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THREE-DIMENSIONAL MODELLING AND ASSESSMENT OF THE EFFECTS OF NEW DEVELOPMENTS IN EXISTING TUNNELS

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To my Parents,

"Nothing can cure the soul but the senses, just as nothing can cure the senses but the soul"

"Lord Henry" in the novel *The Picture of Dorian Gray* by Oscar Wilde

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ABSTRACT

In an urban environment, underground congestion is becoming increasingly important to asset owners and stakeholders. As the complexity of the underground network increases, it becomes fundamental for tunnel engineers to assess the way existing tunnels will perform under new load conditions.

Within the Bond Street Station Upgrade project, which is part of improvement programme across London's transport network undertaken by London Underground Limited, the construction of a new Sprayed Concrete Lining (SCL) passage to improve access to the existing Jubilee Line Platforms, crossed right underneath an existing cast iron Post Office (POT). During the construction process of the tunnel underpass the predictions from ground movements assessments did not correlate with monitoring results, triggering some questions regarding the suitability of the existing methods to predict movements in existing tunnels that this study intends to clarify.

In order to develop a progressive understanding of the Post Office Tunnel expected movements a Xdisp model was developed followed by a two-dimensional numerical model undertaken in PLAXIS 2D. However, the effects of new construction on the vicinity of existing underground structures are difficult to determine using simplistic empirical approaches or 2D analysis. Hence, in order to refine the understanding of the expected movement, a three dimensional finite element method model was also developed using PLAXIS 3D, taking into account the details of the real construction process.

The results obtained from the different approaches were compared with monitoring data provided from the Post Office Tunnel real movement, allowing to a sensitivity analysis to be carried out, in which the influence of various parameters, both geotechnical and structural, was assessed. The results interesting lessons for future similar projects.

KEYWORDS: Existing tunnels, SCL, cast iron tunnels, 3D numerical analysis, soil and tunnel displacements.

RESUMO

Num ambiente urbano, o congestionamento subterrâneo está a tornar-se cada mais importante para os proprietários de túneis já existentes. À medida que a complexidade das redes subterrâneas aumenta, torna-se fundamental para os engenheiros de túneis avaliar o modo como os túneis existentes se comportam sobre novas condições de carregamento.

Enquadrado no projeto de melhoria da estação de Bond Street, que é parte de um extenso programa de melhoramento em toda a rede de transportes de Londres realizado pela empresa London Underground Limited, a construção de uma nova passagem desenvolvida através da técnica de *Sprayed Concrete Lining* (SCL) para melhorar o acesso às plataformas da linha Jubilee, cruzou imediatamente abaixo do túnel de ferro fundido da Post Office. Durante a construção da passagem, os deslocamentos previstos pelas análises anteriormente realizadas não coincidiram com os dados da monitorização, levantando deste modo algumas questões sobre a adequação dos métodos existentes para prever movimentos de túneis existentes, que este estudo pretende clarificar.

De modo a aprofundar progressivamente o conhecimento sobre os movimentos esperados do túnel da Post Office, foi desenvolvido um modelo em Xdisp, seguido de um modelo a duas dimensões desenvolvido em PLAXIS 2D. Contudo, os efeitos que novas construções produzem em estruturas existentes nas proximidades é difícil de interpretar através de abordagens empíricas ou análises 2D. Por isso, de modo a aperfeiçoar o estudo dos movimentos expectáveis, foi ainda desenvolvido um modelo de elementos finitos em PLAXIS 3D, desta forma sendo possível incluir detalhes do processo de construção.

Os resultados obtidos a partir das diferentes abordagens foram posteriormente comparados com os dados fornecidos da monitorização do túnel da Post Office, permitindo deste modo a realização de uma análise de sensibilidade de vários parâmetros, tanto geotécnicos como estruturais. Os resultados revelaram lições interessantes para projetos futuros do mesmo âmbito.

PALAVRAS-CHAVE: Túneis existentes, SCL, túneis em ferro fundido, análise numérica 3D, deslocamentos.

TABLE OF CONTENTS

ACKNOWLEDGMENTS	i
ABSTRACT	iii
RESUMO	v
1. INTRODUCTION	1
1.1. BACKGROUND	1
1.2. MOTIVATION AND GOALS	2
1.3. LAYOUT OF THESIS	3
2. TUNNEL INDUCED GROUND MOVEMENTS	5
2.1. OVERVIEW	5
2.3. PHENOMENOLOGY OF THE TUNNEL CONSTRUCTION	5
2.3. EXCAVATION METHODOLOGIES	8
2.3.1. INTRODUCTION	8
2.3.2. CUT AND COVER TUNNELLING	9
2.3.3. DRILL AND BLAST TUNNELLING	9
2.3.4. TUNNEL BORING MACHINES	9
2.3.4. SEQUENTIAL EXCAVATION METHOD	10
2.3.4.1. New Austrian Tunnelling Method (NATM)	10
2.3.4.2. Sprayed Concrete Lining Method (SCL)	12
2.4. GROUND MOVEMENTS IN GREENFIELD CONDITIONS	14
2.4.1. INTRODUCTION	14
2.4.2. SURFACE MOVEMENTS	15
2.4.2.1. Transverse displacements	15
2.4.2.2. Longitudinal displacements	18
2.4.3. SUBSURFACE MOVEMENT	19
3. LONDON GEOLOGY AND ITS GEOTECHNICAL PROPERTIES	25
3.1. GEOLOGICAL BACKGROUND	25
3.2. LONDON CLAY	27

3.2.1. GEOLOGICAL HISTORY	27
3.2.2. LITHOLOGICAL CHARACTERISTICS AND DIVISIONS OF LONDON CLAY.....	28
3.2.3. GEOLOGICAL STRUCTURES	30
3.2.4. HYDROGEOLOGY.....	30
3.3. GEOTECHNICAL CHARACTERIZATION	31
3.3.1. IN-SITU EARTH PRESSURE COEFFICIENT, K_0	32
3.3.2. UNDRAINED SHEAR STRENGTH, c_u	34
3.3.3. SOIL STIFFNESS – SHEAR MODULUS (G) AND YOUNG’S MODULUS (E).....	36
 4. CASE STUDY.....	 39
4.1. PROJECT DESCRIPTION - BOND STREET STATION UPGRADE.....	39
4.1.1. SITE LOCATION	41
4.1.2. POST OFFICE TUNNELS	42
4.1.3. CONSTRUCTION METHOD	44
4.1.3.1. Under-crossing Post Office Tunnel.....	45
4.1.3.2. Tunnelling works programme	46
4.2. GROUND MODEL AND DESIGN PARAMETERS.....	47
4.2.1. GROUND MODEL	48
4.2.1.1. Ground conditions	48
4.2.1.2. Hydrogeology.....	49
4.2.2. GEOTECHNICAL DESIGN PARAMETERS	49
4.3. MONITORING DATA	50
4.3.1. MONITORING PLAN.....	51
4.3.2. MONITORING DATA	52
4.3.2.1. Vertical displacements.....	53
4.3.2.2. Horizontal displacements	57
4.3.2.3. Longitudinal displacements	62
4.3.2.4. Radial movement.....	63
 5. TWO-DIMENSIONAL NUMERICAL ANALYSIS	 67
5.1. INTRODUCTION	67
5.2. XDISP ANALYSIS/ PRELIMINARY RESULTS	67
5.2.1. ANALYSIS METHODS	68

5.2.2. XDISP MODEL	68
5.2.3. RESULTS	70
5.2.3.1. Surface displacements	70
5.2.3.2. POT displacements	72
5.3. PLAXIS 2D MODEL	74
5.3.1. MODEL FEATURES	74
5.3.1.1. Constitutive material models and parameters	75
5.3.1.2. Geometry and boundaries	77
5.3.1.3. Mesh generation	79
5.3.2. STAGED CONSTRUCTION	81
5.3.3. RESULTS FROM THE BASE MODEL	82
5.3.3.1. Deformed mesh	82
5.3.3.2. Stress state	83
5.3.3.3. Yielding	89
5.3.3.4. Ground displacements	91
5.3.3.5. SCL tunnel' lining forces	92
5.3.4. POT EXPECTED BEHAVIOUR	94
5.3.4.1. Vertical displacements	95
5.3.4.2. Longitudinal displacements	96
5.3.5. SENSITIVITY ANALYSIS	98
5.3.5.1. Young's modulus influence, E'	98
5.3.5.2. Undrained shear strength, c_u	100
5.3.5.3. In-situ earth pressure coefficient, K_0	101
5.3.6. BACK-ANALYSIS	103
 6. THREE-DIMENSIONAL FINITE ELEMENT ANALYSIS ...	 107
6.1. PLAXIS 3D MODELLING	107
6.1.1. GEOMETRY AND BOUNDARIES	107
6.1.2. MESH GENERATION	111
6.2. EXCAVATION SEQUENCE	114
6.3. RESULTS - BASE MODEL	120
6.3.1. SOIL BEHAVIOUR	120
6.3.1.1. Initial deformations - Post Office Tunnel construction	120

6.3.1.2. Deformed after SCL tunnel construction	121
6.3.1.3. Ground displacements.....	123
6.3.1.4. Stress state.....	128
6.3.1.5. Stress paths.....	135
6.3.1.6. Yielding	138
6.3.2. SCL TUNNEL' LINING FORCES	140
6.3.3. POT BEHAVIOUR.....	143
6.3.3.1. Vertical displacements.....	146
6.3.3.2. Horizontal displacements	148
6.3.3.3. Longitudinal displacements	150
6.4. SENSITIVITY ANALYSIS.....	152
6.4.1. INFLUENCE OF THE SOIL PROPERTIES	153
6.4.1.1. Young's modulus influence, E'	153
6.4.1.2. Undrained shear strength, c_u	155
6.4.1.3. In-situ earth pressure coefficient, K_0	157
6.4.2. INFLUENCE OF THE TUNNEL LININGS PROPERTIES.....	159
6.4.2.1. SCL tunnel lining properties	159
6.4.2.2. POT lining properties.....	161
6.4.3. ADDITIONAL STUDIES.....	163
6.4.3.1. Muir Wood approach	163
6.4.3.2. Anisotropic analysis for the POT lining.....	166
7. CONCLUSION.....	171
7.1. CONCLUSIONS.....	171
7.2. FURTHER DEVELOPMENTS	174
BIBLIOGRAPHIC REFERENCES	175
APPENDIX A - VALIDATION OF THE β-METHOD.....	181

LIST OF FIGURES

Figure 1.1 - Urban and rural population of the world (1950-2050) (reproduced from United Nations, 2014)	1
Figure 2.1 - Arching effect: mobilization of the shear strength in the vicinity of the excavation (reproduced from França, 2006)	6
Figure 2.2 - Direction of the principal stresses (reproduced from França, 2006); a) In an intact soil mass; b) After tunnel excavation	7
Figure 2.3 - Three-dimensional arching effect (reproduced from Eisenstein <i>et al.</i> , 1984)	7
Figure 2.4 - Influence of the excavation face: ground mass displacements (after França, 2006)	8
Figure 2.5 - Example of a configuration including twin sidewall drifts – Santiago Metro, Santiago, Chile (Dr. Sauer Group, 2010)	11
Figure 2.6 - 'Type 1' excavation sequence - twin side gallery and central core (ICE, 1996)	13
Figure 2.7 - 'Type 2' excavation sequence - side gallery and enlargement (ICE, 1996)	13
Figure 2.8 - 'Type 3' excavation sequence - top heading/crown, bench and invert (ICE, 1996)	13
Figure 2.9 - 'Type 4' excavation sequence - pilot tunnel and enlargement (ICE, 1996)	14
Figure 2.10 - Geometry of the tunnel induced settlement trough (after Attewell <i>et al.</i> , 1986)	15
Figure 2.11 - Transverse settlement trough (reproduced from Franzius, 2003)	16
Figure 2.12 - Representation of the volume of settlement trough	16
Figure 2.13 - Horizontal surface displacement distribution and strain in the transverse direction (reproduced from Franzius, 2003)	18
Figure 2.14 - Longitudinal settlement profile (reproduced from Franzius, 2003)	19
Figure 2.15 - Variation of trough width parameter i of subsurface settlement troughs with depth (after Mair <i>et al.</i> , 1993)	20
Figure 2.16 - Variation of K of subsurface settlement troughs with depth (after Mair <i>et al.</i> , 1993)	21
Figure 2.17 - Subsurface settlement above tunnel centre line (after Mair <i>et al.</i> , 1993)	22
Figure 2.18 - a) Distribution of i for subsurface settlement troughs with depth; b) Focus of vectors of soil movement (after Grant & Taylor, 2000)	23
Figure 3.1 - Geological map of southeast England displaying London Basin (Woudloper, 2009)	25
Figure 3.2 - Lithological units in London Clay proposed by King, 1981 (adapted by Wright, 2010)	29
Figure 3.3 - Undrained behaviour of reconstituted London Clay in undrained triaxial compression and extension tests; all samples were K_0 -consolidated to $\sigma'_v = 400 \text{ kPa}$ and unloaded to OCRs 1, 1.5, 3 and 7 (reproduced from Hight <i>et al.</i> , 2003, original data by Jardine, 1985)	31
Figure 3.4 - State Boundary surface and Local Boundary surface (reproduced from Zdravkovic and Jardine, 2000, original concept after Gens, 1982)	32
Figure 3.5 - Different approaches on K_0 variation with depth (reproduced from Wright, 2010)	34

Figure 3.6 - Illustration of shear stress - strain behaviour in overconsolidated soils.....	35
Figure 3.7 - Failure envelope for Mohr-Coulomb model.....	36
Figure 3.8 - Variation of London Clay shear stiffness in comparison with strain (reproduced from Wright, 2010).....	37
Figure 4.1 - Location of Bond Street Station in London (Google Maps © 2016).....	39
Figure 4.2 - Bond Street Station, Central and Jubilee lines location (excerpt of the London Underground map).....	40
Figure 4.3 - Overall view of Bond Street Station Upgrade (reproduced from Kumpfmüller <i>et al.</i> , 2014).....	41
Figure 4.4 - View of the site location (Bing Maps © 2016)	42
Figure 4.5 - Typical section through a Post Office Tunnel	43
Figure 4.6 - Representative scheme of cast iron segment's longitudinal section	43
Figure 4.7 - Cross sections, courtesy of HAT (dimensions in mm); a) Ovaloid section; b) POTU section	44
Figure 4.8 - Excavation sequence (HAT courtesy).....	45
Figure 4.9 - Plan view – draft (HAT courtesy)	45
Figure 4.10 - Construction of the underpass: exposing the POT cast iron lining (HAT courtesy).....	46
Figure 4.11 - Schematic representation of the monitoring instruments - typical location in Post Office Tunnel section (looking West)	51
Figure 4.12 - Illustration of the location of the monitoring sections	52
Figure 4.13 - Exact prisms location along the Post Office Tunnel (North elevation).....	52
Figure 4.14 - Raw data recorded by RP3 in all arrays throughout all excavation process: Vertical displacements (CoLOR courtesy).....	53
Figure 4.15 - Vertical displacements along POT; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5.....	55
Figure 4.16 - Schematic illustration of the excavation face reaching the crossing section	56
Figure 4.17 - Raw data recorded in array 5 throughout all excavation process: Horizontal displacements (CoLOR courtesy).....	58
Figure 4.18 - Raw data recorded in array 4 throughout all excavation process: Horizontal displacements (CoLOR courtesy).....	59
Figure 4.19 - Raw data recorded in array 3 throughout all excavation process: Horizontal displacements (CoLOR courtesy).....	60
Figure 4.20 - Horizontal displacements along POT; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5	61
Figure 4.21 - Longitudinal displacements along POT; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5	63
Figure 4.22 - Convergence chord reference system	64
Figure 4.23 - Diametric displacement throughout all excavation process; a) Array 1; b) Array 3; c) Array 5.....	65

Figure 4.24 - Schematic representation of the deformed cross-section of the POT as recorded in array 4 (looking West).....	66
Figure 5.1 - 2D view of the undercrossing section geometry (dimensions in meters)	69
Figure 5.2 - 3D model view	69
Figure 5.3 - Vertical displacement contours at the surface level	70
Figure 5.4 - Horizontal displacement contours at the surface level	71
Figure 5.5 - Deformed grid at surface level	71
Figure 5.6 - Ground surface displacements in the POT axis alignment.....	72
Figure 5.7 - Displacements in the POT crown	73
Figure 5.8 - Displacements in the POT invert	74
Figure 5.9 - Stress circles at yield; a) Mohr-Coulomb failure criterion; b) Tresca failure criterion (reproduced from PLAXIS, 2015a)	76
Figure 5.10 - Influence of the model boundaries on the ground behaviour regarding total displacements	78
Figure 5.11 - Global geometry and boundary conditions - 2D model (dimensions represented in meters)	79
Figure 5.12 - 2D model mesh.....	80
Figure 5.13 - Representation of the mesh quality from PLAXIS 2D model.....	81
Figure 5.14 - Deformed mesh of the 2D model (scaled up 30 times)	83
Figure 5.15 - Initial total stress state; a) Vertical stresses, σ_{zz} ; b) Horizontal stresses, σ_{xx} , σ_{yy}	83
Figure 5.16 - Initial effective stress state; a) Vertical stresses, σ'_{zz} ; b) Horizontal stresses, σ'_{xx} , σ'_{zz} ..	84
Figure 5.17 - Effective vertical stresses in the end of the excavation, σ'_{yy}	85
Figure 5.18 - Direction of the principal stresses after the excavation (arching effect)	85
Figure 5.19 - Effective vertical stresses evolution during the excavation	86
Figure 5.20 - Pore water pressures evolution during the excavation.....	87
Figure 5.21 - Effective horizontal stresses in the end of the excavation, σ'_{xx}	88
Figure 5.22 - Effective horizontal stresses evolution during excavation	88
Figure 5.23 - Tunnel boundary's reference line	89
Figure 5.24 - Yielding history throughout the excavation process; a) Top heading excavation; b) Lining application in the top heading; c) Invert excavation; d) Lining application in the remaining boundary .	90
Figure 5.25 - Vertical displacements in the end of the excavation, u_y	91
Figure 5.26 - Horizontal displacements in the end of the excavation, u_x	92
Figure 5.27 - Diagram of the axial forces in the tunnel lining; a) Temporary stage; b) Final stage	93
Figure 5.28 - Diagram of the bending moments in the tunnel lining; a) Temporary stage; b) Final stage	93

Figure 5.29 - Representation of the settlement trough in its reference baseline	95
Figure 5.30 - POT expected vertical displacements.....	96
Figure 5.31 - Representation of the longitudinal displacements in its reference baseline	97
Figure 5.32 - POT expected longitudinal displacements.....	97
Figure 5.33 - Vertical POT expected displacements for different values of the Young's modulus, E ...	99
Figure 5.34 - Longitudinal POT expected displacements for different values of the Young's modulus, E	100
Figure 5.35 - Vertical POT expected displacements for different values of the undrained shear strength, c_u	101
Figure 5.36 - Longitudinal POT expected displacements for different values of the undrained shear strength, c_u	101
Figure 5.37 - Vertical POT expected displacements for different values of the in-situ earth pressure coefficient, K_0	102
Figure 5.38 - Longitudinal POT expected displacements for different values of the in-situ earth pressure coefficient, K_0	103
Figure 5.39 - Vertical POT expected displacements for different relaxation factors	104
Figure 5.40 - Longitudinal POT expected displacements for different relaxation factors.....	104
Figure 5.41 - Vertical POT expected displacements for a lower bound of the in-situ earth pressure coefficient, K_0	105
Figure 5.42 - Longitudinal POT expected displacements for a lower bound of the in-situ earth pressure coefficient, K_0	105
Figure 6.1 - Global view of the 3D model's geometry; a) Ground model dimensions; b) Post Office and SCL tunnels length and relative position	108
Figure 6.2 - SCL tunnel external surface developed in AutoCAD	109
Figure 6.3 - View of the AutoCAD triangular mesh and indication of the SCL tunnel dimensions (in meters) for the 3D model.....	109
Figure 6.4 - Intersection procedure in PLAXIS; a) Vertical surfaces; b) Top heading/ invert excavation dividing surface.....	110
Figure 6.5 - Top heading/ invert excavation dividing surface created in AutoCAD	110
Figure 6.6 - Post Office and SCL tunnels relative position, reproduced from PLAXIS 3D model	111
Figure 6.7 - Example of a badly shaped mesh.....	112
Figure 6.8 - Coarseness factors adopted for each section of the SCL tunnel.....	113
Figure 6.9 - Final mesh adopted for the base model; a) SCL tunnel lining' mesh; b) POT lining' mesh	113
Figure 6.10 - Representation of the mesh quality scale	114
Figure 6.11 - Representation of the excavation advances modelled in PLAXIS 3D	117
Figure 6.12 - Representation of the different lining's stiffness to model concrete strengthening.....	119

Figure 6.13 - Three-dimensional view of excavation sequences; a) Application of the lining at advance No. 7; b) Excavation at advance No. 16.....	119
Figure 6.14 - Deformed mesh in a cross section perpendicular to POT corresponding to <i>POT construction stage</i> (scaled up 700 times).....	120
Figure 6.15 - POT deformed lining after POT construction (scaled up 400 times).....	121
Figure 6.16 - Deformed mesh of part of the 3D model in the final stage of the excavation (scaled up 400 times).....	122
Figure 6.17 - Vertical displacements in a cross section perpendicular to SCL tunnel and 4.2 m distant from the POT centre-line (u_z).....	123
Figure 6.18 - Horizontal displacements in a cross section perpendicular to SCL tunnel and 4.2 m distant from the POT centre-line (u_y).....	124
Figure 6.19 - Vertical displacements in a cross section perpendicular to SCL tunnel intersecting the POT centre-line (u_z).....	125
Figure 6.20 - Horizontal displacements in a cross section perpendicular to SCL tunnel intersecting the POT centre-line (u_y).....	125
Figure 6.21 - Vertical displacements in a cross section along SCL tunnel (u_z).....	126
Figure 6.22 - Horizontal displacements in a cross section along SCL tunnel (u_x).....	127
Figure 6.23 - Horizontal displacements in a cross section along SCL tunnel representing advance No. 14 (u_x).....	128
Figure 6.24 - Horizontal displacements in a cross section along SCL tunnel representing advance No. 15 (u_x).....	128
Figure 6.25 - Initial total stress state; a) Vertical stresses, σ_{zz} ; b) Horizontal stresses, σ_{xx} , σ_{yy}	129
Figure 6.26 - Initial effective stress state; a) Vertical stresses, σ'_{zz} ; b) horizontal stresses, σ'_{xx} , σ'_{yy} ..	129
Figure 6.27 - Vertical stress state after advance No. 27 was performed (σ'_{zz}); a) Cross section along the SCL tunnel; b) Cross section perpendicular to the SCL tunnel taken 1 m behind the excavation face	130
Figure 6.28 - Representation of the notable point's location.....	131
Figure 6.29 - Evolution of the effective vertical stresses throughout all excavation process.....	131
Figure 6.30 - Horizontal stress state perpendicular to the SCL tunnel longitudinal axis after advance No. 27 was performed (σ'_{yy}); a) Cross section along the SCL tunnel; b) Cross section perpendicular to the SCL tunnel taken 1 m behind the excavation face	132
Figure 6.31 - Evolution of the effective horizontal stresses perpendicular to the SCL tunnel longitudinal axis throughout all excavation process (σ'_{yy}).....	133
Figure 6.32 - Horizontal stress state parallel to the SCL tunnel longitudinal axis after advance No. 27 was performed (σ'_{xx}); a) Cross section along the SCL tunnel; b) Cross section perpendicular to the SCL tunnel taken 1 m behind the excavation face	133
Figure 6.33 - Evolution of the effective horizontal stresses parallel to the SCL tunnel longitudinal axis throughout all excavation process (σ'_{xx})	134

Figure 6.34 - Effective principal stresses direction; a) Cross section along the SCL tunnel; b) Cross section perpendicular to the SCL tunnel taken 1 m behind the excavation face	135
Figure 6.35 - Relative position of the reference points; a) Longitudinal location; b) POTU section ; c) Ovaloid section	136
Figure 6.36 - Stress path in (s,t) space for the POTU section; a) Point A; b) Point B	136
Figure 6.37 - Stress path in (s,t) space for the Ovaloid section; a) Point A'; b) Point B'	137
Figure 6.38 - Stress path in (s,t) space for the Ovaloid section; a) Point A''; b) Point B''	137
Figure 6.39 - Yielding area corresponding to the performance of advance No. 5.....	138
Figure 6.40 - Yielding area; a) Top heading excavation – advance No. 3; b) Invert excavation – advance No. 4	139
Figure 6.41 - Yielding area developed in advance No. 8	139
Figure 6.42 - Arching effect corresponding to advance No. 8	140
Figure 6.43 - Axial forces in the longitudinal direction in the end of the excavation.....	141
Figure 6.44 - Axial forces in the radial direction in the end of the excavation	141
Figure 6.45 - Bending moments in the longitudinal direction in the end of the excavation	142
Figure 6.46 - Bending moments in the radial direction in the end of the excavation	142
Figure 6.47 - Diagram of the axial forces in the tunnel lining; a) Temporary stage; b) Complete section	143
Figure 6.48 - Diagram of the bending moments in the tunnel lining; a) Temporary stage; b) Complete section.....	143
Figure 6.49 - Representation of the POT deformed mesh at advance No. 27	144
Figure 6.50 - Maximum vertical displacement of the POT throughout the entire excavation process	145
Figure 6.51 - Maximum horizontal displacement of the POT throughout the entire excavation process	145
Figure 6.52 - Example of the reference-line used to obtain the POT displacements (along RP3)	146
Figure 6.53 - Vertical displacements along POT obtained from PLAXIS 3D; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5.....	147
Figure 6.54 - Horizontal displacements along POT obtained from PLAXIS 3D; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5	149
Figure 6.55 - Longitudinal displacements along POT obtained from PLAXIS 3D; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5	151
Figure 6.56 - Vertical displacements recorded within RP1 for different values of the Young's modulus, E	154
Figure 6.57 - Horizontal displacements recorded within RP1 for different values of the Young's modulus, E	154
Figure 6.58 - Longitudinal displacements recorded within RP3 for different values of the Young's modulus, E	155

Figure 6.59 - Vertical displacements recorded within RP1 for different values of the undrained shear strength, c_u	156
Figure 6.60 - Horizontal displacements recorded within RP1 for different values of the undrained shear strength, c_u	156
Figure 6.61 - Longitudinal displacements recorded within RP3 for different values of the undrained shear strength, c_u	157
Figure 6.62 - Vertical displacements recorded within RP1 for different values of the in-situ earth pressure coefficient, K_0	158
Figure 6.63 - Horizontal displacements recorded within RP1 for different values of the in-situ earth pressure coefficient, K_0	158
Figure 6.64 - Longitudinal displacements recorded within RP3 for different values of the in-situ earth pressure coefficient, K_0	159
Figure 6.65 - Vertical displacements recorded within RP1 for different values of the concrete lining stiffness, E_c	160
Figure 6.66 - Horizontal displacements recorded within RP1 for different values of the concrete lining stiffness, E_c	160
Figure 6.67 - Longitudinal displacements recorded within RP3 for different values of the concrete lining stiffness, E_c	161
Figure 6.68 - Vertical displacements recorded within RP1 for different values of the POT lining stiffness, E_{POT}	161
Figure 6.69 - Horizontal displacements recorded within RP1 for different values of the POT lining stiffness, E_{POT}	162
Figure 6.70 - Longitudinal displacements recorded within RP3 for different values of the POT lining stiffness, E_{POT}	162
Figure 6.71 - Vertical displacements recorded for the Muir Wood approach within different prisms; a) RP1; b) RP3; c) RP5	164
Figure 6.72 - Horizontal displacements recorded for the Muir Wood approach within different prisms; a) RP1; b) RP5	165
Figure 6.73 - Longitudinal displacements recorded within RP3 for the Muir Wood approach	165
Figure 6.74 - Cast iron tunnels lining joints details (reproduced from Schroeder, <i>et al.</i> , 2011); a) Radial joint; b) Circumferential joint.....	166
Figure 6.75 - Vertical displacements recorded for different values of the POT's longitudinal stiffness, E_{POT}^{long} within different prisms; a) RP1; b) RP3; c) RP5.....	168
Figure 6.76 - Horizontal displacements recorded for different values of the POT's longitudinal stiffness, E_{POT}^{long} within different prisms; a) RP1; b) RP5.....	169
Figure 6.77 - Deformed POT cross-section on the reference section in advance No. 16; a) Base Model; b) Anisotropic approach, $E_{long} = 1.8 \text{ GPa}$	169
Figure 6.78 - Longitudinal displacements recorded for different values of the POT's longitudinal stiffness, E_{POT}^{long} within different prisms; a) RP1; b) RP3.....	170

LIST OF TABLES

Table 4.1 - Cast iron tunnel segment's longitudinal section dimensions	43
Table 4.2 - Underpass advances progress	46
Table 4.3 - Simplified ground profile and geological description.....	48
Table 4.4 - Geotechnical design parameters to be adopted (CH2M, 2013)	49
Table 5.1 - Concrete lining properties introduced in PLAXIS 2D	77
Table 5.2 - Variation of the Young's modulus [MPa]	99
Table 5.3 - Variation of the undrained shear strength [kPa]	100
Table 5.4 - Variation of the in-situ earth pressure coefficient.....	102
Table 6.1 - Actual elastic section properties of the POT lining	115
Table 6.2 - Cast Iron lining properties used in PLAXIS 3D	116
Table 6.3 - Location of the SCL tunnel advances from the POT centre-line	117
Table 6.4 - Concrete lining properties as used in PLAXIS 3D	118

SYMBOLS, ACRONYMS AND ABBREVIATIONS

Latin alphabet

A - Area of section per meter length [m^2/m]

c' - Effective cohesion [kPa]

c_U - Undrained Shear Strength [kPa]

$Chord$ - Length of the reference chord

D - Outer diameter of the tunnel

E - Young's modulus [MPa]

E_{eq} - Equivalent Young's modulus [MPa]

G - Shear modulus [MPa]

h - Overall depth of section [mm]

I - Second moment of area per meter length [m^4/m]

i - Trough width parameter

K_0 - In-situ earth pressure coefficient

K_{nc} - Normally consolidated earth pressure coefficient

k - Trough width parameter in Xdisp

k_h - Horizontal permeability [m/day]

k_v - Vertical permeability [m/day]

OCR - Overconsolidation ratio

p' - Mean effective stress [kPa]

q' - Deviatoric effective stress [kPa]

S_{hx} (S_h) - Horizontal displacement in the transverse direction

S_{hy} (S_L) - Horizontal displacement in the longitudinal direction

S_V - Vertical displacement

$S_{V,max}$ - Maximum vertical displacement

t_{eq} - Equivalent thickness

t_f - Thickness of flange [mm]

t_w - Thickness of web [mm]

u_z, u_x, u_y - Displacements in z -, x - and y -direction, respectively, in PLAXIS

V_L - Volume loss [%]

V_S - Volume of the settlement trough

w - Width of section ring [mm]

z_0 - Depth of tunnel axis below the ground surface

Greek alphabet

γ - Unit weight [kN/m³]

Δ - Chord length variation

ν - Poisson's Ratio

σ'_h - Horizontal effective stress [kPa]

σ'_{nf} - Normal effective stress on the failure plane

σ'_p - Effective pre-consolidation stress [kPa]

σ'_v - Vertical effective stress [kPa]

σ'_{v0} - Effective vertical stress at rest [kPa]

$\sigma_{zz}, \sigma_{xx}, \sigma_{yy}$ - Total stress in z-, x- and y-direction, respectively

$\sigma'_{zz}, \sigma'_{xx}, \sigma'_{yy}$ - Effective stress in z-, x- and y-direction, respectively

τ - Shear stress [kPa]

τ_f - Shear stress on the failure plane

Φ - Cumulative probability function

ϕ' - Effective angle of shearing resistance [°]

ϕ'_c - Critical angle of shearing resistance [°]

ψ - Dilation angle [°]

BBSU - Bond Street Station Upgrade

CoLOR - joint venture between Costain and Laing O'Rourke

CPT - Cone penetration test

GIR - Geotechnical Interpretative Report

HAT - joint venture between Halcrow (now part of CH2M) and Atkins

ICE - Institute of Civil Engineers

ITA – International Tunnelling and Underground Space Association

LUL - London Underground Limited

NATM - New Austrian Tunnelling method

POT - Post Office Tunnel

POTU - Post Office tunnel Underpass

RP - Reference Prism

SCL - Sprayed Concrete Lining

SPT - Standard Penetration Tests

TBM - Tunnel Boring Machine

TfL - Transport for London

1

INTRODUCTION

1.1. BACKGROUND

The increasing urbanization and overall growth of the world's population is projected by the United Nations (2014) to add 2.5 billion people to the urban population by 2050. In 1950, only 30 per cent of the world's population was living in urban areas. However in 2007, for the first time, the global urban population exceeded the global rural population and the world's population has remained predominantly urban thereafter (Figure 1.1). In fact, the urban population is expected to continue growing and by 2050, 66 per cent of the world's population is projected to be urban (United Nations, 2014).

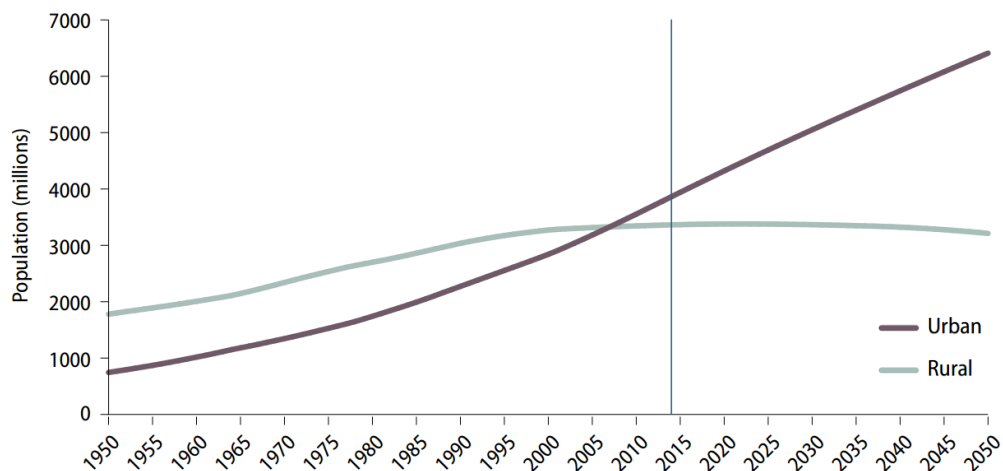


Figure 1.1 - Urban and rural population of the world (1950-2050) (reproduced from United Nations, 2014)

As the population pressure in metropolitan areas rise, and urban space becomes more limited, subsurface structures are becoming more efficient in providing the required infrastructures. The International Tunnelling and Underground Space Association (ITA), founded in 1974, encourages the use of the subsurface presenting strong environmental, social and economical reasons. One of the main environmental reasons they present to go underground, is the *land use and location* that mainly refers to the lack of surface space, which is now the case in nearly all towns and cities around the world.

Major world cities often respond to the increasing pressure with improvements, extensions or additions to their underground infrastructure complex. By expanding their networks of trains for example, cities

are able to answer to the current demand from communities for more areas free of road traffic and therefore promoting more efficient transportation.

London's population has grown strongly in recent years and is expected to reach 10 million people by 2031 (Financial Time's website, March 2014). In order to support the demand of all those extra people, substantial underground transport, power and sanitation projects are being undertaken to help to cope with the London's growth.

Besides its actual extensive underground infrastructure complex for electricity distribution, natural gas and water supply, telecommunications, and underground railway, many other underground structures continue to exist in London even though they are no longer in use.

As such, new tunnel projects have to be constructed within a congested underground space which will inevitably lead to interference in existing surface and subsurface structures. It is well known that ground movements are an inevitable consequence of excavating and constructing a tunnel in a soft soil. The excavation of a tunnel causes relaxation of in-situ stresses, which is partially restricted by the application of the tunnel support. It is not possible to create a void instantaneously and provide an infinitely stiff lining to fill it simultaneously, hence, a certain amount of the deformation of the ground will take place around the tunnel, which will trigger a chain of movements, resulting in settlements at the ground surface.

Therefore, estimating the effects of tunnelling in existing subsurface structures is becoming increasingly important in congested undergrounds such as London's, although it still receives less attention than the assessments of tunnelling effects on buildings.

Typical variety of underground assets to be considered when contemplating construction of a tunnel are for example, other tunnels, pile foundations (both existing and under construction) and pipelines for services. The response of these existing structures to tunnelling induced ground movements depends on their geometry, construction method and overall structural condition. This assessment becomes even more important when the infrastructure is old and vulnerable.

1.2. MOTIVATION AND GOALS

Following the above presented, the main objective of this Master Thesis is the assessment of the effects of new constructions on the vicinity of existing underground structures. During the construction process of a new underground passage in Bond Street station, within an improvement programme undertaken by London Underground Limited (LUL), which was performed in the vicinity of a cast iron tunnel it was noticed that the predictions from previous ground movement assessment did not adjust with monitoring results.

In order to understand the real behaviour of the existing tunnel due to the construction of the new tunnel, different approaches are undertaken. Primarily, an Xdisp model is developed reproducing a greenfield condition of the problem, which is then followed by a two-dimensional numerical model undertaken in PLAXIS 2D.

Notwithstanding, a greenfield approach may not be the best methodology to predict the behaviour of existing structures subjected to tunnel construction, since their presence and, consequently, their stiffness, also alter the ground movements around them. Due to its complexity and regarding the limitations of a 2D analysis, this interaction problem is preferable to be analysed through a three-dimensional numerical method which provides a more flexible tool that has been adopted by many authors.

In fact, it is predicted that sooner than later three-dimensional numerical analysis will become widely available and turn into a standard method for tunnel assessment and design. Hence, a 3D model is prepared in PLAXIS 3D, reflecting the existing structures and the construction stages of the new developments.

All models are built based on a case study suggested by CH2M, which is the company that provided the author all the facilities and working conditions to develop this thesis. The results from the different models are then compared with real monitoring data provided and some conclusions are taken.

The comparison of the obtained results with monitoring data, will allow a better understanding of the influence of various input parameters and material models adopted.

1.3. LAYOUT OF THESIS

This dissertation is structured in seven main chapters, although two main groups are identified. Chapters 2 and 3 together, form the state of art, while the remaining concern to the case study. The subjects developed in this thesis are organized as follows.

Chapter 2 introduces the phenomenology of a tunnel construction. Some excavation methods are also described within this chapter, and, finally, some analytical methods used to predict both surface and subsurface displacements, are presented.

In Chapter 3 the geology of London is briefly described as well as the main geotechnical properties of London Clay. The case study that was the basis of this dissertation was built within the London Clay Formation, which justifies the importance of a detailed characterization of this formation and a good understanding of its parameters.

The fourth chapter presents the case study that bases the numerical models built – the Underpass tunnel at Bond Street station. Primarily, a description of the project is presented, in which the construction methods, tunnels geometry and ground conditions are detailed. Then, the data provided from the monitoring plan installed in the tunnel to be assessed is presented and discussed.

In Chapter 5 the results from two preliminary analysis are presented. In the first instance, an Xdisp analysis is developed, whose results are based on the Gaussian distribution theory. Then, a numerical analysis developed in PLAXIS 2D is presented and the results extrapolated for comparison with the monitoring data as presented in Chapter 4. In addition, a sensitivity analysis is conducted to improve the developed model.

The sixth chapter presents the three-dimensional numerical analysis undertaken in PLAXIS 3D. The results from this model are directly compared with the monitoring data. Furthermore, a sensitivity analysis is also made within PLAXIS 3D, describing the influence of various parameters, both geotechnical and structural.

Finally, Chapter 7 summarizes the final notes and main conclusions of the obtained results, and presents some recommendations for future researches.

2

TUNNEL INDUCED GROUND MOVEMENTS

2.1. OVERVIEW

Tunnelling operations in soft ground lead inevitably to ground movements which in an urban environment can affect existing surface and subsurface structures. These movements occur as a consequence of stress relief before a lining is installed and they manifest themselves at the ground surface as a settlement trough. The magnitude and location of the ground movements depend on a number of factors including geological, hydro-geological and geotechnical conditions encountered, tunnel geometry and depth, as well as the techniques used for tunnel excavation and ground support.

Several techniques were developed throughout the years to perform a tunnel excavation. As stated above, ground movement is not only dependent on the ground conditions but also on the adopted techniques to construct a tunnel. In order to understand the influence of the different techniques on the ground behaviour, a brief introduction on the typical adopted techniques will be presented within this chapter.

Although the influence of the excavation methodologies is highlighted in this chapter, the main aim of the present chapter is to present some of the analytical approaches available in the literature to estimate both surface and subsurface ground movements. The phenomenology of a tunnel construction will be presented as an introduction for the subject.

2.3. PHENOMENOLOGY OF THE TUNNEL CONSTRUCTION

A tunnel construction in a ground mass which is previously in equilibrium, can be interpreted as the removal of the ground stresses in the contour of the excavation. The removal of the soil volume induces a new stress state in the vicinity of the excavation that searches for a new state of equilibrium. This equilibrium can be reached without any support, when the ground is classified as *self-supporting*, or with the application of a specific support system, as for example a sprayed concrete layer in the contour of the excavation in order to control the soil deformation (França, 2006).

The deformations allowed to the ground mass due to an excavation lead to a stress redistribution, in which the soil in the vicinity of the excavation is mobilized. In addition, the strength/stiffness of the lining, already applied in the contour of the excavation behind, is also mobilized due to stress redistribution as well, which then contributes to the generation of a new state of equilibrium. The mechanism that leads to this stress redistribution is called the arching effect.

The arching effect, introduced above, only occurs when the shear strength of the soil is mobilized in the vicinity of the excavation (Langer & Stockmann, 1985). In order to interpret this phenomenon, França, (2006) represents what occurs within a soil area above the crown of a tunnel, which is reproduced in Figure 2.1. The elements of the soil A, B and C are located exactly in the contour of the proposed excavation. After the excavation is performed, there is clear convergence of the tunnel walls inwards the excavation, which affects more element A. Hence, element A moves more than element B, which in turn moves more than element C. The difference between the movements of each element causes shear stresses between them. The relative movement between elements is propagated behind the excavation contour, consequently the soil in the vicinity is mobilized, searching for the equilibrium. This equilibrium is reached by the arching effect which in turn is only possible due to soil's shear strength.

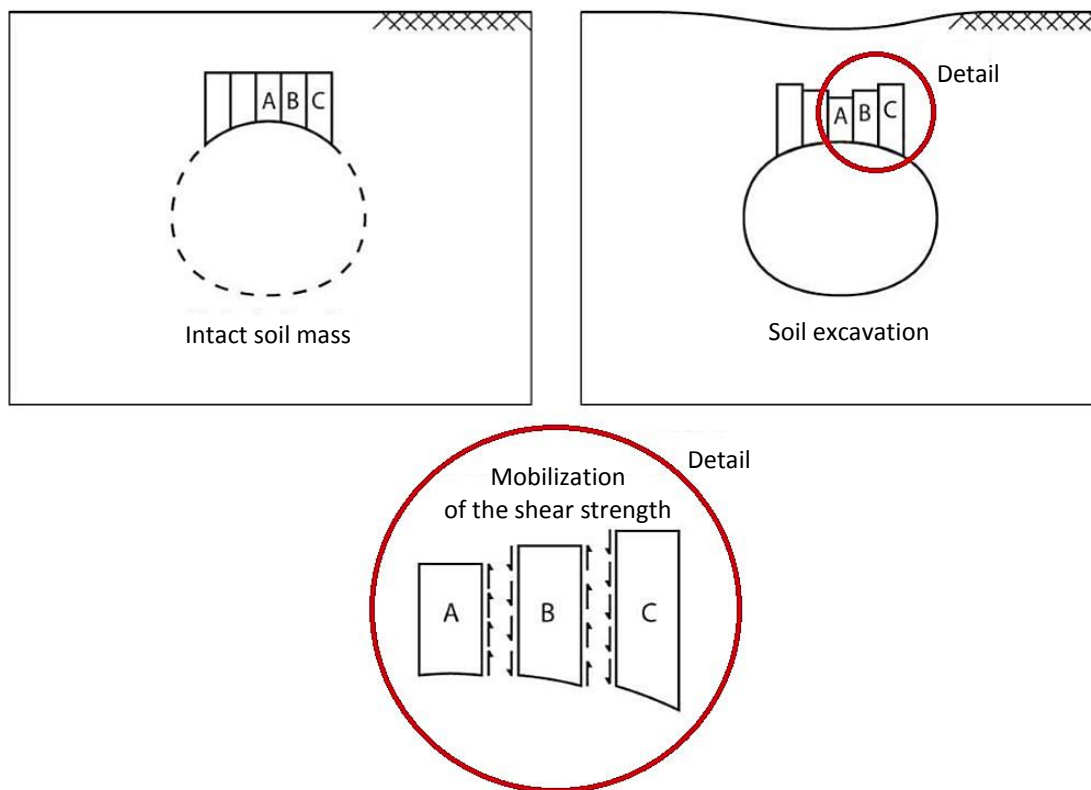


Figure 2.1 - Arching effect: mobilization of the shear strength in the vicinity of the excavation (reproduced from França, 2006)

In general, in an intact soil mass, the direction of the principal stresses before a tunnel excavation is undertaken, coincides with the vertical and horizontal directions (see Figure 2.2 a)). The direction of the principal stresses indicates the planes in which no shear stress occurs, thus it is possible to identify that before an excavation, in an ideal situation, there is no shear stress in the vertical and horizontal planes of the soil mass. However, as it was already introduced, a tunnel excavation necessarily mobilizes shear stresses in the vicinity of the tunnel contour, hence the planes in which no shear stress is identified change, inducing a rotation of the principal stresses direction, which is reproduced in Figure 2.2 b).

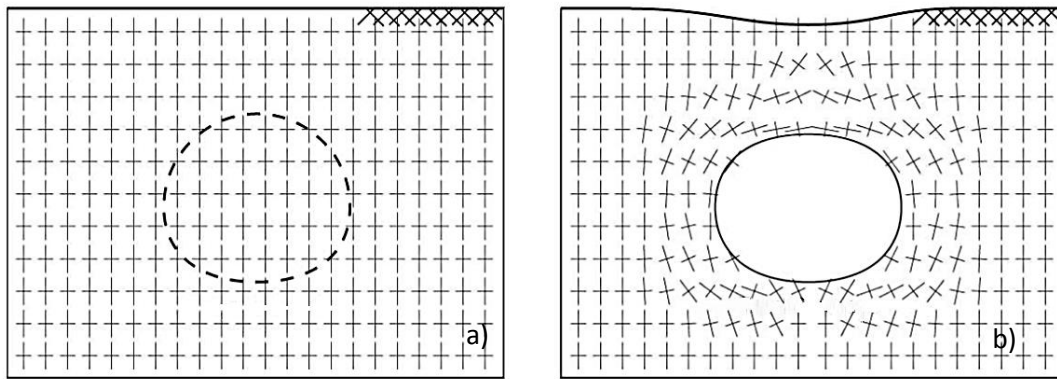


Figure 2.2 - Direction of the principal stresses (reproduced from França, 2006); a) In an intact soil mass; b) After tunnel excavation

The stress redistribution above mentioned, occurs not only in a plane perpendicular to the tunnel axis but also longitudinally, both in vertical and horizontal planes. Figure 2.3 represents the arching effect in the three different planes.

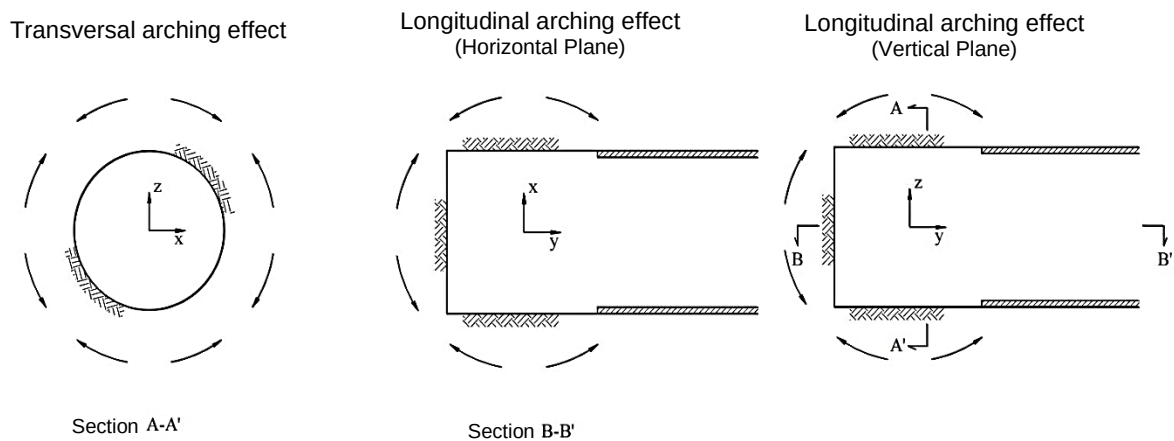


Figure 2.3 - Three-dimensional arching effect (reproduced from Eisenstein *et al.*, 1984)

As it was already introduced, a tunnel excavation leads inevitably to convergence of the ground in vicinity of the excavation. As Almeida e Sousa (1998) presented in his work, ahead of the excavation face the ground displacements are mainly longitudinally to the tunnel axis, and becomes even more important with the advance of the excavation face. In fact, the longitudinal displacements reach a maximum value when the excavation face is crossing a reference section. After the reference section is left behind, the longitudinal displacements start to record an opposite movement, disappearing at a certain distance. On the contrary, the importance of the radial displacements is greater for a certain distance from the excavation face, which is reached after the passage of the excavation face. Figure 2.4 represents the typical displacement field along the excavation axis.

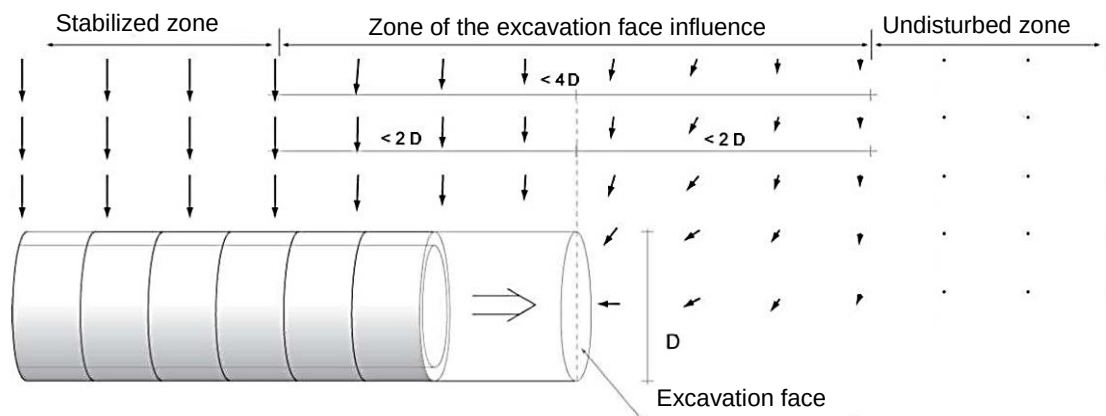


Figure 2.4 - Influence of the excavation face: ground mass displacements (after França, 2006)

The distance at which the effects of a tunnel construction are identified depends on the ground strength and on the support adopted. However, it is commonly adopted that, for the majority of the cases, the effect of the excavation is registered within two diameters, both ahead and behind the excavation face, as represented in Figure 2.4.

Thereafter, a plane strain analysis can be carried out for a section of the tunnel where the excavation face has no more influence since the equilibrium has been reached. However a plane strain analysis must only be applied when both geotechnical and geometrical properties remain constant along the tunnel axis (Almeida e Sousa, 1998). For a section close to the excavation face, a three-dimensional analysis is required.

2.3. EXCAVATION METHODOLOGIES

2.3.1. INTRODUCTION

The adoption of the tunnelling method more suitable for each situation may be dictated by geological and hydrological conditions, purpose, cross-section and length of tunnel, local experience, time/cost considerations and limits of surface disturbance, the latter, especially considered in urban areas.

It is quite common in the engineering literature to find a distinction between tunnelling techniques to be applied in soft ground or in rock, however, tunnelling techniques are now being used in a wider range of ground conditions and this boundary is becoming increasingly blurred. The major difference between both types of ground conditions rely on the stand-up time of the ground, which for soft soils is very short or almost non-existent. Therefore, the adoption of the construction method is driven by the need to support the ground immediately after the creation of the void, ensuring the stability of the created void, and controlling the deformations within acceptable limits.

This section presents a brief review on the most commonly adopted methods regarding the excavation process, comprising the cut and cover method, drill and blast, tunnelling boring machines and the Sequential Excavation Method, which comprises both New Austrian Tunnelling Method (NATM) and Sprayed Concrete Lining Method (SCL).

2.3.2. CUT AND COVER TUNNELLING

Cut and cover is a common and well-known technique for constructing shallow tunnels. It comprises three major parts: the excavation of a trench, which can be braced or anchored, the construction of the tunnel and finally the soil covering of excavated tunnels. This method can easily accommodate changes in tunnel width and non-uniform shapes, therefore it is commonly adopted for the construction of stations.

In locations with no important constraints this technique offers a good alternative to underground construction techniques, since it is often cheaper and more practical for tunnels to be constructed close to the ground surface (until 10-15 m deep). The development of more advanced techniques have been helping the construction of deeper tunnels (30 m) based on this construction method, one example of this techniques is the use of diaphragm walls to sustain the ground while the excavation is performed.

However in an urban environment such as London's, the site constraints are various and the construction of cut and cover tunnels can be very disruptive since the access to the ground surface over extended areas and for long periods of time is difficult (Chapman *et al.*, 2010).

2.3.3. DRILL AND BLAST TUNNELLING

This method involves the use of explosives. On the proposed tunnel face, drilling rings are used to bore blast holes to a designated depth for blasting. Posteriorly, explosives and timed detonators are placed in the blast holes. After blasting is carried out, waste rock and soil are transported out of the tunnel before further blasting.

The drill and blast method for tunnelling can be used in geology ranging from hard rock with low strength, e.g. marl, clay, chalk, to the hardest rock, such as granite or gneiss. Due to this large range of possible usage, drill and blast may be advantageous for very changeable conditions (Chapman *et al.*, 2010).

The ground vibrations induced by this method need to be carefully considered, especially in an urban area, once it can heavily affect surface and subsurface structures. Compared with bored tunnelling (introduced in the following section), blasting generally results in higher but lesser duration of vibration levels.

2.3.4. TUNNEL BORING MACHINES

The term Tunnel Boring Machine (TBM) is universally adopted for all the machines that have a full-face cutting wheel for excavating a tunnel (Guglielmetti, *et al.*, 2008). TBM's are often designed for specific projects, answering certain ground conditions and project specifications with a specific diameter, thereafter tunnelling machines can be found in many different diameters, ranging from microtunnel boring machines, with diameters smaller than 1 m, to machines for large tunnels, whose diameters are greater than 15 m (Chapman *et al.*, 2010).

The general function principle that governs a TBM consists of a cutterhead, rotating on the same axis of the tunnel being excavated, pushed against the excavation face by a series of thrust jacks, which crushes the ground locally by applying intense tensile and shear stresses. The cutterhead has special buckets that allow the muck to be collected and then removed and transported to the exterior of the tunnel.

Due to its characteristics, this tunnelling method is suitable for many different geological conditions, although it may require the selection of appropriate equipment for tunnel face and walls support.

Generally, tunnelling machines are subdivided according to the type of ground that it is able to operate in, and then, more specifically, according to the support typology that the machine is able to apply. Consequently, it is common to find the distinction between rock tunnelling machines and soft ground tunnelling machines, whose main differences are the cutter tools on the cutterhead, which is designed for the specific ground conditions expected, and the tunnel face support techniques, since the stability is generally lower in soft soils.

The machine is often equipped with a closed circular protective shield which allows the application of the tunnel lining almost immediately after excavation. In tunnel boring machines to be used in hard rock, the shield is normally just a small safety crown-shield, mounted at the back of the cutterhead, to avoid it catching on any rock as the ground deforms under high pressures.

All TBM's using segmental lining for the tunnel support, i.e., using prefabricated concrete rings segments bolted together to form the tunnel lining, need to have a tail seal at the rear end of the shield. When the ring segments are being applied onsite, the tunnel boring machine remains stationary. Once the tunnel lining ring is completed, excavation is resumed, until another excavation phase has been executed in order to perform the next lining ring. As the tunnel segments are erected within the tunnel shield, a gap between the segments and the excavated ground is formed, which is posteriorly injected with grout in order to achieve a rigid connection between the ground and the tunnel lining and consequently reducing, by recovering part of the ground movements.

Although TBM's can be used in different ground conditions, one of the general requirements for this tunnelling method is a consistent geology along the route of the tunnel, since the different cutting tools and the stabilization materials are only suitable for a small variation in the characteristics of the material encountered. However, the combination of different cutting tools on the cutterhead can increase the application of machines to a greater range of ground conditions. It is worth referring that extremely hard rock can cause significant wear of the TBM cutterhead and may slow down the progress of the tunnelling works to the point where TBM becomes inefficient and uneconomical and may take longer time than the drill and blast tunnelling method. However, a TBM causes much less damage to the surrounding rock than the use of explosives, providing a more gentle excavation.

2.3.4. SEQUENTIAL EXCAVATION METHOD

2.3.4.1. New Austrian Tunnelling Method (NATM)

The New Austrian Tunnelling Method, NATM, is one of the most adaptable and responsive tunnelling excavation and support methods available. ICE (1996) identifies four main principles to define this excavation method:

- The strength of the ground around a tunnel should be deliberately mobilized to the maximum extent possible;
- Mobilization of ground strength is achieved by allowing deformation of the ground;
- Initial or primary support, which has load-deformation characteristics appropriate to the ground conditions, is installed. Permanent support works are normally carried out at a later stage;
- Instrumentation is installed to monitor the deformations of the initial support system and the build-up load upon it. When appropriate, the results of this monitoring, form the basis for varying the primary (and permanent support), and the excavation sequence.

A main feature of the construction technique of the New Austrian Tunnelling Method (NATM) is that the proposed tunnel is sequentially excavated and supported, and the excavation sequences and face areas to be excavated can be varied (ICE, 1996). The key to minimize surface and subsurface

deformations is to maintain the stability of the exposed excavation face, consequently, since the stability of the excavation face is in inverse proportion to its size, subdividing the cross-section into multiple drifts is advantageous. Typical configurations for NATM tunnels subdivide the excavation face into crown, bench and invert excavations, more complex configurations may include a sidewall drift or even twin sidewall drift (see Figure 2.5). This composition depends mainly on the tunnel size and required geometry, surrounding structures, ground characteristics and surface settlement requirements.



Figure 2.5 - Example of a configuration including twin sidewall drifts – Santiago Metro, Santiago, Chile (Dr. Sauer Group, 2010)

The primary lining, applied sequentially alongside the excavation of each drift, is provided by sprayed concrete in combination with a wiremesh, and when necessary, steel arches and ground reinforcements, such as rock bolts. Ongoing excavation induces stresses redistribution, in which the stresses decrease due to the removal of the soil in the zone of active excavation, and increase ahead and behind the tunnel face. As introduced in the previous section, the final equilibrium occurs within a distance of about two times the tunnel diameter, hence at this point the structural tunnel lining must be effective, which is achieved by closing the tunnel lining in the invert within this distance behind the face. It is then clear that prompt ring closure is essential to minimize ground movements.

The interpretation of the geotechnical monitoring allow changes to be applied in the excavation and support system during the excavation process, to improve the overall tunnelling performance. Lastly, the permanent support is usually provided by a cast in-situ concrete lining, which is normally considered separately for design purposes.

The application of NATM in soft soils is relatively recent compared with its application in hard rock. Since soft soils have a very short stand-up time, some additional measures must be undertaken to ensure the stability of the excavation. The application of these measures is even more important for tunnels constructed in urban areas, where settlements must be limited in order to avoid damaging overlying structures. The principal measures as presented in ICE (1996) are listed below:

- Excavation stages must be sufficiently short, both in terms of dimensions and duration;
- Completion of primary support, in particular the closure of the sprayed concrete “ring” must not be delayed.

Additionally, artificial measures, such as freezing or grout injection, may be used to improve the capacity of self-supporting of the soil, when the stand-up time is almost inexistent. This is when the limit of this construction technique is reached (Chapman *et al.*, 2010).

Overall, this sequential excavation method is relatively slow, nevertheless it is considered to be very useful in areas where existing structures found in the vicinity, such as a sewer, that could not be relocated. NATM offers a flexible construction technique that can cost-effectively adapt to irregular geometries and unforeseen conditions.

2.3.4.2. Sprayed Concrete Lining Method (SCL)

In 1994 two NATM tunnels collapsed while under construction, the first was a tunnel beneath a street in Munich, and the second in the centre of London's Heathrow airport. Following the collapse, 'NATM' became disreputable (Chapman *et al.*, 2010). At that time, the Institute of Civil Engineers (ICE) appointed a Commission to respond to the wide-spread concern, and in 1996 published *Sprayed concrete linings (NATM) for tunnels in soft ground* (ICE, 1996), which led to the use of the term sprayed concrete lining (SCL) instead.

Nowadays, the term 'SCL' is, together with 'NATM', used widely to reproduce any tunnelling method using sprayed concrete as a regular support. The publication that gave birth to the SCL term – ICE (1996) – presents the method as using many construction techniques related to NATM, although it is not related to a specific construction method neither to specific principles connected with NATM.

The SCL construction method, as it is used in soft ground in urban areas throughout the UK, uses an incremental excavation sequence, where a sprayed concrete layer is applied to form a primary lining with, or without, wiremesh, steel fibres, lattice girders, dowels, anchors or bolts. This is then followed by the installation of a permanent lining later on. The primary support particularities are determined in advance of the construction by the designer and are validated during construction by instrumentation and monitoring.

A number of methods have been used for subdividing the face. This subdivision is applied in order to provide a better control of face stability, convergence and settlement, by ensuring earlier support. In stiff competent strata, such as London Clay, full face excavation is possible up to 30 m² in cross section (ICE, 1996), however when advancing full face, a stepped profile of heading and invert is commonly adopted. For face areas greater than 30 m², ICE (1996) suggests four basic excavation sequences, which are represented in Figure 2.6 to Figure 2.9. It should be noted that these suggestions of sequential advances division can also be used with the traditional NATM approach.

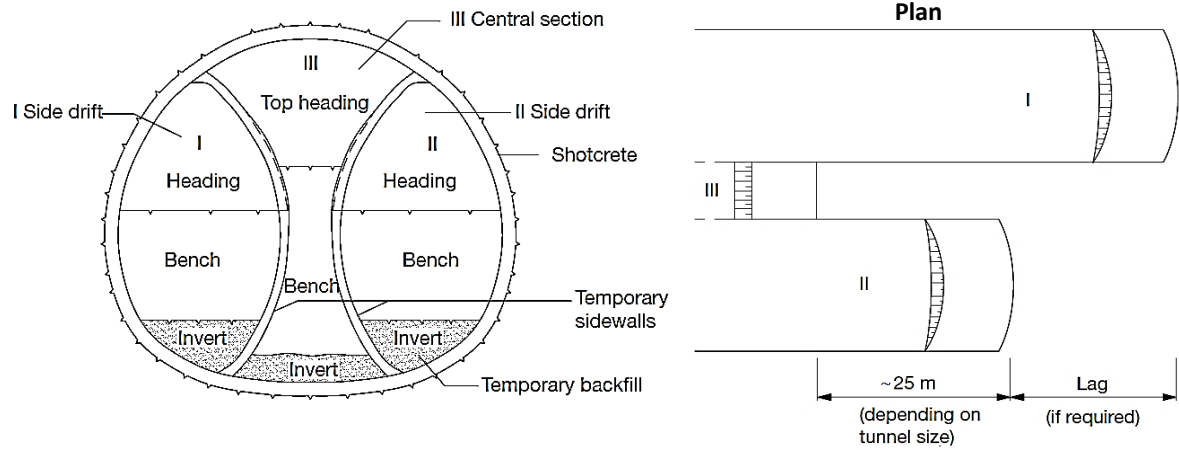


Figure 2.6 - 'Type 1' excavation sequence - twin side gallery and central core (ICE, 1996)

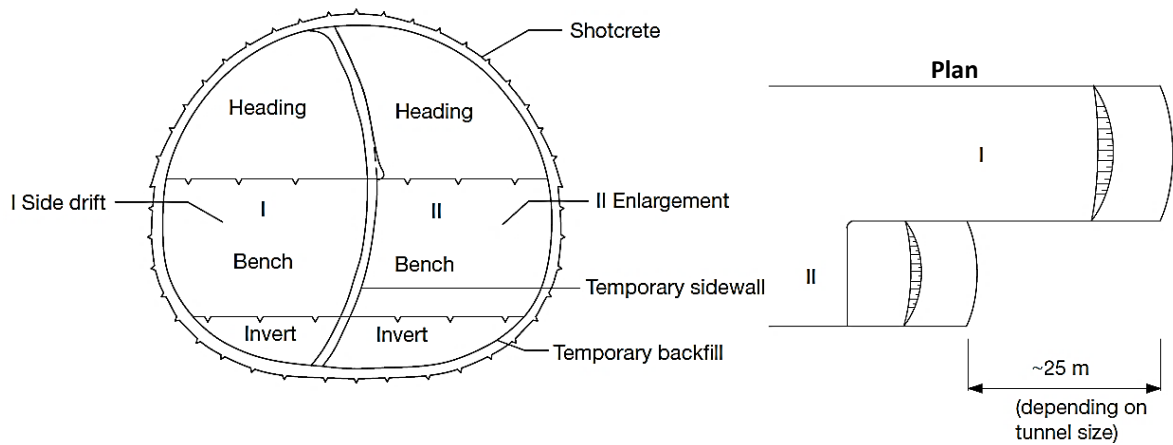


Figure 2.7 - 'Type 2' excavation sequence - side gallery and enlargement (ICE, 1996)

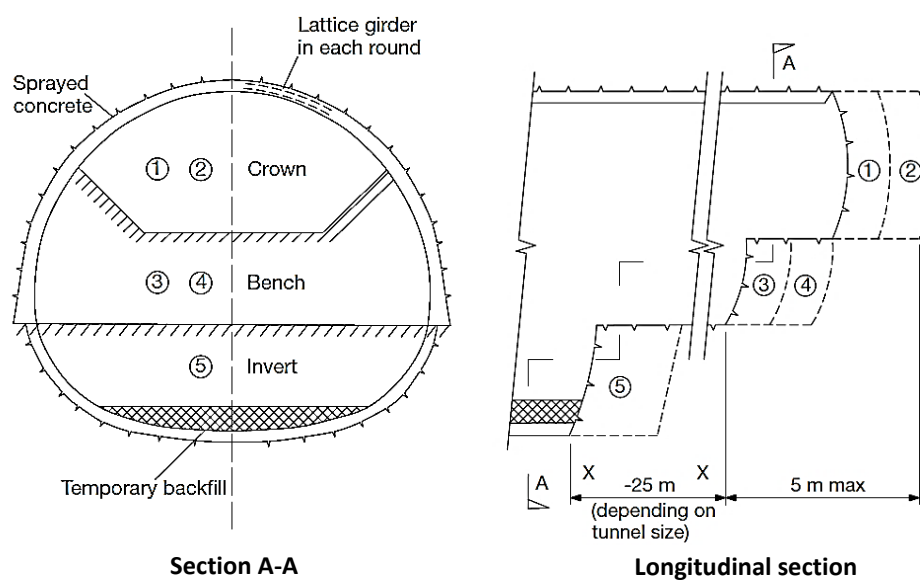


Figure 2.8 - 'Type 3' excavation sequence - top heading/crown, bench and invert (ICE, 1996)

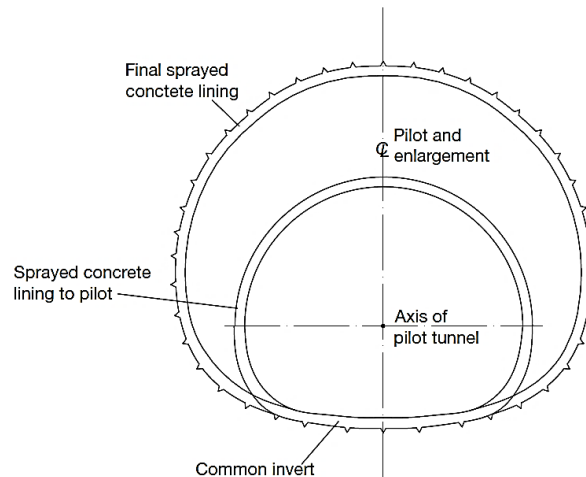


Figure 2.9 - 'Type 4' excavation sequence - pilot tunnel and enlargement (ICE, 1996)

2.4. GROUND MOVEMENTS IN GREENFIELD CONDITIONS

2.4.1. INTRODUCTION

The present section summarizes analytical methods to estimate tunnel induced ground movements. The methods hereafter presented consist on semi-empirical approaches under greenfield conditions, which means that their formulation do not take into account the presence of any other structure in the vicinity. Both surface and subsurface movement will be characterized within this section.

In order to coherently reproduce the ground movement formulations, a coordinate system will be adopted in this section as represented in Figure 2.10. However, when presenting the analysis undertaken within this thesis, a different coordinate system may apply, which will be at that point clarified.

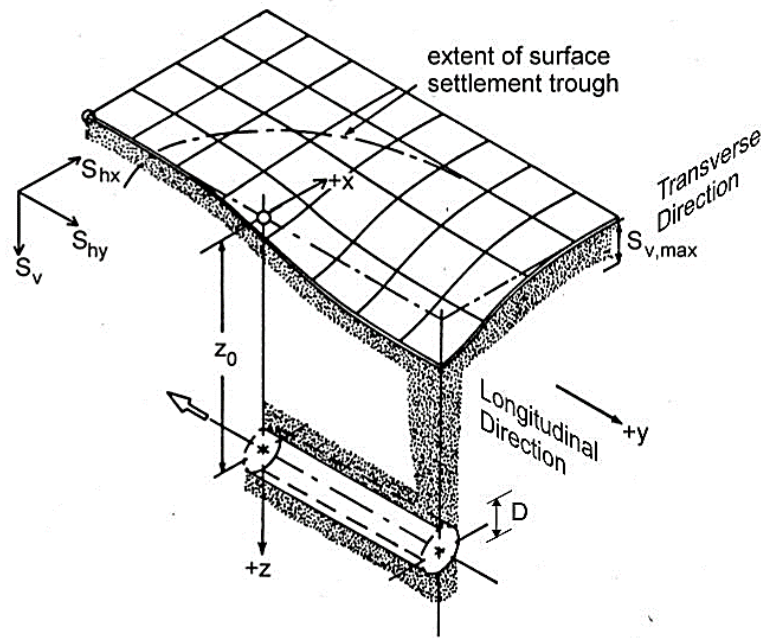


Figure 2.10 - Geometry of the tunnel induced settlement trough (after Attewell *et al.*, 1986)

Figure 2.10 reproduces a complete 3D form of a tunnelling induced settlement trough, where x corresponds to the distance from the tunnel centre-line in the transverse direction, y corresponds to the distance in the longitudinal direction relatively to the excavation face, and z is the depth below the ground surface. The displacements that compose the represented settlement trough will be analysed individually, with S_v referring to vertical displacement, whereas S_{hx} and S_{hy} describe horizontal displacements in the transverse and in the longitudinal direction, respectively.

2.4.2. SURFACE MOVEMENTS

2.4.2.1. Transverse displacements

Greenfield calculations are based in the assumption that the settlement trough perpendicular to the axis of the tunnel being assessed, can be well represented by a Gaussian curve. This mathematical approach has been widely accepted after Peck (1969).

Following this approach, the settlement in the transverse direction can be reproduced by the given equation:

$$S_v(x) = S_{v,max} \cdot e^{-\frac{x^2}{2 \cdot i_x^2}} \quad (2.1)$$

where $S_{v,max}$ represent the maximum settlement registered above the tunnel axis, and i_x the trough width parameter which corresponds to the standard deviation in the original Gaussian function. Figure 2.11 represents a typical settlement trough, in which it can be identified that the maximum slope occurs

at the point of inflection located at the distance i_x from the tunnel centre-line (CL). This point distinguishes the sagging zone and the hogging zone as indicated in Figure 2.11.

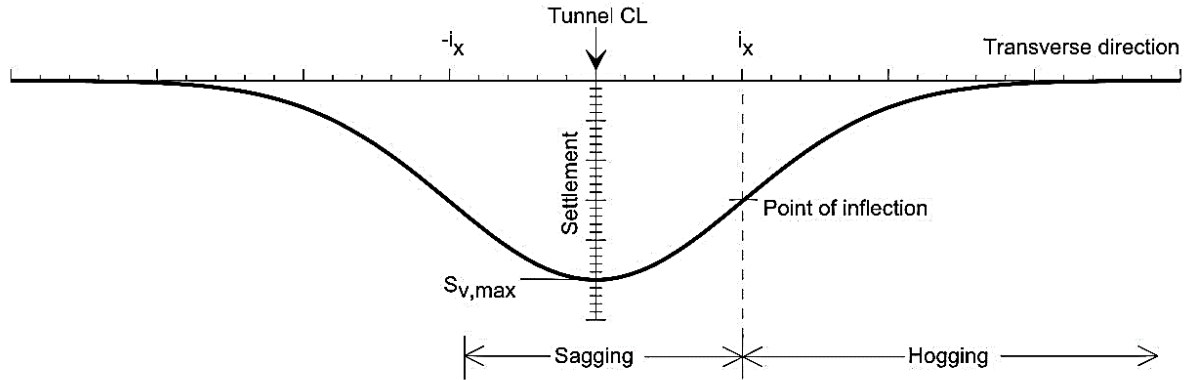


Figure 2.11 - Transverse settlement trough (reproduced from Franzius, 2003)

The volume of the settlement trough per unit length, V_s , can be then expressed by integrating Equation 2.1, which leads to:

$$V_s = \int_{-\infty}^{+\infty} S_v dx = \sqrt{2\pi} \cdot i_x \cdot S_{v,max} \quad (2.2)$$

For materials in which the response of the ground to tunnelling is considered to be undrained, e.g. clays, the volume of the settlement trough is assumed equal to the volume of the soil that moves into the tunnel which is therefore excavated in excess of the theoretical volume of the tunnel. Figure 2.12 represents this relation for a circular tunnel where D corresponds to the outer diameter of the tunnel section.

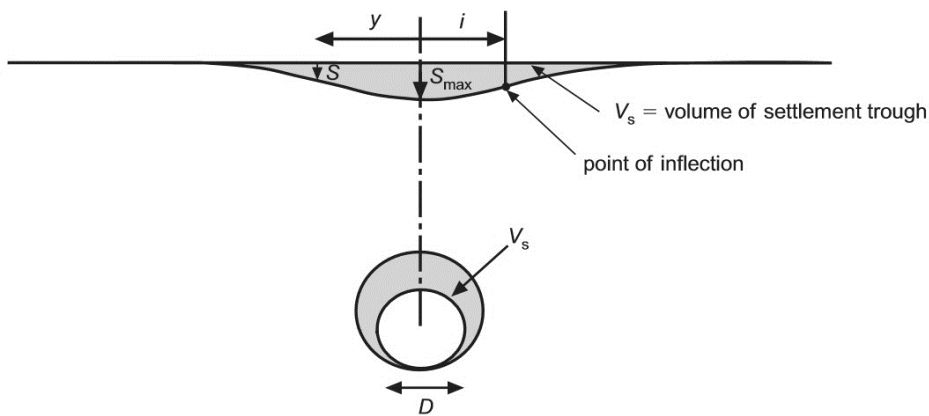


Figure 2.12 - Representation of the volume of settlement trough

The excess volume excavated is commonly known as volume loss, V_L , (usually expressed as a percentage) and can be represented as a proportion of the theoretical tunnel volume (per unit length):

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

The combination of Equations 2.2 and 2.3 allow the transverse settlement profile to be expressed in terms of the volume loss, and, for a circular tunnel may be expressed as:

$$S_V(x) = \sqrt{\frac{\pi}{2}} \cdot \frac{V_L \cdot D^2}{4 \cdot i_x} \cdot e^{-\frac{x^2}{2 \cdot i_x^2}} \quad (2.4)$$

The volume loss can be seen as a measure of the total ground disturbance, and depends on a number of factors such as the type of ground, ground water conditions, tunnelling method, time to provide adequate support and quality of workmanship and management. It comprises face loss and radial convergence (ICE, 1996).

