

1. Introduction – Timber as Temporary Works

Timber is commonly used in construction processes for temporary works, especially in underground construction. Other common materials include steel props. Timber is preferred over steel since it has a smaller density ($\sim 420 \text{ kg/m}^3$), which makes it easy to transport. Furthermore, timber structures give more flexibility because timber can be cut relatively easily on site than steel. Therefore, the design of the temporary structures can be readily modified on site to suit problems arose quickly. These properties make timber more superior as temporary works material in underground constructions where spaces are usually confined.

Despite the advantages gained using timber as underground temporary structures, there are also risks associated. The loads on the underground timber temporary works have not been understood thoroughly. It is very difficult to estimate accurately the load on underground timber structures, especially on structures with a mixture of steel and timber usage, such as the temporary work structure being investigated in this project.

The uncertainty of load estimation described above contributes to a very large factor of safety when designing underground timber temporary works. The timber structures end up being very large. This poses health and safety risks to workers, and also increases the cost and construction time. For example, the cross section of the timber structure being investigated in this report was $(225 \times 100) \text{ mm}^2$, which was relatively large as a temporary structure.

2. Aim

Given the aforementioned problems of inefficient design of timber temporary works, the major aim of this project is to investigate the loadings on these structures, which is expected to be smaller than predicted. Various prediction methods of the load are conducted. Better understanding of the load on the structure will then enable us to modify future designs. Smaller and more efficient timber sections can then be used. Powered tools, which are relatively inconvenient, are used to cut the timber blocks on

site. If smaller structures are possible, the construction method, such as the choice of tools, can be changed. The construction time and costs can be hugely reduced.

3. Overview of Project

This project was divided into two parts to better understand the load on underground timber structures. The first part is an experimental part involving an actual deployment of a wireless sensor network to an underground timber temporary structure in the construction site of Victoria Station, London. The initial deployment plan, as stated in the technical milestone report, was to install the sensor network at Tottenham Court Road Station. However, due to the tight working schedule of the contractor, the interference caused by the deployment became very undesirable and it was cancelled by the contractor. The final deployment was carried out at Victoria Station.

The other major part of the project was a finite element modelling of the structure using Plaxis, a finite element software capable of simulating soil behaviour and its interaction with structures. The model simulates the conditions at various construction stages. Results of these two parts are compared with theories applied for load estimation.

4. Risk Assessment

Risks associated with project were assessed before the commencement of any work. The preparation work of this project involved operating machines in lab, such as operating the instron machine in the fatigue lab for loading tests in CUED. Inductions about safety in lab and about the operations of the machines were conducted in the 1B course. The experiments in this project were carried out under the guidance of technicians. Projective equipment, such as safety goggles, was worn during the experiment.

The Experimental part of this project was conducted in a construction site. CSCS operatives course was taken before the start of the experiment to gain knowledge about safety measures, potential hazards and appropriate actions and behaviour on site. The contractor also gave a site induction and a tunnels/ confined space induction. Personal Protective equipment was worn during the whole course of site work. Lastly, briefings

about the plan of work, potential hazards and other issues on site were conducted every time before the beginning of site work.

The finite element modelling part of the project involved mainly operating computers in the geotechnical lab and CSIC lab, which had limited risk issues.

5. Theories and Literature Review

There are no available representative literature regarding loads and behaviours of timber temporary works, not to mention underground ones. The literature went through in the course of this project are (1) literature related to theories applied to design methods, (2) design consultancy reports on various underground temporary works, (3) guidelines and practice codes of London Underground for preparation of experiment, and (4) documents about theories of finite element modelling of geotechnical structures. Other articles went through describes behaviour of soils, which can aid the estimation of the loads and modelling on underground structures. The following theories are based on findings from them.

5.1 Soil Arching⁽⁶⁾

Soil arching is one of the most common phenomena in soils. This arises from the frictional nature of soil. Soil arching happens when a part of the support of a mass of soil yields while the remainder parts stays stationary in place. When this occurs, the soil joining the yielding part and the stationary part moves out of its initial position between the stationary parts of the soil sample⁽⁶⁾. The shear resistance of the soil at the interface between the yielding and stationary regions restrain this relative movement. This resistance tends to force the yielding part in its original position. It results in a reduction in the pressure acting at the yielding part of the support, with an increase at the stationary part at the interface of the two parts⁽¹⁰⁾. The migration of pressure is referred as soil arching, which happens on underground structures as they move.

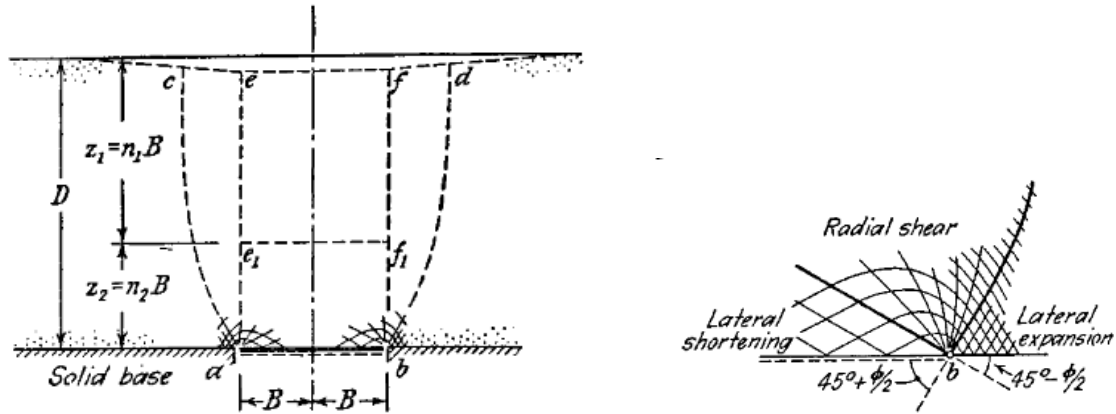


Figure 5.1: Geometry of soil arching (Right: zoomed view at the start of slipping surface)⁽⁶⁾

For example, arching occurs in ideal sand over a locally yielding horizontal support⁽⁶⁾, similar to the plank studied in this project. If the support is gradually lowered, local yielding of the horizontal support of a bed of sand happens. Before, the yielding happens or before arching occurs, the vertical load on the horizontal support is just the full overburden pressure. The full overburden pressure is therefore sometimes used for determining the load on underground structures. As the whole support moves downwards, the soil right above it follows its movement, which causes yielding. Frictional resistance at the interface of the moving and stationary sand is developed. The total pressure on the soil immediately above the yielding strip decreases, this amount of decrease is transferred to the frictional resistance at the boundary of the two regions. The total pressure acting on the adjoining stationary parts increases by the same amount. Local pressure at the yielding region is now only a fraction of the initial overburden pressure. The pressure is hugely transferred to the rigid part of the strip which is at its edges. A zone of discontinuity is formed starting at the edges of the strip. There is then a lateral expansion of the sand from the high-pressure zone to a low pressure one, hence at the edges of the strip towards upper region. This expansion is shown in Figure 5.1a and the local expansion at the edge in Figure 5.1b.

If the downward movement of the strip is in excess, the soil will fail along the boundary of the arch. The analytical formulation of the shape of this boundary has not yet been developed. The average slope angle was approximated through experiment in Table 5.1.

Geometry	Average slope angle
Small $D/2B$	~ 90
Large $D/2B$	$\sim 45 + 0.5\phi$

Table 5.1: Estimated values of slope angle of soil arching ⁽⁶⁾

To conclude, the vertical pressure acting on the lower part of the sand between the boundary, equals the weight of the upper part subtracted by the vertical component of the shear resistance along the boundary ⁽⁶⁾. This contributes to the arching effect.

5.2 Estimation of Load on Underground Timber Structures

There is not a standard method in normal practices for estimating the load on underground temporary works and to design them. The following methods are employed for the design of temporary works in Tottenham Court Road Station and Victoria Station.

5.2.1 Terzaghi's Equation⁽⁶⁾⁽⁹⁾

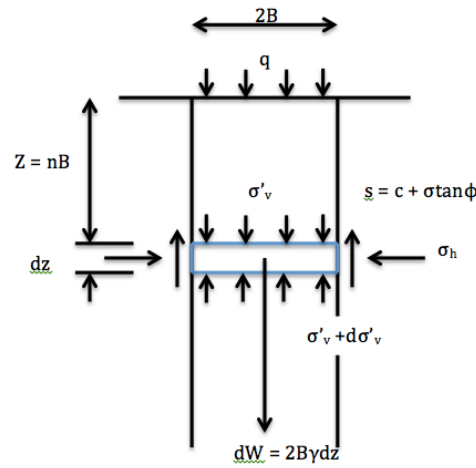


Figure 5.2: Vertical equilibrium of a soil element for illustration of arching and Terzaghi's equation (with vertical slip surface)

Terzaghi's equations are applied to detailed designs of underground structures. Based on the arching effect aforementioned, equations can be derived to estimate the loads on underground structures. According to Terzaghi, the slip boundary can be considered to be vertical ⁽⁶⁾ (Figure 5.2). There are errors associated with this assumption, but the result agrees well enough with experimental data. Considering the small element of soil, with constant K_0 and at a depth of z under surcharge q , between the two vertical slipping boundaries. By writing equations for vertical equilibrium and applying boundary conditions at the surface layer. The following equations for determining pressure on the strip are derived.

Vertical equilibrium of a small element of soil:

$$\frac{d\sigma_v}{dz} = \gamma - \frac{c}{B} - K\sigma_v \frac{\tan\phi}{B}$$

For $\sigma_v = q$ at $z = 0$, the vertical stress is given by:

$$\sigma_v = \frac{B(\gamma - c/B)}{K \tan \phi} [1 - \exp(-K \tan \phi \frac{z}{B})] + q \exp(-K \tan \phi \frac{z}{B})$$

This equation can be applying to various conditions of c and q to obtain:

$c > 0, q = 0$	$\sigma_v = \frac{B(\gamma - c/B)}{K \tan \phi} [1 - \exp(-K \tan \phi \frac{z}{B})]$
$c = 0, q > 0$	$\sigma_v = \frac{B\gamma}{K \tan \phi} [1 - \exp(-K \tan \phi \frac{z}{B})] + q \exp(-K \tan \phi \frac{z}{B})$
$c = 0, q = 0$	$\sigma_v = \frac{B\gamma}{K \tan \phi} [1 - \exp(-K \tan \phi \frac{z}{B})]$

Table 5.2: Terzaghi's equations for estimation of vertical load on underground structures ⁽⁶⁾⁽⁸⁾⁽¹¹⁾

5.2.2 Experience

Another historical method employed by underground temporary work designers for determining the load on structures is by taking it to be equal to total overburden pressure ⁽¹⁸⁾(assuming no arching). However, this was soon discovered to be excessively large. The estimation of load was modified according to previous designers' experiences.

From the design consultancy reports covered, the conventional practice is to take 15% to 30% of the total overburden pressure as the vertical loading acting on underground structures ⁽¹⁸⁾. 30% is often chosen to have a more conservative design ⁽¹⁹⁾. The $K = P_v/P_h$ value is often taken to be in the order of 0.5. This method has been used for over 50 years, without full understanding of the behaviour of the soil and the underground structure. This method was not modified at all during this period either. It is this project's aim is to better understand the load on underground structures and therefore to possibly identify ways to modify this practice.

Note: the design loads of structures considered are included in Session 7 after Session 6 about details on the structures

5.3 Other

Other documents went through included:

- 1) London Underground Standard 1-054 Civil Engineering – Earth Structures
- 2) London Underground Standard S-055 Civil Engineering – Deep Tube Tunnels and Shafts
- 3) London Underground Standard 1-085 Civil Engineering – Fire Safety Performance of Materials

6. Design of the Experiment

This session included essential information for the design of the experiment

Note: Details of some parts of experimental details are included in Session 9: Results and Discussion instead for clearer explanation.

6.1 Wireless Sensor Network

A wireless sensor network was used for the experiment. It was instrumented on underground timber structures to study their behaviour. It comprises of several components. The sensors used in this project are aluminium foil strain gauges and load cells. They are attached onto the timber structure under investigation. These are connected to a sensor node with an antenna, which is also attached in the timber. The data collected are transmitted from the sensors to the node, and in turn to a gateway through the antenna. The gateway is a computer located at the site at a distance from the structure being investigated. The gateway is connected to a 3G network, enabling us to access the data remotely in Cambridge through the network. The following Figure 6.1 shows the layout of the deployment of the sensor network in Victoria Station.

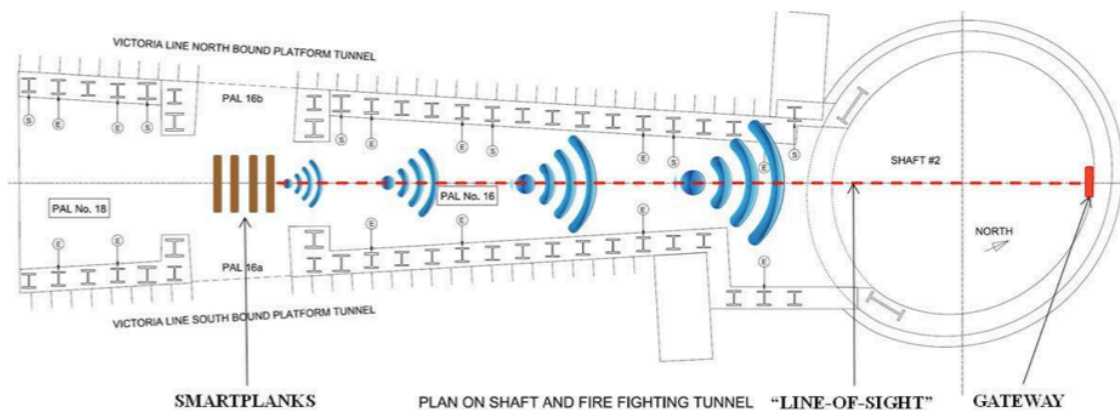


Figure 6.1: Overview of wireless sensor network for deployment

6.2 Timber Structures

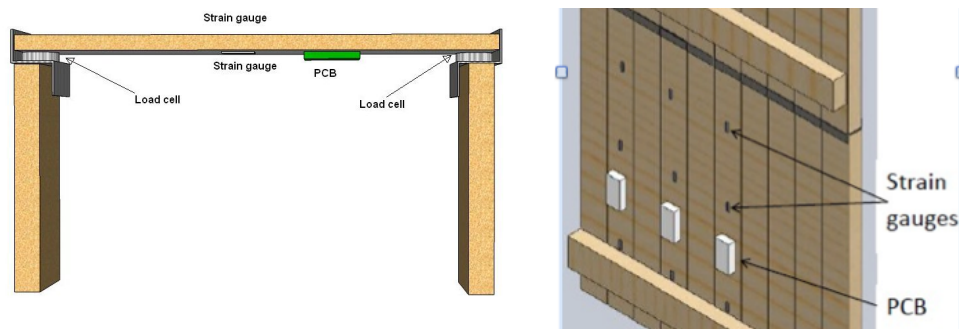


Figure 6.2: Timber temporary structure: (a) Left: idealised type, (b) Right: vertical retaining structure (Stair 14 – Tottenham Court Road Station)

Various types of timber structures were considered in this project. The ideal type of structure (Figure 6.2a) consists of two vertical side trees, and a horizontal head tree supported by the two side trees. This type of structure holds the soil along the tunnel during excavation. The top trees take the bending load from the top while the side trees take the vertical load axially and the lateral bending load. A yankee brob is used as a connection. With this layout, the strain gauges are attached on both the top and under surfaces of the timber plank. The node is attached at available space on the underside for a better strength of signal transmission. The load cell will be located at the top of the side trees at the Yankee brob. The top tree is countersunk by having the side trees shortened to accommodate the load cells. This enables a constant height of all the timber frames.

Another structure considered was a vertical retaining structure at Tottenham Court Road Station, labelled Stair 14 (Figure 6.2b). Similarly, the strain gauges and node are attached on the timber surface. Rows of timber planks are simply supported by two horizontal steel beams at the two ends of the plank. Since it is difficult to locate the load cell to measure horizontal forces, it is not used in this structure.

The last type of structure consists of arrays of head-tree timber planks resting on steel beams. The planks are simply supported. A structure of this type was being studied in detail in this project. It was used as temporary works in Victoria Station for tunnelling activities. Sensors are deployed to this structure. Furthermore, a finite element model was generated to study its behaviour. Details are shown in Session 6.3.2.

6.3 Site Details

Note: Information obtained from site shift reports, design drawings, site visits and meetings. As requested by the author, some documents are intended not to be disclosed and are therefore not included as reference.

