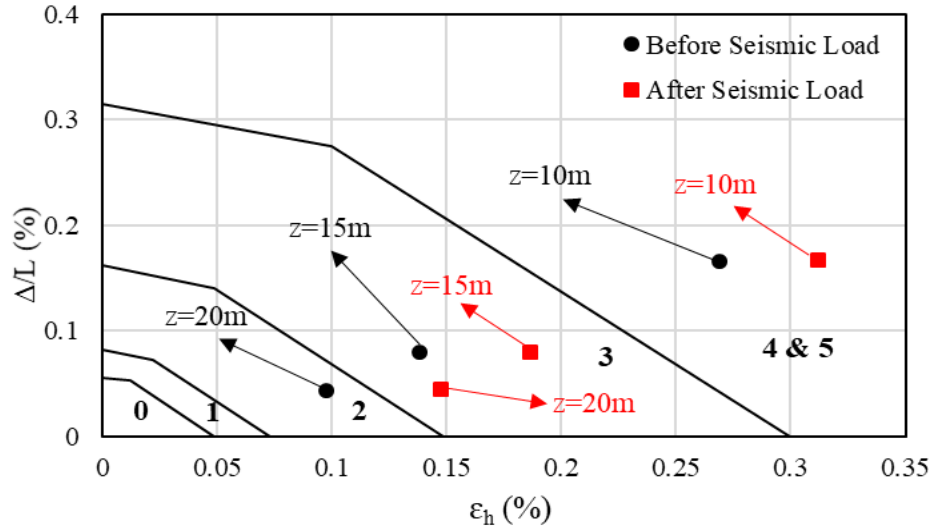


Fig. 7.5 Damage category for various  $z$  and seismic loading as per Burland (1995) and Burland et al. (2001)

It is observed that for all tunnel depths considered, both masonry and framed buildings experience significant increases in  $\epsilon_{hmax}$ ,  $S_{bmax}$ ,  $\beta_{max}$  and  $\epsilon_{lim}$  after seismic loading. These increments result in a shift of damage categories towards more severe levels. Among these, the damage category based on  $\epsilon_{lim}$  is identified as the critical one and is consistent with the damage classifications proposed by Boscardin and Cording (1989), Burland (1995) and Burland et al. (2001) for both masonry and framed buildings, except in the case of masonry building with  $z$  equals to 15. Table 7-2 presents the relative change in  $S_{bmax}$  and  $\epsilon_{lim}$  for masonry and framed buildings due to seismic loading at different tunnel depths.

It is observed that with increasing tunnel depth, the absolute increase in  $S_{bmax}$  and  $\varepsilon_{lim}$  is not significant whereas the percentage increases in  $S_{bmax}$  and  $\varepsilon_{lim}$  consistently rise for both masonry and framed buildings. This trend is primarily attributed to the lower pre-earthquake (static) settlement and strain values at greater tunnel depths, making the seismic-induced increment proportionally more significant. Hence, within the investigated conditions of the present study, the relative influence of seismic loading on damage potential becomes more pronounced at greater tunnel depths when compared to static conditions. Similarly the percentage increase in  $\varepsilon_{lim}$  found to be greater for framed buildings than for masonry buildings, suggesting a comparatively higher relative sensitivity of framed structures to seismic-induced tunnel deformations with respect to their static condition. Similar trend of increasing ground deformations and strains is observed by others (Azadi and Hosseini, 2014; Singh et al., 2017; Zlatanović et al., 2015; Cilingir and Madabhushi, 2011).

Table 7-2 Relative change in  $S_{bmax}$  and  $\varepsilon_{lim}$  due to Seismic loading

| $z$ (m) | increase in $S_{bmax}$ | % increase in $S_{bmax}$ | increase in $\varepsilon_{lim}$ (M) | % increase in $\varepsilon_{lim}$ (M) | increase in $\varepsilon_{lim}$ (F) | % increase in $\varepsilon_{lim}$ (F) |
|---------|------------------------|--------------------------|-------------------------------------|---------------------------------------|-------------------------------------|---------------------------------------|
| 10      | 13.52                  | 39.6                     | 0.0439                              | 8.15                                  | 0.0410                              | 11.73                                 |
| 15      | 14.76                  | 58.5                     | 0.0470                              | 17.12                                 | 0.0478                              | 26.03                                 |
| 20      | 14.92                  | 82.5                     | 0.0512                              | 29.9                                  | 0.0503                              | 41.2                                  |

## CHAPTER 8

### CONCLUSION, FUTURE SCOPE, AND SOCIAL IMPACT

#### 8.1 Conclusion

This research examined tunneling-induced ground deformations and their impacts on nearby buildings through an integrated analytical–numerical approach, with specific reference to the Delhi–Meerut RRTS project. Ground deformations were predicted using step-by-step tunnel construction simulations in PLAXIS 2D. Results are validated against empirical approaches and subsequently applied to building foundations using the Limiting Tensile Strain Method (LTSM) for determination of associated risk to adjacent structures. Parametric studies on tunnel, soil, and structural factors, along with the influence of seismic loading, were systematically evaluated. The key conclusions drawn from this research are summarised below.

- PLAXIS 2D demonstrated strong reliability in predicting ground surface deformations in layered soils, showing good agreement with established empirical approaches.
- Tunnel diameter had a pronounced effect on ground and structural response. Increasing the tunnel diameter from 5.5 m to 7.5 m caused a 66.4% rise in maximum surface settlement, while limiting tensile strains increased by 70% for masonry buildings and 71.8% for framed buildings, elevating their respective damage categories.
- Tunnel depth played a critical role in reducing deformations. Increasing the tunnel axis depth from 10 m to 20 m reduced maximum surface settlement by 47.1%. Correspondingly, limiting tensile strains decreased by 68.3% in masonry buildings and 65.1% in framed buildings, thereby lowering the predicted damage category.
- Volume loss was found to be one of the most sensitive parameters. Doubling the volume loss from 1% to 2% resulted in an 81.2% increase in maximum surface settlement, with limiting tensile strains rising by 66.6% in masonry structures and 78.8% in framed structures, significantly escalating the associated damage risk. Increasing grout pressure was found to be highly effective in reducing volume loss, thereby minimizing ground deformations and mitigating tunneling-induced damage. Hence, proper tail void grouting is essential as a construction control measure to keep volume loss within safe limits.

- An increase in undrained shear strength from 0.75 to 1.25 times the base values, along with a  $4^\circ$  increase in friction angle for the respective soil layers, resulted in an 18.16% reduction in maximum surface settlement. Correspondingly, the limiting tensile strains decreased by 10.8% in masonry buildings and 11% in framed buildings.
- An increase in soil stiffness from 0.5 to 1.5 times the base values for the respective layers led to a 15.85% reduction in maximum surface settlement, with limiting tensile strains decreasing by 30.3% in masonry buildings and 27% in framed buildings.
- Both soil strength and stiffness moderately influenced ground deformations and structural response without significantly altering the overall damage category. However, shear strength had a more dominant effect on settlement control, while stiffness was more influential in governing structural response.
- Based on the parametric analyses, the governing parameters can be ranked according to their relative influence on tunnel-induced response. For both maximum surface settlement and limiting tensile strains, the order of influence is found to be: Volume Loss  $\approx$  Tunnel Depth  $>$  Soil Stiffness, indicating that both volume loss and tunnel depth exerted a comparatively stronger effect on settlement than soil stiffness.
- Masonry buildings experienced up to 55% higher bending tensile strains compared to framed buildings, highlighting their greater vulnerability to tunneling-induced damage. In contrast, framed buildings became more critical in shear response, particularly with increasing height.
- The limiting tensile strains in masonry buildings increase as the height-to-length ratio ( $H/L$ ) rises up to 0.75. Beyond this threshold, the limiting tensile strains begin to decrease with further increases in the ratio. For framed buildings the limiting tensile strains decrease as the height-to-length ratio ( $H/L$ ) rises up 0.8. Beyond this value, the limiting tensile strains exhibit a slight increase with further increases in the ratio, rendering framed buildings more critical in shear beyond this threshold. However, structure parameters including building height and length, do not appear to impact the damage category.
- An increase in the horizontal offset from the tunnel centerline to 24 m resulted in a reduction of limiting tensile strains by 87.7% and 83.8% in masonry and framed buildings respectively along with a corresponding reduction in damage category. This indicates that the location of a building relative to the tunnel alignment is a critical factor in mitigating tunneling-induced impacts.

- Seismic loading significantly amplifies tunneling-induced deformations, with notable increases in maximum surface settlement and limiting tensile strains for both masonry and framed buildings. These increments shift the damage categories to more severe levels, with limiting tensile strain-based classification emerging as the most critical.
- Where the predicted risk of damage remains unacceptable, mitigation measures can be introduced, which may include internal structural measures (e.g., wall thickening, strut stiffening, or sequencing changes), external structural measures (e.g., underpinning or structural strengthening), and ground treatment methods (e.g., jet grouting or compensation grouting). This stage therefore provides the most comprehensive framework for safeguarding structures located within zones of significant tunneling-induced ground movements.

In summary, the study concludes that structural stiffness, building location, and tunnel parameters exert the most significant influence on limiting tensile strains and the resulting damage categories, whereas building height, length and soil parameters have comparatively lesser impact.

## 8.2 Recommended Threshold Values for Practical BRA Design in Indian Urban Tunnelling

Based on the present numerical analyses and observed transition in damage categories, the following threshold values are recommended for practical implementation of the proposed Building Risk Assessment (BRA) framework for Indian tunnelling projects.

Table 8-1 Recommended Threshold for  $V_L$

| $V_L$ (%) | Recommended Risk Level          |
|-----------|---------------------------------|
| < 1       | Low Risk                        |
| 1 - 2     | Moderate Risk                   |
| > 2       | High Risk — mitigation required |

Table 8-2 Recommended Threshold for  $z$

| $z/D$ | Recommended Risk Level          |
|-------|---------------------------------|
| > 3   | Low Risk                        |
| 2 - 3 | Moderate Risk                   |
| < 2   | High Risk — mitigation required |

Table 8-3 Recommended Threshold for  $X$

| $X/D$   | Recommended Risk Level          |
|---------|---------------------------------|
| $> 2$   | Low Risk                        |
| $1 - 2$ | Moderate Risk                   |
| $< 1$   | High Risk — mitigation required |

Table 8-4 Recommended Threshold for  $\varepsilon_{lim}$

| $\varepsilon_{lim}(\%)$ | Recommended Risk Level          |
|-------------------------|---------------------------------|
| $> 0.15$                | Low Risk                        |
| $0.15 - 0.3$            | Moderate Risk                   |
| $< 0.3$                 | High Risk — mitigation required |

### 8.3 Future Scope

While the present research provides valuable insights into tunneling-induced building damage, several simplifications and assumptions were adopted to ensure tractability. Future studies can address these limitations to enhance the accuracy and applicability of results:

#### 8.3.1 Adoption of Advanced Soil Models

The present study employed the Mohr–Coulomb (MC) soil model for its simplicity and wide acceptance in preliminary tunneling design. Future research should incorporate advanced constitutive models, such as the Hardening Soil or Hardening Soil with Small-Strain (HSsmall) models, which capture stress-path dependency, stiffness variation, and non-linear soil behaviour more realistically. This would improve the accuracy of predicted deformations and settlement profiles.