Typical volume losses for open-face tunnelling in soft ground are generally between 1 and 3% (Mair, 1996), whereas for closed-face tunnelling, with significant face support, results in lower values. In order to control surface settlement and consequently the volume loss, ICE (1996) gives some recommendations for tunnels using a Sprayed Concrete Lining. First, the excavation stages must be sufficiently short both in terms of dimension and duration; and second, the closure of the sprayed concrete ring must not be delayed. Recent projects in London Clay using sprayed concrete linings revealed that the typical volume loss between 0.5% and 1.5%.

It is worth referring that regarding shield tunnelling, Attewell (1978) identified different sources for the volume loss, which can be divided in four categories: face loss, which is a result of the radial soil movement towards the unsupported tunnel face (causes the development of surface settlements in front of the tunnel face); shield loss, which corresponds to the radial ground loss around the tunnel shield; ground loss during and subsequent to lining application, which is mainly due to the delay between the performance of the excavation and the lining application; and ground loss after grouting, as the overburden pressure is transferred to the new boundary.

The distinction presented by Attewell allows to see the complexity of this parameter and therefore its importance on the settlement prediction. For this reason, in order to obtain reliable estimates of the surface settlement, the volume loss value is commonly adopted based on values obtained by real monitoring data from similar tunnelling conditions.

Regarding the horizontal displacements of the surface in the transverse direction, O'Reilly & New (1982) showed that it can be derived from the equations above presented, while assuming that the vectors of the resultant displacement point towards the centre of the tunnel. Thus the following equation is assumed to express the horizontal movement in transverse direction:

$$S_{hx}(x) = -\frac{x \cdot S_V(x)}{z_0} \quad (2.5)$$

where z_0 represent the depth of the tunnel axis below the soil surface.

Figure 2.13 represent the distribution of the surface horizontal displacements together with the Gaussian settlement trough, as indicated in the legend.

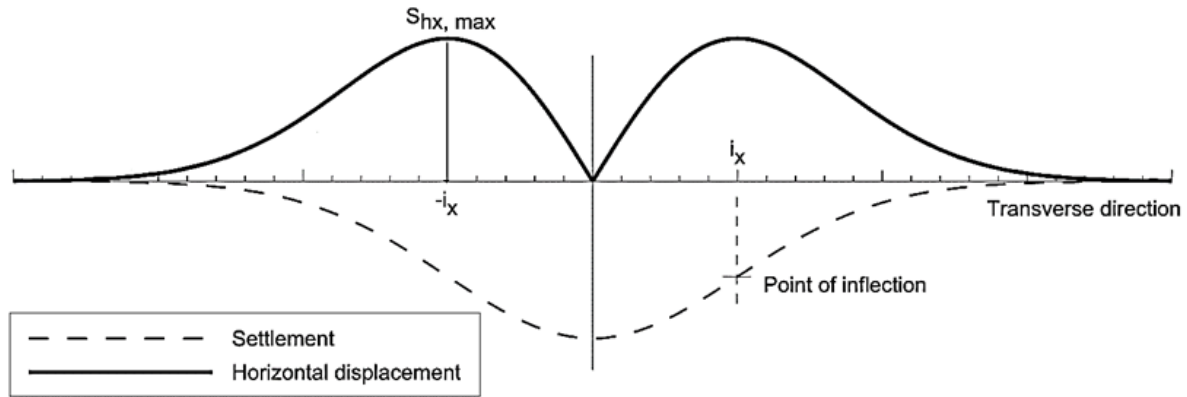


Figure 2.13 - Horizontal surface displacement distribution and strain in the transverse direction (reproduced from Franzius, 2003)

Figure 2.13 allows one to see that the maximum horizontal displacement occurs at the inflection point of the Gaussian curve, whereas in the alignment of the tunnel centre-line it is zero.

2.3.2.2. Longitudinal displacements

From Attewell & Woodman (1982) the longitudinal settlement profile can be derived by assuming a tunnel as a number of point sources in the longitudinal direction and therefore superimposing the settlement craters caused by each point source. If a Gaussian settlement profile is adjusted to the settlement crater, the longitudinal profile can be described by:

$$S_V(y)_{x=0} = S_{V,max} \cdot \Phi\left(\frac{y}{i_y}\right) \quad (2.6)$$

where $\Phi(y)$ is a cumulative probability curve and y is the longitudinal coordinate as represented in Figure 2.10. The cumulative probability curve is defined by the function:

$$\Phi(y) = \frac{1}{i_y \sqrt{2\pi}} \int_{-\infty}^y e^{-\frac{y^2}{2i_y^2}} dy \quad (2.7)$$

Values of $\Phi(y)$ are listed in standard probability tables such as given by Attewell & Woodman (1982).

A typical longitudinal settlement profile can be seen in Figure 2.14.

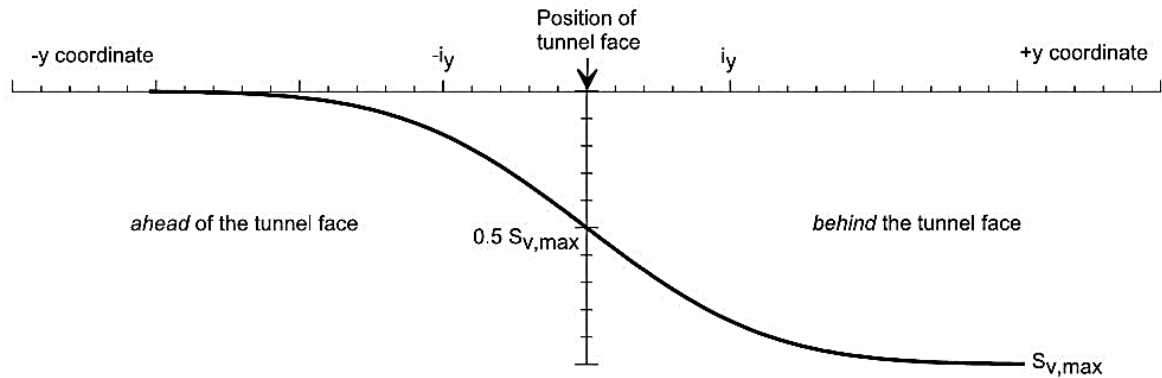


Figure 2.14 - Longitudinal settlement profile (reproduced from Franzius, 2003)

As the previous figure suggests, longitudinal settlement increases in the positive y -direction, which corresponds to the direction of tunnel progression. The maximum value, $S_{v,max}$, is reached at $y = +\infty$, whereas $S_v = 0$ develops at $y = -\infty$. For $y = 0$ the settlement is taken as $S_{v,max}/2$. Actually, Attewell & Woodman (1982) showed that, for stiff clay, face 30-50% of the $S_{v,max}$ may occur above the tunnel, however for convenience a 50% value is assumed.

The width of the longitudinal settlement profile is defined as i_y . Despite the slight discrepancy between the magnitudes of i_x and i_y , which was identified by Attewell *et al.* (1986) where the transverse settlement troughs revealed to be slightly longer than the longitudinal ones, it is common to consider $i_x = i_y$, thus the trough width parameter i will now be used instead of i_x or i_y .

Regarding the horizontal displacements at the ground surface longitudinally to the tunnel axis, Attewell & Woodman (1982) showed that it can be described as follows:

$$S_{hy}(y)_{x=0} = \frac{V_L \cdot D^2}{8 \cdot z_0} \cdot e^{-\frac{y^2}{2 \cdot i^2}} \quad (2.8)$$

2.4.3. SUBSURFACE MOVEMENT

In their work, Mair & Taylor (1993) applied the plasticity solution for the unloading of a cylindrical cavity in order to describe the vertical and horizontal subsurface displacements. The solution revealed that there is a linear relation between S_v/R (or S_{hx}/R) and R/d , where R corresponds to the tunnel radius and d to the vertical (or horizontal) distance from the tunnel centre to the surface. The predicted mathematical correlation was corroborated with field data.

From Mair & Taylor (1993) only vertical soil movement above the tunnel centre-line or horizontal movement at tunnel axis level could be determined. Regarding the shape of transverse settlement troughs, Mair *et al.* (1993) have shown that a Gaussian curve is a fair approach.

In order to predict subsurface displacements, an accurate relation for the trough width parameter, i , must be assumed. From their case studies O'Reilly & New (1982), concluded that the trough width parameter could be related with the corresponding tunnel depth, z_0 , through the simplified equation:

$$i = 0.5 \cdot z_0 \quad (2.9)$$

Equation 2.9, the relation for the trough width parameter to determine the subsurface displacement would be obtained substituting the tunnel depth, z_0 , for $(z_0 - z)$, which corresponds to the vertical distance between the settlement profile and tunnel axis:

$$i = 0.5 \cdot (z_0 - z) \quad (2.10)$$

where z represents the depth at which the subsurface displacements are to be determined.

Figure 2.15 represents field and centrifuge data presented by Mair *et al.* (1993), which showed that subsurface settlement troughs are proportionally with depth, z . The mentioned figure shows the measurements, in which the trough width parameter is normalized by z_0 and plotted against normalized depth, z/z_0 .

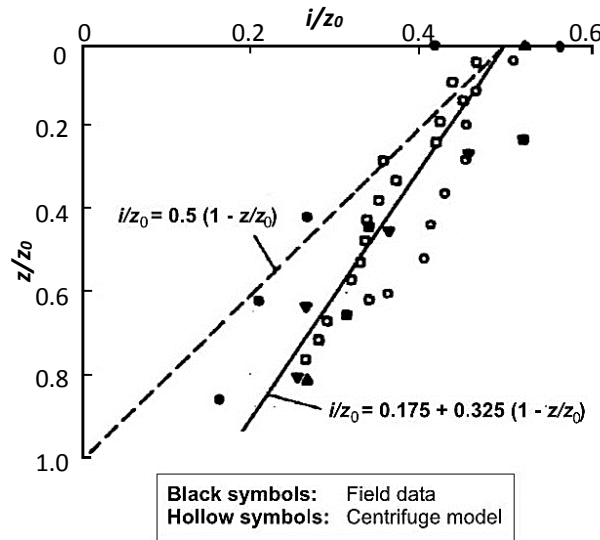


Figure 2.15 - Variation of trough width parameter i of subsurface settlement troughs with depth (after Mair *et al.*, 1993)

It is possible to see that the proposed in Equation 2.10 (dashed line), under predicts i with depth. Hence in order to accurately adjust the measurements the following equation was proposed instead.

$$\frac{i}{z_0} = 0.175 + 0.325 \cdot \left(1 - \frac{z}{z_0}\right) \quad (2.11)$$

assuming that trough width parameter can be expressed by:

$$i = K \cdot (z_0 - z) \quad (2.12)$$

which substituting into Equation 2.11, leads to the variable K expressed as:

$$K = 0.325 + \frac{0.175}{1 - \frac{z}{z_0}} \quad (2.13)$$

Figure 2.16 represents Equation 2.13 as well as the field data already presented in Figure 2.15. It can be seen from the represented plot, which represents K against z/z_0 , that Equation 2.13 fits perfectly well the measurements made by Mair *et al.* (1993), whereas a constant value of $K = 0.5$, as per Equation 2.10 would underestimate the width of subsurface settlement troughs.

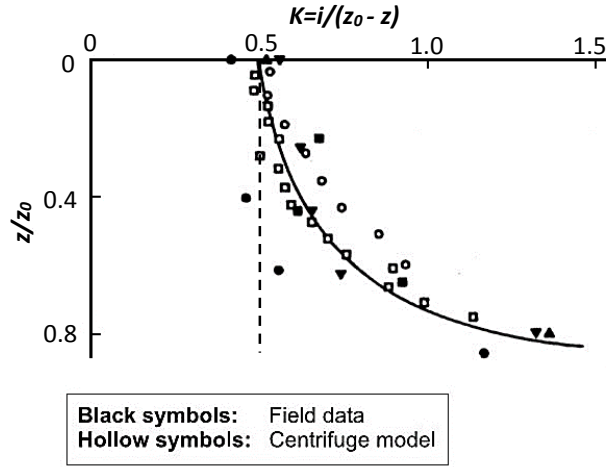


Figure 2.16 - Variation of K of subsurface settlement troughs with depth (after Mair *et al.*, 1993)

The combination of Equations 2.2, 2.3 and 2.11 leads to an expression (Equation 2.14) that allows the maximum settlement, $S_{V,max}$, to be determined for a specific depth, z , as it is presented below.

$$\frac{S_{V,max}}{R} = \frac{1.25V_L}{0.175 + 0.325 \cdot \left(1 - \frac{z}{z_0}\right)} \frac{R}{z_0} \quad (2.14)$$

where R corresponds to the radius of the tunnel.

The following figure (Figure 2.17) shows a plot where the maximum settlement normalized by tunnel radius is plotted against $R/(z_0 - z)$. In the mentioned plot, various approaches are resumed, which can be compared with field data from a tunnel construction in London Clay. The represented curve A is derived from Equation 2.12, whilst curves B and C correspond to Equation 2.14 although for different tunnel depths. The straight solid line corresponds to the plasticity solution presented by Mair & Taylor (1993) as introduced in the beginning of the present subsection.

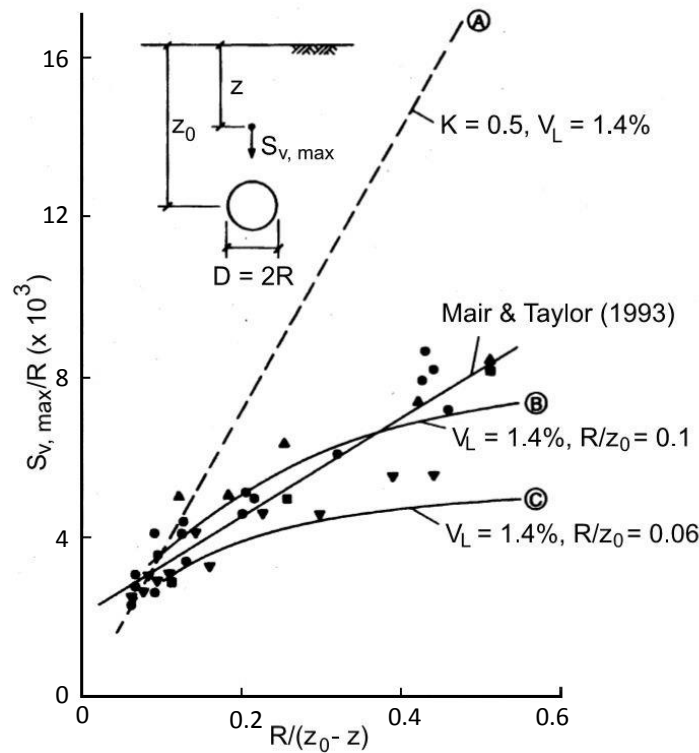


Figure 2.17 - Subsurface settlement above tunnel centre line (after Mair *et al.*, 1993)

Figure 2.17 allows to see that both Mair & Taylor (1993) approach and Equation 2.14 (curves B and C) reasonably agree with the measurements, whilst Equation 2.12 over predicts $S_{v,max}$ of subsurface settlement troughs.

In the previous section, specifically in 2.3.2.1, the horizontal soil displacement was derived from the vertical surface settlements, considering that the resultant displacement vectors point to the centre of the tunnel. However, regarding subsurface displacements, Taylor (1995) stated that in order to achieve constant volume conditions, the displacement vectors should point to the location where the line described by Equation 2.11 intersects with the tunnel centre-line instead. Considering that in the tunnel centre-line $i = 0$, the point corresponds to $0.175 \cdot z_0 / 0.325$ below the tunnel axis level.

However, New & Bowers (1994) highlighted that the assumption that soil particle move towards a single point at the tunnel level is not accurate in the vicinity of the tunnel, which was corroborated by the field measurements of the Heathrow Express. The measurements revealed that the predictions of subsurface troughs based on the assumption that the soil moves towards a single point, were too narrow in the vicinity of the tunnel and consequently the settlements were over-predicted.

Besides, Grant & Taylor (2000) from the results of the centrifuge tests they performed, showed that, close to the surface the settlement troughs were wider than predicted by Equation 2.11, which means that close to surface, horizontal soil displacements away from the tunnel centre-line are underestimated when applying Equation 2.11. Hence, a new and more accurate distribution was proposed by Grant & Taylor (2000) to reproduce the distribution of i with depth, z . This distribution is schematically shown in Figure 2.18 a).

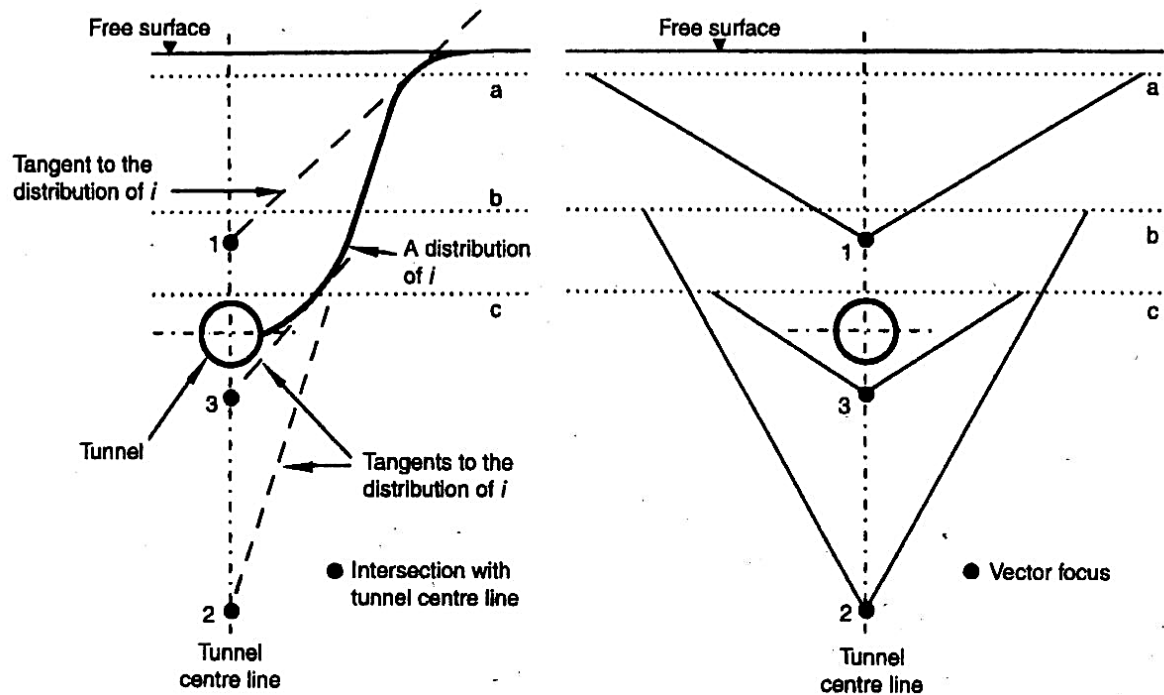


Figure 2.18 - a) Distribution of i for subsurface settlement troughs with depth; b) Focus of vectors of soil movement (after Grant & Taylor, 2000)

Following the stated by Taylor (1995), in which soil displacement vectors are believed to point to the intersection of the distribution of i with z , Grant & Taylor (2000) showed that in fact, the vectors point in the direction of the tangent of the i distribution represented in Figure 2.18 a). Figure 2.18 b) represent the intersection of tangents for different depths with the tunnel centre-line.

Although the tests conducted by Grant & Taylor (2000) revealed that the values of i were greater near the surface than expected in different approaches, they concluded that this difference was related with the free ground surface boundary they assumed in their tests. Actually, the boundary condition they assumed is not representative of urban environments, since the buildings foundations or even the pavements may provide sufficient restrain to decrease the trough width parameter near the ground surface.

3

LONDON GEOLOGY AND ITS GEOTECHNICAL PROPERTIES

3.1. GEOLOGICAL BACKGROUND

The understanding of the geological framework of London has been refined throughout the years to answer scientific, industrial and commercial needs. Its complex nature defines implications in current and future developments which can be surpassed with the numerous investigations that have been made around the London Basin, as identified on Figure 3.1.

Royse *et al.* (2012) realised that the term “London Basin” was first used by William Smith (1815) and George Greenough (1820) to describe the sediments that compose the geology of London. The London Basin’s boundaries are defined by the limit of the Chalk outcrop (Sumbler, 1996), which is defined by the British Geological Survey as a simple unfaulted downward on undifferentiated basement (Sherlock, 1947).

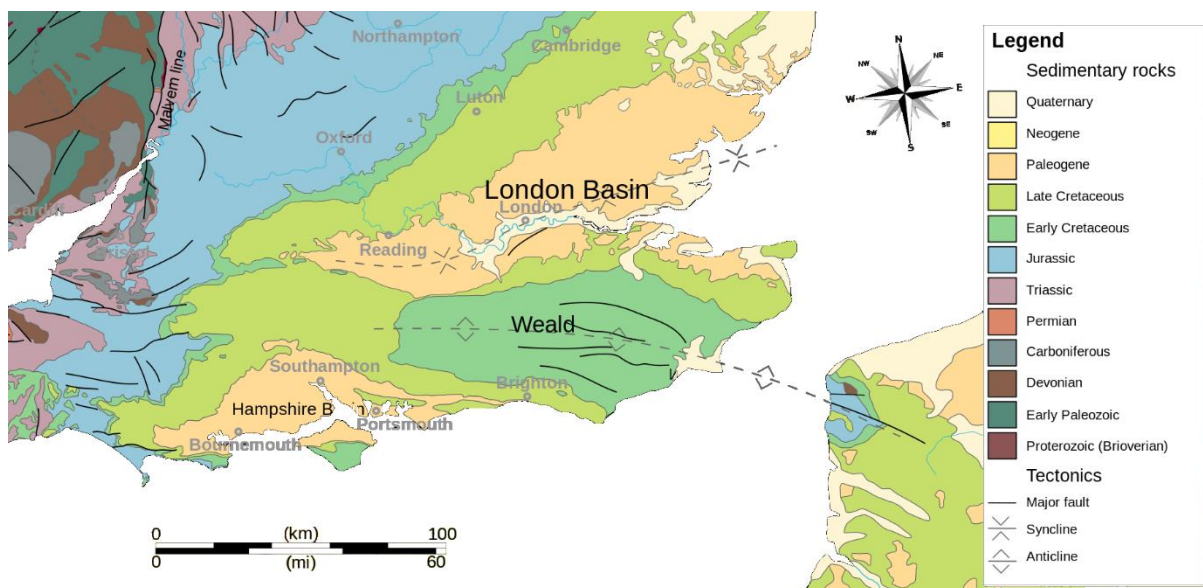


Figure 3.1 - Geological map of southeast England displaying London Basin (Woudloper, 2009)

The mudstones of the Gault Formation are the first sedimentary unit to cover the whole area of the London Platform, which extend to the Worcestershire Basin in the west, to the southern North Sea graben in the east, to the East Midlands Shelf in the north and to the Weald Basin in the south (Sumbler,

1996). The deposit of the mudstones followed a period of intense tectonic activity related to the opening of the northern Atlantic Ocean (Sumbler, 1996).

The mudstones are transgressive across a terrain of eroded Jurassic, Wealden and Lower Greensand sediments, which rest directly on Palaeozoic rocks at the core of the Platform. The Upper Gault is replaced progressively from the south west by arenaceous Upper Greensand facies, which underlies the Chalk over much of the southern area of the Platform from the Thames axis southward with an eastern termination in west Kent (Owen, 1996).

The Chalk is divided at its Formation level as Lower, Middle and Upper, however with the development of many researches the correlation between wire-line logs and boreholes has identified new Formation divisions. Besides, these recent stratigraphical studies allowed to identify the age of the Chalk immediately beneath the sub-Paleogene surface and to locate faults and fault zones previously unidentified.

As engineering projects going deeper beneath London, the more often they are constructed in the Chalk, particularly tunnels. The Chalk was or will be encountered in projects such as the Channel Tunnel Rail Link, Crossrail and Thames Tideway Tunnel. Thereafter it is necessary to have a good understanding of the Chalk elements that could affect the construction in terms of ground conditions, including the size and location of flint bands or other lithological features which might affect the ground stiffness, permeability and groundwater flow level.

Following the tectonic activity recorded in Chalk times, there was a period of erosion and quiescence that placed London on the edge of a sedimentary basin by the start of the Paleogene Period, which succeeding the Mesozoic Era, comprises the epochs Paleocene, Eocene and Oligocene. In this position, Paleogene deposits were laid down during a period of transgressions and regressions result of global sea-level changes. The oldest Formation is that of the Thanet Sand, with outcrops occurring in southeast London.

The Thanet Sand Formation lies uncomfortably under the eroded Upper Chalk surface. Its shallow marine origin led to a largely unfossiliferous layer, which consists mainly of fine grained sand. This Formation initially appeared to be fairly uniform in character, however recent particle size distribution tests indicate that the Thanet Sand shows a coarsening up sequence.

The succeeding Lambeth Group is a complex sedimentary assemblage that was deposited in shallow marine and fluvial to terrestrial environments, this led to a highly variable lithology that is divided in three Formations. The shallow marine Upnor Formation deposited over a large area of the southeast of England is succeeded by the Reading Formation which consists of terrestrial, pedogenically altered and fluvial sediments (mottled clays, silts and sands). In the London area the last Formation is intercepted by the Woolwich Formation consisting of the Shelly Clays and the Laminated Beds (Ellison *et al.*, 2004). The importance of the Lambeth Group relies on its rapidly changing and colourful sequence of sediments which allows to identify fault movements.

The Thames Group records a return to marine conditions induced by global sea-level rise which was preceded by an episode of uplift and erosion. The deposition of the first sediments created the Blackheath Formation and the Harwich Formation, the last formed by thin glauconitic sands and sandy glauconitic clays. Although often thin, these units have great significance for engineering projects due to their high permeable lithology.

Overlying the Harwich Formations is the most well-known of all the Formations in the London Basin, the London Clay Formation, which is a clay with silty and sandy clay intervals, with a maximum thickness of 130m (Ellison *et al.*, 2004). Its presence beneath much of Central London, its relatively

homogeneous nature as well as its competent geotechnical properties makes it an ideal medium for tunnelling, promoting the pioneer development of the London Underground in second half of the 19th century.

Besides its homogeneous nature, King (1981) subdivided the London Clay Formation in five divisions (A to E) by using a combination of biostratigraphy and lithological variation as well as identifying marine flooding events. The London Clay divisions and geotechnical properties will be analysed more accurately in the following subchapter.

The following deposits, Quaternary deposits, provide evidence of an ancient river system, a precursor to the river Thames, and the development of the present River Thames valley. However, the complex and discontinuous nature of its sediments alongside with the incompleteness of the stratigraphical record turn the history's understanding into a difficult analysis.

At that point, Thames was a broadly west to east aligned stream axial to the Basin. In its extreme east a large estuary occurs where the river enters the North Sea. The earliest Thames deposits, The Pebble Gravel Formation, consists of a series of gravels composed essentially of local materials, mainly flint. It is interesting to refer that the Thames system is the largest drainage basin in Great Britain, which explains the large area of river deposits that arose from Thames and its tributaries.

A Glaciation period hampered the drainage system of east and central England, driving the river to adopt a new course through London (Gibbard 1994 and Bridgland 1988). This evolution of the Thames system is defined by a sequence of gravel and sand aggradations under periglacial climate.

The current subdivisions of the River Deposits are based on morphological evidence, however it is being changed through a one based on geological sequences, which is now in a phase of consolidation.

3.2. LONDON CLAY

3.2.1. GEOLOGICAL HISTORY

London Clay is categorized as an over-consolidated, stiff clay of high plasticity, deposited in a low energy marine environment in the Eocene epoch some 50 million years ago and today it is encountered onshore in the London Basin and the Hampshire Basin. Its complex geological history can be described in three major stages, the deposition, erosion of the upper part and redeposition of the Quaternary sediments.

There are many evidences that allow to identify the specifications of the depositional environment of this Formation. While the appearance of phosphatic and claystone nodules, glauconite grains and intense bioturbation indicates that the deposition of sediments was relatively slow, the presence of sand partings and layers represents periodic storms. Furthermore it is possible to find disseminated mica plates, small shell segments commonly less than 5 mm in size and pyrite occurring as small aggregates. Although, it can be said that its dominant clay minerals are illite and montmorillonite, with subsidiary kaolinite (Hight *et al.*, 2003).

Regarding the geological history, the most important evidence in the geotechnics contexts is the maximum thickness of the deposits overlying the London Clay in the past or otherwise the amount of erosion occurred. The relevance of this information relies on the maximum overburden pressure and over-consolidation ratio (OCR) which should be correctly specified in all engineering projects developed in London Clay.

The redeposition process, besides having a great influence on the stress history experienced by the clay, it also has an important role preventing weathering (drying or oxidation). The majority of the London Clay remains unweathered where it has been covered by gravel, however the upper 5 to 10 m has been found to be weathered as it has been exposed. It has been reported that the part of the London Clay that had suffered oxidation, has a lower value of apparent cohesion which should be considered in modelling (Hight, 1993).

3.2.2. LITHOLOGICAL CHARACTERISTICS AND DIVISIONS OF LONDON CLAY

The cycles of sea level change affected enormously the sedimentation in Central London due to its proximity to the depositional basin's western margin. There were three major transgression-regression cycles of relative sea level throughout the formation of the London Basin, the last being identified in the base of the Thames Group. In addition to the major cycles, five sub-cycles of relative sea level change are recognized within the London Clay deposition. The changes in sedimentation as a result of the sub-cycles provide the basis for the division of the London Clay into lithological units. The corresponding five lithological divisions are identified from A to E in ascending order (King, 1981), as shown in Figure 3.2.

Each lithological unit is characterized by a thin basal unit of glauconitic and pebbly sands underlain by an upwardly-coarsening sequence which identifies each sub-cycle (King, 1981). By using the content of fossil fauna and the contents of sand and silt, these divisions have been further subdivided into different units.

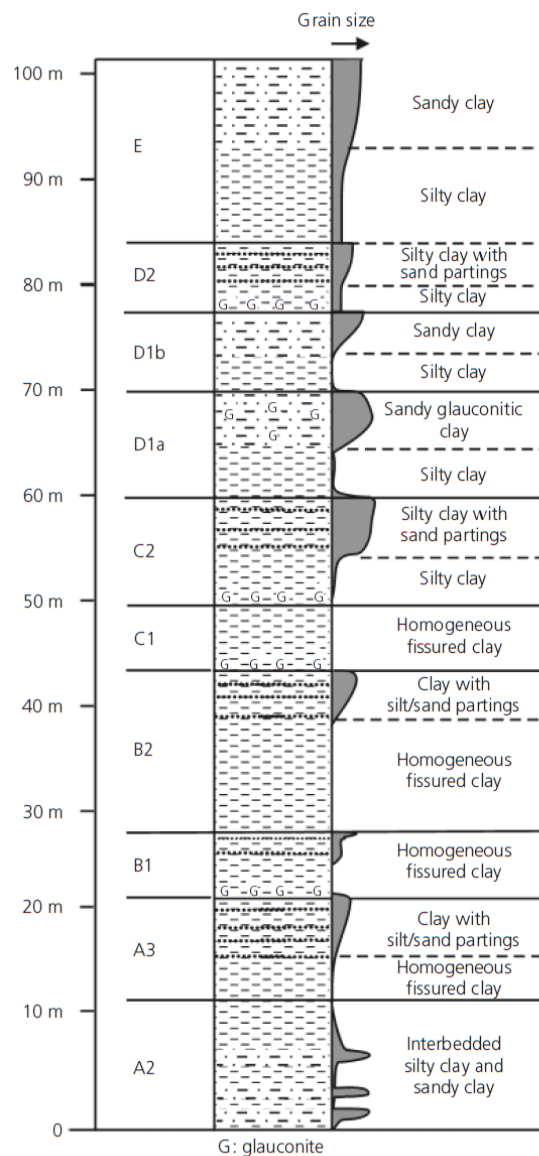


Figure 3.2 - Lithological units in London Clay proposed by King, 1981 (adapted by Wright, 2010)

As the above mentioned figure suggests, the Unit A1 of the London Clay is not commonly represented once one assumes that it is diffused on the underlying layer.

The unit A2, which is approximately 12 m thick, includes a high percentage of silt and some wood fragments and pyrite nodules. This unit contains no claystones, but nonetheless it is known that there are several alternations of sandy and silty clays within this unit, indicating small sea level changes considering that a greater change would lead to a coarsening of the material to be deposited. Frequently, partings and lenses of silt and fine sand are found.

The thickness of the following unit, A3, is similar to the previous one. It comprises a homogeneous although slightly calcareous silty clay layer in the bottom and a claystone layer also near the base. Towards the top there are other thin claystone layers and more commonly silt and sand partings are found.

The next unit, B1, is a thin sandy clay, which together with the unit B2, the last comprising silty clays with numerous claystone and weak silt and sand partings, compose a unit with a combined thickness of 25 m. As shown in Figure 3.2 the boundary between the units A and B represents a glauconitic layer.

The remaining units, C1, C2, C3, D1, D2 and E, has been fully preserved in Hampstead and South Essex, where the original thickness of the London Clay, which is up to 150 m, remains unharmed (King, 1981). In Central London, generally only the lower part of the sequence is conserved, hence only units B and A below can be found.

Despite the above description, the identification of each lithological unit at a specific depth is not straightforward, a mighty geological knowledge and especially experience, are required to distinguish each layer accurately. Moreover, according to what was mention before, some units are likely to be missing due to minor erosion periods and further depositions.

3.2.3. GEOLOGICAL STRUCTURES

The London Basin has rare evidences of folds or faults (Sherlock, 1962), although, resulting from internal distortions of the clay mass it is possible to find occasional extensive shear zones in London Clay, which may be responsible for geotechnical failure in engineering works.

Joints and fissures are the two most common smaller-scale discontinuities encountered in the London Clay. The mean size of the fissures is typically less than 15 cm and it decreases towards the ground surface. Oppositely, their number per unit increases and their spacing becomes smaller, suggesting that stress release and weathering played an important role in fissuring genesis (Ward *et al.*, 1965 and Skempton *et al.*, 1969).

Considering the changing composition amongst different lithological units it is expected the fissuring to vary, however it presents relative constancy in what concerns to their dips, concentrating around 0 to 20 and 70 to 90 degrees. They exhibit no particularly preferred strike.

The joints share similar features with the fissures, the surface is typically matt and implying no trace of shearing along them. Notwithstanding, the joints have systematic orientation and spacing.

3.2.4. HYDROGEOLOGY

The London Basin hosts a regionally important Chalk aquifer, which becomes confined by the London Clay across the central regions. The deep aquifer comprises essentially the lower granular units of the Lambeth Group and the Chalk, together creating a hydraulically connected, layered aquifer system. It is covered by either the London Clay or the clays of the Lambeth Group.

In London, the Terrace Gravels of the river Thames, overlying the London Clay, form a discrete shallower aquifer which is recharged from surface precipitation and from the river Thames. Due to its shallow depth the groundwater flow in the River Terrace Deposits may be influenced by the presence of tunnels, basements and other smaller localised anthropogenic disturbances, hence this structures should be carefully identified and the hydrological conditions clarified for all analysis carried out in these layers.

The London Clay, alongside with the cohesive units of the Lambeth Group, is considered to be non-aquifer with very limited potential to transmit water considering its low permeability. However, recent investigations has indicated that the Lambeth Group may include sand channels and very sandy clay units which have the capacity to transmit limited quantities of water.

3.3. GEOTECHNICAL CHARACTERIZATION

Since the 1960s, the Critical State theory has been the most common approach used for the interpretation of clay behaviour. It assimilates shear and volumetric behaviour in a unified framework founded on the premise of the existence of a Critical State. Jardine (1985) conducted triaxial tests with reconstituted London Clay samples, which results reproduced in Figure 3.3 seemed to indicate the possible existence of the Critical State.

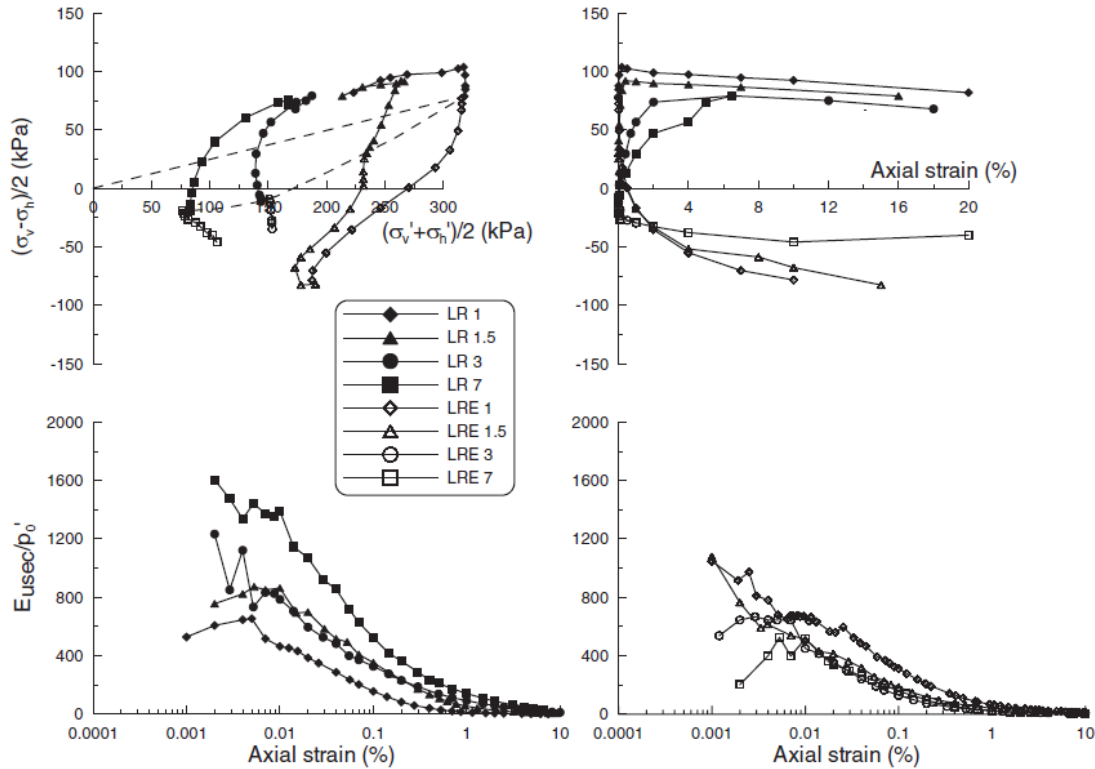


Figure 3.3 - Undrained behaviour of reconstituted London Clay in undrained triaxial compression and extension tests; all samples were K_0 -consolidated to $\sigma_v' = 400 \text{ kPa}$ and unloaded to OCRs 1, 1.5, 3 and 7 (reproduced from Hight *et al.*, 2003, original data by Jardine, 1985)

However, throughout years of laboratory investigation it has been well identified that the behaviour of the natural London Clay is one of the exceptions that do not conform the theory. It normally exhibits brittle behaviour and shearing involves bifurcation at an early stage.

Nevertheless, the concept of the State boundary surface, which is one of the most important concepts in the Critical State theory, allow to interpret the natural London Clay behaviour. The State boundary surface was originally defined in p' - q' - w space (Roscoe & Poorooshasb, 1963) dividing admissible and inadmissible states for a soil. Later, it was redefined by Gens (1982 and 1985) assuming a normalised stress-path (NSP) to represent, as it is known today, the Local Boundary Surfaces, which consider different consolidation regimes (see Figure 3.4).

Burland (1990) showed that the boundary surface of the natural London Clay extended well beyond that of the reconstituted material, indicating that it is due to the effects of the natural soil structure on shear strength.

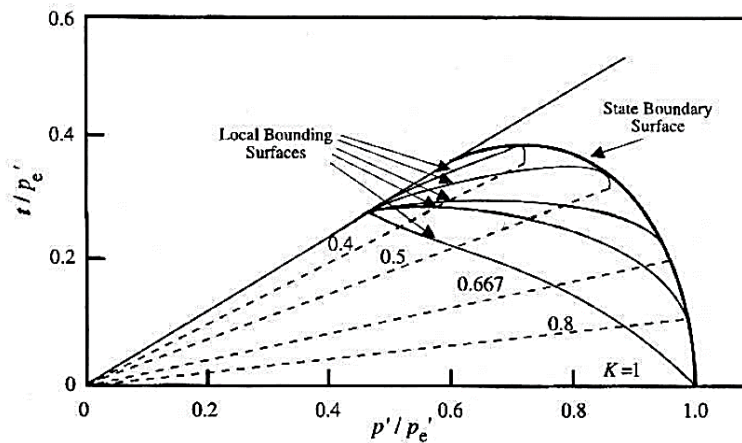


Figure 3.4 - State Boundary surface and Local Boundary surface (reproduced from Zdravkovic and Jardine, 2000, original concept after Gens, 1982)

Although many studies had attempt to generalise the London Clay behaviour, recent investigations focus on interpreting the data in a more direct manner, adopting failure envelopes in either effective stress or total stress terms.

In order to execute a geotechnical analysis considering the influence of a tunnel, the soil parameters typically required are the in-situ stresses, which requires the knowledge of the coefficient of earth pressure, K_0 , and the soil stiffness, commonly presented as the Young's modulus.

Numerous geotechnical investigations should be performed to characterize the ground subsurface conditions, once it is crucial to accurate ground movement predictions. Boreholes are frequently performed to recover soil samples for laboratory testing purposes. In-situ tests are also undertaken in order to obtain information on the stratigraphy, stiffness, strength and permeability of key strata.

The following subchapters will present a summarized characterization of the main geotechnical parameters of London Clay. To support this characterization, results from laboratory and in-situ tests will be presented, as well as empirical expressions. It should be said that, although characterizing London Clay parameters is out of the scope of this thesis, the author found it important to be referred since it allows a better understanding on the London Clay behaviour and the influence of each parameter.

3.3.1. IN-SITU EARTH PRESSURE COEFFICIENT, K_0

The in-situ earth pressure coefficient, K_0 , is defined as the horizontal to vertical effective stress ratio at rest. A tunnel excavation induces ground movements which leads to a new state of stress, once the coefficient K_0 influence the geomechanical behaviour and thus in any geotechnical analysis it is essential to characterize both vertical and horizontal stresses.

The coefficient of earth pressure at rest depends on the stresses that the material has been subjected which is a result of the geological and pore water pressure history. The formula proposed by Jaky (1944) (Equation 3.1), uses the critical angle of shearing resistance, ϕ'_c ($^\circ$), to estimate the normally consolidated earth pressure coefficient K_{nc} .

$$K_{nc} = 1 - \sin \phi'_c \quad (3.1)$$

This formula can be used to estimate the normally consolidated earth pressure coefficient for the original deposition of the London Clay, however, this Formation has suffered several erosion stages and successive sediments deposition hence it cannot reproduce the actual earth pressure coefficient.

Since the London Clay is considered to be an overconsolidated clay, the formula proposed by Mayne & Kulhawy (1982), seems to be more appropriate to estimate the earth pressure coefficient. The proposed model admits that the change in K_0 over the years can be related to the , OCR, according to the following formulas.

$$\frac{K_0}{K_{nc}} = OCR^m \quad (3.2)$$

where m and OCR can be defined as:

$$m = \sin \phi'_c \quad (3.3)$$

$$OCR = \frac{\sigma'_p}{\sigma'_{v0}} \quad (3.4)$$

where σ'_p represents the effective pre-consolidation stress (kPa) and σ'_{v0} the effective vertical stress at rest (kPa).

Alternatively, m can be assumed as $m = 0.5$ (Meyerhof, 1976) or $m \approx 0.1$ as some studies indicate (Lefebvre *et al.*, 1991; Hamouche *et al.*, 1995). Although the last correlations can be applied to estimate the earth pressure coefficient on overconsolidated clays it is still not correct to use them for soils with a more complex history of loading and unloading once it does not take into account processes as creep or cementation.

Over the last 100 years or so, London Clay has felt a reduction in pore water pressure due to extraction from deep wells. This fact has resulted in further increases in the vertical effective stress and consequently in the horizontal effective stress.

Wright (2010) has presented a comparison between different assumptions on the K_0 variation with depth (see Figure 3.5), considering both erosion and later deposition processes, represented as unload and reload curves, respectively, and underdrained conditions. He has also included a specific K_0 profile measured in a site investigation at No. 1 Poultry, which can be well represented by the curve for 50% underdrainage.

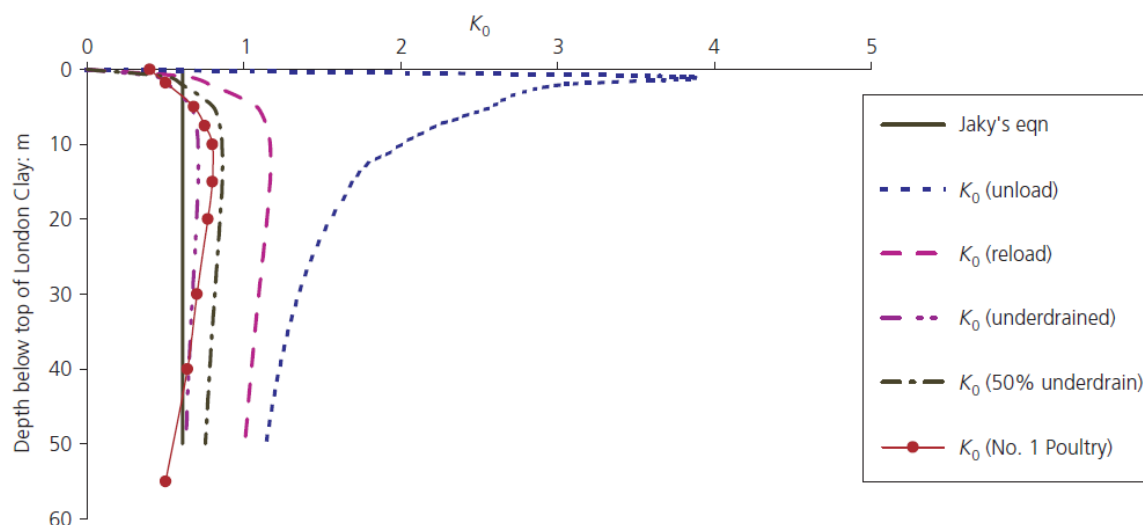


Figure 3.5 - Different approaches on K_0 variation with depth (reproduced from Wright, 2010)

Note should be taken that although many approaches were developed to determine the initial stress, neither a general formula nor a generally applicable experimental procedure are available to date, therefore engineering judgement is required to determine the best approach in each specific context.

3.3.2. UNDRAINED SHEAR STRENGTH, c_u

Ground materials can exhibit either strain-hardening or strain-softening behaviour depending on the initial soil state, soil structure and their changes. Figure 3.6 represents the variation of the shear strength in overconsolidated soils. Once the peak is reached overconsolidated soils tend to exhibit strain-softening, which can be identified by shear banding. The continuous reduction of shear strength leads to a residual state, which, in London Clay can be recognised by reorientation of clay particles in parallel to a slipping surface, at this state, London Clay exhibits a very low shear resistance.

Whenever shear strength is mentioned in this subsection it is always referred to peak shear strength. Besides, London Clay has a typical undrained behaviour when loaded, hence in this subsection it will only be referred characteristics and approaches to estimate the undrained shear strength.

From all the shear strength characteristics of the natural London Clay, the most important that should be taken into account when estimating this parameter is anisotropy. The influence of this characteristic will be briefly explained in the following paragraphs.

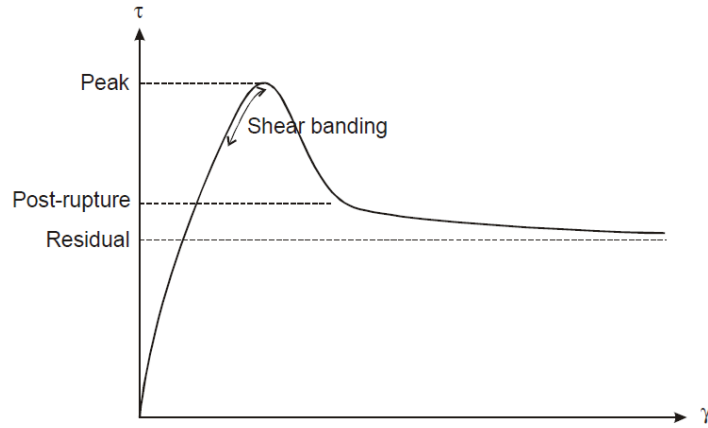


Figure 3.6 - Illustration of shear stress - strain behaviour in overconsolidated soils

There are many ways to estimate the shear strength, either running laboratory tests or in-situ tests. The standard method to investigate the anisotropy in laboratory was to perform triaxial and plain strain tests using samples in which a variety of orientations were covered. Investigations conducted by Ward *et al.* (1959) revealed that the ratio of undrained shear strength in triaxial compression of vertical and horizontal samples, c_{uH}/c_{uV} , was around 1.3 for samples of London Clay.

By performing triaxial compression tests with vertical, diagonal and horizontal samples, Bishop (1966) proposed a formula that can estimate the undrained shear strength in any direction, using θ , which represents the direction of the sample axis measured from the vertical axis.

$$c_u = c_{uV} \cdot (1 - a \sin^2 \theta) \cdot (1 - b \sin^2 2\theta) \quad (3.5)$$

where a and b are constants, which value depends on the material used in the samples as well as the sample size. For a sample with a diameter of 38 mm of London Clay, the constants assume the following values, $a = -0.07$ and $b = 0.30$.

While in total stress terms it can be verified a reduction in shear strength for diagonal samples, in effective stress terms it was not identified significant difference in the strength parameters (Bishop *et al.*, 1965 and Agarwal, 1968). This comparison allowed to conclude that the anisotropic shear strength of the London Clay was simply a consequence of pore water pressure.

The undrained shear strength can also be estimated based on Mohr's circle considerations. The following formula represents the relation between the undrained shear strength and the in-situ effective stresses, which in turn depends on the depth below the ground surface. The Equation 3.6 is easily deduced from Mohr's circle geometry when it touches the failure envelope (see Figure 3.7).

$$c_u = \tau(\text{failure}) = \left(\frac{\sigma'_h \sin \phi' + c' \cos \phi'}{1 - \sin \phi'} \right) \cdot \cos \phi' \quad (3.6)$$

where σ'_h represents the horizontal effective stress, c' the effective cohesion and ϕ' the effective angle of shearing resistance.

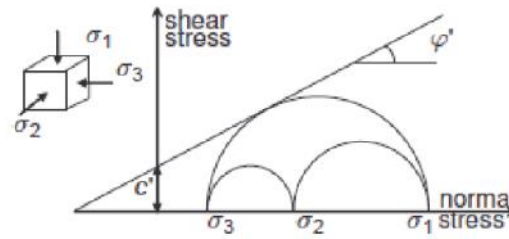


Figure 3.7 - Failure envelope for Mohr-Coulomb model

Wright (2010) presents a typical profile to estimate c_u in London Clay of $50 + 8z$ kPa, where z represents the depth in meters. It can be easily noticed that c_u increases rapidly with depth due to the high overconsolidation ratio and high K_0 values. Although this is a typical profile, in Chapter 4 it will be presented a specific profile for the London Clay units that were found in the construction site. This profiles will be based on the Geotechnical Interpretation Report (CH2M, 2013).

Apart of the laboratory tests and the analytical formulas, vane shear tests, pressuremeter tests, Cone Penetration Tests (CPT) or Standard Penetration Tests (SPT) can also be performed to estimate this parameter in-situ.

3.3.3. SOIL STIFFNESS – SHEAR MODULUS (G) AND YOUNG'S MODULUS (E)

According to Jardine *et al.* (1986), soil stiffness vary with the amount of strain to which it is subjected, which makes it another difficult parameter to assess. As happens with undrained shear strength also the soil stiffness varies with the anisotropy of London Clay.

Triaxial compression tests on horizontally and vertically cut samples performed by Ward *et al.* (1959) revealed that the ratio of undrained Young's Modulus E_{uH}/E_{uV} was between 1.1~2.0. Later, Atkinson (1973 and 1975) reported that the Young's Modulus ratio of the horizontal to the vertical was around 2 in both undrained and drained conditions. This value was the result of triaxial tests and plane strain tests with vertical and horizontal strains performed by him.

Figure 3.8 shows the variation of shear stiffness, G , with strain in London Clay (from Hight and Higgins, 1994). The shear stiffness, G , presented in the graphic, is assumed to be equal to $E/3$ for undrained conditions, it means that the Poisson's ration, ν , is equal to 0.5, as Equation 3.7 shows.

$$G = \frac{E}{2 \cdot (1 + \nu)} \quad (3.7)$$

Note should be taken that the G values are normalized by the mean effective stress, p' :

$$p' = \frac{\sigma'_v + 2\sigma'_h}{3} \quad (3.8)$$

where σ'_v represents the vertical effective stress and σ'_h the horizontal effective stress.

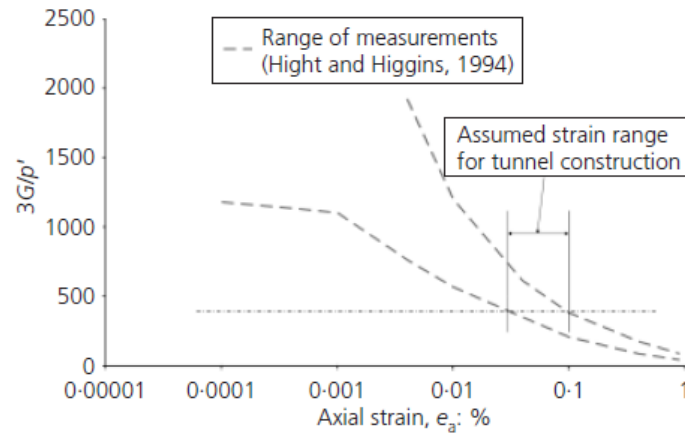


Figure 3.8 - Variation of London Clay shear stiffness in comparison with strain (reproduced from Wright, 2010)

Wright (2010) suggested that an undrained Young's modulus of 250 to 600 times c_u would be a reasonable assumption for predictions of ground deformations for London Clay for tunnels construction.

4

CASE STUDY

4.1. PROJECT DESCRIPTION - BOND STREET STATION UPGRADE

Bond Street Station, located at the Europe's busiest shopping street - Oxford Street, central London -, has suffered several congestion problems caused by heavy demand. Bond Street Underground Station opened in 1900 to be part of the Central London Railway, now the Central Line. In the 1970s major works were carried out to incorporate the new Fleet Line, now known as Jubilee Line, into the station.

Figures from Transport for London (TfL) revealed that more than 185,000 passengers/day currently use this station, and that this number is expected to increase to 225,000 by 2018, when the Elizabeth line (currently designated as Crossrail) connects to this station (TfL's website, May 2016).

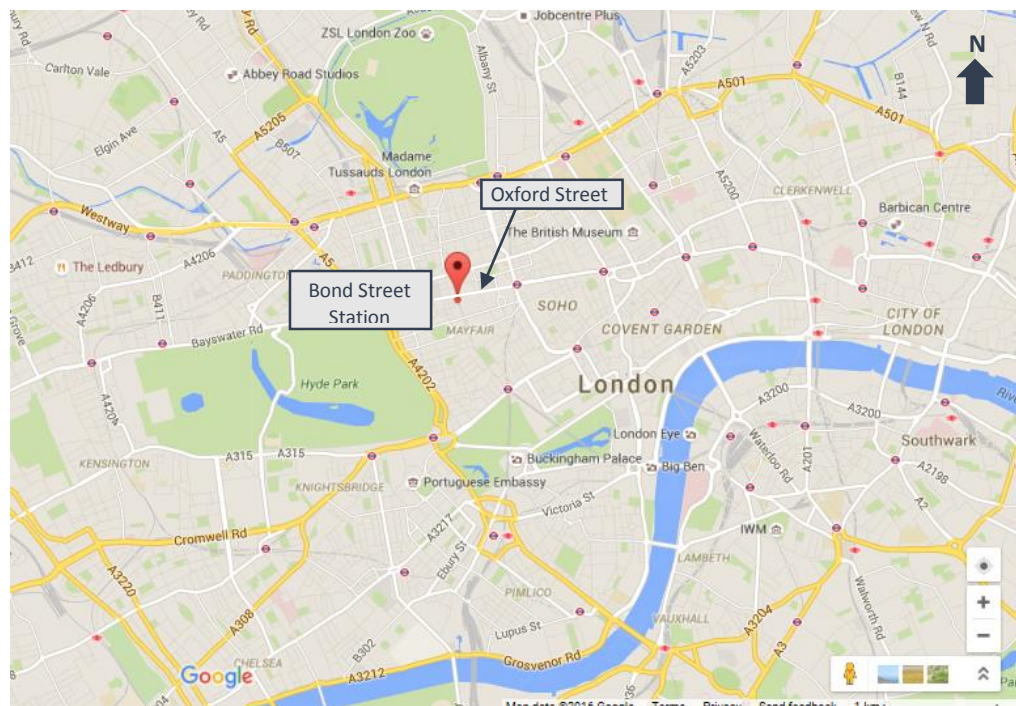


Figure 4.1 - Location of Bond Street Station in London (Google Maps © 2016)

Figure 4.2 corresponds to an excerpt of the London Underground map, which allows to see that Bond street station is attended by two lines of the London Underground network, the Central line, which runs east-west beneath Oxford Street and the Jubilee line, which runs approximately north-south below the

properties along the west side of Stratford Place. In the Central line the Bond Street Station is located between Marble Arch and Oxford Circus and in the Jubilee line, between Baker Street and Green Park.

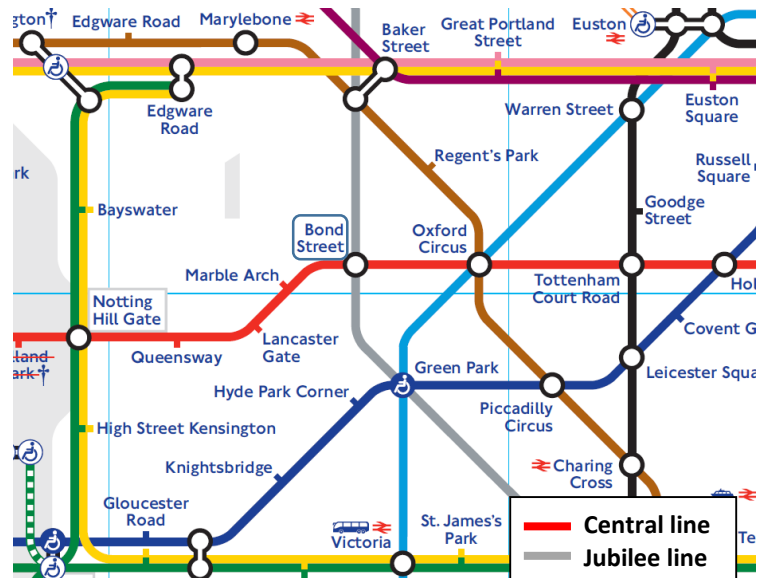


Figure 4.2 - Bond Street Station, Central and Jubilee lines location (excerpt of the London Underground map)

The main goal of London Underground Limited (LUL), part of TfL, with the Bond Street Station Upgrade project (BBSU) was to provide congestion relief, improving the access to the Jubilee line platforms and developing step free access once this station had poor access for mobility impaired passengers.

The Bond Street Station Upgrade main works were based on providing a new entrance on the north side of Oxford Street, a new connection between the Jubilee and Central lines improving the interchange with new lifts and escalators and developing a direct route to the new Crossrail Bond Street Station.

The majority of the tunnelling works has been developed either using sprayed concrete linings (SCL) or performing traditional squareworks. Traditional squareworks are mainly required at joints, and involve complex small, hand excavation, in which timber support is used; when completed, squareworks are presented in form of concrete encased structural box frames. This LUL project faces several site constraints due to the magnitude of its location as well as the London underground complexity. Therefore traditional squareworks was adopted where space constraints from the tunnelling proximity to the existing structures prevented the use of SCL. However in some situations a hybrid SCL and squareworks design was taken.

As part of these works, the construction of the proposed Northern Tunnel to improve access to the existing Jubilee line Platforms, which is represented in Figure 4.3 in soft green, would pass beneath the existing Post Office Tunnel (POT). Thereafter, the proposed Northern Tunnel Underpass, known as the Post Office Tunnel Underpass (POTU), was expected to develop the largest ground movements in this section of the Post Office Tunnel. The POTU was reported as one of the most complex parts of the works developed within the BSSU, therefore the construction of the Underpass will be the focus of the analysis undertaken in thesis.

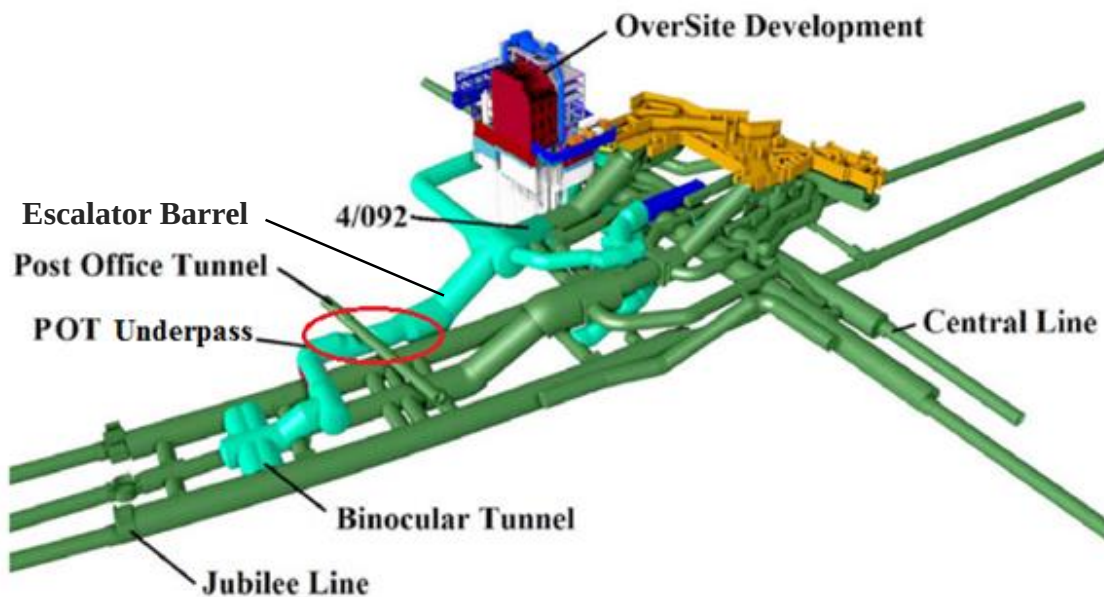


Figure 4.3 - Overall view of Bond Street Station Upgrade (reproduced from Kumpfmüller *et al.*, 2014)

The London Underground complexity is clearly evident in Figure 4.3. In order to compose an efficient Underground station an enormous amount of underground structures has to be performed, from a complex web of train tunnels and pedestrian walkways to escalators. Due to this complexity and with the continuous increase of underground congestion in London, new improvements have to deal with multiple congestions, which make the designers' job even more challenging. The interaction between different tunnels is therefore a subject with increasingly importance.

4.1.1.1. SITE LOCATION

Bond Street is a London Underground station located at Oxford Street near the junction with New Bond Street in City of Westminster, London, W1R 1FE. The site is delineated by Wigmore Street to the north, St Christopher's Place and Gees Court to the west, Marylebone Lane to the east and Oxford Street to the south (Figure 4.4).

The station is located beneath the West One shopping arcade on the corner of Oxford Street and Davies Street and can be accessed by the shopping. The street-level entrances are approximately 200 m west of New Bond Street itself.

Post Office tunnel runs approximately east-west through the north half of the site, crossing underneath the No. 9 Stratford Place building and Stratford Place to the north of Oxford Street.

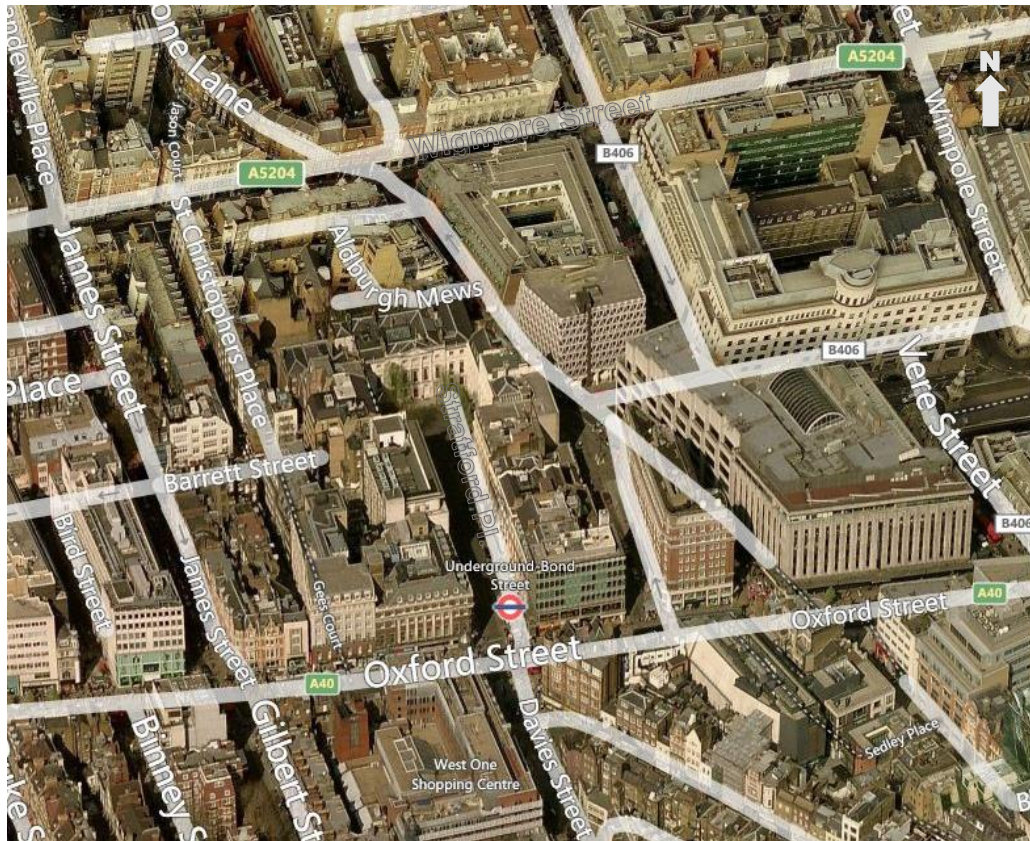


Figure 4.4 - View of the site location (Bing Maps © 2016)

4.1.2. POST OFFICE TUNNELS

The Post Office underground railway was a unique solution to overpass the choked streets of London and reduce the time spent in transporting large volumes of mail across the city. The Post Office tunnel construction started on February 1917 and continued until February 1927 when the first line, between Paddington and West Central District Office became available for training. The complete Mail Rail, connecting Paddington to Whitechapel along 10.5 Km, was commercially operational since then until 2003, when it was closed due to the high railway transportation cost.

Although this Royal Mail' asset is now inactive, the use of the tunnel by third parties has risen recently and is likely to grow with the construction of the Crossrail project.

The tunnel was constructed using bolted cast iron tunnel segments forming tunnel rings with an external diameter of 2971 mm. As shown in Figure 4.5, each ring has 6 segments plus a key segment and each segment had an injection hole about its centre, most of which were fully grouted up to fill the gap between the soil and the segments. The segments were bolted with three bolts to each longitudinal joint, total 21 bolts per section, and each tunnel ring was bolted to the next with 35 bolts. The length of each tunnel ring is 508 mm.

The railway track is supported on the concrete invert and is formed by two tracks, each comprising two running rails and a central contact rail. The following image represents a typical cross section of the Post Office Tunnels, with all its main features.

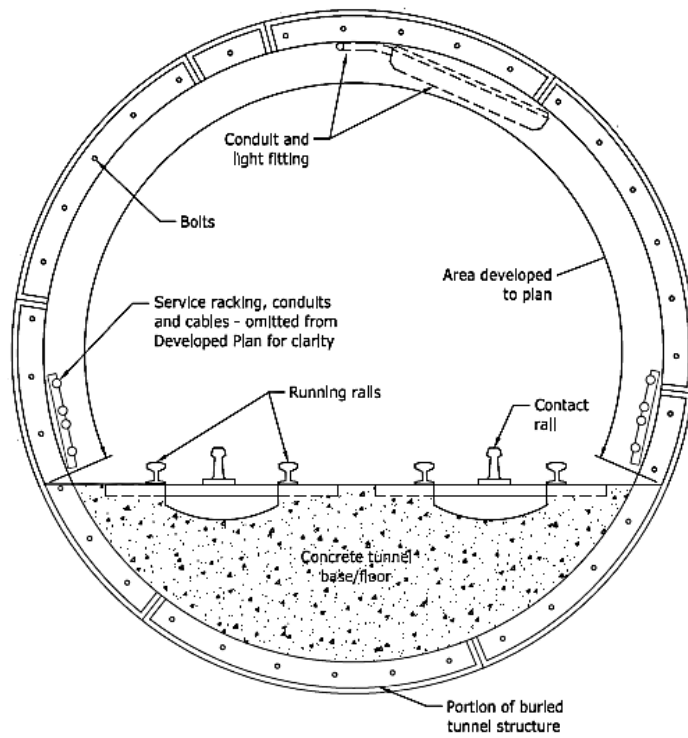


Figure 4.5 - Typical section through a Post Office Tunnel

Figure 4.6 represents schematically the segments's longitudinal section, with its dimensions presented in Table 4.1. This section properties were reproduced from the LUL reports provided to the author and will be used later in the thesis.



Figure 4.6 - Representative scheme of cast iron segment's longitudinal section

Table 4.1 - Cast iron tunnel segment's longitudinal section dimensions

Parameter	Unit	
Overall depth of section, h	mm	114.0
Thickness of flange, t_f	mm	25.4
Thickness of web, t_w	mm	19.0
Width of ring, w	mm	508.0

4.1.3. CONSTRUCTION METHOD

The works undertaken to construct the Underpass were performed by a joint venture between Costain and Laing O'Rourke (CoLOR), which conducted it using the SCL method, with a squatted elliptical cross section beneath the Post Office Tunnel (Figure 4.7 b)) (equivalent to a circular cross section of 5.35 m diameter), and an ovaloid profile for either sides of the POT (Figure 4.7 a)), as it is suggested in Figure 4.8. The thickness of the primary and secondary sprayed concrete linings was assigned for both sections equal to 250 mm.

As Figure 4.8 suggests, away from the crossing section, the excavation was performed by two consecutive top heading excavations of 1 m long, followed by one invert advance of 2 m long, which in turn is performed with 1 m lag. As the excavation face came near the crossing section, the critical region, the excavation was then simply performed by 1 m advances, combining the top heading and invert excavation in an equivalent scheme.

The new sprayed concrete lining Underpass advanced from the south, northwards, at an approximately 15° angle from the perpendicular of the POT, as Figure 4.9 shows.

HAT was appointed as CoLOR's designer, which represent a joint venture between Halcrow (now part of CH2M) and Atkins.

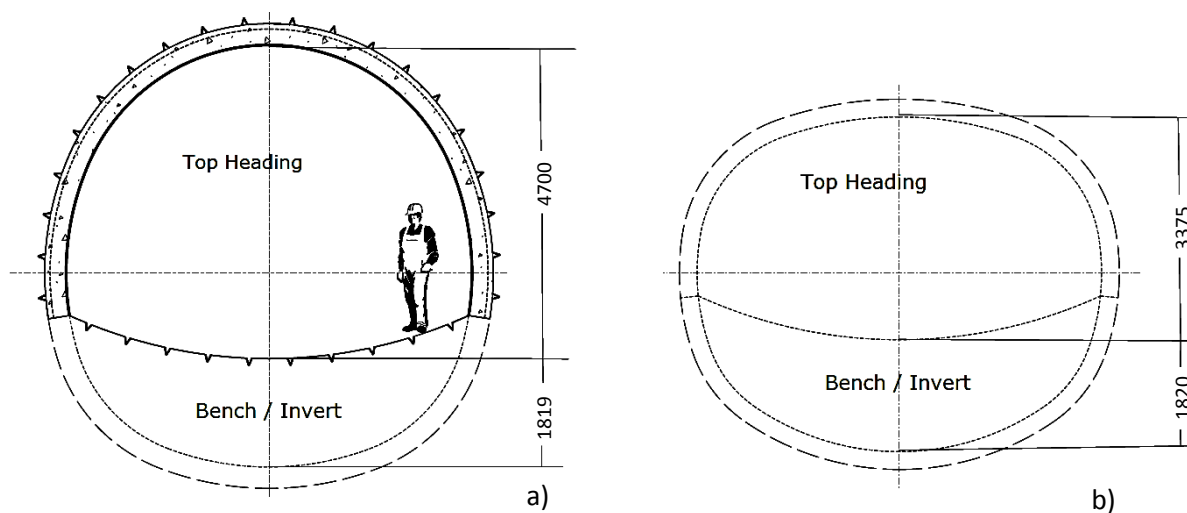


Figure 4.7 - Cross sections, courtesy of HAT (dimensions in mm); a) Ovaloid section; b) POTU section

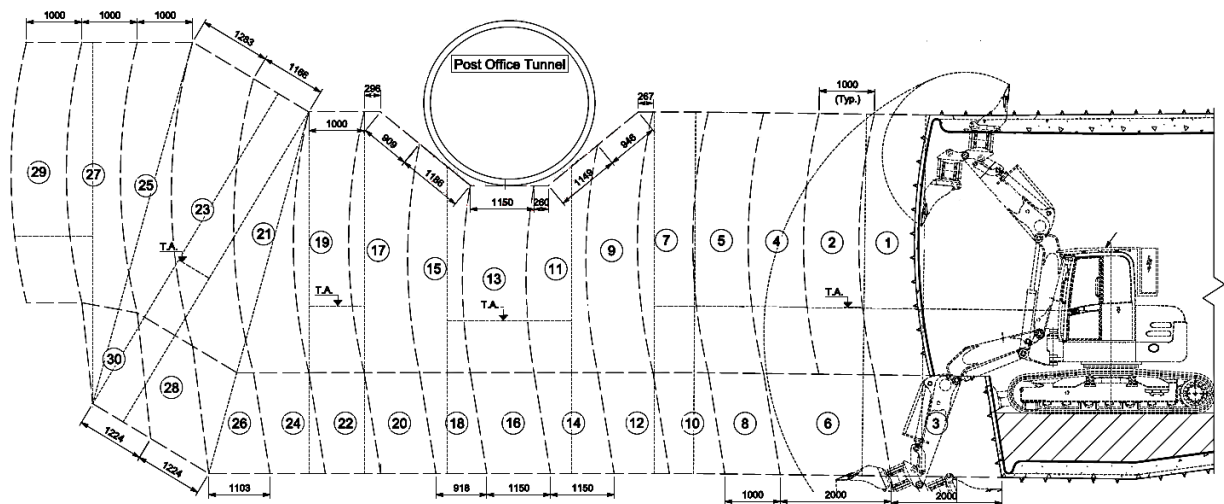


Figure 4.8 - Excavation sequence (HAT courtesy)

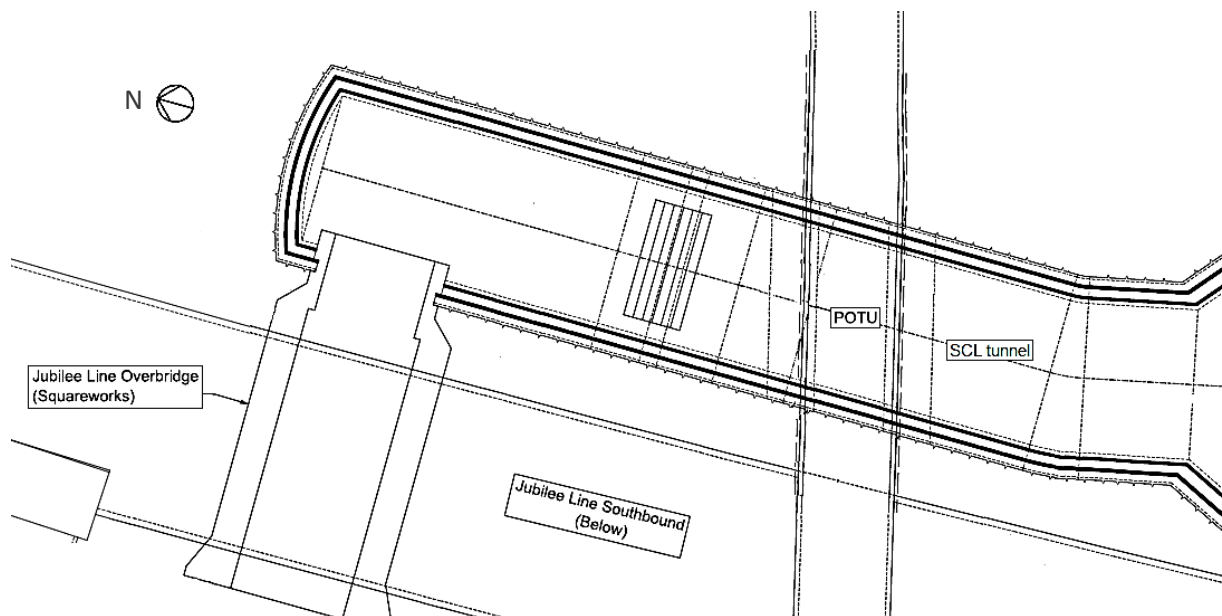


Figure 4.9 - Plan view – draft (HAT courtesy)

4.1.3.1. Under-crossing Post Office Tunnel

When the excavation face was approaching the estimated position of the POT, the works were carefully performed in order to avoid damaging the cast iron lining. Works progressed slowly to expose small sections of the POT, removing the surrounding grout to reveal the external cast iron lining. According to CoLOR reports, the underside of the existing POT was actually exposed during the excavation (see Figure 4.10). After exposing the POT lining, the SCL works continued and the shotcrete was directly sprayed onto the cast iron.