6.3.1 Victoria Station Pal 16/18

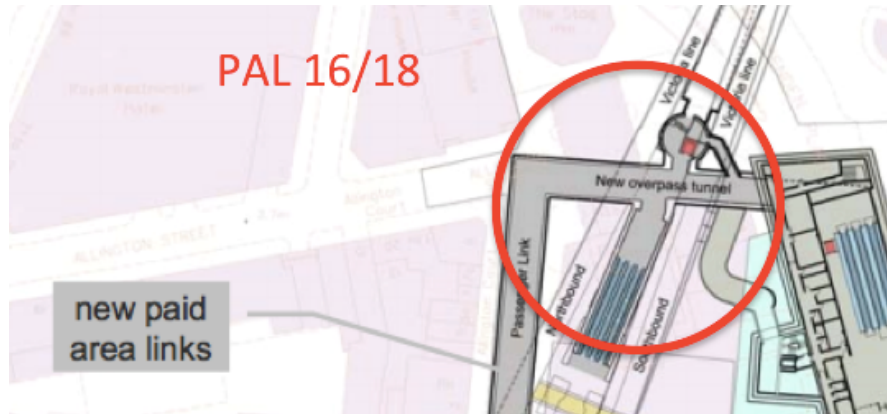


Figure 6.3: Victoria Station Upgrade: Location of Pal16/18 - new access of Victoria Station

The deployment of the wireless sensor network to collect experimental data was carefully designed to suit the condition and the timber temporary structure. As mentioned, the deployment was done at the Victoria Station, London. Site meetings were held with the client and contractor to discuss about issues regarding the deployment.

First of all, it is necessary to know about details of the site. Victoria Station has been experiencing a huge and increasing passenger flow within the station's underground passages and also around the station on the ground level. There are congestions on the platforms of the Victoria Line. The Victoria Station Upgrade project was started in 2011 to improve the current situation. The project involves constructions of a new ticket hall, new passage access passage to the platforms and extension of the existing ticket hall.

Role	Company
Client	London Underground
Contractor	VINCI/BAM Nuttall Joint Venture
Consultant (Permanent works designer)	Mott Macdonald
Consultant (Temporary works designer)	Alan Auld

Table 6.1: Details of Victoria Station Upgrade ⁽²⁰⁾

The deployment was conducted at Pal 16/18, a new step-free lift access to the existing platforms. The construction included a lift shaft (Shaft 2) with a 20m depth down to stratum where the existing Victoria Line tunnels located. The shaft was constructed between the existing tunnels. The linings at the bottom of the shaft were then broke open. A tunnel (Pal 16), which was used as future passenger access was then excavated

in the direction going out radially from the centre of the shaft, and between and parallel to the axes of the existing tunnels as (Figure 6.4). In other words, Pal 16 was constructed in between the existing tunnels. The length of the Pal 16 tunnel passage was about 20m. At the end of this tunnel, a cross passage (Pal 16a & 16b) was constructed to create an access to the existing Victoria Line platforms. Excavation was done laterally in direction perpendicular to the axis of the existing platforms. The excavation would be stopped when it reaches the linings of the platforms.

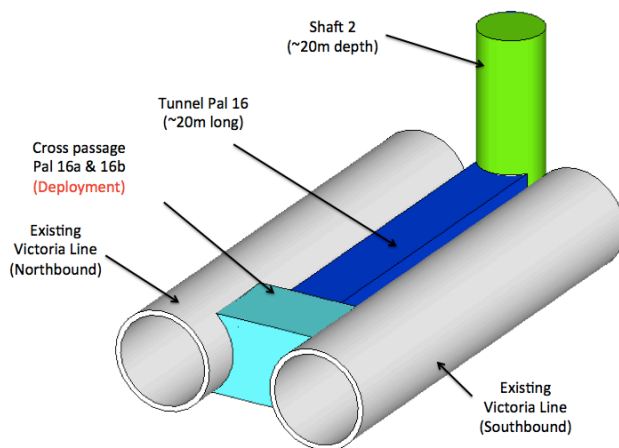


Figure 6.4: Construction works of Pal 16/18

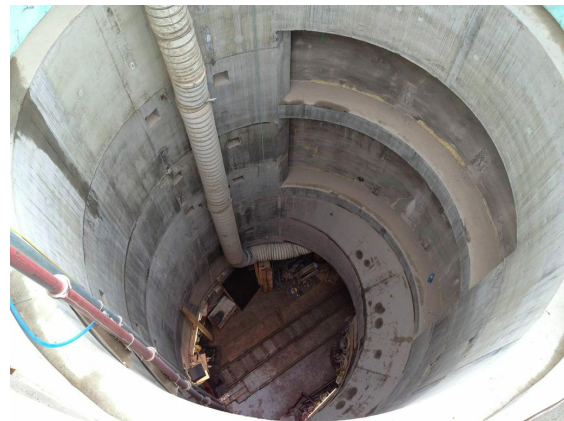


Figure 6.5: Shaft in the construction site (~20m depth)

6.3.2 Details of Sample Timber Structure ⁽¹⁹⁾

A timber temporary structure is used to retain the soil during the excavation of the tunnel (Pal 16) and the cross passage (Pal 16a & 16b). The grade of timber used in the structure was C24 ($E = 9 \text{ GPa}$). This structure was located at a 13.8m depth. Horizontal steel I-beams were employed as permanent works and temporary props connecting and supporting the tunnel linings. This restrained the tunnels from moving towards each other due to the excavation in between. Figure 6.6 shows the completed form of the temporary works for retaining soil at the top of the excavation.

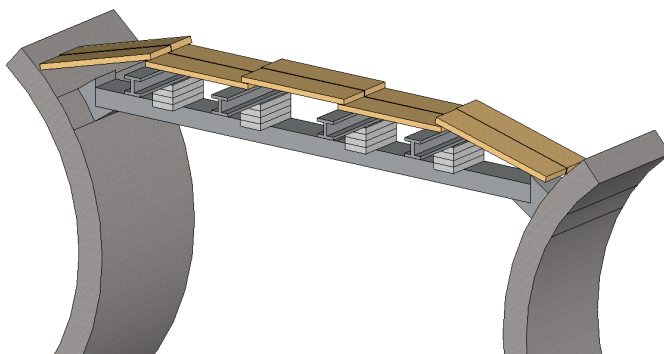


Figure 6.6: Layout of temporary work (Pal 16/18)

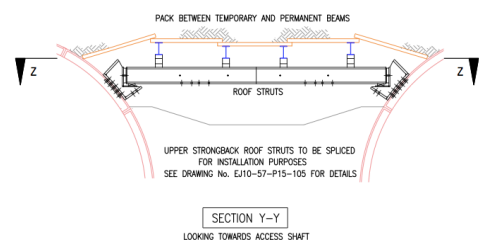


Figure 6.7: Section drawing of temporary work

The planks were labelled 1 to 5 in the order from the left to right. Sensors were only installed on the timber plank at the central span ("The plank" or plank 3). This timber plank was simply supported on the edges of the two planks on its ends (plank 2 & 4). The outermost planks (plank 1 & 5) are simply supported on the tunnel linings, and on plank 2 and 4. Plank 2 and 4 are simply supported on the needle beams running along the axis parallel to the existing tunnels' axes. The needle beams rest on concrete blocks, which in turn rest on a huge horizontal I-beam. This I-beam was a member of the permanent steel works connecting the existing tunnel linings. The load was transferred through the following sequence: Soil - plank - needle beam - concrete blocks - permanent steel roof beam - tunnel linings. The location of deployment was chosen since the timber temporary works would stay in its location for a relatively long period of time giving more a longer available data-logging period, and the location was of the contractor's preference according to the ease of the construction.



Figure 6.8: Completed temporary work (Pal 16/18 – Victoria Station); Instrumented planks circled

Several arrays of timber planks were supported on the same row of needle beams, and then on the same permanent steel beam. At the late stage of the excavation before breaking of the tunnel linings, regions at the same level or above the horizontal permanent beam would be sprayed by concrete. All temporary works above were buried in concrete. The permanent beams and the concrete would then carry the load. Below the permanent steel roof beam, some horizontal steel props were used to restrain lateral movements of the tunnels. These were removed after the spraying of concrete.

6.3.3 Construction Sequence

As the load on the timber was highly dependent on the construction sequences of the temporary structure itself and structures around it. The construction sequence of it was studied in details, especially regarding the excavation sequences.

The construction of the tunnel Pal 16 was completed before the start of excavation of the cross passage Pal 16a & 16b, meaning that the sides and the roof of Pal 16 were completely sprayed by concrete. Before the excavation of the cross passage, the end of Pal 16 was retained by 3 rows of vertically placed timber planks supported by horizontal steel beams.

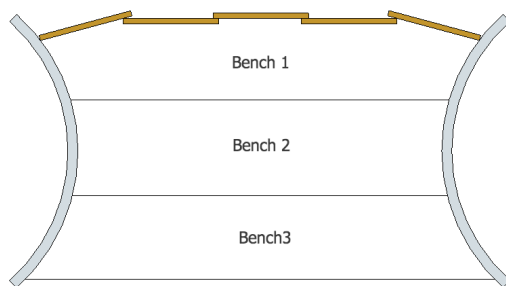


Figure 6.9: Benches of soil of excavation

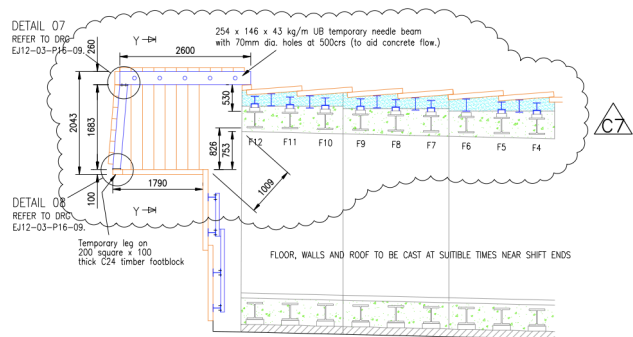
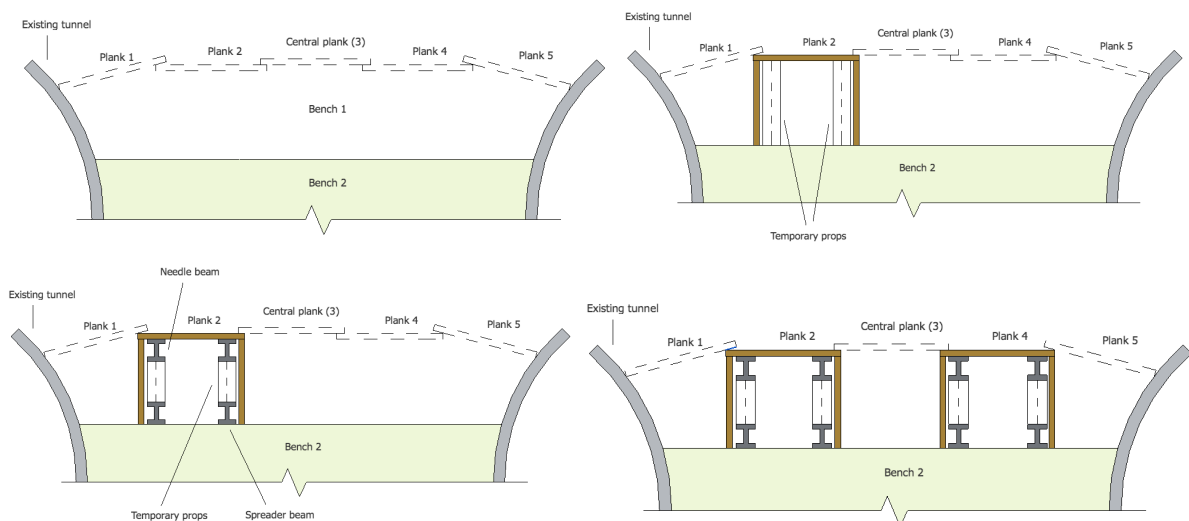


Figure 6.10: Section of side view of excavation of top bench

The soil at the cross passage was divided into three benches (Figure 6.9). The planned locations of the planks are shown in the Figure 6.9. The excavation was done firstly on the top bench. The top row of retaining timber works was removed for the advancement of the excavation (Figure 6.10). Figure 6.11 below shows the excavation sequences.



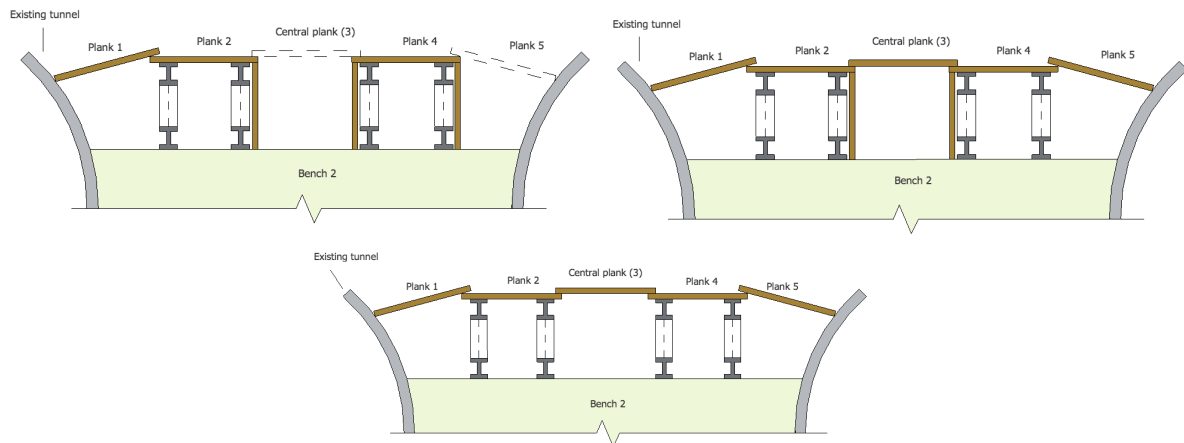


Figure 6.11: Construction sequence of the temporary work and excavation (labelled 1-7, left to right, downwards)

The excavation was started at the top bench. This bench was divided into five regions beneath the locations of the 5 planks. It was first started in the area underneath the intended location of plank 2. The excavation was advanced to a distance equalling the width of a timber plank (0.1m). Then a timber frame, having two side trees and a top tree, would be placed. Props are used to support the top tree in addition to the two side trees (Figure 6.11-2). The excavation would then advance to another 0.1m in the same area. Another set of temporary work would be placed to support the next timber frame. This process was repeated until the advancement of the excavation reached around 1m (10 frames). The height of the props would then be adjusted. Needle beams would be placed below the two ends of the top trees along the ten frames. In addition, spreader beams were placed right below the needle beams at the bottom of this bench along the direction of the excavation. The props stood on the spreader beam and it supported the needle beam above (Figure 6.11-3).

This process was repeated to other areas in the top bench until the whole bench was excavated. The excavation sequence was Area: 2-4-1-5-3. When excavating area 1, the vertical timber plank between the interface of area 1 and 2 was removed, since there was no longer a need to retain the soil laterally (Figure 6.11-5). Plank 1 was supported by the existing tunnel linings on one end, the props and the needle beam supported both plank 1 and 2. This was also the same for area 5 (Figure 6.11-6). When excavating the last central area (area 3), all the vertical timber planks are removed. All the top trees planks were supported by the needle beam and the props (Figure 6.11-7). After the

construction of the horizontal permanent roof beam, the props were removed and replaced by concrete blocks which rested on the roof beam (Figure 6.8).

6.4 Measurement of Load⁽²²⁾

For the experiment, the load on the plank is assumed to be uniformly distributed. For the purpose of timber as temporary structure, the timber is assumed to be a uniform material. This is shown in design reports reviewed ⁽¹⁸⁾⁽¹⁹⁾. Local defects, environmental conditions and aging issues are ignored ⁽²⁾. In conservative designs, all these issues are dealt with by applying factors of safety ⁽²⁾.

The timber plank in the experiment is therefore modelled as a simply supported uniform beam. The beam behaviour follows simple beam theory. The measurement of strain gauges in the experiment can be converted to vertical pressure acting on the timber plank with the following steps ⁽²²⁾.

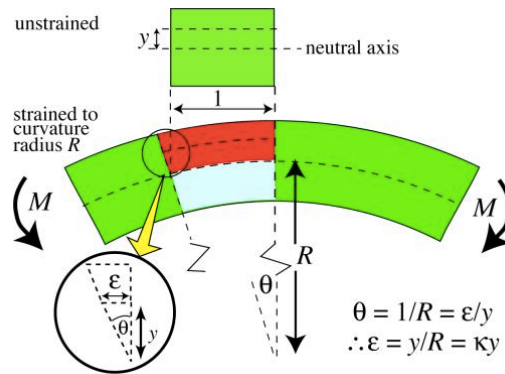


Figure 6.12: Simple beam theory relating strain and curvature ⁽²²⁾

Local strains are converted to the overall pressure on the plank by (assuming uniformly distributed load:

$$\kappa = \frac{\epsilon}{y} = \frac{\epsilon}{d/2} \quad M = EI\kappa \quad M = \frac{\omega x^2}{2} - \frac{\omega lx}{2}$$

The pressure ($P = \omega/b$) acting on the plank is obtained.