#### 8.3.2 Three-Dimensional Modelling

Since tunneling is inherently a three-dimensional process, future investigations should employ 3D numerical analyses to capture longitudinal effects, stress redistribution, and complex soil–structure interaction mechanisms. While 2D plane strain analyses provide reliable predictions of transverse settlements, extending the study to 3D would yield more comprehensive insights, particularly for shorter tunnels or variable-depth alignments.

### 8.3.3 Seismic–Soil–Structure Interaction (SSI)

The present research used the Limiting Tensile Strain Method (LTSM), which assumes that buildings deform according to free-field ground movements, neglecting SSI effects. Future work should explicitly incorporate SSI in numerical models to evaluate the redistribution of strains and potential reduction in building damage, thereby providing a more realistic assessment of structural vulnerability. In addition, dynamic soil–structure interaction models should be employed to better assess earthquake-induced risks in tunneling projects.

### 8.3.4 Refined Validation Approaches

Model outputs were validated using empirical–analytical methods such as Peck’s Gaussian settlement profile and related frameworks, which themselves have inherent limitations. Future studies should expand validation by comparing results with centrifuge modelling, large-scale laboratory tests, or advanced field instrumentation to improve confidence in the predictions. Incorporating in-situ monitoring data such as settlements, structural strains, and crack surveys, combined with back-analysis, would enable more robust model calibration and strengthen the link between numerical predictions and actual building performance.

### 8.3.5 Probabilistic Risk Assessment

Current analyses are deterministic. Incorporating probabilistic frameworks and reliability-based methods could better account for uncertainties in soil properties, construction parameters, and building variability.

### 8.3.6 Integration of Machine Learning for Tunnel-Induced Risk Assessment

The present study adopts a deterministic, simulation-based framework for evaluating tunnel-induced ground deformation and building damage. Future research may explore the integration of Machine Learning techniques to develop a faster, probabilistic, and adaptive risk assessment framework for urban tunneling applications. Machine Learning models can be trained using numerical simulation outputs together with field monitoring data, including surface settlement, volume loss, structural response, and geotechnical instrumentation records, to improve prediction accuracy and computational efficiency. Such an approach may enable rapid forecasting of tunneling-induced impacts under varying geological and construction conditions, thereby improving real-time decision-making during tunnel construction. The successful implementation of ML-based frameworks would require interdisciplinary collaboration among geotechnical engineers, data science researchers, tunnel construction agencies, and instrumentation specialists to ensure the

availability of reliable datasets and standardized monitoring protocols for model training and validation.

## 8.4 Social Impact

The findings of this study extend beyond academic and engineering significance, carrying substantial social implications for urban infrastructure development and community safety. The key social impacts are outlined below:

### 8.4.1 Enhanced Public Safety

By identifying how tunneling parameters, soil conditions, and structural characteristics influence building response, the study contributes directly to reducing risks of structural damage and potential collapse. This ensures the safety of residents, workers, and commuters in densely populated urban areas where tunneling projects are increasingly undertaken.

### 8.4.2 Protection of Heritage and Vulnerable Structures

Many Indian cities, including Delhi, Mumbai, and Kolkata etc., have historically significant and structurally fragile buildings located near proposed tunnel alignments. The insights from this research help in assessing damage potential more accurately and implementing protective measures, thereby safeguarding cultural heritage and community assets.

### 8.4.3 Minimization of Social Disruption

Unplanned or poorly assessed tunneling activities often lead to cracks, service interruptions, and the displacement of residents. The predictive framework developed in this thesis enables proactive identification of at-risk zones, minimizing forced relocation and associated social stress, while maintaining trust in large-scale public infrastructure projects.

### 8.4.4 Support for Sustainable Urban Development

Urban tunneling, particularly for metro and RRTS systems, is critical for easing congestion and reducing environmental pollution. This research facilitates the safe execution of such projects, thereby supporting sustainable mobility and improving the quality of life in rapidly urbanizing regions.

#### 8.4.5 Economic and Livelihood Security

Damage to residential and commercial buildings not only imposes repair costs but also disrupts livelihoods. By providing a robust risk assessment framework, the study reduces the probability of property damage and business interruptions, ultimately protecting household and community-level economic stability.

#### 8.4.6 Informed Policy and Regulatory Frameworks

The outcomes of this study can serve as a reference for policymakers and urban planners to strengthen tunneling-related safety codes and guidelines in India. Establishing evidence-based standards will enhance accountability in construction practices and ensure equitable protection for vulnerable communities.

#### 8.4.7 Community Confidence in Infrastructure Projects

Public opposition to underground construction often stems from fear of structural damage and inadequate risk communication. By demonstrating reliable and transparent methods for predicting and mitigating risks, this study contributes to building public trust and acceptance of large-scale infrastructure projects.

## REFERENCES

1. Agarwal, J. and Sarkar, R., 2024. Greenfield settlements due to tunnelling using tunnel boring machine (TBM) in layered soils: a parametric study. *Sādhanā*, 49(1), p.75.
2. Agarwal, J. and Sarkar, R., 2025. Building risk assessment of existing buildings near a proposed tunnel. *Sādhanā*, 50(3), p.201.
3. Ahmed, M. and Iskander, M., 2011. Analysis of tunneling-induced ground movements using transparent soil models. *Journal of Geotechnical and Geoenvironmental Engineering*, 137(5), pp.525-535.
4. Al-Omari, R.R., Al-Soud, M.S. and Al-Zuhairi, O.I., 2019. Effect of tunnel progress on the settlement of existing piled foundation. *Studia Geotechnica et Mechanica*, 41(2), pp.102-113
5. Amorosi, A., Boldini, D., De Felice, G., Malena, M. and Sebastianelli, M., 2014. Tunnelling-induced deformation and damage on historical masonry structures. *Géotechnique*, 64(2), pp.118-130.
6. Aoyagi, T., 1995. *Representing settlement for soft ground tunneling* (Doctoral dissertation, Massachusetts Institute of Technology). [Online] Available at: <https://dspace.mit.edu/bitstream/handle/1721.1/35037/33669131-MIT.pdf?sequence=2> [accessed 10 November 2025]
7. Armaghani, D.J., 2015. *Tunnel boring machine performance prediction in tropically weathered granite through empirical and computational methods* (Doctoral dissertation, Universiti Teknologi Malaysia). [Online] Available at: <https://openscience.utm.my/items/29d87029-ebd9-465b-957c-528492d65ead/full> [accessed 10 November 2025]
8. Aswathy, M.S., Vinoth, M. and Mittal, A., 2022. Impact of governing factors on prediction of tunneling induced surface settlement in young alluvium deposit. *Indian Geotechnical Journal*, 52(1), pp.13-27.
9. Attewell, P.B., 1978. Ground movements caused by tunneling in soil, Proceedings of International Conference on Large Ground Movements and Structures, London, Pentech, 1978, pp. 812–948.
10. Attewell, P.B. and Farmer, I.W., 1974. Ground disturbance caused by shield tunnelling in a stiff, overconsolidated clay. *Engineering Geology*, 8(4), pp.361-381.
11. Athar, M.F., Sadique, M.R., Alsabhan, A.H. and Alam, S., 2022. Ground settlement due to tunneling in cohesionless soil. *Applied Sciences*, 12(7), p.3672.

12. Azadi, M. and Hosseini, S.M.M., 2014. The influence of tunnels on the nearby buildings under seismic loadings conditions, case study: Shiraz underground-Iran, *Proceedings of 14th World Conference on Earthquake Engineering*, pp. 1-8.
13. Basile, F., 2014. Effects of tunnelling on pile foundations. *Soils and Foundations*, 54(3), pp.280-295.
14. Bobet, A., 2001. Analytical solutions for shallow tunnels in saturated ground. *Journal of engineering mechanics*, 127(12), pp.1258-1266.
15. Boldini, D., Losacco, N., Bertolin, S. and Amorosi, A., 2018. Finite element modelling of tunnelling-induced displacements on framed structures. *Tunnelling and underground space technology*, 80, pp.222-231.
16. Boone, S.J., 1996. Ground-movement-related building damage. *Journal of geotechnical engineering*, 122(11), pp.886-896.
17. Boonpichetvong, M., Netzel, H. and Rots, J.G., 2005. Numerical analysis of soil foundation-building interaction due to tunnelling, *Proceedings of the 5th International Symposium (IS-A'dam 2005)-Geotechnical aspects of underground construction in soft ground*, Amsterdam, Netherlands, December, 2005, pp. 479-485.
18. Boscardin, M.D., 1980. *Building response to excavation-induced ground movements*. University of Illinois at Urbana-Champaign.
19. Boscardin, M.D. and Cording, E.J., 1989. Building response to excavation-induced settlement. *Journal of Geotechnical engineering*, 115(1), pp.1-21.
20. Bowles, J.E., 1996. *Foundation analysis and design*. 5th edn. New York: McGraw-Hill.
21. Burd, H.J., Houlsby, G.T., Augarde, C.E. and Liu, G., 2000. Modelling tunnelling-induced settlement of masonry buildings. *Proceedings of the institution of civil engineers-geotechnical engineering*, 143(1), pp.17-29.
22. Burland, J.B., 1995. Assessment of damage to buildings due to tunnelling and excavations, *Proceedings of the 1st International Conference on Earthquake Geotechnical Engineering*, IS-Tokyo'95.
23. Burland, J.B., Broms, B.B. and De Mello, V.F., 1977. Behaviour of foundations and structures, *Proceedings of the 9th International Conference on State-of-the-Art Report on Soil Mechanics and Foundation Engineering II*, Tokyo, Japan, pp.495-546.
24. Burland, J.B. and Wroth, C., 1974. Settlement of buildings and associated damage. *Proceedings of the Conference on Settlement of Structures*, Cambridge, London, England, pp.611-654.
25. Burland, J.B., Standing, J.R. and Jardine, F.M., 2001. Building response to tunneling. *Case studies from construction of the jubilee line extension*. London, pp.134-145.