Figure 4.10 - Construction of the underpass: exposing the POT cast iron lining (HAT courtesy)

4.1.3.2. Tunnelling works programme

The Underpass tunnelling works was programmed to commence right after the excavation of the inclined escalator barrel, which leads to the Underpass (see Figure 4.3). However, CoLOR reported problems with material supply, which had to be resolved before the Underpass tunnelling works started. This interruption was accepted since the works underneath the POT had to be undertaken with consistent advances in order to reduce the risk of damaging the POT.

Once the conditions were assembled, the tunnelling resumed on the 10th of November of 2014, and the under-crossing reached on the 27th of November of 2014. Table 4.2 summarizes the Underpass construction sequence as shown in Figure 4.8, relating each advance with the respective date and its relative distance to the POT centre-line. Although Figure 4.8 represents advances from No. 1 to No. 30, the author only had access to information regarding advances No.1 to No. 23. The horizontal distance from the POT centre-line of the remaining advances can be seen in Figure 4.8, however the corresponding date is not available, hence only advances from No. 1 to No. 23 will be taking into account and represented in Table 4.2, furthermore is not expected that the excavation after advance No. 23 has great impact in terms of inducing movements in the POT.

Table 4.2 - Underpass advances progress

Advance No.	Horizontal Distance from POT Centre-Line [m]	Date
1	-7.225	10/11/2014
2	-6.225	22/11/2014
3	Same as advance No. 1	22/11/2014
4	-5.225	23/11/2014
5	-4.225	24/11/2014
6	Same as advance No. 4	24/11/2014

7	-3.225	25/11/2014
8	Same as advance No. 5	25/11/2014
9	-2.225	25/11/2014
10	Same as advance No. 7	26/11/2014
11	-1.150	27/11/2014
12	Same as advance No. 9	27/11/2014
13	0	28/11/2014
14	Same as advance No. 11	28/11/2014
15	1.034	28/11/2014
16	Same as advance No. 13	29/11/2014
17	1.993	29/11/2014
18	Same as advance No. 15	30/11/2014
19	2.993	30/11/2014
20	Same as advance No. 17	01/12/2014
21	3.993	01/12/2014
22	Same as advance No. 19	02/12/2014
23	4.993	02/12/2014

Note that this table only represents the distance from the top heading advances since they are the ones with more interest in the displacements as they are excavated 1 m ahead from the invert advances. The distance of each advance from the POT centre-line (measured along the SCL tunnel position) corresponds to the distance from the POT centre-line to a middle point of each advance.

4.2. GROUND MODEL AND DESIGN PARAMETERS

The site ground conditions and appropriate geotechnical design parameters for each formation encountered, were detailed in the Geotechnical Interpretative Report (GIR) provided to the author CH2M (2013). This report provides a summary of the ground investigation works undertaken which comprised boreholes performed with sampling and in-situ testing, stratigraphic logging and a laboratory testing programme.

This subchapter intends to present a brief description of the site geological conditions and which geotechnical design parameters will be used in the models developed in the following chapters. The importance of this subchapter relies on the influence of the ground behaviour in the tunnel performance, either regarding displacements or forces. It is important to perfectly understand the ground conditions to adopt adequate design parameters in order to determine realistic results.

4.2.1. GROUND MODEL

4.2.1.1. Ground conditions

The following table summarizes the main features of the encountered formations and its divisions. From the GIR it was possible to identify the level of each stratum, which is presented from the top of the stratum, and a brief geological description.

Table 4.3 - Simplified ground profile and geological description

Stratum		Level of top of the stratum [mLUD]	Thickness [m]	Geological description
Superficial material	Made Ground	123	2	Tarmacadam and concrete encountered at the surface; beneath, the made ground is consisted of a mixture of granular and cohesive material: - dark brown to brown, slightly sandy slightly gravelly CLAY, with ash, fragments of brick, flint and clinker; - brown slightly clayey gravelly SAND, with red brick, flint and occasional clinker; - GRAVEL, with flint and brick to a dark brown slightly clayey slightly gravelly SAND with flint and brick, to brown and dark brown very sandy GRAVEL, with fragments of brick, mortar and flint.
	River Terrace Deposits	121	6	Dense becoming medium dense, orange brown very sandy GRAVEL which is sub-angular to rounded fine to coarse flint; fine to coarse SAND. Soft to firm orange brown slightly gravelly CLAY; above, a thin layer of soft slightly sandy slightly gravelly CLAY.
London Clay Formation	LC A3	115	11	Stiff to very stiff fissured brown grey CLAY; fissures are closely spaced, randomly orientated and generally matt and smooth. Rare pyritised wood fragments, occasional pyrite nodules, occasional mudstone, rare shell fragments and rare burrows were encountered.
	LC A2	104	11	Stiff to very stiff bioturbated slightly sandy CLAY, with frequent partings of grey fine sand and silt; occasional to frequent dark brown and white silt and light grey clay, variably interbedded with stiff to very stiff fissured grey brown CLAY. Fissures were closely spaced, randomly orientated, smooth and polished.

				Grey burrows infilled with grey silt and clay, rare pyrite nodules, rare shells and rare to occasional lignite fragments, were occasional encountered.
Lambeth Group	Upper Mottled	93	5	Stiff to very stiff fissured mottled grey, blue, red and brown CLAY; fissures are very closely spaced to closely spaced randomly orientated, smooth, matt and polished.
	Lower Mottled	88	12	Stiff to very stiff fissured mottled red, brown, yellow, grey CLAY; fissures are very closely to extremely closely spaced, randomly orientated, smooth, matt, slightly polished and occasionally striated.

Note that the level of the strata was represented in mLUD, this unit derived from London Underground Datum which is a specific tunnel datum based on an ordnance datum, used in designing tunnels that pass below the sea level. In order to avoid dealing with negative elevations, this datum was created subtracting 100 m to the ordnance datum.

4.2.1.2. Hydrogeology

Two major aquifers exist within the vicinity of the site: the Upper Aquifer, within the superficial deposits of Made Ground and River Terrace Deposits; and the Lower Aquifer, comprising the basal units of the Lambeth Group. However neither of the aquifers have any influence on the site hydrology.

The groundwater levels in the River Terrace Deposits at the boreholes performed near the No 2 Stratford Place, were recorded at approximately 117 mLUD.

4.2.2. GEOTECHNICAL DESIGN PARAMETERS

The geotechnical design parameters are a fundamental aspect to the accuracy of the results obtained by numerical analysis, they represent the real properties of the soil and its definition should be as cautious as possible in order to approximate the modelled materials to the reality.

This section presents a summarized table including the recommended geotechnical parameters presented in the GIR. These parameters were estimated using numerous laboratory and in-situ tests, hence its accuracy is reliable.

Table 4.4 - Geotechnical design parameters to be adopted (CH2M, 2013)

Geotechnical parameters	Unit	Superficial material		London Clay Formation		Lambeth Group	
		Made Ground	River Terrace Deposits	LC A3	LC A2	Upper Mottled	Lower Mottled
Unit weight, γ	kN/m ³	19	19	20	20	21	20

Stiffness	Drained vertical Young's Modulus, E'	MPa	10	50	$32+2.8z_1$	$60+z_2$	100	120
	Drained Poisson's Ratio, ν'	-	0.3	0.3	0.2	0.2	0.2	0.2
Strength	Undrained Shear Strength, c_u	kPa	n/a	n/a	$80+7z_1$	$150+2.5z_2$	250	250
	Effective cohesion, c'	kPa	0	0	5	10	10	10
	Angle of shearing resistance, ϕ'	° (degrees)	28	38	22	24	26	26
Initial conditions	In-situ earth pressure coefficient, K_0	-	0.53	0.38	1.2	1.2	1.2	1.1
Flow parameters	Vertical permeability k_v	m/day ($\times 10^{-15}$)	11570	11570	1.157	1.157	1.157	1.157
	Horizontal permeability k_h	m/day ($\times 10^{-15}$)	11570	11570	5.787	5.787	11.57	11.57

* z_1 and z_2 represent the depth in m below the top of LC A3 or LC A2 as appropriate.

For tunnel lining design the GIR presents two more different estimates for the in-situ earth pressure coefficient, which represent lower bounds. For tunnels to be constructed at least one diameter away from an existing tunnel, $K_0 = 0.8$ should be assumed for London Clay as well as for the Upper Mottled layers, whereas for the Lower Mottled $K_0 = 0.7$; for the remaining layers the same values are considered. For tunnels to be constructed within one diameter of an existing tunnel, the in-situ earth pressure should be assumed even lower, which would be $K_0 = 0.65$ for London Clay and Upper Mottled and, $K_0 = 0.6$ for the Lower Mottled, the remaining layers assume the same value as represented in Table 4.4.

4.3. MONITORING DATA

The impacts of a tunnel excavation in the surrounding area are numerous, vertical and horizontal displacements, drawdown of water level, variation of stress, radial displacements along the lining of the excavation, among others. Therefore, these impacts must be recorded, studied and, if necessary, minimized to reduce the risk of damage to the surrounding structures, whether they are installed in the

ground surface or in the subsurface as well. To register all the impacts it is required an appropriate monitoring plan, which, whenever possible, must include real time monitoring to allow a quick response.

The excavation of the underpass beneath the Post Office Tunnel induced movements in the POT itself, which due to its close proximity to the excavation had to be carefully monitored. This subchapter will present the monitoring plan and the data registered from the Post Office tunnel. The information that will be presented here was provided to the author through spreadsheets (RAW data) performed by a specific software used by the joint venture CoLOR.

4.3.1. MONITORING PLAN

The monitoring plan consisted of using fixed prisms together with an automatic theodolite so it could provide real-time data. The real-time data allowed a continuous record of the displacements of the Post Office Tunnel during the progress of the excavation underneath.

The POT monitoring consists in five prisms along the tunnel lining to allow any ovalisation to be interpreted and control the trigger levels. Figure 4.11 represents the location of the monitoring prisms. Ideally, the RP2 and RP4 prisms (note that RP stands for Reference Prism) should be installed in the horizontal axis of the tunnel, however due to existing utilities located at the tunnel lining this was not possible. Once the invert of the Post Office tunnel is filled with mass concrete the RP1 and RP5 prisms were installed on the lowest level possible.

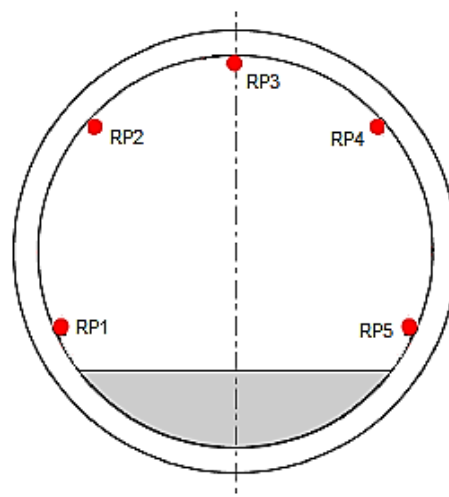


Figure 4.11 - Schematic representation of the monitoring instruments - typical location in Post Office Tunnel section (looking West)

Along the Post Office Tunnel there were eight monitoring sections, which are represented in Figure 4.12. Each section had a similar position for the prisms as represented in Figure 4.11.

As Figure 4.12 suggests, the centre-line of array 4 is located exactly on the vertical axis of the new tunnel, hence the centre-line of this section (array 4) will be used as a reference to define the relative position of all arrays along the POT. Note that array 4 will be hereafter referred as POT reference section. The exact relative position of the remaining arrays is presented in Figure 4.13 as well as the exact vertical position of the prisms in a longitudinal view. In this figure it is possible to identify that the prisms position slightly vary from the represented in Figure 4.11 through all monitoring sections. This difference is explained by obstruction that impacted the installation of the instruments, however this fact

does not create a considerable influence in the monitoring results, once these are based on the exact prisms coordinates.

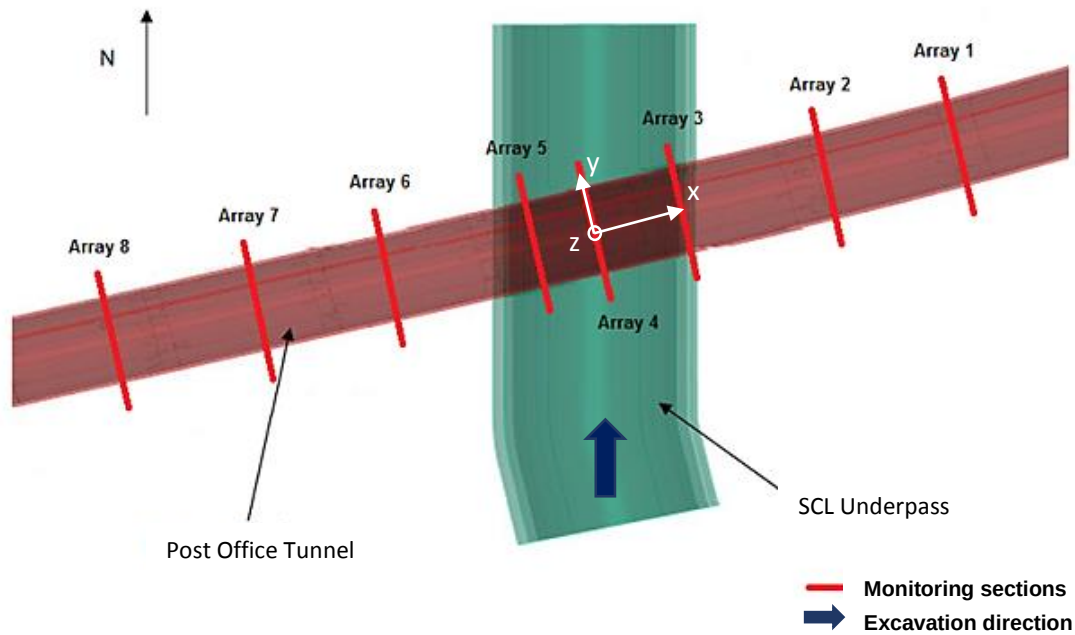


Figure 4.12 - Illustration of the location of the monitoring sections

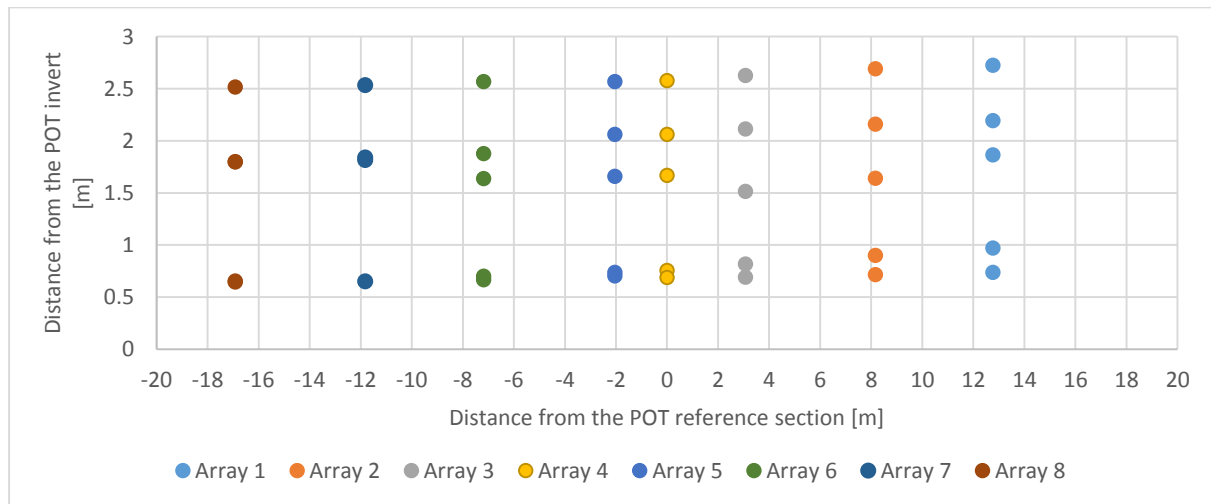


Figure 4.13 - Exact prisms location along the Post Office Tunnel (North elevation)

4.3.2. MONITORING DATA

The present section reviews the monitoring data provided to the author and presents the analyses performed using the mentioned information. This section will be divided in four different subsections, each subsection will approach a specific movement of the POT.

The monitoring data presented in this section was recorded from the 19th of July of 2014 to the 28th of February of 2015 so it could take into account the influence of the stages before the Underpass excavation works until all displacements stabilized.

It is worth referring that although this interval comprises the displacements until the 28th of February of 2015, the author is not able to specify the distance of the excavation face from the POT centre-line after the 2nd of December of 2014 (see 4.1.3.2.). Hence, whenever the author includes the influence of the excavation face in the analysis presented in this section, it will always be related to the timeframe presented in Table 4.2.

4.3.2.1. Vertical displacements

Beginning with settlement analysis, it is more useful to start presenting the RAW data recorded by a specific prism in all arrays throughout the excavation process. It will allow to quickly understand the main influence of the excavation advances in the POT behaviour. Figure 4.14, represents the settlements captured by RP3 in all arrays, throughout all excavation process. This prism was selected as a reference for this representation because it is the prism located on the vertical axis thus it reproduces an average behaviour of the tunnel settlements.

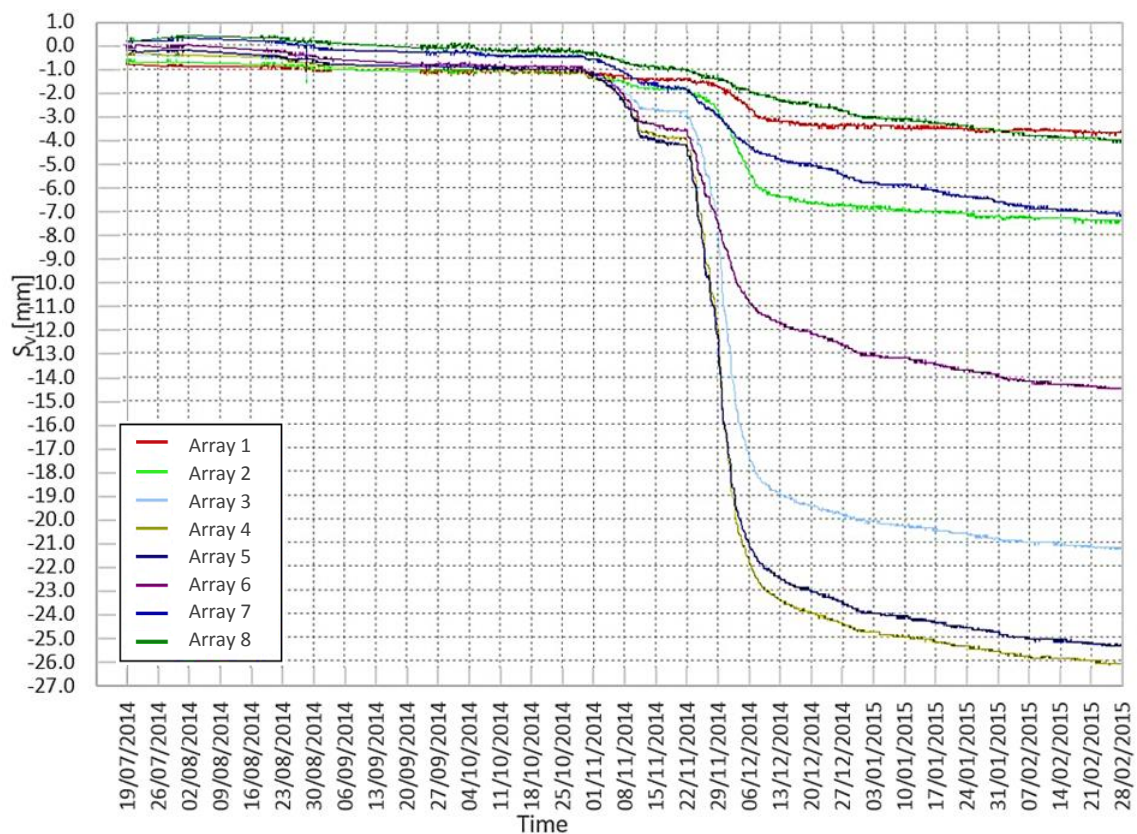


Figure 4.14 - Raw data recorded by RP3 in all arrays throughout all excavation process: Vertical displacements (CoLOR courtesy)

It is clearly identifiable when the excavation face reaches the crossing section as displacements increase abruptly, which corresponds to the 28th of November of 2014. It is possible to recognise that arrays 4 and 5 are the ones that recorded higher displacements, which is perfectly understandable once they are

the arrays nearest the cross section where the POT lining becomes exposed. The maximum settlement recorded by this prism corresponds to array 4 and was approximately 26 mm. As arrays become more distant, the recorded displacements decrease, which evidences that settlements follow an approximately normal distribution with the distance of the arrays.

In the beginning of November 2014, arrays 4 and 5 started to record some slight displacements, which evidences that the excavation face was approaching the crossing section, however, almost one week later (approximately on the 10th of November) the recorded displacements stagnate, until the middle of the mentioned month. The reached displacements level evidences the problem regarding the materials supply reported by CoLOR. Since the tunnelling works to construct the Underpass had to be performed with consistent advances, CoLOR demanded to stop the works until the problem was resolved. Once the problem was resolved the works resumed which can be identified in the plot by the abrupt increase of recorded displacements.

The plot represented in Figure 4.14 identifies the behaviour of the POT due to excavation advances, however only vertical displacements recorded by prisms RP3 are presented. To understand the general behaviour of the POT through all arrays, i.e. to identify the movement along the POT, it is important to also clarify the movements recorded in the remaining prisms. The following figure, Figure 4.15 represents the settlements recorded by each prism separately, along all arrays. The points represented in the different curves correspond to the vertical displacements registered by the prisms in each array. The curves were then designed adjusting the points of each advance. Hence, the abscissa of the plots corresponds to the distance from the POT reference section (array 4) measured along the POT, according with the axis represented in Figure 4.12, consequently each point in the curves correspond to the position of the different arrays (see Figure 4.13 to locate the different arrays).

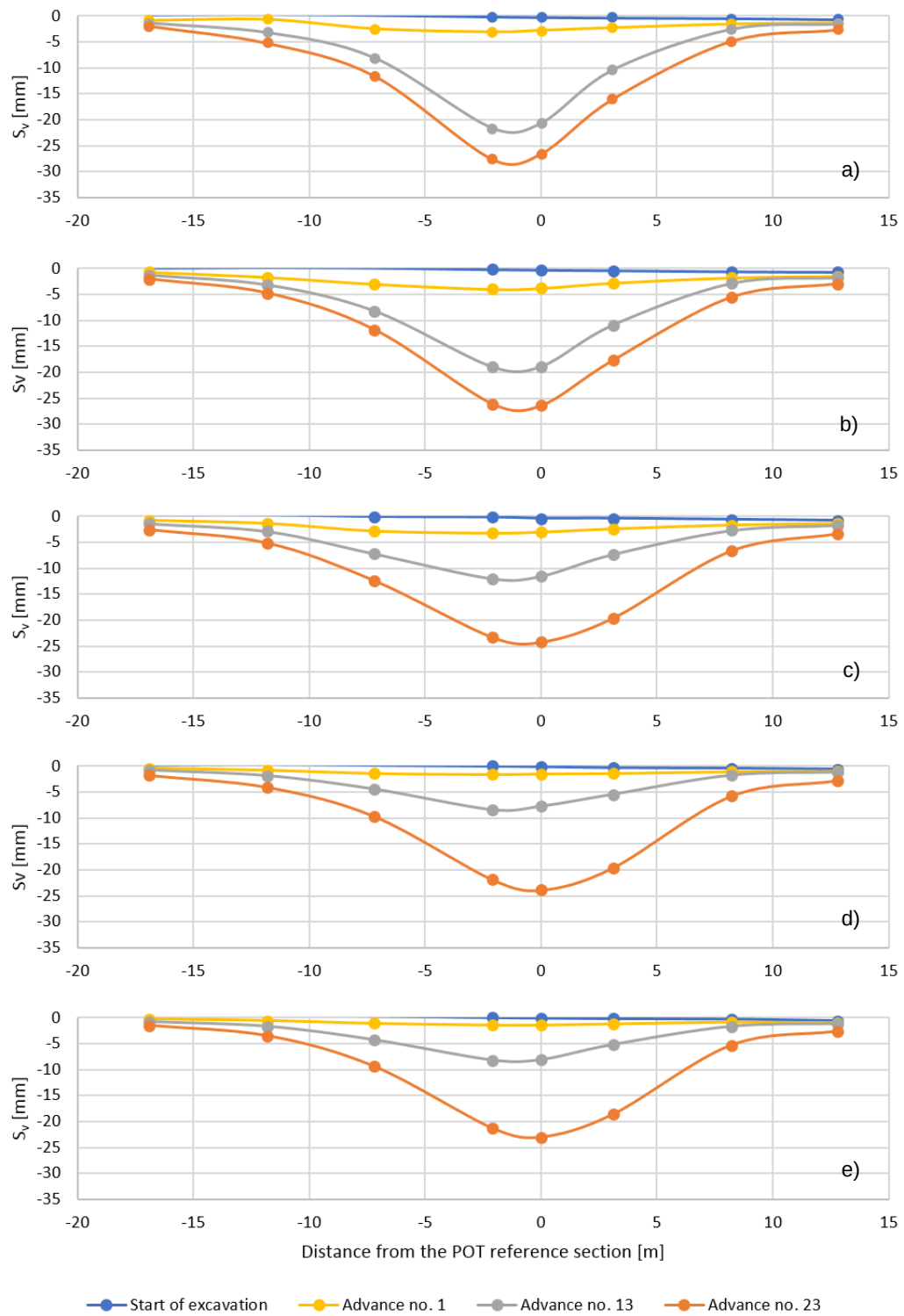


Figure 4.15 - Vertical displacements along POT; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5

The presented plots evidence the settlements through four different stages: in the start of the excavation, which represents a specific stage where the excavation face is distant enough to have no influence on POT displacements; in advance No. 1 (as represented in Figure 4.8), where the excavation face is

approximately one diameter distant from the POT centre-line; in advance No. 13, which represents the under-crossing stage; and in advance No. 23, which is the last stage the author had access to.

Theoretically, the end of the Underpass tunnelling works should be represented through a stage where the excavation face was at least two diameters distant from the cross-section in order to reproduce a situation in which the excavation face had no influence on the POT. However, is it acceptable to select advance No. 23 as the final stage in this analysis since the displacements recorded in this stage were almost coincident with the ones recorded in the 28th of February of 2015, which is the latest recorded date provided to the author.

It can be seen that the maximum settlement recorded in the last stage (advance No. 23), which is approximately 28 mm, was recorded in array 5 by prism RP1. As mentioned before, the largest movements were expected to occur in array 4 section as it is representative of the central section, however, in fact, since the tunnels are not perpendicular, the new tunnel excavation face first reaches array 5, as Figure 4.16 suggests. Therefore, the largest displacements may actually be expected in prism RP1 in array 5. As it is possible to identify, prism RP1 is the one closest to the excavation face as it is simultaneously located at the side of the progression of the excavation and closest to the Post Office Tunnel invert (see Figure 4.11 to locate the different prisms, note that the mentioned figure represents the prisms location looking west in the POT).

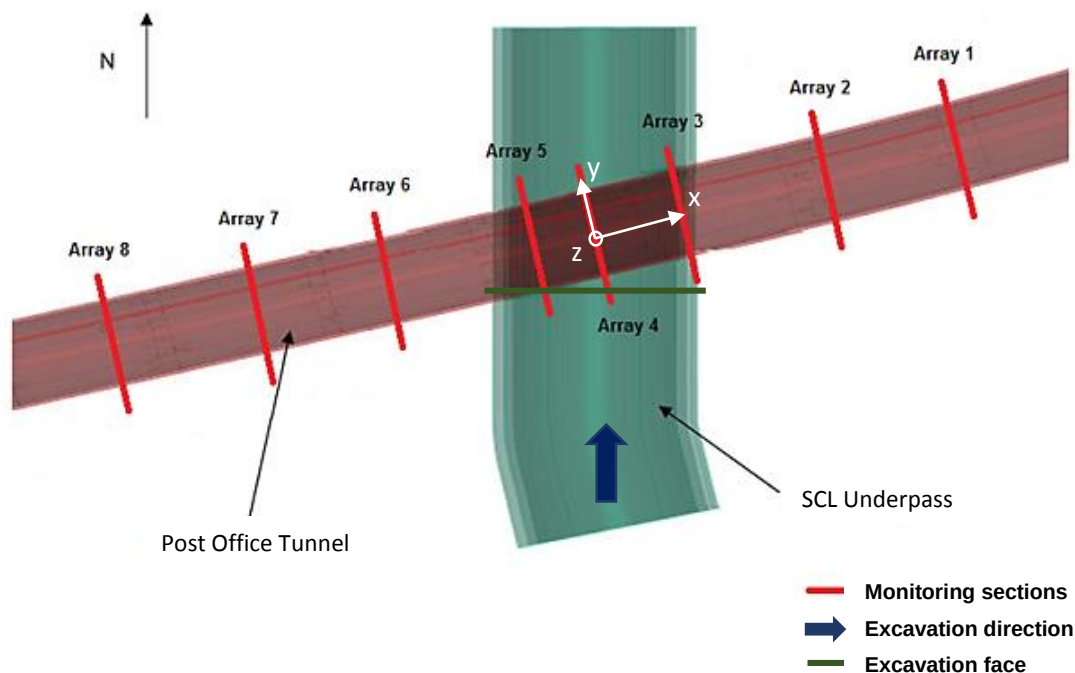


Figure 4.16 - Schematic illustration of the excavation face reaching the crossing section

Regarding the advance No. 13, which represents the under-crossing stage, it is even more noticeable that the proximity of the prisms to the excavation face is the most prominent aspect, as RP1 and RP2 were the prisms that recorded the larger settlements in that stage, which were between 20 and 23 mm. The remaining prisms recorded much lower displacements, which reveals that the displacements of the POT were quite localized when the new tunnel's excavation face reached the Post Office Tunnel location. Besides, the disparity between the results from prism RP1 and RP5 may indicate a rotation of

the tunnel. This non-uniform movement of the POT is explained by its lining properties. As already presented the POT is a cast iron tunnel formed by numerous segments, hence the presence of the joint allows the tunnel to adjust to the soil movements. The deformability of the radial joints allows the cross section of the tunnel to deform, in this case, towards the excavation face. This movement will be further explained in the end of this chapter.

In the last stage, PR1 and RP2 were still the prisms recording the greater displacements, which was around 28 mm, all remaining prisms recorded a similar settlement, approximately 25 mm, which may reveal that at that stage the displacements had already stabilized and the whole POT had already been affected by the excavation, in contrast with the situation identified in advance No. 13 where only the part of the tunnel closest to the excavation face was affected.

4.3.2.2. Horizontal displacements

Although the vertical displacements are much greater than the horizontal displacements, it is also important to understand the real horizontal movement of the POT once it allows to completely define its behaviour due to the proximity of the excavation face. In this subsection the horizontal displacement to be characterized correspond to displacements perpendicular to the POT longitudinal axis.

Due to its proximity to the new tunnel excavation works, arrays 3, 4 and 5 are the ones that better record the horizontal movements. Figure 4.17, Figure 4.18 and Figure 4.19, reproduce the lateral movement recorded throughout all the excavation process, in array 5, 4 and 3, respectively. For each array all related prisms are represented, thus the difference between the displacements recorded by each prism can be identified.

The displacements reproduced in these figures were registered based on the local axis of each prism which are in accordance with the axis first represented in Figure 4.12. Hence negative values represent movements towards the advancing tunnel, i.e. southwards.

As described for the vertical displacements, for the horizontal displacements it is also possible to identify the material shortage recorded (between the 10th and the 22th of November), followed by the passage of the face excavation beneath the POT centre-line (27th of November of 2014) which is clearly identified by a peak movement recorded by all prisms throughout all arrays.

Once it is the first to be reached by the excavation face, array 5 is the first array to be represented and analysed within this subsection (Figure 4.17). It can be seen that once again, prism RP1 recorded the greater movements since it is located at a lower level and is the first to be affected by the excavation face. As mentioned above the under-crossing is clearly identified by a peak movement registered by all prisms, which corresponds to a uniform displacement towards the excavation face (peak movement registered between 22nd and 29th of November). As the excavation face moves forwards and the advance No.13 is performed (see Table 4.2), the horizontal displacements decrease substantially in absolute value, which shows that all prisms move in the opposite direction following the progress of the excavation (movement identified between the 29th of November and the 6th of December).

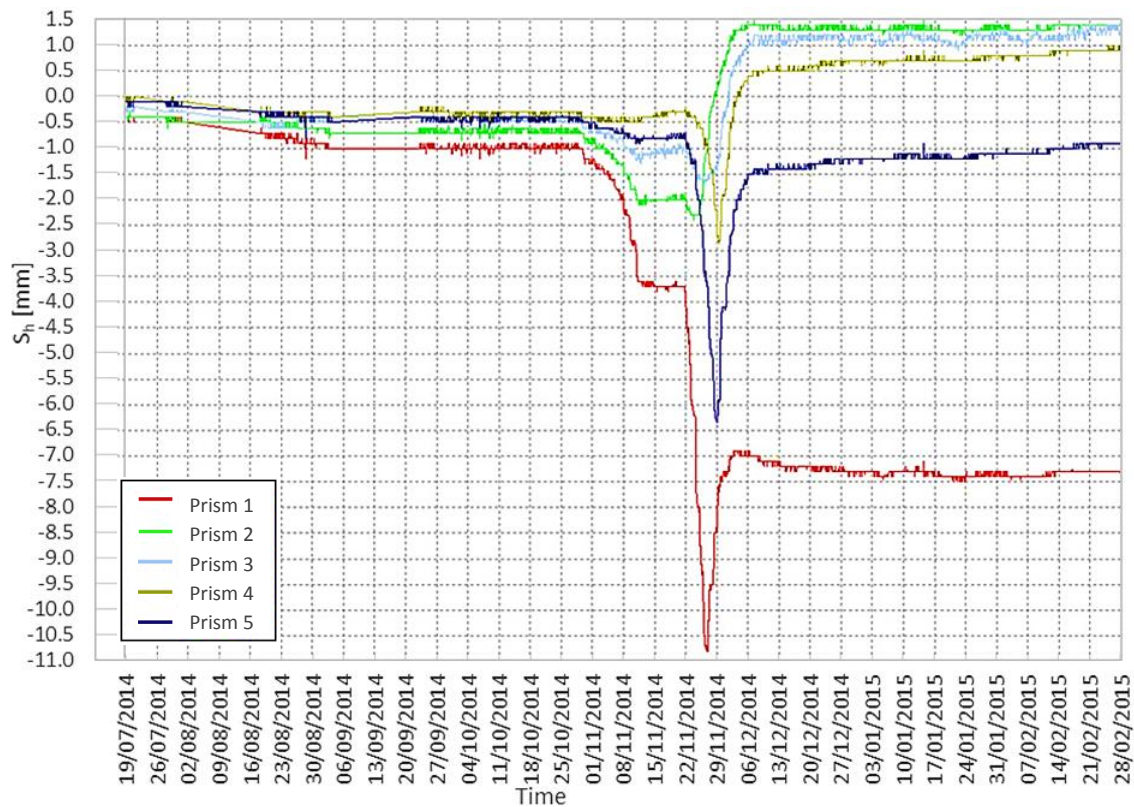


Figure 4.17 - Raw data recorded in array 5 throughout all excavation process: Horizontal displacements (CoLOR courtesy)

The displacements recorded in array 4 are much similar with the recorded in array 5 with the exception of prism RP5 that records greater displacements in this array (see Figure 4.18). In fact, the final position of this prism is approximately the initial one. This return to the initial position may be eased by the central position of this array.

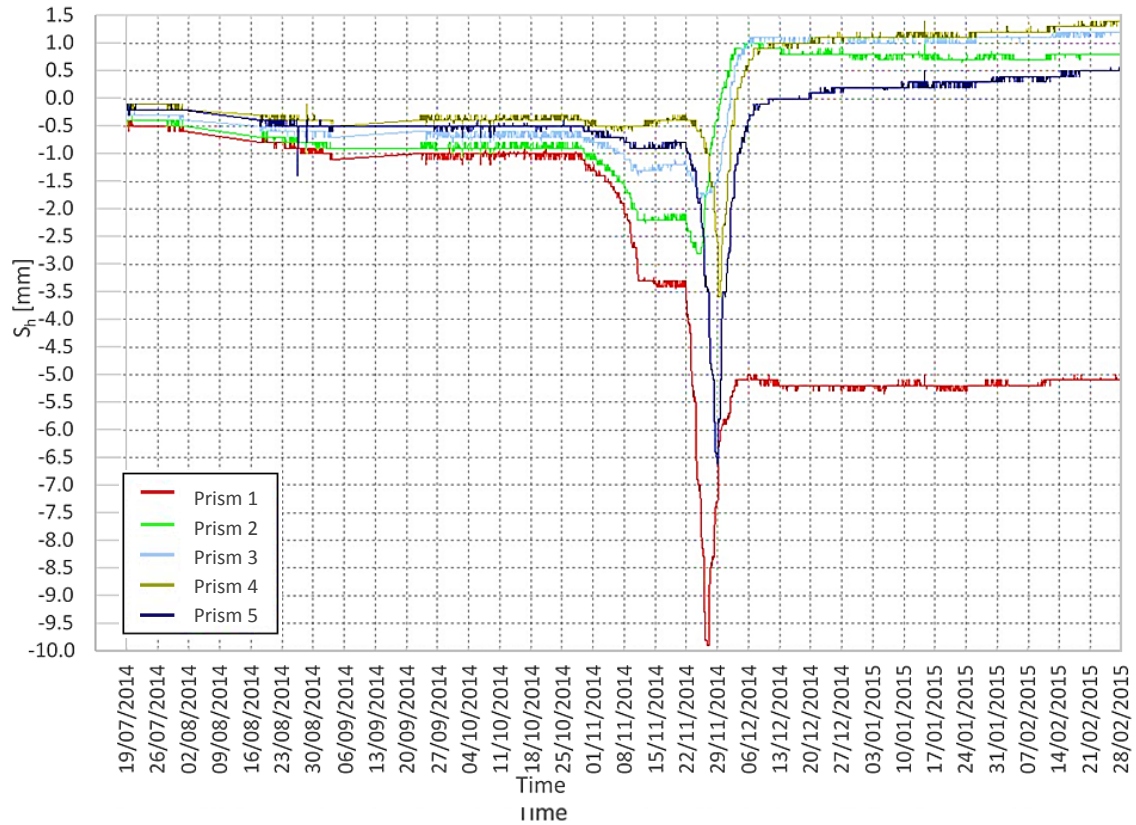


Figure 4.18 - Raw data recorded in array 4 throughout all excavation process: Horizontal displacements (CoLOR courtesy)

Regarding the displacements recorded by array 3, the peak movement due to the approximation of the excavation face is much less significant. Since this is the third array to be reached, when the excavation face starts affecting the prisms located in this arrays, the peak movement was already absorbed in the zone of the arrays 5 and 4, which means that at this stage the displacements may propagate more uniformly through the POT. However, as it can be identified in Figure 4.19, the positive horizontal displacements are greater than recorded in the other arrays. As this is the last array being “affected” by the excavation face, the movement of the POT within its area will be mainly northwards, in fact, RP5 which is the prism located further north, recorded the major displacements in this array.

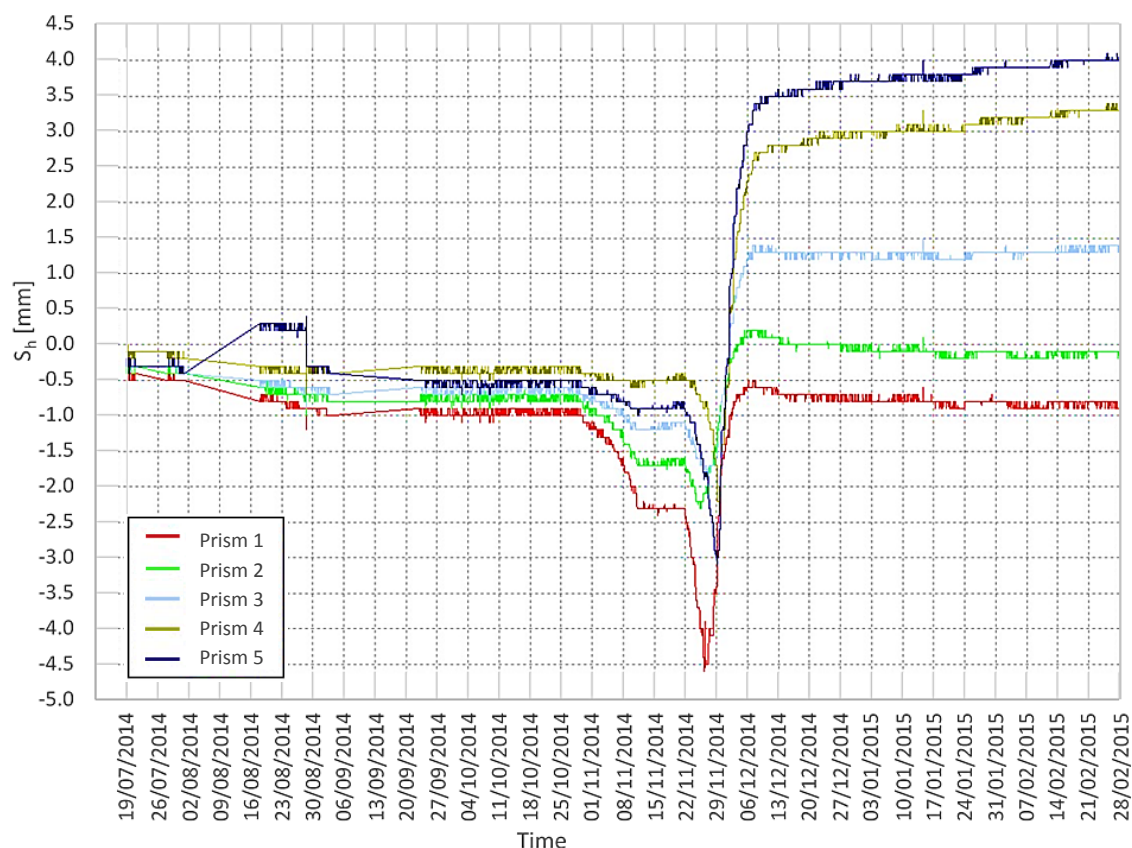


Figure 4.19 - Raw data recorded in array 3 throughout all excavation process: Horizontal displacements (CoLOR courtesy)

From the analysis of these arrays, one can conclude that, in general the POT moves towards the position of the excavation face. When the face of the excavation is south the POT, which corresponds to the time period between the start of the excavation until 27th of November 2014, the tunnel has negative displacements (see Figure 4.12 for axis reference), whereas when the excavation face is north the POT, the tunnel moves in the opposite direction although the displacements magnitude is slightly lower in this direction. Note should be taken that all plots represent cumulative displacements, thus while the maximum displacements towards south can be read directly in the plot (approximately 11 mm in array 5 – see Figure 4.17), the movement northwards has to be interpreted as it is concealed in the movement southwards, however as Figure 4.19 shows the maximum northwards displacement was also very important. The maximum absolute displacement in this direction, which was recorded by prism RP5, was approximately 7 mm.

Figure 4.20 represents the horizontal displacements recorded by each prism along the POT throughout different stages of the excavation. The excavation stages here mentioned are the same as analysed for the vertical displacements (see Figure 4.15): start of the excavation, advance No. 1, advance No.13 and advance No. 23.

The displacements reproduced in the following figure resume the movement characterized previously through the analysis of Figure 4.17, Figure 4.18 and Figure 4.19.

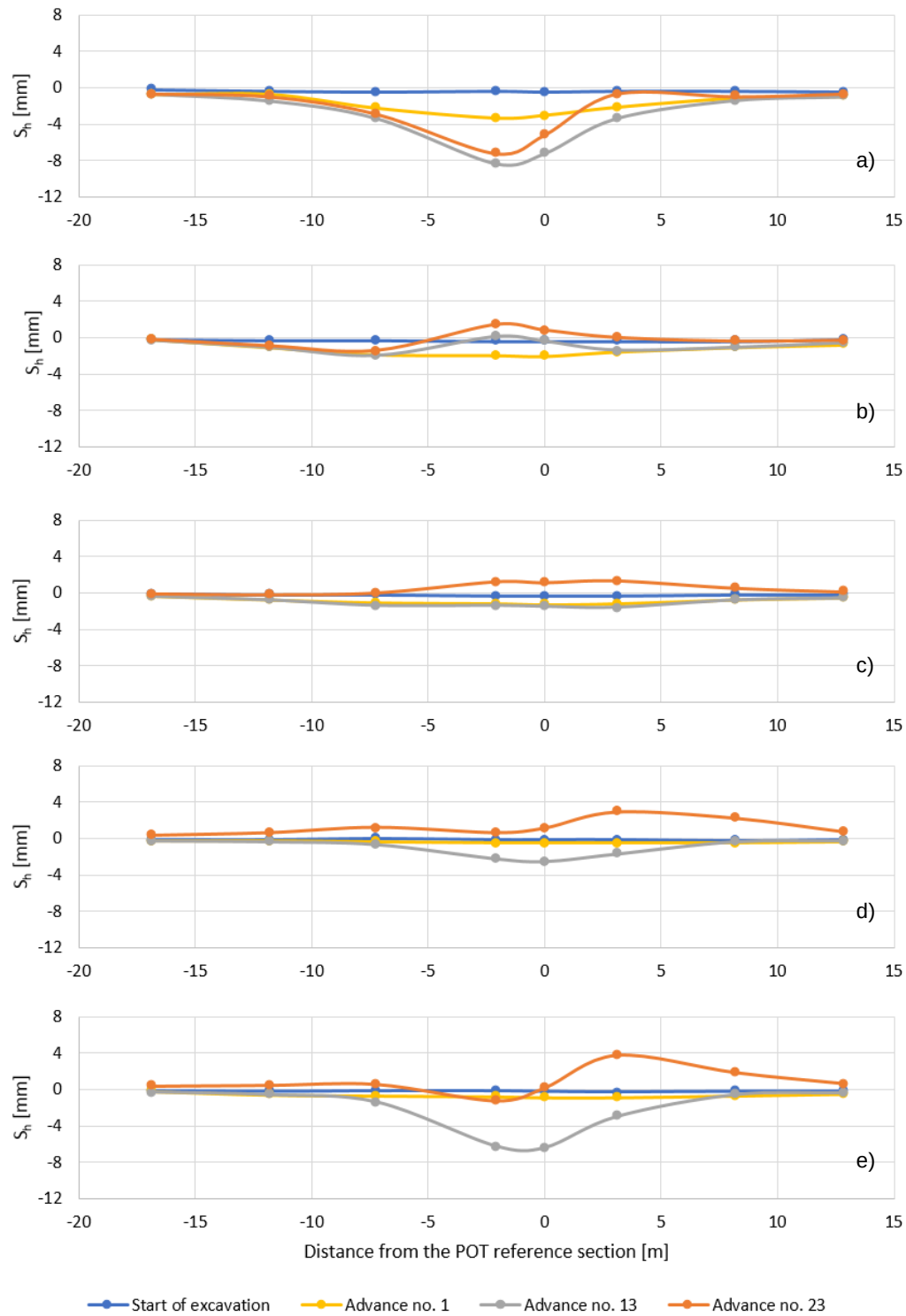


Figure 4.20 - Horizontal displacements along POT; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5

As previously stated, RP1 recorded the major horizontal displacements southwards, however as the excavation moves forward these displacements decrease, which results from the movement of the POT in the opposite direction. According to the discussed from Figure 4.17, the plot represented in Figure 4.20 a) shows that the displacements northwards are more significant in array 3 since the negative

displacements are much lower in this section. The prism RP5 recorded displacements similar to those registered by RP1 until the advance No.13 was performed, nevertheless after that advance RP5 recorded greater displacements than RP1. As already introduced this prism is the one located further north, hence it is the prism more affected by the lining flexibility when the excavation face in north the POT longitudinal axis.

The remaining prisms recorded much lower displacements which is related with its relative position. Once RP2, RP3 and RP4 are located in an upper level, they are still more restrained as they are more distant from the excavation of the SCL tunnel. Besides, the radial joints provide a certain flexibility within the cross section, which may allow the tunnel into a skew ovaloid.

4.3.2.3. Longitudinal displacements

The longitudinal displacements recorded by the different prisms throughout all arrays are represented within Figure 4.21 with respect to advancing tunnelling works as per Figure 4.15. As it can be seen the maximum longitudinal movements were occurring about the POT crown, which were recorded by prism RP3. This movement is comprised of soil horizontal action, however the discrepancy in the values recorded in different prisms induce that the longitudinal displacements are due to structural behaviour, otherwise all prisms located in the same array would have similar displacements. This discrepancy is more intense in array 6, where the longitudinal displacements in the final stage of the excavation (advance No. 23) range from 3 mm recorded by prism RP3 and 0.7 mm recorded by RP1.

The smallest movements were, in fact, recorded by the prisms located in a lower level, hence it may infer that the movements of the POT is founded about an axis of rotation near the POT invert. Not that the longitudinal displacement recorded in the base of the POT may be restricted by the concrete invert in the base of the POT, which leads to lower displacements in the prisms located at a lower level.

It is worth referring that the longitudinal displacements are not symmetric in relation to the POT reference section, which would be expected since this is the section located on the vertical axis of the new tunnel. However, as already explained, array 5, which is approximately -2 m from the POT reference section, is the first array to be reached by the excavation, thus it is expected to have greater displacements than, for example, the approximately symmetric array, which is array 3. Besides, the 15° deviation of the POT to the perpendicular to the SCL tunnel, induces that all arrays located in the negative part of the axis presented may experience greater longitudinal displacements.

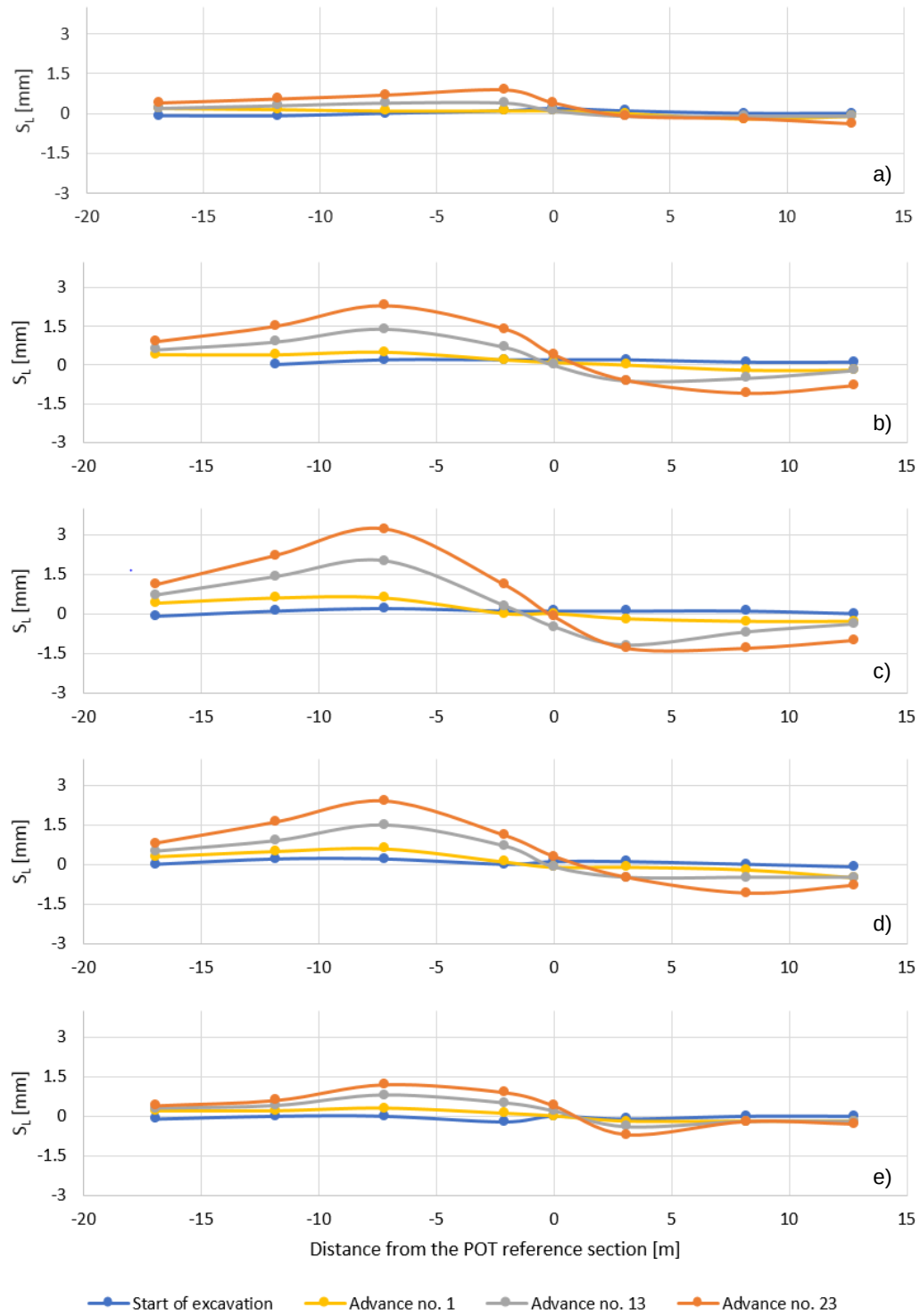


Figure 4.21 - Longitudinal displacements along POT; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5

4.3.2.4. Radial movement

After the interpretation of the displacements along the three-perpendicular directions, the analysis will now be focused on the convergence/divergence of the POT section, which is highly influenced by the

segment joints within each ring. In order to develop this study a chord reference system was developed, which is represented in Figure 4.22, therefore the length variation of those chords throughout all excavation process about different arrays will be presented. The information in the plots presented in this section was developed based on the raw data provided between the 19th of July of 2014 and the 28th of February of 2015 as per Figure 4.14.

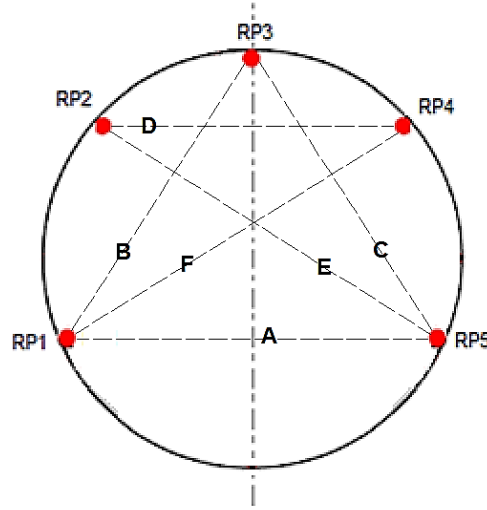


Figure 4.22 - Convergence chord reference system

The initial length of the chords was determined based on the exact position of each prism before the tunnelling works begun, thereafter the length variation, hereafter represented by Δ , was calculated considering the initial length of each chord and compared with the chord length determined through the coordinates of each prism after movement. The following equations clarify the determination of the diametric displacements, the length of the chords, defined as *Chord*, was calculated as represented in Equation 4.1:

$$Chord_a = \sqrt{(y_{i+1} - y_i)^2 + (z_{i+1} - z_i)^2} \quad (4.1)$$

where a represents the chord letter, whereas i and $i + 1$ correspond to the prisms related with each chord as represented in Figure 4.22.

In order to clarify the diametric displacements, the following equation was used to verify if the prisms of each chord were converging or diverging.

$$\Delta = Chord_a - Chord_{a,initial} \quad (4.2)$$

where $Chord_{a,initial}$ corresponds to the initial length of each chord, and $Chord_a$ the chord length after movement. Therefore if the length variation, Δ , is negative it means that the prisms converged, on the other hand, if Δ is positive the correspondent prisms diverged.

The following plots represent the diametric displacement of the POT in arrays 1, 3 and 5, these arrays were chosen as a reference since they represent the general behaviour of the POT. The remaining arrays registered a similar behaviour to one of these three, which will be further clarified in the following paragraphs.

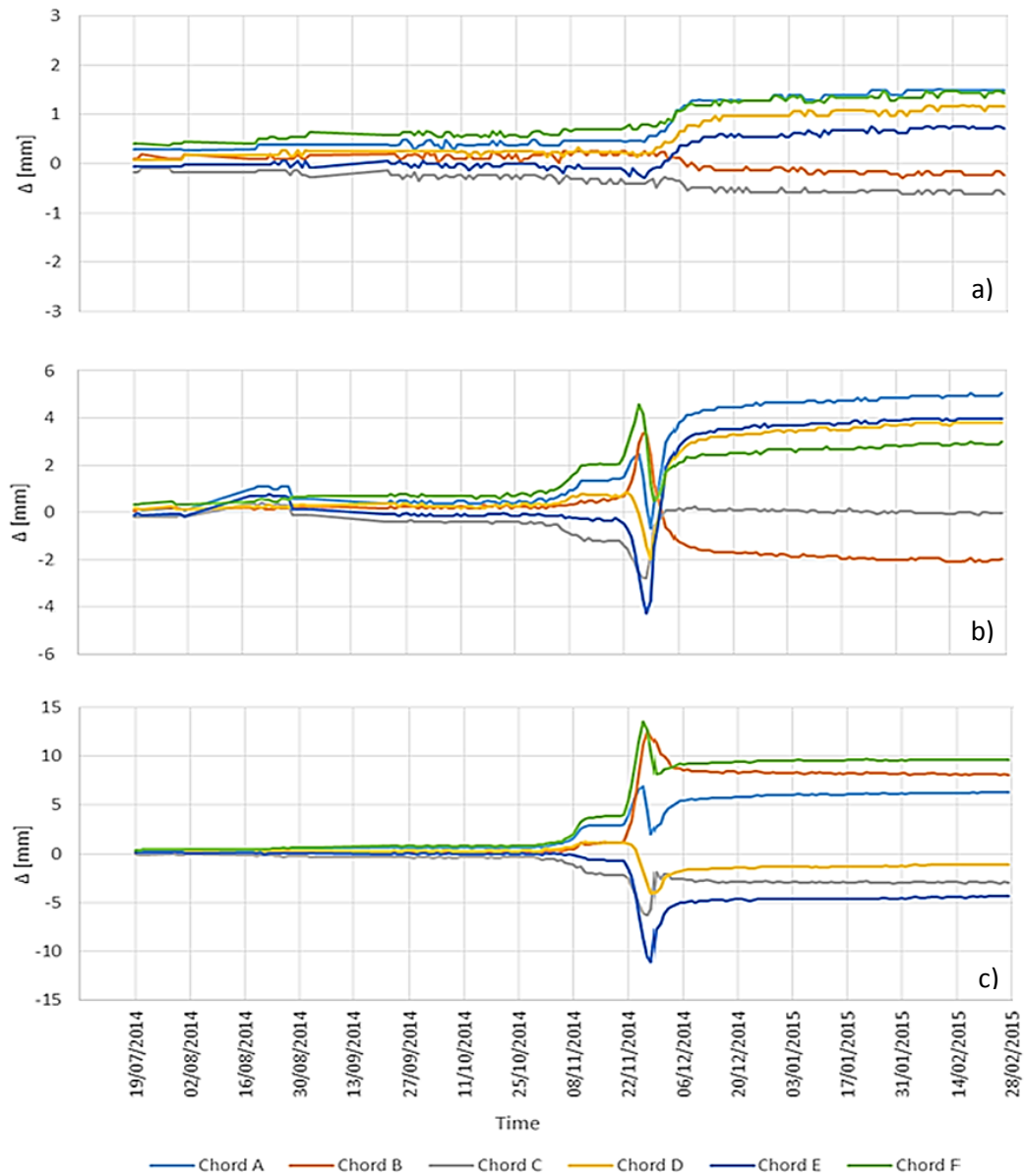


Figure 4.23 - Diametric displacement throughout all excavation process; a) Array 1; b) Array 3; c) Array 5

As Figure 4.23 b) and c) suggest the maximum convergence occurred approximately when the tunnelling face is beneath the POT, which corresponds to the 28th of November of 2014, besides it is possible to identify that the maximum displacement occurred within array 5 (Figure 4.23 c)) as a divergence of approximately 13.5 mm registered in Chord F.

In Figure 4.23 a), which corresponds to the diametric displacements registered in array 1, the tunnelling progress it is not as clear, with the maximum movement being only 1 mm of divergence, registered in

chord A. In fact this array registered small displacements due to its distance to the SCL tunnel excavation, arrays 7 and 8 recorded similar values since they are also distant from the excavation. The displacements registered in array 2, followed the same tendency as the previous although they were slightly greater (approximately 2 mm).

In array 3 (Figure 4.23 b)) the scenario is different with all chords now registering either divergence or convergence in a considerable level. It can be identified that with the excavation face approaching the POT centre-line, chords A, B and F registered divergence, whereas the remaining chords registered convergence. This record clearly identifies the major movement of prism RP1 while others remain in the same position. In fact this movement was already identified in the previous sections as RP1 is the first prism to record any displacement. However, after the undercrossing is performed the length variation of the chords alter with only chord B registering convergence after it. This modification shows that, after the undercrossing RP5 also moved significantly as chord E registered approximately 4 mm in the end of the excavation. The author identified that array 6 recorded similar displacements.

The diametrical displacements recorded within array 5 were similar to those recorded in array 3, however the major movement of RP1 is even clearer, since A, B and F only registered divergence throughout all excavation process, whereas C, D and E registered convergence.

In conclusion, one may notice that the POT in general registered an elongation of its cross-section towards the excavation. The following figure represents the predicted position for the POT cross-section when the advance No.13 was performed and in the end of the excavation (see Figure 4.8 for excavation advances interpretation). This representation was developed based on the real prisms position within array 4; note that the prisms displacement were scaled up 20 times.

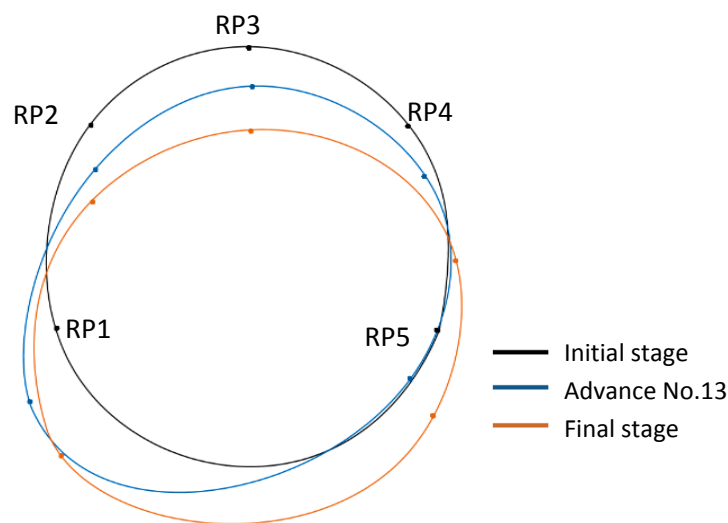


Figure 4.24 - Schematic representation of the deformed cross-section of the POT as recorded in array 4 (looking West)

According with the presented above, Figure 4.24 allows one to identify that when the tunnelling face reaches the POT centre-line, which corresponds to advance No. 13, prism RP1 moves towards the excavation elongating the section. Part of this movement is then recovered with the advance of the excavation towards the opposite side. Simultaneously, prism RP5 starts recording movement in the opposite direction, leading to a slightly more circular deformed.

5

TWO-DIMENSIONAL NUMERICAL ANALYSIS

5.1. INTRODUCTION

With the recent advances in the computational power and processing speed of desktop computers and the development of flexible and powerful numerical analysis codes, the designer has now several techniques available. However, the users of such tools should be aware that numerical and modelling errors are inevitable when simulating geotechnical problems.

The present chapter presents the analysis of the case study made both in Xdisp and in PLAXIS 2D. For both analyses, the author provides a description of the modelling process in each program and a discussion of the obtained results. Note that Xdisp does not correspond to a numerical method, however its inclusion in this chapter provides a preliminary result of the expected movements which then allow a better understanding of the POT movement and the importance of the various approaches.

5.2. XDISP ANALYSIS / PRELIMINARY RESULTS

In order to achieve a reliable prediction of ground movements, a preliminary assessment was carried out with the use of Xdisp. Xdisp is a simple software that calculates ground movements induced by tunnelling in terms of three dimensional displacements, besides it allows to subsequently assess buildings and utilities potential damages.

The equations used are based on the normal probability (Gaussian) distribution theory, the user is required to define the estimated volume loss, above the tunnel due to deformation, V_L . Xdisp will then use this to define the settlement profile at the surface or at a specified depth. Several methods of solution are available to create the profile of settlement above the tunnels.

A tunnel is taken as a cylindrical excavation in soil, hence the geometry of the new undercrossing tunnel had to be simplified in order to be modelled in this software. Instead of using the real cross sections, Figure 4.7, the author assumed two circular sections, the largest with a diameter of 6.5 m and the smallest with a diameter of 5 m. The transition sections, which correspond to the sections where the diameter decreases from 6.5 m to 5 m or vice-versa, were divided into small sections, each with a progressively smaller or larger diameter, respectively.

Since the Post Office Tunnel was already installed in place before the excavation of the Underpass, it was modelled in Xdisp as a utility, however no assessment was performed since the author had not the properties required for this purpose.

The calculated Xdisp displacements do not take the stiffness of the Post Office Tunnel into account, hence the preliminary results are conservative displacement values. In fact, this program is usually used to estimate the worst-case settlement and lateral movements.

5.2.1. ANALYSIS METHODS

The geometry of the settlement trough provided by Xdisp is uniquely defined by specifying a value for the volume loss and the width of the trough relative to the depth of the tunnel, which is identified in Xdisp as trough width parameter, k . Displacements due to SCL tunnelling works undertaken to LUL within London Clay are usually based on a ground volume loss of 1.5% and a user-specified k value of 0.5. These values had been adopted from consideration of monitoring case history data throughout the works undertaken through London Clay and will be adopted in this analysis.

The complete 3D form of a tunnelling induced settlement trough provided by Xdisp was based on the equations proposed by Attewell and Woodman (1982), which are used in terms of a specific function obtained from the numerical integration of the normal probability function. Those equations are applicable to surface and subsurface movements. In combination with these equations a specific k derivation method had to be applied because unlike other methods, to set the three-dimensional form of movement available in Xdisp, this does not define its own k derivation method. Note that k derivation method refers to the method used to estimate the trough width parameter.

The width of the settlement trough perpendicular to the tunnel axis is defined in Xdisp in terms of distance from the tunnel centre-line to the inflection point of the curve, i . Although Xdisp provides a range of options to calculate this parameter, the author adopted the *user-specified k* as the k derivation method, which correlates the mentioned distance with k through the equation presented below.

$$k = \frac{i}{z} \quad (5.1)$$

where z corresponds to the axis level of the tunnel to the ground surface in metres.

When subsurface displacement calculation is required, the user has another range of methods to choose from in order to calculate displacements below the surface level. Since the *user-specified k* method was the one adopted to perform the calculation of the displacements at the surface, the author adopted the only method that allowed one to specify the k value instead of estimating it, which was the *New and Bowers (1994)* method. In fact, it is known that this method can also be used to calculate surface displacements as it was validated through the Heathrow Express Trial Tunnel, however Xdisp does not provide this option.

5.2.2. XDISP MODEL

As already mentioned the SCL tunnel excavation was modelled as a cylindrical excavation in soil, whereas the POT was modelled as a utility. The utility dimensions used in this model were those presented in section 4.1.2. for the POT, therefore the internal diameter and the wall thickness were assumed 2933 mm and 19 mm respectively. Note that the wall thickness was assumed as the thickness of the section's web, which is conservative. Since no utility damage assessment was carried out, these were the only input data concerning POT modelling.

Figure 5.1 and Figure 5.2 present the model developed in Xdisp. The length of both tunnels was adopted based on an iterative process assuring that the displacements above the undercrossing section were the most accurate possible and no residual displacements were found on the POT. This led to a length of approximately 124.2 m for the utility and 70 m for the SCL tunnel. The dimension of the crossing section was slightly simplified in order to create the cylindrical sections throughout all the SCL tunnel.

The POT was modelled in place with a deviation angle of 15° from the perpendicular of the SCL tunnel axis, as represented in the drawings provided in the last chapter. The global x , y and z coordinates used in Xdisp interface are aligned with the SCL tunnel excavation, thus x is parallel to the tunnel axis, y is perpendicular to the tunnel and z represents the depth of the tunnel, which is in accordance with the specified in 4.2.1.1, where the ground level is assumed located at $z = 123\text{m}$.

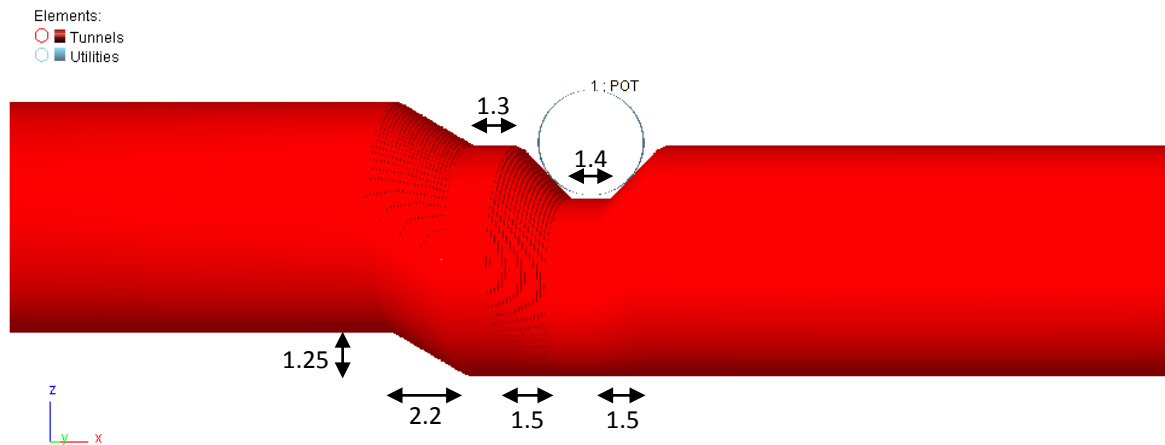


Figure 5.1 - 2D view of the undercrossing section geometry (dimensions in meters)

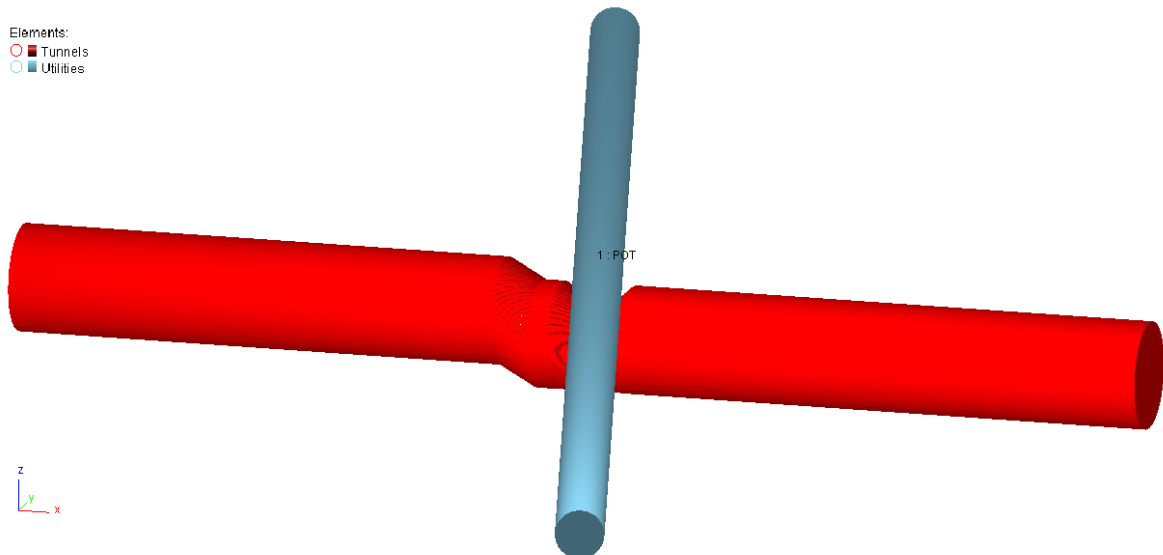


Figure 5.2 - 3D model view

5.2.3. RESULTS

5.2.3.1. Surface displacements

Xdisp provides a feature that allows to represent the displacements through contours in a plan view. The following images were composed through that feature and represent the vertical and the horizontal displacements at the ground surface (Figure 5.3 and Figure 5.4, respectively).

In these plots the SCL tunnel is represented in green whilst the POT as a utility through a blue line representing its longitudinal axis. The interval between the contours which represent the vertical displacement is 2.5 mm, reaching a maximum of 12.5 mm contour. For the horizontal displacements the interval is smaller, 1 mm, and the maximum contour value is 4 mm, located laterally to the SCL tunnel.