7. Design Load of Structures

7.1 General Information

After knowing the details of the temporary work (e.g. geometry and location), the design load can be estimated. This session summarises the result from temporary works design reports of the underground timber structures studied. Designs of timber structures take

into account the factored life and dead loads. These are compared with the basic working stresses of the type of timber chosen ⁽¹⁾. The working stress of the timber has to be greater than the load. Factors of safety are commonly applied to account for non-uniformity, moisture content, grains' orientation, duration of load, modes of load ⁽²⁾ (e.g. cyclic). However, in the design of underground temporary timber structures, these factors are not always taken into account. Un-factored working stresses are compared with the factored load ⁽¹⁹⁾. Lastly, issues associated with aging of timber are not considered since the working lives of temporary structures are short ⁽¹⁾⁽²⁾.

7.2 Tottenham Court Road – Stair 14 ⁽¹⁸⁾

Stair 14 (Figure 6.2b) at Tottenham Court Road Station was a vertical structure located at 20m depth. The parameters of the soil, which was London Clay, at this depth were substituted into the Terzaghi's equation (Session 5.2.1) and the conventional load prediction method (Session 5.2.2) to obtain values of the design load. Short-term parameters (undrained) and long-term parameters (drained) were applied. The calculated results are studied and the final design load was chosen.

	Terzaghi's equations		Experience		Final design
kN/m ²	Short-term	Long-term	15% overburden	30% overburden	
P _v	69.3	118.2	57	114	60
P _h	22.9	39.4	28.5	57	30

Table 7.1: Load estimation and design load for Tottenham Court Road Station – Stair 14

7.3 Victoria Station – Pal 16/18 ⁽¹⁹⁾

This structure was located at a 13.8m depth (more details in Session 6.3.2). The designer assumed the vertical load on the structure to be 30% of the sum of the full overburden pressure and the surcharge applied on the ground surface.

This gave a design load of:

$$P_v = 0.3(\gamma z + q) = 0.3[(19)(13.8) + 10] = 81.7kPa$$

This value was compared with the experimental and finite element simulation results.

8. Details of Finite Element Modelling - Plaxis

Note: Details of various parts of theories regarding formulations of Plaxis are included in Session 10: Results and Discussion instead for clearer explanation.

8.1 General Information

The finite element model for analysing the timber temporary work considered in this project was generated using Plaxis. Plaxis is a finite element software which simulates the behaviour of soils and its interaction with structures.

A 2D plain strain model was employed. 6-node triangular elements were generated to model the soil behaviour around the timber temporary work. A 6-node element has 3 stress points. The shape function on each side of the element was in the order of 2 and was therefore able to conduct second order interpolation, which is sufficient for our simulation since there was nothing varying in any higher orders.

Effective stress analysis and equivalent effective stress analysis were carried out instead of a total stress analysis as planned in the milestone report. This was because after careful consideration, the plan of the modelling was changed to include the tunnelling phases and the consolidation phases starting from the construction of the Victoria Line tunnel 50 years ago. Total stress approach is only suitable for undrained problems and is thus not applicable for consolidation analysis.

8.2 Mohr-Coulomb Model ⁽⁷⁾⁽¹²⁾⁽¹⁵⁾

The Mohr-Coulomb model (perfect plasticity) was used to simulate the behaviour of soils. Stress and strain tensors in 2D problem are expressed as:

$$\sigma = \begin{pmatrix} \sigma_x & \sigma_{xy} \\ \sigma_{yx} & \sigma_y \end{pmatrix} \quad \varepsilon = \begin{pmatrix} \varepsilon_x & \varepsilon_{xy} \\ \varepsilon_{yx} & \varepsilon_y \end{pmatrix}$$

Terzaghi's principle is applied.

$$\sigma = \sigma' + \sigma_w$$

Parameters ⁽¹²⁾ of soil data to be input for a Mohr-Coulomb model are:

- 1) Young's modulus, E
- 2) Poisson's ratio, ν
- 3) Internal friction angle, ϕ
- 4) Cohesion, c
- 5) Dilatancy angle, ψ
- 6) The rate of change of Young's modulus along depth, $E_{\text{increment}}$
- 7) The rate of change of cohesion, $c_{\text{increment}}$
- 8) The soil in the model was assumed to have no tensile strength. Tension cut-off was set at zero

The Mohr-Coulomb model assumes elastic perfectly plastic behaviour of soil. The yielding envelope is determined by the friction angle as shown in the Figure 8.1 below. Both the effective stress analysis (drained) and equivalent effective stress analysis (undrained) were conducted using this model with different soil parameters.

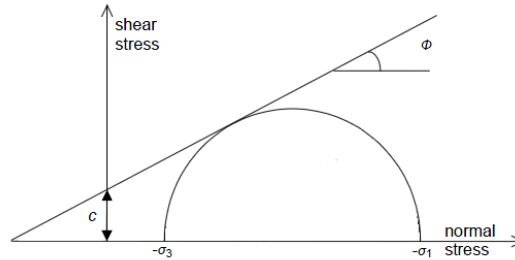


Figure 8.1: Mohr-Coulomb model of Plaxis ⁽¹²⁾

8.3 Element Formulations ⁽¹³⁾

In finite element analyses, interpolation functions (also called shape functions) are used to model data values within an element based on known values or parameters at the nodes. Each node has a number of degree-of-freedom that maps the discrete value of unknowns modelled. For example, degrees-of-freedom are displacements and pore pressure values in consolidation problems. Taking displacement as an example, it is mapped by the following equation:

$$u(\xi) = \sum_{i=1}^n N_i(\xi) v_i$$

where:

- v_i = nodal values,
- $N_i(\xi)$ = value of the shape function of node i at local position ξ
- $u(\xi)$ = the modelled resulting value at position ξ , and
- n = the number of nodes in an element.

For the 6-node triangular elements used in the model of this project, each side of the triangle has 3 nodes, forming a 3-node line element as below.

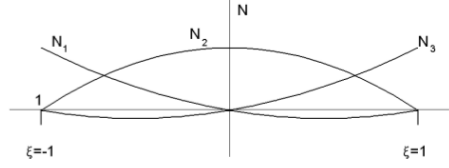


Figure 8.2: 3-node line element and shape functions ⁽¹³⁾⁽¹⁷⁾

As the basic principle of finite element analysis, a shape function of a node has a value of 1 at its node and equals to 0 at other nodes. For the 3-node line element, nodes 1, 2 and 3 are located at $\xi = -1, 0$ and 1 respectively. For the 2D 6-node element, two local coordinates (ξ, η) are provided. The following gives the corresponding shape functions:

$$\begin{aligned} N_1 &= \zeta(2\zeta - 1) & N_2 &= \xi(2\xi - 1) & N_3 &= \eta(2\eta - 1) \\ N_4 &= 4\zeta\xi & N_5 &= 4\xi\eta & N_6 &= 4\eta\zeta \end{aligned}$$

where :

$\zeta = 1 - \xi - \eta$ is an auxiliary coordinate used by Plaxis.

8.4 Numerical Integration ⁽¹³⁾⁽¹⁷⁾

Integrations are often carried out for the elements during a simulation, integrals are approximated using numerical methods. For a line element, the integral is approximated by taking k sample points in the element.

$$\int_{\xi=-1}^1 F(\xi) d\xi \approx \sum_{i=1}^k F(\xi_i) w_i$$

where:

$F(\xi_i)$ = the value of the function F at position ξ

w_i = weight factor at point i (relative significance of point)

Two methods are used by Plaxis, the Newton-Cotes Integration and Gauss Integration. The modelled values are calculated for each element within the model, after that, the elements are then mapped to the global matrix.

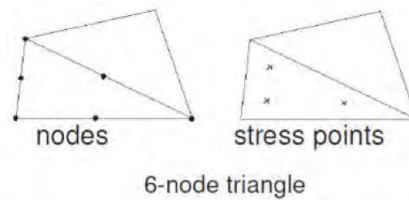


Figure 8.3: 6-node triangular element and stress points for simulation calculations ⁽¹⁵⁾

For a triangular element, integrals are approximated as:

$$\iint F(\xi, \eta) d\xi d\eta \approx \sum_{i=1}^k F(\xi_i, \eta_i) w_i$$

Gauss integration is used for triangular elements, a 6-node element has 3 approximation points. Gauss method has specified number and locations of approximation points between nodes. Gauss integration of k-points can carry out integrations of polynomial with an order of $2k-1$. Parameters are specified as follows.

Point	ξ_i	η_i	ζ_i	w_i
1, 2 and 3	1/6	1/6	2/3	1/3

Table 8.1: Gauss integration points for a 6-node element

8.5 Differentiation⁽¹³⁾

Differentiations are also often needed. For example, it is needed to obtain strain values from displacements. Strain is given by:

$$\underline{\varepsilon} = B_i \underline{v}$$

where B_i is the strain interpolation matrix equalling partial derivatives of the shape function (the third element of the vectors correspond to shear stress and strain)

In a 2D plain strain ($\varepsilon_z = \gamma_{yz} = \gamma_{xz} = 0$), the strains in the global Cartesian coordinates are obtained by summing up the contributions from all nodes as follows.

$$B_i = \begin{pmatrix} \frac{\partial N_i}{\partial x} & 0 \\ 0 & \frac{\partial N_i}{\partial y} \\ \frac{\partial N_i}{\partial y} & \frac{\partial N_i}{\partial x} \end{pmatrix} \quad \text{and} \quad \begin{pmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{pmatrix} = \sum_i B_i \begin{pmatrix} v_{x,i} \\ v_{y,i} \end{pmatrix}$$

A Jacobian maps the local coordinates derivatives to that in the global Cartesian coordinates.

$$\begin{pmatrix} \frac{\partial N_i}{\partial \xi} \\ \frac{\partial N_i}{\partial \eta} \end{pmatrix} = \begin{pmatrix} \frac{\partial x}{\partial \xi} & \frac{\partial y}{\partial \xi} \\ \frac{\partial x}{\partial \eta} & \frac{\partial y}{\partial \eta} \end{pmatrix} \begin{pmatrix} \frac{\partial N_i}{\partial x} \\ \frac{\partial N_i}{\partial y} \end{pmatrix} = J \begin{pmatrix} \frac{\partial N_i}{\partial x} \\ \frac{\partial N_i}{\partial y} \end{pmatrix} \quad \rightarrow \quad \begin{pmatrix} \frac{\partial N_i}{\partial x} \\ \frac{\partial N_i}{\partial y} \end{pmatrix} = J^{-1} \begin{pmatrix} \frac{\partial N_i}{\partial \xi} \\ \frac{\partial N_i}{\partial \eta} \end{pmatrix}$$

8.6 Element Stiffness Matrix

The element stiffness matrix is given by the following equation and approximation. The element stiffness matrix is made up of sub-matrices K_{ij}^e , with i and j as local nodes. dV indicates an integration over the volume. This is the volume per unit depth in the 2D model, which equals the area dA .

$$K^e = \int B^T D^e B dV \quad \text{and} \quad K_{ij}^e = \sum_k B_i^T D^e B_j w_k$$

8.7 Consolidation ⁽⁷⁾⁽¹³⁾

Consolidation behaviour of the soil is also modelled in Plaxis. This function was used to simulate the behaviour of soil between the time when the tunnel was constructed and the present time of the upgrade project. This session explains the finite element discretisation for the consolidation theory. Terzaghi's effective stress theory and Biot's theory of consolidation are used.

Stresses in soil is given by:

$$\begin{pmatrix} \sigma_x \\ \sigma_y \\ \sigma_{xy} \end{pmatrix} = \begin{pmatrix} \sigma'_x \\ \sigma'_y \\ \sigma'_{xy} \end{pmatrix} + \underline{m}(p + p_{excess}) \quad \text{and} \quad \underline{m} = \begin{pmatrix} 1 \\ 1 \\ 0 \end{pmatrix}$$

P, the steady state pore pressure values is calculated by the program according to the input of the location of the water table. Same as ordinary materials, a stiffness matrix of the soil related the strain increment and the effective stress increment. Again, finite element approximations are applied to simulate the actual stress and pressure values.

The expressions used are ⁽¹²⁾:

$$\underline{u} = N \underline{v} \quad \underline{p} = N \underline{p}_n \quad \underline{\varepsilon} = B \underline{v}$$

where,

$\underline{v}$ = the nodal displacement vector

$\underline{u}$ = continuous displacement vector within an element

$\underline{p}_n$ = excess pore pressure vector

$\underline{p}$ = the excess pore pressure

N = the shape function

B = the strain interpolation function (partial derivatives of the shape function)

A small element of soil is considered, and by applying the above equations to the equilibrium of the element, the following approximation can be obtained.

$$\int B^T d\underline{\sigma} dV = \int N^T d\underline{f} dV + \int N^T d\underline{t} ds + \underline{r}_0$$

$$\underline{r}_0 = \int N^T \underline{f}_0 dV + \int N^T \underline{t}_0 dV - \int B^T \underline{\sigma}_0 dV$$

where,

$\underline{f}$ = force due to self-weight

$\underline{t}$ = traction forces on surface

$\underline{r}_0$ = residual force vector (which is equal to zero, error in previous iteration step will results in a small finite value, the error is self-correcting after summing up the contributions from all elements)

Rearranging the equilibrium equation by substituting the discretion formulas,

$$K d\underline{v} + L d\underline{p}_n = d\underline{f}_n$$

In the above governing equation, K is the stiffness matrix, L is the coupling matrix, and $d\bar{f}_n$ is the incremental load vector. These equations are applied when modelling consolidation problems. These matrices are given in the following Table 8.2.

Stiffness Matrix	Coupling Matrix	Incremental Load
$K = \int B^T M B dV$	$L = \int B^T \underline{m} N dV$	$d\bar{f}_n = \int N^T d\bar{f} dV + \int N^T d\bar{t} ds$

Table 8.2: Matrices used in finite element model (consolidation)

It is noted that the Poisson's ratio value applied by Plaxis is 0.495 instead of 0.5 in reality to avoid singularity problem of matrices ⁽¹⁴⁾.

9. Results and Discussion – Deployment

This session describes the development process, testing, and the final implement of the wireless sensor network. The analysis of the data collected with the system is discussed.

9.1 Preparation Work

9.1.1 Mechanical Joint Design

Before the actual deployment of the wireless sensor network to the site, various preparation works was done. The first procedure was the design of mechanical attachment methods of the strains gauges and load cells.

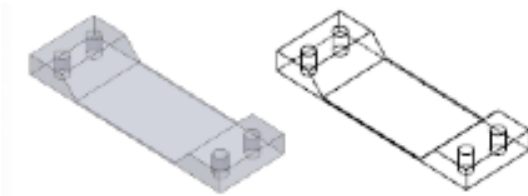


Figure 9.1: Design of aluminium alloy plate for strain gauges installation

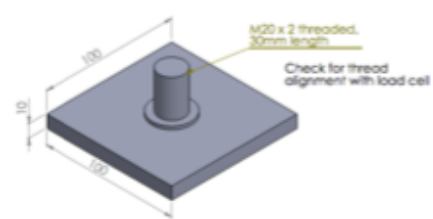


Figure 9.2: Design of aluminium steel plate for load cell installation

The aluminium foil strain gauges are usually attached using epoxy adhesives, which is inflammable. It was of a very strong preference from the contractor that epoxy was not used in our deployment on site. Using inflammable material on site required COSHH assessment, which was not desirable. An aluminium plate (Figure 9.4) was designed for the attachment of the strain gauges on the timber plank. Aluminium alloy 6028 was chosen since it was readily available, relatively not stiff, high yield stress and easy to machine. The strain gauges were glued onto the middle thin part of the plate on site. A thin region prevented the plate from causing local reinforcement. The plates with the

gauges were screwed to the surface of the timber plank. The sides of the plate were designed to be thick to ensure a good load transfer from the plank to the gauge. It is to ensure the gauge recorded the same change in strain on the extreme fibre of the timber.

Attachment method was also designed for the load cell. The resistance of circuit in the load cell changed as it was compressed. The load that caused the compression was deduced from the resistance. A steel plate with high strength was designed to ensure a uniform loading onto the load cell. Unfortunately, the layout of the temporary work in Victoria Station would not allow us to install the load cell. Inserting it would destroy the uniformity of the temporary work. Countersinking the load cell between the needle beam and plank would cause a huge reduction in size of the timber plank. This violated the safety requirement from the contractor. Load cells were not used in the end. It was saved for future deployments. The experiment would still be fine without the load cell, since it was only for further confirming the load exerted on the plank.

The designs were manufactured in the CUED workshop. Two types of strain gauge plates were manufactured. The first had a thickness of 0.5mm while the other had a thickness of 1mm. The former had to be fabricated by hand tools while the later can be machined. It was intended that both types were to be tested (Session 9.4).

9.1.2 Other Preparations

The electrical and software of the sensor nodes were prepared by the software team. This consists of a set of software codes written using Contiki, the design and building of the circuit on the PCB board. The whole system were connected together and tested before the deployment. Short and long run tests were done. The strain gauges were connected to a Whitestone Bridge circuit for auto-logging by the nodes.