26. Camós, C., 2015. *Sensitivity of urban buildings to tunneling-induced settlements*. (Doctoral dissertation, Department d'Enginyeria de la Construcció). [Online] Available at: [https://www.academia.edu/15316251/Sensitivity\\_of\\_Urban\\_Buildings\\_to\\_Tunneling\\_Induced\\_Settlement\\_PhD\\_Dissertation](https://www.academia.edu/15316251/Sensitivity_of_Urban_Buildings_to_Tunneling_Induced_Settlement_PhD_Dissertation) [accessed 10 November 2025]
27. Camós, C., Molins, C. and Arnau, O., 2014. Case study of damage on masonry buildings produced by tunneling induced settlements. *International Journal of Architectural Heritage*, 8(4), pp.602-625.
28. Celestino, T.B., Gomes, R.A. and Bortolucci, A.A., 2000. Errors in ground distortions due to settlement trough adjustment. *Tunnelling and underground space technology*, 15(1), pp.97-100.
29. Chakeri, H. and Ünver, B., 2014. A new equation for estimating the maximum surface settlement above tunnels excavated in soft ground. *Environmental earth sciences*, 71(7), pp.3195-3210.
30. Cho, S.H., Kim, J., Won, J. and Kim, M.K., 2017. Effects of jack force and construction steps on the change of lining stresses in a TBM tunnel. *KSCE Journal of Civil Engineering*, 21(4), pp.1135-1146.
31. Chen, J., 2011. *The Finite Element Method: Its Fundamentals and Applications in Engineering*. World Scientific Publishing Company.
32. Clarke, J.A. and Laefer, D.F., 2014. Evaluation of risk assessment procedures for buildings adjacent to tunnelling works. *Tunnelling and Underground Space Technology*, 40, pp.333-342.
33. Cilingir, U. and Madabhushi, S.G., 2011. Effect of depth on seismic response of circular tunnels. *Canadian Geotechnical Journal*, 48(1), pp.117-127.
34. Comodromos, E.M., Papadopoulou, M.C. and Konstantinidis, G.K., 2014. Numerical assessment of subsidence and adjacent building movements induced by TBM-EPB tunneling. *Journal of Geotechnical and Geoenvironmental Engineering*, 140(11), p.04014061.
35. Cording, E.J., 1991. Control of ground movements around tunnels in soil, Proceedings of the Ninth Pan American Conference, Vina del Mar, Chile. Sociedad Chilena de Geotechnica, August, Vol. 4, pp. 2195-2244.
36. Cording, E.J., Long, J.L., Son, M., Laefer, D. and Ghahreman, B., 2010, Assessment of excavation-induced building damage. Proceedings of the Earth Retention Conference 3, August, 2010, pp. 101-120.
37. Dalgic, K.D., Hendriks, M.A. and Ilki, A., 2017. Building response to tunnelling-and excavation-induced ground movements: Using transfer functions to review the

limiting tensile strain method. *Structure and Infrastructure Engineering*, 14(6), pp.766-779.

38. Devriendt, M., 2011. Tunnel induced ground movements and control instrumentation. In *Course on Tunnel Design & Construction* (pp. 1-17). Thomas Telford Publishing.
39. Dias, T.G.S. and Bezuijen, A., 2015. TBM pressure models: Observations, theory and practice, *Proceedings of the 15th Pan-American Conference on Soil Mechanics and Geotechnical Engineering – Geotechnical Synergy*, Buenos Aires, pp. 347–374.
40. Dimmock, P.S. and Mair, R.J., 2008. Effect of building stiffness on tunnelling-induced ground movement. *Tunnelling and Underground Space Technology*, 23(4), pp.438-450.
41. Dutta, S. and Ghosh, A., 2025. Pasternak's two-parameter non-linear soil model for prediction of pile response due to tunnelling induced ground movement. *International Journal of Geotechnical Engineering*, 19(1-3), pp.106-123.
42. ElMouchi, A.M., Hassan, A.M. and Amer, M.I., 2018. Performance of an existing raft foundation rested on granular soils due to TBM tunneling process. *Arabian Journal of Geosciences*, 11(6), p.108.
43. El-Nahhas, F.M., El-Mossallamy, Y.M. and El-Shamy, A.A., 2015. 3D analysis of ground settlement induced by mechanized tunnelling, *Proceedings of the 14th International conference on structural and geotechnical engineering*, December.
44. El Naggar, A., Youssef, M.A. and El Naggar, H., 2020. Effect of tunneling on shallow foundations, *Proceedings of the World Congress on Advances in Civil, Environmental & Materials Research (ACEM20)*, GECE, Seoul, Korea, August, 2020.
45. Fargnoli, V., Boldini, D. and Amorosi, A., 2013. TBM tunnelling-induced settlements in coarse-grained soils: The case of the new Milan underground line 5. *Tunnelling and Underground Space Technology*, 38, pp.336-347.
46. Farrell, R., Mair, R., Sciotti, A. and Pigorini, A., 2014. Building response to tunnelling. *Soils and foundations*, 54(3), pp.269-279.
47. Fattah, M.Y., Shlash, K.T. and Salim, N.M., 2011. Settlement trough due to tunneling in cohesive ground. *Indian Geotechnical Journal*, 41(2), pp.64-75.
48. Fattah, M.Y., Shlash, K.T. and Salim, N.M., 2013. Prediction of settlement trough induced by tunneling in cohesive ground. *Acta Geotechnica*, 8(2), pp.167-179.
49. Finno, R.J., Voss Jr, F.T., Rossow, E. and Blackburn, J.T., 2005. Evaluating damage potential in buildings affected by excavations. *Journal of geotechnical and geoenvironmental engineering*, 131(10), pp.1199-1210.

50. Fok, P., Neo, B.H., Goh, K.H. and Wen, D., 2012. Assessing the impact of excavation-induced movements on adjacent buildings. *The IES Journal Part A: Civil & Structural Engineering*, 5(3), pp.195-203.
51. Franzius, J.N., Potts, D.M. and Burland, J.B., 2006. The response of surface structures to tunnel construction. *Proceedings of the Institution of Civil Engineers-Geotechnical Engineering*, 159(1), pp.3-17.
52. Franza, A., Marshall, A.M. and Zhou, B., 2019. Greenfield tunnelling in sands: the effects of soil density and relative depth. *Géotechnique*, 69(4), pp.297-307.
53. Fu, J., Yang, J., Klapperich, H. and Wang, S., 2016. Analytical prediction of ground movements due to a nonuniform deforming tunnel. *International Journal of Geomechanics*, 16(4), p.04015089.
54. Fu, J., Yu, Z., Wang, S. and Yang, J., 2018. Numerical analysis of framed building response to tunnelling induced ground movements. *Engineering Structures*, 158, pp.43-66.
55. Gaba, A.R., Simpson, B., Beadman, D.R. and Powrie, W., 2003. Embedded retaining walls: guidance for economic design. *Proceedings-institution of civil engineers geotechnical engineering*, pp.13-16.
56. Ghahremannejad, B., Surjadinata, J., Poon, B. and Carter, J.P., 2006. Effects of tunnelling on settlements and structural forces within model pile foundations. *Proceedings of the International Conference on Physical Modelling in Geotechnics*, Hong Kong.
57. Giardina, G., 2013. *Modelling of settlement induced building damage*. (Doctoral dissertation, Ipskamp Drukkers, Enschede, The Netherlands). [Online] Available at: <https://repository.tudelft.nl/record/uuid:8523bca8-91d3-4da5-abd2-2ff2cfddf9a0> [accessed 10 November 2025]
58. Giardina, G., DeJong, M.J. and Mair, R.J., 2015. Interaction between surface structures and tunnelling in sand: Centrifuge and computational modelling. *Tunnelling and Underground Space Technology*, 50, pp.465-478.
59. Giardina, G., Marini, A., Hendriks, M.A., Rots, J.G., Rizzardini, F. and Giuriani, E., 2012. Experimental analysis of a masonry façade subject to tunnelling-induced settlement. *Engineering Structures*, 45, pp.421-434.
60. Giardina, G., Van de Graaf, A.V., Hendriks, M.A., Rots, J.G. and Marini, A., 2013. Numerical analysis of a masonry façade subject to tunnelling-induced settlements. *Engineering structures*, 54, pp.234-247.
61. Goh, K.H. and Mair, R.J., 2014. Response of framed buildings to excavation-induced movements. *Soils and Foundations*, 54(3), pp.250-268.

62. Golpasand, M.R.B., Nikudel, M.R. and Uromeihy, A., 2016. Specifying the real value of volume loss (VL) and its effect on ground settlement due to excavation of Abuzar tunnel, Tehran. *Bulletin of Engineering Geology and the Environment*, 75(2), pp.485-501.
63. Grant, R.J. and Taylor, R.N., 2000. Tunnelling-induced ground movements in clay, *Proceedings of the institution of civil engineers-geotechnical engineering*, 143(1), pp.43-55.
64. Guglielmetti, V., Grasso, P., Mahtab, A. and Xu, S., 2008. *Mechanized Tunnelling in Urban Areas: Design Methodology and Construction Control*. Boca Raton, FL: CRC Press.
65. Hansmire, W.H. and Cording, E.J., 1972. Performance of a soft ground tunnel on the Washington Metro, *Proceedings of North American Rapid Excavation and Tunneling Conference Proceedings*, Chicago, Illinois, June 5-7, 1972, 2, pp.371-389.
66. Harris, D.I. and Franzius, J.N., 2005, December. Settlement assessment of running tunnels-a generic approach, *Proceedings of the International symposium of geotechnical aspects of underground construction in soft ground*, pp. 225-230.
67. Hashash, Y.M., Hook, J.J., Schmidt, B. and Yao, J.I.C., 2001. Seismic design and analysis of underground structures. *Tunnelling and underground space technology*, 16(4), pp.247-293.
68. He, S., Li, C., Wang, D. and Liu, X., 2021. Surface settlement induced by slurry shield tunnelling in sandy cobble strata—A case study. *Indian Geotechnical Journal*, 51(6), pp.1349-1363.
69. Hsieh, P.G. and Ou, C.Y., 1998. Shape of ground surface settlement profiles caused by excavation. *Canadian geotechnical journal*, 35(6), pp.1004-1017.
70. Huat, C.Y., Armaghani, D.J., Lai, S.H., Motaghedi, H., Asteris, P.G. and Fakharian, P., 2024. Analyzing surface settlement factors in single and twin tunnels: a review study. *Journal of Engineering Research*.
71. Islam, M.S. and Iskander, M., 2021. Twin tunnelling induced ground settlements: A review. *Tunnelling and Underground Space Technology*, 110, p.103614.
72. Jacobsz, S.W., Standing, J.R., Mair, R.J., Hagiwara, T. and Sugiyama, T., 2004. Centrifuge modelling of tunnelling near driven piles. *Soils and foundations*, 44(1), pp.49-56.
73. Jain, S., Srivastava, A.K., and Pal, S. 2025. Analysis of Underground Structure for Racking Forces with Soil Structure Interaction. In *International Conference on Advances in Materials, Modeling, and Analysis for Sustainable and Resilient Infrastructure Singapore*: Springer Nature Singapore, pp. 383-392.