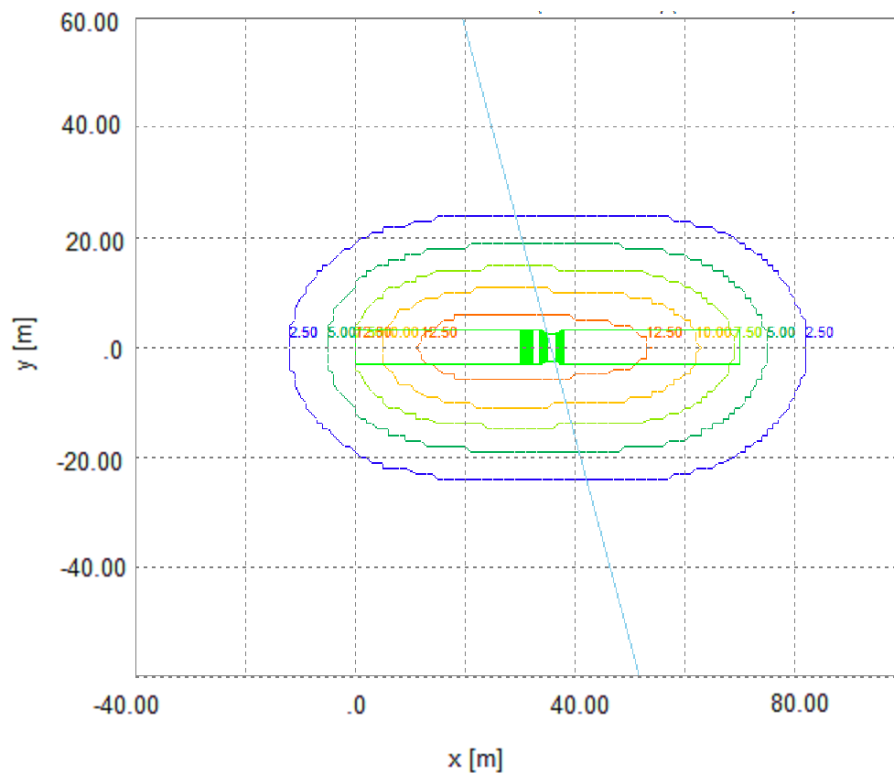


Figure 5.3 - Vertical displacement contours at the surface level

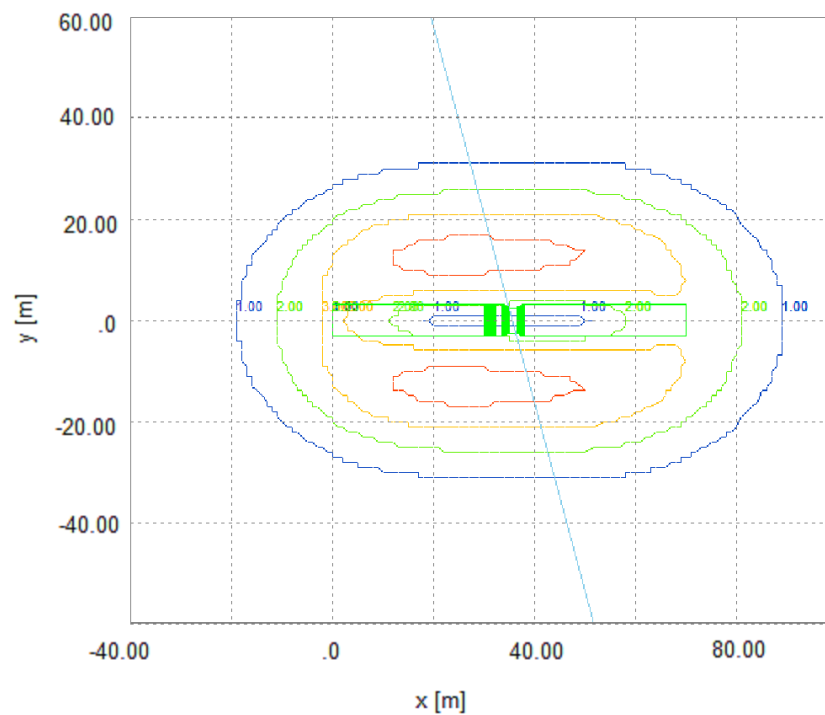


Figure 5.4 - Horizontal displacement contours at the surface level

The displacements represented in the previous figures can also be displayed by a deformed grid (Figure 5.5), which represents the ground surface. It was identified that the maximum vertical displacements at the ground surface was 14.35 mm, whilst the maximum horizontal displacement was 4.33 mm towards the excavation.

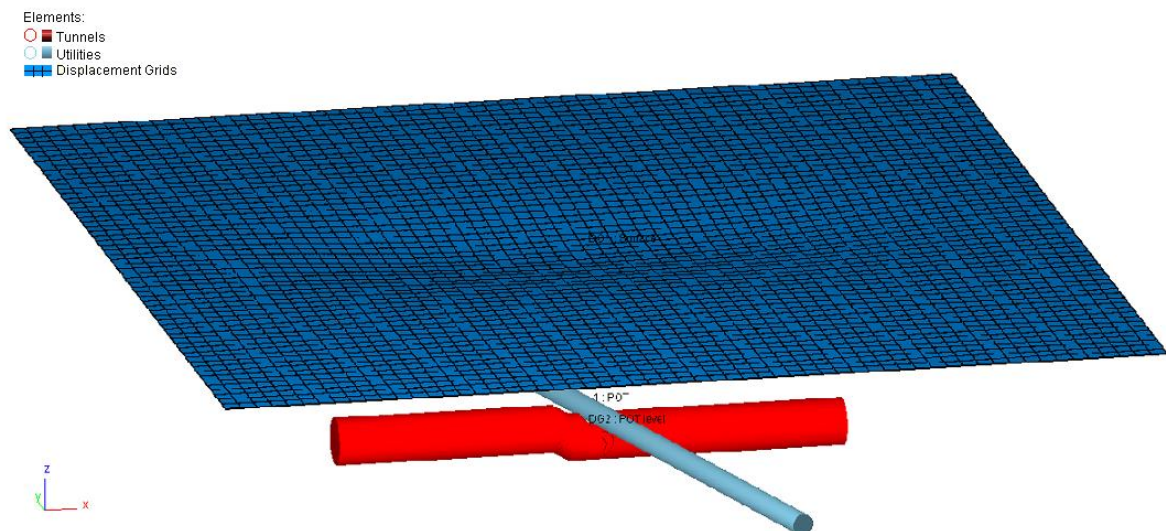


Figure 5.5 - Deformed grid at surface level

The plot presented in Figure 5.6 represents the ground surface displacements, both vertical and horizontal, through a line aligned with the POT longitudinal axis, consequently the ordinate is defined as a distance from the edge of the POT identified by the coordinates (19.60, 60,00), see Figure 5.3.

Since the total length of the POT is 124.2 m, the POT reference section is located at 62.1 m from the axis origin, which is expected to correspond to the section with the greater vertical displacements. It can be identified that the width of the settlement trough is quite extensive which denotes that the displacements are widely spread towards the surface. It should be noted that due to the simplicity of the software the soil had to be modelled as a unique layer, hence its properties were homogenised and only reproduced by the two above mentioned parameters. This considerable simplification of the soil properties may have led to a wider settlement trough than in the reality.

Although the plot represents the horizontal displacements in both x - and y -direction, it does not correspond exactly to the perpendicular and longitudinal displacements of the POT since this is not perfectly perpendicular to the SCL tunnel. However, as there is a small deviation, the POT perpendicular displacements can be approximate to horizontal displacements in x -direction, whilst the longitudinal displacements of the tunnel can be approximate to the horizontal displacements in y -direction. Hence it can be assumed that the longitudinal displacements in the surface along the POT longitudinal axis reached a maximum of 4 mm moving towards the SCL tunnel centre-line, while the perpendicular were almost insignificant.

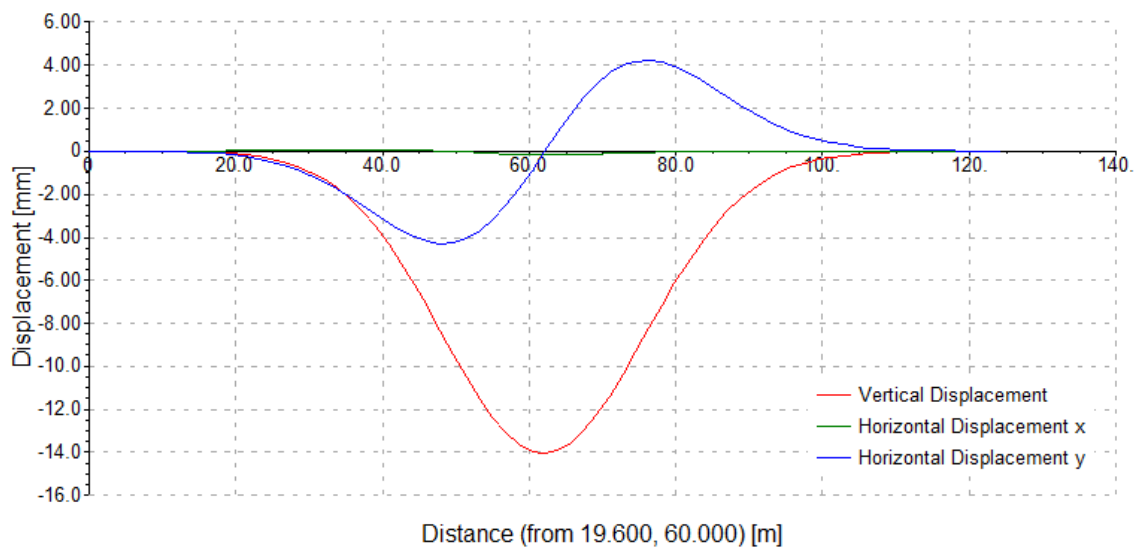


Figure 5.6 - Ground surface displacements in the POT axis alignment

5.2.3.2. POT displacements

Since the POT stiffness is not taken into account, the displacements presented in the following plots are “greenfield” displacements, thus they correspond to conservative results, i.e., the obtained values may be slightly larger than expected in the reality. Similarly to what was mentioned about the horizontal displacements on the plot represented in Figure 5.6, the same simplification was endorsed in this section, i.e., the POT perpendicular displacements are approximate to horizontal displacements in x -direction, and the longitudinal displacements are approximate to the horizontal displacements in y -direction.

Regarding the movement of the POT crown along its length (Figure 5.7) the maximum vertical displacement at this level was 27.9 mm, whereas the maximum POT perpendicular displacement was approximately 8.0 mm. In fact, the vertical displacements recorded by the monitoring prisms located at the crown of the POT are very similar to those obtained in Xdisp, with just an error of 10%. Although the perpendicular displacements are in the same magnitude from the two sources, the scenario regarding the longitudinal displacements is different with Xdisp returning results more than twice the recorded values. This discrepancy may be explained by the fact that the POT longitudinal stiffness is ignored.

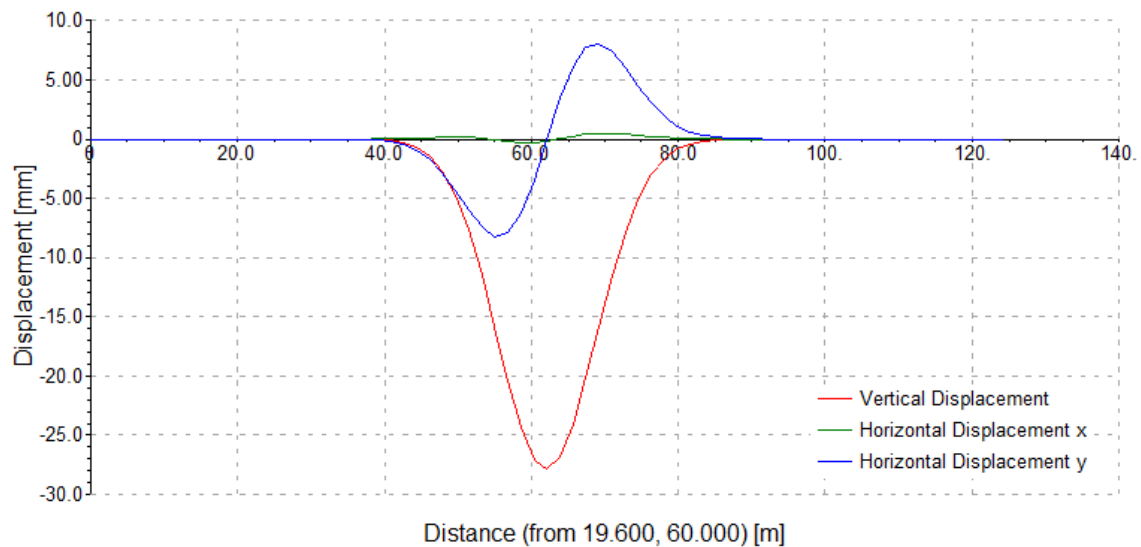


Figure 5.7 - Displacements in the POT crown

Figure 5.8 represents the displacements expected in the POT invert, however this cannot be directly compared with the monitoring data since there were not prisms at a so lower level. Despite this, the values presented in this plot are very conservative, and once more explained by the fact that the tunnel stiffness was not considered.

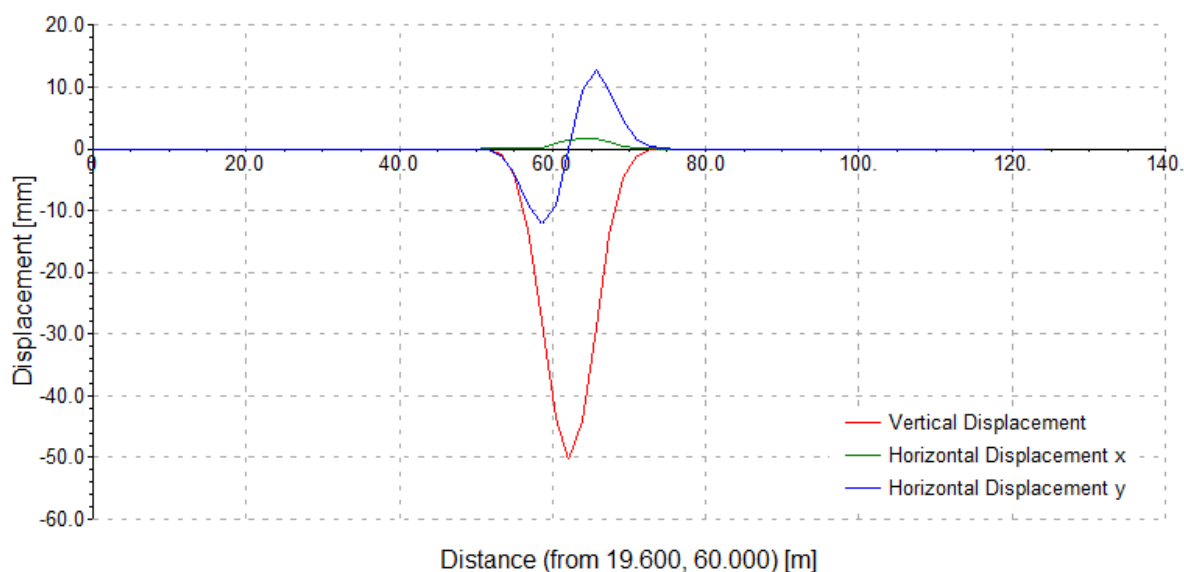


Figure 5.8 - Displacements in the POT invert

5.3. PLAXIS 2D MODEL

PLAXIS is a finite elements program that has been specifically developed since 1987 for geotechnical applications. The software was developed for the analysis of deformation, stability and groundwater flow in geotechnical engineering in which soil models are used to simulate the ground behaviour.

Three different type of calculations are available within this software, namely Plastic, Consolidation and Safety analysis. Since the project developed in this thesis required mainly information regarding ground movements, which in turn is engaged with a stress-strain condition, the calculation type adopted for this project was Plastic.

The software provides a large range of constitutive material models, which allows the mechanical behaviour of the soil to be modelled at various degrees of accuracy. The user must have a good understanding of the soil models and its parameters in order to reproduce the soil behaviour the most accurate possible for each situation.

It is then clear that the accuracy of the results obtained with this program depends on the user's level of expertise, however in most of the situations, PLAXIS grants a considerable level of accuracy which makes it a desired program in the geotechnical field. It provides a tool for practical analysis which presents the user with wide information regarding displacements, stresses and strain in soil and structural elements.

Regarding PLAXIS 2D, it is a special purpose two-dimensional finite element program which simulates either plane strain or axisymmetric conditions to model real situations. This subchapter presents the simplified two-dimensional modelling of a cross section of the SCL tunnel excavation.

5.3.1. MODEL FEATURES

Since this project had a very complex geometry an initial analysis was carried out with the use of a plane strain condition in order to simplify the assessment. A plane strain model is used in two-dimensional

finite element analysis for geometries with uniform cross sections and corresponding stress state and loading scheme over a certain length perpendicular to the cross section (PLAXIS, 2015b).

Since a two-dimensional finite element analysis assumes the same loading scheme and therefore the same geometry for a certain length perpendicular to the represented cross section, it is not possible to include in the same model both Post Office Tunnel and SCL tunnel since they are not parallel. Hence the 2D model only allows the cross-section of the new tunnel to be modelled. Naturally the cross section to be adopted was the POTU section as shown in Figure 4.7 since the interest of this model is to understand the POT behaviour during the construction of the Underpass.

5.3.1.1. Constitutive material models and parameters

The soil is one of the most complex materials and the process of selecting a constitutive model to simulate its behaviour is probably the most important part of the numerical modelling. Geotechnical applications require mighty constitutive models in order to reproduce the non-linear, time-dependent and anisotropic behaviour of the soils. Since soil is a multi-phase material additional features have to be incorporated to deal with pore water pressures.

Regarding geotechnical deformation problems the behaviour of the soil is formulated based on a mathematical framework that represents a relationship between stress and strain, in fact the material models are often expressed in a form that relates infinitesimal increments of stress with infinitesimal increments of strain. Over the years many constitutive models were developed for modelling different types of soil and nowadays one can find from simple linear elastic perfectly-plastic to complex non-linear anisotropic models.

It is known that elasticity theory, which is based on the assumption that no residual strains remain after loading and unloading a material, cannot capture the non-linear and irreversible behaviour of soils.

The Mohr-Coulomb model available in PLAXIS, is a simple and well-known linear elastic perfectly-plastic model often used to model all types of soils. When loaded, the soil suffers elastic deformations controlled by its stiffness, E , and the Poisson's ratio, ν , which means that the linear elastic part of the Mohr-Coulomb model is based on Hooke's law of isotropic elasticity (PLAXIS, 2015a). When the soil reaches the yielding point its behaviour becomes perfectly plastic, which in turn is commanded by the dilation angle, ψ . The perfectly plastic part of this model is then based on the Mohr-Coulomb failure criterion which is formulated in a non-associated plasticity framework. This framework executes the calculation of plastic strains using a plastic potential function different from the yield function which leads to some limitations since the uniqueness of the elastoplastic solutions cannot be proven (Brinkgreve, 2013).

The Mohr's Coulomb failure criterion is represented by the tangent to the failure circles performed with different initial effective stresses, this line, assumed to be straight can be expressed as represented in Equation 5.2.

$$\tau_f = c' + \sigma'_{nf} \cdot \tan \phi' \quad (5.2)$$

where τ_f and σ'_{nf} represent the shear and normal effective stresses on the failure plane, c' the effective cohesion and ϕ' the angle of shearing resistance of the material.

When a total stress analysis is required due to the undrained soil behaviour, PLAXIS simply assumes the cohesion parameter equal to the undrained shear strength, c_u , in combination with $\phi' = 0$, which means that the Mohr's Coulomb failure criterion reduces to the well-known Tresca criterion (see Equation 5.3).

$$\tau_f = c_u \quad (5.3)$$

The following figure represents the meaning of the Coulomb's envelope for each approach.

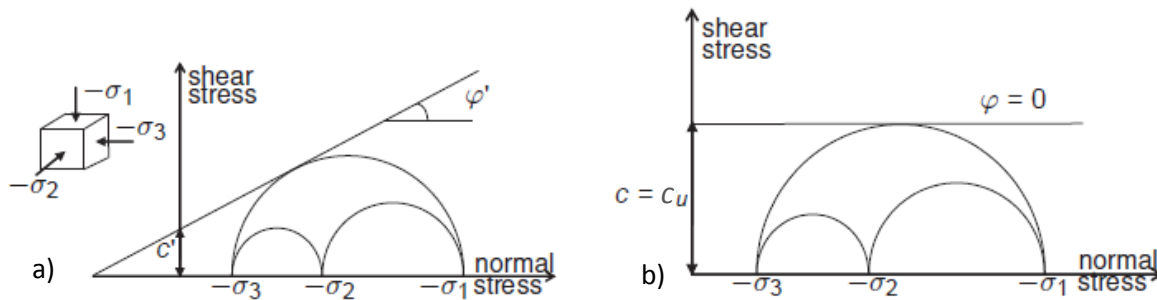


Figure 5.9 - Stress circles at yield; a) Mohr-Coulomb failure criterion; b) Tresca failure criterion (reproduced from PLAXIS, 2015a)

Although some authors assume a drained analysis more suitable to model an excavation, for London Clay this procedure is not the most appropriate. Since the London Clay is an overconsolidated clay with low permeability, an undrained analysis is the better approach to obtain accurate results.

There are three methods available in PLAXIS to perform undrained analysis, usually known as A, B or C. The decision for one of the three methods is based on the main aim that each analysis has. The method C consists on a total stress analysis which means that no pore pressures are calculated in the analysis. For the purpose of this thesis this may be a good approach since consolidation is not modelled, however it requires undrained values of stiffness and strength to be inputted into the model. In the previous chapter the reader had the chance to consult the design parameters available for this numerical analysis and as it is possible to identify only drained values of stiffness were provided, hence method A or B may be more appropriate.

The main difference between method A and B relies in the strength parameters. Whilst in method B undrained shear strength is directly inputted into the model, in method A this parameter is estimated based on drained strength and stiffness values, thus the last may overestimate the undrained shear strength. Since the undrained shear strength values were already available for each layer to be modelled, the author adopted the method B as the most appropriate for this analysis.

For the London Clay Formation and for the Lambeth Group, as the soil behaviour was modelled as *undrained* comprising the method B, the only strength parameter inputted was the undrained shear strength as presented in Table 4.3 from the previous chapter. As previously referred, the plastic part of Mohr-Coulomb's model is commanded by the dilation angle, ψ , which needs to be specified for drained analysis. Typically $\psi = \phi' - 30^\circ$ is a good approximation to apply in sands, but for undrained stress analysis use $\psi = 0^\circ$ (Lees, 2012), which is what PLAXIS does automatically.

The first two layers forming the Superficial Material, present high permeability compared with London Clay or the Lambeth Group, in order to accurately model this two soil layers the soil behaviour was assumed now as *drained*. As mentioned in the previous paragraph the dilation angle has an important role for drained materials, however since this parameter was not provided in the GIR and considering that the global behaviour of the soil is undrained, the dilation angle is assumed for this layers as for an undrained material, $\psi = 0^\circ$.

Regarding the remaining strength parameters to input in PLAXIS to model the drained materials, c' and ϕ' , only the effective cohesion suffered some simplifications. Although the first two layers were reported with null cohesion, this parameter was considered equal to 1 *kPa* in order to improve numerical stability. The angle of shearing resistance was kept as reported.

Although its limitations and simplicity, Mohr-Coulomb's model is still the best approach for this case study when considering the information available regarding the. This statement is based on the company's design experience.

Regarding the concrete lining of the SCL tunnel, it was modelled in PLAXIS as a plate, which is a structural object used to model slender structures in the ground with a significant flexural rigidity and a normal stiffness. In addition to the plate, an interface was added for a proper modelling of soil-structure interaction. PLAXIS Reference Manual (PLAXIS, 2015b) refers interfaces as joint elements that “may be used to simulate, for example, the thin zone of intensely shearing material at the contact between a plate and the surrounding soil”, which is fundamental to accurate the results between the two elements.

The concrete lining was modelled as a linear elastic material which is based on Hooke's law of isotropic elasticity. The author had not access to the concrete properties that were used on site, thus based on the company experience the properties adopted for the primary lining's concrete are presented in Table 5.1. The concrete assumed was a C20/25 and the thickness of the lining was obtained through the provided drawings, 250 mm.

Table 5.1 - Concrete lining properties introduced in PLAXIS 2D

Parameter	Unit	Concrete lining
Material type	-	Elastic, Isotropic
Normal stiffness, EA	kN/m	7.5×10^6
Flexural rigidity, EI	kNm^2/m	3.906×10^4
Weight, w	$kN/m/m$	25
Poisson's ratio, ν	-	0.2

5.3.1.2. Geometry and boundaries

The creation of a geometrical model requires the selection of arbitrary boundaries to limit the model size and consequently the calculation times. It should be mentioned that this is the first simplification applied to the model once the reality does not involve boundaries, however several authors have given recommendations to select the model boundaries that does not affect the main variables and phenomena to be analysed.

Whenever a “natural” boundary is identified, which may appear in the form of a bedrock layer, it should be taken as the bottom boundary of the numerical model. The ground model as identified in the previous chapter does not contain any bedrock layer thus other criteria should be adopted.

According to Meissner (2002), the distance between the structure to be analysed and the model vertical boundaries should be at least 2 or 3 times the characteristic length of the structure, which in this case is the tunnel diameter. It is considered that the factor 2 may work for ultimate limit state conditions and for analysis of structural forces whereas the factor 3 is required for deformation analysis. This concept is also frequently applied to define the depth of the bottom boundary.

Although these recommendations had been followed it was assumed that other considerations should be taken since the main condition was not satisfied, i.e., it was acknowledged that the position of the boundaries influenced the total ground displacements.

Figure 5.10 represents a PLAXIS 2D model in which the vertical boundaries are within 30 m away from each side of the tunnel and the bottom boundary 17 m underneath the base of the tunnel. Since the tunnel diameter is approximately 5 m, the boundaries position are in accordance with the recommendations proposed by Meissner (2002). However, the figure allows to identify that the total displacement along the vertical boundaries is different from zero in all its extension, which demonstrates that the boundaries were probably affecting the ground behaviour deviating the model from the reality.

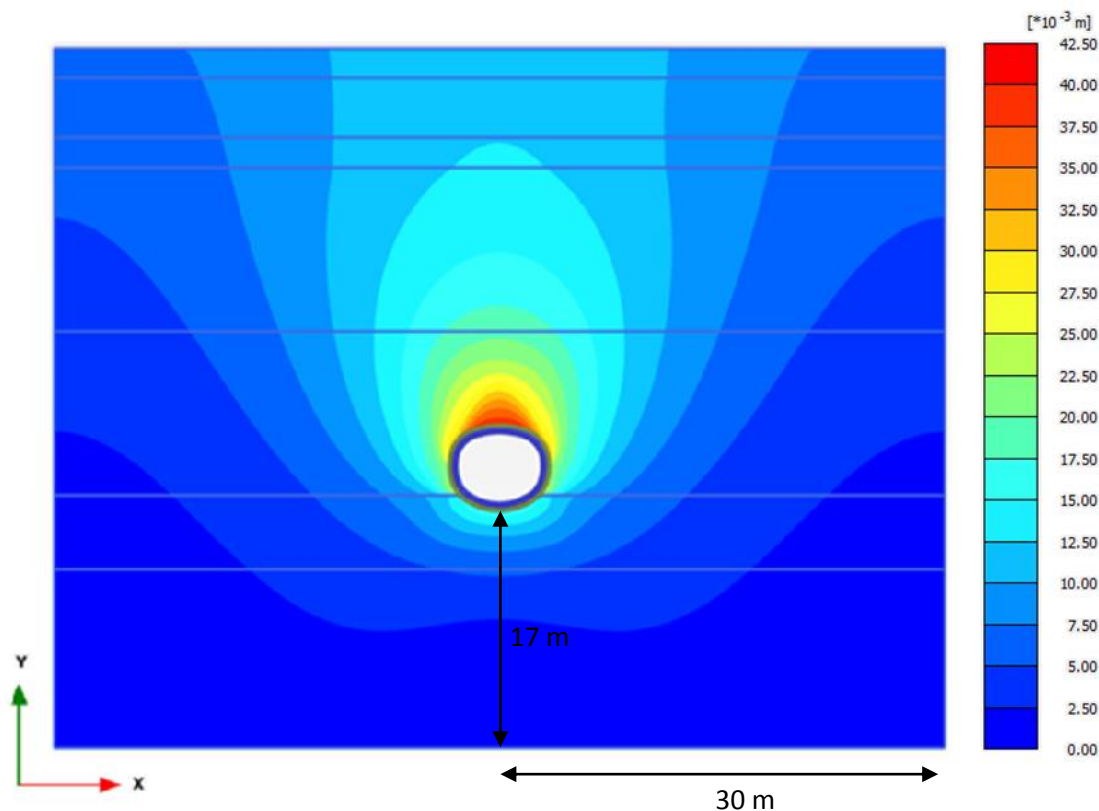


Figure 5.10 - Influence of the model boundaries on the ground behaviour regarding total displacements

In order to minimize any influence of the boundaries in the ground behaviour some other models with different dimensions were developed. Ultimately it was identified that a distance of 70 m between the vertical boundaries and the tunnel was enough to mitigate any displacement at the boundaries. The bottom boundary was defined at 27 m beyond the base of the tunnel.

Note should be taken that although there is only information from the boreholes until 76 m LUD deep (see Table. 4.3) it were assumed the same soil properties to enlarge the model depth to 66 m LUD. This

assumption was taken to prevent any influence of the bottom boundary in the tunnel behaviour, which is expected to suffer heave.

Concerning the global boundary conditions, PLAXIS automatically imposes a set of general fixities that represents the real deformation behaviour. For this model PLAXIS assumed that the vertical boundaries were fixed in horizontal direction and free in vertical direction allowing any vertical displacements to occur, which was what was pretended for this model. The model bottom boundary was fixed in all direction whereas the ‘ground surface’ was free.

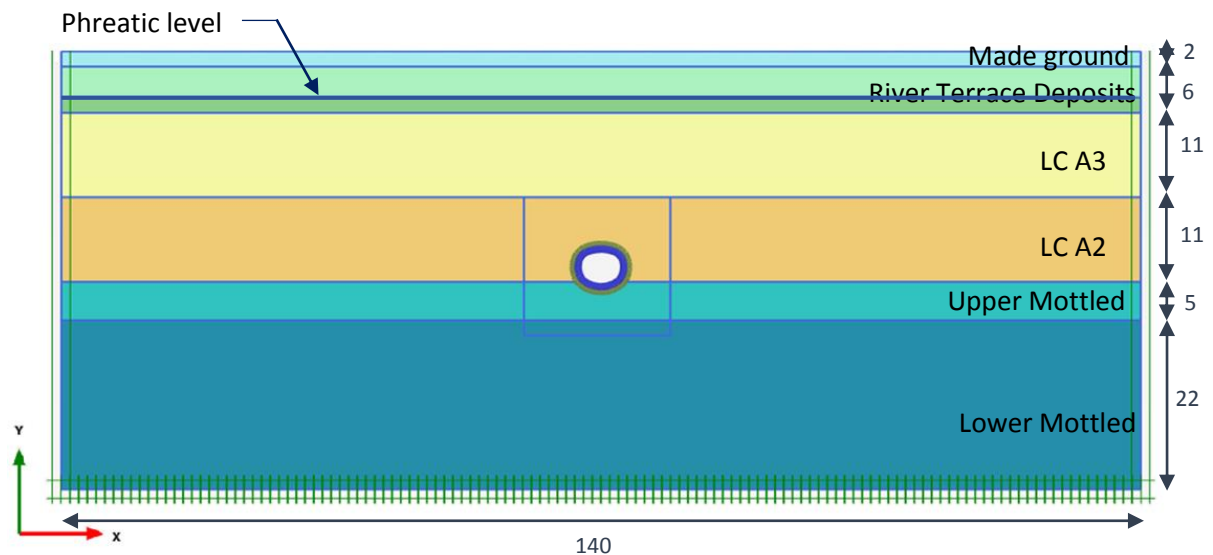


Figure 5.11 - Global geometry and boundary conditions - 2D model (dimensions represented in meters)

5.3.1.3. Mesh generation

The modelling of soil layers generally follows a continuum approach that is based on continuous functions in which quantities like displacements, stress and strain are homogenised. The numerical model is discretized and transformed to a finite element model by dividing the soil continuum into a calculation grid with cells, which is represented by the mesh and its elements. The quantities are still continuous and follow a specific order of interpolation within the elements that contain the material properties.

The discretisation consists mainly of generating a mesh with a specific coarseness and defining an order of interpolation within elements. This process defines the accuracy of the results since it delineates the precision at which the continuity of the solution is approximated. A finer mesh and a higher order of interpolation provides a more accurate solution (Brinkgreve, 2013).

The basic type of elements to model soil layers provided by PLAXIS 2D are 15-node and 6-node triangle elements. The first type of elements, which the author adopted, provide a fourth order interpolation for displacements whereas the numerical integration comprises twelve Gauss points. In consequence a mesh composed of this type of elements concedes a finer distribution of nodes and therefore more accurate results.

Additionally PLAXIS offers a range of predefined values for the element distribution from very coarse to very fine. The mesh of the developed model, which can be seen in Figure 5.12, was created based on a very fine element distribution. Furthermore PLAXIS allows the user to select automatic mesh

refinements in order to ensure a good quality mesh around structural elements, tool which the author has benefited.

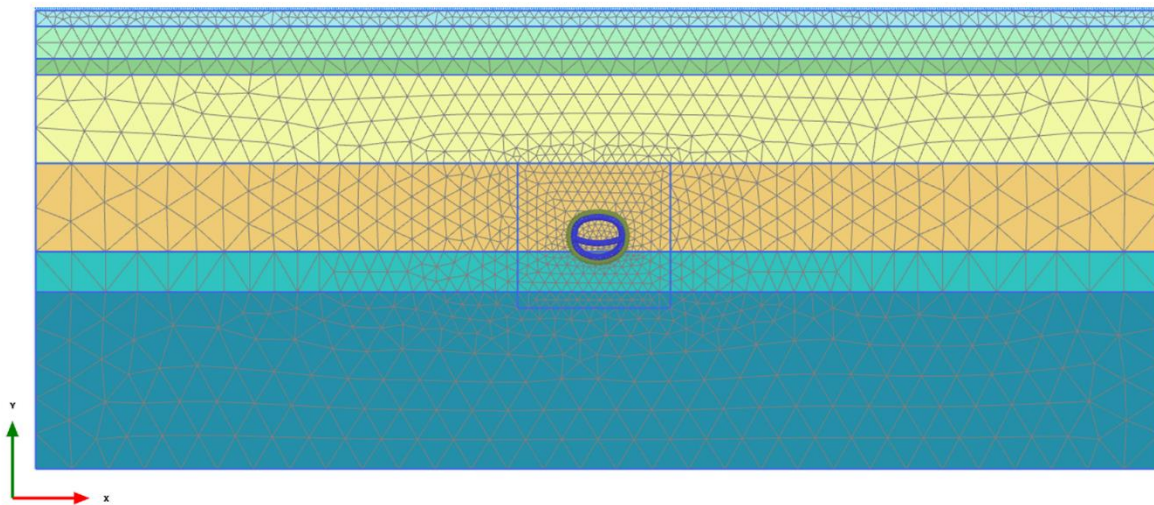


Figure 5.12 - 2D model mesh

Although the software confer a specific tool to refine the mesh where large stress concentrations or large deformation gradients are expected it was still necessary to create a cluster around the tunnel in order to get an even more refined mesh in this area. This option was taken because the quality of the mesh without the cluster, which PLAXIS provides, was lower than expected for modelling this situation. Therefore a mesh refinement was undertaken in all layers inside the cluster, reducing the coarseness factor to 0.3. The coarseness factor corresponds to a feature available in PLAXIS that gives an indication of the relative element size regarding the target element size as defined by the element distribution parameter. By default the coarseness factor is set to 1, consequently, when it is set to 0.3, the element size reduces to 30% of the target element size.

It may be noted that the coarseness factor could be reduced through all the layers in which the tunnel is installed, however as Potts & Zdravkovic (2001) refer the best mesh is not necessary the one with the largest number of elements but the one which use a smart and efficient way of meshing with local refinements where required.

Additionally, once the first two layers are considerably thin compared with the remaining ones, the coarseness factor was too high to create mesh elements with similar dimensions along the layers, thus it was necessary to reduce their mesh coarseness factor. After some attempts the author realised that reducing the coarseness to 0.5 was enough to obtain the quality required. Lastly the created mesh was constituted by 2880 elements and 23537 nodes.

Figure 5.13 represents the mesh quality provided from a specific tool in PLAXIS, this tool compares the shape of the elements created with an equilateral triangle shaped element and assign it a value ranging from 0 to 1, where 1 corresponds to a perfectly equilateral triangle. As it can be seen almost every element as a good quality.

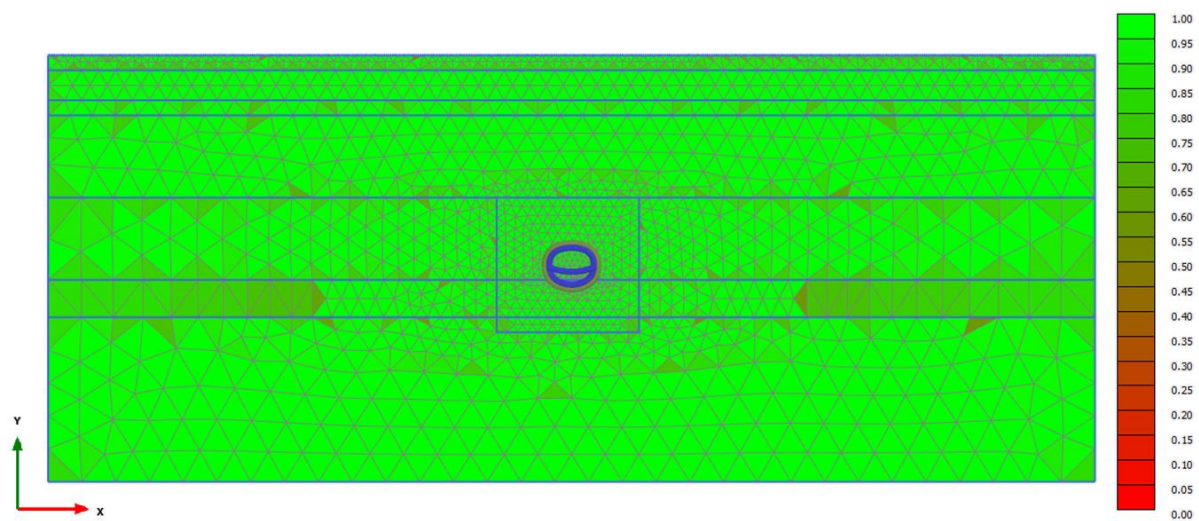


Figure 5.13 - Representation of the mesh quality from PLAXIS 2D model

5.3.2. STAGED CONSTRUCTION

In order to create a reliable and practical model some assumption are made to simulate the excavation. The following list resume the most important assumptions that were made.

- The soil strata are horizontally bedded and of uniform thickness;
- The groundwater head remains constant throughout the excavation process at 6 m below ground level (i.e. $z = 117 \text{ m}$);
- A surcharge of 10 kN/m^2 acts at ground level;
- The primary sprayed concrete lining thickness remain constant;
- The sprayed concrete is applied and gains its full strength instantaneously when the 1 m advance is excavated.
- The tunnel face is unsupported and deforms to equilibrium during each advance.

In order to reproduce the real works as accurate as possible the tunnel excavation was divided into top heading and invert excavation, as presented in the detailed design (refers to Figure 4.7 b)). Hence, the creation of the initial stress field, which was generated in the first stage by means of K_0 procedure, is followed by three more stages: the first corresponds to the application of the surcharge, the second to the excavation of the top heading, which will be divided into two different phases as will be explained later on this subsection, and the last to the excavation of the tunnel invert.

The K_0 procedure is a specific calculation method that takes into account the loading history of the soil, this method is adopted over the *Gravity loading* method, also available in PLAXIS, which generate the initial stresses based on the volumetric weight of the soil instead. For an overconsolidated clay as London Clay, the loading history is a controversial factor that determines the accuracy of the initial stress state simulation, hence once the in-situ earth pressure coefficient is provided by geotechnical reports it must be embraced and directly inputted into the model, instead of allowing the software to estimate it based in the Poisson's ratio.

In PLAXIS 2D a *Staged construction* loading type, which was the adopted type for this analysis, allows the user to specify a new state that is supposed to be reached at the end of the calculation phase. In order to simulate the construction process of sprayed concrete lining tunnels the three-dimensional arching effect needs to be taken into account, hence it is necessary to split the *staged construction* process into

more than one calculation phase, one to simulate the excavation of soil volume and another to simulate the application of the lining.

The three-dimensional arching effect that occurs within the soil, is emulated in PLAXIS by using the β -method, (Convergence confinement method - Schikora & Fink, 1982), which divide the initial stresses, p_k , acting around the location where the tunnel is to be constructed, into a part $(1 - \beta) \cdot p_k$ that is applied to the unsupported tunnel, and a part $\beta \cdot p_k$ that is applied to the supported tunnel. The β -value is usually based on the designer's experience, which depends mainly on the ratio of unsupported tunnel length and the equivalent tunnel diameter. For SCL tunnels excavated in London Clay, the company's experience state that $\beta = 50\%$ is considered a good approach to simulate the three-dimensional arching effect.

Instead of entering a β -value, PLAXIS use a reduced ultimate level of $\sum M_{stage}$, which is compared with the $(1 - \beta) \cdot p_k$ component. Hence, by assuming $\sum M_{stage} = 0.5$, one considers that only 50% of the stress state is applied to the excavation phase, whilst the remaining stress is applied simultaneously with the tunnel lining in a consecutive phase.

Summing up, the staged construction results in six calculation phases:

- 1) Generation of the initial stress field;
- 2) Application of the surcharge;
- 3) Excavation of the top heading;
- 4) Application of the lining in the top heading;
- 5) Excavation of the invert;
- 6) Application of the lining in the invert.

The groundwater head has a large influence on the pore water pressures generated around the tunnel location and therefore on the pore water pressures generated due to the tunnel construction. Note should be taken that the simulation of the excavation by deactivating the soil inside the tunnel only affects the soil stiffness and strength, without additional input the water pressures remain inside the tunnel. Thus to remove these water pressures, i.e. to remove the water inside the tunnel, the *WaterConditions* must be set to dry in the excavation stage.

5.3.3. RESULTS FROM THE BASE MODEL

5.3.3.1. Deformed mesh

Regarding the deformation of the mesh in the final stage of the excavation, Figure 5.14, it can be seen that there is a clear convergence of the soil around the tunnel, which is reflected by settlements in the crown of the tunnel and by heave in the base. This representation aim to qualitatively summarize the behaviour of the soil around the tunnel therefore the deformation is scaled up 30 times.

Regarding the displacements in the ground surface, they are imperceptible in this figure, however they were identified, which demonstrates that the displacements are propagated to the surface although the little influence.

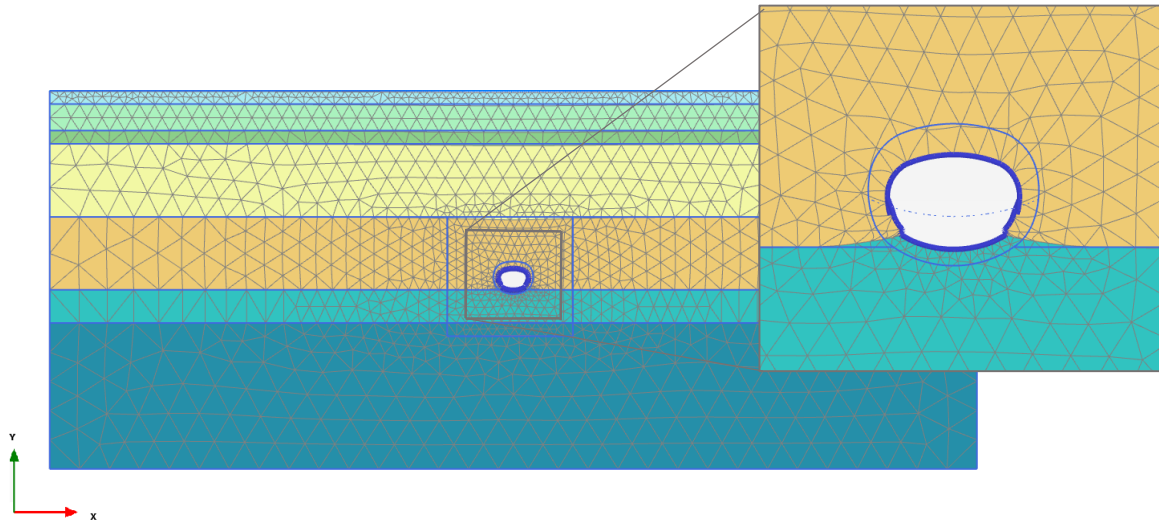
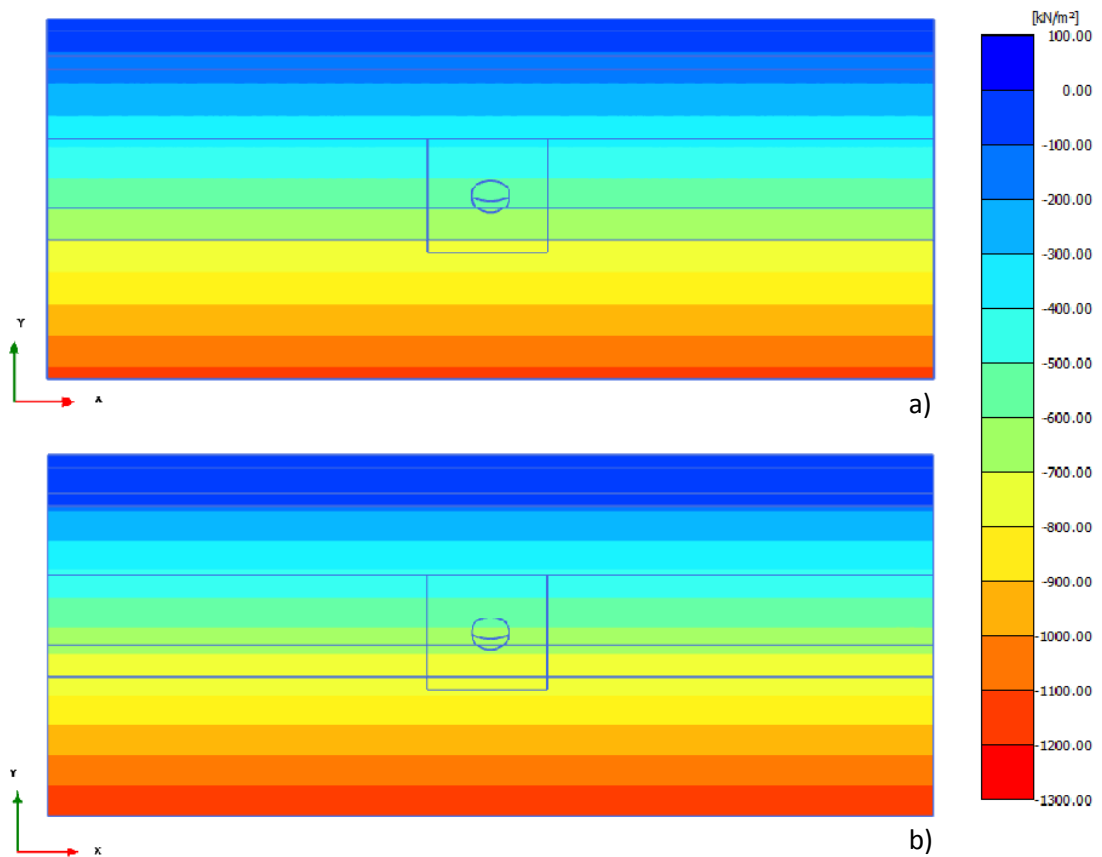


Figure 5.14 - Deformed mesh of the 2D model (scaled up 30 times)

5.3.3.2. Stress state

The initial stress field generated by means of the K_0 procedure as previously explained, is represented in the following figures. The presentation of the stress field will be based on both total and effective stresses, Figure 5.15 and Figure 5.16 respectively, bear in mind that the groundwater head is 6 m below the ground surface.

Figure 5.15 - Initial total stress state; a) Vertical stresses, σ_{zz} ; b) Horizontal stresses, σ_{xx} , σ_{yy}

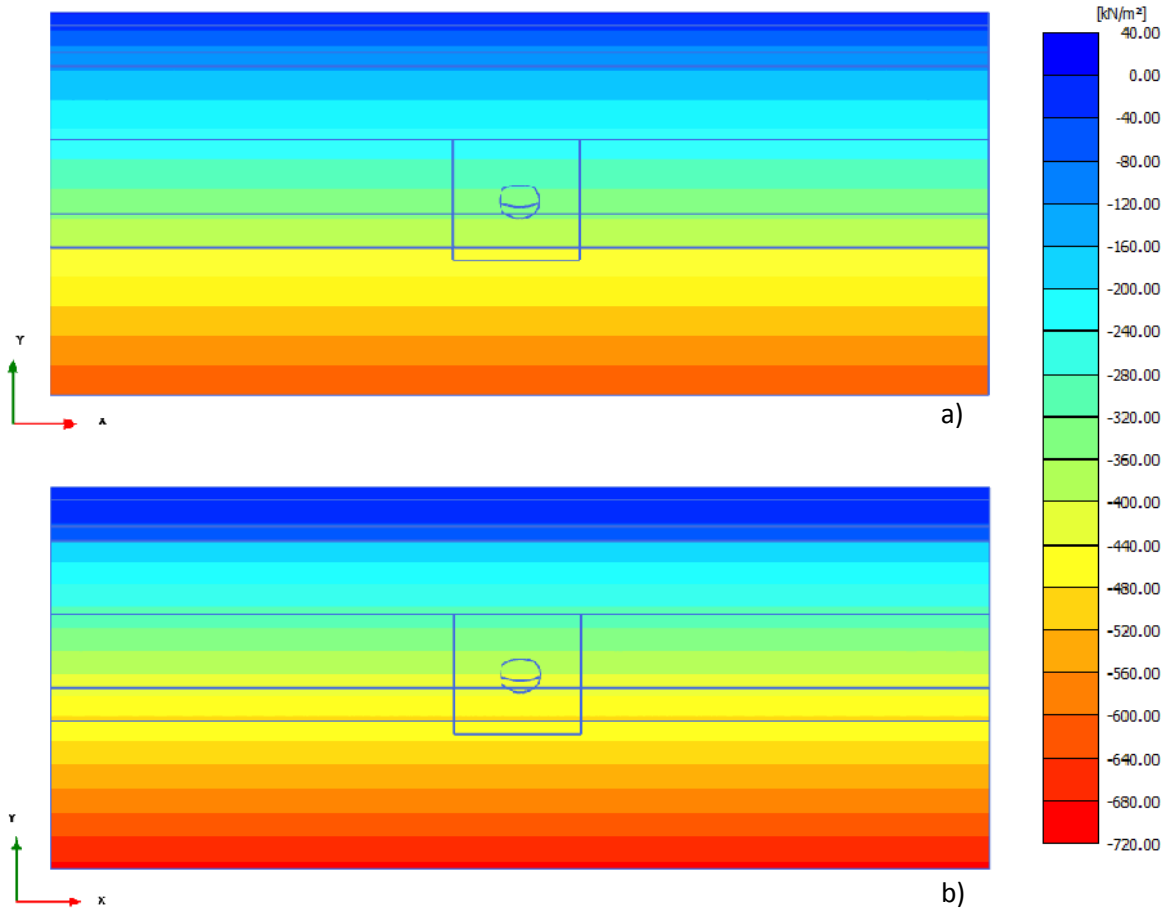


Figure 5.16 - Initial effective stress state; a) Vertical stresses, σ'_{zz} ; b) Horizontal stresses, σ'_{xx} , σ'_{yy}

The vertical total stresses, represented as σ_{yy} , reached a maximum value of 1139.0 kN/m^2 , and the horizontal total stresses, which are equal in the plane and out of the plane, $\sigma_{xx} = \sigma_{zz}$, a maximum of 1202.0 kN/m^2 . As expected, the horizontal stresses are greater than the vertical ones, once the in-situ earth pressure coefficient is greater than one through the London Clay and the Mottled Formations. However in the two shallower layers an opposite situation is identified once the in-situ earth pressure coefficient is smaller than one. In the deepest soil layer, the Lower Mottled it can be identified a little gap in the boundary with the Upper Mottled layer in hat concerns to horizontal stresses This gap is explained by the small difference in the in-situ earth pressure coefficient for the two different layers, which is 1.2 for the Upper Mottled and 1.1 for the Lower Mottled. The relation $\sigma_{xx} = \sigma_{zz}$ is explained with the assumption of $K_{0x} = K_{0z}$, since no anisotropic behaviour was adopted.

Regarding the effective stresses, the maximum value for the vertical effective stresses, represented as σ'_{zz} , is 629.0 kN/m^2 and for the horizontal effective stresses, σ'_{xx} and σ'_{zz} , 691.9 kN/m^2 .

After the excavation is completed, the scenario is quite different, as represented in Figure 5.17, the vertical stresses in the crown and in the base of the tunnel decrease, whilst near the tunnel's side walls increase substantially. This scenario is a result of the stresses redistribution induced by the arching effect, which can be clearly identified through the direction of the principal stresses in Figure 5.18. As the vertical stresses in the crown and in the base of the tunnel decrease due to soil removal in between,

the soil strength near the sides of the tunnel is mobilized, which then causes the stresses to increase in this area.

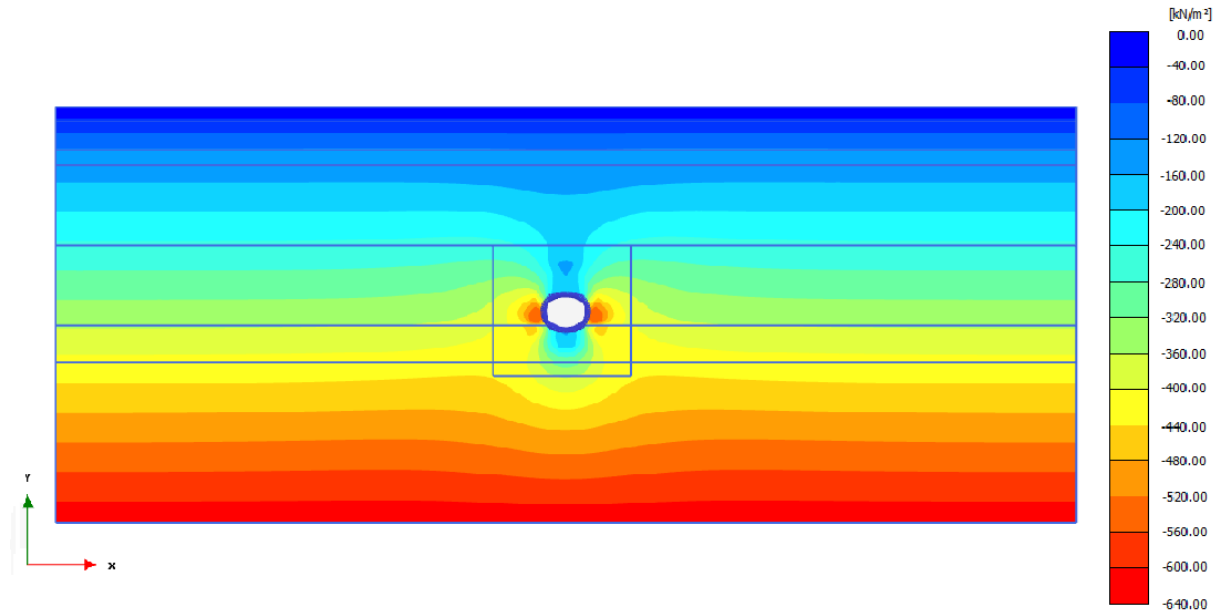


Figure 5.17 - Effective vertical stresses in the end of the excavation, σ'_{yy}

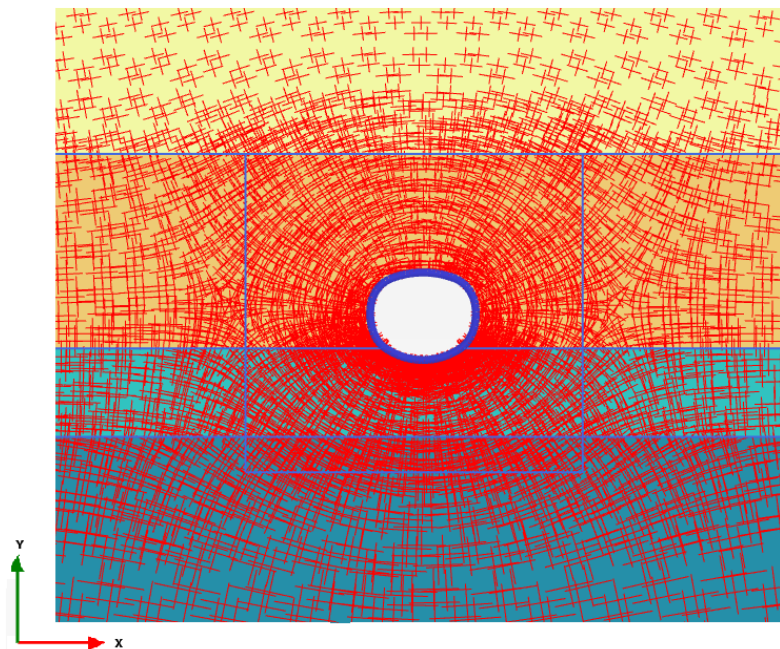


Figure 5.18 - Direction of the principal stresses after the excavation (arching effect)

The plot represented in Figure 5.19 shows the evolution of the vertical stresses in three different points due to the excavation of the tunnel throughout all stages. Point A and Point B represent, respectively, the stresses evolution in a point 1 m above the crown of the tunnel and 1 m below the base of the tunnel;

whereas Point C corresponds to a point located on the tunnel's horizontal axis, which is also 1 m distant of the tunnel lining.

The vertical stresses are represented as effective stresses, σ'_{yy} , hence it also has the influence of the pore water pressures. As an example of this influence, it can be identified that in the end of the excavation the effective vertical stresses decreased more in Point B than in Point A, which is expected because the pore water pressure increases in depth, thus is greater in Point B than in Point A. This comparison is possible because the pore water pressure decreases in the same proportion during the tunnel construction for both points, as it can be seen in Figure 5.20.

Note that the following plots were reproduced from PLAXIS 2D output, hence the abscissa corresponds to the internal reference of each phase (listed as "step"). Each calculation phase is generally divided into a number of steps, which are automatically generated in PLAXIS to take into account the non-linear behaviour of the soil. However, for an efficient interpretation, the steps must be read as follows:

- Step 0 - Generation of the initial stress field;
- Steps [0-2] - Application of the surcharge;
- Steps [2-5] - Excavation of the top heading;
- Steps [5-8] - Application of the lining in the top heading;
- Steps [8-15] - Excavation of the invert;
- Steps [15-18] - Application of the lining in the invert.

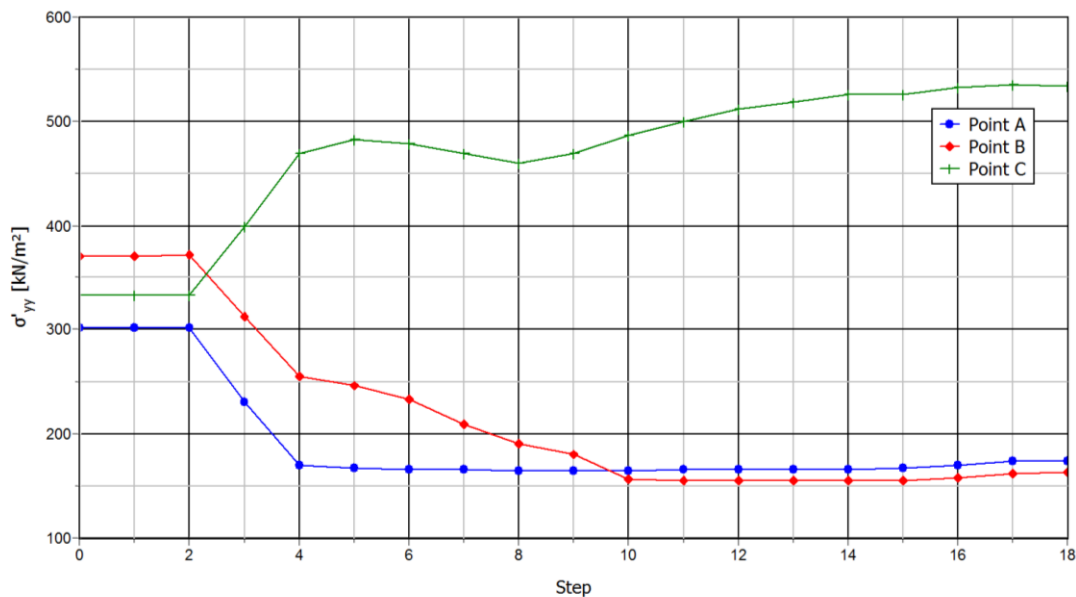


Figure 5.19 - Effective vertical stresses evolution during the excavation

Figure 5.19 corroborates the above explained regarding the vertical stresses. Whilst the vertical stresses decrease in the crown and in the base of the tunnel (Points A and B), in the sides of the tunnel the vertical stress increases. Regarding Point A, it can be seen that this point is only affected by the excavation of the top heading - steps [2-5], where it register a decrease of approximately 47% of the initial effective vertical stresses.. The point near the invert of the tunnel - Point B - registered a decrease in the vertical stresses until step 10, which induces that this point only reached the equilibrium after the excavation of the invert as expected. A similar situation occurred with Point C, located near the tunnel side walls, only

reaching the equilibrium after the application of the lining on the invert. It is interesting to highlight that the evolution of the vertical stresses in Point C has a relative minimum identified in step 8. This relative minimum was reached due to the application of the lining in the top heading, which allowed to a new stress redistribution in the vicinity.

The following figure represents the evolution of the pore water pressures during the excavation. The removal of a soil volume generates negative pore water pressures in the vicinity of the excavation, inducing water flow towards the excavation. Hence, as expected, the pore water pressures decrease until the excavation of whole tunnel section is performed. Note that during the application of the lining in the invert (steps [15-18]), the pore water pressures slightly increase, which is due to the completion the tunnel lining.

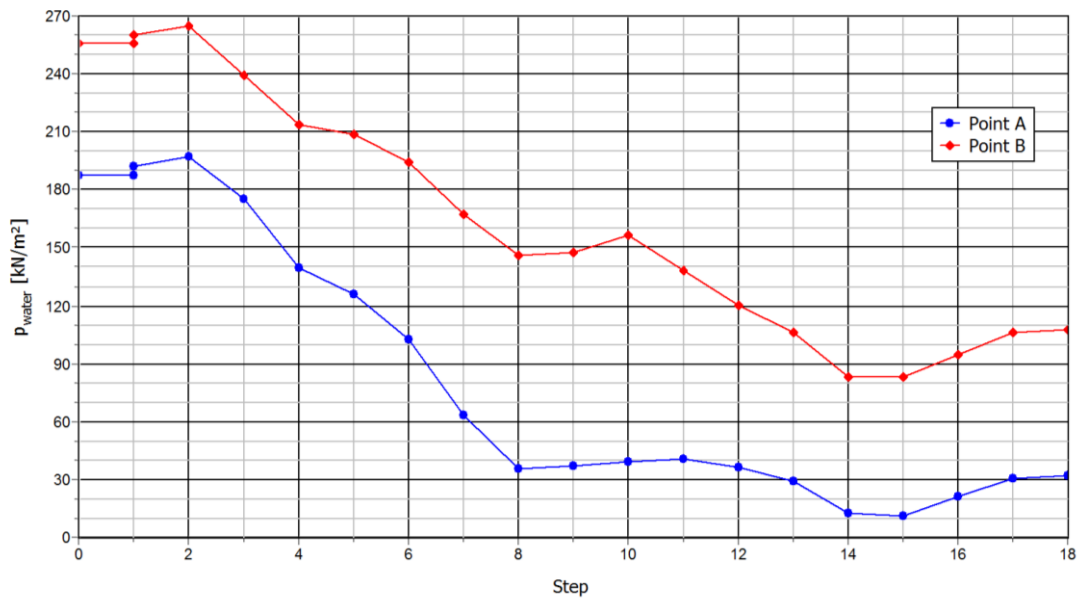


Figure 5.20 - Pore water pressures evolution during the excavation

Regarding the horizontal stresses, the arching effect also explains the recorded differences in the stress state, Figure 5.21. As the horizontal stresses decrease in the sides of the tunnel due to the removal of the soil, the stress redistribution ensures that the soil above and under the tunnel assimilates the new stress state, thus the horizontal stresses increase in this areas. Note that Figure 5.21 represents the horizontal stresses in x -direction.

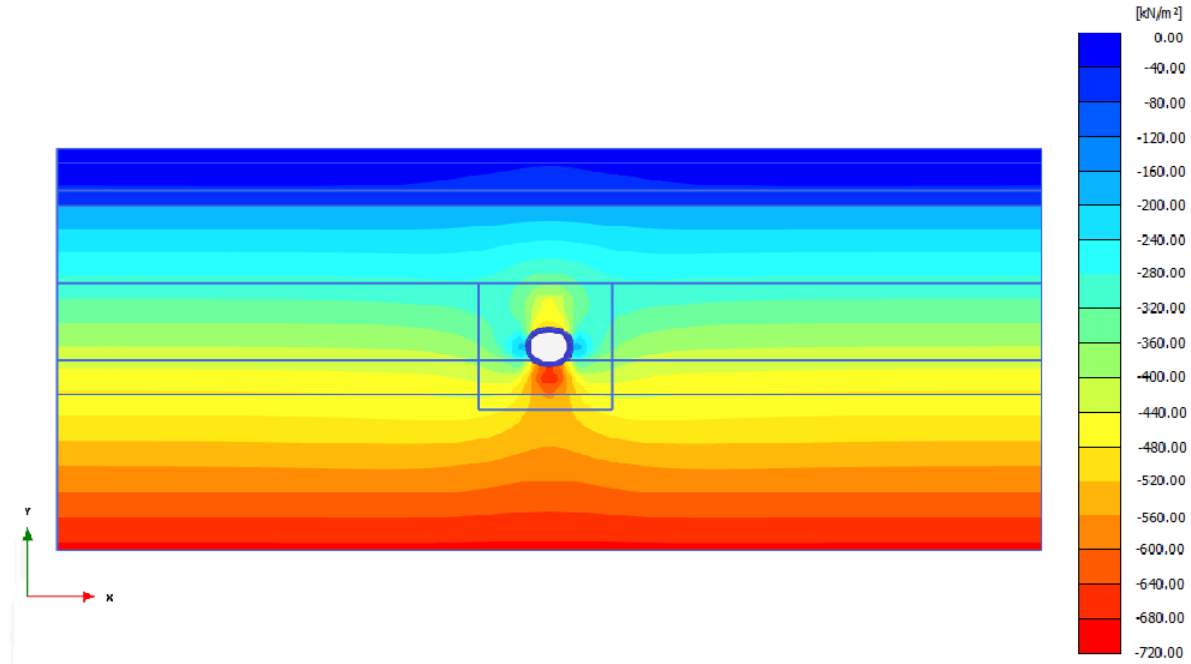


Figure 5.21 - Effective horizontal stresses in the end of the excavation, σ'_{xx}

The following plot, similarly to the represented in Figure 5.19, translates the evolution of the effective horizontal stresses (σ'_{xx}) throughout all excavation process. It can be seen that Point B recorded a greater increase in the horizontal stresses than Point A. This difference occurs due to the arching effect induced during the excavation of the invert; whilst Point A reached the equilibrium after the application of the lining in the top heading, Point B suffered a new stress redistribution while the excavation of the invert, which in turn increases again the horizontal stresses in the base of the tunnel.

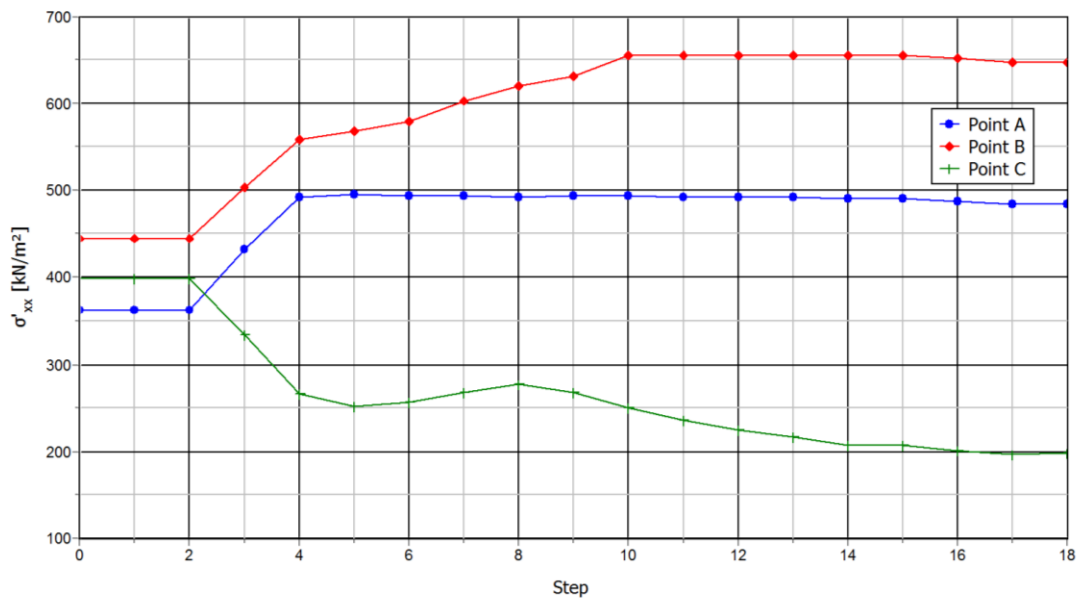


Figure 5.22 - Effective horizontal stresses evolution during excavation

In order to verify if PLAXIS applied the β -method correctly to simulate the relaxation, an assessment was carried out to the soil stresses around the tunnel boundary. This assessment intended to verify if the stresses in the soil perpendicularly to the tunnel boundary after the application of the mentioned method were actually 50% of the stresses in the initial state. It should be mentioned that this assessment was carried out in an initial stage of this study, which intended to calibrate the model before any results were analysed.

The assessment was performed in Excel, in which the normal stresses on the elements located in a line around the tunnel boundary were inspected (see Figure 5.23 to locate the reference line represented in red). The normal stresses in that line corresponding to the initial stress state are represented in the fifth column of the spreadsheet, which is presented in appendix A. The normal stresses along the same line in the consecutive stage, which corresponds to the excavation of the top heading are represented in the sixth column. In fact, the reference line only corresponds to the upper part of the tunnel boundary because this is the part that is actually being exposed after the excavation of the top heading, thus the part that needs to be inspected.

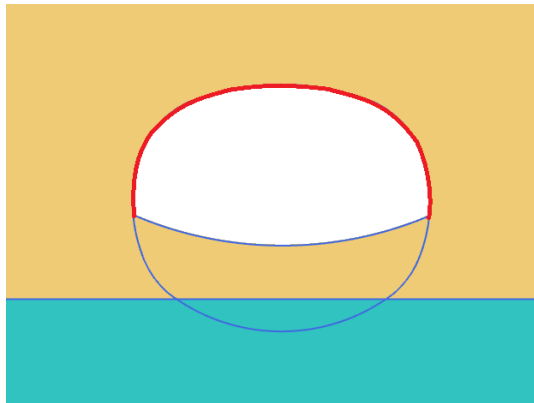


Figure 5.23 - Tunnel boundary's reference line

Through the spreadsheet analysis one can conclude that the program is actually applying the method correctly because, in the exposed boundary, where the normal stresses should be zero, they are in fact 50% of the initial stresses, which is the aim of the method.

Note should be taken that the values highlighted in orange in Appendix A correspond to the vertices of the line, which may evidence a numerical issue due to the concentration of stresses resulting in a relation that is not 50%.

5.3.3.3. Yielding

A perfectly-plastic model is a constitutive model with a fixed yield surface, which means that it is fully defined by model parameters and is not affected by straining. For Mohr-Coulomb model the yield surface is formulated based on the failure envelope previously presented. Hence for stress states represented by points within the yield surface, the behaviour is purely elastic and all strains are reversible, whereas for stress states lying on the yield surface the soil behaviour becomes elastic perfectly-plastic. When this situation occurs PLAXIS identifies these points as plastic points.

It is expected that the soil develop plastic points and also, as a result from a new stress state the same points return to an elastic condition if they return to a state within the yield surface again. However the strains developed through the plastic behaviour are irreversible.

There are two types of plastic points represented in PLAXIS, the “failure points” and “tension cut-off points”. The first indicates that the stresses lie on the surface of the failure envelope, which means that the shear strength reached the limit. This situation occurs when the normal stress applied in a point is significantly small compared with the shear stress, leading to the failure of the point. Regarding the tension cut-off points, these indicate a situation in which tensile stresses had developed, which is allowed when the shear strength is sufficiently small. Hence the soil may also fail in tension instead of shear. In this analysis no tension cut-off points had developed.

When the area of the plastic points developed is too extensive, failure mechanisms may develop, thus it is important to identify where the yielding occur. The following figures represent the plastic points developed due to the excavation through different stages.

Figure 5.24 represents the yielding area developed through the four excavation stages. It can be identified that between stages some points return to a state within the yield surface, which can be clearly identified between the invert excavation stage and the application of the corresponding lining.

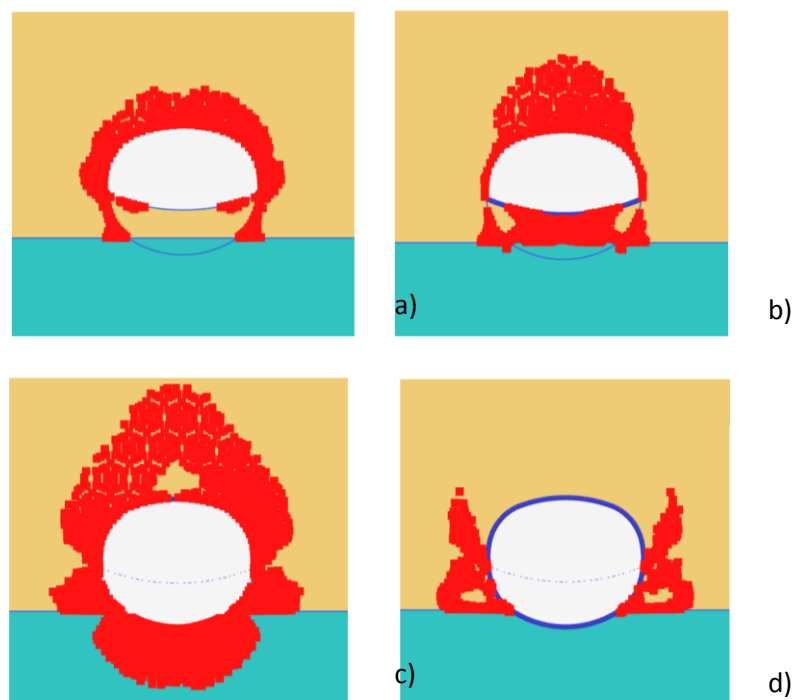


Figure 5.24 - Yielding history throughout the excavation process; a) Top heading excavation; b) Lining application in the top heading; c) Invert excavation; d) Lining application in the remaining boundary

The Figure 5.24 c) represents the critical stage, which corresponds to the generation of the larger yielding surface, this highlights the importance of splitting the excavation in top heading/invert not just to control the displacements but also, and most importantly, to prevent failure mechanisms to develop, because the yielding area would certainly be even larger if the tunnel section was excavated in one step.

However Figure 5.24 d) which corresponds to the final stage, presents a yielding area much smaller than the previous one since the concrete lining is now completely installed, thus the soil is now partly

supported by the lining strength and the ground arching effect is fully mobilized with a more circular and closed tunnel shape.

It is possible to identify through all the stages that the lower soil layer represented in blue, which corresponds to the Upper Mottled Formation, presents less plastic points than the upper one – London Clays unit A2. This fact occurs because the strength of the soil in the lower layer is greater than the strength of the lowest part of the London Clay Formation. Actually, the boundary between this two layers as it was modelled, do not correspond to a real situation since the strength of the soil in this boundary has a gap of 72.5 kPa , situation that hardly occurs in the reality once the transition between different layer is often quite diffuse. The undrained shear strength of the soil in the boundary is 177.5 kPa for the London Clay and 250 kPa for the Upper Mottled. These values result from the prediction of the geotechnical parameters in the GIR, in which the undrained shear strength of London Clay is said to increase in depth.

5.3.3.4. Ground displacements

As already mentioned on the beginning of this section the soil converge towards the tunnel. Figure 5.25 represents the heave effect which corresponds to a displacement of 18.7 mm whereas the crown settles 35.7 mm . The identified soil behaviour was expected since the stresses relief induced by the removal of a soil volume cause the soil to move towards the empty space.