9.2 Calibration



Figure 9.3: Experimental setup for Calibrations of a) Left - Strain gauges (3-point bending test), b) Right - load cells (axial compression)

Before the deployment, calibrations must be conducted to study the relationship between the actual desired data and the signal collected (i.e. strain – resistance for strain gauge; & load – resistance for load cell). Loading tests were done to conduct the calibration. Initially, both calibrations were carried out using the instron machine.

The strain gauge was attached to the designed plate, which was screwed onto a sample plank at the mid-point on the tension side. The plank was loaded by a 3-point rig of the machine. The load cell is loaded by a punch. The following graph shows the calibration curve of the load cell. The load-resistance behaviour shown a straight line as expected.

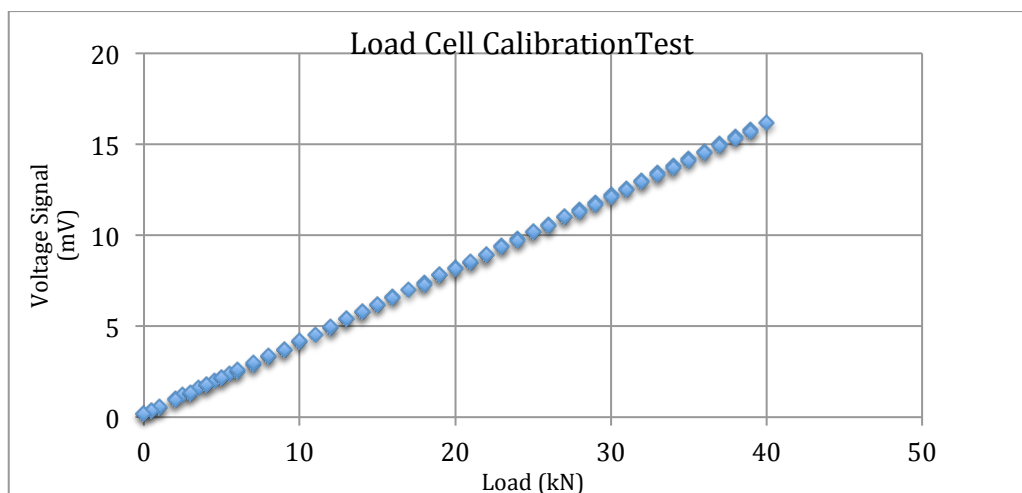


Figure 9.4: Calibration result of load cell

The instron machine worked by a displacement – control manner. The load are increased and logged automatically by the computer of the machine. The load was

increased continuously in a quick rate without pauses. The resulting graph showed an unclear relationship between resistance and load for the strain gauges.

It was suspected that there was a finite response time for the strain gauge. The calibration was therefore repeated using hanging weights to load the plank. Loads were increased in increments with pauses each time. During this test, three strain gauges were tested side-by-side at the same time with different attachment method:

- 1) The strain gauge was directly glued onto the plank by epoxy,
- 2) The strain gauge was glued onto the designed plate ($t = 0.5\text{mm}$), which was screwed onto the plank (to test whether this setup works)
- 3) The strain was glued onto an aluminium plate ($t = 1\text{mm}$) and the plate was also glued onto the plank (to test the effect of local reinforcement, and to differentiate the effect of attachment with screws and that with epoxy).



Figure 9.5: Setup for calibration of strain gauges and comparison of attachment method

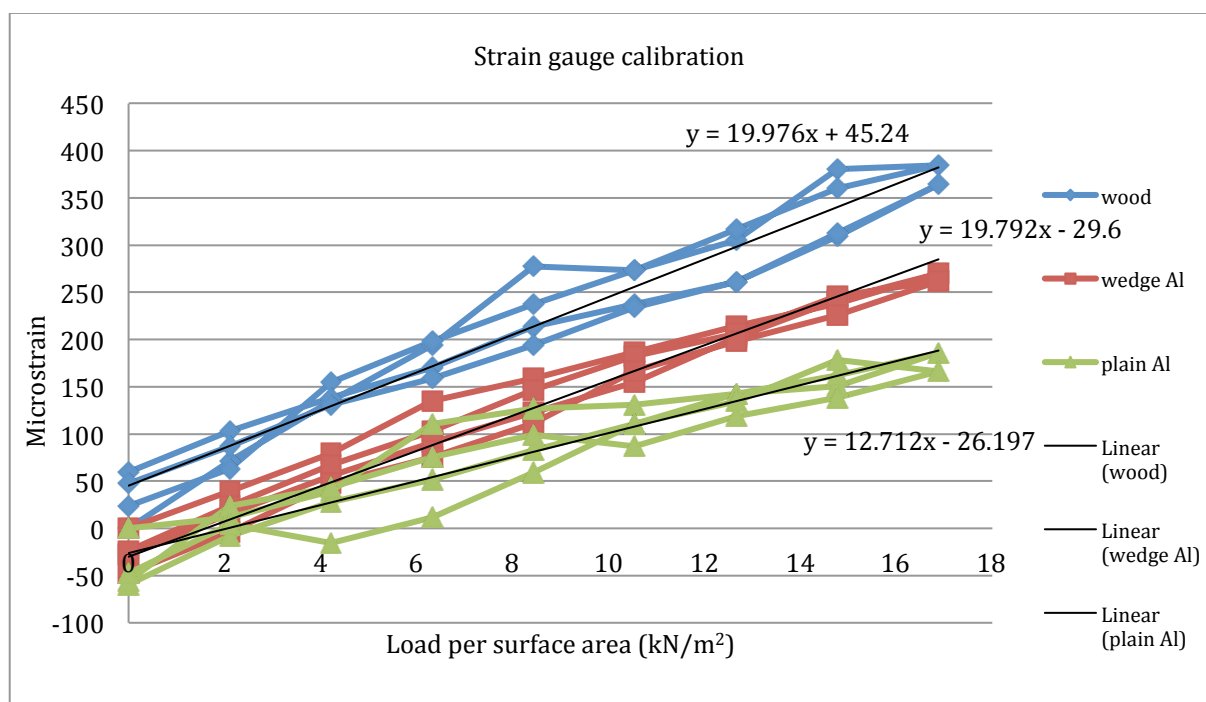


Figure 9.6: Result of loading and reloading test of three strain gauge test results

With the incremental increase of load, the results show a repeatable linear relationship between load and resistance. The resistance values were converted to strain values according to its specifications. The 3 sets up resistance data were compared. The results were repeatable during loading and unloading. The initial offset of the graph of the three sets of data was due to the slight variation in resistance of each strain gauge. The strain

gauges were planned to measure change in strain during the deployment. The gradient of the graph, the change in strain (resistance) relative to that in load, was the vital parameter. The data of the strain gauge directly stuck onto the plank (blue) and the one on the designed plate (red) gave a very similar gradient. The small 0.9% difference was acceptable. The effect of the extra aluminium plate was negligible. The gradient of the data of the strain gauge (green) on the thick plate had a smaller gradient, meaning that the plank might be locally reinforced, since doubling the thickness would increase the second moment of area by eight times. Equations of calibration lines are shown in graph.

It is concluded that the designed plate was capable as a mechanical attachment method of the strain gauge for the deployment. 0.5mm thickness is thin enough. This setup was then used for the deployment. For the 1mm plate, more tests were planned for the deployment to test the effect of it.

9.3 Comparison with Theoretical Values

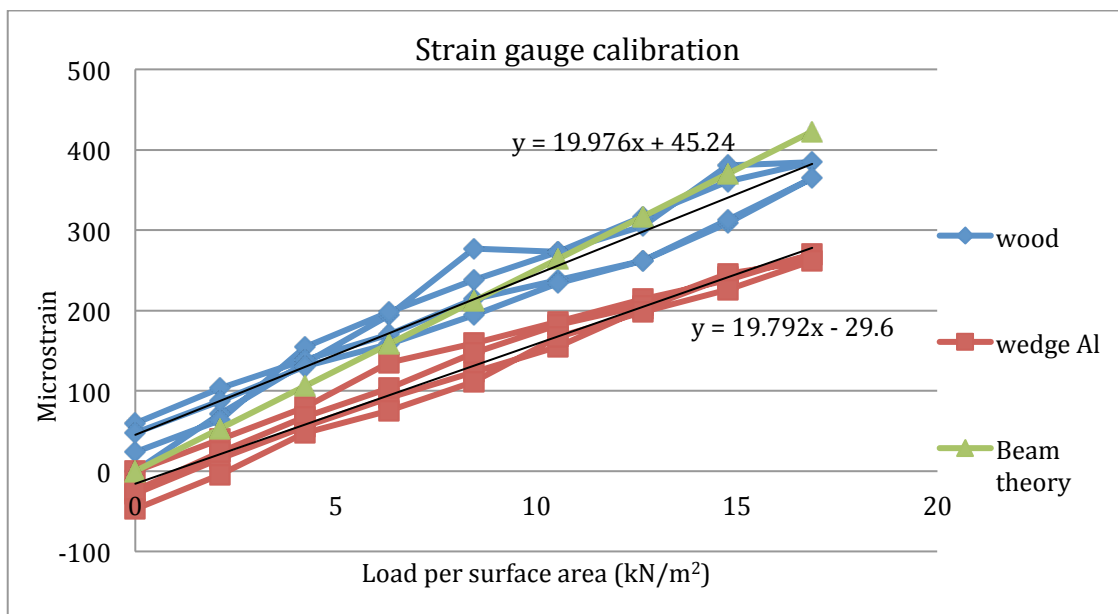


Figure 9.7: Comparison between theoretical beam theory data and calibration result

A line representing the strain values of corresponding load applied in the calibration calculated by beam theory was plotted with the data. The strain at mid-point at the tension extreme fibre was calculated according to the dimensions of the plank used ($E = 5.5\text{GPa}$, $b = 0.093\text{m}$, $d = 0.033\text{m}$ & $L = 0.4\text{m}$). The experimental and theoretical lines lied close to each other, proving that the measured values agreed well with the theory. Hence, the setup was capable of doing the planned measurements. The strain values

measured can be converted to corresponding load by applying the beam theory of a uniformly distributed load (Session 6.4). Again, the initial offset of the data was due to slight variation in every strain gauge used. The measurements were taken as changes relative to the strain value at the initial unloaded condition.

9.4 Experimental Layout

After discussion with the contractor, it was agreed that our sensor system (without load cells) would be installed to the central plank of the temporary work (plank 3 in Figure 6.6). The sensor network was deployed to four central planks. A total of sixteen strain gauges would be installed, with four gauges on each plank. The instrumented planks are numbered 20, 21, 22 and 23. The gateway computer was placed at the bottom of the shaft, at a position with a best line-of-sight to the instrumented planks (Figure 6.1).

The planks were spread along the whole cross passage. The layout plan of the gauges on the planks is shown below.

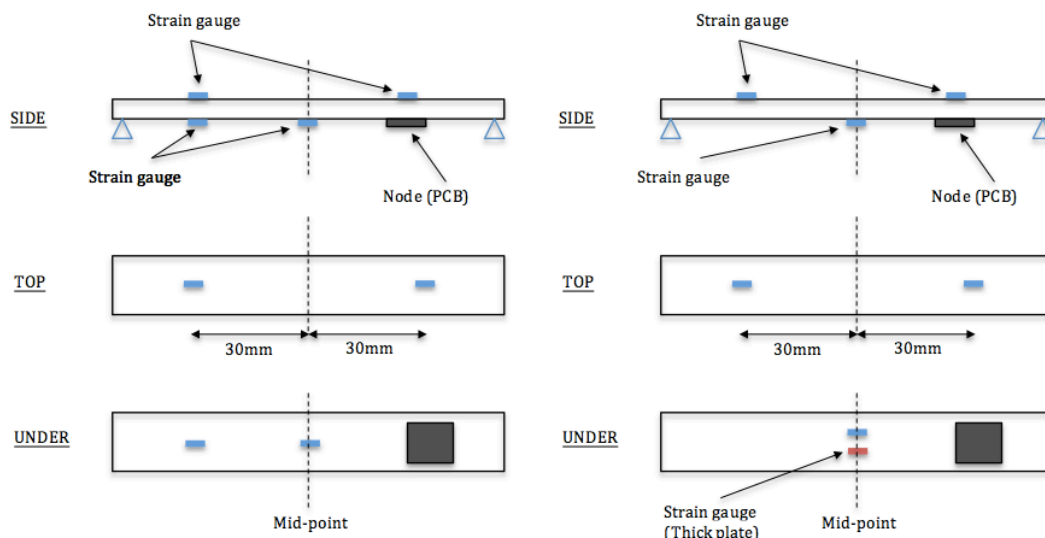


Figure 9.8: Layout plan of sensors, a) Left – Plank 20, 21, 23, b) Right – Plank 22

Two layout plans were designed. Both plans have two strain gauges on the topside (compression side), 30mm from the mid point of the plank. And the sensor nodes with the antenna were always mounted on the underside to ensure a good signal transmission. The first layout (Figure 9.8a) has a strain gauge at the mid point of the plank and another one 30mm from the mid point on the underside. It was assumed that the plank would experience a uniformly distributed load. The maximum bending

moment and maximum strain at extreme fibre would be present at the mid point on the underside (tension side). A strain gauge located at this point can take the strain measurement. Plank 20 and 21 had all four strain gauges attached using the thin plate. This provided an extra set of data as a repeat to ensure reliability. Plank 23 also had the same layout but had all the strain gauges on thicker plate instead. This gave a set of data to compare with the other two sets for identifying the effect of the thicker plate.

Plank 22 had another layout, with 2 strain gauges installed side-by-side at the mid-point of the underside. One was attached using the thin plate and the other using the thicker plate. This also enabled us to identify any differences between the two setups. All the strain gauges were connected to the sensor node.

The planks were instrumented on site a week before the planks were being put into the designated loading position. The gateway was installed at the bottom of the shaft, the 3G signal was tested during the instrumentation. The instrumented planks were wrapped with tape to protect them from any physical damages. They were stored in a covered area to prevent the electronic components from being damaged by rainwater.

It was not allowed for us to place the plank at the loading position ourselves. The miners were briefed about the installation of the plank. The underside of the plank was sprayed by paint to clearly show the orientation of the planks. Also, if there were any final adjustments of the span length of the planks, the miners would cut even lengths on both ends and to ensure the planks were placed carefully with the central strain gauge located at the mid-span. The following are photos of the instrumented planks.



Figure 9.9: Instrumentation of plank on site

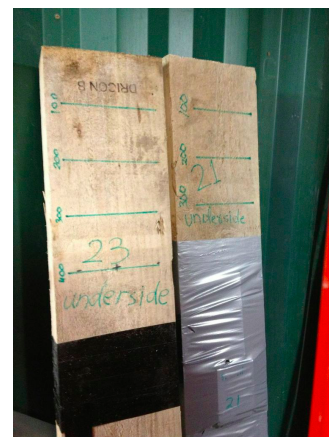


Figure 9.10: completed Instrumented planks



Figure 9.11: Attached strain gauge



Figure 9.12: Instrumented plank with sensor node (underside)

9.5 Problems Encountered

There were problems during the deployment of the first two planks (planks 22 and 23). These were tackled and solved promptly.

9.5.1 Corrosion of the Sensor Nodes

When the first two planks were located in their working place and being switched on. No signal was detected by the gateway. It was suspected that the signal strength was not strong enough. However, there was still no detected signal after the signal strength was strengthened (next session).

The deployed planks are not accessible, so investigation was carried out with the remaining two planks (plank 20 & 21). The tapes were unwrapped. It was discovered that there were significant amount of oxidised components in the sensor nodes, caused by the unexpectedly high humidity in the environment. Moreover, the batteries in a sensor node had exhausted. It was accidentally switched on during the storage period.



Figure 9.13: (a) Left: Damaged PCB board by oxidation (b) Middle: Damaged Sensor node casing by oxidation (c) Right: Repaired and protected sensor node

The oxidised components and the batteries were replaced with new ones immediately. Extra water proofing precautions were employment to protect the sensor nodes.

Waterproof membrane were wrapped around the PCB board within the sensor node. Then the plank was wrapped again and stored.

9.5.2 Weak Signal

The repaired sensor nodes worked normally before installation. Afterwards, the signal then became weak and some signal packages were lost. The signal was likely to be interfered by the large amount of steel beams nearby or by the construction works. A number of relay nodes were then installed along the tunnel leading to the planks. The signals could then be transmitted from the sensor node through the relay nodes to the gateway. Four relay nodes were installed and data was then collected properly.



Figure 9.14: a) Left: Relay node, b) Middle: Installation of Relay node in Pal 16, c) Right: Installation of Relay node in Pal 16

9.5.3 Loss of Data

Due to the oxidation problem, the first two planks became faulty. There was no data collected from these two planks (plank 22 and 23). The other two planks were repaired and tested again before they were deployed. Data was collected and stored properly. However, after the extraction of data from the gateway computer. It was discovered that the data from plank 21 had fluctuating values throughout the construction phases. Since all the strain gauges give fluctuating values. This was a hint that the sensor node might be damaged during the construction. Other one set of data was obtained from plank 20. Unfortunately, it was not possible to differentiate the effect between the two attachment methods without the data set from planks 22 and 23. The data from plank 20 is analysed in the next session.