74. Jenck, O. and Dias, D., 2003. Numerical analysis of the volume loss influence on building during tunnel excavation. *Proceedings of the 3rd International FLAC Symposium on FLAC and FLAC3D Numerical Modeling in Geomechanics*, Swets & Zeitlinger, Lisse, pp.145–152.
75. Kanagaraju, R. and Krishnamurthy, P., 2020. Influence of tunneling in cohesionless soil for different tunnel geometry and volume loss under greenfield condition. *Advances in Civil Engineering*, 2020(1), p.1946761.
76. Khabbaz, H., Gibson, R. and Fatahi, B., 2019. Effect of constructing twin tunnels under a building supported by pile foundations in the Sydney central business district. *Underground Space*, 4(4), pp.261-276.
77. Lai, H., Zhang, J., Zhang, L., Chen, R. and Yang, W., 2019. A new method based on centrifuge model test for evaluating ground settlement induced by tunneling. *KSCE Journal of Civil Engineering*, 23(6), pp.2426-2436.
78. Leca, E. and New, B., 2007. Settlements induced by tunneling in soft ground. *Tunnelling and Underground Space Technology*, 22(2), pp.119-149.
79. Lee, S.W., 2002. The use of compensation grouting in tunnelling: a case study. *Proceedings of the Institution of Civil Engineers-Geotechnical Engineering*, 155(2), pp.101-109.
80. Lee, K.M., Rowe, R.K. and Lo, K.Y., 1992. Subsidence owing to tunnelling. I. Estimating the gap parameter. *Canadian geotechnical journal*, 29(6), pp.929-940.
81. Lei-Kilany, M.E., El-Sayed, T.A. and Mohab, R.A., 2017. Numerical analysis of tunnel boring machine in soft ground. *Jokull Journal*, 67(5).
82. Loganathan, N. and Poulos, H.G., 1998. Analytical prediction for tunneling-induced ground movements in clays. *Journal of Geotechnical and geoenvironmental engineering*, 124(9), pp.846-856.
83. Loganathan, N., Poulos, H.G. and Stewart, D.P., 2000. Centrifuge model testing of tunnelling-induced ground and pile deformations. *Geotechnique*, 50(3), pp.283-294.
84. Manchanda, H. and Agarwal, J., 2022. Building risk assessment for urban rail transit systems in India – current practices and challenges, *Proceedings the 11th Annual Conference on Deep Foundation Technologies for Infrastructure Development in India*, Tirupati, September 15–17, 2022.
85. Mair, R.J., Taylor, R.N. and Bracegirdle, A., 1993. Subsurface settlement profiles above tunnels in clays. *Geotechnique*, 43(2), pp.315-320.
86. Mair, R.J. and Taylor, R.N., 1997. Theme lecture: bored tunnelling in the urban environment. In: *Proceedings of the 14th International Conference on Soil Mechanics and Foundation Engineering (ICSMFE)*, Hamburg, Germany, Vol. 4, pp. 2353–2385.

87. Mair, R.J., Taylor, R.N. and Burland, J.B., 1996. Prediction of ground movements and assessment of risk of building damage due to bored tunnelling, Proceedings of the 4th International Symposium of International Conference of Geotechnical aspects of underground construction in soft ground, London, pp. 713-718.
88. Malhotra, M., Sahu, V., Srivastava, A. and Misra, A.K., 2019. Impact of pile foundations adjacent to tunnels in sandy stratum. *Sādhanā*, 44(8), p.184.
89. Marshall, A.M., Farrell, R.P., Klar, A. and Mair, R., 2012. Tunnels in sands: the effect of size, depth and volume loss on greenfield displacements. *Géotechnique*, 62(5), pp.385-399.
90. Mathew, G.V. and Lehane, B.M., 2013. Numerical back-analyses of greenfield settlement during tunnel boring. *Canadian Geotechnical Journal*, 50(2), pp.145-152.
91. Mathew, G.V. and Lehane, B.M., 2014. Measured and back analysed soil structure interaction effects in a layered stratigraphy during tunnel boring. *Geotechnical and Geological Engineering*, 32(4), pp.873-884.
92. Mei, Y. and Song, Q., 2021. Analytical solution for settlement of homogeneous structure where the tunnel passes underneath and its application. *KSCE Journal of Civil Engineering*, 25(9), pp.3556-3567.
93. Miliziano, S. and de Lillis, A., 2019. Predicted and observed settlements induced by the mechanized tunnel excavation of metro line C near S. Giovanni station in Rome. *Tunnelling and Underground Space Technology*, 86, pp.236-246.
94. Mirhabibi, A. and Soroush, A., 2012. Effects of surface buildings on twin tunnelling-induced ground settlements. *Tunnelling and Underground Space Technology*, 29, pp.40-51.
95. Mirhabibi, A. and Soroush, A., 2020. Three-dimensional simulation of interaction between surface buildings and twin tunnelling regarding the surface settlement. *Geotechnical and Geological Engineering*, 38(5), pp.5143-5166.
96. Moh, Z.C., Ju, D.H. and Hwang, R.N., 1996, April. Ground movements around tunnels in soft ground, Proceedings of the International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, Balkema AA London, 730, pp. 725-730.
97. Möller, S.C. and Vermeer, P.A., 2008. On numerical simulation of tunnel installation. *Tunnelling and Underground Space Technology*, 23(4), pp.461-475.
98. Mroueh, H. and Shahrouh, I., 2003. A full 3-D finite element analysis of tunneling–adjacent structures interaction. *Computers and Geotechnics*, 30(3), pp.245-253.

99. Murphy, J., Gaynor, S. and Laefer, D.F., 2010. Predicted tunnel-induced settlement and damage to Findlater's Church with respect to freefield and constructed side considerations, Proceedings of the GeoFlorida 2010: Advances in Analysis, Modeling & Design, pp.1690-1699.
100. Naqvi, M.W., Akhtar, M.F., Zaid, M. and Sadique, M.R., 2021. Effect of superstructure on the stability of underground tunnels. *Transportation Infrastructure Geotechnology*, 8(1), pp.142-161.
101. Netzel, H., 2005. Review of the limiting tensile strain method for predicting settlement induced building damage, Proceedings of the 5th International Symposium TC28 Geotechnical Aspects of Underground Construction in Soft Ground. Amsterdam, the Netherlands CRC Press, pp.159-164.
102. Netzel, H., 2009. Building response due to ground movements. (Doctoral dissertation, Delft University of Technology, The Netherlands). [Online] Available at: <https://repository.tudelft.nl/record/uuid:511732e9-a084-430d-a4da-b11a3227be88> [accessed 10 November 2025]
103. Obrzud, R., 2010. *The hardening soil model: A practical guidebook*. Zace Services.
104. O'Reilly, M.P. and New, B.M., 1982, Settlement above tunnels in the United Kingdom – their magnitude and prediction, Proceedings of the 3rd International Symposium on Tunnelling, Brighton, London, pp. 173–181.
105. Oteo, C. and Moya, J.F., 1979, September. Estimation of the soil parameters of Madrid in relation to the tunnel construction. Proceedings of the 7th Euro conference on soil mechanics and foundation engineering, 3, pp. 239-47.
106. Peck, R.B., 1969. Deep excavations and tunneling in soft ground, Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, State-of-the-Art Volume, pp. 225–290.
107. Polshin, D.E. and Tokar, R.A., 1957. Maximum allowable non-uniform settlement of structures, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, London: Butterworth's, 1, pp. 402-405.
108. Potts, D.M. and Addenbrooke, T.I., 1997. A structure's influence on tunnelling-induced ground movements. *Proceedings of the Institution of Civil Engineers-Geotechnical Engineering*, 125(2), pp.109-125.
109. Potts, D.M. and Zdravković, L., 2001. *Finite element analysis in geotechnical engineering: Theory and application*. London: Thomas Telford.
110. Rankin, W.J., 1988. Ground movements resulting from urban tunnelling: predictions and effects. *Geological Society, London, Engineering Geology Special Publications*, 5(1), pp.79-92.

111. Rebello, N., Sastry, V.R. and Shivashankar, R., 2014. Study of surface displacements on tunnelling under Buildings Using 3DEC numerical modelling. *International Scholarly Research Notices*, 2014(1), p.828792.
112. Rezaei, A.H. and Ahmadi-adli, M., 2020. The volume loss: real estimation and its effect on surface settlements due to excavation of Tabriz Metro tunnel. *Geotechnical and Geological Engineering*, 38(3), pp.2663-2684.
113. Sagaseta, C., 1987. Analysis of undrained soil deformation due to ground loss. *Geotechnique*, 37(3), pp.301-320.
114. Sarkar, C., Mukherjee, S. and Kumar Pitchumani, N., 2021, November. Effect of Overburden Pressure and Soil Parameters on Tunnel-Induced Ground Settlement. In *Stability of Slopes and Underground Excavations, Proceedings of the Indian Geotechnical Conference 2020*, Singapore: Springer Singapore, 3, pp. 327-341.
115. Schiena, F., Lusini, E., Fazio, A.L. and Graziani, A., 2024. A strain applied method for FEM-2D modelling of TBM tunnels in coarse-grained soils with comparative analysis of case histories. *Tunnelling and Underground Space Technology*, 153, p.106001.
116. Shahin, H.M., Nakai, T., Ishii, K., Iwata, T. and Kuroi, S., 2016. Investigation of influence of tunneling on existing building and tunnel: model tests and numerical simulations. *Acta Geotechnica*, 11(3), pp.679-692.
117. Shiau, J. and Sams, M., 2019. Relating volume loss and greenfield settlement. *Tunnelling and Underground Space Technology*, 83, pp.145-152.
118. Shinde, B.S., Galatage, A.A., Ingole, P.S. and Bhosale, M.S., 2020, A review paper on numerical modelling of building response to underground tunnelling. *International Research Journal of Engineering and Technology (IRJET)*, 7(8), pp. 3854–3857.
119. Singh, M., Viladkar, M.N. and Samadhiya, N.K., 2017. Elasto-plastic analysis of Delhi metro underground tunnels under seismic condition, *Proceedings of the 16th world conference on earthquake engineering (16 WCEE)*, Santiago, Chile, January, 2017, pp. 9-13.
120. Singh, S.K., Kundu, S. and Singh, U., 2020, Impact of underground construction in urban-built environments, *NBM & CW Magazine* [online]. Available at: <https://www.nbmcw.com/article-report/infrastructure-construction/transportation-metrorail-airways-waterways/impact-of-underground-construction-n-urban-built-environments.html> [Accessed 21 August 2025]
121. Skempton, A.W. and MacDonald, D.H., 1956. The allowable settlements of buildings. *Proceedings of the Institution of Civil Engineers*, 5(6), pp.727-768.