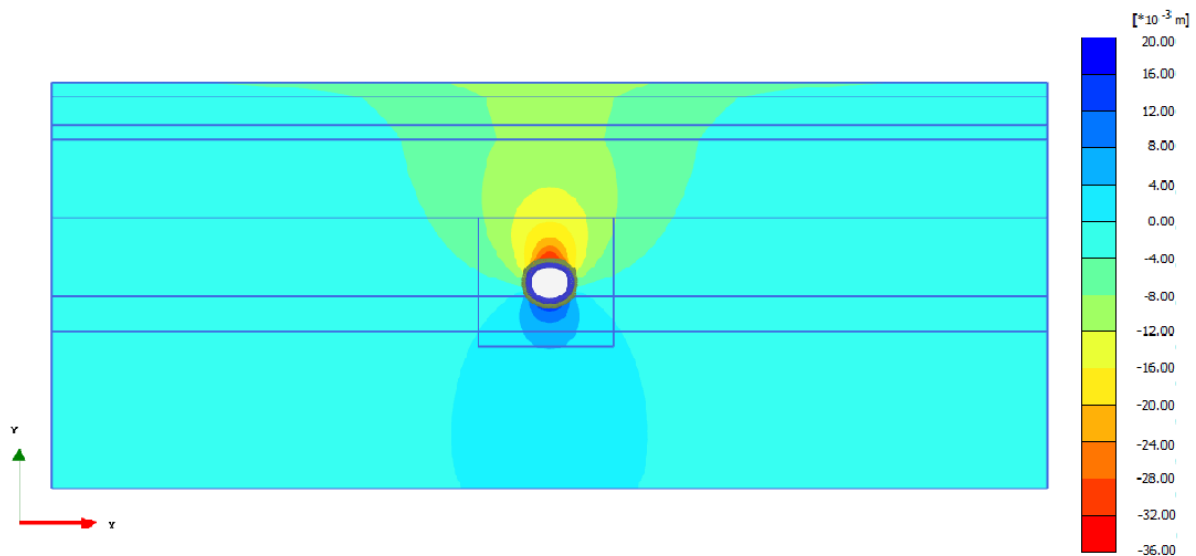


Figure 5.25 - Vertical displacements in the end of the excavation, u_y

It was identified that the maximum vertical displacement, 37.51 mm , was in fact reached in the previous stage, in which the concrete lining was not yet installed in the invert. After the application of the lining there are an important stress redistribution that allows the displacements to experience a slightly decrease due to the recovery of part of the elastic displacements.

In addition the horizontal displacements, reflect an expected symmetrical movement towards the excavation, reaching a maximum of 24.9 mm , as Figure 5.26 shows.

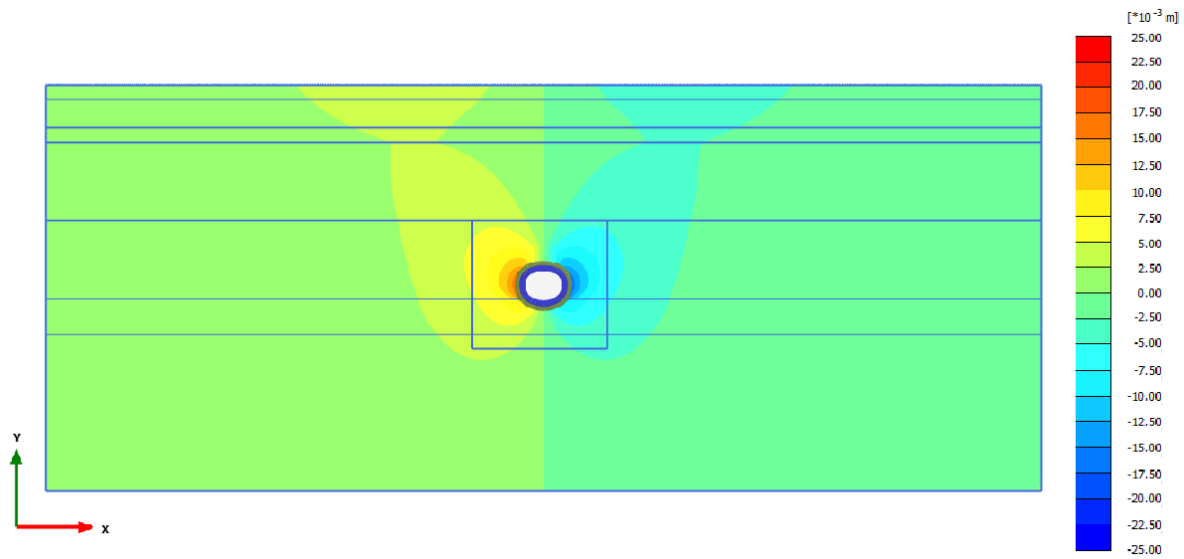


Figure 5.26 - Horizontal displacements in the end of the excavation, u_x

5.3.3.5. SCL tunnel' lining forces

It is known that the geometry of the tunnel cross section has a major influence in the forces developed in the lining. A circular cross section, for example, has a great behaviour regarding compression forces. However the structural forces not only depend on the cross section geometry but also on the geotechnical background.

Usually the axial forces in the tunnel lining slightly increase in depth due to the increase in the soil stresses, however this behaviour is not identified in this lining in the final stage of the excavation, Figure 5.27 b), where the axial forces in the invert are slightly lower than the axial forces in the crown of the tunnel. This fact may be explained by the excavation sequence. The removal of the temporary invert allows the lining in the tunnel walls to deform towards the tunnel, consequently at that stage the axial forces will decrease in that area. The application of the lining in the invert will then follow the stress relief in the tunnels walls and generate a new stress state. These alterations in the stress state in the vicinity of the tunnel explain the fact that in the tunnels walls the axial forces are lower in the final stage than in the temporary stage.

The diagram of the axial forces developed in the concrete lining is represented in Figure 5.27. The maximum value reached in the final stage, 763.4 kN/m corresponds to the forces in the crown of the tunnel, which was developed after a stresses redistribution due to the completion of the lining.

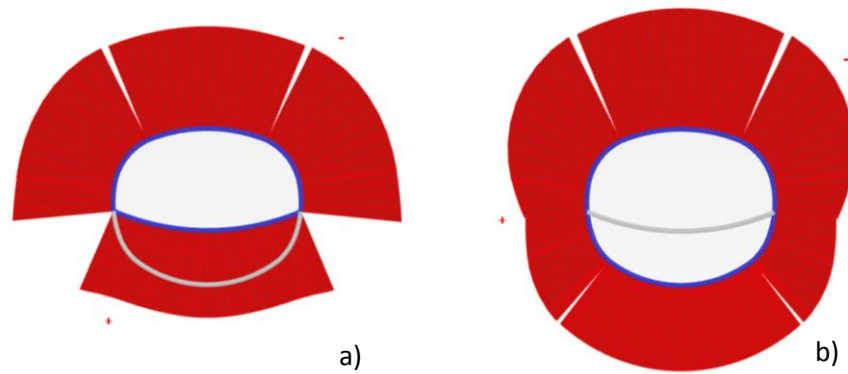


Figure 5.27 - Diagram of the axial forces in the tunnel lining; a) Temporary stage; b) Final stage

Regarding the bending moments in the lining, represented in Figure 5.28, the maximum value reached in the final stage was 135.7 kN m/m . This value corresponds to the bending moment in the most critical part of the cross section's geometry, as it can be seen in Figure 5.28 b). In this zone, the transition between the different arches that compose the geometry is sharp, which induces an abrupt transition in the shear forces. The geometry in the lower part of the tunnel is clearly more efficient since the bending moments are very low in this area, the transition between arches is softer.

In the temporary stage, Figure 5.28 a) the bending moments in the transition between the walls and the temporary invert is also really high because the transition complete an almost perfect perpendicular angle, which is really inefficient. Some techniques are applied during the tunnel construction to surpass this problem (e.g. to make this transition more circular), however none was considered in this model because there was not sufficient information from the detailed design regarding this aspect.

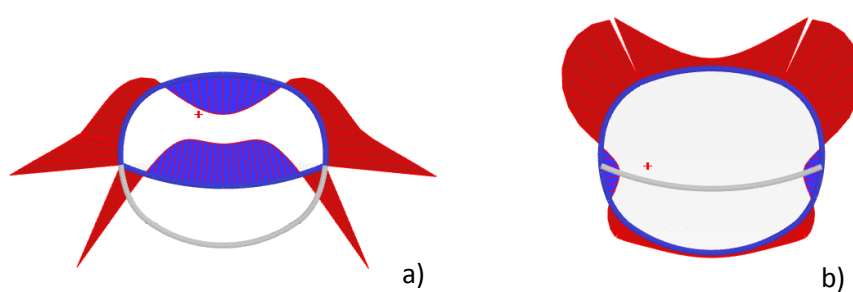


Figure 5.28 - Diagram of the bending moments in the tunnel lining; a) Temporary stage; b) Final stage

Considering that the concrete assumed for the SCL in this analysis was C20/25, the design value of concrete compressive strength within EC7, f_{cd} , is approximately 13.33 MPa . In order to verify if a linear elastic model is suitable for this analysis, the maximum compressive stress due only to the axial forces, σ_c , will be compared with the design value of concrete compressive strength. As previously presented, the maximum axial forces acting in the tunnel lining was 763.4 kN/m , considering that the cross section area per metre is $0.250 \text{ m}^2/\text{m}$, it leads to a compressive stress of 3.05 MPa . Although only the axial forces were considered it can be seen that σ_c is much smaller than f_{cd} , hence it can be assumed that a linear elastic model is reasonable within this analysis.

As explained before, the strategy used to model the SCL tunnel was the β -method which assumes a stress relief equal in all tunnel contour. França (2006) has shown through a three-dimensional analysis that actually the stresses relief in the tunnel invert is greater than the relief above the tunnel crown for tunnels performed in NATM, which is expected since the invert is usually performed using longer advances. This reasoning can be extrapolated to SCL tunnels due to the similitude between the two excavation methods. Hence one can say that the strategy used in the β -method may overestimated the lining forces in the invert since the stress relief is actually greater than the one modelled.

5.3.4. POT EXPECTED BEHAVIOUR

It was already explained that the 2D model do not take into account the Post Office Tunnel presence, however the soil behaviour in the area where the POT would be installed was analysed and the POT behaviour was correlated with the displacements recorded in the area, which means that the POT stiffness is not considered as well as the POT lining weight. This correlation was assumed since the results would be conservative, as the POT lining stiffness would certainly decrease the displacements around the SCL tunnel.

The results obtained from PLAXIS 2D model will be compared with the displacements recorded by the monitoring prisms. In order to describe the vertical displacements expected in the POT, the prisms RP1 and RP5 were defined as the reference prisms, since they are located at a lower level. In fact, the prism RP1 was the prism recording greater vertical displacements, thus it should be assumed as a reference; since RP5 is located at the same level it was chosen also as a reference in order to represent an average of the vertical displacements at that level. As it was presented in the previous chapter, the displacements recorded by RP1 and RP5 are relatively different because the excavation face first reaches the RP1 position. Although the developed 2D model considers the excavation sequence in a specific section, i.e., simulates the top heading/invert excavation, it does not consider the advance of the excavation longitudinally, hence the 2D model results should be compared with an average behaviour represented by the two prisms.

Regarding the horizontal displacements, since a bi-dimensional model does not allow to verify the displacements perpendicularly to the represented cross section, only the displacements perpendicular to the SCL tunnel will be analysed, which in turn will be correlated with the longitudinal displacements of the POT. This correlation is possible because the POT is approximately perpendicular to the SCL tunnel, thus the horizontal displacements, u_x , obtained in PLAXIS can be assumed as the POT expected longitudinal displacements. The longitudinal displacements will then be compared with the results recorded by the RP3 prisms. RP3 is the prism located nearest the POT crown, thus is approximately located on the POT longitudinal axis, which may represent the clearer results, besides this is the prism recording the greater longitudinal displacements as shown in Chapter 4.

In order to accurately compare the results from PLAXIS with the monitoring data from the specific prisms some assumptions were followed. Prisms RP1 and RP5 are on average 80 cm above the predicted base of the POT. Assuming a margin of 20 cm between both tunnels, (i.e. a margin that takes into account the external distance between the SCL tunnel and the POT as well as their lining thickness), those prisms are expected to be approximately 1 m above the SCL tunnel crown. Thus all plots presented in this section corresponding to vertical displacements were then produced based on the displacements read in a baseline located 1 m above the SCL tunnel crown. A similar strategy was used to represent the longitudinal displacements, however RP3 is on average located at 2.4 m above the predicted base of the POT, hence the plots produced to this effect were based on the recorded displacements in a baseline located 2.6 m above the SCL tunnel.

5.3.4.1. Vertical displacements

The predicted vertical displacements of the POT are presented in Figure 5.29, which are represented through the reference baseline for the vertical displacements. As it was previously explained, it can be assumed that the POT position would be along this baseline. The magnitude of the displacements are presented in Figure 5.30.

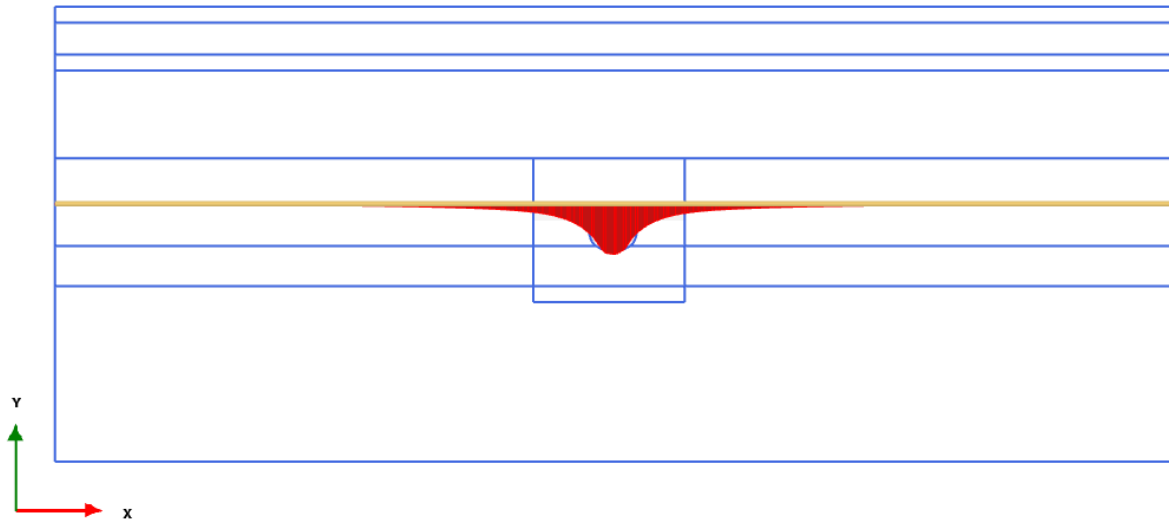


Figure 5.29 - Representation of the settlement trough in its reference baseline

The plot represented in Figure 5.30 reproduces the settlement trough calculated using PLAXIS (blue line), as well as the recorded vertical displacements by prism RP1 and RP5 throughout all arrays, which can be identified by the relative distance from the POT reference section (see section 4.3.1). It should be noted that the vertical displacements reproduced from PLAXIS are in a plan perfectly perpendicular to the longitudinal axis of the SCL tunnel, however the distance from the various arrays to the POT reference section, as presented in section 4.3.1, was measured along the POT longitudinal axis, hence considering that the POT is not perfectly perpendicular, the arrays position was rebated to a plan perpendicular to the SCL tunnel. Therefore the abscissa represents directly the distance from the SCL tunnel centre-line.

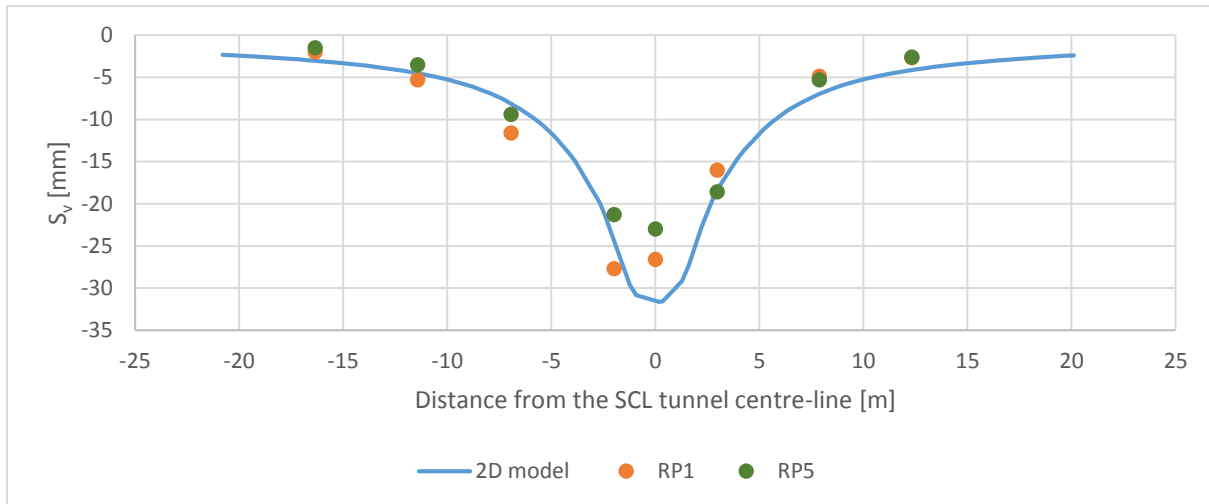


Figure 5.30 - POT expected vertical displacements

Figure 5.30 allows to identify that the magnitude of the vertical displacements reproduced from PLAXIS complies with the maximum vertical displacement recorded by the monitoring system. While the maximum recorded vertical displacement was approximately 29 mm, the vertical displacements obtained by PLAXIS was 31 mm, which represents a great approximation. However one may notice that the settlement trough obtained from the 2D model is narrower than the predicted from the monitoring data (see Figure 4.15 for further interpretation).

The reader may be reminded that this numerical model do not consider the POT position, therefore the stiffness of its lining is not taken into account, hence although the magnitude of the displacements is accurate it may be overestimated. Accordingly, the shape of the settlement through would be wider if the POT lining stiffness would be considered, since it may dissipate the displacements along the POT.

Although its simplicity, the 2D model returned satisfactory results in what concerns to vertical displacements, which may indicate that the geotechnical parameters adopted represent a satisfactory preliminary approximation of the real soil behaviour, however further analysis will be developed within this chapter.

5.3.4.2. Longitudinal displacements

The diagram of the horizontal displacements obtained from PLAXIS is presented in Figure 5.31 along the baseline assigned to analyse horizontal movements.

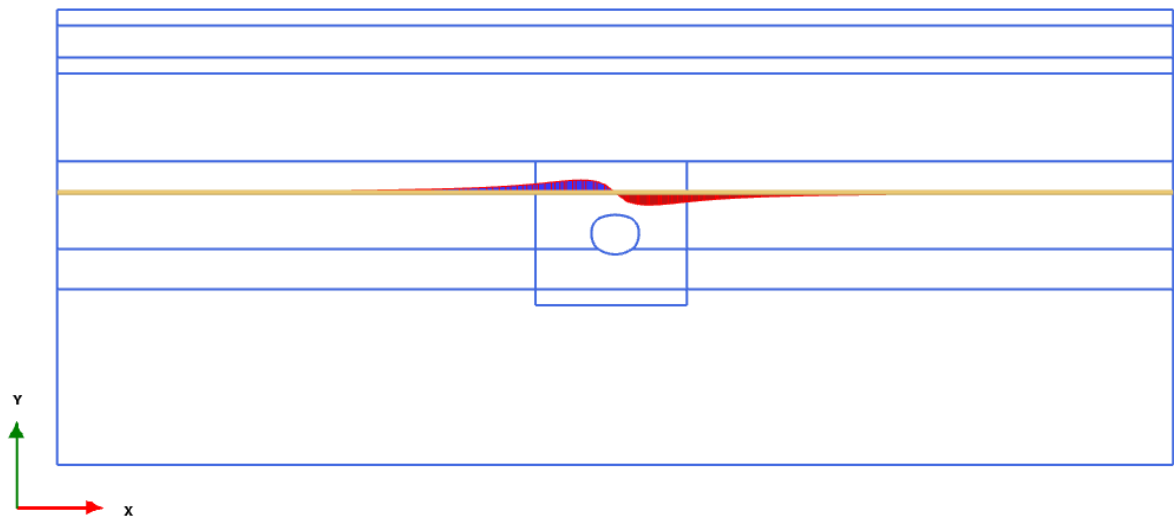


Figure 5.31 - Representation of the longitudinal displacements in its reference baseline

Figure 5.32 reproduces the longitudinal displacements obtained from PLAXIS as well as the displacements recorded in prisms RP3 in all arrays. The simplifications made to accurately reproduce the results in Figure 5.30 were also adopted to represent the longitudinal displacement recorded from the prisms, which means that the arrays position was rebated.

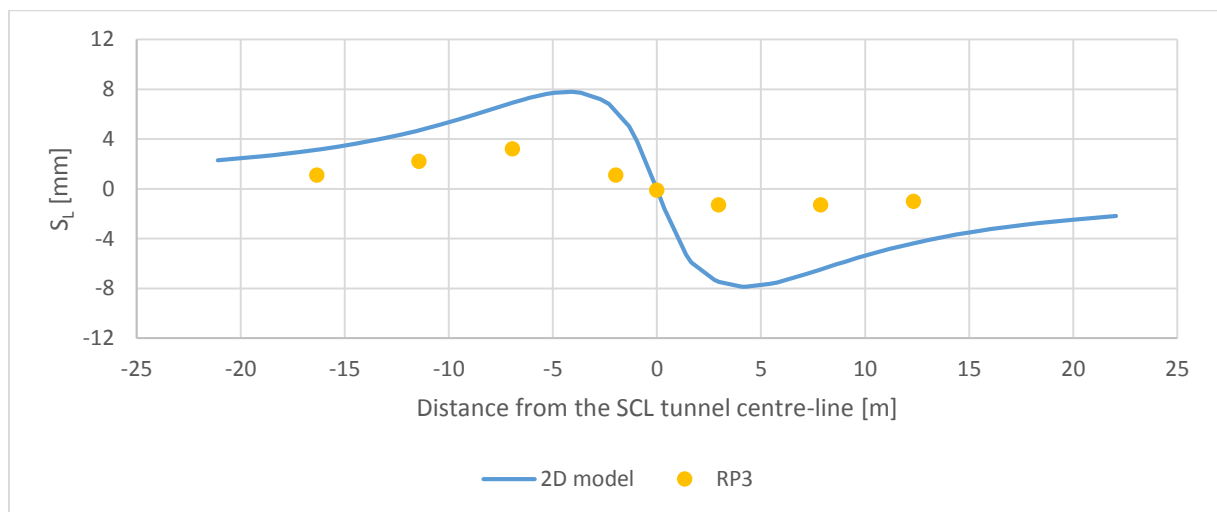


Figure 5.32 - POT expected longitudinal displacements

As it can be seen in the previous figure the longitudinal displacements obtained from PLAXIS do not coincide with the monitoring data, the maximum expected longitudinal displacement is approximately 8 mm whilst the monitoring prisms revealed that the POT only moved 3 mm towards the new tunnel excavation.

Similarly to what discussed in the vertical displacements section, the fact that the POT stiffness is not considered in the analysis may induce in greater displacements, which means that the displacements are

overestimated. In fact, this simplification may be quite contested once the lining stiffness it probably the most important parameter to be considered in the POT movement prediction.

One of the most controversial geotechnical parameter in analysis developed in London Clay is the in-situ earth pressure coefficient, K_0 . Actually this was the only parameter for which the GIR presented a range of options. Within the London Clay, K_0 is expected to be around 1.0 and 1.3 (from Geotechnical Interpretative Report), however regarding tunnel lining design the same report provide a different approach for tunnels to be constructed within one diameters of existing tunnels, which corresponds to a lower bound. Therefore the in-situ earth pressure coefficient used in this analysis might had been assumed greater than intended, which lead to higher horizontal displacements. The influence of this parameter will be examined in the following section.

5.3.5. SENSITIVITY ANALYSIS

In order to understand the influence of each geotechnical parameter involved in this analysis and therefore develop a back-analysis, a sensitivity study was carried out based on the model previously presented. For this sensitivity analysis both soil strength and soil deformation parameters were modified, however only parameters that can show variability in the expected results for the POT were considered.

Regarding the soil strength parameters, only the undrained shear strength, c_u was modified. The effective cohesion, c' , and the angle of shearing resistance, ϕ' , presented in Table 4.4, were only used in the first two soil layers as they were modelled with drained behaviour, hence modifying these parameters would only affect the results in the two first layers. As mentioned in the previous paragraph, only parameters that could affect directly and significantly the displacements of the POT were considered, since the POT does not intersect the two first layers, the influence of this parameters was not studied. Following the same reason, the influence of the soil deformation parameter: dilation angle, ψ , was not studied as it only influence drained materials, solely the influence of the Young's modulus, E' , was studied. Finally, the in-situ earth pressure coefficient, K_0 , was also analysed.

The results that will be presented throughout this analysis will only reflect the modification of the parameters in the layers intersected by the tunnels – London Clay Unit A2 and Upper Mottled. The influence of the modifications simultaneously in all layers was also analysed but will not be presented since the results obtained from this two approaches are very similar, which proves that those layers do not influence significantly the POT movements.

To notoriously identify the influence of the parameter, this study comprises the analysis of a lower and an upper bound of the parameters, which generally correspond respectively to half and twice the best estimate as presented in Table 4.4. Throughout this section both vertical and horizontal displacements will be analysed with respect to the parameters modifications. Finally, this study uses the same reference baselines previously presented.

5.3.5.1. Young's modulus influence, E'

The modification of the Young's modulus directly affect the soil displacements, therefore the POT movement, however its influence is not directly proportional because only the elastic part of the displacements is affected by the elastic modulus. As previously discussed the final displacements are also affected both by the yielding area developed around the tunnel and by the lining properties.

The parameters adopted for this analysis are resumed in Table 5.2. As shown previously, the Young's modulus increase in depth within London Clay, for this sensitivity study the increment is also modified in the same proportion.

Table 5.2 - Variation of the Young's modulus [MPa]

Soil layer	Lower bound	Best estimate	Upper bound
London Clay unit A2	$30+0.5z_2$	$60+z_2$	$120+2z_2$
Upper Mottled	50	100	200

* z_2 represent the depth in m below the top of LC A2

As it can be identified in Figure 5.33 and Figure 5.34 the modification of the elastic modulus do not affect much the shape of the curves, but mainly its magnitude. By increasing the Young's modulus to twice its best estimate, the maximum vertical displacement reduces approximately 40%, which corresponds to a decreasing from approximately 31.7 mm to 18.4 mm. When the Young's modulus is reduced to half of its best estimate, there is also a variation of approximately 40% in the vertical displacements, which corresponds to an increase from 31.7 mm to 54.1 mm.

The variation measured through this study shows that reducing or increasing the Young's modulus in the same proportion leads to a similar behaviour in terms of displacements, which induces that the influence of the Young's modulus is quite uniform.

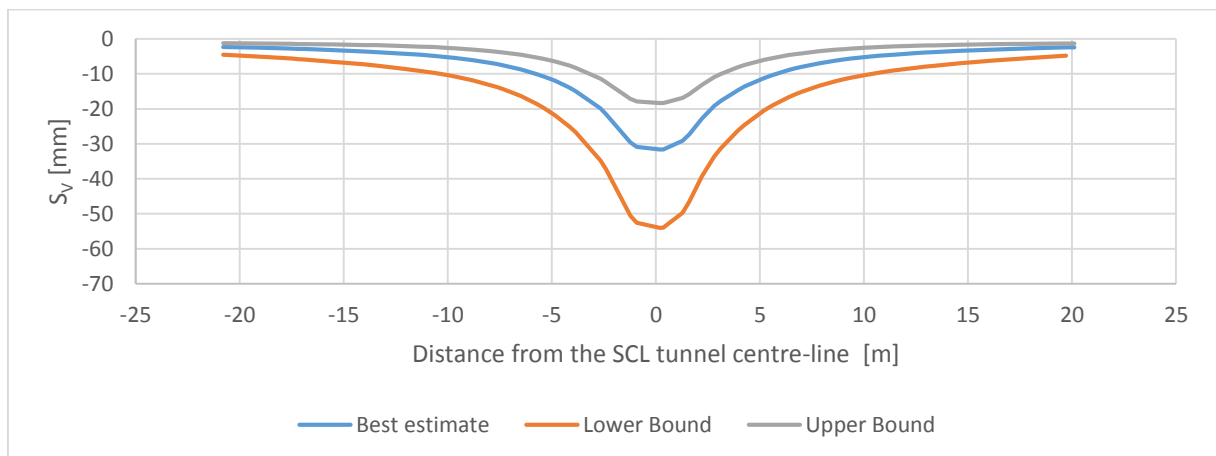


Figure 5.33 - Vertical POT expected displacements for different values of the Young's modulus, E

Regarding the horizontal displacements, the same modifications in the Young's modulus values led to variations of the displacements around 45%, which is a little higher than the percentage determined for the vertical displacements. However the relation between the lower bound and upper bound results is still quite uniform.

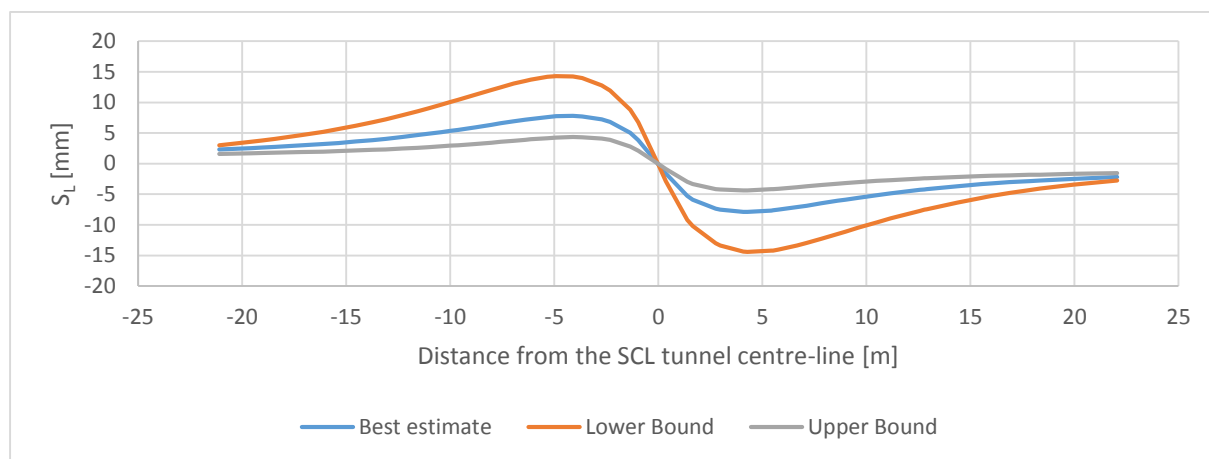


Figure 5.34 - Longitudinal POT expected displacements for different values of the Young's modulus, E

Note should be taken that the variation assumed for this parameter in the London Clay and in the Upper Mottled is not realistic since the real variation for this parameter in this Formations is much less extensive, however this study allowed one to identify the high importance and the influence of this parameter either for vertical and horizontal movements.

5.3.5.2. Undrained shear strength, c_u

Undrained materials studied using a linear elastic perfectly-plastic model, have the undrained shear strength as the only parameter, within the soil strength parameters, with significant importance. The proposed variation for this parameter within this sensitivity analysis (Table 5.3) is not applicable for any of the layers, i.e. any of the presented variations can be identified within London Clay nor Upper Mottled formation, however this modification allow to clearly identify the influence of this parameter. In fact, the gradient of the undrained shear strength used in the lower bound had to be considered the same as the gradient of the best estimate otherwise the soil would collapse, as it was verified.

Table 5.3 - Variation of the undrained shear strength [kPa]

Soil layer	Lower bound	Best estimate	Upper bound
London Clay unit A2	$75+2.5z_2$	$150+2.5z_2$	$300+2.5z_2$
Upper Mottled	125	250	500

It may be concluded by analysing Figure 5.35 and Figure 5.36 that decreasing the undrained shear strength as represented in the previous table would lead to a critical scenario. The vertical displacements increased from 31.7 mm to 174.9 mm whereas the horizontal movement increased from 7.8 mm to 51.7 mm, which would correspond to an unacceptable situation within an asset damage control point of view. This discrepancy may be associated with the considerable yielding area formed around the tunnel.

Whilst the influence of the Young's modulus is quite uniform, the influence of the undrained shear strength is completely the opposite, increasing the undrained shear strength leads to an improvement of the displacements much less significant than the deterioration caused by the decreasing of this parameter in the same proportion.

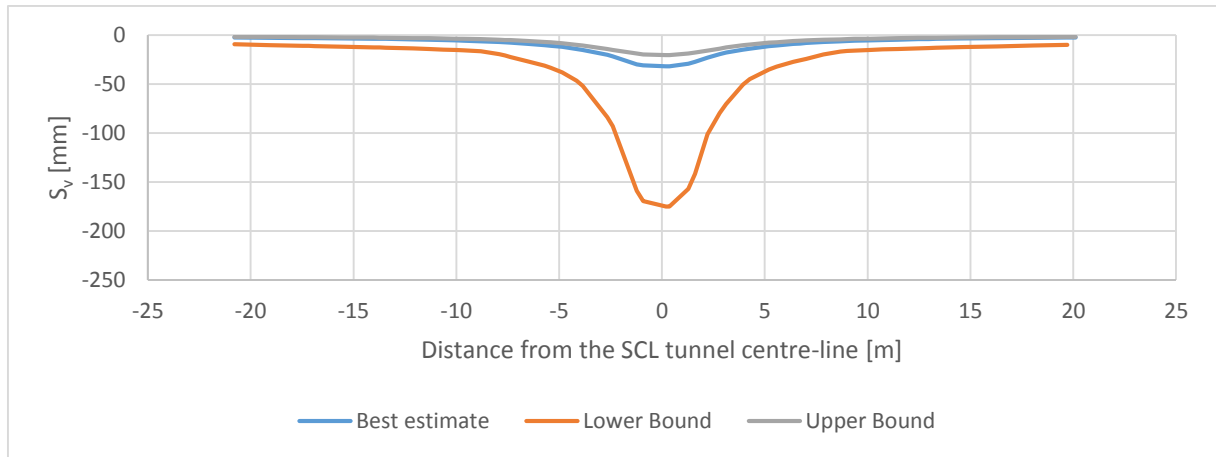


Figure 5.35 - Vertical POT expected displacements for different values of the undrained shear strength, c_u

There is no significant difference in the shape of the curves, which shows that the undrained shear strength mainly affects the maximum displacements. This parameter defines whereas a point is in a plastic condition or not, thus if this strength parameter is lower, for the same stress state, there will be more points on the yielding surface which inevitably leads to greater displacements.

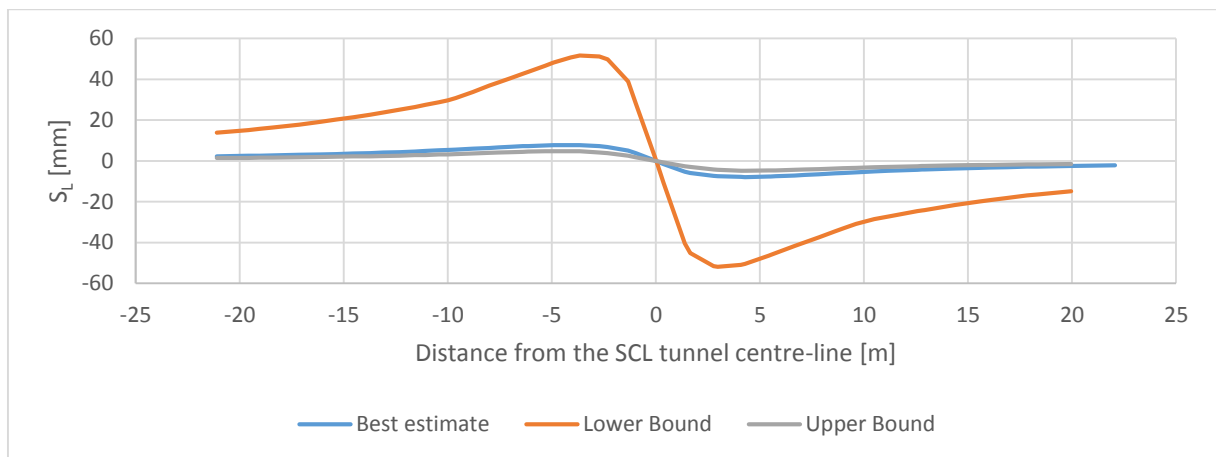


Figure 5.36 - Longitudinal POT expected displacements for different values of the undrained shear strength, c_u

Although this parameter reveal to be of high importance in the soil behaviour, it was perfectly defined in the GIR, hence it was not considered in the back-analysis. Besides, increasing the undrained shear strength, which would be the aim of the back-analysis since it would improve the results towards the real displacements, would represent a non-conservative strategy.

5.3.5.3. In-situ earth pressure coefficient, K_0

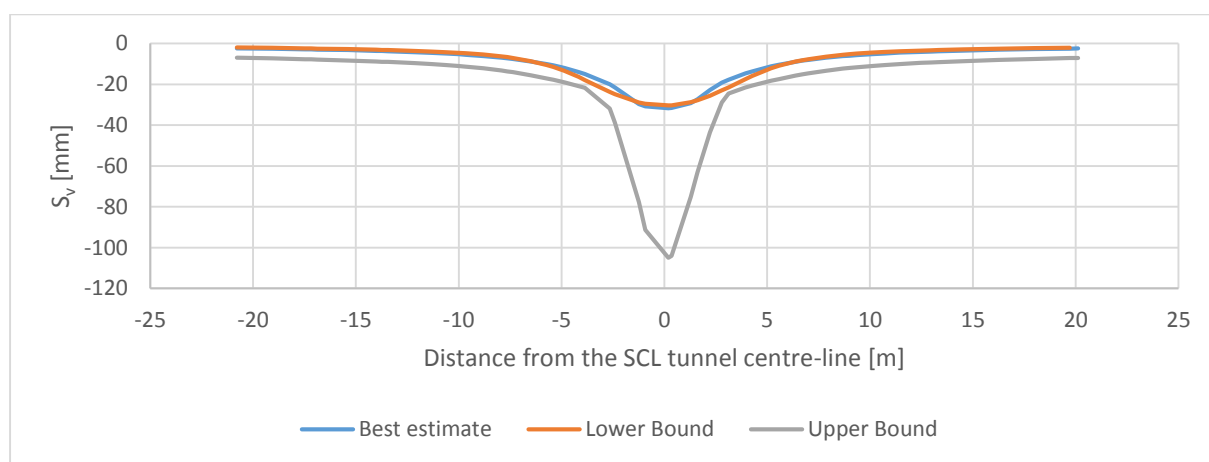
The in-situ earth-pressure coefficient is a fundamental parameter in what concerns to displacements due to unloading. This parameter determines the horizontal stress state, therefore the magnitude of stress relief due to excavation is highly influenced by this parameter, which in turn affects the magnitude of the displacements.

Table 5.4 presents the two extremes values considered for this analysis, although they are not representative for this soil layers it allows one to qualitatively identify the influence of the in-situ earth pressure coefficient. As discussed previously, this parameter is quite controversial and various approaches are taken to select the estimate that better represents the soil behaviour especially near existing structures, hence it is really important to characterize its influence since it can significantly alter the magnitude of the displacements.

Table 5.4 - Variation of the in-situ earth pressure coefficient

Soil layer	Lower bound	Best estimate	Upper bound
London Clay unit A2	0.6	1.2	2.4
Upper Mottled	0.6	1.2	2.4

Figure 5.37 and Figure 5.38 allow one to identify that the modification in K_0 alters not only the maximum displacement values but also the shape of the curves. Whilst $K_0 = 0.6$ almost does not alter the maximum vertical displacement, it alters the shape of the curve, which becomes slightly wider. These results reveal that a lower value of the in-situ earth pressure coefficient allows displacements to propagate further away from the excavation. On the contrary, an increase in this parameter results in a narrower effect, which leads to a substantial increase in the maximum vertical displacement, in this case from 31.7 mm to 105 mm. This substantial effect is explained by the high horizontal stresses initially installed and consequently by the huge stress relief due to excavation. In fact, the horizontal displacements also suffer a considerable alteration, which is related with this effect, Figure 5.38.

Figure 5.37 - Vertical POT expected displacements for different values of the in-situ earth pressure coefficient, K_0

Although the effect of the increase in the in-situ earth pressure coefficient leads to a substantial increase in the maximum values, it can be identified that, mainly in Figure 5.38, the displacements also increase throughout all the extension of the soil layer although in a lower level. This extended response of the soil reveals that the in-situ earth pressure coefficient has indeed a wider effect throughout the soil layers, which expose that the displacements propagate through all the extension of the soil. However, the width of the settlement trough is small, which allow one to conclude that the influence of increasing this parameter is even more important in the vicinity of an excavation.

In addition, it may be concluded that the in-situ earth pressure coefficient affect more significantly the horizontal displacements than the vertical ones. In this analysis for an increase of 100% in the in-situ earth pressure coefficient, the vertical displacements increase in approximately 300% whereas the horizontal displacements increase approximately 500%, which corresponds to a variation from 7.8 mm to 41.7 mm.

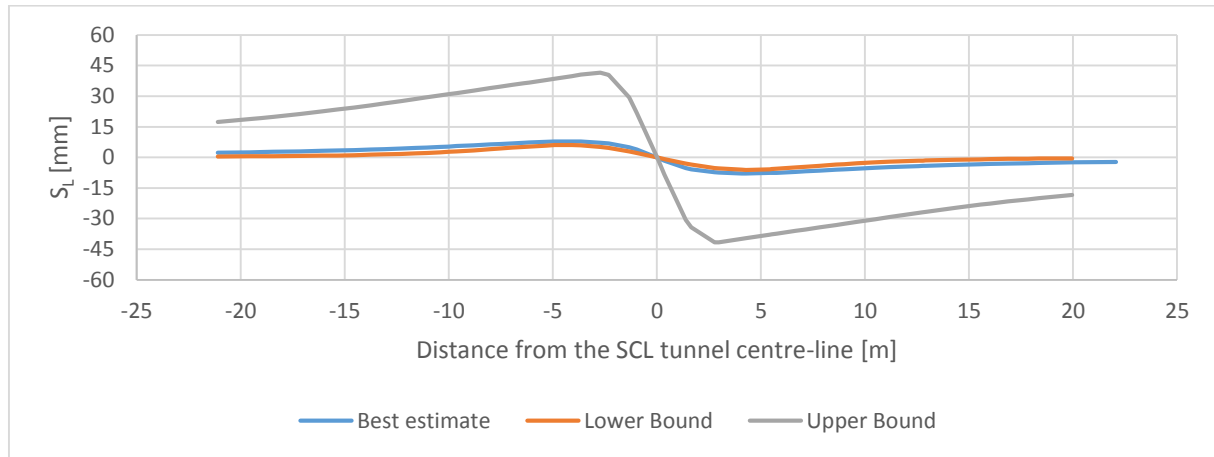


Figure 5.38 - Longitudinal POT expected displacements for different values of the in-situ earth pressure coefficient, K_0

Regarding the results from the base model, it was possible to verify that the POT expected longitudinal displacements were greater than measured by the monitoring system, whereas the results for the vertical displacements were satisfactory. This sensitivity analysis revealed that decreasing the in-situ earth pressure coefficient almost do not affect the maximum value of the vertical displacements but it slightly decreases the longitudinal displacements. Consequently, this parameter will be used as a variable in the back-analysis to improve the 2D model results.

5.3.6. BACK-ANALYSIS

Although this section is presented as a back-analysis it is, in fact, a model improvement process relying on the monitoring data, which assigns the character of back-analysis to this process.

The previous section presented the influence of the most important parameters for this analysis, and consequently defined which parameters would improve the results from the 2D model. It should be noted that the 2D model is quite limited and that a back-analysis based on this model will not reveal any reliable conclusion since the POT is not included. However, the author identified the potential to validate the geotechnical parameters initially assumed.

For this analysis, the in-situ earth pressure coefficient, whose influence was already introduced in the previous section, and the relaxation factor were taken into consideration. The soil relaxation was initially assumed based on the β -method, which was 50%. Although this parameter was adopted based on the company's experience as an initial estimation it can vary for different situations, thus its influence should also be assessed.

Considering that the horizontal displacements were greater than recorded by the monitoring system one may assume that the relaxation factor adopted initially was greater than the real relaxation, thus a lower

relaxation factor was assessed. For this purpose a relaxation factor of 40% was adopted, which reflects in $\beta = 0.6$ for the β -method.

The following figures, Figure 5.39 and Figure 5.40, present a comparison between the results obtained from PLAXIS with 50% relaxation ($\beta = 0.5$) and with 40% relaxation ($\beta = 0.6$), for vertical and horizontal displacements, respectively. For the comparison represented in Figure 5.39 the displacements recorded by the prisms RP1 and RP5 were included, as well as the results from prism RP3 for the horizontal displacements comparison, Figure 5.40.

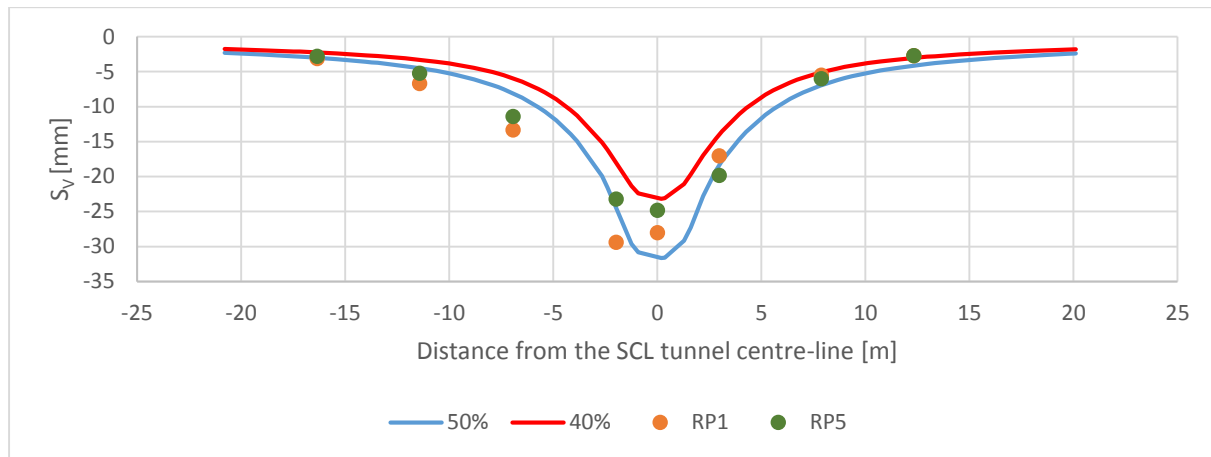


Figure 5.39 - Vertical POT expected displacements for different relaxation factors

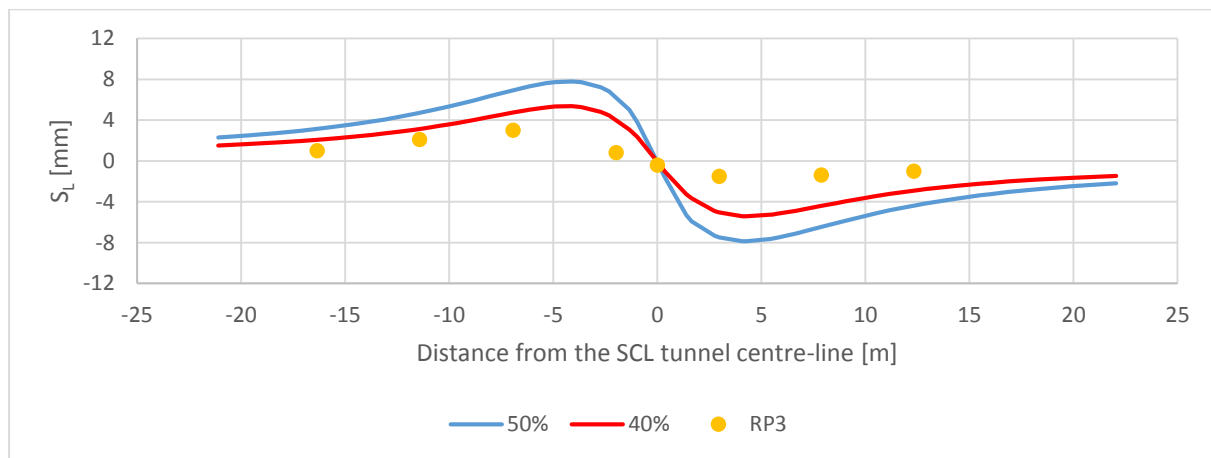


Figure 5.40 - Longitudinal POT expected displacements for different relaxation factors

As it can be seen, a forty per cent relaxation factor is a better approximation to the horizontal behaviour, however the vertical displacements are better represented by the curve formulated based in 50% relaxation, which means that this strategy would improve the results for the horizontal displacements but worsen the vertical ones. Considering this indeterminacy the volume loss was calculated for both models in order to verify which approach would be the best for this situation.

From the Xdisp analysis it was possible to verify that the volume loss assumed, $V_L = 1.5\%$, was a good approximation of the reality since the displacements obtained in that analysis were very similar to those registered by the monitoring system. Therefore the best approach was considered the one with a volume

loss nearest 1.5%. In fact, the model based on $\beta = 0.5$, presented a volume loss of 1.1% whilst the other model 0.8%. In conclusion the first model, also mentioned as the base model, had a good estimate of the relaxation factor.

However, the horizontal displacements still need some improvements, thereafter the in-situ earth pressure was altered based on the model developed with 50% relaxation. As discussed in 4.2.2. the in-situ earth pressure coefficient may assume lower values for analysis where an excavation is undertaken within one diameters of an existing tunnel, hence for this analysis this parameter was assumed as $K_0 = 0.65$ for both London Clay Unit A2 and Upper Mottled layers.

The following figures resume the influence of the above mentioned parameter. It can be seen that in general, for both vertical and horizontal displacements, the results obtained with a lower value of the in-situ earth pressure coefficient are closer to the monitoring data.

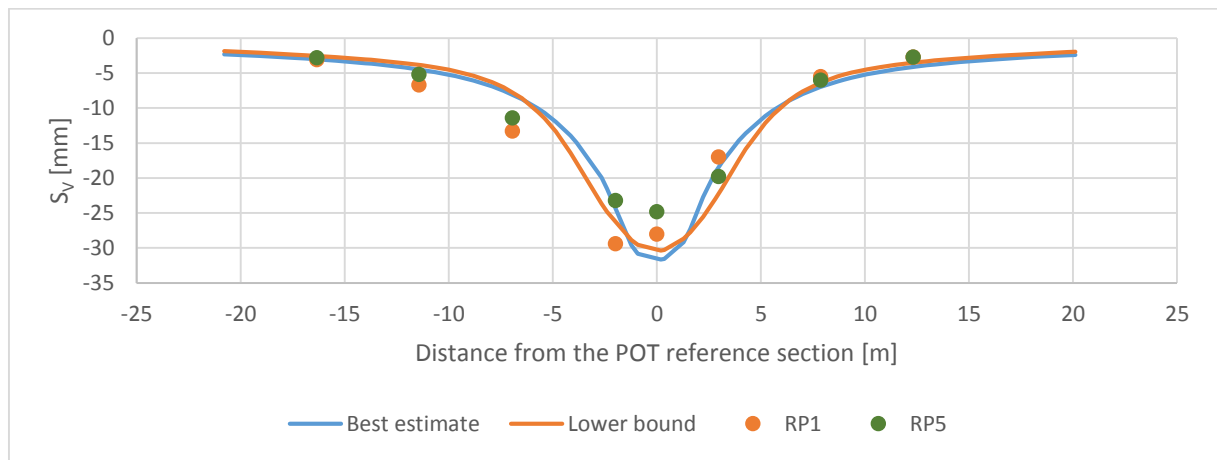


Figure 5.41 - Vertical POT expected displacements for a lower bound of the in-situ earth pressure coefficient, K_0

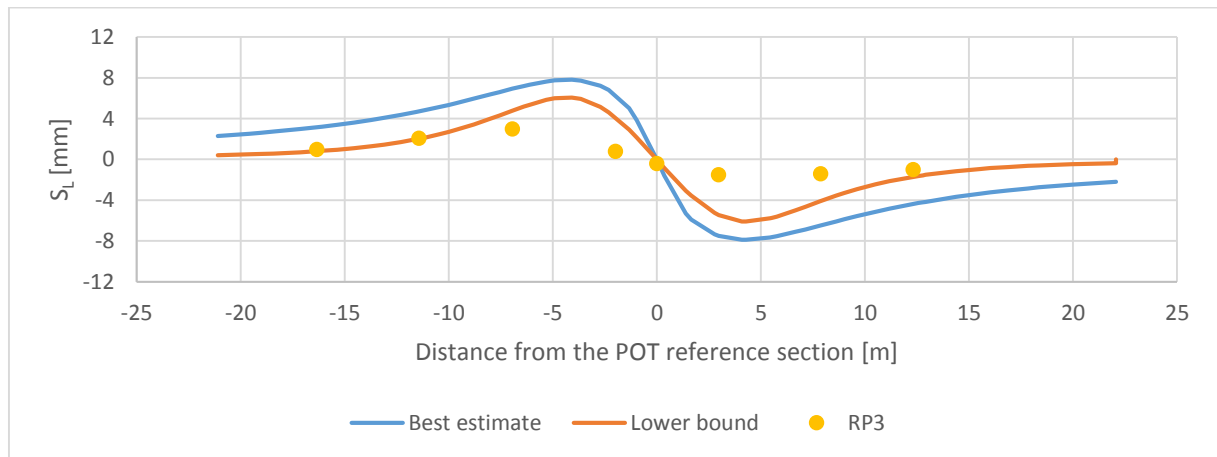


Figure 5.42 - Longitudinal POT expected displacements for a lower bound of the in-situ earth pressure coefficient, K_0

Although the improvement undertaken in this analysis there is still many uncertainties and the limitation of the model do not allow for an accurate prediction. Therefore, the variables considered in this analysis will be further assessed within PLAXIS 3D in the next chapter.

6

THREE-DIMENSIONAL FINITE ELEMENT ANALYSIS

6.1. PLAXIS 3D MODELLING

PLAXIS 3D is a special purpose three-dimensional finite element program that, similarly to PLAXIS 2D, is used to perform deformation and stability analysis.

The model developed in PLAXIS 2D presented in the previous chapter, allowed the author to familiarize with the PLAXIS interface and to develop computational abilities required in this software that revealed to be an advantage in the modelling of the three-dimensional situation, which will be detailed throughout the present chapter.

6.1.1. GEOMETRY AND BOUNDARIES

Before going into any detail it should be clarified that the main aim of this numerical analysis was to control the Post Office Tunnel movements and compare it with the monitoring data obtained during the SCL tunnel construction, in order to develop a back analysis relying on its results. Hence, the 3D model geometry was developed based on the influence of the new excavation on the Post Office Tunnel, thus only the closest part of the new tunnel was modelled. Besides as it was already mentioned in the previous chapter, the author only had access to monitoring data regarding excavation advances in the vicinity of the POT.

Regarding the model dimensions, the first concern was to create a quadrangular model in which it could be conceivable to include both tunnels and achieve the minimum boundaries interference possible. According to the PLAXIS 2D model dimensions which were already introduced, the vertical boundaries should be 70 m away from the new tunnel axis, however for a 3D model this is not efficient since it would lead to a model with thousands of elements and the returning benefits of this option would not be considerable.

Lastly, in order to create a model with realistic dimensions balancing the influence of the boundaries in the tunnel behaviour, Meissner (2002) recommendations were taking into account, i.e., the boundaries were positioned approximately 4 diameters away from the new tunnel axis. Consequently the developed model ended with equal length and width, which is 50 m. Figure 6.1 a) represents the model dimensions created for this analysis.

Regarding the depth of the model, whilst in PLAXIS 2D the deepest layer - Lower Mottled - was modelled with 22 m thickness, in this model the thickness of that layer was assumed 12 m. This difference was taken because 22 m of thickness would lead to a model deeper than pretended, and

considering that the volume of the model has a direct influence on the number of mesh elements created, it was important to optimize the dimensions in order to also optimize the mesh. Besides, later on, the lower boundary proved to have no influence on the results. Lastly, considering the remaining layers' thickness as presented in Table 4.3, the model resulted in 47 m deep, as reproduced in Figure 6.1 a).

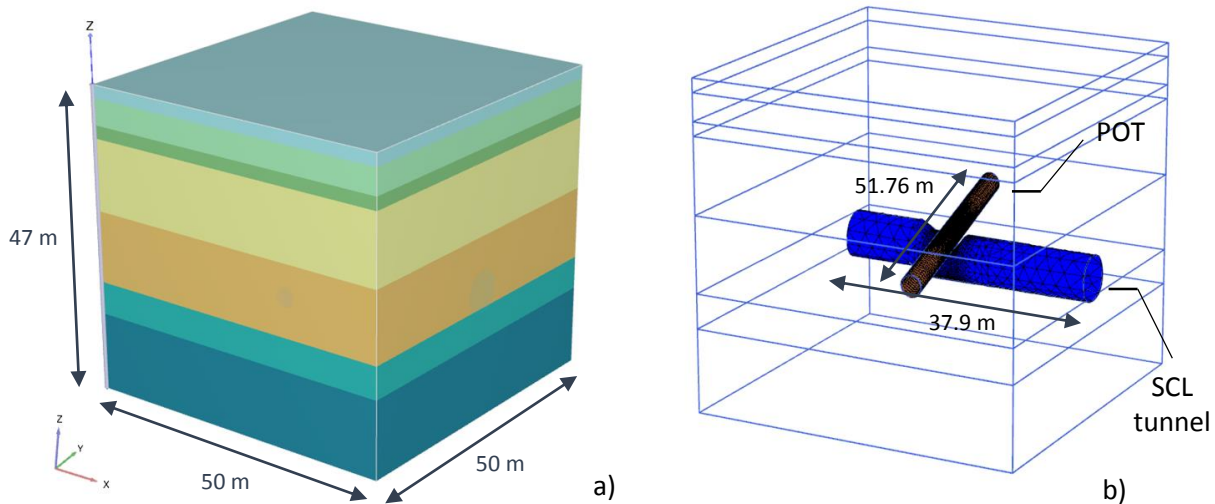


Figure 6.1 - Global view of the 3D model's geometry; a) Ground model dimensions; b) Post Office and SCL tunnels length and relative position

As Figure 6.1 a) may suggest by its colours, the soil layers were modelled based on the same assumptions taken in the 2D model, thus the stratigraphy as well as the constitutive models and its parameters are the same (view section 5.3.1.1.).

The model boundary conditions on the subject of deformations are similar to the applied in PLAXIS 2D, the general fixities imposed to vertical boundaries with their normal in x -direction are fixed in x -direction and free in the other directions, and for vertical boundaries with their normal in y -direction they are fixed in y -direction and free in x - and z -direction. The model bottom boundary is fixed in all directions and the 'ground surface' is free in all directions. This model's axis reference is represented within Figure 6.1.

The tunnels length (see Figure 6.1 b)) were assumed based on the few information provided to the author, hence the length adopted for the new tunnel was selected assuring that the initial and the final part of the tunnel, i.e. the boundaries of the tunnel, would not affect the interaction between both tunnels. Note that the SCL tunnel axis is aligned with the x -direction.

Whilst the POT was directly designed on the software since it was roughly modelled as a simple cylinder with 3 m diameter, the challenging SCL tunnel geometry was imported from an AutoCAD model, which was constructed based on the cross-sections represented in Figure 4.7. Some simplifications were made in the tunnel's geometry in order to design a tunnel the most uniform possible or it would be impossible for PLAXIS to generate an acceptable mesh for this tunnel. As a result, the tunnel external surface was constructed with the transition sections between the two different cross-sections perfectly connected through a continuous and uniform surface, as reproduced in Figure 6.2.

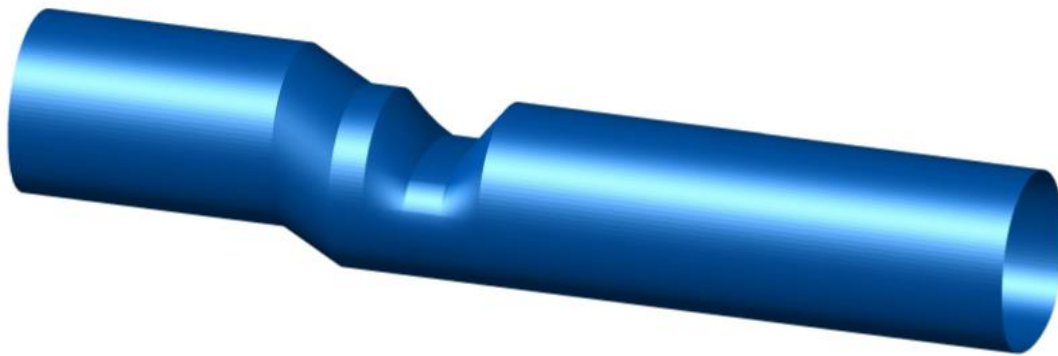


Figure 6.2 - SCL tunnel external surface developed in AutoCAD

Before importing the tunnel external surface, an AutoCAD mesh had to be generated so that PLAXIS could read the geometry without any problem. For this process a triangular mesh was created considering a maximum angle between new faces of 40 degrees, following the recommendations presented in the PLAXIS knowledge base. Figure 6.3 shows the generated mesh in AutoCAD as well as the SCL tunnel dimensions within each section. It is possible to identify the lower part of the tunnel surface, i.e. the tunnel's invert, is completely continuous which allowed PLAXIS to create an optimized surface even though the upper part was still complex.

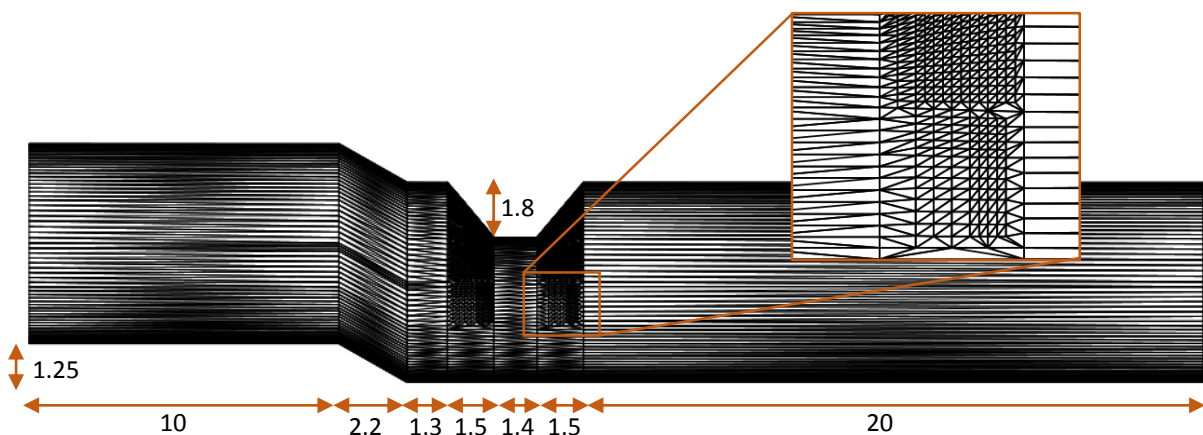


Figure 6.3 - View of the AutoCAD triangular mesh and indication of the SCL tunnel dimensions (in meters) for the 3D model

In order to model the excavation sequence, which comprises the 1 m meter advances as they were performed in the reality and the top heading and invert excavation plan, new surfaces were added later in PLAXIS to the SCL tunnel external surface. Although the excavation face was in reality constructed using a curved surface as shown in Figure 4.8, the model does not include this aspect since it would exacerbate the complexity of the PLAXIS mesh to be generated, compromising the efficiency of the model. However, the author assumed that modelling the excavation face in each advance as a vertical plan surface would not affect significantly the accuracy of the results.

Furthermore the location of each advance was slightly altered from the real location in order to simplify the model and achieve an optimized and efficient finite element mesh. The real location of the

excavation face as it may be seen in Figure 4.8, in some advances was too close to the crossing section, consequently too close to some vertices but not perfectly aligned with them, which as it will be explained later, could lead to numerical problems. Thus the author assumed small simplifications as result of this concern.

The vertical surfaces were designed in PLAXIS and they consisted in square surfaces intersecting the new tunnel itself, see Figure 6.4 a). This procedure was taken so that PLAXIS could perfectly intersect all surfaces preventing the creation of any tiny elements or empty spaces between the surfaces. After intersecting the tunnel external surface and the vertical surfaces, the residual surfaces was deleted avoiding its interference in the mesh generation.

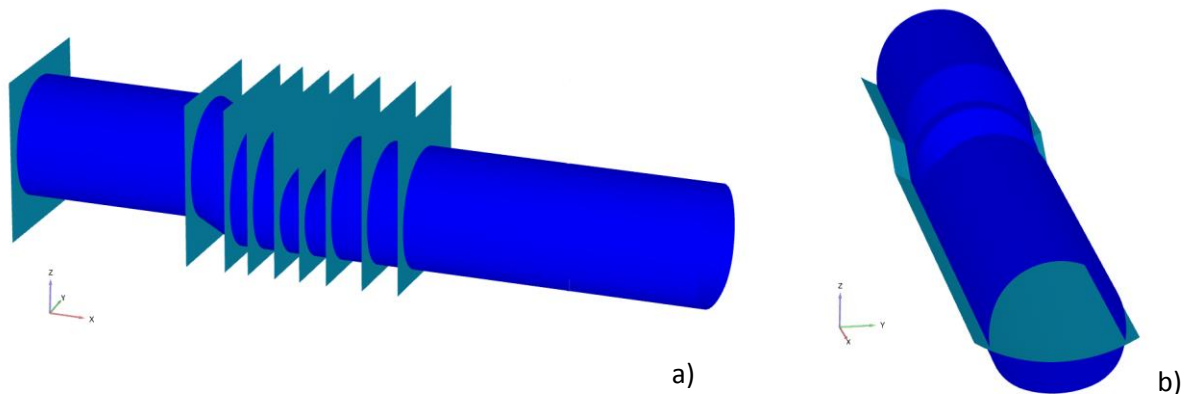


Figure 6.4 - Intersection procedure in PLAXIS; a) Vertical surfaces; b) Top heading/ invert excavation dividing surface

Regarding the curved surface that divides the top heading and the invert excavation, it was also created in AutoCAD as it is represented in Figure 4.7. The creation of this surface directly in PLAXIS was not feasible since the software did not intersect the horizontal part with the inclined one correctly. Hence a new AutoCAD model was developed creating a continuous surface along the SCL tunnel, which consisted on a slightly curved surface extended beyond the new tunnel external surface. After separately importing this surface to the structure already created in PLAXIS, this detail, i.e. the slightly larger surface, was fundamental to intersect all surfaces assuring that the structure was perfectly connected and any atypical element was created. Lastly the residual surfaces were deleted.

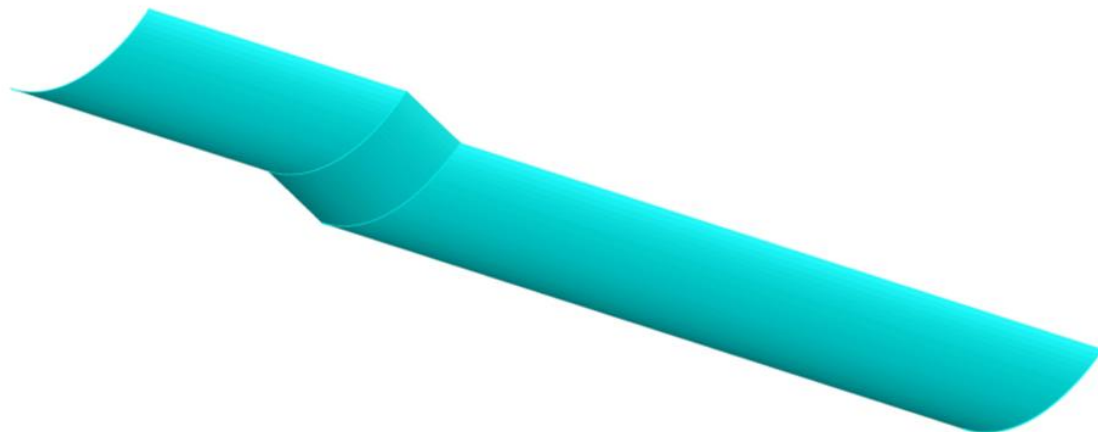


Figure 6.5 - Top heading/ invert excavation dividing surface created in AutoCAD

One last aspect of the geometry which was also very important regarding the mesh generation was the POT depth. Considering the information provided, the base of the POT was expected to be at 97.5 mLUD, which means that it would be almost tangent to the SCL tunnel crown on the crossing section, however regarding the assumed geometry for the Underpass (see Figure 6.3), the POT could not be created tangent to the SCL tunnel crown or it would intersect the inclined surfaces. Besides, PLAXIS would not allow a mesh to be generated in this circumstances, this software would intersect the two tunnels instead of creating them as two separate structures or would not even be able to generate a mesh. In fact as it will be explained in the following section, the position of the POT also represented an obstacle regarding the process of mesh generation due to its close proximity to the SCL lining. Finally, both tunnels were created 60 cm distant from each other as Figure 6.6 suggests, this relative position was optimized since it was the one that allowed the software to generate a mesh, if the tunnels were closer than this, as they were in the reality, PLAXIS would not generate a mesh.

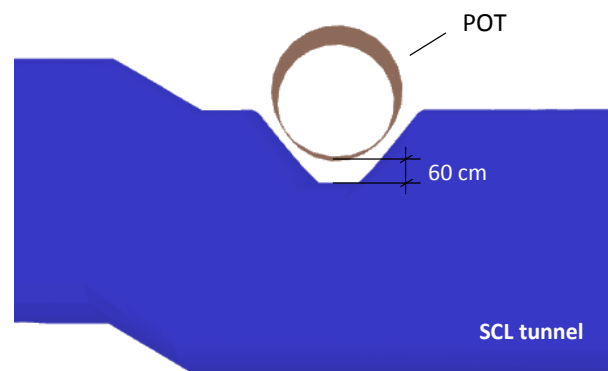


Figure 6.6 - Post Office and SCL tunnels relative position, reproduced from PLAXIS 3D model

6.1.2. MESH GENERATION

Before proceeding with the definition of the construction stages, a mesh has to be generated which takes into account the soil stratigraphy as well as all structural elements and loads. According to the recommendations presented in PLAXIS Reference Manual (PLAXIS, 2015c), the mesh should be “sufficiently fine to obtain accurate numerical results” but no too fine as it can “lead to excessive calculation times”. As the previous section may suggest, the geometry of this problem is complex and there are specific sections on the imported surfaces that led to problems when generating the mesh. As follows the mesh generation was an iterative process that comprised geometry simplifications, coarseness factors modifications and element properties adjustments that will be identified in the following paragraphs.

To model soil layers, PLAXIS 3D uses 10-node tetrahedral elements. In addition, special types of elements are used to model structural behaviour. For plates used to simulate tunnel linings, 6-node plate elements are applied which are compatible with the 6-node face of a soil element. Moreover 12-node interface elements are used to simulate soil-structure interaction behaviour.

In order to identify any problem in the geometry, as for example intersection problems or tiny elements, an initial attempt to create a coarse mesh was carried out. At this stage the mesh was generated using the default properties presented by PLAXIS to a coarse element distribution. Since the geometry in the transition section is too complex, PLAXIS was not able to adapt coarse elements to the tunnel surface, hence no mesh was generated. A finer mesh was developed to the same purpose and still the software

could not generate it. This first strategy allows to identify geometry problems and correct them before any mesh refinement, therefore saving time.

After performing some adjustments in the geometry, which consisted in slightly modifying the advances location, as for example aligning them with the tunnel vertices, or modifying the POT position as it was already introduced in the preceding section, alongside with the decrease of the polyline angle tolerance to allow the program to create elongated elements which could adapt to the tunnel surface, it was possible to generate a mesh.

However, as the following figure suggests (Figure 6.7), the created plate elements were not equilateral triangles, which led to calculation problems. Since the elements were badly shaped when a calculation was performed, PLAXIS returned an error identifying that the jacobian matrix was less than or nearly zero. The solution was to create a better shaped mesh without the elongated elements that were present in the last mesh.

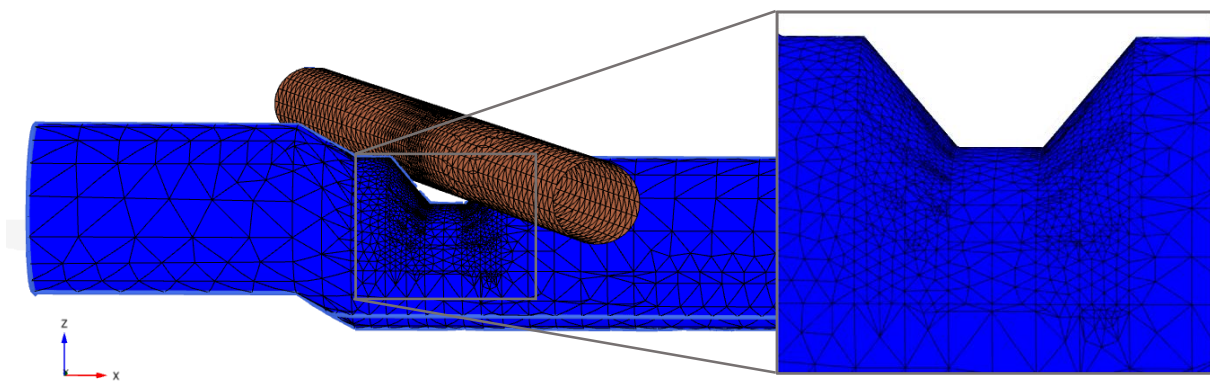


Figure 6.7 - Example of a badly shaped mesh

In order to achieve a better solution, the polyline angle tolerance was now increased to 35 degrees, as well as the surface angle tolerance. This procedure prevents any elongated elements to be created, assuring that the elements are approximately equilateral triangles. Notwithstanding, this procedure was only effective concomitantly with the modification in the coarseness factor on the most complex part of the tunnel lining. Figure 6.8 represents the coarseness factors adopted for each part. In order to create a progressive mesh, i.e. a mesh in which the elements size increases from the underpass section to the edges of the tunnel, resulting in a more continuous and therefore in a better shaped mesh, the coarseness factor followed the same idea, ranging from 0.08 in the centre of the tunnel (transition sections) to 0.5 near the edges.

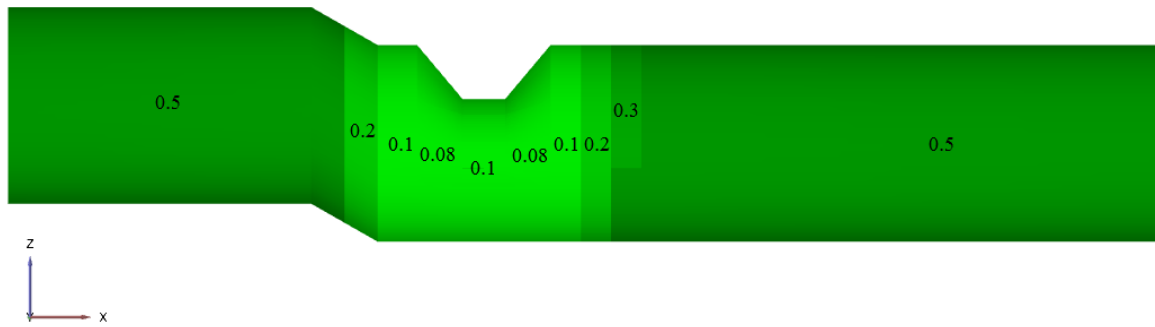


Figure 6.8 - Coarseness factors adopted for each section of the SCL tunnel

In addition, the coarseness factor was also modified in the POT plate, mainly in its centre. This modification was necessary since the elements near this section were really small as a result of the proximity of both tunnels and the coarseness factors used for the Underpass.

Some attempts were carried out using these simplifications although not all the combinations were successful. For all the attempts that PLAXIS was able to generate a mesh, the quality of the mesh was inspected. As expected for much attempts the mesh quality was really low, which encouraged the author to search for a better combination. Lastly the author managed to create a mesh consisting of a total of 99033 elements and 127383 nodes, which was adopted as the basis of the base model to be developed. Figure 6.9 represents the mentioned mesh. As it can be seen, in comparison with the mesh presented in Figure 6.7, the triangular elements are much more equilateral and the transition between elements dimensions more gradual, which leads to an improvement on the accuracy of the results as well as in the calculation time.