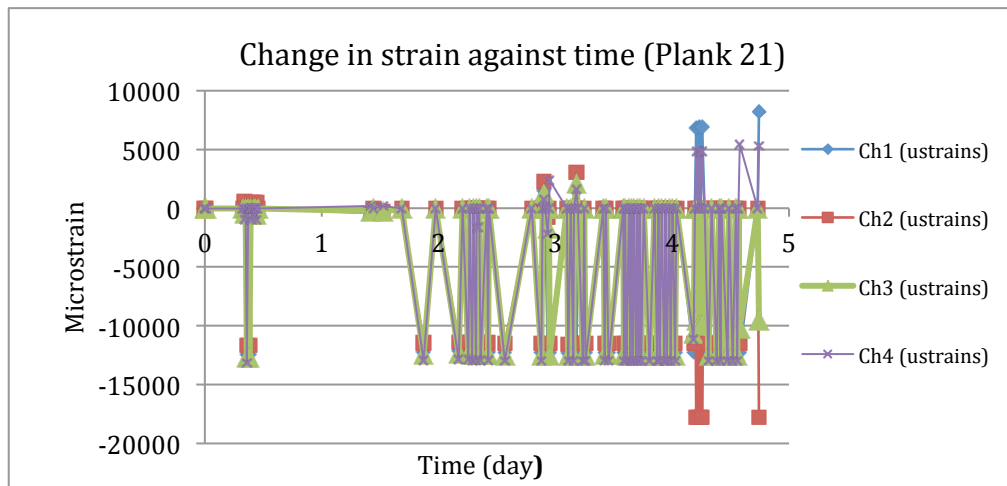


Figure 9.15: Faulty data from plank 21

9.6 Results

Planks 22 and 23 were put into place as the temporary structure on 21 Apr 2013 at 10:00am. The sensor nodes were switched on at the same time. They were discovered to be faulty with no detected signal.

Planks 20 and 21 were placed into the loading position on 24 Apr 2013 at 10:00pm (the construction site worked on a 24-hour-7day basis). The sensor nodes were also switched on. By the time that the instrumented planks were to be installed, the two side planks were already in place. The central planks (planks 20 and 21) were put into place by first excavating the layer of soil beneath its location. Then, masses of soil above the end points of the two side planks and the soil above the designated location of the central plank were excavated (Figure 9.16). At this moment, it was witnessed that the soil could hold it self without failing before central planks were put in. The two ends of the side beam supported the soil above the open space.

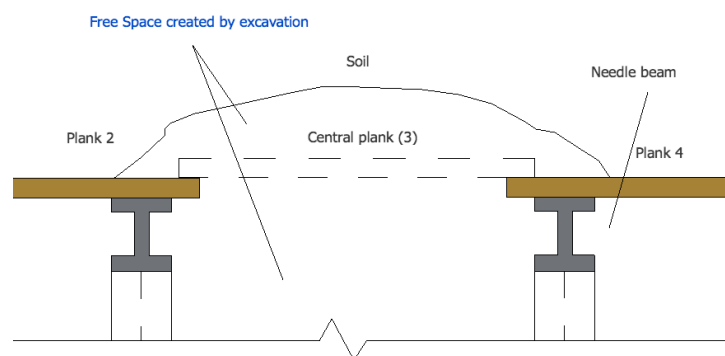


Figure 9.16: construction and excavation

Afterwards, the central planks were put on top of the two ends of the side planks. The load on the plank at this time was only caused by its self-weight. At 11:00pm of the same day, the free space between the top of the plank and the soil was grouted. Graph xx below was plotted with the change in strain of the strain gauges against time. The reference strain values (zero change) were taken as the strain value before the grouting starts. The x-axis represents the time, with the start point as the time when grouting was conducted, which was the time when the plank was loaded. The corresponding change in strain values relative to the reference values were plotted. A thermistor was installed within the sensor node, the temperature was also plotted against time to better indicate activities around the plank. Table 9.1 below shows the positions of the strain gauges.

Plank 20	Side	Location along span
CH1 (gauge 1)	Top (Tension)	30mm from mid-point
CH2 (gauge 2)	Top (Tension)	30mm from mid-point
CH3 (gauge 3)	Underside (Compression)	Mid-point
CH4 (gauge 4)	Underside (Compression)	30mm from mid-point

Table 9.1: Locations of strain gauges on Plank 20

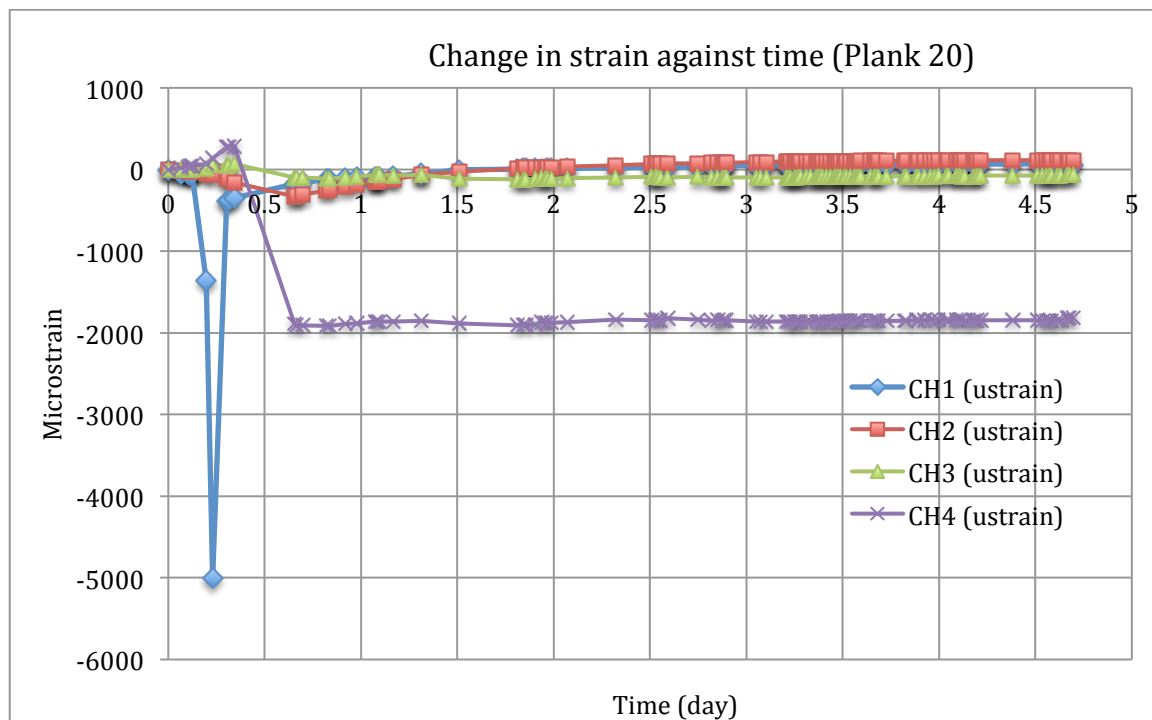


Figure 9.17: Deployment result – Change in strain values with time of strain gauges on plank 20

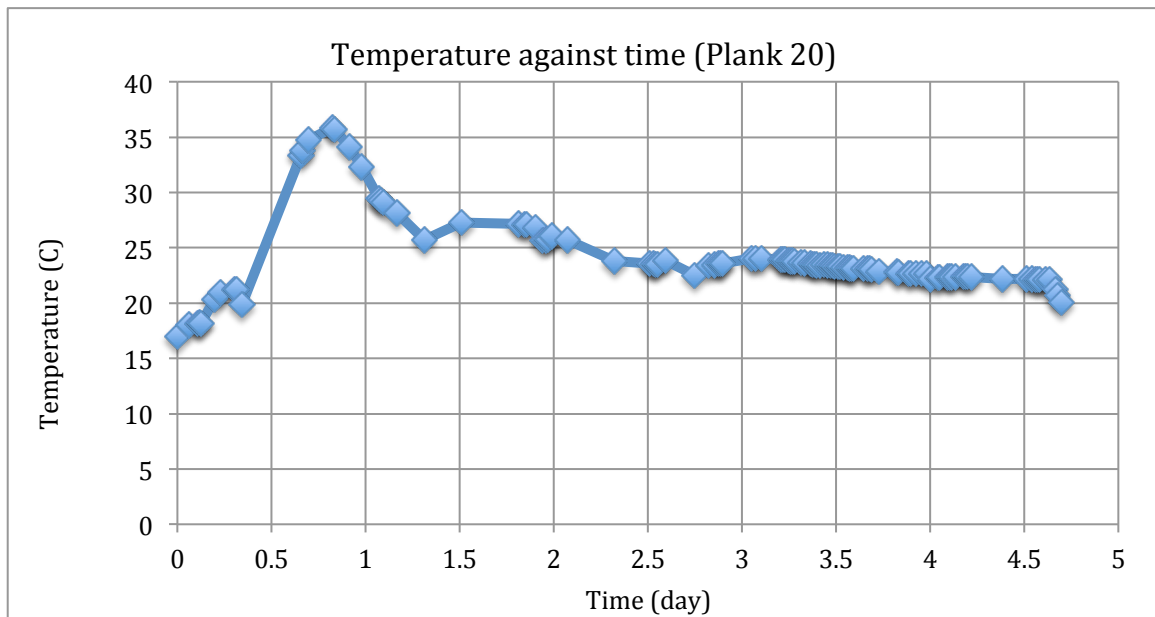


Figure 9.18: Deployment result – Change in temperature with time of thermistor on plank 20

From Figure 9.17, there are only changes in strain over time during the start (small x-values). The values then settle and approaches zero for gauges 1, 2 and 3. Gauge 4 gives a very large constant residual value. It was concluded that gauge 4 had become faulty after about $t = 0.6$ days (~ 25 Apr 2013 12:00pm). Or some local effect happened at its location, causing the big jump in strain values and the effect stayed unchanged. To look into details the changes at the beginning, the following graph was plotted zooming into range of $x = 0 - 0.4$ days.

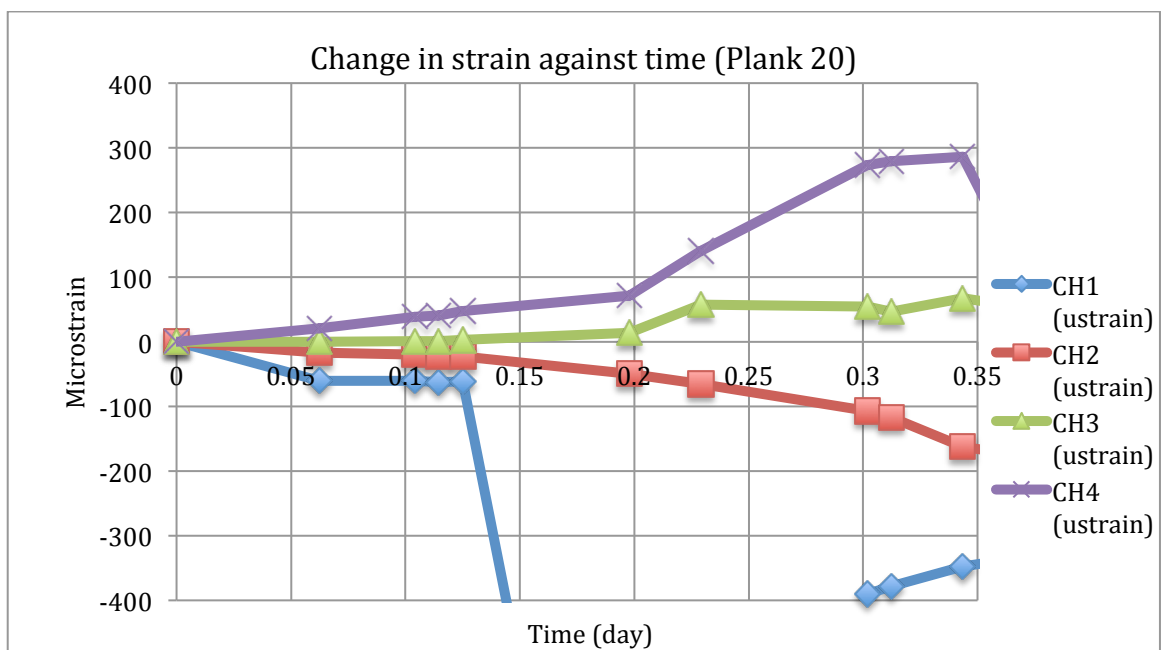


Figure 9.19: Deployment result – Change in strain values with time of strain gauges on plank 20 ($x = 0$ to $x = 0.35$)

From the graph, gauge 1 and 2 gave negative values representing compression as expected, while gauge 3 and 4 gave positive values representing tension. This showed that the grout had applied a downward pressure from the top of the plank. From the Figure 9.18, the temperature started to rise from $t = 0$, this was caused by the heating effect during the setting of the grout. The strain values after $t = 0.35$ days began to settle and approaches to zero (except gauge 4). The load values corresponding to these strain values were computed according to beam theory (Session 6.4) or by the calibration curve. Again, a uniformly distributed load was assumed. The parameters of the C24 timber planks used were $E = 9 \text{ GPa}$, $b = 0.225\text{m}$, $d = 0.1\text{m}$, and $L = 1\text{m}$. The location of the strain gauges were taken into account for the calculations (Table 9.1). The span length was modified to 1m before the construction by the contractor.

It was noted that the thermal coefficient of expansion of timber was in the order of about $2 \times 10^{-6} \text{ /m/}^\circ\text{C}$, varying across type of timber and the environmental conditions, such as moisture ⁽³⁾⁽⁵⁾. The strain by temperature along grains, which was in the direction where the strain values were measured in this experiment, was smaller than that across grain given by the coefficient ⁽³⁾. The effect of temperature was assumed to be negligible.

	Gauge 1	Gauge 2	Gauge 3	Gauge 4
Micro-strain (change)	-347	-162	67	286
Local bending moment (Nm)	624.9	291.7	120.7	515.0
Corresponding Pressure (kPa)	34.7	16.2	4.2	28.6

Table 9.2: Result - Corresponding load for strain measurements

Gauge 3 was at the mid-point of the span and should have the greatest strain. Its strain was smaller than the other three values, showing that there were experimental errors at this point. One possible reason was a locally strengthened point on the timber. Taking the mean of other values, the average pressure acting on the plank was 26.5kPa which was a lot smaller than the design load of 81.7kPa. The design load was overestimated.

The large peak given by gauge 1 at $t = 0.2$ was due to local effect. As the value restored after the peak, the effect had gone and the gauge worked fine afterwards. This peak

value corresponded a very large load ($\sim 320\text{kPa} > \text{full overburden}$), which was not possible and was therefore ignored.

From the change in strain graph, all the values approached zero after the peak values at $t = 0.35$ days. The hardening of the grout caused this. The temperature graph showed a rise followed by a fall, which corresponds to the setting and hardening of the grout. The strength of the grout had grown and the plank no longer held the load. Any load from the soil above was withstood by the grout instead. Moreover, recalled that during the construction process, the soil could hold itself between the supports of the central plank even before the plank was placed. This showed that there was significant arching happening. The soil arched between the supports. Loads were concentrated there. The pressure at mid span of the plank was very small due to the effect of arching.

The grouting injection pressure used was $8 \text{ bar} = 800\text{kPa}$. However, the vertical pressure on the plank must be a lot smaller. A 800kPa pressure was very large, even greater than overburden, the spreading of the grout above the plank had caused the pressure to reduce greatly. Moreover, the quality of the grout might not be good enough to provide a uniform pressure on the plank. There was error in the calculations above, but it was believed that the pressure value calculated was close to the actual value. There was a difference of $\sim 50\text{kPa}$ from the measured pressure from the design load. The actual load was concluded to be smaller.

In the future, if there is a good analytical prediction method of the behaviour of the soil under arching effect, the size and section of the plank can be modified to only withstand the load from the soil after arching has happened and the load from the grout. A smaller size can be used. This conclusion was only made based on this one set of results, more experiments must be conducted in the future in order to confirm this finding.

10. Results and Discussion - Finite Element Analysis

This session describes the details regarding the finite element modelling of the temporary work and the soil surrounding it. The development process of the model, the results and any findings are discussed.

