NATM, EPBM and Cut and Cover Tunnelling Applications in the Project of Izmir Met

By Basar Arioglu, Ali Yuksel, Sezgin Kurtuldu and Ergin Arioglu

The Izmir Metro System is planned as a high capacity system covering the whole urban area according to the master transportation plan. It will consist of 45 km of double track in total when completed. The first stage of 11.4 km including backbone system between Ucyol and Halkapinar, workshop, depot area and Bornova branch (Figure 1) were successfully constructed by Yapı Merkezi-Adtranz Consortium in less than four years. Due to the complex soil conditions, varying overburden and dense urbanization, three different tunnelling techniques were applied between Ucyol and Basmane Stations.

Nenehatun – NATM tunnel

The section of Izmir LRTS located between Ucyol and Konak Stations was constructed as deep tunnel because of the elevation difference of 70 m and the high density of the buildings on the alignment.

Bahribaba and Yeşilyurt shafts, 12 and 32 m deep respectively, provided the access to tunnel entries. Two switch tunnels for Buca junction were located between the Yeşilyurt access and the Ucyol station tunnel. The Station Tunnel was situated between 291th to 440th metres of alignment. Inclined escalator tunnels provided the connection of the station tunnel with the ticket hall. A double track tunnel is constructed at the remaining 1,384.7 m (Table 1).

Geology – geomechanical properties of rocks

Upper Miocene aged sedimentary and volcanic rocks are encountered at Nenehatun tunnel alignment (Figure 2). The Andesites are moderately to highly weathered, slightly to highly jointed, between weak to strong. Cooling joints are smooth and have clay fillings while tectonic joints have rough surfaces. Sedimentary rocks are composed of interbeddings of claystone, sandstone, and conglomerate. They are classified as weak rock.

Geological mapping was performed and geomechanical index properties like RQD, joint set number, condition of joint surfaces, condition of weathering of the rock at the face, were determined every 5 to 10 m along the tunnel alignment. Variation of RQD and Q values are presented in Figure 2.
Crushed zones that are 1 to 3 m in width intersect nearly perpendicular to tunnel axis in a distance between 20 to 50 m and dip in direction of the advance. Rock is completely weathered in these zones and overbreak occurred during the excavation at the roof of the tunnel due to the tunnel water.

The section of the rock where the Ucyol Station is located is heavily jointed and heavily weathered. This caused some overbreak and created difficulties in the installation of the rock bolts.

At the end of the Ucyol Station grey, medium to slightly jointed, high strength andesite (UCS = 100 to 120 MPa) was encountered. It was difficult to excavate this section by road header. Pick consumption increased by 33% compared to the average value. The vibrations during the excavation caused serious problems on the gearbox of the cutterhead. For this reason, a hydraulic hammer OK RH9 instead of Roadheader was used for the excavation.

Between 850 to 1130 m sedimentary rocks composed of the interbeddings of claystone, sandstone and conglomerate were encountered. During the excavation of these slightly to medium cemented and low strength sedimentary rocks, there was no difficulty of excavating but due to the movements of the road header and due to the seepage water, the rock at the bottom of the tunnel was disturbed and loss of strength occurred. Therefore the handling of the equipment became difficult. As the convergence of the tunnel increased, it was necessary to support the bottom with an invert.

A length of 150 m of the tunnel until Bahribaba exit was under sea level. In this section the andesite was medium to highly weathered, medium to closely jointed and with clay fillings and moderate to high strength.

**Excavation and support**

The excavation and supporting operations were designed according to New Austrian Tunnelling Method (5). The variations in the geology and deformations (convergence and surface settlement) were monitored by continuous geological mapping and surveying.

Excavation was done in a top heading and bench sequence. After two rounds of excavation at the top heading, the excavation of the bench started.

Under favourable ground conditions type B support was applied allowing 3 to 6 m bench length. It included 20 cm reinforced shotcrete, steel lattice girders with 1 to 1.3 m intervals, seven to eight rock bolts with 3.85 m length each. Under unfavourable conditions type C support was executed with a bench length of 2 to 4 m. It included 25 cm shotcrete with a double layer of steel wire mesh, steel lattice girders with 0.8 to 1 m intervals, and 11 to 12 rock bolts per round. As the geological conditions varied frequently
and the alignment was located in an urban area, type A support (15 cm shotcrete, no lattice girders) proposed during the design phase, was not used. The maximum settlement observed on the 53 deformation points was 9 mm and no damage was recorded (3).

The enlarged section of the first switch tunnel was divided into left and right faces. In the second switch tunnel, the excavation method was changed to the method used in the double track tunnel, by observing convergence measurements and the evaluation of the advance rate. The advance rate was increased to 1.07 and 1.22 m/d in the second and the third tunnel, respectively while 0.78 m/d were achieved in the first switch tunnel. Therefore there was a gain of 136 days in total excavation time and 65 000 man-hours, where saved compared to the method applied in the first switch tunnel (4).

After the shafts were completed, tunnel excavation started at Üçyol access on 26. December 1996 and at Bahri Baba access on 3. June 1997. The excavation was completed when the faces met each other on 28. August 1998 – 808 m from the Üçyol Access (3).

Both from Üçyol and from Bahribaba, the advance rate of the first 100 m was realized with an average of 1.5 m/d. Due to the learning effect and the better organisation in the loading and transport operations, the production was increased and very high rates up to 5.2 m/d was reached (Table 2). Considering working days, the average rate in the double truck tunnel was determined as 2.6 m/d at each face (2, 4).