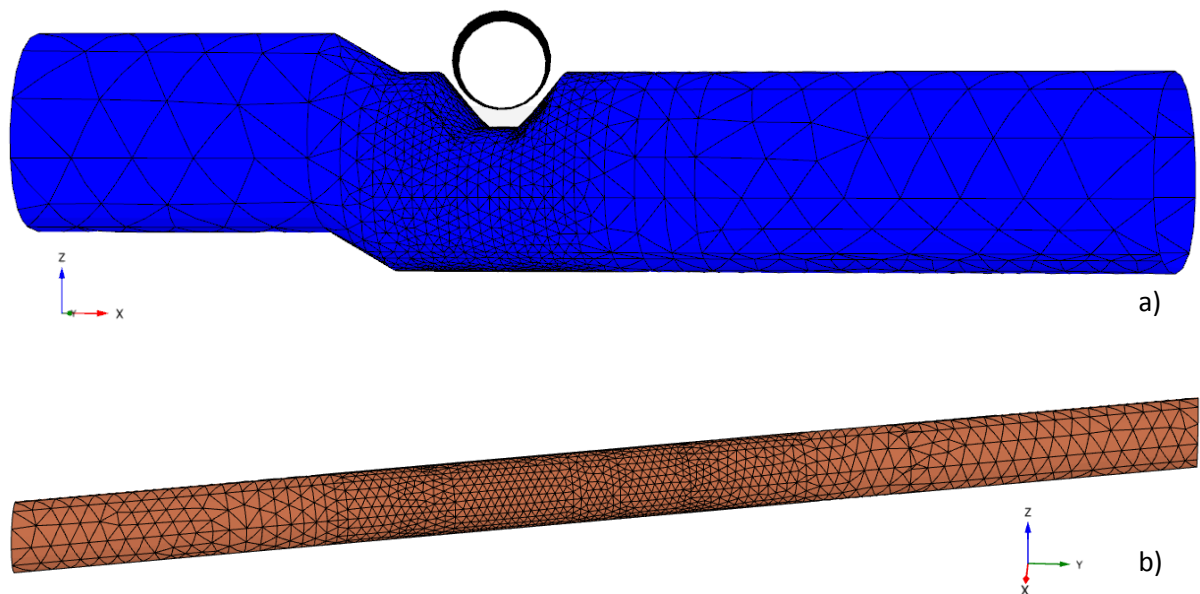


Figure 6.9 - Final mesh adopted for the base model; a) SCL tunnel lining' mesh; b) POT lining' mesh

Figure 6.10 represents the poorest elements found on the adopted mesh. As it is possible to identify the poorest elements are the ones located between the POT lining and the top of the soil layer where it is

installed. The top of the soil layer works as a boundary forcing the elements to a specific size, hence this affects its quality. The same problem occurred underneath the SCL tunnel but not with the same intensity.

Apart from these poor elements, most of the mesh is acceptable and its coarseness is satisfactory. Other attempts in which the coarseness was even better led to thousands of elements and therefore to excessive calculation times. The base model balanced these two aspects resulting in a suitable mesh for its purpose.

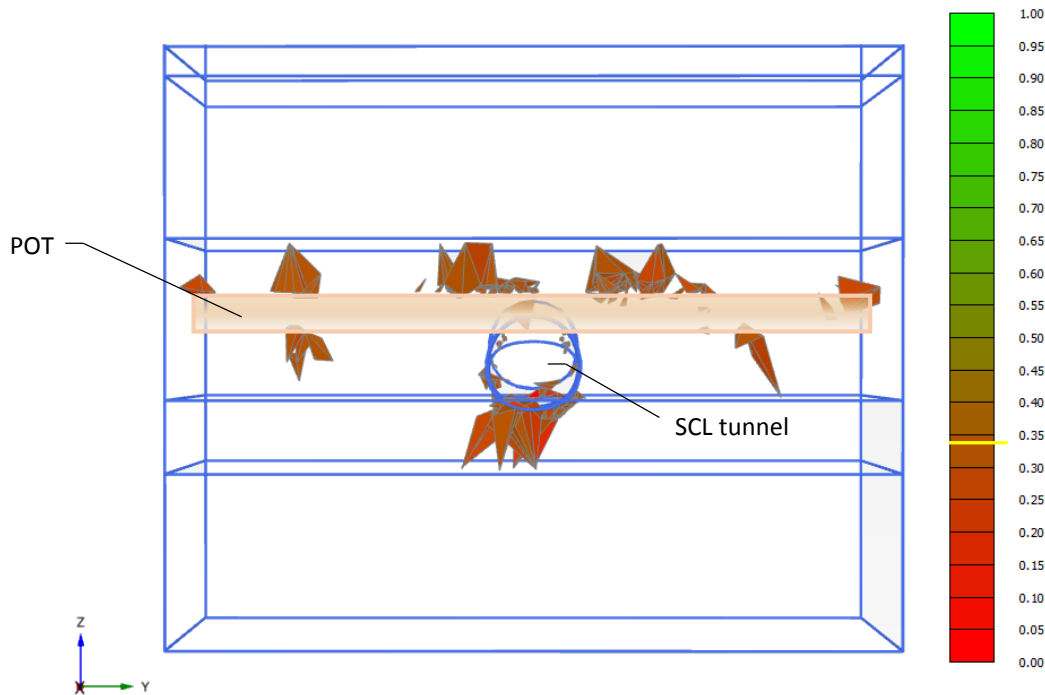


Figure 6.10 - Representation of the mesh quality scale

6.2. EXCAVATION SEQUENCE

The first stage of the tunnel excavation was, as done in PLAXIS 2D, the generation of the initial stress field through the K_0 procedure, which was already explained in 5.3.2. The following stage, as modelled also in PLAXIS 2D, corresponds to the application of the surcharge.

The 3D model's major advantage lies in the fact that it allows to investigate the interaction between the POT and the excavation of the new SCL tunnel. Therefore, the next step corresponds to the construction of the POT in the ground model and generating the correspondent stress field, in order to posteriorly initiate the excavation of the new tunnel. This step is hereafter referred as *POT construction stage*.

As the model description already suggested, the POT lining was simulated in PLAXIS 3D as a plate with uniform thickness. The formulation of the equivalent thickness and therefore the equivalent Young's modulus are presented below. The properties of the cast iron tunnel segment's section assumed for this analysis were those presented in section 4.1.2.

Regarding the section dimensions presented in Table 4.1, the resultant area and the second moment of area of the lining section are summarized in the following table.

Table 6.1 - Actual elastic section properties of the POT lining

Parameter	Unit	
Area of section per meter length, A	m^2/m	2.85×10^{-2}
Second moment of area per meter length, I	m^4/m	2.829×10^{-5}

In addition, the Young's modulus for the cast iron lining, E , was assumed equal to 100 GPa , also based on the information from the LUL reports (CH2M, 2013). Hence, in order to determine an equivalent thickness, t_{eq} , both flexural rigidity, EI , and normal stiffness, EA , were assumed constant. Thus, for one meter length of tunnel:

$$EI = E_{eq} \cdot I_{eq} = E_{eq} \cdot \frac{t_{eq}^3}{12} \quad (6.1)$$

and,

$$AE = E_{eq} \cdot t_{eq} \quad (6.2)$$

where E_{eq} and I_{eq} represent the equivalent Young's modulus and the equivalent second moment of area, respectively. Dividing Equation 6.1 by 6.2 results in the following equation.

$$I/A = t_{eq}^2/12 \Leftrightarrow t_{eq} = \sqrt{12 \cdot I/A} \quad (6.3)$$

Regarding the parameters presented in Table 6.1, from the Equation 6.3 results $t_{eq} = 0.109m$. Therefore, considering Equation 6.2, which can be presented as:

$$E_{eq} = AE/t_{eq} \quad (6.4)$$

results $E_{eq} = 26.11 GPa$.

This formulation was adopted so that the POT lining could be modelled as a monolithic structure, the most accurately possible. The parameters inputted in the software are represented in Table 6.2. Note that this formulation was based on the longitudinal section dimensions of a POT segment, hence the equivalent Young's modulus calculated through this formulation corresponds to the radial stiffness of the POT. However, at this stage the stiffness of the POT was assumed similar radially and longitudinally since the POT lining was modelled with isotropic behaviour.

Note should be taken that although the POT lining formed by bolted cast iron segments, the lining joints will not be modelled within this analysis due to limitations of the software which has not an efficient way of reproducing those joints. Although PLAXIS 3D allows connections to be created, the user has

to manually reproduce each segment by dividing the POT plate into segments of a specific length, which in this case would be 508 mm. Considering that the POT was modelled with approximately 52 m it would be necessary to define approximately 100 sections in order to reproduce the longitudinal joints, besides it would be even necessary to reproduce the radial joints by diving each created segment ring in 6 different plates. This procedure would be excessively time consuming and considering the time available for the production of this thesis was not considered feasible. Thereafter, the POT lining was modelled as a simple monolithic structure which is the industry's standard practise.

Table 6.2 - Cast Iron lining properties used in PLAXIS 3D

Parameter	Unit	Cast Iron lining
Material behaviour	-	Elastic, Isotropic
Equivalent thickness, t_{eq}	m	0.109
Weight, γ	kN/m^3	71
Equivalent Young's modulus, E_{eq}	kN/m^2	26.11×10^6
Poisson's ratio, ν	-	0.26
Shear modulus, G	kN/m^2	10.36×10^6

The POT modelling in PLAXIS did not take into account the construction process of this tunnel. This simplification was assumed because there was no information available from the POT construction, and its correspondent movements, besides it will not affect significantly the results from the SCL tunnel excavation, which is the main aspect of the 3D model. In the end, POT was simulated through a cylindrical excavation, crossing the entire model in its specific relative position to the SCL tunnel, (i.e. 15° deviated from the perpendicular to the SCL tunnel future position), simultaneously with the application of its lining.

The POT installation itself induced settlements around the tunnel that do not need to be considered, since only the displacements induced in the POT by the SCL tunnel excavation will be analysed. In fact this displacements were cleared from the model in order to allow a direct comparison between monitoring data and PLAXIS results regarding POT displacements. In conclusion, by resetting the displacements in the model at this stage, it results that the POT displacements will only be the result of the new tunnelling works, which is the aim.

As was already introduced, the SCL tunnel excavation sequences do not perfectly follow the real excavation plan, however all simplifications follow conservative procedures which the author believed not to have significant influence on the analysis goals. Figure 6.11 represents the excavation sequence created for this purpose and adopted throughout the analysis.

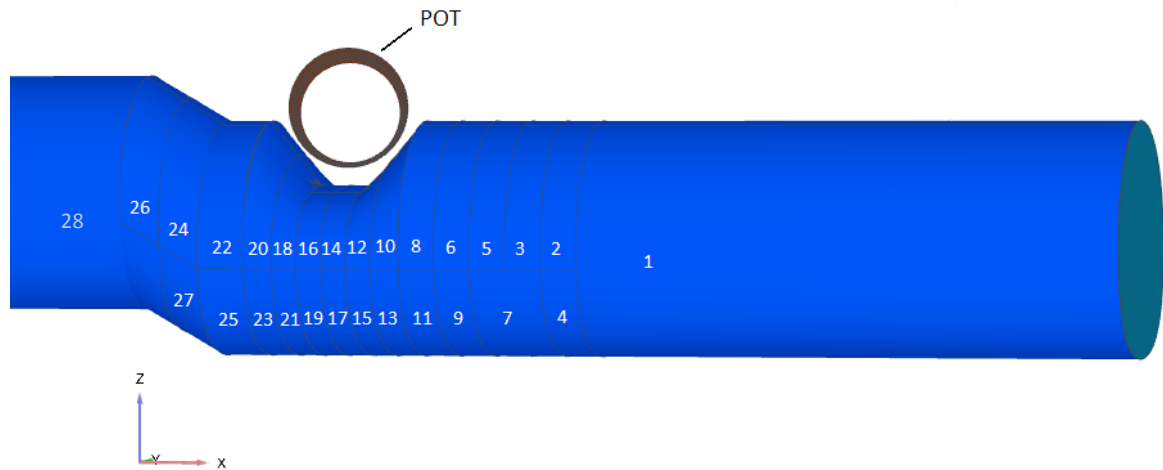


Figure 6.11 - Representation of the excavation advances modelled in PLAXIS 3D

Table 6.3 clarifies the position of each advance regarding the POT centre-line. All distances were defined from the excavation face of each advance to the POT centre-line, which coincides with the face of the advance No. 14.

Table 6.3 - Location of the SCL tunnel advances from the POT centre-line

Advance No.	Horizontal Distance from POT Centre-Line [m]	Advance No.	Horizontal Distance from POT Centre-Line [m]
1	7.2	15	Same as advance No. 12
2	6.2	16	-0.7
3	5.2	17	Same as advance No. 14
4	Same as advance No. 2	18	-1.4
5	4.2	19	Same as advance No. 16
6	3.2	20	-2.2
7	Same as advance No. 5	21	Same as advance No. 18
8	2.2	22	-3.5
9	Same as advance No. 6	23	Same as advance No. 20
10	1.4	24	-4.6
11	Same as advance No. 8	25	Same as advance No. 22
12	0.7	26	-5.7
13	Same as advance No. 10	27	Same as advance No. 24
14	0	28	-15.7

The excavation process was divided in two different stages for each advance: the first stage simulates the excavation and the second stage the application of the concrete lining. It was assumed that the soil and initial ground support deforms to equilibrium after each 1 m advance before the primary sprayed lining is applied, furthermore no time effects was taken into account for the PLAXIS plastic calculations. As this process consists in the excavation of 1 m advances, shifting between top heading and invert excavation, the support is immediately applied 1 m behind the excavation face. It should be referred that this process was not applied in advances No. 1 and No. 28, in which the excavation and the application of the lining were modelled within the same stage. It was assumed that detailing this advances as much as the remaining excavation would lead to a surplus of unnecessary information to be processed without any relevance to the analysis in hand. Regarding advance No. 1, its only purpose was to locate the excavation face in the area from which the monitoring plan started recording the influence of the new tunnel excavation on the POT, thus it was not necessary to detail the excavation throughout its total length. Besides, modelling simultaneously the excavation and the application of the lining did not interfere significantly with the POT displacements. With respect to advance No. 28, it was uniquely developed to finish the excavation of the modelled tunnel, since the information regarding the monitoring system only recorded the influence until the corresponding advance to the No. 27.

Similarly to the POT lining, the SCL tunnel concrete lining was also modelled as a plate, whose properties and material model adopted, were the same used in the model developed in PLAXIS 2D (see Table 6.4). Although PLAXIS 3D allows for anisotropic material behaviour in plate elements, the concrete lining was modelled as an isotropic material.

Table 6.4 presents as well the properties adopted for the temporary concrete lining, which is applied in the excavation face, in each advance, and in the temporary invert when a top heading excavation is performed. This lining is only applied subsequently to the excavation in order to model the ground arching effect mobilization and the delay in closing the face and the invert. Posteriorly, this lining is removed so that another excavation advance is performed.

Table 6.4 - Concrete lining properties as used in PLAXIS 3D

Parameter	Unit	Primary concrete lining	Temporary concrete lining
Material behaviour	-	Elastic, Isotropic	Elastic, Isotropic
Thickness, d	m	0.25	0.075
Weight, γ	kN/m^3	25	25
Young's modulus, E	kN/m^2	30×10^6	30×10^6
Poisson's ratio, ν	-	0.2	0.2
Shear modulus, G	kN/m^2	12.50×10^6	12.50×10^6

Although the most commonly used constitutive model for sprayed concrete is a linear elastic one with a constant stiffness (Thomas, 2009), this material exhibits a very complex behaviour, whilst in tension sprayed concrete is initially a linear elastic material, when under compression, in its hardened and hardening form, it behaves as a nonlinear elastoplastic material. Besides, as it is evident, the stiffness of the sprayed concrete varies considerably during its early ages (Thomas, 2009).

In order to improve this simple elastic analysis Thomas (2009) suggests to incorporate the increase in magnitude of the concrete stiffness with age. In fact, as it is a time-dependent material the best approach

to model it in PLAXIS, is to consider different stages for the concrete lining stiffness. Based on the company's experience two stages were considered for the concrete strengthening, in which was assumed that the sprayed concrete reaches its total stiffness after two advances are performed. For the first stage, which refers to the application of the lining right after the excavation, the concrete lining stiffness was adopted with 1/3 of its total, thereafter in the ensuing advance the same lining section reached now its total stiffness, whereas the stiffness of the lining to be applied in the new advance is again considered with 1/3. Figure 6.12 clarifies this procedure. The lining represented in green has a Young's modulus equal to 10 GPa (applied after the excavation stage) and the blue lining corresponding to full stiffness, 30 GPa.

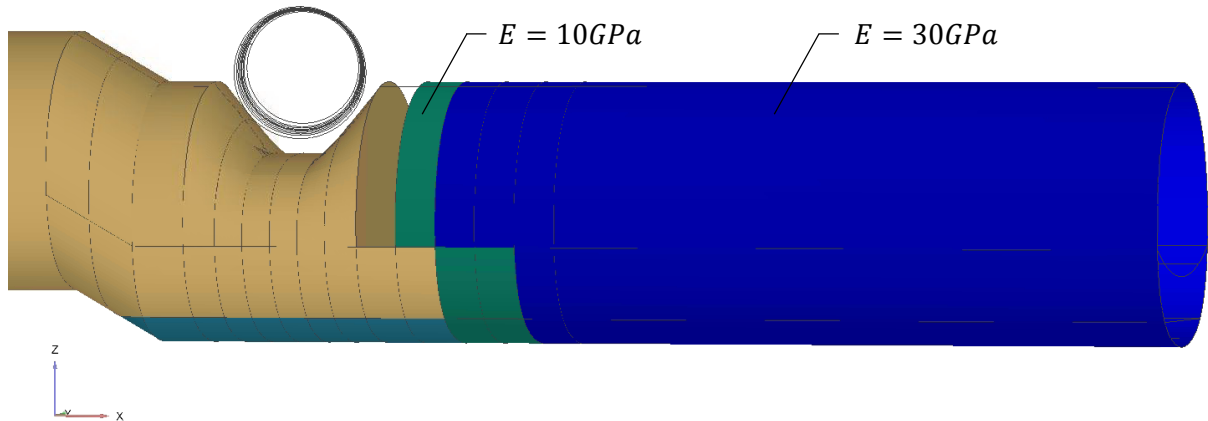


Figure 6.12 - Representation of the different lining's stiffness to model concrete strengthening

Figure 6.13 corresponds to a three-dimensional view of the excavation process, where it is possible to identify the different stages of the sprayed concrete, as well as the sprayed concrete that is applied in the excavation face between consecutive excavations. The concrete applied in the face of the excavation was also modelled with only 1/3 of its total stiffness, i.e. 10GPa.

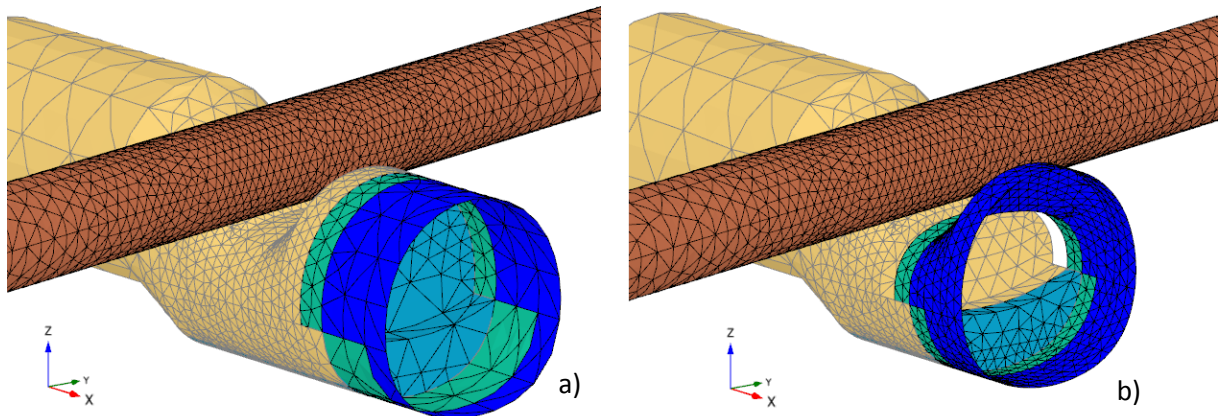


Figure 6.13 - Three-dimensional view of excavation sequences; a) Application of the lining at advance No. 7; b) Excavation at advance No. 16

6.3. RESULTS - BASE MODEL

6.3.1. SOIL BEHAVIOUR

6.3.1.1. Initial deformations - Post Office Tunnel construction

As already mentioned the simulation of the POT construction consisted on excavation and construction of the lining simultaneously. This strategy was adopted once it was necessary to induce in the model a stress state similar to the one experienced by the soil in the beginning of the SCL tunnelling works. Figure 6.14 corresponds to a cross section intersecting the middle of the POT, thus there is no influence of the boundaries on its behaviour. The scale of the deformed mesh represented in the figure is clearly exaggerated in order to understand more accurately the behaviour of the soil around the POT.

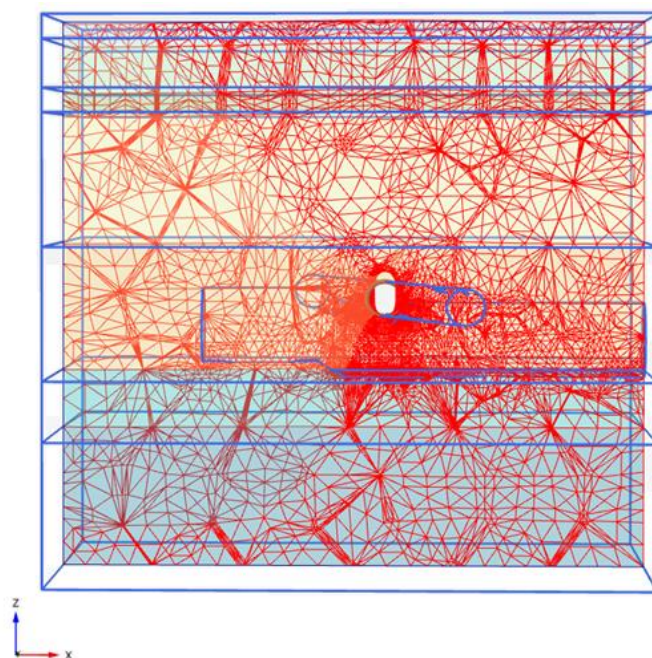


Figure 6.14 - Deformed mesh in a cross section perpendicular to POT corresponding to *POT construction stage* (scaled up 700 times)

Although the lining stiffness is considered to be uniform through its section, as it can be seen in Figure 6.14 and also in Figure 6.15, that there is a small difference between the vertical and horizontal displacements, which is related with the in-situ earth pressure coefficient, K_0 . The mentioned figures allow one to identify heave on the crown of the POT, which reached a maximum of 0.8 mm, whereas the sides converged 0.6 mm into the tunnel. This small difference is identified because for London Clay, K_0 usually assumes values greater than one, which results in higher horizontal stresses compared with the vertical stresses, leading to higher horizontal displacements.

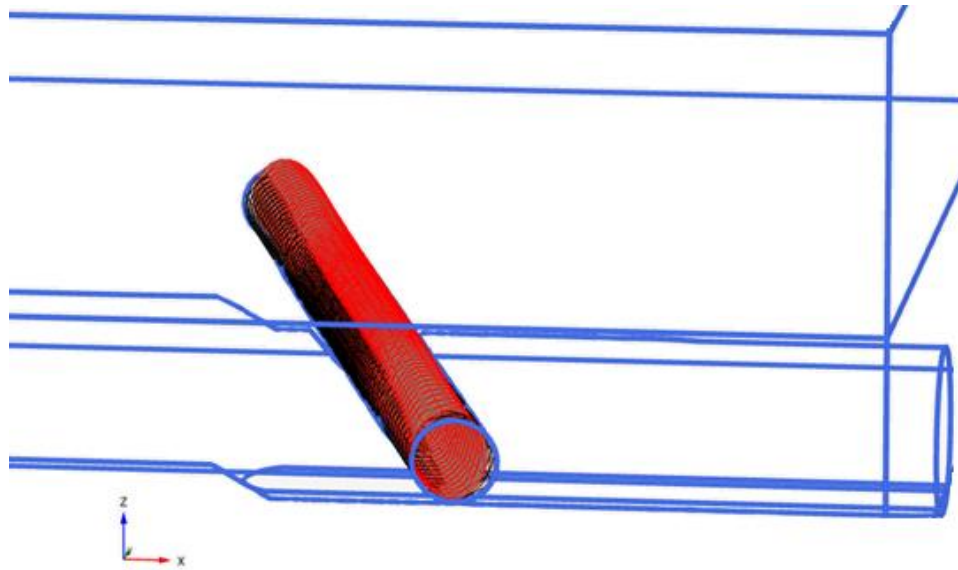


Figure 6.15 - POT deformed lining after POT construction (scaled up 400 times)

Note should be taken that this initial small displacements around the POT will not be considered in the following stages. As mentioned in the previous section, the displacements of this stage were reset to zero because they correspond to initial and former movements, therefore not considered in the current monitoring data, i.e., the monitoring system started recording the POT displacements some weeks before the new tunnelling works had begun, thus this procedure ensure that the displacements obtained by PLAXIS in the POT, due to the excavation of the new tunnel, have the same reference as the monitoring system, i.e. initial zero displacements. This stage was created mainly to induce the stress state due to the presence of the POT in the new excavation.

6.3.1.2. Deformed after SCL tunnel construction

The deformed mesh presented in Figure 6.16 summarizes the behaviour of the soil due to the excavation of the new tunnel. It represents the excavation in the final stage, i.e. in advance No. 28, which means that the critical part of the excavation was already performed. Once again, the scale is exaggerated in order to understand the global behaviour of the soil. The deformed mesh represents total displacements.

In the presented figure the first layers of the soil were hidden in order to identify the ground movements around the tunnel excavation more easily, since the displacements at the ground surface are insignificant, almost not identified, due to the dissipation of the vertical displacements away from the excavation as discussed in Chapter 2.

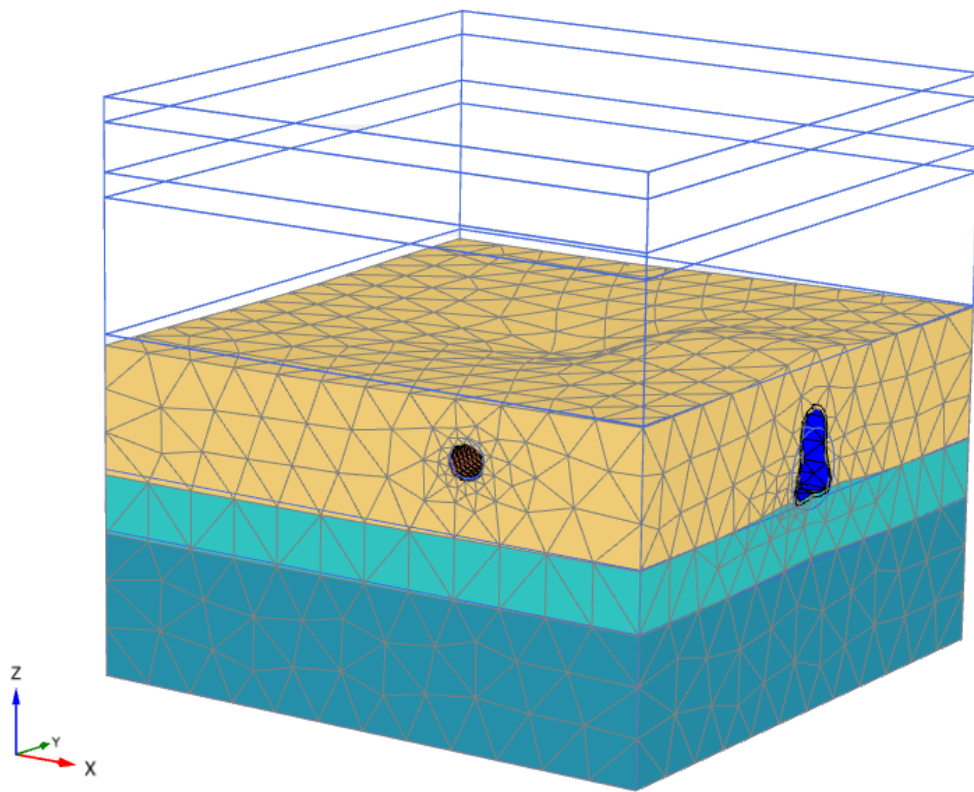


Figure 6.16 - Deformed mesh of part of the 3D model in the final stage of the excavation (scaled up 400 times)

As expected, the soil in the vicinity of the SCL tunnel converge inwards the excavation, however it is possible to identify that the horizontal displacements are higher than the vertical ones, mainly in the part of the tunnel nearest the boundary. This behaviour is explained by the influence of the K_0 in the soil. Essentially, it can be identified some heave in the soil above the excavation and also beneath it, however this last movement is explained due to removing a large volume of the soil. This process results in a decreasing of the vertical stresses which allows the soil below the excavation to heave. The heave in the crown in the first part of the tunnel is identified because the excavation and the support was applied at the same time, strategy used by the author to be able to excavate a large volume of the soil without compromising the tunnel stability and the results from the tunnel excavation process near the POT. This strategy does not allow the soil to settle because the lining is built simultaneously with its total stiffness. Hence, the movements of this part of the tunnel, which corresponds to the advance No. 1 should not be taken into account.

Nearest the POT the soil behaviour is different. There is no heave and the movement of the soil is completely towards the excavation. The presence of the POT reduces the volume of the soil available above the SCL tunnel which decreases its global strength and also considering that the POT lining weight is not insignificant, results in the soil between the two tunnels to move towards the SCL tunnel excavation.

Right after the crossing section some heave is identified again. Note that this heave is once again caused by the strategy applied to excavate the remaining tunnel which is excavating and applying the lining in the same stage (corresponding to advance No. 28 as per Figure 6.11).

6.3.1.3. Ground displacements

The present subsection aim to characterize the soil displacements in the vicinity of the SCL tunnel after the excavation is completed, hence the results discussed within this subsection were obtained regarding the completion of advance No. 27, unless otherwise stated. Note that advance No. 28 will not be taken into account as already explained.

Figure 6.17 represents a cross section 4.2 m distant from the POT centre-line, approximately located in the area of advance No. 5. This highlights the movement of the soil explained previously, i.e., heave in the base of the excavation and settlement of the crown.

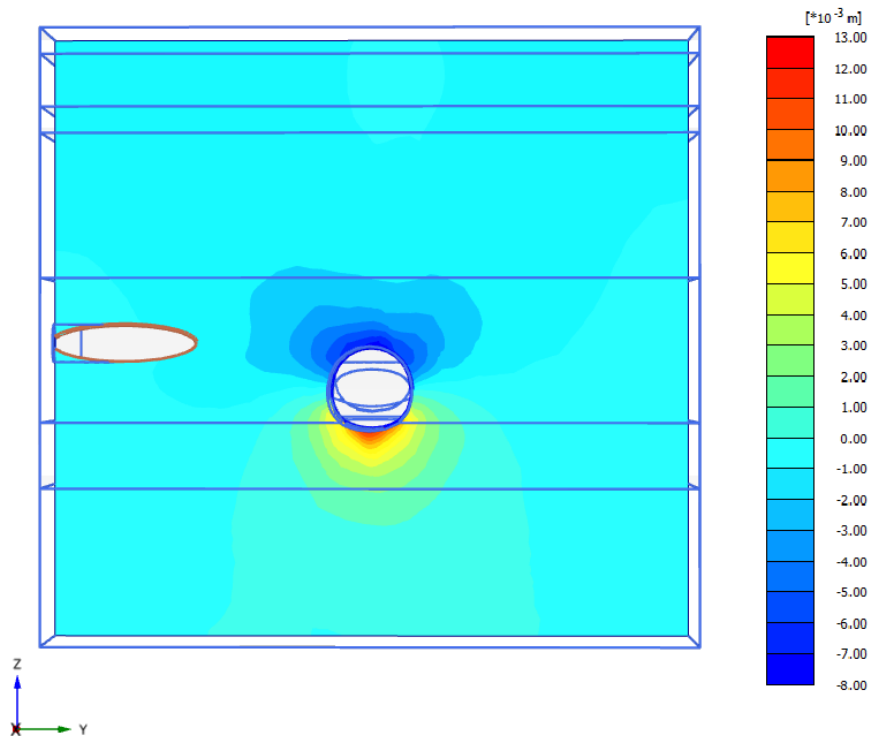


Figure 6.17 - Vertical displacements in a cross section perpendicular to SCL tunnel and 4.2 m distant from the POT centre-line (u_z)

It can be seen that the heave effect is more important than the settlement, reaching 13 mm in comparison with the 7.5 mm of vertical displacement in the crown of the tunnel. This difference may be related with the fact that the invert excavation, in particular the advance No. 7, corresponds to a 2 m advance which allows the soil to move more in comparison with the 1 m advances. Besides, unlike advanced constitutive material models, the Mohr-Coulomb model does not consider different types of stiffness modulus to predict the soil behaviour during loading, unloading or reloading, thus some displacements may be different to the measured in the reality. In fact, Mohr-Coulomb model may indicate a heave effect during tunnel construction which is not in accordance with the measured in the reality (Vakili, 2014). In the lateral part of the tunnel vertical displacements are null.

Regarding the horizontal displacements perpendicular to the tunnel axis, represented in Figure 6.18, they are approximately symmetric in relation to the vertical tunnel axis, reaching a maximum of 16 mm towards the excavation in this part of the tunnel.

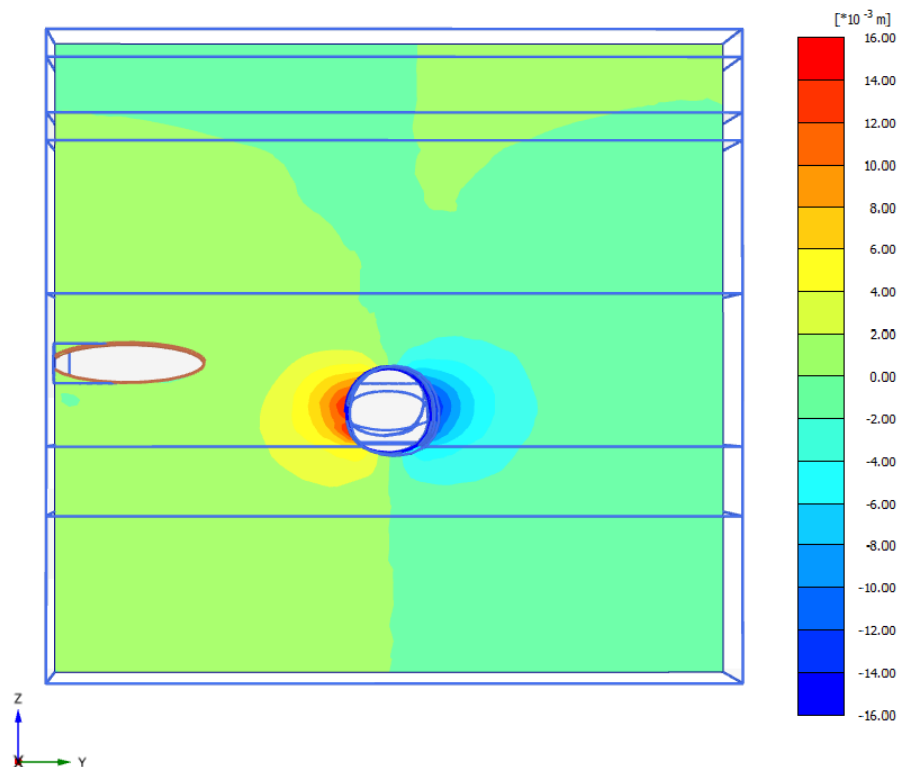


Figure 6.18 - Horizontal displacements in a cross section perpendicular to SCL tunnel and 4.2 m distant from the POT centre-line (u_y)

In the undercrossing section, whose soil displacements are presented in Figure 6.19 and Figure 6.20, the scenario is similar, however the presence of the POT changes the behaviour of the ground in what concerns to vertical displacements. The soil movement between both tunnels propagates to the crown of the POT, which moves inwards the excavation, this is due to the lining stiffness that affects the global displacements.

It is clear that the presence of the POT influences the dissipation of the vertical displacements around the SCL tunnel, as it can be identified in Figure 6.19. The maximum vertical displacement reached in the undercrossing section was approximately 7 mm whereas the heave reached 12 mm. The small difference in the displacements between the two sections analysed, (4.2m away of the POT centre-line - Figure 6.17 - and the undercrossing section - Figure 6.19) is explained by the difference in the tunnel cross-section.

The tunnel cross-section right underneath the POT is substantially smaller than the cross section of the advance No. 5, which allows less soil to be mobilized, hence is expected that this cross section presents a slightly better global behaviour. This may reveal that the diameter of the cross-section has a significant influence in the convergence of the tunnel. This effect would be more intense if the POT was not present since more soil would be available and a full arching effect could be reached.

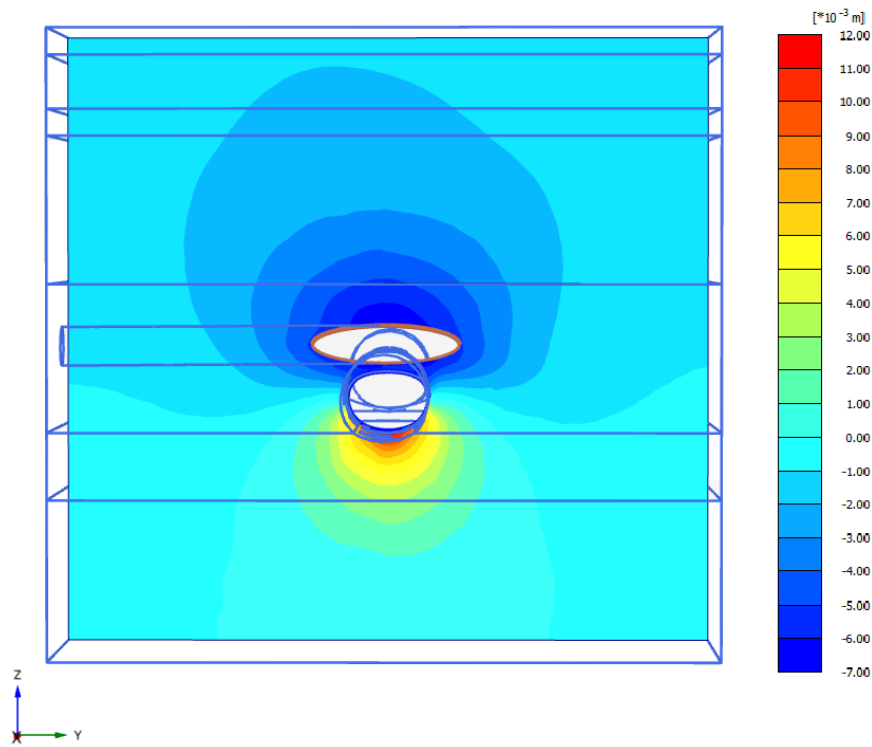


Figure 6.19 - Vertical displacements in a cross section perpendicular to SCL tunnel intersecting the POT centre-line (u_z)

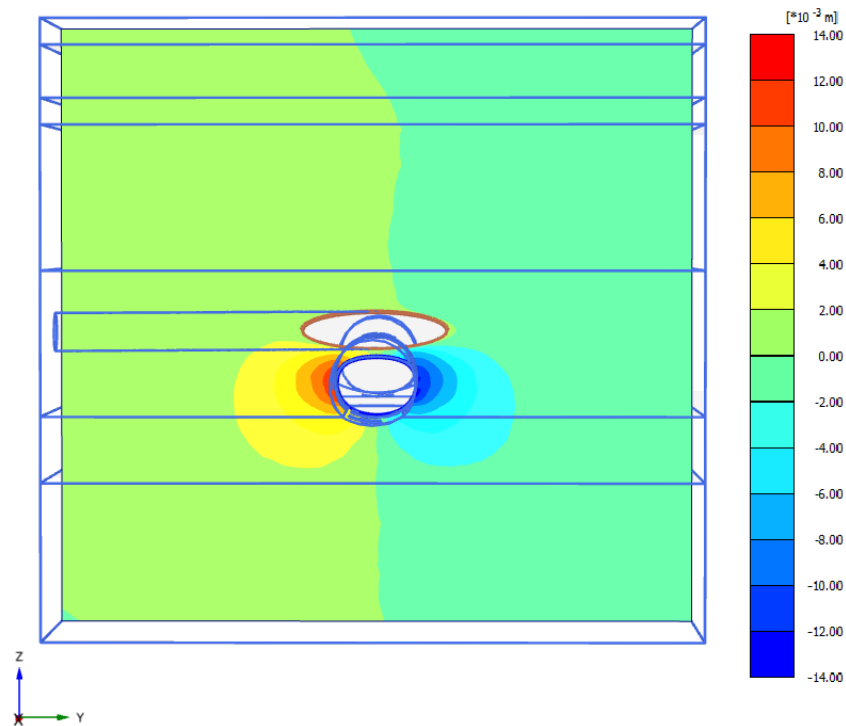


Figure 6.20 - Horizontal displacements in a cross section perpendicular to SCL tunnel intersecting the POT centre-line (u_y)

The horizontal displacements registered within this section (Figure 6.20) follow the tendency above mentioned for the vertical displacements, i.e. it records a small decrease compared with the section away from the POT centre-line. In this section, the horizontal displacements reached a maximum of 14 mm towards the tunnel. It can be identified within this figure that in fact the arching effect is not fully mobilized due to the POT presence, since the horizontal movement propagates less in this section compared with the previously analysed.

Figure 6.21 and Figure 6.22 represent a cross section along the SCL tunnel in which the previously explained behaviour of the ground is summarized (concerning vertical and horizontal displacements, respectively). Regarding the vertical displacements, it was identified that, along the SCL tunnel, the maximum heave was 13.7 mm and the maximum settlement was 20.3 mm, located in the crown of the ascending part of the SCL tunnel. The non-expected location of the maximum displacements may be explained by the vertex located in the crown of the tunnel in that section. Once again, the area around advance No. 1 should not be taken in consideration since the adopted strategy did not allow the tunnel to move accurately.

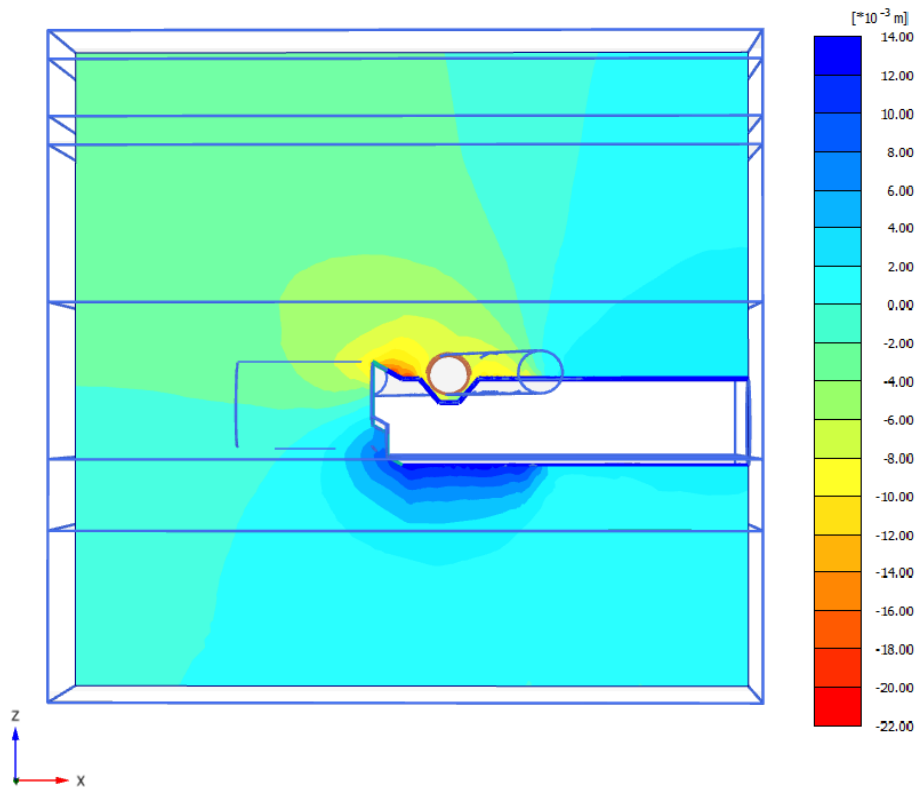


Figure 6.21 - Vertical displacements in a cross section along SCL tunnel (u_z)

Regarding the horizontal displacements parallel to the SCL tunnel axis, as it is possible to identify in Figure 6.22, the major movements are located in the face of the excavation, reaching 31.8 mm towards the excavation. The different colours in the figure describe the influence of the excavation face in the soil ahead which moves towards the face. The effect is spread almost to 3 diameters ahead the excavation face.

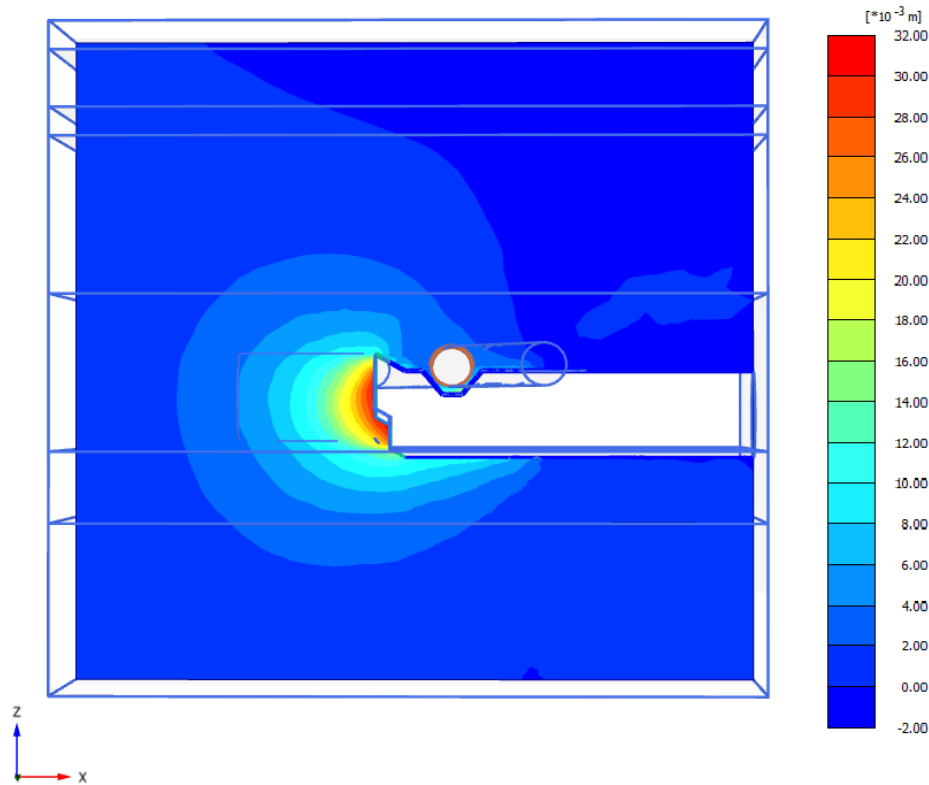


Figure 6.22 - Horizontal displacements in a cross section along SCL tunnel (u_x)

The following sequence of figures (Figure 6.23 and Figure 6.24) aim to identify the importance of the excavation of the top heading and the invert in distinct moments. For this purpose the author represented two of the most critical advances, No. 14 and No. 15, (Figure 6.23 and Figure 6.24, respectively). In advance No. 14 the maximum horizontal displacements inwards the excavation face is 25.6 mm whereas in advance No. 15 increases to 26.2 mm. Advance No. 15 corresponds to an invert excavation. Although the face of the advance No. 15 is 1 m behind the face of the top heading excavation, the increase in horizontal displacement is significant. It might be obvious that if the top heading and invert face excavation were aligned, the effect would be even more intense, hence this is a good practice and standard methodology adopted in SCL tunnels to minimize the movement in the excavation face and improve global stability, leading ultimately to less movements at the surface as explained in Chapter 2.

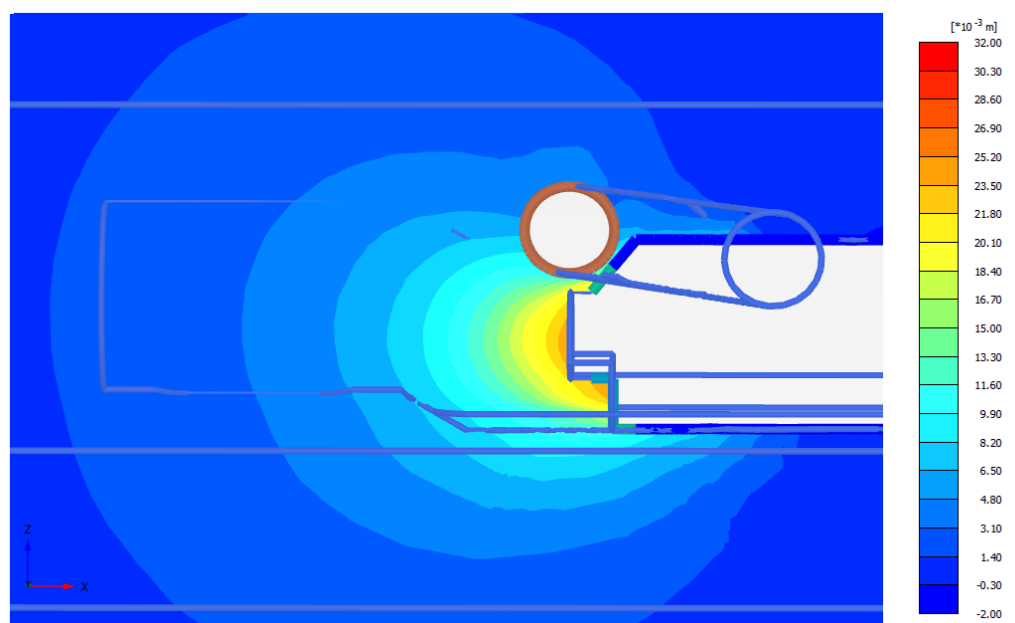


Figure 6.23 - Horizontal displacements in a cross section along SCL tunnel representing advance No. 14 (u_x)

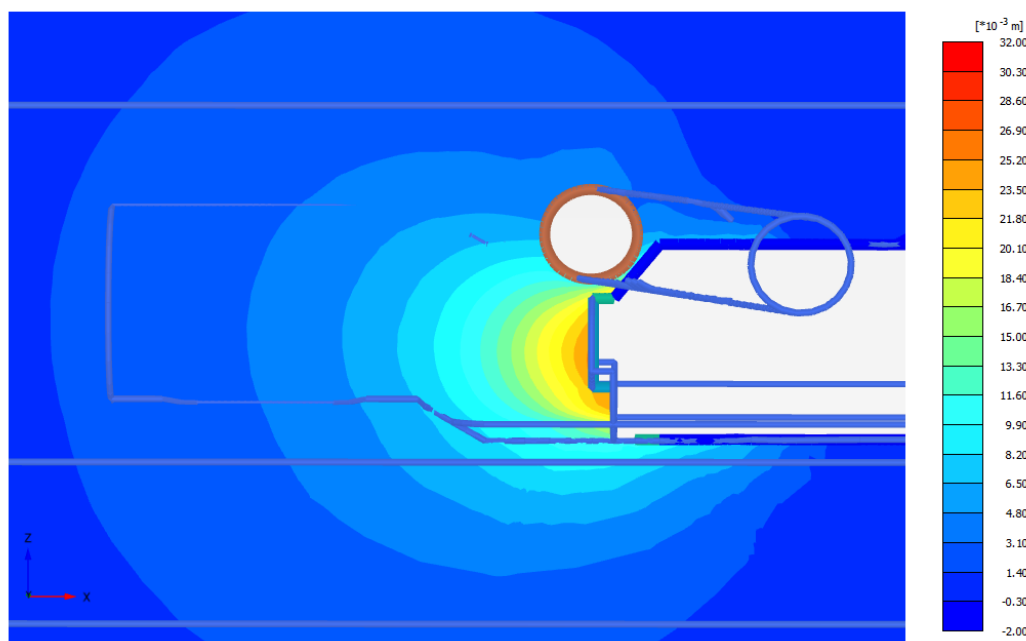


Figure 6.24 - Horizontal displacements in a cross section along SCL tunnel representing advance No. 15 (u_x)

6.3.1.4. Stress state

The initial stress state generated by means of the K_0 procedure provided by PLAXIS, is represented in the following figures. The introduction of the stress state was divided in total stresses and effective stresses.

The vertical total stresses, represented as σ_{zz} , present a maximum value of 939.0 kN/m^2 , whereas the horizontal total stresses, which are the same in both directions, $\sigma_{xx} = \sigma_{yy}$, a maximum of 991.9 kN/m^2 (Figure 6.25). As already expected, the horizontal stresses are greater than the vertical ones, once the in-situ earth pressure coefficient is greater than one through the London Clay and the Mottled Formations.

Regarding the effective stresses, Figure 6.26, since the groundwater head remains 6 m below the surface level, the maximum value for the vertical effective stresses, represented as σ'_{zz} , is 529.0 kN/m^2 and the horizontal effective stresses, σ'_{xx} and σ'_{yy} , 581.9 kN/m^2 .

Note should be taken that the maximum stresses values are lower than those presented in the previous chapter because in the 3D analysis the deepest layer was modelled with a thickness of 12 m instead of 22 m as modelled in PLAXIS 2D. This difference as already explained, has no influence on the results expected for the SCL or Post Office Tunnels.

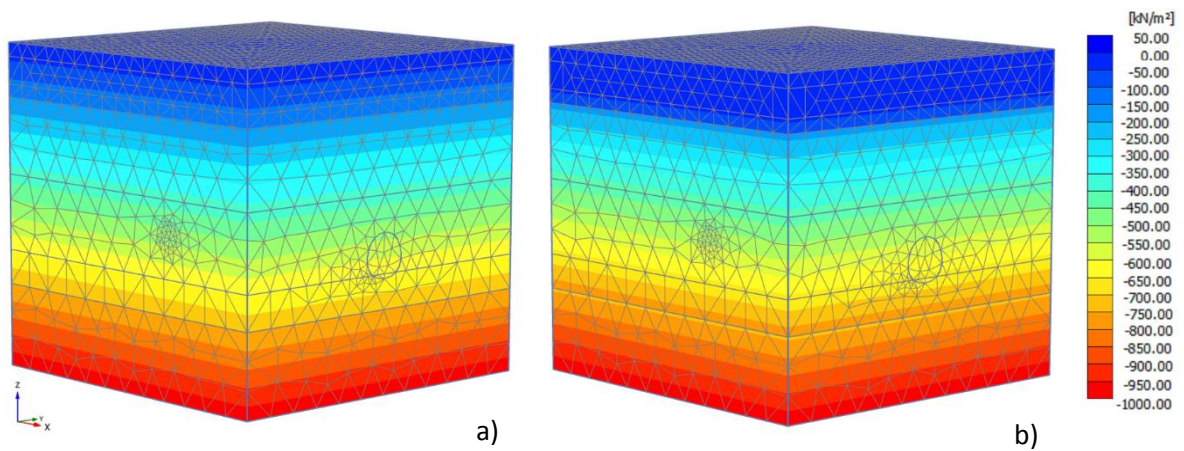


Figure 6.25 - Initial total stress state; a) Vertical stresses, σ_{zz} ; b) Horizontal stresses, σ_{xx} , σ_{yy}

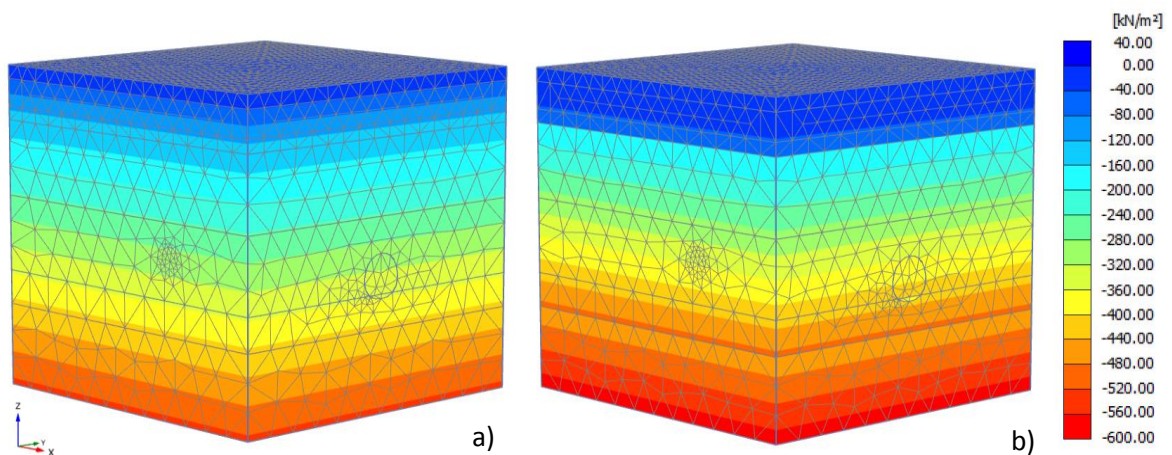


Figure 6.26 - Initial effective stress state; a) Vertical stresses, σ'_{zz} ; b) horizontal stresses, σ'_{xx} , σ'_{yy}

Regarding the vertical stress state developed due to the excavation sequences it is possible to identify in advance No. 27 that the total stresses in the crown of the SCL tunnel decrease as well as the stresses underneath the tunnel (see Figure 6.27). The scenario in the vicinity of advance No. 1 is slightly different however this difference is once again explained by the strategy used to excavate that volume of the soil

(excavation and lining application simultaneously). Near the tunnel's side walls the vertical stresses increased.

These alterations in the stress state were induced by the arching effect, both longitudinal and transversal. As a result, the vertical stresses above and underneath the SCL tunnel decrease as the vertical stresses in the between turn to zero due to the removal of the soil. Consequently, the vertical stresses near the side walls increase to equalize the stresses.

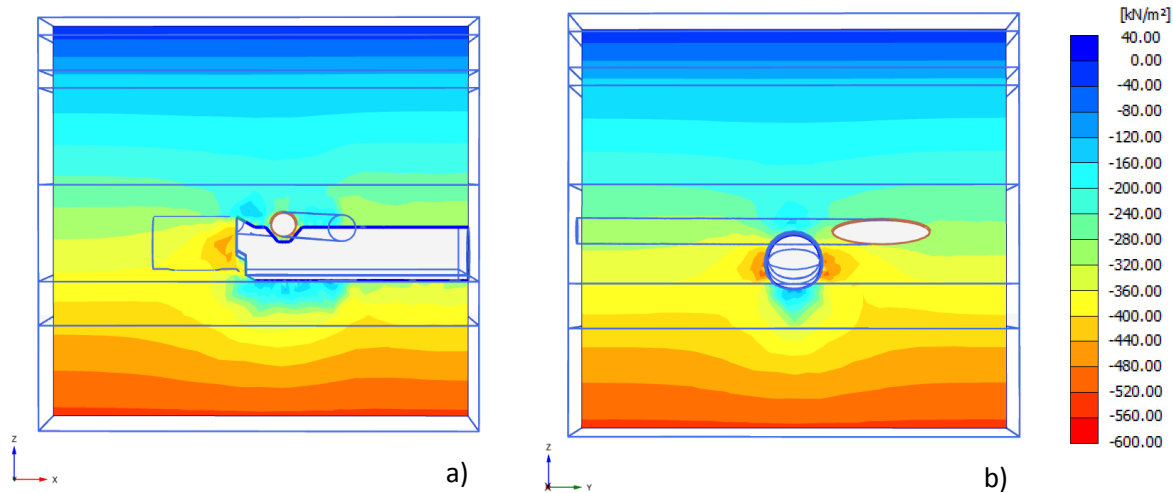


Figure 6.27 - Vertical stress state after advance No. 27 was performed (σ'_{zz}); a) Cross section along the SCL tunnel; b) Cross section perpendicular to the SCL tunnel taken 1 m behind the excavation face

It is possible to identify that the effective vertical stresses ahead of the excavation face also increased substantially, (Figure 6.27 a)). This fact occurs because the longitudinal arching effect contributes to increasing the vertical stresses ahead of the excavation face. As the vertical stresses above the tunnel decrease, the soil ahead is mobilized as shown in Chapter 2.

The plot represented in Figure 6.29 represents the effective vertical stresses in four notable points throughout the excavation process: Point A is located between the SCL tunnel and the POT; Point B is located in a horizontal plan containing the tunnel axis; Point C is located beneath the SCL tunnel invert; and Point D in the SCL tunnel axis. All points are located in the same vertical section, which is the critical section beneath the POT. Points A, B and C are located 30 cm away of the SCL lining (see Figure 6.28).

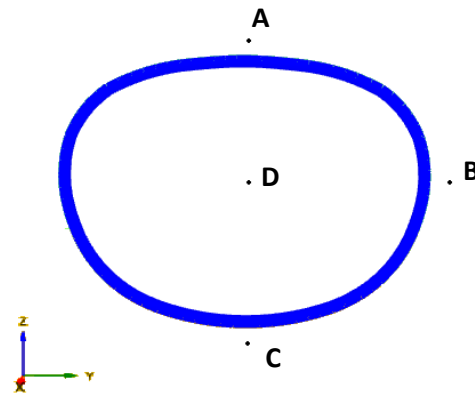


Figure 6.28 - Representation of the notable point's location

The curves represented in Figure 6.29 allow to identify the amplitude of the modifications on the stress state above mentioned for the areas in the vicinity of the points. Note that the abscissa of the plot refers to the distance from the excavation face to the POT centre-line (see Table 6.3). The first point in each curve represent the initial stress state. The remaining points in the curves represent each excavation advance.

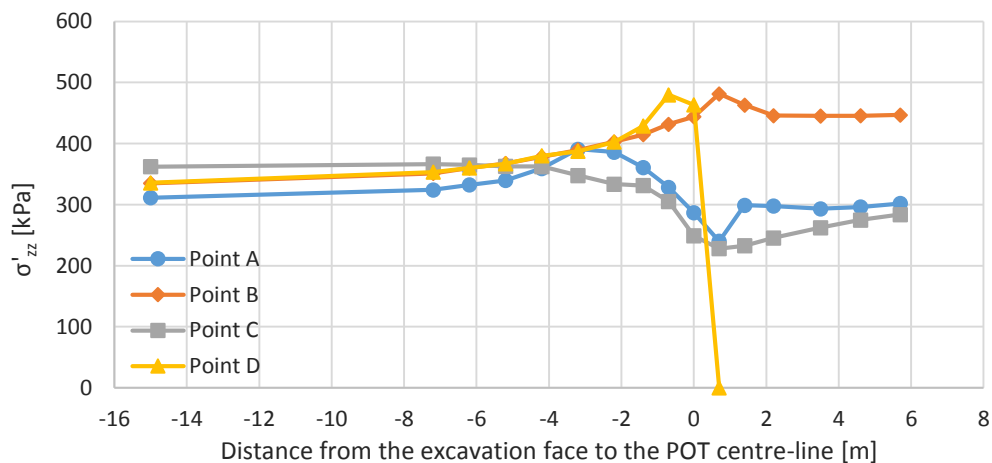


Figure 6.29 - Evolution of the effective vertical stresses throughout all excavation process

Figure 6.29 allows to observe that, as previously explained, the vertical stresses near the tunnels walls (Point B) increase as the excavation proceeds and the greater increase in the vertical stresses occur exactly when the excavation face reaches the reference section. After the excavation face crosses the reference section, the vertical stresses slightly decrease due to the application of the lining (applied 1 m behind the excavation face), which allows a new stress state to be reached as the lining strength is mobilized.

Near the crown of the SCL tunnel (Point A) the vertical stresses slightly increase when the first advances are performed due to the longitudinal arching effect mobilized, which causes a stress increase in the points ahead. However, when the excavation face gets near the crossing section (approximately -2 m from the reference section) the vertical stress in the crown of the tunnel start to decrease reaching the

minimum when the excavation passes the crossing section. Soon after, a slightly increase is registered which is also due to the application of the lining.

Regarding Point C, which is located near the invert of the tunnel, the vertical stresses record a decrease until the excavation face reaches the reference section, as previously seen. After the excavation crosses the location of the point the stresses slightly increase due to the new stress state generated by the strengthening of the lining.

For Point D, the scenario is different once this point is intersected by the excavation, thus when the excavation face reaches its location the stresses decrease to zero since the volume of the soil containing that point is removed. When the excavation was getting closer to that point, the vertical stresses increased due to the longitudinal arching effect ahead of the excavation face (see Figure 6.27 a)).

Also due to the arching effect, the horizontal stress state suffers alterations too, both longitudinally and perpendicularly to the SCL tunnel. Regarding the stresses perpendicularly to the tunnel axis (Figure 6.30) there is a slightly increase on the stresses in the crown of the tunnel and a larger increase underneath the tunnel. The modification in the stress state underneath the tunnel is more significant once the stresses are transferred mostly to this area due to the presence of the POT above. As a result of the unloading, the confinement stresses decrease in the tunnel walls and therefore the horizontal stresses decrease in the same proportion.

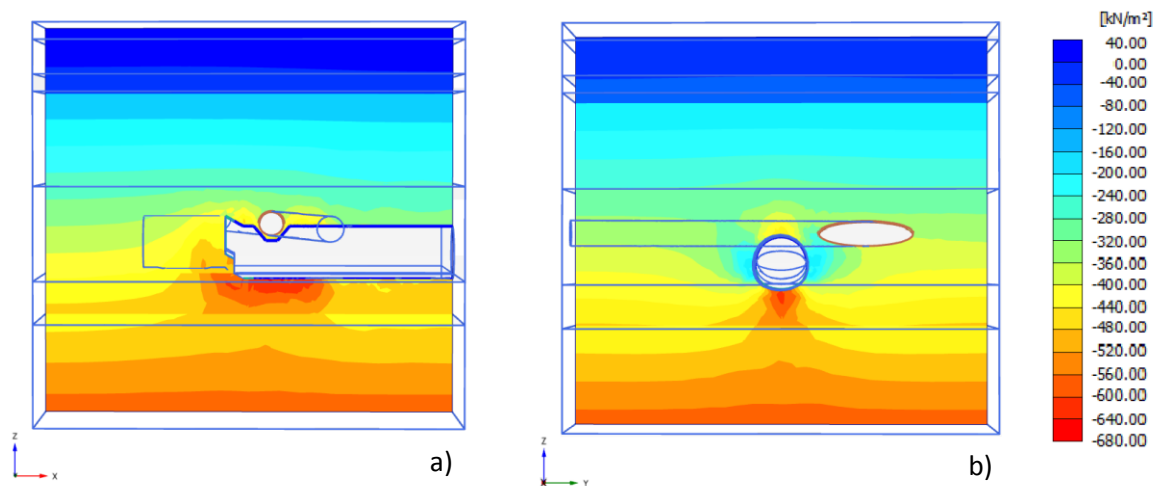


Figure 6.30 - Horizontal stress state perpendicular to the SCL tunnel longitudinal axis after advance No. 27 was performed (σ'_{yy}); a) Cross section along the SCL tunnel; b) Cross section perpendicular to the SCL tunnel taken 1 m behind the excavation face

The plot represented in Figure 6.31, reproduces the evolution of the horizontal stresses throughout all excavation process (for the notable points detailed in Figure 6.28) perpendicularly to the SCL tunnel longitudinal axis.

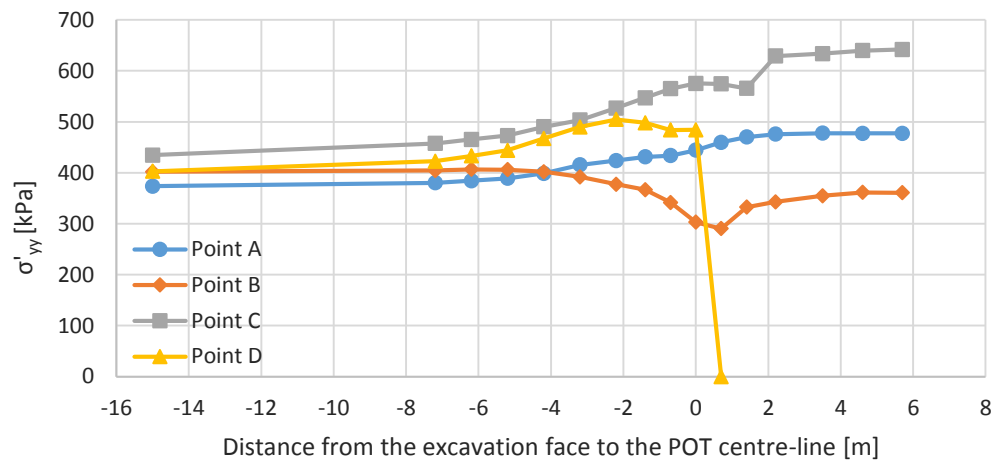


Figure 6.31 - Evolution of the effective horizontal stresses perpendicular to the SCL tunnel longitudinal axis throughout all excavation process (σ'_{yy})

The stress behaviour described in Figure 6.30 can be clearly identifiable in Figure 6.31. Points A and C recorded a horizontal stress increase during all excavation process, although more significant in Point C, which corresponds to the tunnel invert, as previously detailed. Regarding Point B, as it was previously mentioned, the soil near the tunnel walls registered a decrease in the horizontal stresses perpendicular to the SCL tunnel cross section, which is related with the removal of the soil. Point D recorded an increase in σ'_{yy} when the excavation face was getting closer, and turned zero with the passage of the excavation face.

Longitudinally, the scenario of the horizontal stresses is slightly different (see Figure 6.32). Although it is similar to the perpendicular stresses in the tunnel invert, which suffer an increase, the major alteration relies on the stresses ahead of the excavation face, where the horizontal stresses decrease significantly due to the unloading. The region near to the tunnel walls register a slightly increase.

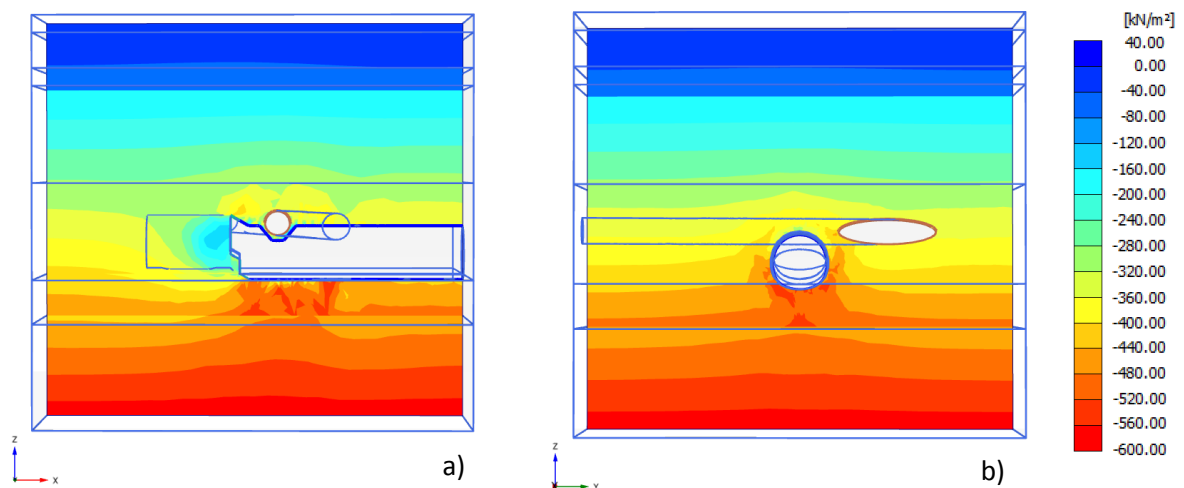


Figure 6.32 - Horizontal stress state parallel to the SCL tunnel longitudinal axis after advance No. 27 was performed (σ'_{xx}); a) Cross section along the SCL tunnel; b) Cross section perpendicular to the SCL tunnel taken 1 m behind the excavation face

It can be seen in the plot presented in Figure 6.33 that, until the excavation face is approximately -1 m from the reference section, all points suffer a decrease in the stresses longitudinally to the SCL tunnel. The passage of the excavation face by the reference section then nullifies the horizontal stresses in Point D, and causes an increase in the stresses in all other points, which can be correlated with the effect identified in Figure 6.32 b). However contrary to what would be expected, after the identified increase, the horizontal stresses longitudinally to the SCL tunnel axis decrease again, which may be related with the application of the lining since it induces a new stress state. The proximity of the reference points to the SCL tunnel lining (30 cm) may also slightly affect the results. Note that Point A registered the greater decrease. This point is located in an underprivileged area since the arching effect is not fully mobilized due to the confinement of this area between both tunnels.

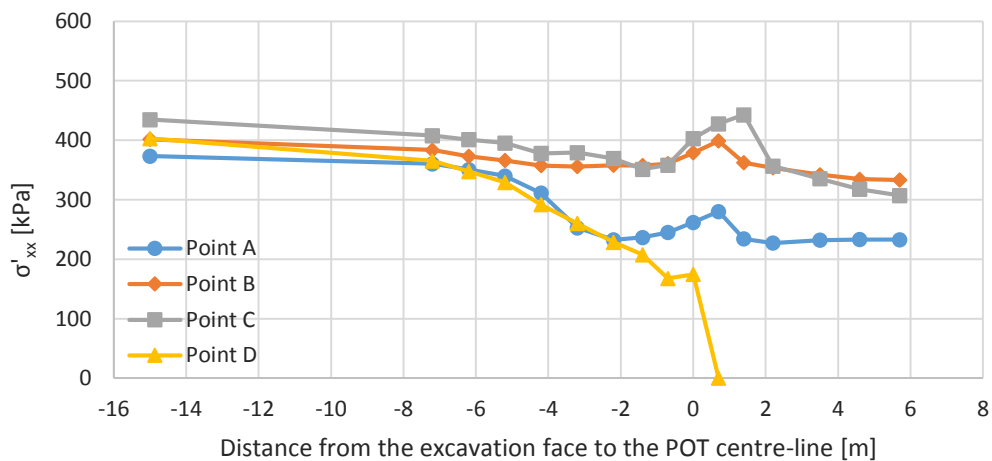


Figure 6.33 - Evolution of the effective horizontal stresses parallel to the SCL tunnel longitudinal axis throughout all excavation process (σ'_{xx})

The following figures represents the direction of the principal stresses in a cross section along the tunnel, which allows one to identify the longitudinal arching effect, and, in a cross section perpendicular to the tunnel, which in turn represents the transversal arching effect.

Even though Figure 6.34 b) shows the interference of the POT, it is still possible to identify the mentioned effect around the SCL tunnel.