10.1 Geometry and Meshing

10.1.1 Final Model

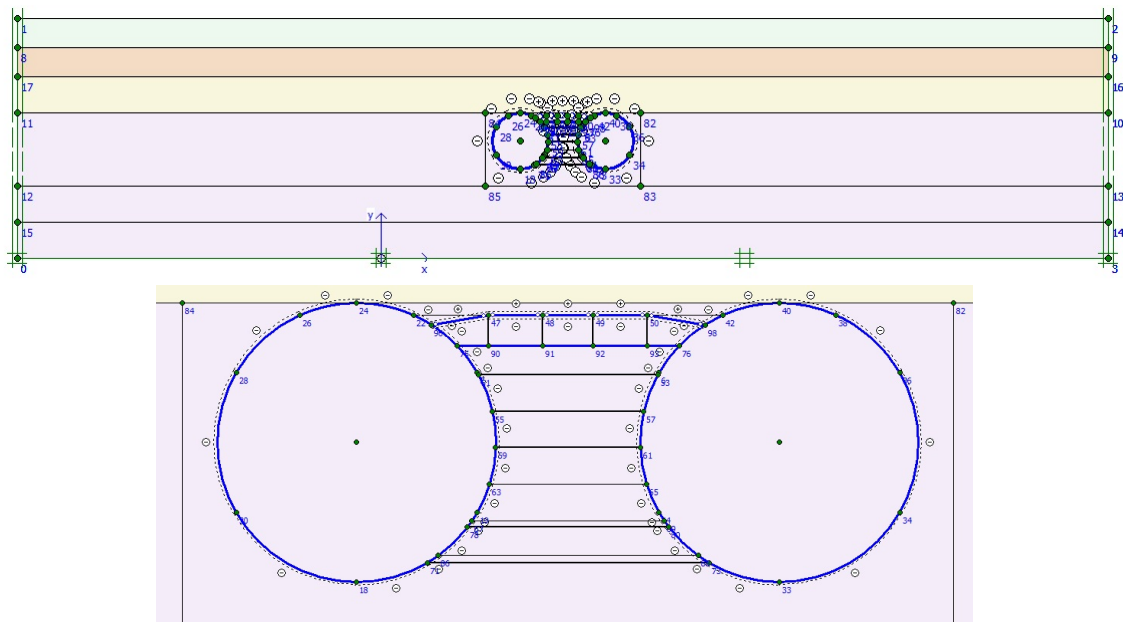


Figure 10.1: Geometry of the Finite Element Model

The geometry of the cross passage and the temporary work were input to the software. The geometry inputs were identical to values specified in the design consultancy reports Table 10.1. The geometry of the final model is shown in Figure 10.1 above. Mechanical properties of structure were also input to the software, such as those for tunnel linings. The geotechnical report gave the soil strata. The timber temporary structure is fully within London clay. The section of the ground at the site is shown in Figure 10.2.

Parameter	Value (units)
Radius of tunnel	3.8m
Span of each timber plank	1.4m
Depth of plank from ground surface	13.8m
Depth of model	33m
Width of model	150m

Table 10.1: Dimensions of relevant structure for finite element model

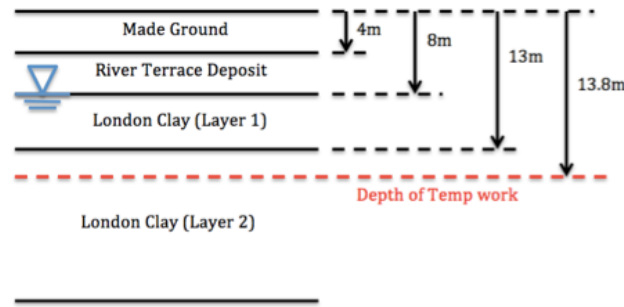


Figure 10.2: Soil strata at the Victoria Station Site

Plate elements are used to model the timber plank and the permanent works. Supports of the planks are modelled using point-to-point anchors. A point-to-point anchor is a 2-node elastic spring that can sustain both tensile and compressive load and that connected relevant structures. Point-to-point anchors were also added between the tunnels to simulate the temporary props used during the excavation of the cross passage. Plaxis also has a specific function for modelling tunnels. Interface elements are placed at the surfaces of structures that were in contact with soil.

For example, the support of the planks is a needle beam that was held by concrete blocks, which in turn sat on a permanent roof beam. This roof beam sat on the existing tunnel linings. In the model, the supports of the planks were modelled by point-to-point anchors connecting the junction of the planks (plate elements) and the roof beam (another plate element). The roof beam is connected to the tunnel linings.

After inputting the geometries and material parameters, a mesh of elements was generated. The mesh was refined to be denser around the location of planks and tunnels to ensure a high accuracy in area of interest. The elements were significantly smaller in this area. Figure 10.3 shows the final mesh, which consisted of 22800 elements.

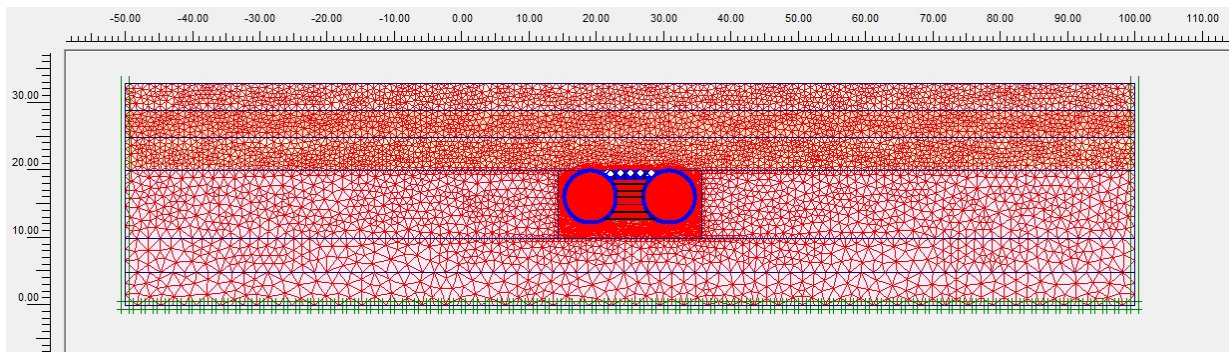


Figure 10.3: Mesh generated for final model

10.1.2 Preliminary Models

Before the final model was generated, preliminary designs of the model were done. They were kept being modified to best suit the conditions at the construction site and to match against any sudden changes happened at the site that affect the temporary work.

Preliminary design 1

This design Layout (Figure 10.4) did not work. Fixed end anchors were first used to model the supports of the timber planks. A fixed end anchor is an elastic spring that fixed a point to a designated space. As in this model, the supports were fixed at a point in the soil. During the tunnelling and the consolidation phase, the tunnel had a significant

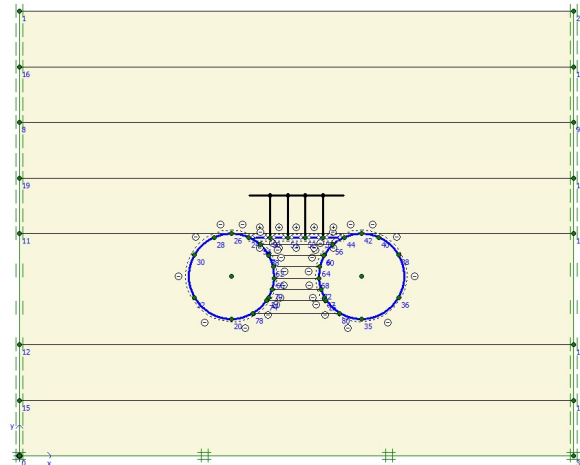


Figure 10.4: Layout of preliminary design of FE model – using fixed-end anchors

displacement relative to its initial defined position by the input geometry. As the planks were connected to the tunnel linings in the input, the planks and the connected point of the anchors on them moved with the tunnels. The other ends of the anchors were fixed in space, a huge extension was induced and thus a huge stress. Therefore, the anchor failed. This would not happen in reality since the supports were designed according to the final location of the tunnel (after the displacements). Therefore, the layout was modified to have the anchors moving with the tunnels.

Preliminary design 2

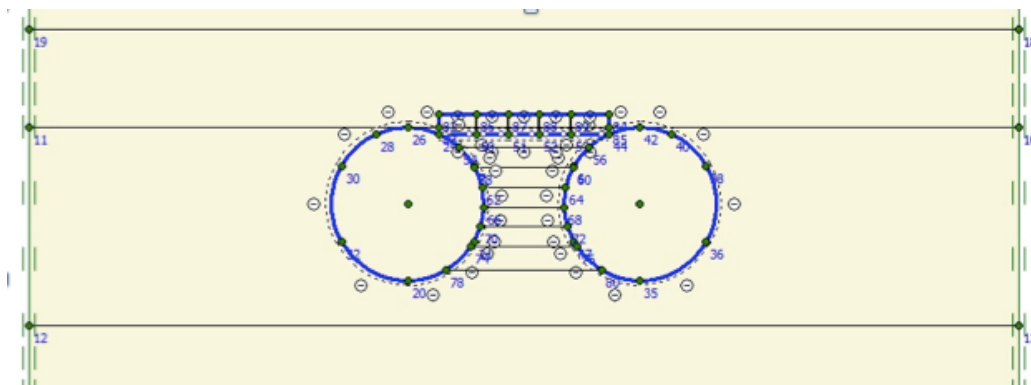


Figure 10.5: Layout of preliminary design of FE model – using point-to-point anchors connected to a frame

A frame, which is a plate element, was added to the previous design at the end points of the planks where they were connected to the tunnel linings (Figure 10.5). The supports were now modelled as point-to-point anchors connecting the point of junction of the timbers and the point on the frame vertically above the support. The frame would move together with the tunnel during the phases before the excavation. The behaviour of the support was modelled properly. From the result of this model, the frame interacted with the soil surrounding it, which was directly above the timber planks. This caused a huge error in modelling the vertical stresses on the plank. This must be prevented.

The model was modified to have the anchors below the planks connected to a horizontal plate element supported by the tunnel linings. This best simulated the actual layout in the construction site and eliminates any disturbances in the soil above the planks. The final design shown above (Figure 10.1) was then generated. The width of the model was also extended to better simulate the settlements of the soil. Furthermore, these preliminary designs only contains one type of soil stratum, which was London Clay, this was improved in the final design to include other layers of strata to match the actual environment.

10.2 Boundary Conditions and Initial Stresses ⁽¹⁴⁾⁽¹⁵⁾⁽¹⁶⁾

As mentioned, effective stress analysis and equivalent effective stress analysis were carried out for drained and undrained conditions respectively. The made ground, the river terrace deposit were located above the water table, and were therefore always in drained conditions. For the London clay, undrained conditions are applied in all construction phases, including the tunnelling for the Victoria Line, the excavation in the current upgrade project and the construction of the temporary works. Drained conditions are only used for the consolidation phase for the London clay.

After generating the mesh and specifying soil parameters, the initial conditions were generated before the start of the analysis. In both effective stress analysis (drained) and equivalent effective stress analysis (undrained), the water table is considered at the top of the London clay layer. The water pore pressure was generated according to this level. Whenever a layer of soil is removed, the layer would be set to dry to eliminate the water pore pressure ⁽¹⁶⁾.

In effective stress analysis, the initial stresses within the soil were dependent on the input values of K_0 and internal friction angle. For the equivalent effective stress analysis, the initial conditions were also dependent on the level of water table, and the K_0 and internal friction angle of the soil stratum. However, the input values of cohesion and internal friction angle were derived from the undrained parameters instead. These values were calculated based on the knowledge of the effective stress profile of the soil.

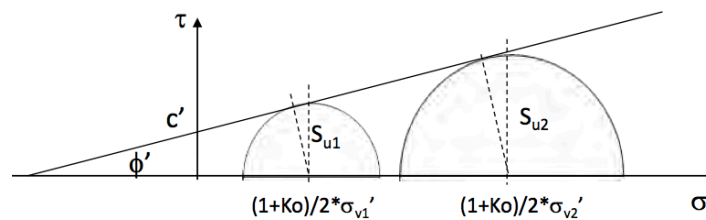


Figure 10.6: Mohr-Coulomb model for equivalent effective stress analysis

The following parameters were input to Plaxis for further analysis.

	Effective Stress Analysis				Equivalent Effective Stress Analysis	
Parameter	Made Ground	River Terrace Deposit	London Clay (layer 1)	London Clay (layer 2)	London Clay (layer 1)	London Clay (layer 2)
γ (kN/m ³)	18	18	20	20	20	20
K_0	0.5	0.5	1.2	1.2	1.2	1.2
E (kPa)	10	25	2.4×10^4	2.9×10^4	2.4×10^4	2.9×10^4
ν	0.2	0.25	0.2	0.2	0.2	0.2
ϕ (°)	25	25	25	25	33.4	33.6
c (kPa)	$1 \times 10^{-4} \sim 0$	$1 \times 10^{-4} \sim 0$	5	5	62	60
ψ (°)	0	0	0	0	0	0
Permeability (m/day)	1×10^{-9}	1×10^{-9}	1×10^{-9}	1×10^{-9}	1×10^{-9}	1×10^{-9}
$E_{\text{increment}}$ (kPa/m)	0	0	1000	1167	1000	1167
$C_{\text{increment}}$ (kPa/m)	0	0	0	0	6.6	6.6

Table 10.2: Input parameters of soil strata to Plaxis

The initial pore water pressure and the initial soil stresses were as follows:

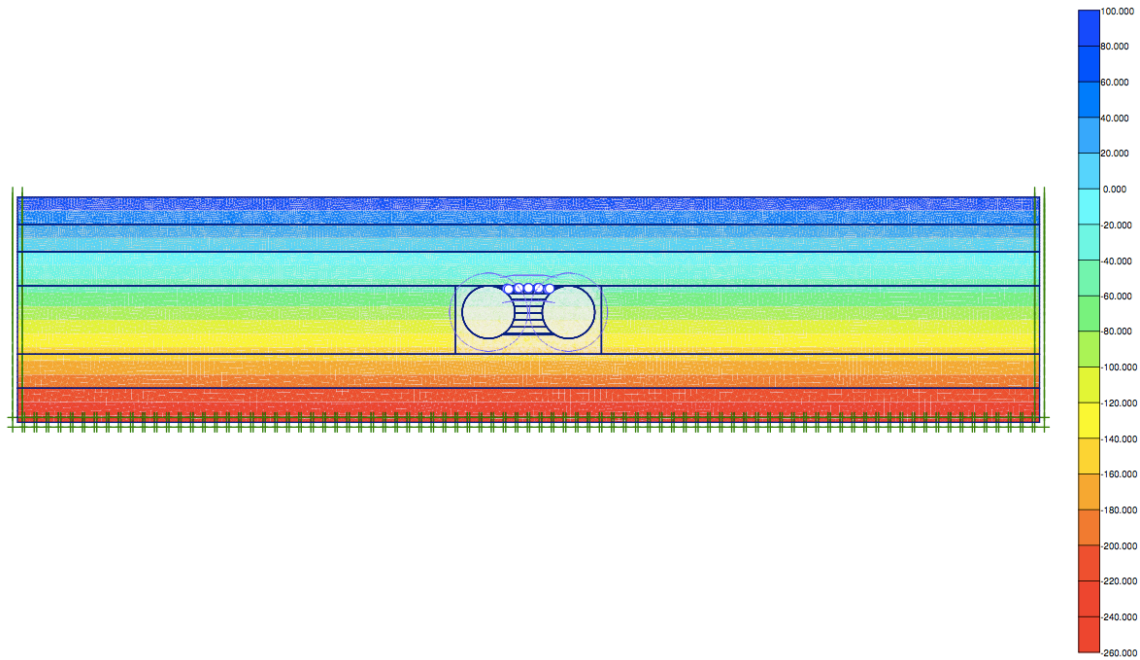


Figure 10.7: Initial water pore pressure diagram (Scale: -260 to 100kPa)

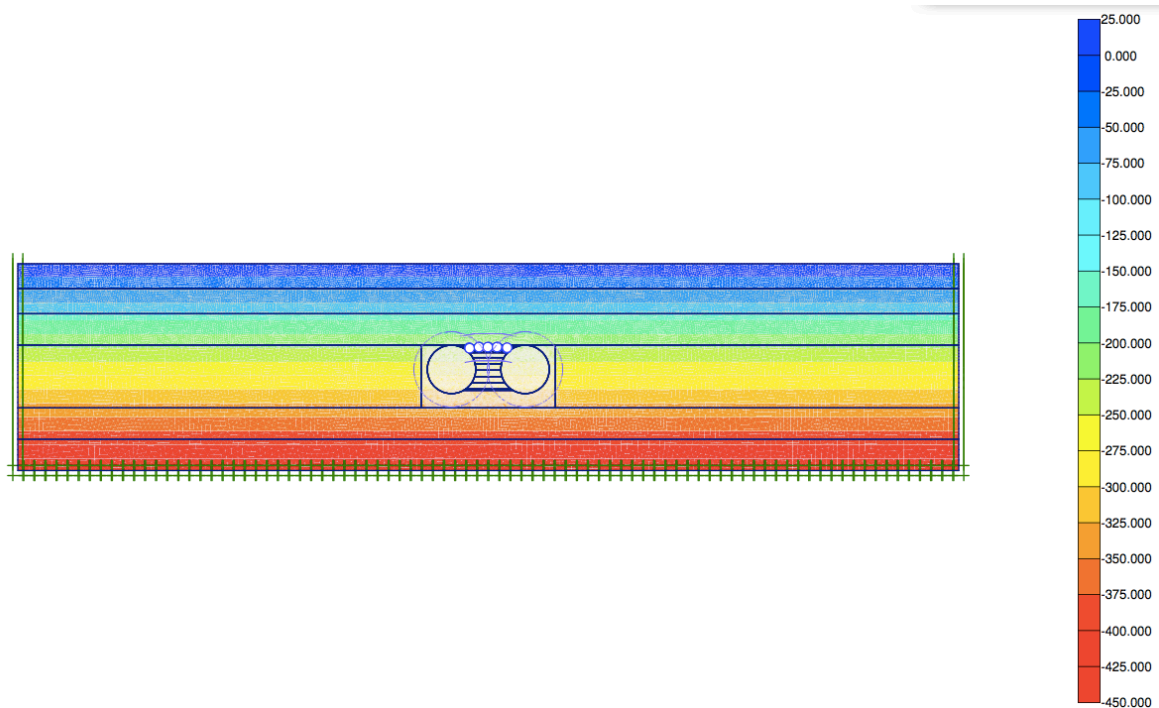


Figure 10.8: Initial soil stress diagram(Scale: -450 to 25kPa)

10.3 Results

10.3.1 Tunnelling

The following diagrams show the deformed mesh of the soil after the tunnelling. According to the skill and construction technology during the Victoria Age when the tunnel was constructed, a contraction ratio of 3% of the tunnel was assumed.