122. Son, M., 2015. Response analysis of nearby structures to tunneling-induced ground movements in sandy soils. *Tunnelling and Underground Space Technology*, 48, pp.156-169.
123. Son, M., 2016. Response analysis of nearby structures to tunneling-induced ground movements in clay soils. *Tunnelling and Underground Space Technology*, 56, pp.90-104.
124. Son, M. and Cording, E.J., 2005. Estimation of building damage due to excavation-induced ground movements. *Journal of geotechnical and geoenvironmental engineering*, 131(2), pp.162-177.
125. Son, M. and Cording, E.J., 2007. Evaluation of building stiffness for building response analysis to excavation-induced ground movements. *Journal of geotechnical and geoenvironmental engineering*, 133(8), pp.995-1002.
126. Srinivasan, R. and Bhavsar, R.C., 1983. *Harbour, Dock and Tunnel Engineering*. Charotar Publishing House.
127. Sugiyama, T., Hagiwara, T., Nomoto, T., Nomoto, M., Ano, Y., Mair, R.J., Bolton, M.D. and Soga, K., 1999. Observations of ground movements during tunnel construction by slurry shield method at the Docklands Light Railway Lewisham Extension-East London. *Soils and foundations*, 39(3), pp.99-112.
128. Tian, X., Gu, S. and Huang, R., 2017, September. Theoretical Analysis on the Effect of Tunnel Excavation on Building strip foundation. In *IOP Conference Series: Materials Science and Engineering*, 231(1), p. 012088.
129. Timoshenko, S., 1957. *Strength of Materials-Part 1 Elementary theory and problems*. London: D van Nostrand Inc Co. Inc., London, England.
130. Timoshenko, S.P. and Gere, J.M., 1972. *Mechanics of materials*. van nordstrand reinhold company. New York.
131. Torp-Petersen, G.E. and Black, M.G., 2001. Geotechnical investigation and assessment of potential building damage arising from ground movements: CrossRail. *Proceedings of the Institution of Civil Engineers-Transport*, 147(2), pp. 107-119.
132. Verruijt, A. and Booker, J.R., 1996. Surface settlements due to deformation of a tunnel in an elastic half plane. *Geotechnique*, 46(4), pp.753-756.
133. Vorster, T.E., Klar, A., Soga, K. and Mair, R.J., 2005. Estimating the effects of tunneling on existing pipelines. *Journal of Geotechnical and Geoenvironmental engineering*, 131(11), pp.1399-1410.
134. Vu, M.N., Broere, W. and Bosch, J., 2016. Volume loss in shallow tunnelling. *Tunnelling and Underground Space Technology*, 59, pp.77-90.

135. Wang, F., 2021. Empirical evidence for estimation of subsurface settlement caused by tunneling in sand. *Underground Space*, 6(5), pp.577-584.
136. Wang, J.N., 1993. *Seismic design of tunnels: a simple state-of-the-art design approach*. Parsons Brinckerhoff.
137. Wang, F., Miao, L., Yang, X., Du, Y.J. and Liang, F.Y., 2016. The volume of settlement trough change with depth caused by tunneling in sands. *KSCE Journal of Civil Engineering*, 20(7), pp.2719-2724.
138. Wang, J. and Wen, Z., 2021. Analysis of construction impact of a large diameter shield tunneling side-crossing viaduct pile foundations in short distance. *Geotechnical and Geological Engineering*, 39(8), pp.5587-5598.
139. Whittle, A.J., Hsieh, Y.M., Pinto, F. and Chatzigiannelis, Y., 2001. Numerical and analytical modeling of ground deformations due to shallow tunneling in soft soils, Proceedings of first MIT Conference on Computational Fluid and Solid Mechanics, pp. 546-549.
140. Wood, A.M., 1975. The circular tunnel in elastic ground. *Geotechnique*, 25(1), pp.115-127.
141. Wu, H.N., Shen, S.L. and Yang, J., 2017. Identification of tunnel settlement caused by land subsidence in soft deposit of Shanghai. *Journal of Performance of Constructed Facilities*, 31(6), p.04017092.
142. Yahya, S.M. and Abdullah, R.A., 2014. A review on methods of predicting tunneling induced ground settlements. *Electronic Journal of Geotechnical Engineering*, 19, pp.5813-5826.
143. Yiu, W.N., Burd, H.J. and Martin, C.M., 2017. Finite-element modelling for the assessment of tunnel-induced damage to a masonry building. *Géotechnique*, 67(9), pp.780-794.
144. Zhang, L., Wu, X., Skibniewski, M.J. and Chen, H., 2014. Safety management for existing buildings in tunnel construction, In ISARC, Proceedings of the International Symposium on Automation and Robotics in Construction, 31, p.1.
145. Zhang, X., Qu, H., Xu, Y., Zhang, L. and Zhang, Z., 2024. Investigating the damage to masonry buildings during shield tunneling: A case study in Hohhot Metro. *Engineering Failure Analysis*, 160, p.108147.
146. Zhang, Z.X., Liu, C., Huang, X., Kwok, C.Y. and Teng, L., 2016. Three-dimensional finite-element analysis on ground responses during twin-tunnel construction using the URUP method. *Tunnelling and Underground Space Technology*, 58, pp.133-146.

147. Zheng, G., Lu, P. and Diao, Y., 2015. Advance speed-based parametric study of greenfield deformation induced by EPBM tunneling in soft ground. *Computers and Geotechnics*, 65, pp.220-232.
148. Zhao, C., Lavasan, A.A., Barciaga, T., Zarev, V., Datcheva, M. and Schanz, T., 2015. Model validation and calibration via back analysis for mechanized tunnel simulations–The Western Scheldt tunnel case. *Computers and Geotechnics*, 69, pp.601-614.
149. Zhao, C., Schmüdderich, C., Barciaga, T. and Röchter, L., 2019. Response of building to shallow tunnel excavation in different types of soil. *Computers and Geotechnics*, 115, p.103165.
150. Zienkiewicz, O.C. and Taylor, R.L., 2000. *The finite element method: Volume 1, The basis*. 5th edn. Oxford: Butterworth-Heinemann.
151. Zlatanović, E., Lukić, D.Č., Prolović, V., Bonić, Z. and Davidović, N., 2015. Comparative study on earthquake-induced soil–tunnel structure interaction effects under good and poor soil conditions. *European Journal of Environmental and Civil Engineering*, 19(8), pp.1000-1014.
152. CIRIA (1995) SPT methods and use. CIRIA Report R143. London: Construction Industry Research and Information Association.
153. IS 1893 (Part 1): 2016 – Criteria for earthquake resistant design of structures, Part 1: General provisions and buildings. New Delhi: Bureau of Indian Standards.
154. IS 2131: 1981 – Method for standard penetration test for soils. New Delhi: Bureau of Indian Standards.
155. IS 2720 (Part 2): 1980 – Methods of test for soils, Part 2: Determination of water content. New Delhi: Bureau of Indian Standards.
156. IS 2720 (Part 3/Sec 1): 1980 – Methods of test for soils, Part 3: Determination of specific gravity, Section 1: Fine soils. New Delhi: Bureau of Indian Standards.
157. IS 2720 (Part 3/Sec 2): 1980 – Methods of test for soils, Part 3: Determination of specific gravity, Section 2: Coarse soils. New Delhi: Bureau of Indian Standards.
158. IS 2720 (Part 4): 1985 – Methods of test for soils, Part 4: Grain size analysis. New Delhi: Bureau of Indian Standards.
159. IS 2720 (Part 8): 1983 – Methods of test for soils, Part 8: Determination of water content–dry density relation using heavy compaction. New Delhi: Bureau of Indian Standards.
160. IS 2720 (Part 5): 1985 – Methods of test for soils, Part 5: Determination of liquid and plastic limit. New Delhi: Bureau of Indian Standards.

161. IS 2720 (Part 12): 1981 – Methods of test for soils, Part 12: Determination of shear strength parameters using triaxial shear test without pore pressure measurement. New Delhi: Bureau of Indian Standards.
162. IRC: 6-2017 – Standard Specifications and Code of Practice for Road Bridges, Section II: Loads and Stresses. Indian Roads Congress, New Delhi, India.
163. <https://timesofindia.indiatimes.com/city/agra/agra-homes-crumble-as-metro-tunneling-sparks-safety-crisis/articleshow/115775628.cms>
164. <https://timesofindia.indiatimes.com/city/bengaluru/bmrcl-shifts-residents-from-unsafe-shivajinagar-buildings/articleshow/79817799.cms>
165. <https://timesofindia.indiatimes.com/city/mumbai/rd-caves-in-near-metro-7a-work-site-at-sahar-9-families-shifted/articleshow/112768292.cms>
166. <https://timesofindia.indiatimes.com/city/delhi/heritage-building-falls-to-delhi-metros-tunnel-work/articleshow/47364512.cms>
167. <https://timesofindia.indiatimes.com/city/chennai/another-chennai-church-develops-cracks-due-to-metro-rail-tunnelling-work/articleshow/21455516.cms>
168. <https://timesofindia.indiatimes.com/city/chennai/metro-rail-tunnelling-has-damaged300-plus-buildings-since-mid-2012/articleshow/61886194.cms>
169. The Metro Railguy Home page <https://themetrorailguy.com/delhi-meerut-rrts-information-route-maps-fares-tenders-updates/>
170. BidAssist Home page <https://bidassist.com/delhi-tenders/national-capital-region-transport-corporation/detail-2a42fa4a-5b37-46b5-98b1-7e3bdeb4585b>
171. Bentley Systems (2022) PLAXIS 2D Tutorial manual.
172. Bentley Systems (2024) PLAXIS 2D Reference manual.
173. Bentley Systems (2022) PLAXIS 2D Material models manual.
174. Bentley Systems (2013) PLAXIS 2D Material models manual.

## ANNEXURE

### PARA WISE RESPONSE BY MS. JYOTI AGARWAL (2K20/PHDCE/04) TO FOREIGN EXAMINER

1. **Comment:** Integrating ML would transform the present deterministic, simulation-based framework into a fast, probabilistic, adaptive, and generalizable risk assessment system, significantly advancing both research and practice in urban tunneling. To implement this, future researchers should collaborate with:
  - Computer science/ data science departments for ML model development and optimization.
  - Tunnel construction agencies (e.g., NCRTC for RRTS) to gain access to field monitoring datasets for training and validation.
  - Geotechnical instrumentation firms to standardize data collection formats suitable for ML pipelines.

**Response:** Thank you for the valuable suggestion. The potential integration of Machine Learning for developing a faster, probabilistic, and adaptive tunnel-induced risk assessment framework has been acknowledged as an important future research direction. Accordingly, the future scope section of the revised thesis (Section 8.3.6 Page No. 106) has been updated to discuss how Machine Learning techniques, combined with field monitoring datasets, may enhance the predictive capability and generalizability of the present deterministic simulation-based approach. The revised discussion also highlights the importance of interdisciplinary collaboration with data science experts, tunnel construction agencies, and geotechnical instrumentation stakeholders for effective model development, validation, and standardized data acquisition.