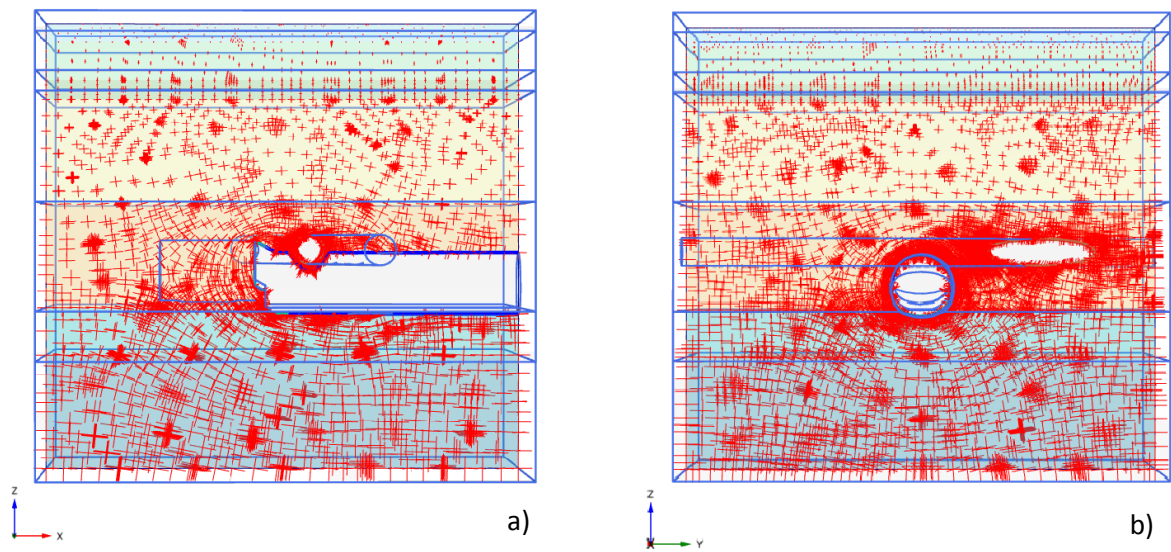


Figure 6.34 - Effective principal stresses direction; a) Cross section along the SCL tunnel; b) Cross section perpendicular to the SCL tunnel taken 1 m behind the excavation face

6.3.1.5. Stress paths

A stress analysis through σ'_{zz} , σ'_{xx} , σ'_{yy} , as it was presented in the previous subsection, is very useful and intuitive since the stresses magnitude is associated with a specific direction which allows to an immediate comprehension of the stresses alterations. However, the soil behaviour, either in resistant or deformational terms is strongly dependent on the stress path experienced, hence throughout this section it will be presented the stress paths for notable points in the vicinity of the SCL tunnel.

Figure 6.35 represents the relative location of the reference points. As it can be seen, two different sections were chosen for this analysis, one in the centre of the Underpass and another 4.2 m away from the first section, hereafter referred as Ovaloid section. Considering that the stress paths of the points located at the POTU section is strongly affected by the near presence of the POT, the Ovaloid section was chosen in order to reproduce a typical stress behaviour of points in the vicinity of the SCL tunnel in which the POT presence is less significant. It should be mentioned that a section at a greater distance from the Underpass would not reproduce the typical stress behaviour during the SCL tunnel excavation, since it would be affected by the strategy used to excavate the first advance, in which the excavation is performed simultaneously with the application of the lining.

For the POTU section two points were referred: one 30 cm above the SCL tunnel crown (which is located exactly between the SCL and the Post Office tunnels) – Point A, and the other located on the plane containing the longitudinal axis of the SCL tunnel, also 30 cm away of the SCL tunnel walls – Point B. For the Ovaloid section two points were referred also 30 cm away of the SCL tunnel lining, Point A' located near the crown and Point B' located near the tunnel walls. Two other points were added in this section which were aligned with the previous but 1 m away instead - Point A'' and Point B''.

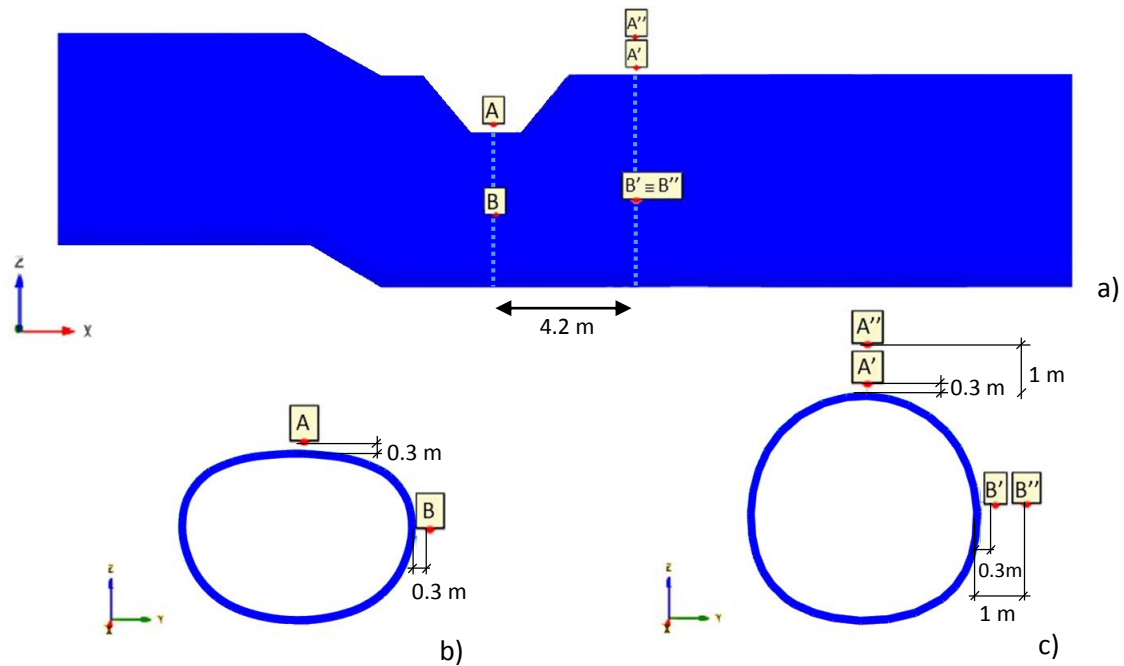


Figure 6.35 - Relative position of the reference points; a) Longitudinal location; b) POTU section ; c) Ovaloid section

Figure 6.36, Figure 6.37 and Figure 6.38 represent the stress path for the six points represented above, as referenced. Since the analysis undertaken consisted on an undrained analysis, the yield surface is represented within each plot as the undrained shear strength of the point in assessment, c_u , which depends on its depth (see Table 4.4). Each point on the stress path represents the stress state at a different advance.

It should be mentioned that since the excavation is being performed within soil layers in which $K_0 > 1$, the principal stresses may suffer a rotation during the excavation, affecting in turn the stress path. However an analysis on the stress path regarding yielding can still be accurately done.

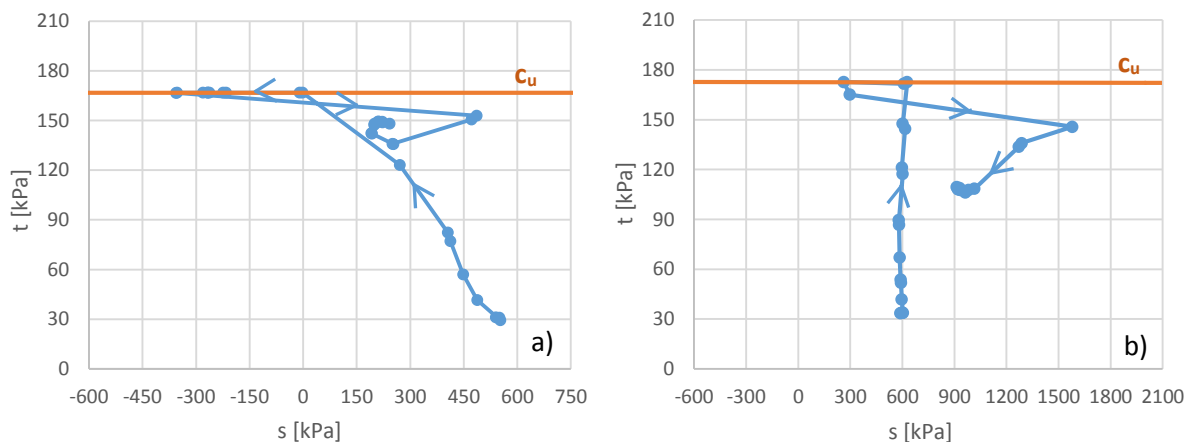


Figure 6.36 - Stress path in (s,t) space for the POTU section; a) Point A; b) Point B

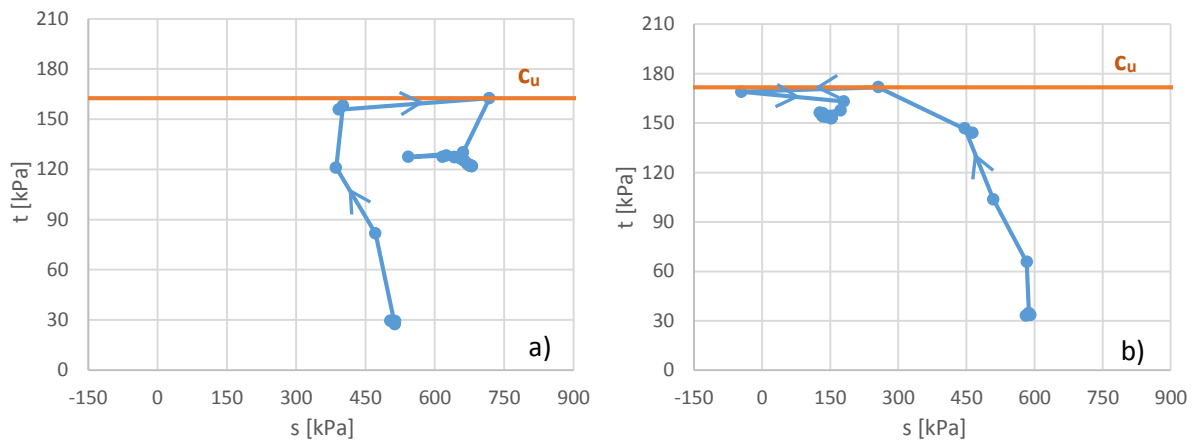


Figure 6.37 - Stress path in (s,t) space for the Ovaloid section; a) Point A'; b) Point B'

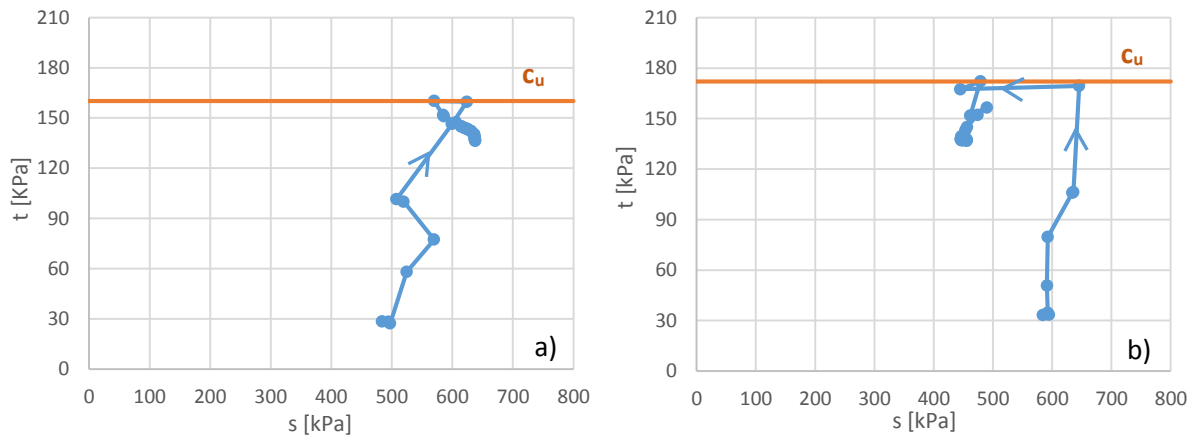


Figure 6.38 - Stress path in (s,t) space for the Ovaloid section; a) Point A''; b) Point B''

It can be identified that all reference points, although in different levels, suffered yielding as stress path reaches the represented yield surface. For some points, as for example Point A from Figure 6.36, the stress path coincides with the yield surface in a considerable extension, which reveals that for this points there is a major need for stress redistribution while the excavation is being performed in the vicinity. Only when the lining is applied in the area around the points (which corresponds to the application of the lining in advance No. 16 for reference points in the POTU section, and in advance No. 5 for the points in the Ovaloid section), the stress state returns to a position within the yield surface, since the strength of the lining contributes for the soil resistance.

Whilst Point A and Point B, Figure 6.36, suffer considerable alterations on the stress path, the remaining points present a more stable stress path, which is expected since the critical section is the one containing the first two points. Besides it can be seen that the points represented in Figure 6.37 and Figure 6.38 first reached the equilibrium than Point A and B, which is indicated by the cluster of points in the end of their stress paths.

6.3.1.6. Yielding

Unlike the model developed in PLAXIS 2D, the 3D model takes into account the influence of the POT, which means that the results from the last will always be more critical since there is less soil available and a stiff structure nearby. Thus it is expected the yielding area developed through the different stages to be larger, besides this will reflect more accurately the arching effect developed.

Figure 6.39 corresponds to advance No. 5, which represents the excavation face at approximately 4 m behind the crossing point. This representation aims to interpret the developing of plastic points without the interference of the POT. It is possible to identify that yielding occurs mainly in points ahead the excavation face. The yielding of this points is related with the increase of vertical stresses due to longitudinal arching effect induced by the excavation and with the decrease of horizontal stresses in the tunnel axis direction, due to the removal of the soil behind. Furthermore they follow the path or the mobilized soil since the load is being transferred from the top of the excavation to the bottom, corresponding to the mobilization of the vertical arching effect.

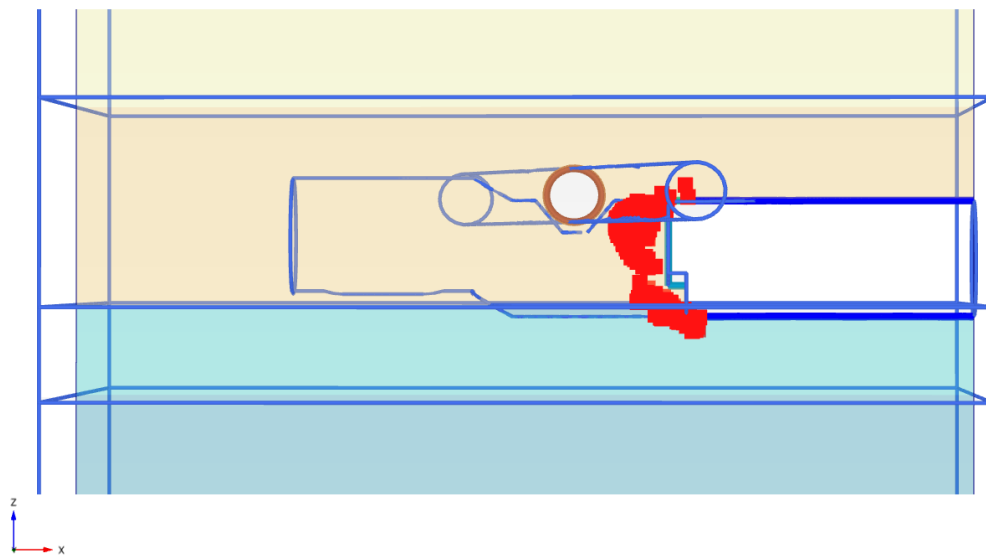


Figure 6.39 - Yielding area corresponding to the performance of advance No. 5

The following figure, Figure 6.40, represents a cross section approximately 1 m ahead the excavation face. When the excavation of the advance No. 3 is performed, which corresponds to the excavation of the top heading, a large area of yielding is identified, Figure 6.40 a), whereas the yielding area corresponding to advance No. 4, Figure 6.40 b), which is an invert excavation, is less extensive. This comparison gives strength to the previously explained that the division of the excavation in top heading/invert is really an important measure to control yielding. Before the excavation of the advance No. 4 is performed, shotcrete is applied to the excavation face of the top heading which induces a new stress state preventing the creation of a failure mechanism in the excavation face and reducing ground movements. The closing of the excavation face is furthermore a good practice adopted in SCL tunnels in soft soils (Thomas, 2009).

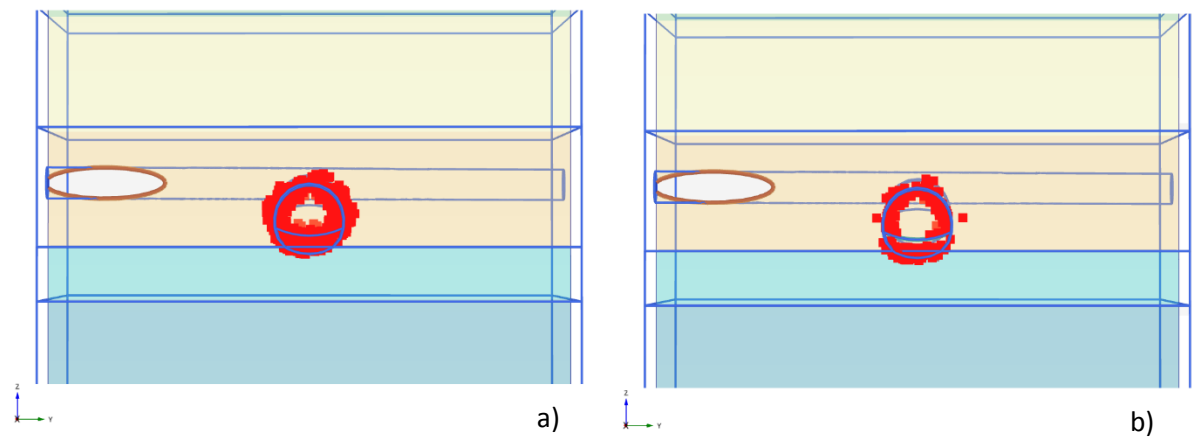


Figure 6.40 - Yielding area; a) Top heading excavation – advance No. 3; b) Invert excavation – advance No. 4

When the excavation face gets closer to the POT position the yielding area extends towards the POT lining, as it can be seen in Figure 6.41 which corresponds to advance No. 8. This outcome is again explained by the longitudinal arching effect, which can be seen through the direction of the principal stresses for this advance (Figure 6.42). In comparison with the arching effect generated in advance No. 5, in this stage once the POT lining is near the excavation face, the strength of the soil is mobilized more ahead because the presence of a stiff structure restricts the volume of the soil to be mobilized.

In Figure 6.41, the points represented in white identify tension cut-off points which correspond to points which failure was due to tension instead of shear, as it was explained in the previous chapter.

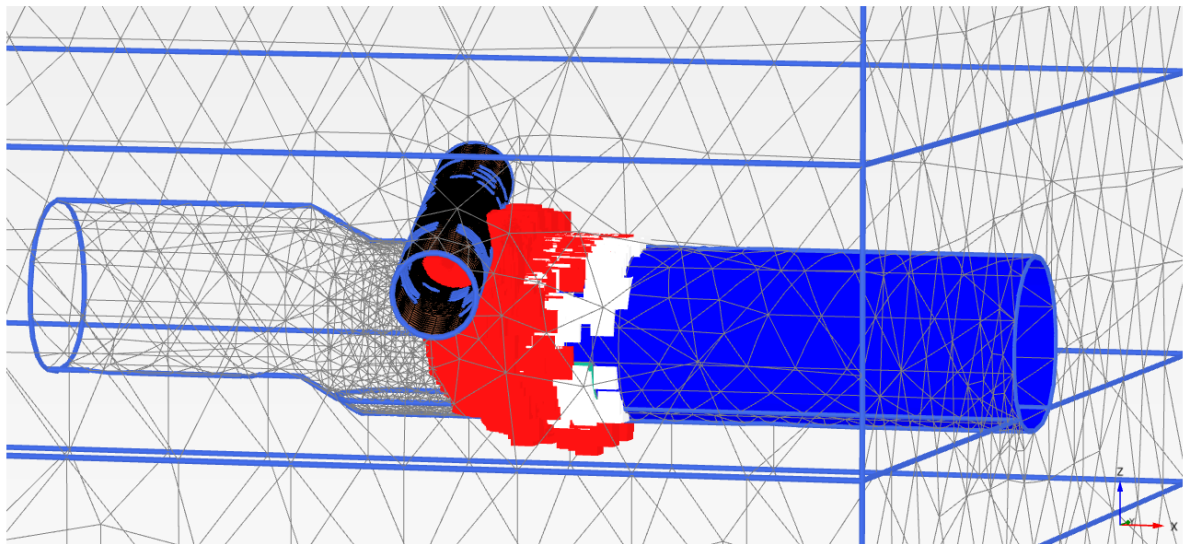


Figure 6.41 - Yielding area developed in advance No. 8

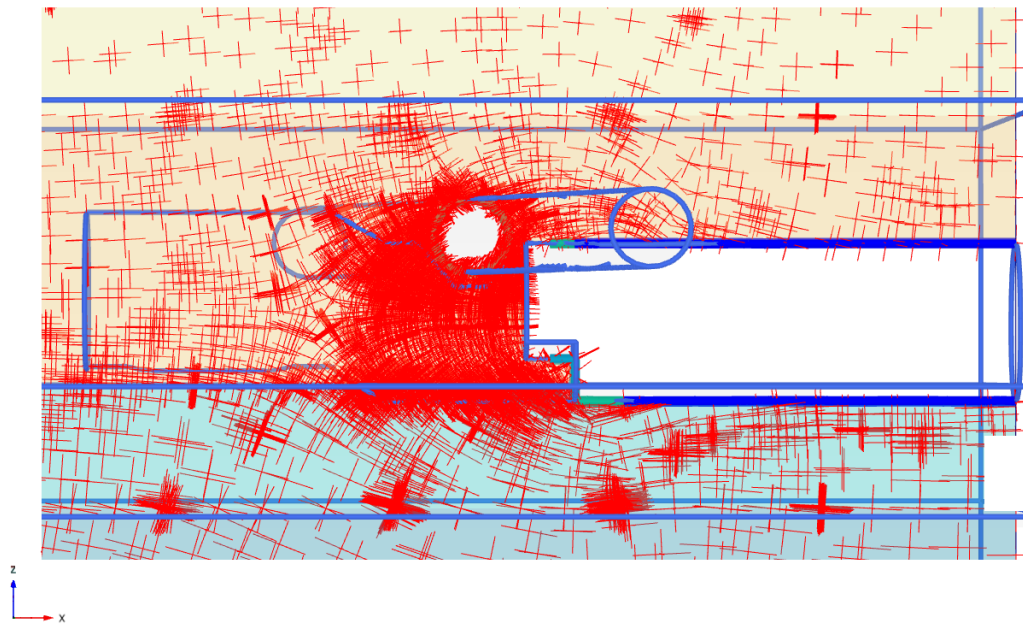


Figure 6.42 - Arching effect corresponding to advance No. 8

6.3.2. SCL TUNNEL' LINING FORCES

The forces developed in the lining of the new excavated tunnel will be presented in this subsection. For this analysis, the author will focus the attention on the forces developed in the end of the excavation in order to understand the critical sections in the SCL tunnel regarding the lining forces. The lining forces will also be presented in the crossing section, which corresponds to a SCL tunnel section beneath the POT. This allows to a quick comparison between the forces developed in the 2D model for the same section. Note that in the following figures positive values correspond to tensile stresses, whilst negative values represent compressive stresses.

Regarding the axial forces longitudinally to the tunnel (parallel to the SCL tunnel longitudinal axis) it can be seen in Figure 6.43 that, in the end of the excavation, i.e., when all the lining in the model is with its full strength, the axial forces are concentrated in the POTU section, mainly in the side walls of the tunnel, which in turn correspond to tensile stresses. The maximum compressive stresses occur in the upper vertex of the transition between the Ovaloid section and the POTU section.

The axial forces in the radial direction, as it can be seen in Figure 6.44, mainly registers compression in all tunnel lining, however it can be identified that in the upper part of the transition between the Ovaloid section and the POTU section some tensile stresses developed which are represented with the darker blue of the represented scale.

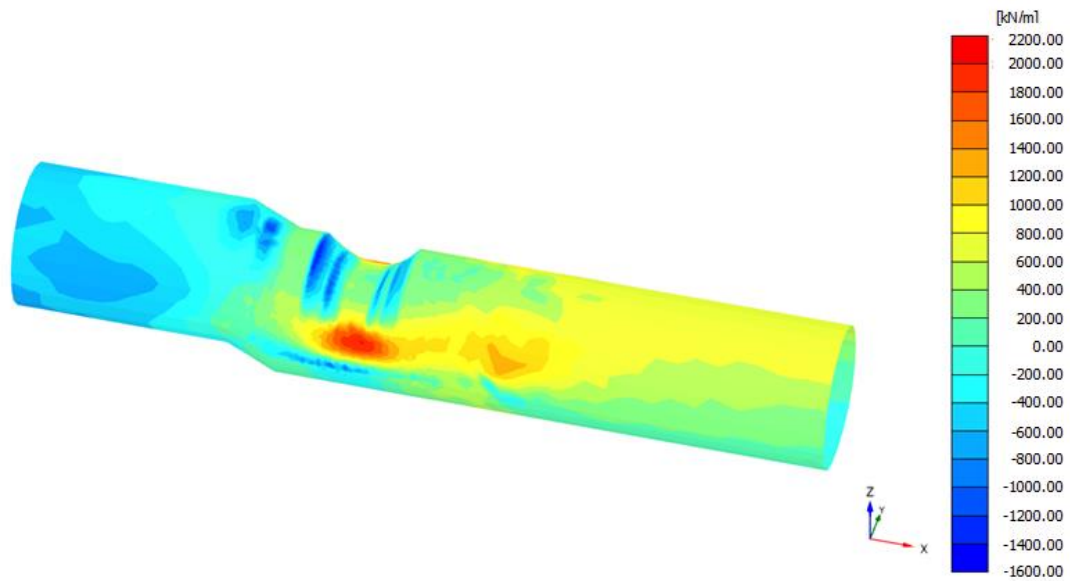


Figure 6.43 - Axial forces in the longitudinal direction in the end of the excavation

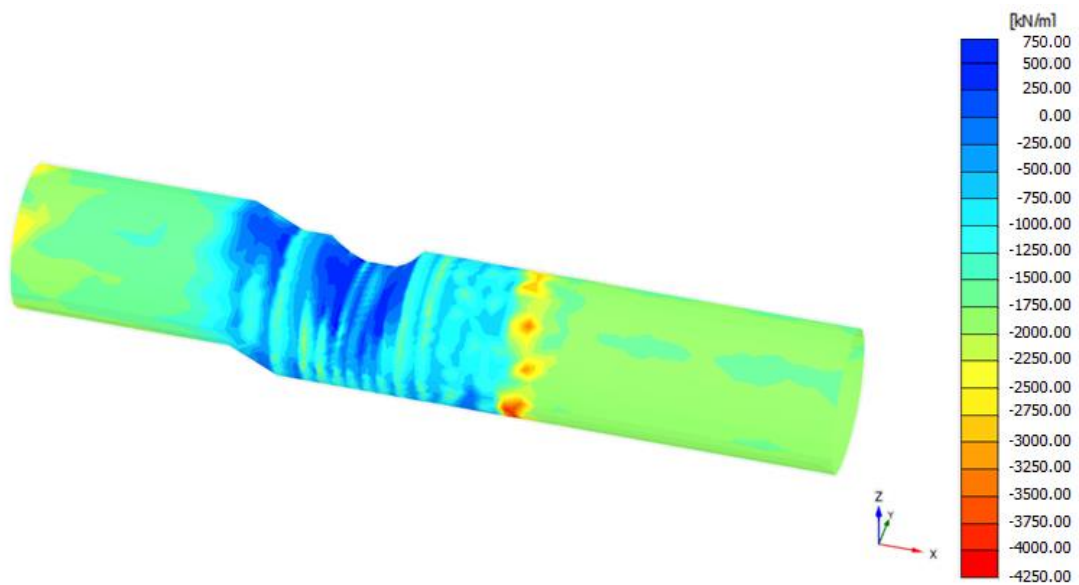


Figure 6.44 - Axial forces in the radial direction in the end of the excavation

Figure 6.45 and Figure 6.46 represent respectively, the bending moments in the longitudinal and in the radial direction. It can be seen that the bending moments in the longitudinal direction concentrate in the vertexes of the transition sections, with the positive bending moments in the upper vertexes and the negative bending moments in the lower vertexes, inducing the convergence of the tunnel lining in the crossing section. In what concerns to the bending moments in the radial direction it can be seen that the critical section is also near the lower vertexes where the negative moments assume a certain relevance.

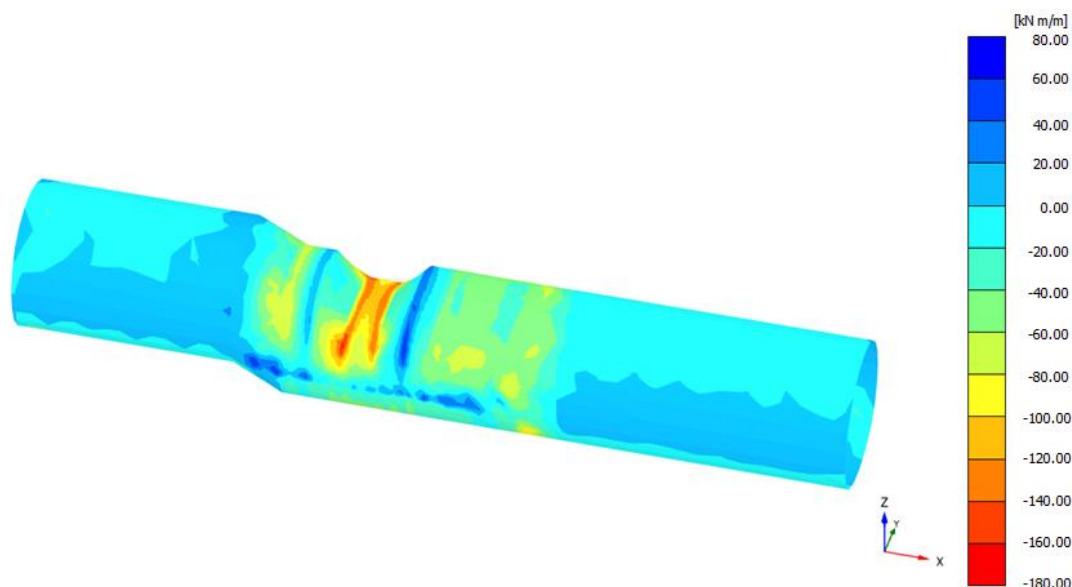


Figure 6.45 - Bending moments in the longitudinal direction in the end of the excavation

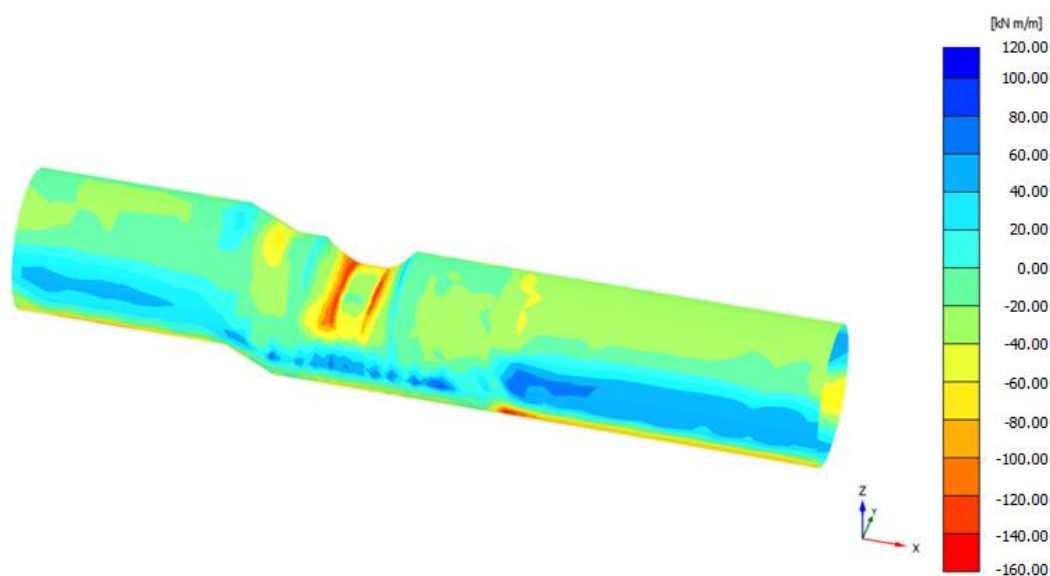


Figure 6.46 - Bending moments in the radial direction in the end of the excavation

Although the diagrams of forces are expected to be considerable uniform along a tunnel cross-section (radially), as it can be seen in the following figures, the PLAXIS 3D diagrams for the represented cross section, present some peaks values which may be related with the fact that three-dimensional elements are being intersected in order to reproduce a two-dimensional behaviour. Thus, only the average behaviour must be interpreted.

Figure 6.47 and Figure 6.48 represent respectively, the axial forces and the bending moments for a section beneath the POT. Each figure reproduce the diagram of forces in two different stages, in a temporary stage (a) and when the lining is completed (b). Note that the temporary stage corresponds to

a situation where the lining is also already complete, however only the part applied in the top heading is with its full strength. A situation where the lining in the invert was not yet applied was not possible, since the lining in the top heading would only be with 1/3 of its total strength. However, once the lining in the invert is only with 1/3 of its total strength, does not result in significant forces.

The maximum axial force registered in this cross section for the temporary stage was a compression of 1230 kN/m , whilst when the lining is complete, the axial forces slightly decrease to 1092 kN/m . This slight reduction is due to the completion of the lining which allows for a better stress redistribution along the cross-section. Note that the axial forces obtained in PLAXIS 2D for the same section were lower than those registered in the PLAXIS 3D model, which is mainly due to the presence of the POT in the latter. As it was already possible to verify, the presence of the POT alters the stress redistribution in its vicinity, which directly affect the lining forces since less volume of soil is mobilized. The weight of this structure may also affect the results to a certain extent.

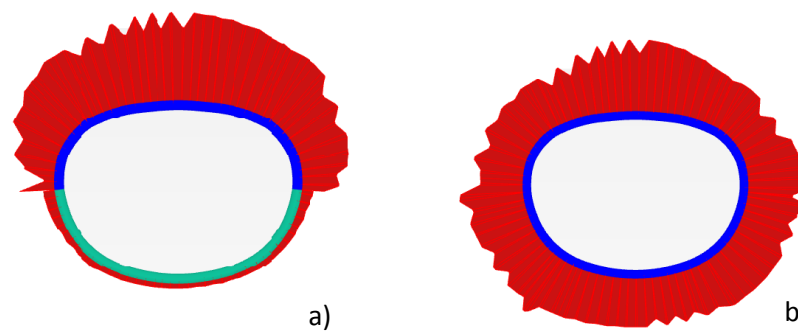


Figure 6.47 - Diagram of the axial forces in the tunnel lining; a) Temporary stage; b) Complete section

The diagram of the bending moments obtained in PLAXIS 3D for the complete section (Figure 6.48 b)) was very similar to the one represented from PLAXIS 2D for the same situation, however the maximum values vary. The maximum bending moment obtained for this section in the present model was 43.22 kN m/m , which is quite lower than registered in the 2D model.

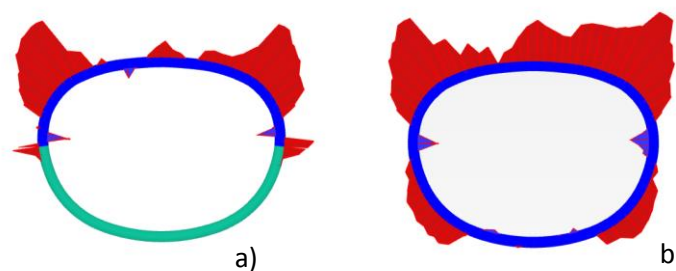


Figure 6.48 - Diagram of the bending moments in the tunnel lining; a) Temporary stage; b) Complete section

6.3.3. POT BEHAVIOUR

Within the scope of this thesis, the displacements on the POT recorded in PLAXIS, induced by the excavation of the new tunnel will be clearly identified throughout this subsection. These displacements will be directly compared with the monitoring data, which means that the analysis will focus on the displacements registered along the POT aligned with the position of the monitoring prisms.

Throughout all excavation process, the POT suffered a combination of vertical and horizontal displacements that culminated in a final position as represented in Figure 6.49. The mentioned figure represents the total displacements registered by the POT in the last performed excavation advance, i.e., advance No. 27. As it was already explained, the last excavation advance corresponds actually to advance No. 28, however this advance will not be considered in the results since it does not represent accurately the excavation process, which was in fact performed by 1 m advances rather than 10 m straight as modelled. Besides, from the monitoring data the author only had access to information regarding the advance corresponding to advance No. 27, thus the influence of the excavation after this advance was not considered.

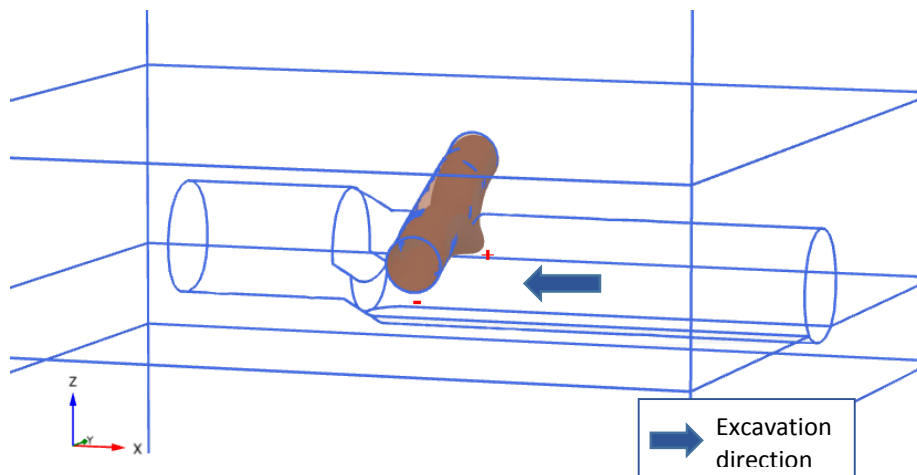


Figure 6.49 - Representation of the POT deformed mesh at advance No. 27

It can be seen in Figure 6.49 that the POT final position, is slightly directed towards the right, which indicates that the major movements registered in the POT were in the first advances of the SCL tunnel excavation. As was already presented, the soil ahead of the excavation face moves inwards the excavation, this tendency reflects on the POT lining when the excavation face approaches it.

The POT moves towards the right once the soil is removed in that area. Although some of the displacements are compensated after the application of the sprayed concrete lining, the POT does not totally return to its initial position. Consequently, even after the excavation face moves to the other side of the POT, the initial displacements will continue to reflect.

One of the major advantages of this three-dimensional analysis is the fact that it allows to track the behaviour of each structure throughout all excavation process. Whilst the 2D model only performed the excavation in one specific section, the 3D analysis allows the effects of excavation advances to be clearly detailed and thereafter to identify areas of potential damages to the structures or critical stages. Although the 3D model presented in this chapter had all excavation advances detailed approximately as performed on-site, this subsection will focus on some specific advances since it would be exhaustive to present the influence of all advances on the POT structure. The selection of those advances over the others will be further explained in the following paragraphs.

In order to identify which advance corresponded to the critical one for each movement, and thereafter to detail the POT movement the most accurately possible, the following plots were developed. The plots represented in Figure 6.50 and Figure 6.51, represent the maximum vertical and horizontal displacements

of the POT, respectively, throughout all excavation process. The points between each advance correspond to the application of the lining further to the excavation of the corresponding advance.

The maximum longitudinal displacement of the POT throughout all excavation process will not be presented since it does not correspond to a critical movement, i.e. the magnitude of the longitudinal displacements is much lower compared with the vertical and horizontal ones. Note that the aim of this plots was to identify which advance was the critical in order to define a reference-advance in the following analysis. Although the longitudinal displacements will also be detailed within this section it is more important to identify which was the critical advance for vertical or horizontal displacements due to its magnitude.

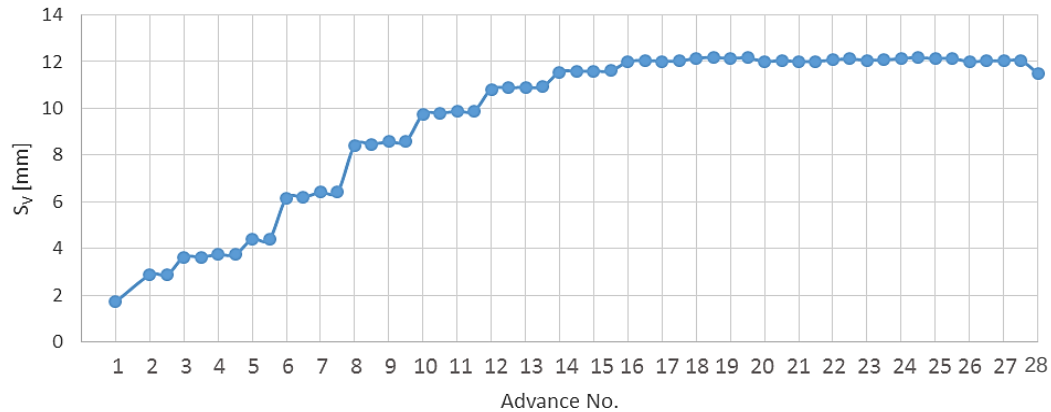


Figure 6.50 - Maximum vertical displacement of the POT throughout the entire excavation process

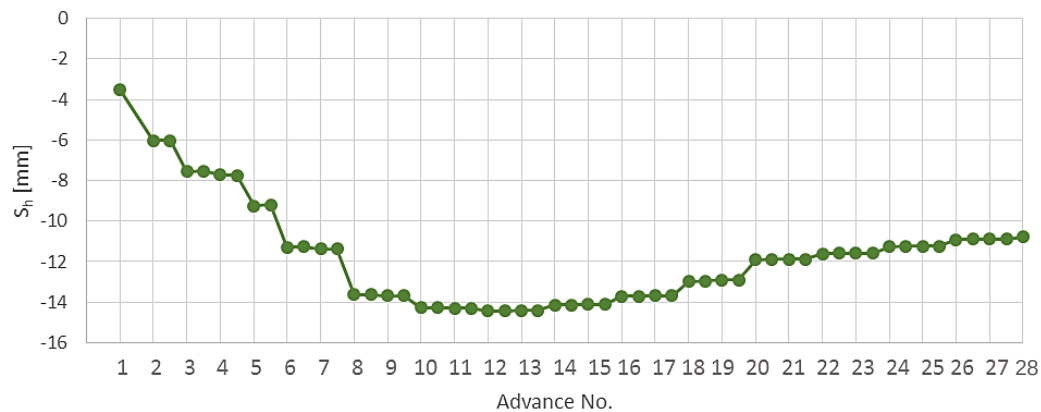


Figure 6.51 - Maximum horizontal displacement of the POT in the transverse direction throughout the entire excavation process

As it can be seen in Figure 6.50 the critical advance for the vertical displacements is advance No. 16, which was expected since it corresponds to the undercrossing advance, thus the advance that exposes much the POT lining. For the horizontal displacements (Figure 6.51), the critical advance is the early No. 12, which from Figure 6.11 can be extrapolated once the following advances are already beneath the POT, thus they almost do not induce horizontal displacements. This can be seen in Figure 6.51 as the maximum horizontal displacement is approximately constant from advance No. 12 until advance No. 16. Although advance No. 12 was identified as the critical for the horizontal displacements, the fact

is that the horizontal displacements remains approximately constant until advance No. 16, thus considering that the last was identified as the critical for the vertical movement, this reveals to be more important within this analysis and will be adopted as a reference.

Note that this plots accentuate the fact that advance No. 28 should not be taken in consideration, as it can be seen, for the vertical displacements there is a decrease in the maximum value for advance No. 28, which may not correspond to the real behaviour, bear in mind that the excavation and the application of the lining were performed simultaneously in advance No. 28.

According to the above described, the author adopted as references the advances No. 2, 16 and 26 performed in PLAXIS, which corresponds to the advances No. 1, 13 and 23 performed on site (Figure 4.8). This advances were adopted as references based on the same principals embraced to represent the displacements recorded by the monitoring system (section 4.3.2): start of the excavation, critical stage (which corresponds to undercrossing POT), and end of the excavation of the Underpass.

The effects of the excavation process in the POT obtained through PLAXIS will now be directly compared with the monitoring data presented within Chapter 4, hence vertical, horizontal and longitudinal displacements will be detailed in the following subsections.

6.3.3.1. Vertical displacements

Similarly to what was detailed in Figure 4.15, the plots represented in Figure 6.53 correspond to the vertical displacements recorded in the position of the each monitoring prism along POT, for advances No. 2, 16 and 26. In fact, these plots must be directly compared with those presented in Figure 4.15, as they represent the same movement for the same excavation advance although obtained in PLAXIS. The same reference was used to represent the abscissa of the presented plots, which was the section corresponding to array 4 (see Figure 4.13 for arrays location).

Figure 6.52 represents a typical reference-line used to obtain the displacements of the POT. This reference-line reproduces the position of the prisms RP3 along all arrays of the POT, which is then used to directly extract from the software the displacements due to the tunnelling works and therefore represent them in a plot. A similar strategy was used to reproduce the displacements recorded within the position of the remaining prisms along POT. This procedure allow one to interpret the POT movement obtained in PLAXIS and promptly compare it with the displacements recorded by the monitoring system.

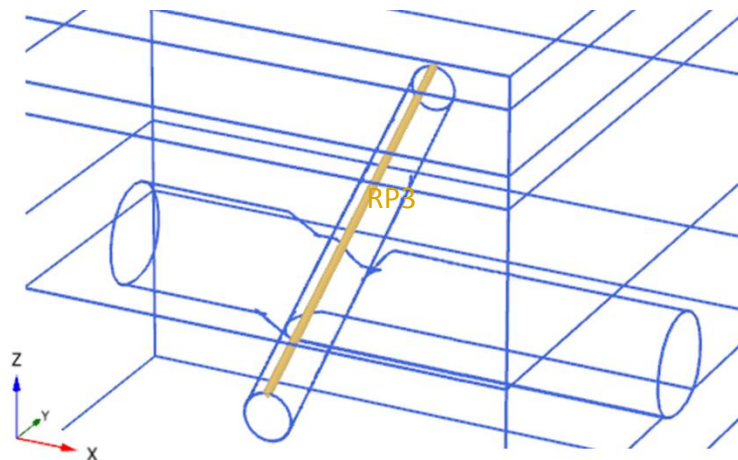


Figure 6.52 - Example of the reference-line used to obtain the POT displacements (along RP3)

Note that in order to simplify this comparison, the relative position of the prisms, which was used to describe the POT movement from PLAXIS, will be roughly described as RP1 to RP5 as specified, thus in Figure 6.53, a) corresponds to the vertical displacements recorded within the position of the RP1, b) corresponds to the RP2 and so forth.

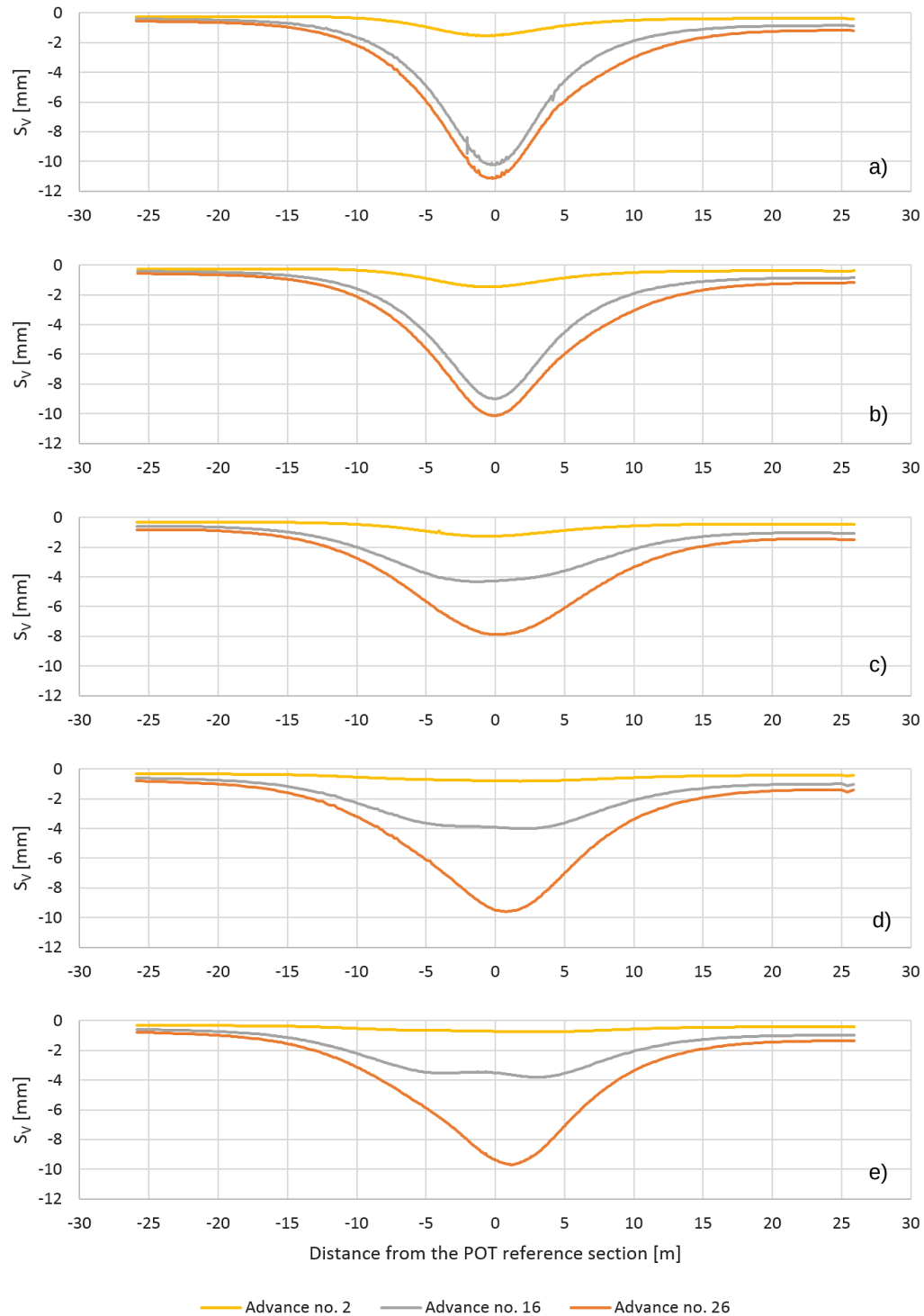


Figure 6.53 - Vertical displacements along POT obtained from PLAXIS 3D; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5

Regarding the vertical displacements recorded in the position of prisms RP1 along the POT, Figure 6.53 a), whilst the maximum vertical displacement recorded by RP1's along all arrays for the last recorded stage was 28 mm, the maximum vertical displacements recorded in PLAXIS for the same advance, was 11 mm, which corresponds to a difference of 60%. For the advance corresponding to the undercrossing, hereby defined as advance No. 16, the difference was a little lower, with the vertical displacements recording 10 mm, comparing with the 21.7 mm recorded by the monitoring system (corresponding to an error of approximately 53%). Although the considerable difference, the global movement of the POT reproduced from PLAXIS is similar to the real movement, as the maximum vertical displacement was also recorded within the position of the prisms RP1 since this is the prism closest to the excavation face.

Within the POT crown, whose displacements were registered within the position of the RP3's along POT, the maximum vertical displacement obtained through the PLAXIS model was 7.8 mm in advance No. 26, whereas in the reality the crown of the POT settled approximately 25 mm within array 4. This corresponds to the greatest difference between monitoring system and PLAXIS model results, which is approximately 68%. Since the maximum vertical displacements recorded by the monitoring system in the last recorded advance was approximately similar in all prisms (approximately 25 mm), it can be interpreted that the POT moved uniformly downwards, which contrasts with the registered by PLAXIS where the crown of the tunnel settled less than the rest of the tunnel's lining. This difference is explained by the assumed properties for the POT lining, which as it was already explained, did not accurately reproduce the real POT lining, since the tunnel lining's joints were not modelled.

Actually, the highlighted fact corresponds to one of the limitations of this three-dimensional model, the presence of those joints allows the POT to move more than as modelled in PLAXIS, as joints allow rotation between each segment. Thereafter the global difference recorded between monitoring system and PLAXIS model regarding vertical displacements may be related with this fact.

It can be seen from Figure 6.53 that, although the substantial difference in the maximum displacements, the 3D model developed represents quite well the influence of the excavation sequences on the POT. Through section 4.3.2.1., it was possible to identify that the first prisms to record the influence of the excavation face were prisms RP1 due to its proximity to the excavation; from this model, this influence is also clearly identified mainly in the curves corresponding to advance No. 16. Whilst for this advance the RP1 recorded a maximum vertical displacement of approximately 10 mm, RP4 and RP5 recorded approximately 4 mm which at this stage is acceptable. It should also be highlighted that the increase in the maximum displacement recorded from the advance No. 16 to advance No. 26, in prisms RP4 and RP5, which was approximately 60%, is similar between monitoring data and PLAXIS model results.

6.3.3.2. Horizontal displacements

Figure 6.54 represents the horizontal displacements corresponding to the movement perpendicular to the POT longitudinal axis for the same excavation advances as per Figure 6.53.

It should be referred that, since the POT longitudinal axis is deviated 15° from the perpendicular to the y -direction of the model, the horizontal displacements represented hereby were in fact defined as a results from the lining displacements in x - and y -direction. A similar procedure was taken to reproduce the longitudinal displacements which will be presented in the following section.

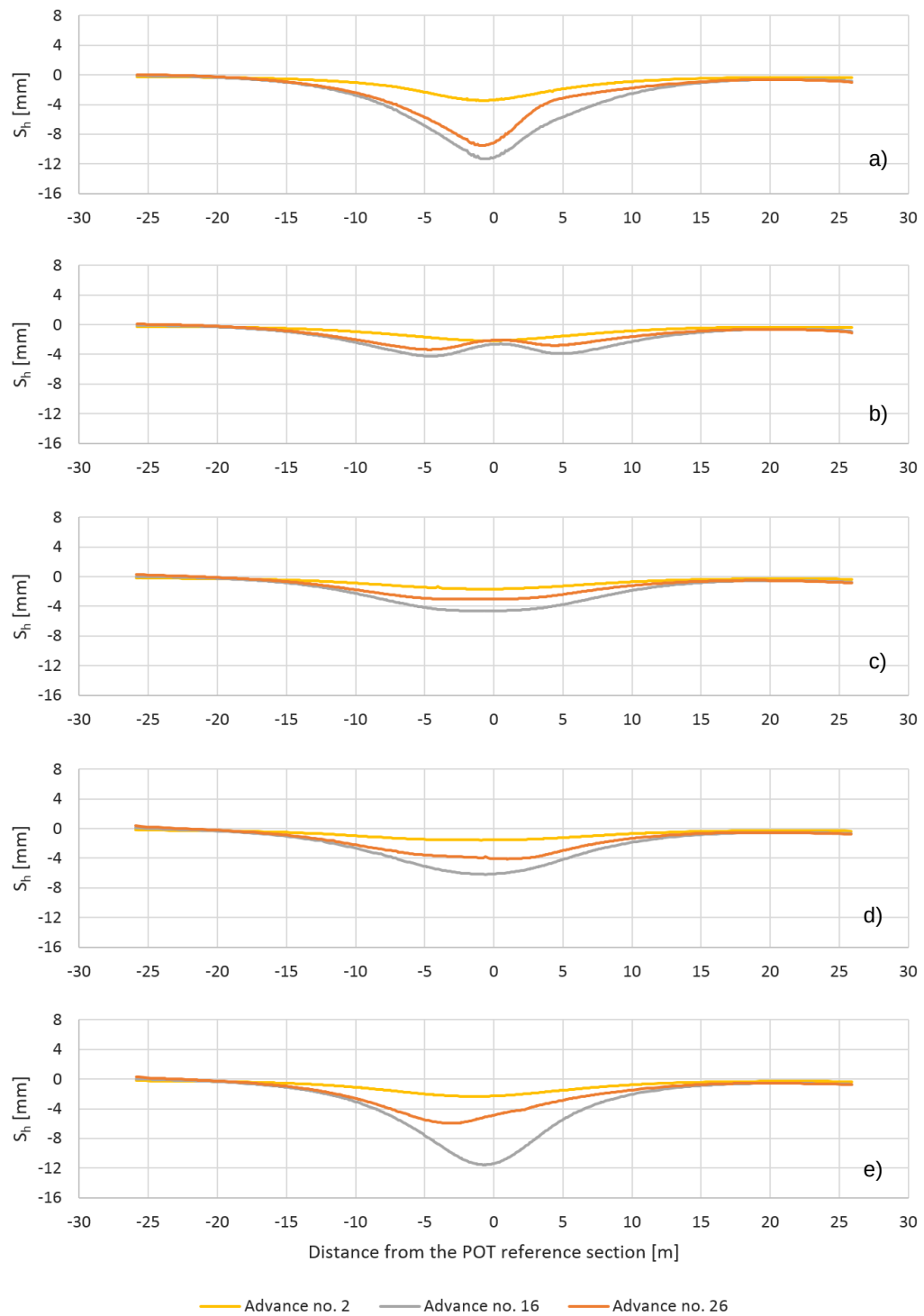


Figure 6.54 - Horizontal displacements along POT obtained from PLAXIS 3D; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5

Regarding the horizontal movement reproduced in Figure 6.54, unlike the vertical behaviour, the horizontal displacements obtained from PLAXIS were slightly greater than recorded by the monitoring system. Whilst the monitoring data revealed that the maximum horizontal displacement was 8.4 mm, recorded within prism RP1, the PLAXIS model returned a maximum value of 11.3 mm, also within the

RP1 position. From the back-analysis undertaken in Chapter 5 it was possible to identify that modifying the value of the in-situ earth pressure coefficient, K_0 , from 0.65 to 1.2, lead to greater horizontal displacements whereas the vertical displacements remained approximately similar. This fact may suggest that the recorded differences for the horizontal displacements between PLAXIS model and monitoring data are related with the in-situ earth pressure coefficient assumed for this analysis, which must had been assumed lower in order to also obtained lower horizontal displacements, without compromising the vertical displacements results. However the influence of this parameter will be further investigated in the following section.

It can be seen in Figure 6.54 a) that although the difference in the maximum values the PLAXIS model accurately reproduced the influence of the excavation progress, once it can be identified that the maximum displacement was recorded in the area around the position of array 5 (approximately -2 m away from the POT reference section) which was the array that recorded the major displacements within prisms RP1 as previously explained in Chapter 4. Besides, the recovery of the horizontal displacements due to the advance of the excavation, is also clearly identified between advances No. 16 and No. 26. Although the displacements recorded for advance No. 26 are greater than recorded from the monitoring system, the magnitude of the recovery between advances No. 16 and 26 is approximately constant, which denotes that the major difference between reality and PLAXIS model results relies on the maximum displacements recorded within advance No. 16. This difference is particularly identified in the area covered by prisms RP4 and RP5, which are the prisms that are most affected when the excavation reaches advance No. 18 as represented in Figure 6.11.

6.3.3.3. Longitudinal displacements

Similarly to the figures presented in the previous subsections, Figure 6.55 represent the longitudinal displacements along POT axis registered within the alignment of prisms RP1 to RP5, which is represented in a) to e), respectively. The displacements in this five alignments were reproduced from three different stages of the construction: advance: No. 2, No. 16 and No. 26. Note that as referred in the preceding subsection this movement was also composed of displacements in x - and y -direction.

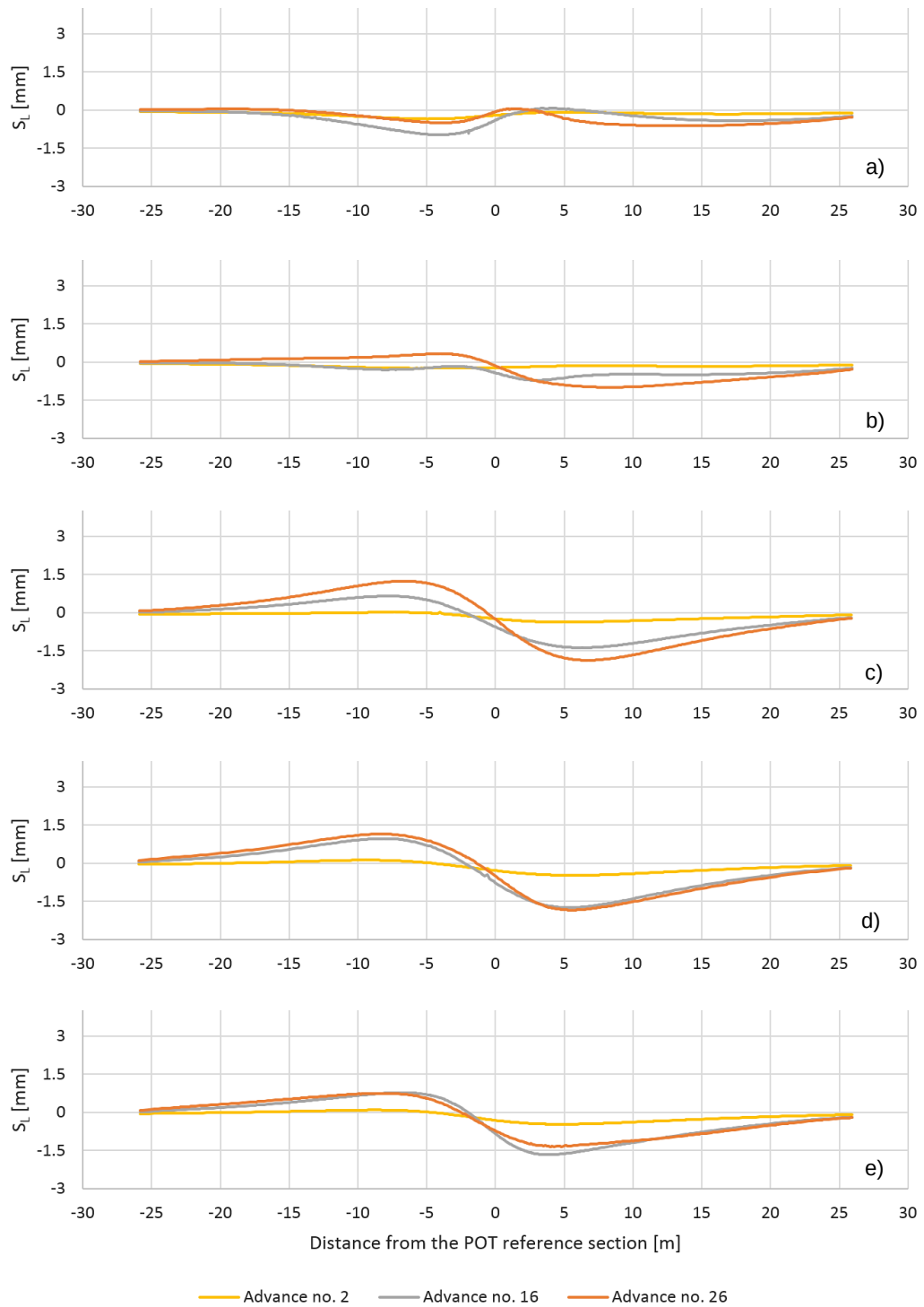


Figure 6.55 - Longitudinal displacements along POT obtained from PLAXIS 3D; a) RP1; b) RP2; c) RP3; d) RP4; e) RP5

In what concerns to longitudinal displacements, this movement was quite different to the registered by the monitoring system mainly in RP1 and RP2, besides the maximum displacements were slightly lower than the registered in the monitoring system (compare with Figure 4.21).

Whilst the monitoring data exposed an axis of rotation near the tunnel invert due to the discrepancy between longitudinal displacements within RP3 and the lowest RP1 and RP5, the PLAXIS model do not register this discrepancy as the RP3 moves approximately the same as RP5. This difference between monitoring data and PLAXIS results may rely on the fact that the joints were not considered, which prevent rotation to be identified between segments.

Another important aspect from the comparison between the results of the numerical model and the monitoring data, is that, unlike the real movement, the longitudinal displacements registered within PLAXIS were approximately symmetric in relation to the POT reference section (mainly registered within RP3, RP4 and RP5). Once again this difference is related with the fact that the POT lining was modelled as a monolithic structure which may induce a similar behaviour on both sides.

6.4. SENSITIVITY ANALYSIS

Within the same aim of the sensitivity study developed through the PLAXIS 2D model, the influence of various parameters will now be assessed through the developed 3D base model. Hence, besides the influence of each parameter on the soil behaviour and consequently on the POT behaviour, it will also be possible to detail the influence of the three-dimensional effect on each analysis.

The results that will be presented in this sensitivity study refer to the POT movement and were obtained from advance No. 27, which was the last advance considered in this model. Note that advance No. 28 was also performed, however as already explained, its results will not be considered.

Similarly to the exposed in the sensitivity study developed in Chapter 5, the influence of both soil strength and soil deformation parameters was controlled, which, considering only the parameters that revealed to have considerable influence on this analysis, resulted in an analysis of the influence of the undrained shear strength, c_u , the influence of the Young's modulus, E' , and finally the influence of the in-situ earth pressure coefficient, K_0 . For those, an upper and a lower bound of the parameter value was considered, which will be stated within each respective subsection. In relation to the soil parameters only the layers intersected by the tunnels, i.e., London Clay Unit A2 and Upper Mottled, suffered modifications.

Besides the influence of the soil properties, also the influence of the lining properties will be assessed within this section. Although the concrete stiffness assumed different levels within consecutive advances so that the model could incorporate the fact that concrete is a time-dependent material, this is still a very variable parameter for sprayed concrete. Thus, in order to determine the influence of the concrete stiffness in a tunnel construction in London Clay, a lower bound for the concrete stiffness was chosen to be compared with the base model, which was half of the assumed in the last. Regarding the POT lining, as it has been discussed throughout this chapter, it was modelled through a very simple model, which do not consider the lining segments nor the joints allowing freedom in rotation. It is known that the presence of the joints influence the stiffness of the lining, reducing it, thus the influence of the POT lining stiffness will also be evaluated within this sensitivity study.

Additional studies were carried out regarding the POT lining, which incorporated more accurately the influence of the lining joints. Those studies consisted in a Muir Wood approach, which takes into account the number of segments in each ring and an anisotropic analysis, as the lining stiffness is considerably different longitudinally and circumferentially (Schroeder, *et al.*, 2011). The adopted strategies will be further explained on the respective section.

In order to assemble this study as efficient as possible, only the maximum displacements of each direction registered within the alignments of the monitoring prisms will be compared. This strategy

prevent this study to be too exhaustive and allows the global behaviour to be adequately analysed. Hence, for the vertical and horizontal displacements the results presented within this study were registered from the alignment along POT for RP1, which was the prism recording the maximum vertical displacement, as well as the maximum horizontal displacement. For the longitudinal movement, the monitoring data revealed that RP3 was the prism recording the maximum longitudinal displacement, thus the results regarding this displacements presented hereafter were registered within its alignment.

6.4.1. INFLUENCE OF THE SOIL PROPERTIES

The present subsection follows the idea of the sensitivity study developed within PLAXIS 2D, which aim to evaluate the importance of each soil parameter regarding the general behaviour of the soil and thereafter the POT's behaviour. Note that for this calculations the POT is modelled with the same lining properties, thus the POT behaviour is uniquely affected by the modifications on the soil parameters. The lower and upper bounds considered in this study were exactly the same as applied in the 2D study, which were half and twice the best estimate of the considered parameter. Once again this variation is not realistic for the soil layers affected, however it allows to identify the influence of each parameter in an expedited way.

Note that the results for the best estimate hereafter presented correspond to the results obtained from the base model previously described. However, whilst in section 6.3.3 the last advance to be considered was advance No. 26 so it could be directly compared the displacements recorded by the monitoring system, in this study, the results presented from the base model refers to advance No. 27 which is the last reasonable advance in the model developed. Assuming this fact, some slightly differences in the maximum displacements are expected to be seen.

6.4.1.1. Young's modulus influence, E'

Figure 6.56, Figure 6.57 and Figure 6.58 represent the displacements of the POT for different values of the Young's modulus of the soil. The bounds considered for the Young's modulus in this analysis were exactly the same as represented in Table 5.2, which affect the London Clay and the Upper Mottled layers in the same proportion.

As it was already mentioned in Chapter 5, the Young's modulus influence is not directly proportional since it only affects the elastic part of the displacements. In fact, from the base model description, one could conclude that the yielding area has greater significance in the 3D model once the POT affects the stress redistribution, consequently it will also have more influence on the displacements of the POT.

A quick view on the following plots allow one to verify that reducing or increasing the Young's modulus in the same proportion leads to a similar behaviour, i.e. the displacement curves regarding the same displacement present similar shapes, which shows that the influence of the Young's modulus is quite uniform throughout all the extension of the POT. However the magnitude of the displacements registered for the different bounds is very different from the reproduced in the 2D analysis.

Figure 6.56 reproduces the vertical displacements along POT from the best estimate, i.e. from the base model, from a model developed with the lower bound assumed for the Young's modulus and from a model developed with the upper bound.

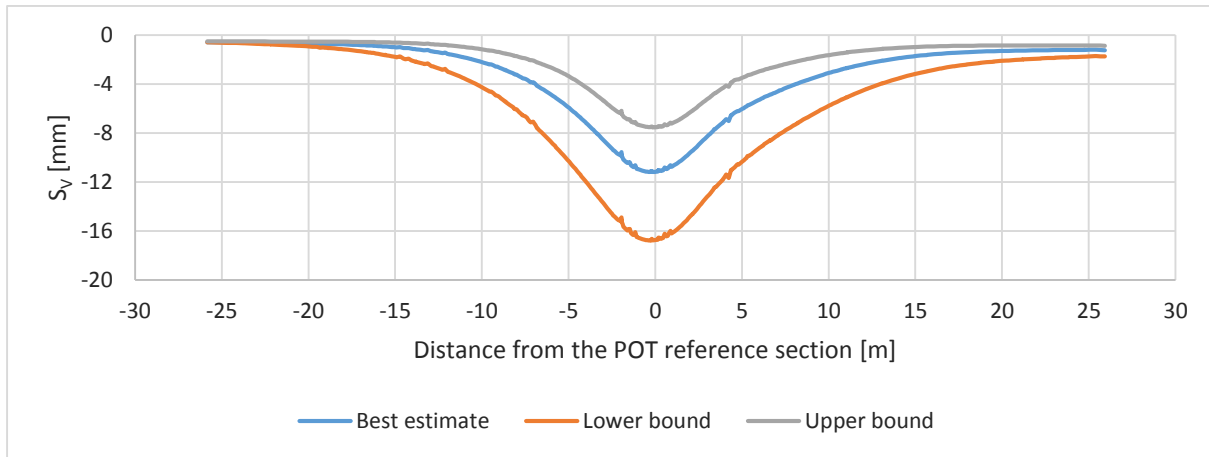


Figure 6.56 - Vertical displacements recorded within RP1 for different values of the Young's modulus, E

From Figure 6.56 it can be seen that increasing the Young's modulus to twice its best estimate (refers to Upper bound) leads to a decrease in the vertical displacements of approximately 33%, which corresponds to a variation from 11.2 mm to 7.5 mm. Regarding the analysis made with the lower bound, it can be seen that the displacements increased in the same proportion, i.e. 33%, which corresponds to an increase from 11.2 mm to 16.8 mm.

Comparing this variations with the registered in the analysis from the 2D model, one can see that in the 3D model the same variation of the deformation parameter, E' , led to lower variations on the vertical displacements. This difference between models, which is from approximately 40% in the 2D model to 33% in the 3D model, may be related with the three-dimensional arching effect. Whilst in the 2D model this effect is imposed through specific techniques, a three-dimensional numerical model requires no artifice. Despite being strongly structured, the techniques used in the two-dimensional models still have some limitations and do not reproduce the same ground behaviour as a tri-dimensional model. Hence considering that the arching effect allows the stresses to redistribute more effectively, it is reasonable that a 2D model experience greater variations on the displacements than a 3D model.

The following plot, Figure 6.57, represents the horizontal displacements for the same variation of the Young's modulus, as per Figure 6.56.

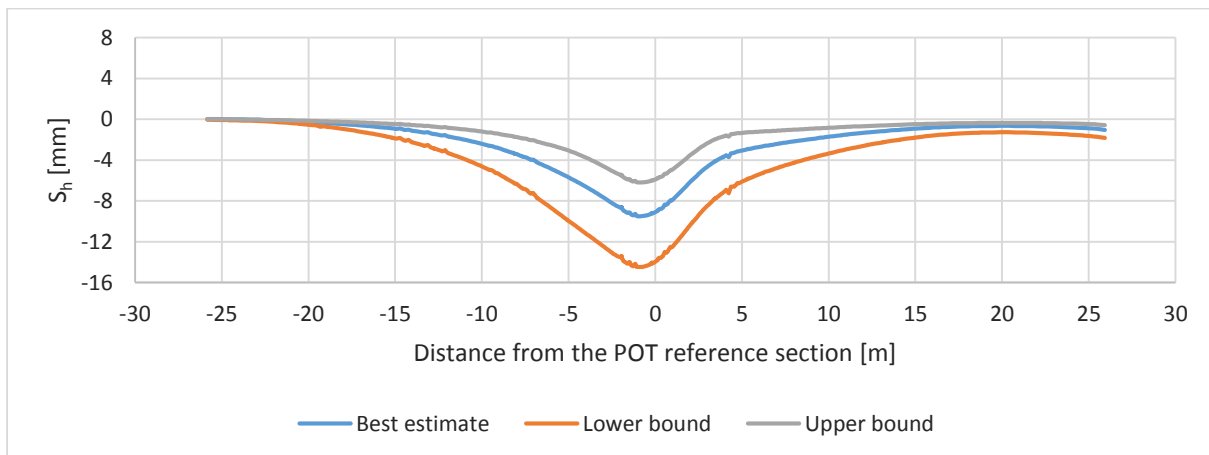


Figure 6.57 - Horizontal displacements recorded within RP1 for different values of the Young's modulus, E

As it can be seen, the horizontal displacements follow the same tendency as the vertical ones, i.e. the increasing and the decreasing in the maximum displacements occurs in the same proportion, which is now approximately 35%.

Regarding the longitudinal displacements, Figure 6.58, it can be seen that the variation of the maximum longitudinal displacement between base model and upper bound model is approximately 27% (which represents a variation from 1.9 mm to 1.3 mm), as well as between base model and lower bound. Note that the shape of the displacements curves remain similar.

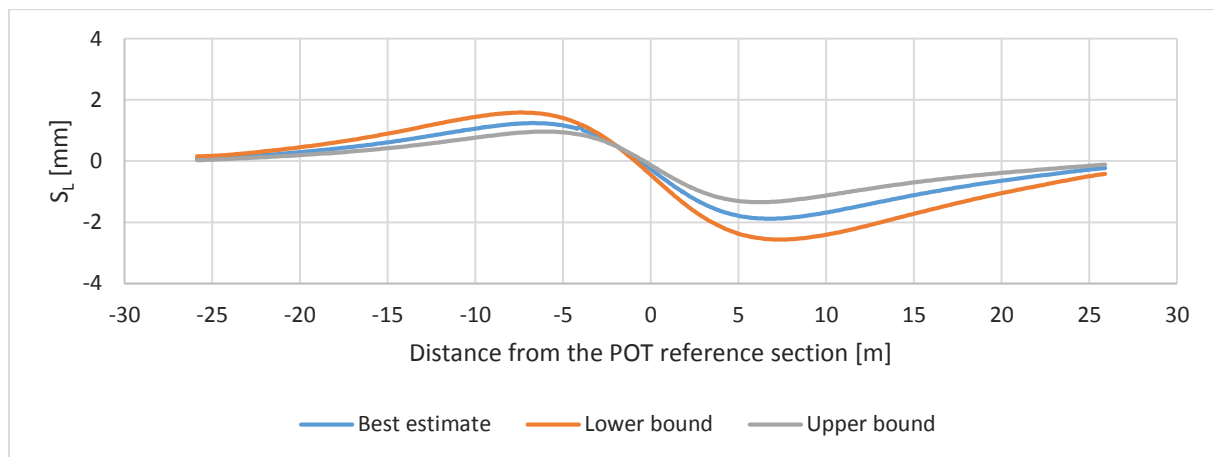


Figure 6.58 - Longitudinal displacements recorded within RP3 for different values of the Young's modulus, E

As it was possible to verify, the displacements variation registered in the 2D model (approximately 45% between best estimate and bounds) was greater than the one recorded within the 3D model, this difference may be affected by the POT presence. Bear in mind that the 2D model do not consider the POT lining stiffness which is expected to reduce the longitudinal displacements of the soil in the vicinity of the POT. In fact, whilst the 2D model developed with the lower bound of the parameter being assessed presented a maximum longitudinal displacement of approximately 15 mm, the 3D model only presented approximately 2.6 mm.

6.4.1.2. Undrained shear strength, c_u

For the soil strength parameter analysed within this sensitivity study, the variation on the values was the same applied in Chapter 5, which is represented in Table 5.3. Note that the lower and upper bound only affected the London Clay and Upper Mottled Formations, for London Clay the gradient of the undrained shear strength was assumed the same as the best estimate in order to prevent failure.