The results shown a settlement trough at the ground surface as expected. The maximum settlement was 65.7mm. The settlement at the edge of the model was 4.8mm that was small enough to be neglected. This showed that the width of the model was large enough to model significant changes. It was also observed that the tunnels moved downwards and towards each other, the maximum total displacement of the tunnel was 102.4mm.

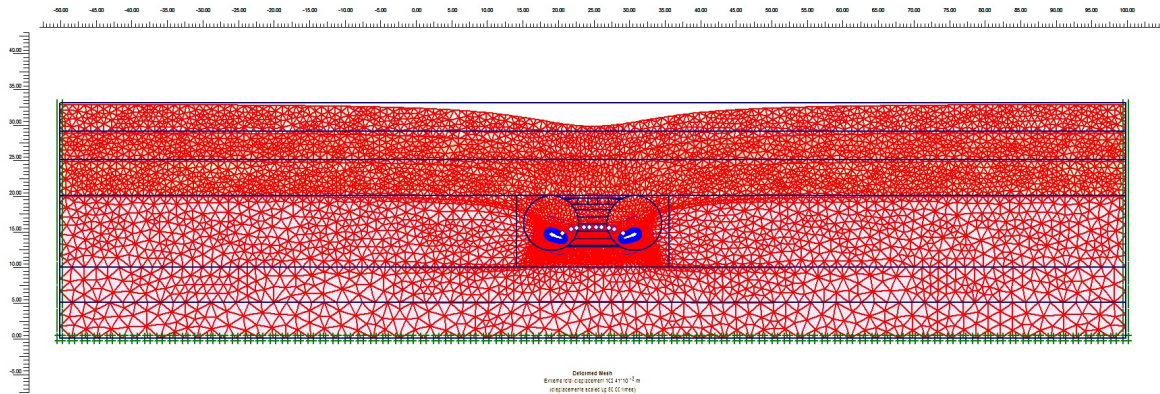


Figure 10.9: Deformed mesh after tunnelling phase (Scale = 50x)

10.3.2 Consolidation

A consolidation phase was added between the tunnelling phase and the construction phase of the temporary works. This worked with the theory stated in Session 8.7. Excess pore pressure generated during the tunnelling phase was allowed to dissipate for 50 years, which was about the age of the Victoria Line Tunnels. The effective stress of the soil would be closer to real values; the results of later phases would be more accurate.

Two modes of consolidation were available in Plaxis. The first one was time increment; the second was minimum pore pressure. In the time increment mode, the consolidation period was input, the phase would stop after the simulation within this period had finished. The second one involved an input of a targeted minimum pore pressure value (1kPa in this project), the calculations in the software would stop if this value was reached. The later option involved a significantly shorter calculations time of around 30 minutes while first mode took about 8 hours for each run.

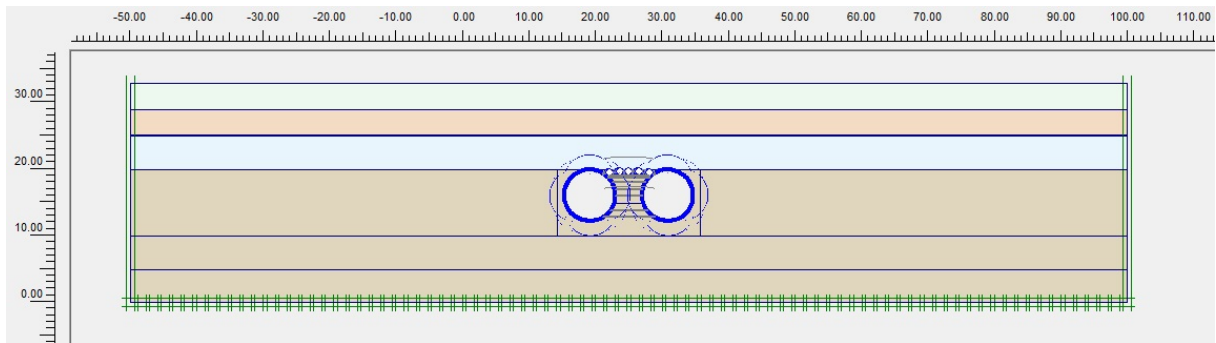


Figure 10.10: Result of consolidation (time increment mode: 50years) – Excess ore pressure plot (zero everywhere)

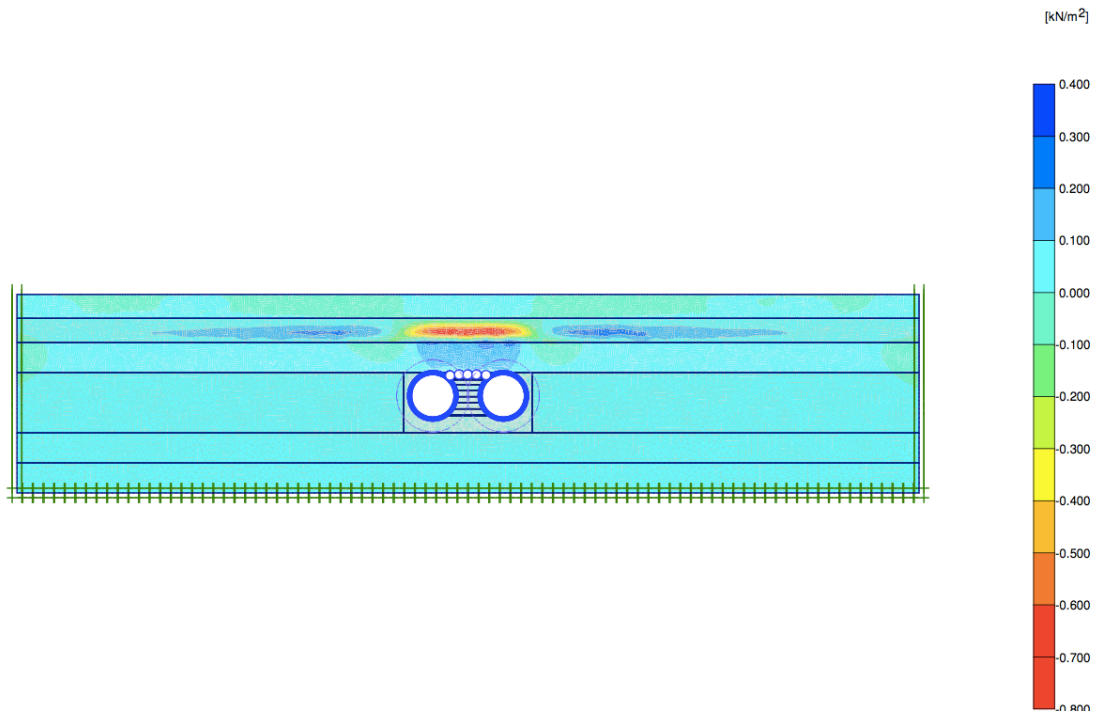


Figure 10.11: Result of consolidation (minimum pressure: 1kPa) - Pore pressure plot (Scale: -0.8 to 0.4 kPa)

The first plot using time increment mode of 50 years, no shadings are shown. The results showed that all the excess pore pressure was dissipated. The results from the minimum pore-pressure mode gave a shading contour ranging from -0.8kPa to 0.4kPa, which was close enough to the result of the previous mode. Further calculations were conducted using the minimum pore pressure mode to save calculation time. With the appropriate input used, both modes give very close results.

10.3.3 Loading on Timber Plank

After the consolidation phase, the soil in bench 1 which was immediately below the planks was excavated (Session 6.3.3). The timber planks, the supports and the horizontal beam were activated at this stage. The displacement and bending moment acting on the central plank were plotted as below.

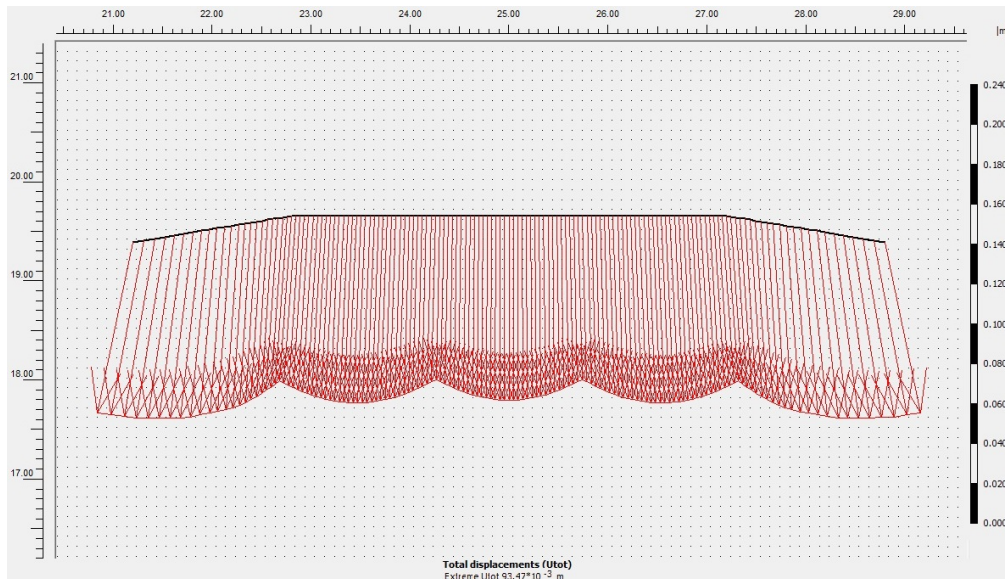


Figure 10.12: Total Displacement of the central timber plank after construction (Max: 93.43mm, legend unit: 40mm)

The displacement in the diagram was measured from the initial location of the geometry input of the plank which was before the tunnelling phase. This was the reason why the displacement was large at 93.5mm. The displacement of the plank after it was being constructed relative to the start of the construction was calculated by subtracting the displacement of the plank by that of the tunnel (96.8mm) at this phase. This gave a maximum displacement of the plank of $96.8 - 93.5 = 3.3\text{mm}$.

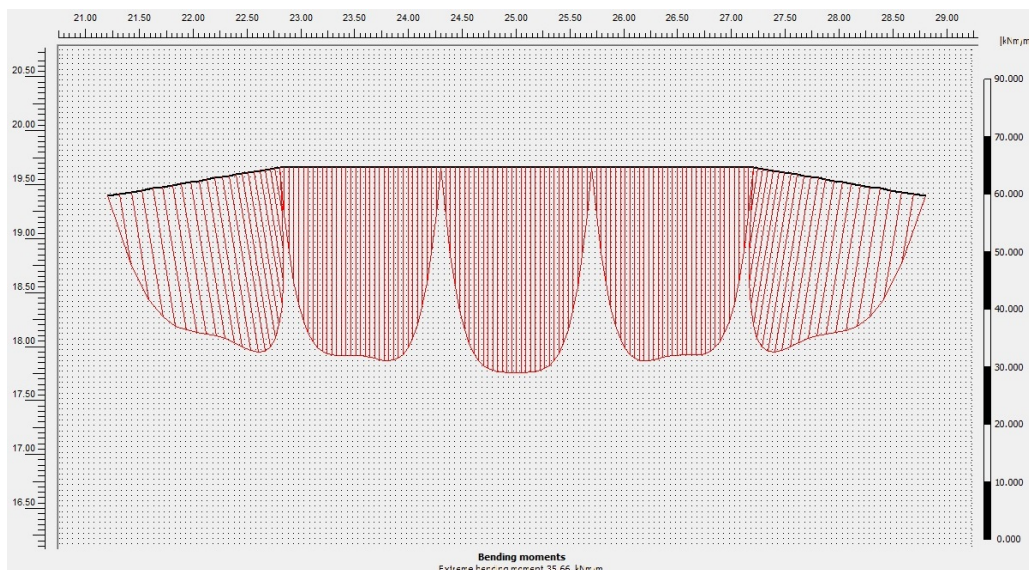


Figure 10.13: Bending moment diagram of the central timber plank after construction (Max: 35.66kNm/m, Legend unit: 10kNm/m)

The maximum bending moment of the plank was $35.66\text{kNm/m} = 8.02\text{kNm}$. The bending moment diagram along the span was not a perfect parabola, proving that the timber did not experience a uniformly distributed load. Since the load is not uniform, the pressure profile on the plank could not be worked backwards from the bending moment diagram.

The bending of moment diagram had steepest gradient at the ends of the planks, and a small increase in the mid-span, showing that the loads were concreted at the ends of each plank. However, this bending moment diagram is not accurate enough to model the bending moment on the actual plank. The supports used in the model were point anchors with no width dimension. To look at the load on the plank, the total vertical stress of the soil immediately above the central plank was plotted (Figure 10.14).

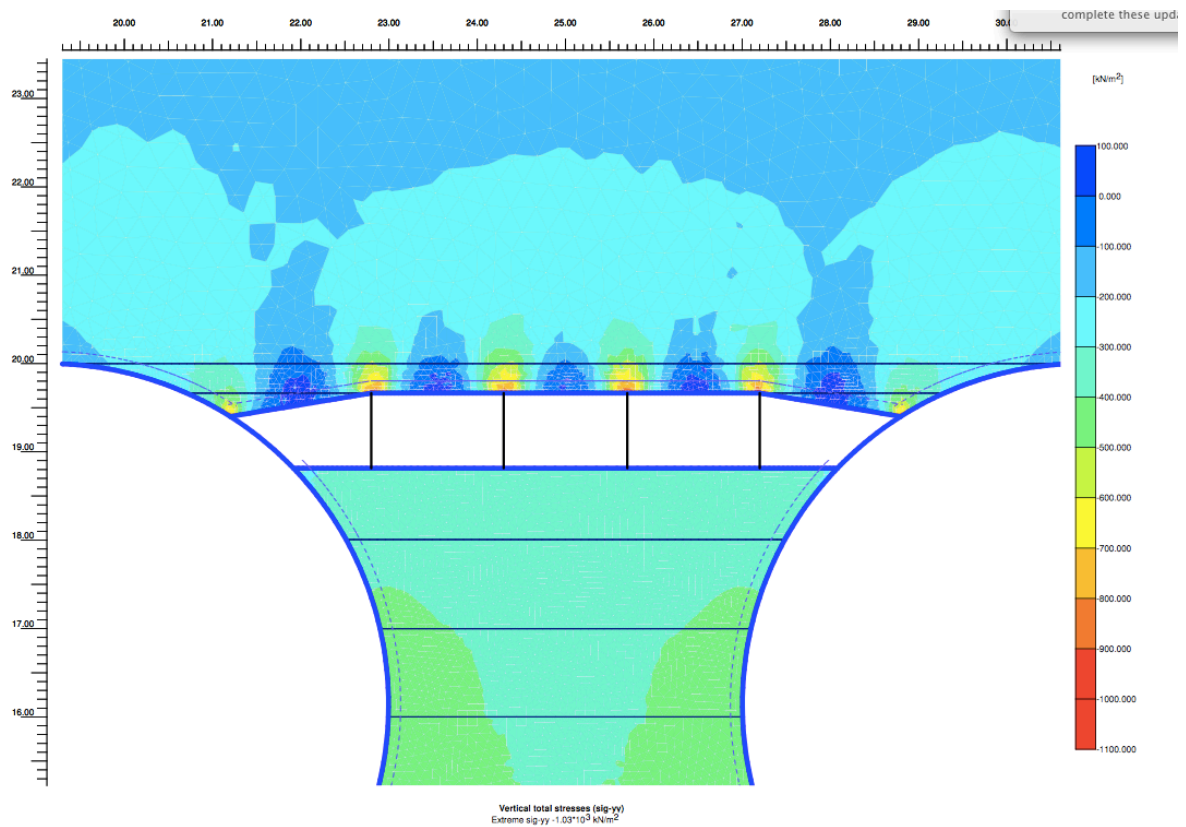


Figure 10.14: Total vertical stresses in soil after the construction of the temporary works (Scale: -1100 to 100kPa)