## PARA WISE RESPONSE BY MS. JYOTI AGARWAL (2K20/PHDCE/04) TO INDIAN EXAMINER

### Chapter 1

1. **Comment:** The thesis mentions tunneling-related distress in Indian metro projects, including 150 buildings in Agra, 300 buildings in Chennai, a 24 ft deep and 10 ft wide cave-in in Mumbai, and a 10 ft sinkhole in Bengaluru. Please clarify how these reported incidents quantitatively justify the need for the present Building Risk Assessment framework.

**Response:** The comment is well appreciated. The intent of presenting tunnelling-related incidents in Indian metro projects was to highlight the practical significance and urgency of Building Risk Assessment (BRA) under Indian urban tunnelling conditions. In response to the comment, the revised thesis has been strengthened by incorporating a clearer quantitative justification for the need of the present BRA framework, emphasizing how such incidents demonstrate the potential consequences of uncontrolled ground deformation and the necessity for predictive risk assessment. The corresponding discussion has been incorporated in the revised thesis (Section 1.2, Page No. 3).

2. **Comment:** The Delhi–Meerut RRTS project has a total length of approximately 82.15 km, with underground sections of 5.6 km and 5.68 km. Please explain why Package 4 was selected for detailed study instead of the other underground stretch.

**Response:** Package 4 was selected for detailed analysis due to the availability of comprehensive geotechnical data, tunnel geometry, and alignment details necessary for numerical modelling and Building Risk Assessment (BRA). Furthermore, this section passes through a densely urbanized environment, including the Anand Vihar and Khichdipur region, where the proximity of existing structures increases the importance of evaluating tunnelling-induced ground deformation and associated building risk. The subsurface profile of the selected stretch, consisting of mixed soil strata, was also considered representative for assessing tunnelling-induced settlement under Indian urban conditions.

### Chapter 2

1. **Comment:** The empirical trough width parameter  $k$  is reported as 0.4–0.7 for cohesive soils and 0.2–0.3 for cohesionless soils. Since the study area contains both SM and CL layers, how was the appropriate effective  $k$  selected?

**Response:** The comment is well appreciated. Since the study area comprises both SM and CL soil strata, a single soil-specific  $k$  value was not directly adopted from the literature. Instead, an effective trough width parameter was derived by fitting the PLAXIS 2D greenfield settlement profile to a Gaussian settlement curve. This

approach yielded a trough width parameter of  $i = 7.92$  m, corresponding to a  $k$  value of 0.528, which lies within the acceptable range for mixed ground conditions. Accordingly, this value was adopted for the empirical verification of the numerical model. The corresponding clarification has been incorporated in the revised thesis (Section 5.3, Page No. 58).

2. **Comment:** Table 2.1 summarizes 43 previous studies. Please clarify how many of these studies are directly related to Indian tunnelling conditions and how the present work quantitatively differs from them.

**Response:** The comment is well appreciated. The revised thesis clarifies that 7 out of the 43 reviewed studies are directly related to Indian tunnelling conditions. It has also been emphasized that while previous Indian studies mainly focused on greenfield settlements, tunnel–pile interaction, or isolated aspects of tunnel–structure response, the present study differs by developing an integrated Building Risk Assessment (BRA) framework. The proposed framework incorporates limiting tensile strain, parametric analysis, and seismic effects to comprehensively evaluate structural vulnerability under Indian urban tunnelling conditions. The corresponding discussion has been incorporated in the revised thesis (Section 2.2.4, Page No. 20).

### Chapter 3

1. **Comment:** The LTSM assumes that the building fully follows the greenfield settlement profile. Since this is conservative, please quantify how much reduction in strain may occur if soil–structure interaction is considered.

**Response:** The comment is well appreciated. The revised thesis clarifies that the LTSM assumption, wherein buildings are assumed to fully follow the greenfield settlement profile, represents a conservative approach, as soil–structure interaction (SSI) may significantly reduce tunnelling-induced strains depending on structural stiffness, geometry, and ground conditions. It has also been highlighted that literature reports reductions ranging from negligible values for flexible structures to over 90% for stiff buildings. The corresponding discussion has been incorporated in the revised thesis (Section 3.5, Page No. 37).

2. **Comment:** The thesis used a modified shear factor form of 1.2 instead of 1.5, which increases strain by about 20–25% for practical  $L/H$  ratios. Please explain how this affects the final damage classification.

**Response:** The comment is well appreciated. The modified shear form factor of 1.2 instead of 1.5 increases the predicted tensile strain by approximately 20–25% for practical  $L/H$  ratios, leading to a comparatively more conservative damage assessment. However, the effect on final damage classification is case-dependent, and in most scenarios this primarily influences cases lying near damage category boundaries, while structures clearly within a category remain largely unaffected.

3. **Comment:** Stage-I BRA excludes buildings with settlement less than 10 mm and slope less than 1/500. Please justify whether these limits are adequate for old, weak, or heritage structures.

**Response:** The comment is well appreciated. The revised thesis clarifies that the Stage-I BRA screening limits of settlement  $< 10$  mm and slope  $< 1/500$  were adopted as preliminary thresholds based on commonly accepted tunnelling criteria corresponding to negligible to very slight damage potential. However, it has also been acknowledged that old, weak, or heritage structures may exhibit higher vulnerability even at lower deformation levels due to material degradation, pre-existing defects, and reduced structural redundancy. Therefore, such structures are recommended for individual assessment irrespective of the preliminary screening limits. The corresponding discussion has been incorporated in the revised thesis (Section 3.5, Page No. 37).

## Chapter 4

1. **Comment:** Table 4.2 classifies eight soil layers with  $N_{60}$  increasing from 8 in Layer 1 to 60 below 40 m. Please discuss how this variation affects tunnel-induced ground deformation.

**Response:** The comment is well appreciated. The progressive increase in SPT  $N_{60}$  values with depth (from approximately 8 near the ground surface to 60 below 40 m depth) reflects a corresponding increase in soil stiffness, consistent with practical subsurface conditions as these values are derived from field investigations. This depth-dependent stiffness variation plays a significant role in controlling tunnel-induced ground deformation. Although a volume loss of 1.5% is applied around the tunnel cavity in the present study, the resulting settlement reflected at the ground surface is approximately 1% (as discussed in Section 5.3, Page No. 58), primarily due to the confinement provided by the deeper, stiffer soil strata. The presence of dense/stiff underlying layers restricts deformation propagation toward the surface, thereby reducing the magnitude of surface settlement. The corresponding clarification has been incorporated in the revised thesis (Section 5.3, Page No. 58).

2. **Comment:** Groundwater was encountered at about 15 m below ground level, close to the tunnel depth range. Please explain how groundwater influence was considered in the PLAXIS modelling.

**Response:** The comment is well appreciated. The Groundwater influence was explicitly incorporated in the PLAXIS modelling through the assignment of a phreatic level at the observed groundwater table ( $\sim 15$  m below ground level) to generate hydrostatic initial pore water pressures. A phreatic analysis was performed assuming no groundwater flow during excavation, while the tunnel lining was considered impermeable, thereby accounting for groundwater effects as discussed in Section 5.2 (Page No. 53).

3. **Comment:** Table 4.4 uses  $E'_{ref}$  values ranging from 8000 kPa to 60000 kPa and undrained shear strength values of 90 kPa and 130 kPa for CL layers. Please justify these values, since field modulus tests were not performed.

**Response:** The comment is well appreciated. The revised thesis clarifies that, in the absence of field modulus tests, the adopted stiffness parameters ( $E'_{ref} = 8000\text{--}60000$  kPa) were estimated using established empirical correlations with SPT N values, as recommended in CIRIA Report 143 (1995) as discussed in section 4.2.2 (Page No. 48). Furthermore, the undrained shear strength values adopted for the CL layers (90 kPa and 130 kPa) were derived from SPT N– $C_u$  correlations considering the measured plasticity index obtained from Atterberg limit tests. The adopted  $C_u$  values were also verified against laboratory test results available in the Geotechnical Investigation Report and found to be reasonably consistent, as discussed in Section 4.2.1 (Page No. 47).

4. **Comment:** The tunnel diameter is 6.5 m, lining thickness is 300 mm, and concrete grade is M50. Please explain whether the lining stiffness values of  $EA = 10.6 \times 10^6$  kN/m and  $EI = 79.5 \times 10^3$  kN·m<sup>2</sup>/m were independently verified.

**Response:** The comment is well appreciated. The independent stiffness values of the tunnel lining ( $EA = 10.6 \times 10^6$  kN/m and  $EI = 79.5 \times 10^3$  kN·m<sup>2</sup>/m) were not assumed arbitrarily but were calculated based on the geometric and material properties of the 300 mm thick M50 concrete segmental lining used in the project. The axial stiffness ( $EA$ ) and bending stiffness ( $EI$ ) were determined from the modulus of elasticity of M50 concrete and the cross-sectional properties of the lining, following standard structural mechanics relations adopted in PLAXIS 2D for plate elements.

## Chapter 5

1. **Comment:** The PLAXIS 2D model domain was taken as 10 times the tunnel diameter in all directions. Please justify whether this boundary distance is sufficient for shallow tunnel cases.

**Response:** The comment is well appreciated. A PLAXIS 2D model domain extending to 10 times the tunnel diameter (10D) in all directions was adopted to minimize boundary effects and avoid artificial restraint of tunnel-induced stress redistribution and deformation. Tunnelling literature generally recommends boundary distances ranging from approximately 4D–15D horizontally and 3D–8D vertically, depending on tunnel geometry and ground conditions. The adopted 10D extent lies within these recommended ranges and is considered adequate, including for shallow tunnel cases. Furthermore, the settlement profiles presented in the thesis diminish well before reaching the model boundaries, confirming negligible boundary influence on the numerical results. The corresponding justification has been incorporated in the revised thesis (Section 5.2, Page No. 53).

2. **Comment:** The baseline tunnel model uses  $D = 6.5$  m,  $z = 15$  m, and volume loss  $V_L = 1.5\%$ . Please explain how the adopted volume loss was selected for EPB-TBM tunnelling in Delhi soils.

**Response:** The comment is well appreciated. The revised thesis clarifies that the adopted volume loss ( $V_L = 1.5\%$ ) was selected based on the range commonly reported for EPB-TBM tunnelling in soft ground and alluvial deposits. Previous studies indicate that volume loss typically varies from approximately 0.1% to 3.9%, with most reported values for well-controlled mechanized tunnelling lying within 0.2%–2.0%. Considering that the Delhi subsurface profile predominantly consists of alluvial silty sand (SM) with intermediate low-plasticity cohesive layers (CL), an intermediate value of 1.5% was adopted to realistically represent EPB-TBM tunnelling conditions in mixed ground while maintaining a reasonably conservative basis for settlement and damage assessment. Furthermore, the adopted value produced realistic surface settlement behaviour consistent with expected tunnelling performance. The corresponding clarification has been incorporated in the revised thesis (Section 5.2, Page No. 53).

3. **Comment:** The surface settlement trough volume obtained from PLAXIS is about 1%, while imposed tunnel volume loss is 1.5%. Please explain this difference and its effect on damage assessment.