Figure 6.59, Figure 6.60 and Figure 6.61 represent respectively the maximum vertical, horizontal and longitudinal displacements of the POT, for the two bounds considered for the undrained shear strength as well as for the best estimate of this parameter.

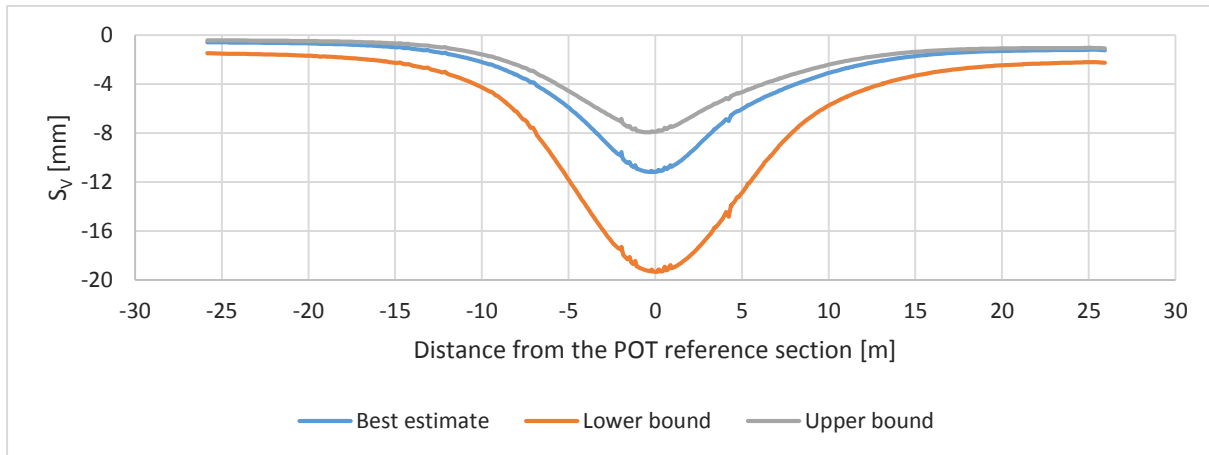


Figure 6.59 - Vertical displacements recorded within RP1 for different values of the undrained shear strength, c_u

In Figure 6.59 it can be seen that the increase on the vertical displacements is greater than the decrease for the same proportion on the undrained shear strength, i.e., the percentage of increase in the vertical displacements when the undrained shear strength is reduced for half of its best estimate is greater than the percentage of the decreasing displacements when the undrained shear strength is increased for twice of its best estimate. Note that this is a strength parameter thus decreasing the strength of the soil will obviously make it more susceptible to deformation, as the yielding area increases substantially.

Comparing these results with the ones obtained from the 2D model (see Figure 5.35), one may notice that the influence of the undrained shear strength is much more significant in the 2D model, however it should be referred that once again the lining stiffness of the POT was not considered in that analysis, thus in the 3D model, part of the displacements induced from a reduction on the undrained shear strength are absorbed by the POT which dissipate them through its extension. In fact it can be seen that the displacement curve for the lower bound is narrower in the 2D model than in the 3D model, which induces that the displacements are dissipated along the POT.

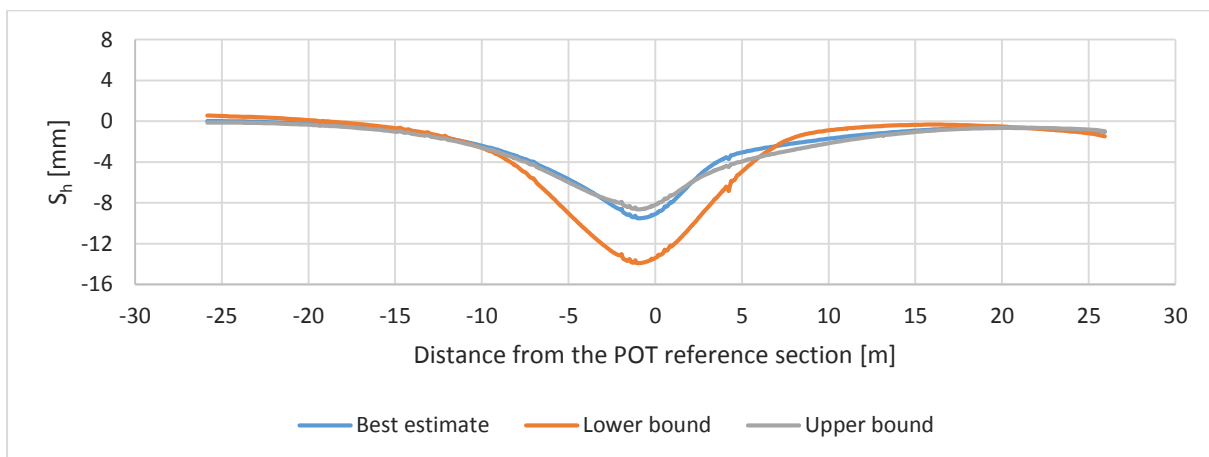


Figure 6.60 - Horizontal displacements recorded within RP1 for different values of the undrained shear strength, c_u

In what concerns to displacements perpendicular to the POT longitudinal axis, it can be seen in Figure 6.60 that the upper bound of the undrained shear strength almost does not affect the behaviour of the

POT, whereas reducing it induces the same tendency of the vertical displacements, i.e. increase on the maximum displacements whilst the shape of the curve remains approximately similar. It can still be identified a distribution of the displacement along the POT as the curve becomes slightly wider.

Regarding the longitudinal displacements, it can be seen in Figure 6.61 that, within the POT, they follow the same tendency as the vertical ones, presenting a much smaller maximum displacement for the lower bound in the 3D than in the 2D, which again indicates the influence of the POT lining stiffness.

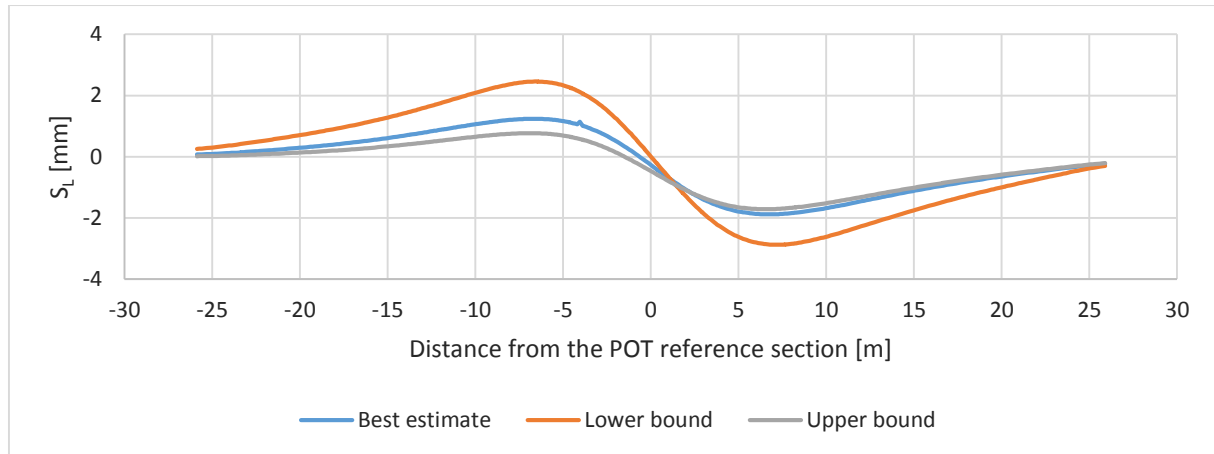


Figure 6.61 - Longitudinal displacements recorded within RP3 for different values of the undrained shear strength, c_u

From the previous plots it was possible to identify that, when the undrained shear strength is decreased the relation between vertical, horizontal and longitudinal displacements is quite similar, with all recording similar variation. The influence of this parameter is of some interest for the case study as it improves some of the results obtained for the POT, note that if the undrained shear strength was lower than reported by the geotechnical reports (which would approximate the model to the lower bound hereby assumed), it would lead to greater vertical and longitudinal displacements, which would be closer to the reality, however it would also affect the horizontal displacements leading to displacements greater than recorder for the monitoring data. It should be mentioned that although this analysis aim to clarify the influence of each parameter individually, the results obtained from PLAXIS are a complex combination of a large range of parameters, consequently it makes the search for a more accurate model a very difficult process.

6.4.1.3. In-situ earth pressure coefficient, K_0

The in-situ earth pressure coefficient, as already shown in the 2D analysis, is one of the most interesting parameters to be assessed within excavations on London Clay, as it has a great influence on the stress relief. The bounds considered for this parameter in this subsection were the same as assumed in Table 5.4, where only the two mentioned layers were affected. The following plots, Figure 6.62, Figure 6.63 and Figure 6.64, reproduce the influence of this parameter in this case study.

Similarly to what was identified in the sensitivity study developed in the 2D model, the lower bound assumed for the in-situ earth pressure coefficient almost does not affect the vertical displacements obtained for the POT, as they are almost coincident with the results from the base model, reproduced in Figure 6.62 as the best estimate. On the contrary, the upper bound of this parameter has a certain

influence on the POT movement, which as it can be identified, propagates in all its extension. This effect was also identified in the analysis done in the 2D model, and proves that the in-situ earth pressure coefficient has a wide effect on the ground behaviour. The increase in this parameter leads to greater horizontal stresses which after an excavation produces a significant stress relief that propagates in all extension of the soil layers. Comparing the results in this analysis with the presented in Figure 5.37, it can be identified that the maximum vertical displacement registered with the upper bound is much lower in the 3D analysis than in the 2D, once again, this difference is mainly due to the presence of the POT, which prevents the soil to deform much.

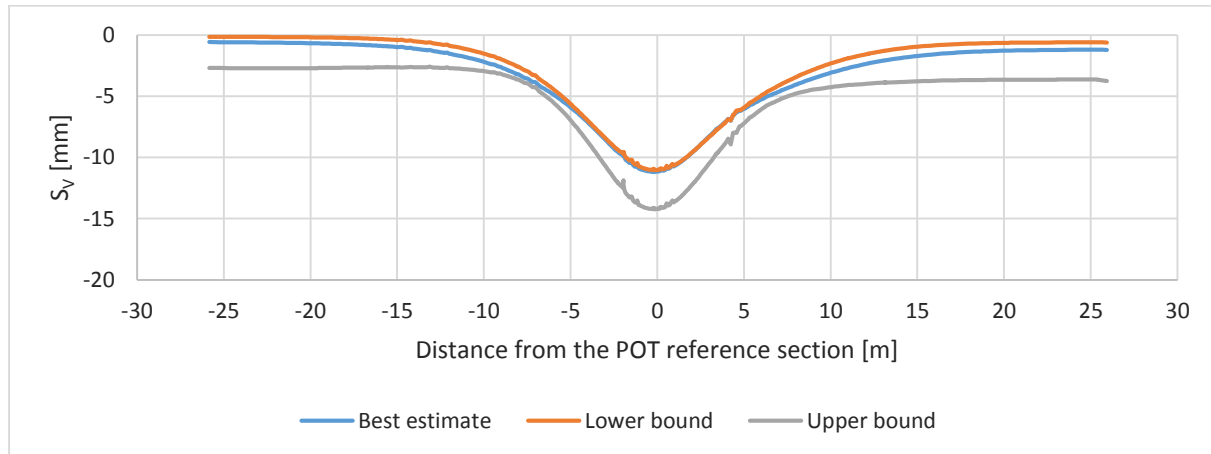


Figure 6.62 - Vertical displacements recorded within RP1 for different values of the in-situ earth pressure coefficient, K_0

Whilst a lower value for the in-situ earth pressure coefficient almost does not affect the vertical displacements, it affects the horizontal movements although in a lower level. Figure 6.63 reproduces the influence of the parameter being analysed in the horizontal direction.

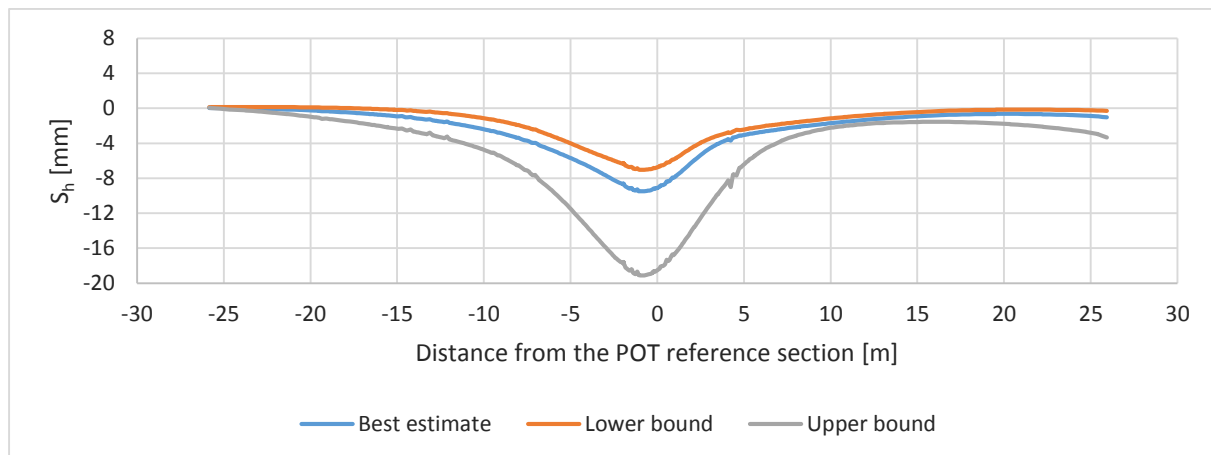


Figure 6.63 - Horizontal displacements recorded within RP1 for different values of the in-situ earth pressure coefficient, K_0

Section 6.3.3. allowed to verify that the horizontal displacements obtained from the base model were slightly greater than recorded by the monitoring system. From Figure 6.63 one can identify that reducing K_0 to 0.6 (see Table 5.4) also reduces the horizontal displacements, thus applying this modification to

the base model would improve it, however only regarding horizontal displacements, as the influence of this modification almost does not affect the vertical displacements neither the longitudinal displacements (Figure 6.64).

An increase on the in-situ earth pressure coefficient leads to wider horizontal and longitudinal displacements similarly to what was identified in the vertical displacements.

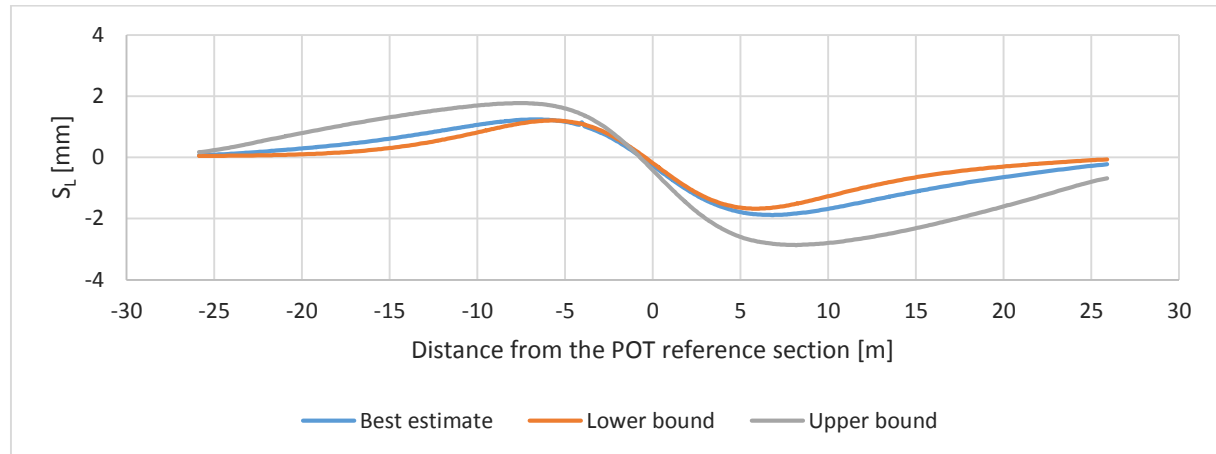


Figure 6.64 - Longitudinal displacements recorded within RP3 for different values of the in-situ earth pressure coefficient, K_0

Similarly to the movements already characterized, the longitudinal displacements recorded in the 3D model for the upper bound of the in-situ earth pressure coefficient are much lower than the ones recorded in the 2D model (see Figure 5.38) due to the presence of the POT.

6.4.2. INFLUENCE OF THE TUNNEL LININGS PROPERTIES

Although the soil properties are generally the more difficult to characterize and simulate, to model the tunnel lining properties is also a very challenging aspect in the numerical models. Due to some limitations, the software does not reproduce accurately the structural behaviour of the tunnel linings, the sprayed concrete lining because it does not simulate its curing, and, for the POT lining, because it does not reproduce the lining segments, hence in order to determine the influence of the lining stiffness in the developed model the lining properties will be investigated within this section.

6.4.2.1. SCL tunnel lining properties

The properties assumed for the sprayed concrete applied in the new tunnel, as discussed in section 6.2, considered two different levels of stiffness to simulate the concrete curing, $E = 10 \text{ GPa}$ in a first stage of the application of the lining and $E = 30 \text{ GPa}$ in the ensuing stage. For this sensitivity study two different levels were assumed, in the first stage the Young's modulus was reduced to 5 GPa and in the final stage the stiffness was assumed as $E = 15 \text{ GPa}$. The temporary sprayed concrete applied in the excavation face between advances was also modelled with $E = 5 \text{ GPa}$.

This study intends to clarify the influence of the support stiffness on the POT behaviour, thus, the maximum vertical, horizontal and longitudinal displacements obtained for this modification are

presented as lower bound in the following Figure 6.65, Figure 6.66 and Figure 6.67, respectively. The results from the base model are also presented (best estimate).

Although this modification consisted of reducing the global stiffness of the support to half its initial assumed value, it can be seen that it does not affect significantly the displacements of the POT. Whilst in Figure 6.65 it can be identified a slightly increase on the vertical displacements, Figure 6.66 and Figure 6.67 show that the stiffness of the concrete almost does not affect the horizontal and longitudinal movement of the POT.

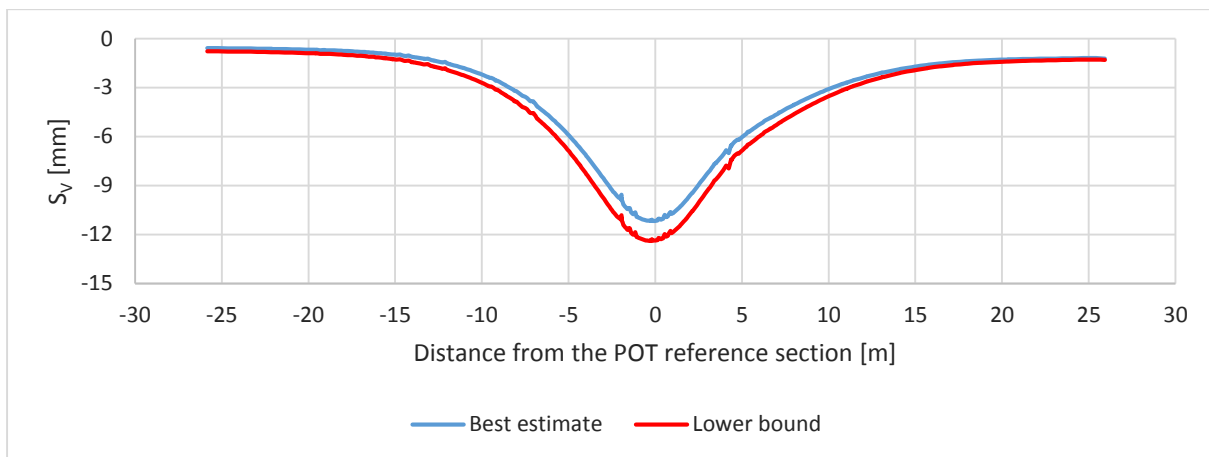


Figure 6.65 - Vertical displacements recorded within RP1 for different values of the concrete lining stiffness, E_c

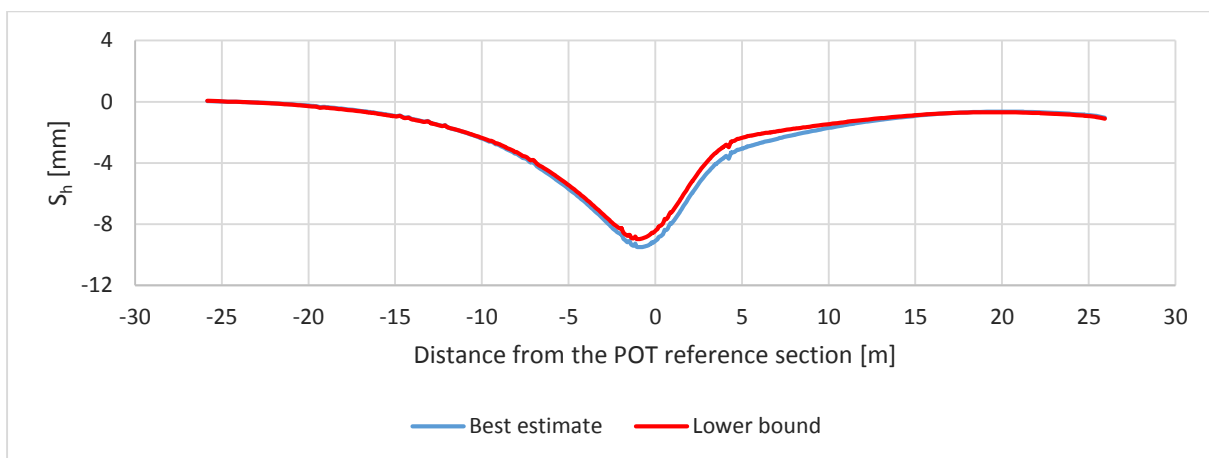


Figure 6.66 - Horizontal displacements recorded within RP1 for different values of the concrete lining stiffness, E_c

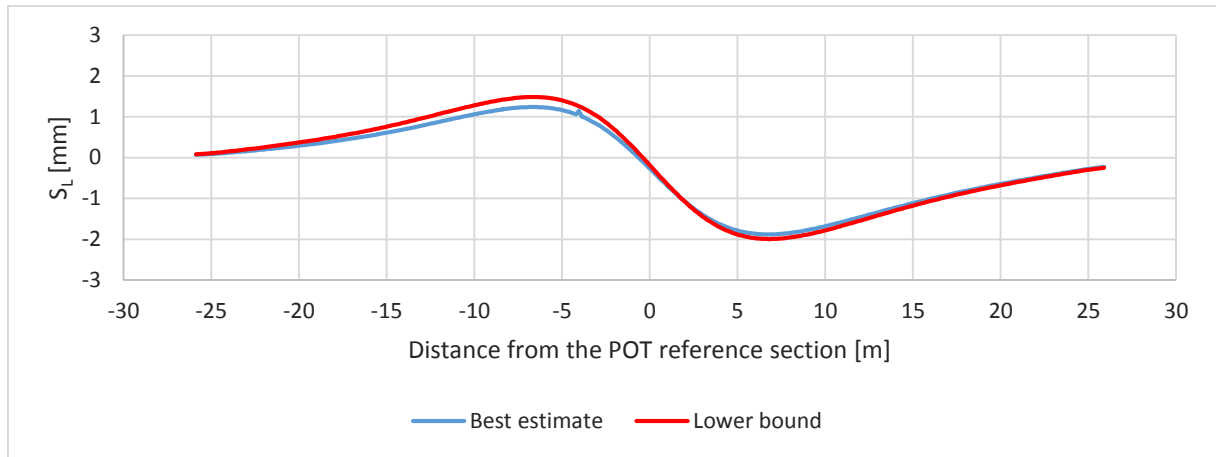


Figure 6.67 - Longitudinal displacements recorded within RP3 for different values of the concrete lining stiffness, E_c

It is known that London Clay has a very satisfactory behaviour in what concerns to tunnelling works, from this sensitivity study on can conclude that in fact, reducing in the Young's modulus of the support does not affect significantly the ground deformation, which may be related with the three-dimensional arching effect. The stress redistribution acts in order that the soil maintain a satisfactory behaviour in the vicinity of the excavation even when the support is less resistant.

6.4.2.2. POT lining properties

Since this study focuses on the POT behaviour, the stiffness of its lining is expected to have a significant importance. In order to understand its influence, two different models with different stiffness for the Pot lining were developed to compare the behaviour of the POT with the one registered in the base model. Maintaining all the remaining parameters equal to those assumed in the same model, the stiffness of the POT was modified to half its equivalent value (refers to Table 6.1) and to one fifth.

Figure 6.68, Figure 6.69 and Figure 6.70 reproduce the influence of the POT lining stiffness on the vertical, horizontal and longitudinal displacements, respectively.

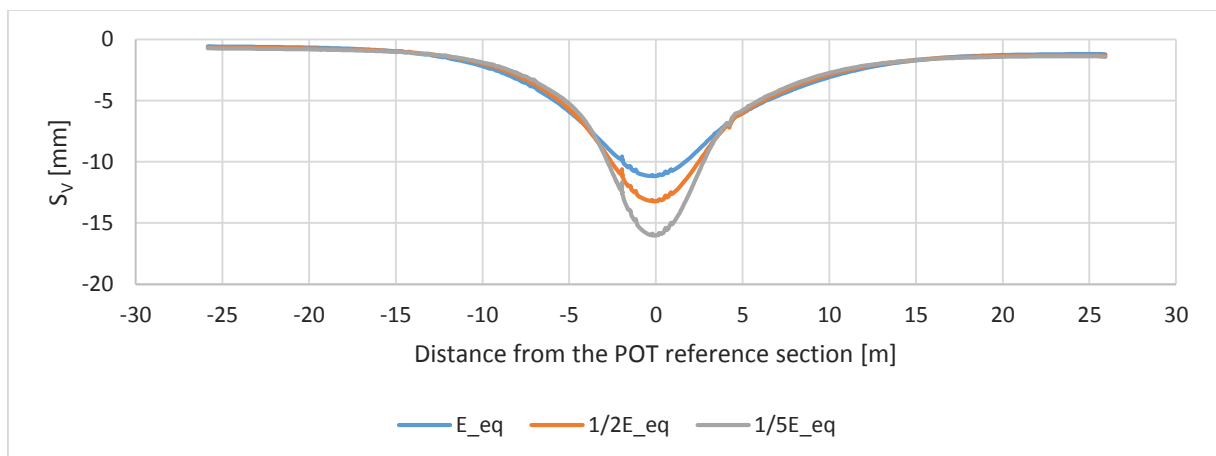


Figure 6.68 - Vertical displacements recorded within RP1 for different values of the POT lining stiffness, E_{POT}

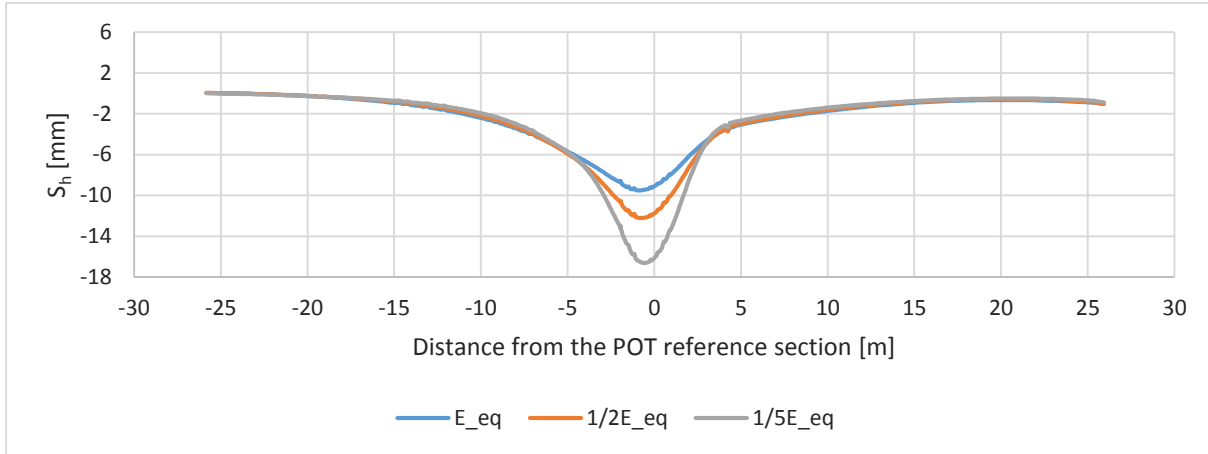


Figure 6.69 - Horizontal displacements recorded within RP1 for different values of the POT lining stiffness, E_{POT}

It can be seen that, for the vertical and horizontal displacements, decreasing the lining stiffness only affects the maximum value, i.e. only affect the behaviour of the POT directly above the excavation. To reduce the lining stiffness to 50% of its value causes the maximum vertical displacements to increase 16%, whereas the horizontal value increases 22%; to reduce it for 20% of its value leads to an increase in the vertical displacements of 30% whilst the horizontal displacement increases 43%. The percentage in the horizontal displacements may be higher due to the in-situ earth pressure coefficient, which is greater than one.

Away from the excavation (i.e. away from the POT reference section). Reducing the lining stiffness has no influence, which can be identified in Figure 6.68 and Figure 6.69 through the overlapping between the different curves. This fact shows that away from the excavation, the POT behaviour is mainly affected by the soil parameters, whilst in the vicinity of the excavation the stiffness of the lining represents a more important role, which is understandable since the removal of the soil mobilizes this area.

For the longitudinal displacements the scenario is similar (Figure 6.70) reducing the lining stiffness to 50% of its value causes the maximum displacements to increase 25%, whilst reducing it to 20% of its value leads to an increase on the maximum value of 46%

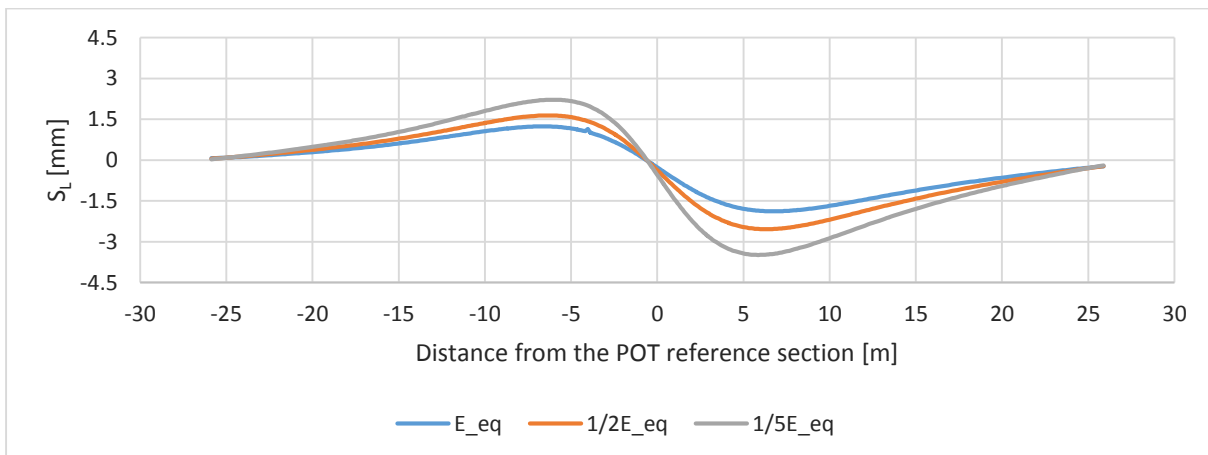


Figure 6.70 - Longitudinal displacements recorded within RP3 for different values of the POT lining stiffness, E_{POT}

Although this subsection focuses on the POT stiffness influence, this study is still very limited since the segments are not modelled, thereafter the behaviour of the POT is more uniform than in the reality. The following section will present different techniques in order to improve the model regarding the POT lining.

6.4.3. ADDITIONAL STUDIES

As it was already introduced, the simplification taken on the POT lining simulation is one of the major limitations of the developed three-dimensional model, therefore some additional studies were carried out in order to reproduce the influence of the lining joints. These studies consisted of a Muir Wood approach and an anisotropic analysis for the POT lining, which act on the POT lining stiffness.

For this study, besides the vertical and horizontal displacements recorded in the alignment of prisms RP1 and the longitudinal displacements recorded by RP3, also the vertical and horizontal displacements in RP5, and the vertical in RP3 will be analysed. This additional references were assumed since the analysis in this section act directly in the POT lining behaviour, thus it is of much interest to see the influence of this approaches on the global POT behaviour.

6.4.3.1. Muir Wood approach

When a lining is composed of segments, the stiffness at the joints may be appreciably less than elsewhere (Muir Wood, 1975), hence in order to include this reduction in the lining stiffness due to the presence of the joints, a Muir Wood approach was embraced within this sensitivity study.

From Muir Wood (1975), the effective value of the second moment of area for each ring, I_e , can be defined as presented in Equation 6.5.

$$I_e = I_j + \left(\frac{4}{n}\right)^2 \cdot I \quad (6.5)$$

where I_j corresponds to the second moment of area at the joints, n the number of equal segments in each ring and I the second moment of area of the cross section. From Muir Wood (1975) the following relation is embraced: $I_j \ll I$, thus assuming that each POT lining ring is composed of 6 equal segments, which is a simplification from the reality, it leads to an effective value of second moment of area for each ring, I_e , approximately equal to 44% of the actual second moment of area, i.e.:

$$I_e \approx 0.44 \cdot I \quad (6.6)$$

From Table 6.1, the second moment of area of the section assumed for this analysis is $2.829 \times 10^{-5} \text{ m}^4/\text{m}$, or $1.437 \times 10^{-5} \text{ m}^4/\text{segment}$. Consequently, the effective second moment of area within a POT ring can be assumed as $0.638 \times 10^{-5} \text{ m}^4/\text{segment}$.

Although this approach only considers the influence of the segments within each ring, the reduction on the lining stiffness will be assumed equal for both radial and longitudinal directions. Thereafter, considering the effective second moment of area, from Equation 6.3, a new equivalent thickness for the

POT lining is determined, which for this case is $t_{eq} = 0.0727 \text{ m}$, consequently, from Equation 6.4, the Young's modulus must now be assumed as $E_{eq} = 39.17 \text{ GPa}$.

Further to the foregoing reasoning, a new model was developed assuming the new properties for the POT lining. The results obtained from this model are presented in Figure 6.71, Figure 6.72 and Figure 6.73 as well as the results of the base model in order to allow for an accurate comparison.

The vertical displacements were analysed in the alignment of RP1, RP3 and RP5. As it can be seen in Figure 6.71 only RP1 registered a different behaviour for the Muir Wood approach, in which the maximum displacement registered an increase of 7%. Note that this approach only affected the second moment of area, which in relation with the base model, led to a decrease in the equivalent thickness, but an increase in the equivalent Young's modulus. The sensitivity study developed within the tunnel lining properties allowed to see that a reduction on the Young's modulus of the POT lining lead to an increase in the vertical displacements, however the slightly increase that is possible to identify in the plot regarding RP1 (Figure 6.71 a)), may be related with the decrease of the equivalent thickness.

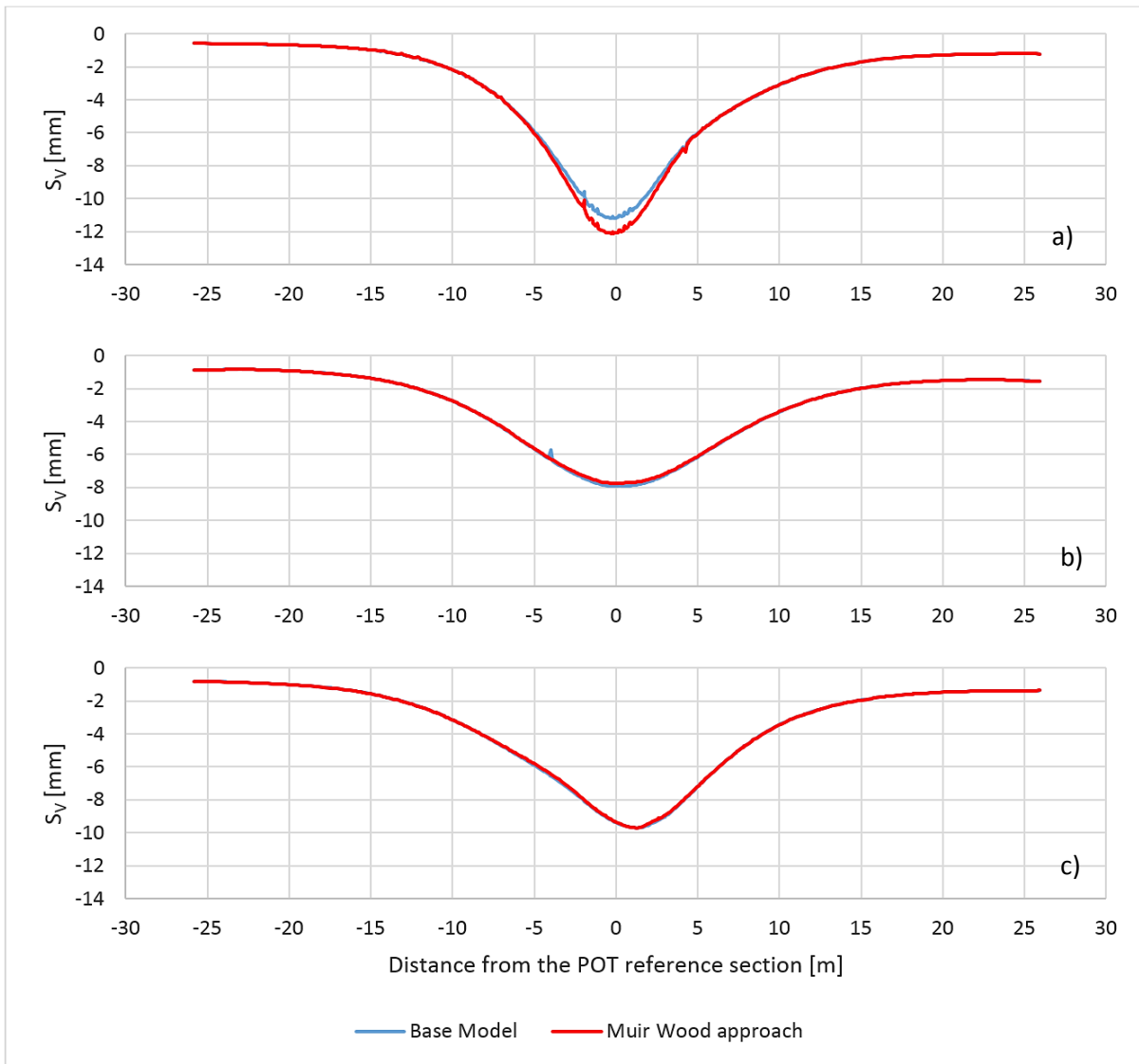


Figure 6.71 - Vertical displacements recorded for the Muir Wood approach within different prisms; a) RP1; b) RP3; c) RP5

Regarding the horizontal displacements, Figure 6.72, once again the only slightly registered differences occurred within RP1, which is related with its proximity to the excavation face in the early stages of the excavation. The longitudinal displacements within RP3 (Figure 6.73) registered exactly the same as the base model.

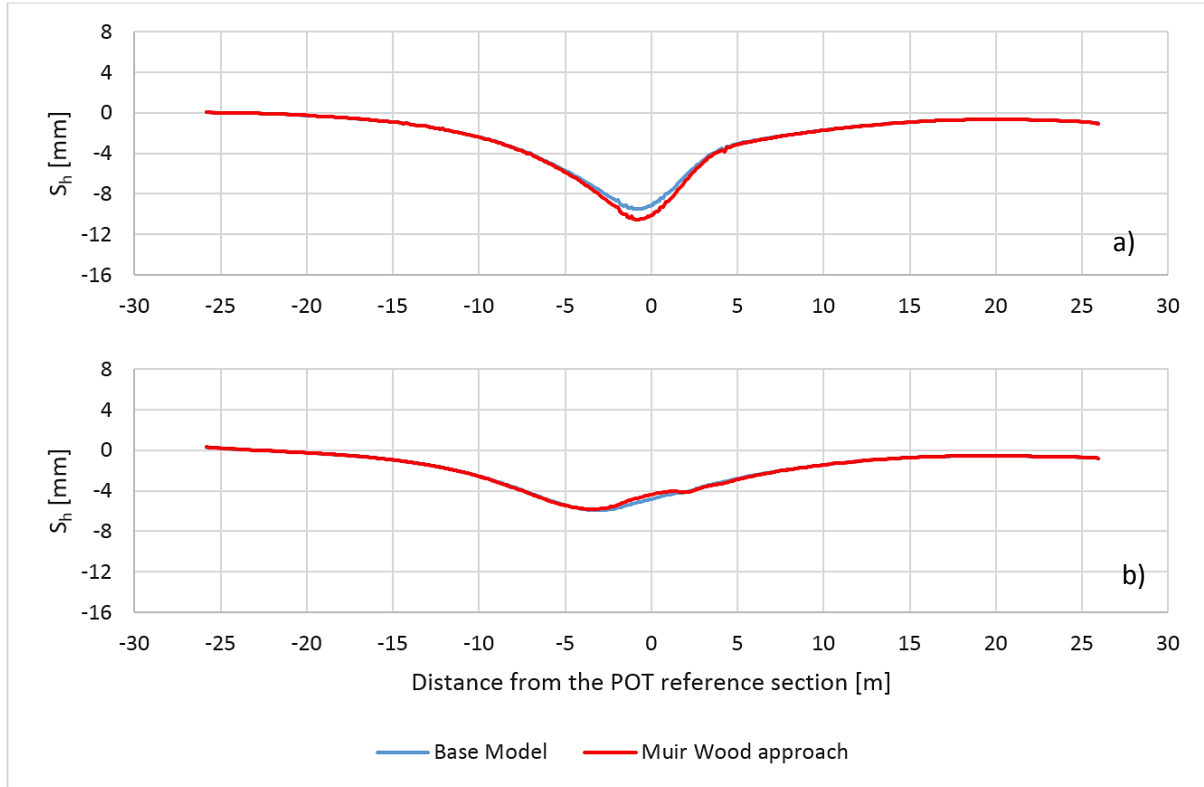


Figure 6.72 - Horizontal displacements recorded for the Muir Wood approach within different prisms; a) RP1; b) RP5

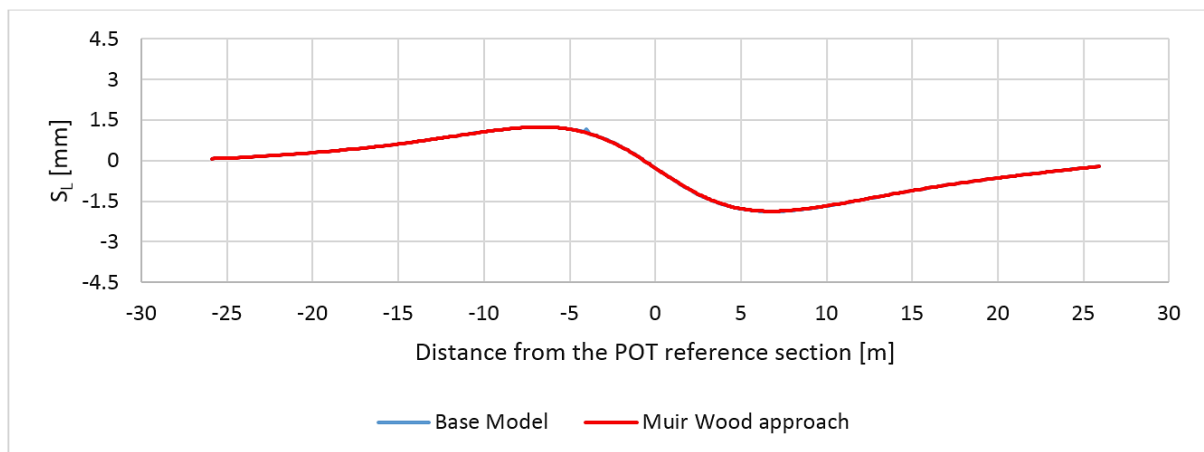


Figure 6.73 - Longitudinal displacements recorded within RP3 for the Muir Wood approach

Although Muir Wood is a well-known approach, one can conclude from this study that it may not be suitable for deformation analysis of this particular case since no significant variation was identified. In

fact, the proposed approach of the Muir Wood is included in a study regarding the loading on a circular tunnel, hence it may be more advisable for analysis regarding structural forces than deformational aspects.

6.4.3.2. Anisotropic analysis for the POT lining

The base model description showed that the POT lining was modelled with a linear elastic model with isotropic behaviour, which means that the lining stiffness was assumed equal for both radial and longitudinal directions. However, it is known that the lining stiffness for cast iron tunnels is more flexible longitudinally than circumferentially due to the extent of the caulking groove (Schroeder, *et al.*, 2011). Figure 6.74 represents the caulking groove in the two different type of joints, radial and circumferential. As it can be seen, the caulking groove is of very limited depth for the radial joint but extends all the way to the lining skin for the circumferential joint, hence it allows the lining to move more longitudinally than circumferentially.

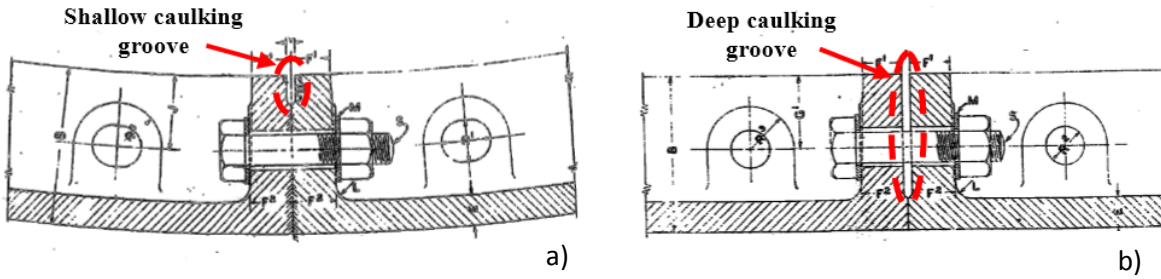


Figure 6.74 - Cast iron tunnels lining joints details (reproduced from Schroeder, *et al.*, 2011); a) Radial joint; b) Circumferential joint

In order to incorporate this effect in the model developed, a new approach was carried out, in which the POT lining was modelled with anisotropic behaviour. Given the difficulties in estimating an average longitudinal stiffness Schroeder, *et al.* (2011) considered an extreme condition reducing the second moment of area in longitudinal direction, I^{long} , by a factor of 100, whilst the other parameter remain constant. However, for an anisotropic analysis conducted in PLAXIS only the Young's modulus can be modified in different directions, the second moment of area, I , which is only dependent on the lining thickness, is assumed constant in both directions. Hence, following the idea presented above, the relation exposed in Equation 6.7 is assumed for this analysis.

$$(EI)^{long} = \frac{(EI)^{circ}}{100} \quad (6.7)$$

where EI represents the flexural stiffness in longitudinal or circumferential direction as indicated. The foregoing discussion culminates in a reduction factor directly applied in the longitudinal Young's modulus, i.e.:

$$E^{long} = \frac{E^{circ}}{100} \quad (6.8)$$

Nevertheless, due to numerical limitations of the software, the relation presented by the Equation 6.8 cannot be applied. PLAXIS imposes the following relation for the Young's modulus:

$$\nu_{POT} < \sqrt{E_1/E_2} \quad (6.9)$$

where ν_{POT} represents the Poisson's ratio of the lining material to be modelled, and E_1 and E_2 represents the Young's modulus in longitudinal and circumferential direction, E^{long} and E^{circ} , respectively. Considering that cast iron presents $\nu = 0.26$ (as modelled in the base model), it leads to the following relation.

$$E_1 > 0.0676 \cdot E_2 \quad (6.10)$$

Finally, in order to develop an anisotropic analysis respecting this condition, the following relation was adopted. Note that the author assumed a small margin in the longitudinal stiffness for the condition to be applied without numerical problems.

$$E^{long} = \frac{E^{circ}}{14.5} \quad (6.11)$$

Therefore, a new numerical model was developed in which, based on the equivalent Young's modulus of the POT from Table 6.2, it was assumed $E^{circ} = 26.11 \text{ GPa}$ and $E^{long} = 1.8 \text{ GPa}$.

The following plots, Figure 6.75, Figure 6.76 and Figure 6.78, reproduce the results for the anisotropic analysis developed based on the conditions presented above, which is to be compared with the results from the base model. In addition to the relation presented in Equation 6.11, another model was developed also assuming an anisotropic behaviour, in which the longitudinal stiffness was reduced by a factor of 10, i.e., $E^{long} = 2.61 \text{ GPa}$.

Figure 6.75 represents the results of the three approaches recorded in the alignment of RP1, RP3 and RP5 regarding the vertical displacements. It can be seen that reducing the longitudinal stiffness, either to 2.61 GPa or to 1.8 GPa , leads to a similar increase in the vertical displacements registered in the alignment of RP3 and RP5. However, in the alignment of RP1 (Figure 6.75 a)), the POT behaved in an unexpected way. Whilst reducing the longitudinal lining stiffness for 2.61 GPa did not show significant influence, reducing it for 1.8 GPa caused a decrease in the vertical displacements compared with the base model, which was unexpected.

It is possible to see that a similar situation occurred in the horizontal movement (Figure 6.76). In the alignment of RP1, the behaviour of the POT is almost coincident with the registered by the base model when 2.61 GPa is applied to the longitudinal stiffness. When the longitudinal stiffness is assumed as 1.8 GPa , the horizontal displacements also register an unexpected decrease.

This unexpected behaviour for both vertical and horizontal displacements recorded in the alignment of RP1 induces that the relation $E^{long} = E^{circ}/14.5$ forces the POT section to maintain its approximately circular section, whilst in an isotropic analysis this restriction did not occur. Figure 6.77 presents the deformed cross-section of the POT in the reference section when advance No. 16 is performed. The deformed cross-section is reproduced from the base model and from the model developed through the

anisotropic approach with $E^{long} = 1.8 \text{ GPa}$ (Figure 6.77 a) and b), respectively), both in the same scale. It is possible to see that the isotropic model led to a greater ovalisation within the POT than the one that occurred in the anisotropic approach. Thus, whilst the base model registered large vertical and horizontal displacements in the alignment of RP1 allowed by the ovalisation identified, the anisotropic model presented lower movements in these directions.

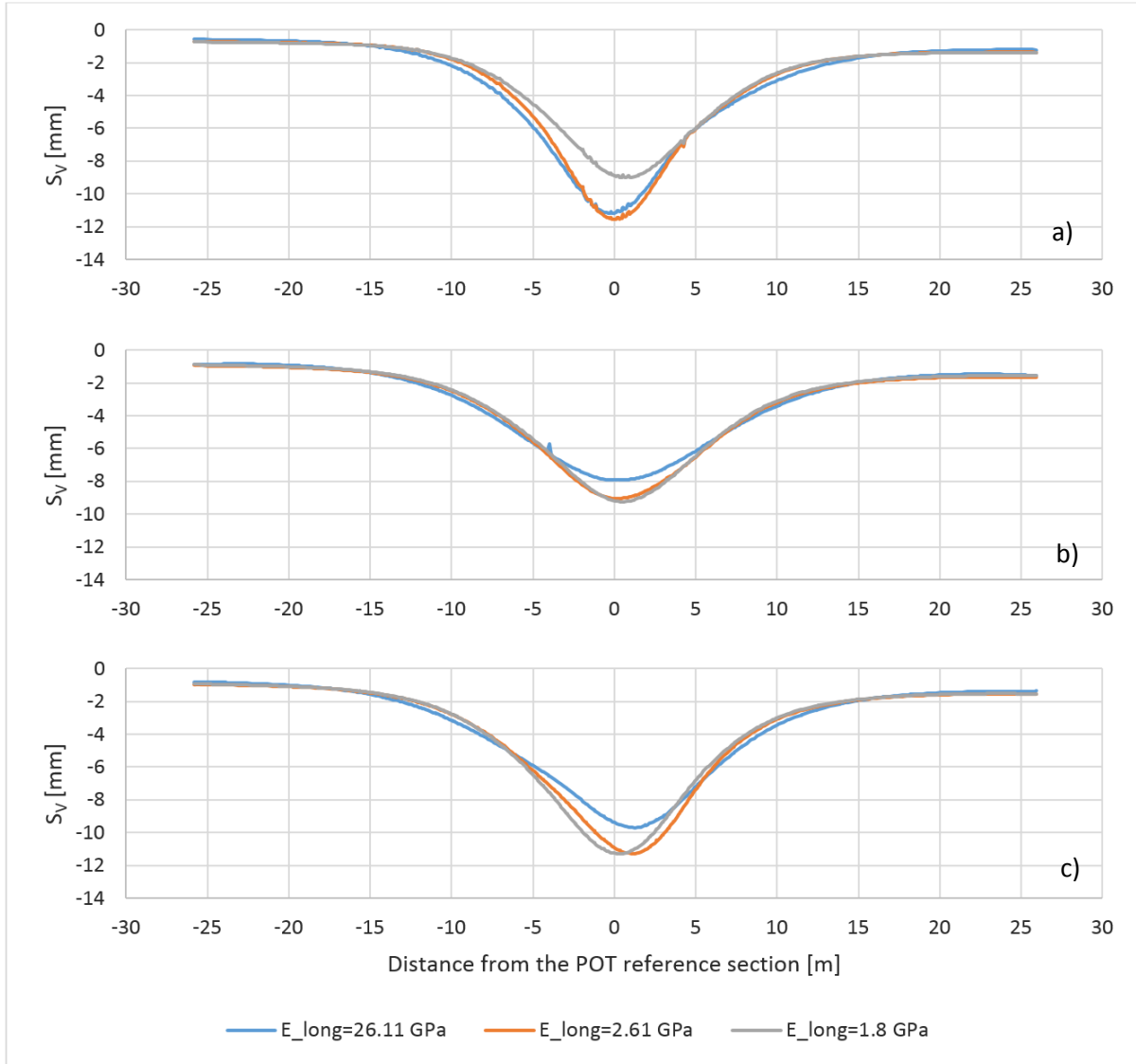


Figure 6.75 - Vertical displacements recorded for different values of the POT's longitudinal stiffness, E_{POT}^{long} within different prisms; a) RP1; b) RP3; c) RP5

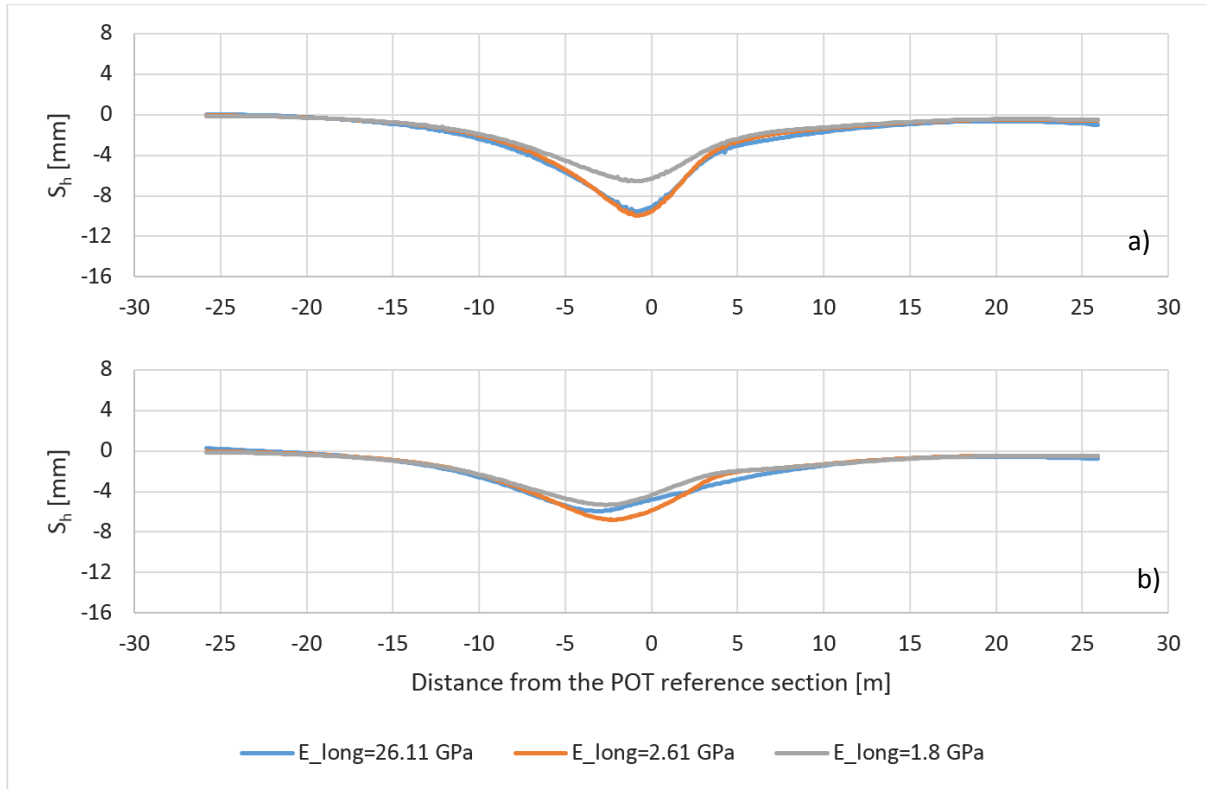


Figure 6.76 - Horizontal displacements recorded for different values of the POT's longitudinal stiffness, E_{POT}^{long} within different prisms; a) RP1; b) RP5

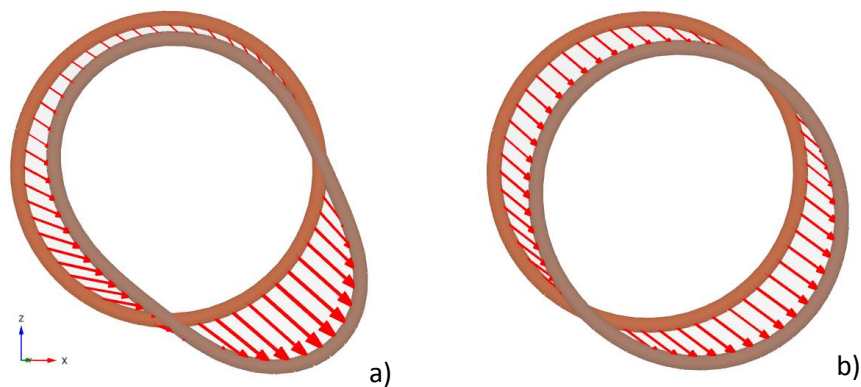


Figure 6.77 - Deformed POT cross-section on the reference section in advance No. 16; a) Base Model; b) Anisotropic approach, $E^{long} = 1.8 \text{ GPa}$

In what concerns to the longitudinal displacements, in order to determine the influence of an anisotropic analysis in the global behaviour of the POT, the results obtained in the alignment of RP1 are also presented in Figure 6.78. It can be seen that the longitudinal behaviour of the POT is quite consistent once the reduction of the longitudinal stiffness to 2.61 GPa and to 1.8 GPa led to similar behaviours, i.e., there was an increase in the longitudinal displacements in same magnitude for both approaches. It can be seen that, in fact, the longitudinal movement was the one that registered greater influence

regarding the modification in the longitudinal stiffness, leading to better results when compared with the monitoring data.

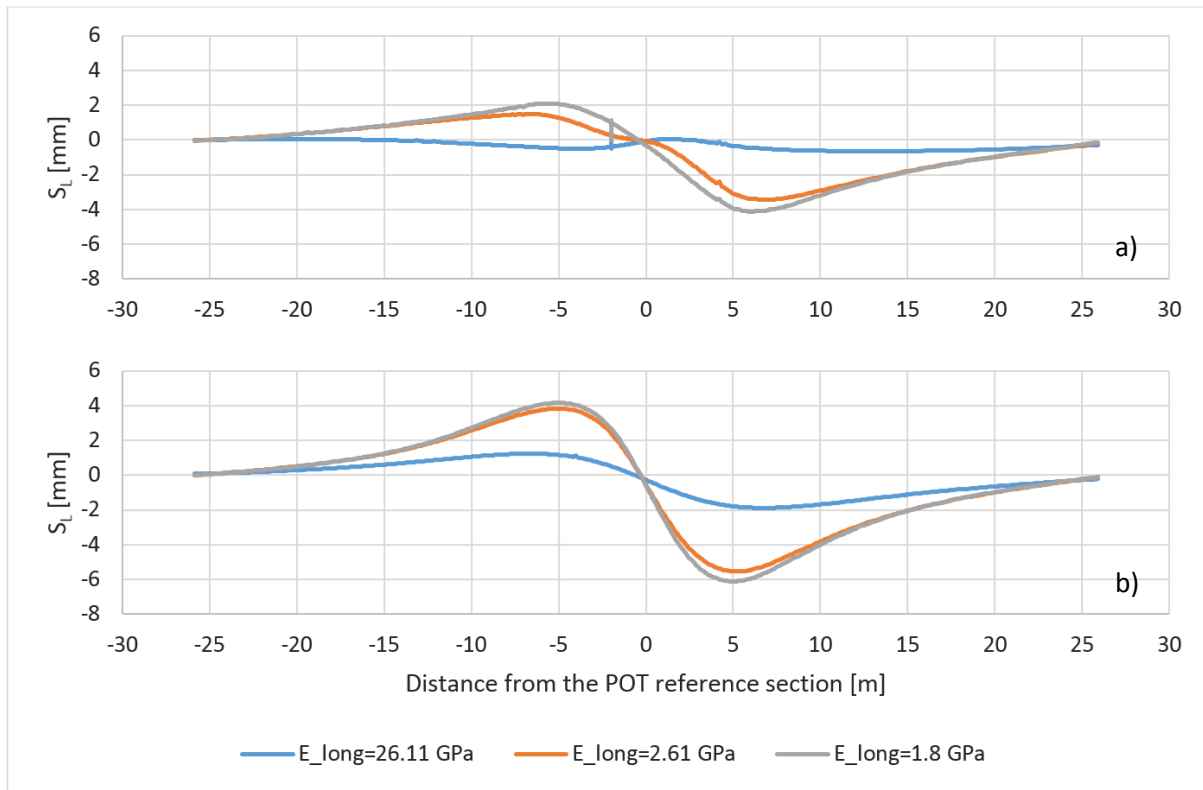


Figure 6.78 - Longitudinal displacements recorded for different values of the POT's longitudinal stiffness, E_{POT}^{long} within different prisms; a) RP1; b) RP3

Although the unexpected behaviour in the vertical and transverse directions, the anisotropic analysis led to slight improvements in the model results, inducing that this may be a better approach to model cast iron tunnels. However, this should be further analysed in order to understand the limitations of PLAXIS in what concerns to anisotropic behaviour.

7

CONCLUSION

7.1. CONCLUSIONS

The displacement prediction of an existing structure due to a new tunnel construction is a very complex subject, which makes it difficult to establish an analytical approach to reproduce them. A literature review, as presented in Chapter 2, revealed that, although there are strongly based formulations to predict both surface and subsurface ground movements, there is still very little information regarding displacements of underground existing structures due to tunnelling. As an example, Mair (2008) studied the effects of tunnelling on buried pipes and presented an analytical solution, however, this analytical approach is not suitable for cast iron tunnels, since the latter have a more complex behaviour than pipelines. Due to these limitations and all the specificities surrounding these analysis, the numerical methods are of paramount importance to predict the behaviour of existing underground structures due to tunnelling.

The geotechnical characterization of the London Clay and the overall geology of London, which was presented in Chapter 3, revealed to be of extreme importance to the author. Since the tunnels within the case study were mainly constructed within London Clay, this chapter allowed the author to be acquainted with the main characteristic of the London Clay and the referenced values of its properties. Although the author recognizes that the characterization of this soil was not as complete as it would be preferable, it highlighted the main properties that could influence the results of the numerical models.

Regarding the real movement of the cast iron tunnel of Post Office (POT), which was possible to characterize through the monitoring data provided, it could be seen that, as expected, the general movement of the POT is towards the excavation, both in vertical, horizontal and longitudinal directions. When the excavation face crosses underneath the POT, the movement of this tunnel starts to be recorded towards the opposite direction, which emphasizes the influence of the excavation face regarding displacements in the vicinity. The presence of several monitoring prisms along the POT section, was a key aspect to interpret the details of this movement, which is highly influenced by the relative movement of the lining joints. The results from the displacements registered in each monitoring prism showed that the cast iron tunnels have a real complex movement when exposed to new load conditions.

Although the displacements obtained from Xdisp do not reproduce the real situation, since a greenfield condition is assumed, the results determined for the crown of the tunnel were quite satisfactory when comparing the vertical displacements with monitoring data, which induces that the main parameters of this analysis, trough width parameter, $k = 0.5$ and volume loss, $V_L = 1.5\%$, represent a reasonable estimate for the prediction of displacements within London Clay. It was possible to see, however, that

the displacements measured in the tunnel invert were too far from the monitoring data, which is explained by the fact that the stiffness of the existing structure is not taken into account. In fact, one may conclude that an analysis of this nature should not be undertaken in greenfield conditions, since the stiffness of the existing structure has a considerable influence on the surrounding soil movement. Another limitation of this program that may explain the discrepancy in the longitudinal results, is the fact that it models the soil only based on two empirical parameters, which is too simplistic for such a complex material as soil. However, Xdisp is still a very practical and reliable tool to predict displacements of a new tunnel construction, although only in greenfield conditions.

It is indubitable that a tunnel excavation is a three-dimensional problem, however this problem is often modelled two dimensionally, once a three-dimensional analysis requires major computational resources, storage and time. Thereafter, a two dimensional analysis leads to a delicate simplification of the reality and should only be performed when the three dimensional effect can be accurately simulated, which in this case study was reproduced using the β -method. Regarding the present case, the interaction between the POT and the new tunnel could not be modelled in a plain strain condition, which led to an analysis also undertaken in greenfield conditions. Bearing this in mind, it is, in fact, possible to see that the Xdisp results and the results from the PLAXIS 2D model were quite similar. This similitude allowed the author to verify that the method adopted to simulate the three-dimensional effect - β -method - with a 50% relaxation factor (which is the commonly adopted value for the relaxation within London Clay) was a good approach for this problem since it led to a similar volume loss.

The sensitivity analysis performed with the 2D model, allowed to see that the soil parameters, such as the undrained shear strength and the in situ earth pressure coefficient, have a significant influence in the results of the numerical analysis. Thus, the correct characterization of these parameters is really important for the success of the analysis. As it was highlighted throughout this thesis and corroborated by the Geotechnical Interpretative Report (GIR), the in situ earth pressure coefficient is considered a very controversial parameter, especially in London Clay. It could be seen through the GIR, that different bounds are presented for this parameter whenever a tunnel is to be constructed in the vicinity of an existing structures, which emphasize the importance of studying its influence. In fact, the lower bound of this parameter led to better results in the PLAXIS 2D model regarding the horizontal displacements.

Regarding the three-dimensional analysis conducted in PLAXIS 3D, this project revealed that this software presents many obstacles in the modelling process of an interaction between two very close tunnels. Although PLAXIS has many internal features to model a tunnel construction, a complex geometry, such as the one studied here, still need to be imported and even simplified so that PLAXIS can generate an acceptable finite element mesh. It was possible to observe that, for more complex geometries it is important to have a good control on the specific properties of the mesh elements, which includes mainly the control on the polyline and surface angle tolerances and the refinement of the coarseness factor. In what concerns to the construction method, i.e., the simulation of the excavation sequence, the author realized that the most efficient way to model it, was to make some adjustments in the intersecting surfaces ensuring that the program creates accurate advances. These adjustments consist mainly on extending the surfaces to properly intersect different objects.

It could be seen that the results from the 3D base model were much lower than the results obtained in the 2D model. In the first instance, one may conclude that this discrepancy is due to the presence of the POT, whose stiffness provides some resistance to the soil movement. Besides, when comparing the results from a 2D model with the results from a 3D model, it is expected that the excavation process, as it was modelled in PLAXIS 3D, may reduce the displacements around the tunnel in construction, since the latter allows the arching effect to be progressively mobilized for smaller sections of tunnel, thus redistributing the stresses more efficiently. However, the monitoring data showed that in the reality the

displacements of the POT were greater than obtained in the PLAXIS 3D model (except for the horizontal displacements that were lower in the reality than the PLAXIS 3D returns). This may expose some limitations of the developed numerical model, which will be discussed in the following paragraphs.

The sensitive analysis undertaken within PLAXIS 3D showed that the K_0 parameter also improved the results of the PLAXIS 3D model when a lower bound was applied for this parameter. This result corroborates the idea made up with PLAXIS 2D that a lower bound for this parameter must be considered for tunnels constructed in the vicinity of existing structures.

One of the most dubious aspects of the 3D modelling discussed in Chapter 6 was the simulation of the POT lining. As it was presented, the POT lining is composed by cast iron segments that allow rotation to occur between joints. However, the model adopted to simulate the POT lining did not take into account this feature, which may be pointed as a limitation of the 3D model. Actually, the vertical displacements obtained through the numerical model were much lower than recorded in the monitoring data, inducing that the model adopted for the POT lining would not be the most appropriate. In addition, the difference between PLAXIS results and monitoring data, may have been enlarged by the fact that unlike the reality, there was a certain amount of soil between both tunnels. As reported, the POT was exposed during the construction of the underpass, which was not possible to model in PLAXIS, thus it might have prevented some displacements to occur.

Different approaches were adopted in an attempt to simulate the effect of the segment joints. First, still considering the initial isotropic linear elastic model, the effect of reducing the Young's modulus of the POT lining's material was studied; then, a Muir Wood approach was applied to the study, which reduces the stiffness of the lining due to the presence of the segment joints; and finally, the effect of an anisotropic model was investigated. The first attempt, revealed that reducing the Young's modulus of the lining, leads to a global increase in the POT displacements (i.e. an increase on the vertical, horizontal and longitudinal displacements in the same proportion) which did not adjust with the monitoring results, hence it was possible to conclude this is not a good approach to take into account the effect of the joints. It could also be seen that, although the Muir Wood approach provides a formulation to reduce the stiffness of the lining to take into account the joints, it did not represent any improvement on the model results. Thereafter, one can conclude that there is no advantage in applying a Muir Wood approach for the assessment of the effects of tunnelling on a cast-iron tunnel. In the end, the anisotropic analysis was the only one that led to interesting results.

As Schroeder *et al.* (2011) stated, the different extent of the caulking groove for the radial and circumferential joints leads inevitably to an anisotropic behaviour of the cast iron tunnels, however it is very difficult to estimate an average longitudinal stiffness to reproduce this anisotropy. Due to limitations of the software the relation $E^{long} = E^{circ}/14.5$ was the acceptable upper limit for the anisotropic analysis. However, the results revealed some numerical issues for this relation, which may be related with the complexity of geometry and stiffness of the various materials. Thus, to assume a longitudinal stiffness equal to ten per cent of the circumferential stiffness proved to be a best estimate to simulate the effect of the joints in the POT lining.

The complexity of the interaction between two tunnels was quite evident throughout this dissertation. One can conclude that, although numerical models are an excellent approach to analyse a problem of this nature, many improvements must be implemented in this type of analysis in order to get reliable results. This may involve new researches on the subject and probably the development of new numerical tools. The following section presents some suggestions for further researches on the subject.

7.2. FURTHER DEVELOPMENTS

One of the identified limitations of the developed PLAXIS models was the constitutive material model adopted for the soil. Soil is a very complex material that exhibits non-linear behaviour and shows differences in primary loading, unloading and reloading with stress dependent stiffness and undergoes plastic deformation. Besides, it also experiences small strain stiffness at very low strains and upon stress reversal. Vakili *et al.* (2014) stated that this general behaviour is not possible to be accounted for in a simple elastic perfectly plastic Mohr-Coulomb model, which directly affects the results obtained in PLAXIS. Thereafter, a study with an advanced constitutive material model, such as Hardening Soil model already implemented in PLAXIS or other, may represent an advantage for this case study. However, an advanced model requires a more detailed geotechnical and geological characterization, which leads to an increase in the cost of the analysis. Nevertheless, a cost-benefit analysis could be made and the advantages/disadvantages registered for future researches.

In order to account with the segment joints of the POT, it may be advantageous to develop a new model, in which each joint of the cast iron tunnel is modelled. This procedure would certainly be excessive time consuming and the memory necessary for that analysis unsustainable. Thereafter, one should considered the use of a different software, to allow a more efficient analysis of this situation. A new software would also enable the modelling of the POT closer to the real position which practically tangent to the SCL tunnel.

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APPENDIX A - VALIDATION OF THE β -METHOD

Structural element	Node	Wall node	Local number	σ_N [kN/m ²]		Relation between the stresses in the top heading excavation stage and in the initial state
				Initial Stress State	Top heading excavation Stage	
NegativeInterface_17_1	10111	8844	1	-535.72	-266.94	0.50
Element 1-4 (Interface)	10104	8833	2	-530.43	-265.22	0.50
(LC A2)	10103	8832	3	-528.18	-264.09	0.50
	10102	8831	4	-524.34	-262.17	0.50
	10101	8830	5	-522.52	-261.26	0.50
NegativeInterface_17_1	10101	8830	1	-522.52	-261.26	0.50
Element 1-3 (Interface)	9487	8793	2	-520.19	-260.09	0.50
(LC A2)	9486	8792	3	-519.89	-259.95	0.50
	9485	8791	4	-518.05	-259.03	0.50
	9481	8790	5	-518.31	-259.15	0.50
NegativeInterface_17_1	9481	8790	1	-518.31	-259.15	0.50
Element 1-2 (Interface)	9474	8773	2	-518.01	-259.01	0.50
(LC A2)	9473	8772	3	-519.80	-259.90	0.50
	9472	8771	4	-520.07	-260.03	0.50
	9471	8770	5	-522.36	-261.18	0.50
NegativeInterface_17_1	9471	8770	1	-522.36	-261.18	0.50
Element 1-1 (Interface)	9422	8175	2	-524.15	-262.08	0.50
(LC A2)	9421	8174	3	-528.02	-264.01	0.50
	9420	8173	4	-530.19	-265.09	0.50
	9419	8172	5	-535.29	-266.81	0.50
NegativeInterface_16_1	9419	8172	1	-538.80	-270.32	0.50
Element 2-6 (Interface)	8750	7981	2	-544.64	-272.32	0.50
(LC A2)	8749	7980	3	-561.39	-280.70	0.50
	8748	7979	4	-564.68	-282.34	0.50
	8744	7978	5	-579.47	-289.74	0.50
NegativeInterface_16_1	8744	7978	1	-579.47	-289.74	0.50
Element 2-5 (Interface)	7764	7759	2	-586.51	-293.25	0.50
(LC A2)	7765	7758	3	-606.84	-303.42	0.50
	7766	7757	4	-607.19	-303.60	0.50
	7760	7756	5	-619.57	-309.86	0.50
NegativeInterface_18_1	9571	8813	1	-622.83	-311.53	0.50
Element 3-8 (Interface)	9567	8812	2	-610.63	-305.32	0.50
(LC A2)	9566	8811	3	-611.28	-305.64	0.50
	9565	8810	4	-590.02	-295.01	0.50
	9587	8848	5	-583.30	-291.65	0.50
NegativeInterface_18_1	9587	8848	1	-583.30	-291.65	0.50
Element 3-7 (Interface)	9590	8847	2	-567.06	-283.53	0.50
(LC A2)	9589	8846	3	-564.48	-282.24	0.50
	9588	8845	4	-545.47	-272.73	0.50
	10111	8844	5	-539.44	-270.66	0.50
NegativeInterface_15_1	7760	7756	1	-619.25	-309.54	0.50
Element 4-9 (Interface)	6942	7033	2	-622.55	-311.27	0.50
(LC A2)	6943	7032	3	-630.51	-315.26	0.50
	6944	7031	4	-632.54	-316.27	0.50
	6945	7030	5	-637.95	-457.04	0.72
NegativeInterface_19_1	8855	8182	1	-637.39	-459.43	0.72
Element 5-10 (Interface)	8858	8185	2	-633.00	-316.50	0.50
(LC A2)	8857	8184	3	-630.80	-315.40	0.50
	8856	8183	4	-624.64	-312.32	0.50
	9571	8813	5	-622.26	-310.97	0.50