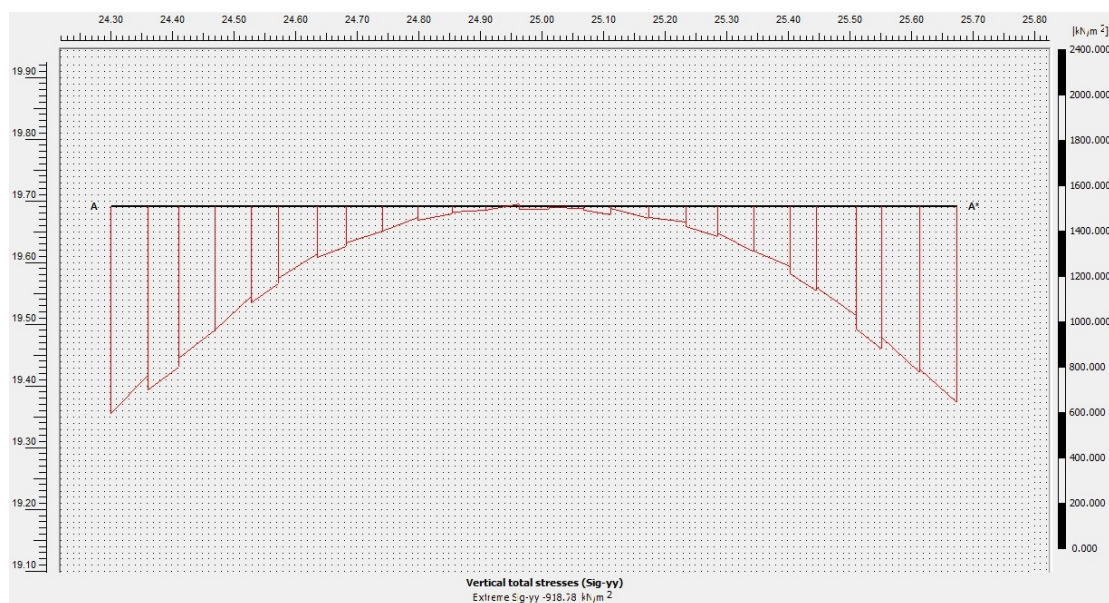


Figure 10.15: Total vertical stress profile on the central plank (Max: 918.78kPa, Legend unit: 200kPa)

The stress profile indicated very large pressure acting on the two ends of the span of the plank. The maximum pressure was 918.78kPa, which was a lot greater than the full overburden pressure of 81.7kPa. However, the pressure decreased drastically at the mid-span to be nearly zero. This showed a load concentration at the two ends at the supports, and the mid-span was barely taking any load. This could be explained by the phenomenon of arching (Session 5.1).

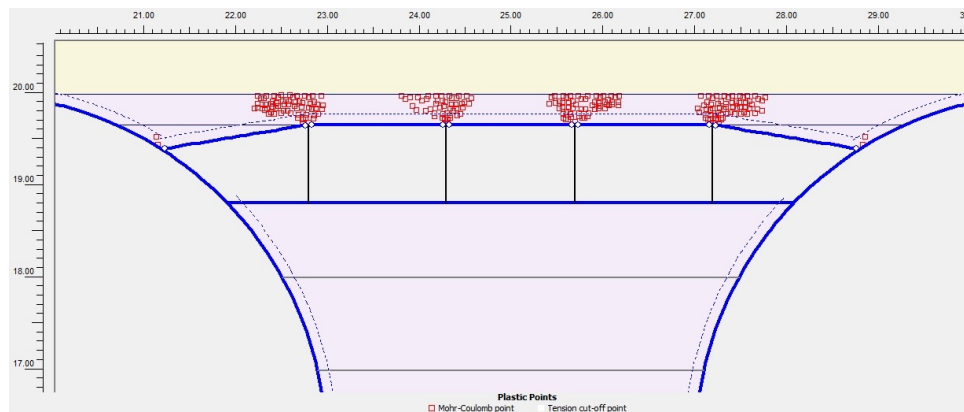


Figure 10.16: Plastic points (red) during the phase of construction of the temporary work

Since the mid-span of the plank deformed more relative to its supports. The soil immediately above the centre of the plank moved while that above the supports was stationary. Arching happened and the pressure was transferred from the centre to the supports. The supports in the model were point anchors. The needle beams at the construction site has a width of 0.147m which was 10.5% of the total span of the plank. A large percentage of the concentrated load was only taken by the supports. The actual load of the timber planks were a lot smaller than the full overburden pressure, and even the design load of 81.7 kPa. The Figure 10.16 shows the plastic points in the soil. The plastic points were also concentrated at the supports, showing that the shear stresses in the soil there were higher and soil has exceeded the Mohr-Coulomb strength criterion.

This model also showed that the load on the plank was smaller than the design load due to effect of arching. Arching was solely withstood by the shear resistance of the soil. The soil could be held by its shear strength with the arching mechanism. Since the temporary work only withstood the load for a relatively short period of time, it can be concluded that the shear strength would not have a significant change due to drainage. The vertical pressure acting on the mid span of the plank was zero. Theoretical, the soil arch can hold the soil without the span of the plank, providing a sufficient width of the

support. However, any extra movement of the support or vibrations of the soil would cause this strength to decrease. Therefore, the planks are still needed, especially for this site, since the frequently traffic within the existing tunnels would cause vibrations.

10.3.4 Final Phase – Completed Excavation

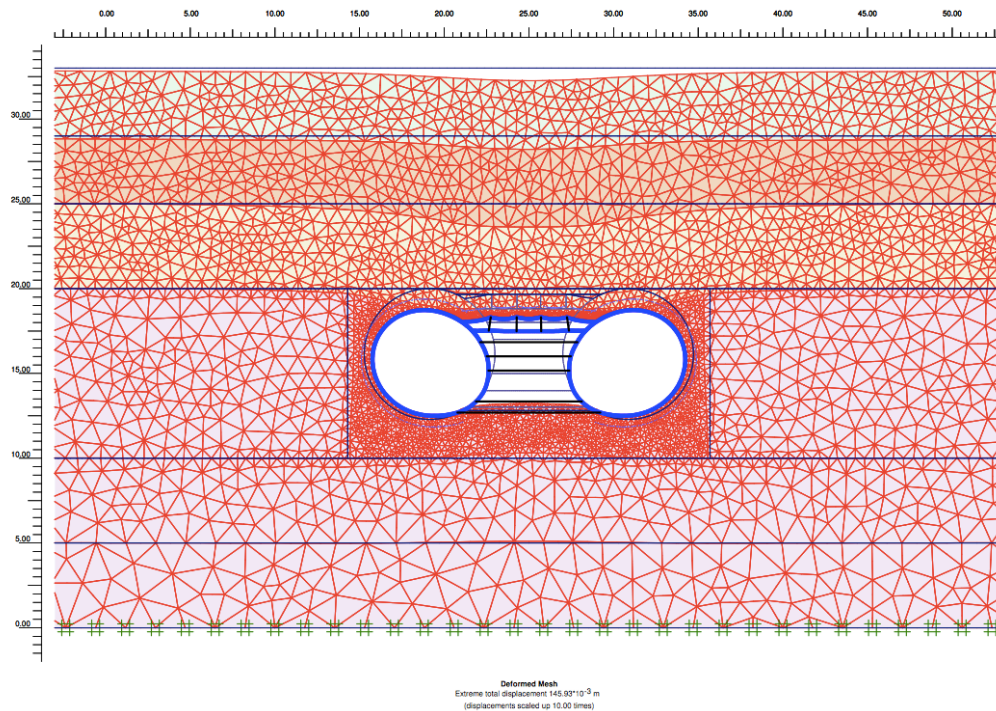


Figure 10.17: Deformed mesh after the completion of excavation

The excavation was carried out in layers of 1m, same as the actual excavation process at the site. After the whole cross passage was excavated, the deformed mesh was in Figure 10.17. The vertical stress profile on the planks still followed the same shape but with reduced magnitude. After the completion of the whole excavation process, all five planks had deformed more, including the supports relative to the tunnel linings, it was suggested that a larger scale of arching happened and load was transferred to the tunnel linings during the excavation process. The bending moment of the tunnel linings increased by 192 kNm/m through the excavation stages.

It was observed that the tunnels tended to move towards the empty space between them as expected. The horizontal movements were restrained by the temporary props and would then be prevented by the spraying of concrete. The displacement and bending moments on the tunnel linings of the tunnel on the left were plotted.

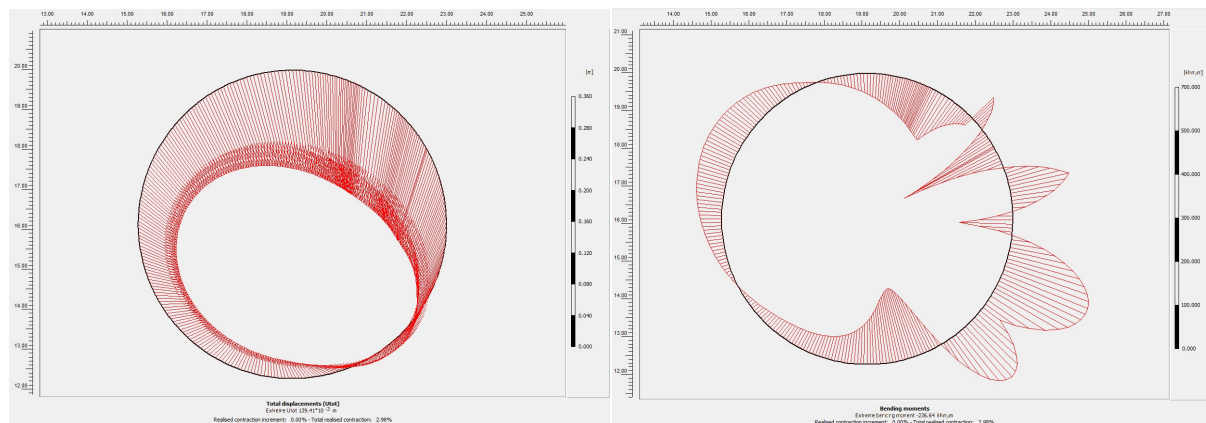


Figure 10.18: a) Left - Total displacement profile of tunnel lining after construction (Max: 139.4mm, legend unit: 40mm); b) Right – Bending moment diagram of tunnel lining after construction (Max: -236.64kNm/m, legend unit: 100kNm/m)

10.4 Comparison between Different Conditions

The following Table summarises the loads on the planks of various conditions modelled:

- 1) Less stiff supports
- 2) Rigid tunnel linings
- 3) 1% initial contraction percentage of tunnelling

(only one parameter was changed each time compared with the initial model)

	Real parameters	Less stiff supports	Rigid tunnel linings	1% contraction
<u>After construction of temporary work</u>				
Max. vertical load on plank[at supports] (kPa)	918.78	915.02	895.61	1000
Vertical load at mid-span on plank (kPa)	6.34	6.31	~0	6.97
Max bending moment on plank [at mid-span] (kNm)	8.02	7.95	7.93	7.61
Max bending moment on tunnel lining (kNm/m)	44.65	48.44	174.58	23.81
<u>After completion of excavation</u>				
Max. vertical load on plank [at supports] (kPa)	879.43	870.20	894.28	989.04
Vertical load at mid-span on plank (kPa)	6.11	~0	~0	6.96
Max bending moment on plank [at mid-span] (kNm)	7.90	7.81	7.97	7.48
Max bending moment on tunnel lining (kNm/m)	236.64	236.51	964.80	933.22

Table 10.3: Finite element modelling results on timber planks and tunnel linings at difference stages of excavation and with various conditions

The vertical total stress profiles along the plank had the same shape in all cases. There were only differences in the magnitudes. The pressures on the planks were always about zero at mid-span. The load on the planks decreased as the excavation proceeded.

It was observed that the relative stiffness between structures had a significant effect on the distribution of the vertical pressure. A stiffer material attracted more load. As stiffer structures deform less, the soil surrounding it stayed relatively stationary. By arching, the load was transferred to stiffer structures. From Table 10.3 above, the load transferred to the tunnel linings from the planks and the supports of the planks. This was shown in both cases of less stiff supports and rigid linings, and was exceptionally significant in the later case. The bending moment in the linings increased in a great magnitude.

Significant change was observed in the case of 1% contraction. With a small contraction percentage, the ground surface level rose instead of having a settlement trough as in other cases after the tunnelling phase. This was due to the buoyancy effect of the tunnel. A settlement profile was only given after the final stage of excavation. But there was not much changes in the vertical stress on the planks. But with this contraction percentage, the bending moment on the tunnel linings had a huge difference; more models should be conducted with different percentage values to further investigate its effect.

11. Future works and improvements

11.1 General

The conclusions and observations from this project were made according to only one data set and one simulation model. The findings are not reliable enough for only one temporary structure was studied.

The basic equipment is now developed. More deployments should be carried out in the future on structures similar to the one investigated in the project and also structure with completely different designs. This also applies to the finite element modelling part. It is needed to seek more opportunities of deployments and more structures should be studied. Experiments on effect of soil arching should be carried out if possible.

11.2 Improvement of Sensor Nodes

The sensor nodes used in this experiment were fragile. They were damaged physically and by oxidation. The sensors nodes have to be protected by waterproof materials in future deployments. The instrumented planks should be stored off-site in dry areas before construction if possible. Moreover, the battery size has to be increased to achieve

a longer data logging time. Lastly, the circuit on the PCB board consists of six voltage regulators for ensuring constant supply of power level to the sensor circuit. Each sensor was connected one voltage regulator and there were two more in the circuit. The voltage regulator consumes a significant amount of power. The circuit can be modified to contain less voltage regulators to extend the battery life. Lastly, the sampling rate can be reduced at a considerable period of time after the installation of the plank.

11.3 Earlier Planning with Contractor

For this project, the construction of Pal 16/18, including the excavation and construction of the timber temporary works, was on the critical path of the contractor's schedule. This limited the available working time on site. Future deployments should be planned earlier and together with the contractor, so that the best way and location of deployment can be decided. A problem in the project was that sensors were broken during the construction. In the future, with better cooperation with the contractor, the student can put the plank in place. If this is not possible, the installation can be done under the supervision of appropriate personnel who knows details about the project.

11.4 Modelling

The modelling was done using computers in geotechnical lab in CUED. The memory sizes of these computers were limited. It was not possible to have too many elements in the model, not to mention elements with higher order. Future modelling can be conducted using computers with a larger memory size to obtain a more accurate result. Apart from that, there are limitations on the available functions of Plaxis. For example, a support with finite width was not possible. Apart from that, the tunnel linings modelled by Plaxis are defaulted to be impermeable. In reality, some linings are permeable which allows the drainage of water through them. Plaxis is not capable of doing this type of drainage problems. Other software can be used together with Plaxis in the future for comparison.

12. Conclusion

The loads on underground timber structures are not fully understood and are often over estimated. The structures are designed to be large in size which results from huge factors of safety caused by the uncertainties on load estimations. The design methods

are not based on well-developed studies. Even the Terzaghi's equations are empirical. This project aims to better understand the loads of these structures in order to have more efficient designs in the future. This helps to reduce safety risks, the cost and time of construction.

Experiment and finite element modelling were done to an underground timber temporary structure at Victoria Station. Both the sensor network used and the model generated were developed to best suit the purpose of the project's aim. The wireless sensor network, consisting of strain gauges and load cells as sensors, was developed in mechanical, electrical and software aspects. Various conditions of the environment were modelled using Plaxis.

Both the experimental data and the finite element analysis showed that the load on the timber planks was smaller than the design load of 81.7kPa. It was observed that soil arching plays a significant role in the behaviour of soil and thus the loads on underground structures. The load concentrated on stiffer structure with relatively small displacements. In the temporary work considered, the load was transferred from the mid-spans of the timber planks to the supports and to the tunnel linings. Experimental data gave a load of about 26.5kPa. The finite element analysis gave an approximately zero load at the mid-span planks and a concentrated load of ~900kPa at the supports. However, there is still no analytical method developed to estimate the arching behaviour.

The findings in this report were based on one set of experimental data and finite element model of only one structure. The conclusions made were only preliminary. The most important future work is to gather as much data as possible and from different types of underground temporary structures. This enables more analysis to be done to further confirm the findings in this project. Research and experiments to understand soil arching should also be conducted.

Lastly, valuable experiences were gained given the chance to work on site and learn about real construction works and methods. Construction works such as tunnelling, piles constructing, jet grouting, etc. were seen during the course of the project. The basic finite element modelling procedures and possible pitfalls were also learnt.