**Response:** The comment is well appreciated. The revised thesis clarifies that the difference between the imposed tunnel volume loss (1.5%) and the resulting surface settlement trough volume (~1%) is primarily attributed to the increase in soil stiffness with depth, where the deeper, stiffer strata provide greater confinement and reduce deformation transmission to the ground surface. Consequently, not all tunnel cavity volume loss is reflected as surface settlement, which is consistent with practical observations in layered ground conditions. Since the Building Risk Assessment (BRA) in the present study is based on the computed free-field ground deformations obtained from PLAXIS, this reduction in surface settlement is inherently accounted for in the damage assessment, thereby enabling realistic rather than overly conservative predictions of structural response. The corresponding clarification has been incorporated in the revised thesis (Section 5.3, Page No. 58).

4. **Comment:** The derived settlement trough parameter is  $i = 7.92$  m and  $k = 0.528$ . Please compare this value with the recommended range for cohesive and cohesionless soils.

**Response:** The comment is well appreciated. The revised thesis clarifies that the derived trough width parameter ( $k = 0.528$ , with  $i = 7.92$  m) lies within the range recommended in the literature for cohesive soils (0.4–0.7) and also falls within the reported range for stiff clays (0.4–0.6) and granular soils (0.2–0.5) under mixed ground conditions. Since the study area comprises both cohesionless (SM) and cohesive (CL) strata, the obtained value is considered reasonable and representative of the prevailing subsurface conditions, as discussed in Section 2.2.1 (Page No. 9) and Section 5.3 (Page

no. 58). Furthermore, the trough width parameter is related to tunnel depth through  $i = kz$ ; therefore, using the tunnel axis depth ( $z = 15$  m) and the derived  $k = 0.528$ , the corresponding trough width parameter ( $i = 7.92$  m) is obtained.

## Chapter 6

1. **Comment:** Tunnel diameter increase from 5.5 m to 8.5 m caused more than 100% increase in settlement. Please explain the governing mechanism behind this large increase.

**Response:** The comment is well appreciated. The governing mechanism behind the substantial increase in settlement has been clarified in the revised thesis in Section 6.4 (Page No. 95). This increase is primarily attributed to greater excavation-induced ground disturbance and stress redistribution associated with larger tunnel diameters, which increase soil deformation and enlarge the settlement influence zone. Furthermore, the maximum surface settlement is directly proportional to the square of tunnel diameter ( $D^2$ ) under a constant percentage of volume loss imposed on the tunnel cavity. Therefore, increasing tunnel diameter significantly amplifies the surface settlement response, which explains the observed increase exceeding 100%.

2. **Comment:** Tunnel depth increase from 10 m to 30 m significantly reduced settlement. Please comment on the optimum or minimum tunnel cover depth.

**Response:** The comment is well appreciated. The influence of tunnel depth on settlement and the minimum tunnel depth required for risk reduction have been clarified in the revised thesis in Section 6.4 (Page No. 95). Based on the results of the present study, a tunnel depth of approximately 20–25 m may be considered an effective minimum tunnel axial depth for reducing settlement-related risks, as the reduction in settlement beyond this depth becomes comparatively smaller. However, it should be noted that the optimum minimum cover depth is site-specific and depends on ground conditions, tunnel diameter, and allowable settlement criteria.

3. **Comment:** Increasing volume loss from 0.5% to 2% caused a very large increase in settlement. Please explain the sensitivity of damage assessment to TBM operational control.

**Response:** The comment is well appreciated. The sensitivity of settlement-induced damage assessment to TBM operational control has been clarified in the revised thesis in section 6.4 (Page No. 95). Results highlight the strong sensitivity of settlement response to TBM operational control, as volume loss directly reflects the effectiveness of excavation face support and annular gap management during tunnelling. In TBM tunnelling, volume loss is strongly influenced by operational parameters such as face pressure control, shield overcut, TBM conicity, tail void closure, and grout injection pressure. Poor operational control may result in excessive ground loss, leading to amplified soil deformation and increased settlement-induced damage risk to adjacent structures. Therefore, maintaining a low and controlled

volume loss through optimized TBM operational parameters is essential to minimize settlement and reduce damage risk to nearby infrastructure.

4. **Comment:** Please explain why soil strength variation produced different settlement reductions under constant grout pressure and constant volume loss conditions.

**Response:** The comment is well appreciated. The difference in settlement reduction under constant grout pressure and constant volume loss conditions has been clarified in the revised thesis in Section 6.4 (Page No. 96). Under the constant volume loss condition, the imposed volume loss around the tunnel cavity remains fixed irrespective of soil stiffness. Since the amount of ground loss is predefined and controlled, variation in soil stiffness produces only a marginal effect on the surface settlement profile, resulting in nearly overlapping settlement troughs. Therefore, the sensitivity of settlement to changes remains limited under constant  $V_L$  conditions. In contrast, under the constant grout pressure condition, the tunnel response becomes strongly dependent on soil stiffness. For the same applied grout pressure, soils with lower stiffness experience greater deformation and compressibility, resulting in larger excavation-induced ground movements and deeper settlement troughs.

5. **Comment:** Please explain why masonry buildings showed more critical damage response compared to framed structures.

**Response:** The comment is well appreciated. The results indicate that masonry buildings exhibit a more critical damage response than framed structures primarily due to their relatively low stiffness ratio ( $E/G$ ) and limited capacity to accommodate differential ground movements. Masonry structures generally possess low tensile strength and brittle behaviour, making them more susceptible to cracking under tunnelling-induced tensile and shear strains. In contrast, framed structures exhibit greater structural flexibility and load redistribution capacity, enabling them to better tolerate differential settlement and induced deformation without significant damage.

6. **Comment:** Please define the approximate safe building offset distance beyond which tunnel-induced damage becomes negligible.

**Response:** The comment is well appreciated. Based on the settlement profiles and damage assessment obtained in the present study, tunnel-induced building damage reduces significantly with increasing offset distance from the tunnel centerline. It was observed that beyond an approximate offset distance of 4.0 times the tunnel diameter, the influence of tunneling-induced ground deformation becomes considerably less pronounced, and the associated damage generally becomes negligible for both masonry and framed structures under the analysed conditions. However, this distance is not universal and depends on factors such as soil stiffness, tunnel depth, volume loss, building stiffness, and foundation type. Therefore, the reported offset distance should be considered indicative for the present case study rather than a generalized threshold.

## Chapter 7

1. **Comment:** The seismic study considers tunnel depths of 10 m, 15 m, and 20 m with  $D = 6.5$  m. Please justify why only pseudo-static analysis was adopted instead of full dynamic time-history analysis.

**Response:** The comment is well appreciated. The use of pseudo-static analysis instead of full dynamic time-history analysis was adopted considering the objective and scope of the present study. This study is primarily intended for the screening-level assessment of structures assigned to moderate and higher damage categories for Stage 3 assessment. The objective was to obtain an approximation of the extent to which seismic acceleration increases the limiting strains in buildings and influences the damage assessment. Therefore, a simplified pseudo-static approach was considered appropriate for evaluating the relative influence of seismic loading on tunnel-induced ground response at tunnel depths of 10 m, 15 m, and 20 m. The pseudo-static method is widely adopted in geotechnical engineering for preliminary and parametric seismic assessment, as it incorporates earthquake effects through equivalent seismic coefficients while maintaining computational efficiency. In contrast, full dynamic time-history analysis requires site-specific ground motion records, advanced constitutive modelling, damping calibration, and significantly higher computational effort. Such analysis becomes more relevant when structures are classified under moderate or higher damage categories, where structure-specific modelling is required to accurately estimate induced strains and assess structural response. At that stage, full dynamic time-history analysis may be conducted for detailed assessment. However, this level of analysis is beyond the scope of the present thesis.

2. **Comment:** Table 7.2 shows that seismic loading increases  $S_{max}$  by 39.6%, 58.5%, and 82.5% for tunnel depths of 10 m, 15 m, and 20 m, respectively. Why does the percentage increase become higher at greater depths?

**Response:** The comment is well appreciated. A clarification regarding the increasing percentage rise in  $S_{max}$  with tunnel depth has been incorporated in the revised thesis in Section 7.1 (Page No. 101). The results indicate that the absolute increase in settlement due to seismic loading is only marginally different with tunnel depth, increasing from 13.52 mm at  $z = 10$  m to 14.92 mm at  $z = 20$  m. Therefore, no generalized conclusion regarding increased seismic influence with tunnel depth has been drawn based solely on the absolute settlement increment. However, the percentage increase in  $S_{max}$  becomes progressively higher because the pre-earthquake (static) settlement decreases significantly with increasing tunnel depth due to greater confinement and reduced surface settlement. Consequently, a comparable seismic-induced settlement increment becomes proportionally more significant at greater tunnel depths, resulting in percentage increases of 39.6%, 58.5%, and 82.5% for  $z = 10$  m, 15 m, and 20 m, respectively. Hence, within the investigated conditions of the present study, the relative influence of seismic loading on settlement and damage assessment becomes more pronounced at greater tunnel depths when compared to static conditions.

3. **Comment:** Under seismic loading,  $\varepsilon_{lim}$  increases by 8.15%, 17.12%, and 29.9% for masonry buildings and by 11.73%, 26.03%, and 41.2% for framed buildings at  $z = 10, 15$ , and  $20$  m, respectively. Why is the percentage increase higher for framed buildings?

**Response:** The comment is well appreciated. The higher percentage increase in  $\varepsilon_{lim}$  for framed buildings has been clarified in the revised thesis in section 7.1 (Page No. 101). The seismic-induced absolute increase in  $\varepsilon_{lim}$  is found to be nearly similar for masonry and framed buildings; however, the percentage increase in  $\varepsilon_{lim}$  is observed to be greater for framed buildings at all tunnel depths. This trend is primarily attributed to the lower pre-earthquake (static)  $\varepsilon_{lim}$  values in framed buildings compared to masonry buildings. Therefore, within the investigated conditions of the present study, framed buildings demonstrate a comparatively higher relative sensitivity to seismic-induced tunnel deformations with respect to their pre-earthquake (static) condition.

## Chapter 8

1. **Comment:** The thesis concludes that volume loss control, tunnel depth, and soil stiffness are key parameters. Please rank these parameters in terms of maximum surface settlement and limiting tensile strains.

**Response:** The comment is well appreciated. Based on the parametric analyses, the governing parameters have been ranked according to their relative influence on tunnel-induced response in revised thesis in section 8.1 (Page No. 103). Considering the absolute change observed in both maximum surface settlement and limiting tensile strains, the order of influence was found to be: Volume Loss  $\approx$  Tunnel Depth  $>$  Soil Stiffness. The results indicate that volume loss and tunnel depth exert nearly comparable and significantly greater effects on settlement and structural response compared to soil stiffness.

2. **Comment:** The thesis proposes a transferable BRA framework for Indian tunnelling projects. Please specify the quantitative thresholds for volume loss, tunnel cover, building offset, and tensile strain that should be recommended for practical design?

**Response:** The comment is well appreciated. The threshold values for volume loss, tunnel cover, building offset, and limiting tensile strain recommended for practical implementation of the proposed Building Risk Assessment (BRA) framework for Indian tunnelling projects, based on the present numerical analyses and observed transition in damage categories, have been added in the revised thesis in Section 8.2 (Page No. 104-105). These are following:

Threshold for  $V_L$

| $V_L$ (%) | Recommended Risk Level |
|-----------|------------------------|
| $< 1$     | Low Risk               |
| 1 - 2     | Moderate Risk          |

|       |                                 |
|-------|---------------------------------|
| $> 2$ | High Risk — mitigation required |
|-------|---------------------------------|

Threshold for  $z$

| $z/D$   | Recommended Risk Level          |
|---------|---------------------------------|
| $> 3$   | Low Risk                        |
| $2 - 3$ | Moderate Risk                   |
| $< 2$   | High Risk — mitigation required |

Threshold for  $X$

| $X/D$   | Recommended Risk Level          |
|---------|---------------------------------|
| $> 2$   | Low Risk                        |
| $1 - 2$ | Moderate Risk                   |
| $< 1$   | High Risk — mitigation required |

Threshold for  $\varepsilon_{lim}$

| $\varepsilon_{lim}(\%)$ | Recommended Risk Level          |
|-------------------------|---------------------------------|
| $> 0.15$                | Low Risk                        |
| $0.15 - 0.3$            | Moderate Risk                   |
| $< 0.3$                 | High Risk — mitigation required |

## LIST OF PUBLICATIONS AND THEIR PROOFS

### SCI Indexed Journals

1. Agarwal, J. and Sarkar, R., 2025. Building risk assessment of existing buildings near a proposed tunnel. *Sādhanā*, 50(3), p.201.

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### Building risk assessment of existing buildings near a proposed tunnel

JYOTI AGARWAL and RAJU SARKAR\*

Civil Engineering Department, Delhi Technological University, Shahbad Daulatpur Main Bawana Road,  
Delhi 110042, India  
e-mail: [rajusarkar@dce.ac.in](mailto:rajusarkar@dce.ac.in)

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**Abstract.** The present study introduces a novel hybrid approach for building risk assessment of existing buildings adjacent to proposed tunnel construction, employing a design methodology based on the limiting tensile strain method integrated with advanced PLAXIS 2D simulations. Ground deformations induced by tunnelling were modelled in PLAXIS 2D for controlled volume loss ( $V_L$ ) scenarios and validated against traditional empirical-analytical approaches. The deformation results obtained by PLAXIS 2D were directly applied beneath the building foundations to evaluate its impact on structural integrity, and maximum tensile strain values were calculated to predict potential damage levels and classify risk categories. To understand the sensitivity of these predictions, a detailed parametric analysis was conducted, exploring the influence of key variables, including tunnel diameter ( $D$ ), tunnel axial depth from the ground surface ( $z$ ) and volume loss ( $V_L$ ), soil parameters such as soil stiffness ( $E'_{ref}$ ) and shear strength parameters (undrained shear strength,  $s_u$  and friction angle,  $\phi'$ ) and stiffness of structure ( $E/G$ ) on the maximum tensile strains in buildings and damage category. Results reveal that structural stiffness and tunnel parameters have the most pronounced effect on tensile strain and potential damage, with soil parameters playing a secondary role.

**Keywords.** Tunnelling; ground deformations; PLAXIS 2D; building risk assessment; damage classification.

### 1. Introduction

As urban rail transit systems undergo rapid development in nearly all major cities across India, there is a growing concern for the potential risks posed to existing structures and historical sites in close proximity. Safeguarding these existing structures emerges as a significant challenge, particularly when the infrastructure development includes the construction of underground tunnels that traverse through age-old urban areas. Design and construction of these long and shallow tunnels pose significant risks to adjacent structures because of ground deformations caused by tunnelling [1, 2]. Ground deformations caused by tunnelling during construction may damage adjacent infrastructures and utilities if not accurately taken into consideration in design. The primary cause of ground deformations associated with tunnelling using modern tunnel boring machines (TBMs) is volume loss ( $V_L$ ).  $V_L$  is defined as the volume of ground lost around the tunnel cavity ( $V_T$ ) relative to the tunnel volume per unit length [3–8]. This volume loss leads to ground deformations. The assessment of risk of damage for buildings potentially affected by tunnelling-induced

conditions of buildings before, during and after tunnel construction, and (b) the building risk assessment (BRA), which estimates the potentially expected damages based on predicted deformations and the inherent vulnerability of the structures [9]. There have been some unpleasant incidents in the past in India that further strengthen the need for rigorous analysis of all existing buildings and structures prior to the start of construction. An unfortunate incident occurred during the underground tunnelling work for the Central Secretariat—Kashmiri Gate line, Delhi, India. In this particular case, a 113-year-old heritage building within the tunnel construction zone experienced partial collapse due to the leakage of the sewer line resulted in soil weakening and vibrations reaching the building [10]. In another incident in Kolkata, a three-storey building in the Bowbazar area collapsed a day after cracks emerged, following tunnel boring work when underground water suddenly entered the tunnel and the soil above became unstable. The impact zone of this incident was extensive, with approximately 52 buildings being damaged [11]. Several additional incidents, not addressed here, but are in connection with the main topic of the present study. These occurrences underscore

2. Agarwal, J. and Sarkar, R., 2024. Greenfield settlements due to tunnelling using tunnel boring machine (TBM) in layered soils: a parametric study. *Sādhanā*, 49(1), p.75.

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## Greenfield settlements due to tunnelling using tunnel boring machine (TBM) in layered soils: a parametric study

JYOTI AGARWAL\* and RAJU SARKAR

Civil Engineering Department, Delhi Technological University, Shahbad Daulatpur Main Bawana Road, Delhi 110042, India  
e-mail: jyotiagarwal\_2k20phdce04@dtu.ac.in; rajusarkar@dce.ac.in

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**Abstract.** As India's cities continue to increase in population, a large number of new metro lines are being built to accommodate the mobility of residents. For construction of these metro lines, underground tunnels are being constructed to alleviate traffic problems in congested areas such as Delhi, Chennai, Mumbai, etc., where land scarcity is a major issue. Although tunnel boring machines (TBMs) are used in the construction of metro tunnels, ground settlements on the surface are inevitable. Tunnelling induced ground settlements may have adverse impact on the structures that are above the ground level as well as adjacent to the excavations. In the present study, ground settlements caused by TBM tunnelling are determined in greenfield conditions, using a finite element method (FEM) based software, PLAXIS and parametric study is performed after validation of numerical model with empirical results. Ground settlements obtained using PLAXIS 2D are compared with those obtained using empirical methods as available in literature. This study captures the effect of governing tunnel and soil parameters such as diameter of tunnel ( $D$ ), depth of the tunnel axis from ground surface ( $z$ ), grout pressure ( $GP$ ), undrained shear strength of soil ( $s_u$ ) and soil modulus ( $E'_{ref}$ ), covering the most practical range, on ground settlements under greenfield condition. The settlement results of this study are validated and compared closely with earlier experimental and observational findings that have been published in the literature. It is observed from the various parameters studied that they are having a major impact on the ground settlements during tunnel construction. Therefore, a tunnel designer should consider the effect of all the governing factors in design prior to the tunnel construction to keep the maximum surface settlements within the permissible limit set by the standards.

**Keywords.** Tunnelling; greenfield settlements; volume loss; parametric study.

### 1. Introduction

Research in the transportation services for better management and infrastructure development has been prompted by population growth, urbanisation, and the quick expansion of emerging economies. Delhi and Mumbai are predicted to rank among the world's two most populous cities by the year 2035. Rapid transit system advancements have followed the rise in land scarcity, urbanisation, and industrialisation caused by the rapid population growth rate [1]. With little room for alteration above ground, tunnels have become a more popular option, making greater use of the underground area is a good alternative. Tunnelling induced ground settlement is a complex phenomenon that depends on a variety of factors including the soil and tunnel

caused by tunnelling and compared recorded ground movements with results from existing empirical and analytical approaches [2]. Loganathan *et al* [2] evaluated the tunnelling-induced ground deformations and their impact on adjacent pile foundations by conducting three centrifuge model tests in clays. Jacobsz *et al* [3] carried out a study using centrifuge to examine how tunnelling in dense, dry sand will affect surrounding single piles and found that volume loss resulted by tunnelling may cause large pile settlements within a zone of influence. Marshall *et al* [4] analyse ground movement by conducting centrifuge tests due to tunnelling in sands and assessed the impact of tunnel dimensions and volume loss on soil deformation under greenfield conditions. Many researchers have given analytical methods to anticipate tunnelling-induced ground

## Papers Presented in International Conferences

3. Agarwal, J., Sarkar, R. (2024). Twin Tunneling-Induced Ground Deformations Under Greenfield Conditions: Parametric Study. In: Sivakumar Babu, G.L., Mulangi, R.H., Kolathayar, S. (eds) Technologies for Sustainable Transportation Infrastructures. SIIOC 2023. Lecture Notes in Civil Engineering, vol 529. Springer, Singapore.



4. Agarwal, J., Sarkar, R. (2024). Effect of Tunnelling on an Adjacent Pile. Paper presented at the 4th International Conference on Futuristic and Sustainable Aspects in Engineering and Technology (FSAET-2023), GLA University, Mathura, India, November 28–30, 2023.





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## CURRICULUM VITAE/ BRIEF PROFILE

**Ms. Jyoti Agarwal** is currently pursuing her Ph.D. in Geotechnical Engineering at *Delhi Technological University (DTU), New Delhi*. She holds an M.Tech. in *Soil Dynamics* from *IIT Roorkee*, where she was awarded the *DAAD Scholarship* to conduct her dissertation research at the *University of Stuttgart, Germany*. She completed her B.Tech. in Civil Engineering from *JSS Academy of Technical Education, Noida (UPTU)*.

With over **10 years of professional experience**, she has worked with leading consultancy organizations such as *AIMIL Ltd., TYPISA India* and *Rodic Consultants* as a Geotechnical Consultant and Design Engineer. Her expertise spans the geotechnical design and analysis of underground metro structures, deep excavations, foundations, slope stability, and building risk assessment for tunnelling projects.

Her ongoing doctoral research focuses on *tunnelling-induced ground deformations and their effects on nearby structures*. She has published several research papers in reputed journals and international conferences. In recognition of her outstanding contributions to research, she was honoured with the **Commendable Research Award for Excellence in Research (2025)** by *Delhi Technological University*.

Ms. Agarwal has also received multiple academic distinctions, including the *Indian Service of Engineers Prize*, and has consistently demonstrated excellence in both academic and professional domains.