The amount of labour in the double truck tunnel and enlarged tunnel were 14.5 and 17.2 man-hours/m³, respectively. As a result of the evaluation of the different tasks of tunnel excavation and support operations, it was concluded that the necessary time for one excavation and support cycle (i.e. 0.8 to 1.3 m) was 670 minutes: 29 % was for excavation, 27 % for shotcrete, 13 % for rock bolt, 11 % for wire mesh 10 % for steel girder, and 9 % for standstill and breakdown respectively (2, 4).

**Ummuhan Ana – EPBM twin tunnels**

Due to the density of the buildings, main traffic lines of the city and the high costs for the relocation of the infrastructure facilities located between Konak, Cankaya and Basmane Stations and in order to prevent damage to the archaeological remains located in the recent fills, it was decided that this section should be constructed as a tunnel (6).

As the alignment included a small radius (R = 250 m) and as the neighbour and adjacent buildings had pile foundations and/or deep basements (maximum 5 m), a twin tunnel system was selected. Tunnels consisted of four tubes of a total length of 2 753 m each having a 6.5 m exca-
vation diameter with 5.7 m inside diameter.

The section between Basmane and Cankaya Stations is located under a street where two to three storey historical masonry buildings and six to eight storey modern buildings are situated at each side. The section between Konak and Cankaya Stations is underpassing buildings. The depth of the tunnel which was 6 to 7 m under the street, reached down to 13 m under the buildings to avoid basements.

Geology – geomechanical properties of layers
In Ummuhan Ana Tunnel route clays, completely weathered andesite and recent sea sediments, composed of silt, clayey silt and sandy gravels were encountered (Figure 3). They were classified as silt and clayey silt having cross bedded sand layers and sand pockets between Konak and Cankaya Stations. In the beginning of the tunnel from the Konak access, the tunnel passed through silt layers. Yellow clays were encountered on lower parts of the face with increasing depth of the tunnel, around the midway of the Konak and Cankaya alignment. The tunnel route passed through the sandy gravel and silt layers between Basmane and Cankaya stations. Water level is approximately 1.5 m below surface at Konak and 2.5 to 4 m and 5 to 6 m at around the Cankaya and Basmane Stations respectively.

EPB tunnelling machine
Due to the difficult geological conditions and the buildings situated in the neighbourhood of the alignment, during the feasibility stage it was decided to excavate the tunnel by equipment that was capable to excavate the tunnel full face. An EPB shield was selected (6, 7). The shield diameter was 6.54 m and the diameter of the cutter head was 6.56 m. The total length of the shield was 7.3 m and its weight is 325 t. A screw conveyor attached to the excavation chamber removed the excavated material from the front. For continuous monitoring of the face pressure, sensors were placed in the excavation chamber and the screw conveyor. For ensuring the face stability it was necessary to provide the correct face pressure and control the unloading of the excavation chamber. Soil conditioning was done to achieve suitable plasticity, lower internal friction, lower permeability and soft consistency of the soil (8). Soil conditioning improved soil properties thus it was possible to safely obtain the face stability. In addition to this, the required torque of the cutter head was reduced and the wearing of the screw conveyor decreased.

The conditioning of the gravely sandy soils was carried out by foam, which was composed of environmental friendly polymers and surfactants, and that of clayey, silty soils by bentonite slurry. The consumption of the foam per unit volume of the excavated material was 300 to 500 l/m³, 0.01 to 0.5 kg/m³ and 0.5 to 1 kg/m³ of
polymers and surfactants respectively. (9).

**Excavation with EPB machine**

Excavated material from the face is passed through the openings on the cutter head to the excavation chamber. By removing the excavated material from the excavation chamber, the machine pushes itself ahead by a total 28 jacks with a capacity of maximum 44 300 kN. In the same time, the void (≈12 cm thickness) between the segments and the excavated ground was immediately filled by grout. Immediate filling of this void and the controlled take of the excavated material minimized ground settlements. Bentonite was added to provide flowability and fly ash was used to safe cement. Natural sand (0 to 5 mm) was used as aggregate in the mix.

After the completion of one advance step (120 cm), the segments (seven plus one key-stone) were placed by an erector, equipped with a vacuum device. The Segments are 120 cm long and 30 cm thick. High strength concrete (BS 45) was used. Elastomeric gaskets were used to ensure water tightness.

The excavated material was transferred to belt conveyor and muck was carried with muck wagons pulled by diesel locomotives. Two service trains consisting of six muck cars, three segment cars and one grout unit were used. The TBM was guided with a single laser guidance system that was fully computer controlled.

**Excavation progress**

The TBM, which was specially designed and produced by Herrenknecht for the Izmir Metro Project, arrived on site in May 1997. Preparation of the access shaft at Basmane Station, assembling of parts and installation of compressed air, water, and cooling units took four months. Excavation of drive 1 and drive 2 were executed from Basmane Station to Cankaya Station, while drive 3 and drive 4 were executed from Konak Station to Cankaya Station.

The first ring was placed at Basmane Station on 25. August 1997. Advance rates were realized around 3 m/d during the first month. Excavation was done in one shift in the beginning. In this period the crews became more skilled, working systems were established and clogging problems of grout were solved by improvements of the grout mix. Then the daily advance rates gradually increased and reached up to 24 m/d. Average advance rate was 8 m/d for this drive.

The shield parts of the EPB machine were dismantled at Cankaya Station and transferred to Basmane Station and again assembled in two months. All TBM equipment and service units were moved to Konak Station area for the excavation of drives 3 and 4. The average advance rate was 12.5, 13.3 and 19.7 m/d for drive 2, 3 and 4 respectively. The maximum advance rate achieved was 30 m/d during the excavation of drive 4.
Unacceptable wearing of the screw conveyor was found during the assembling at Konak station before the excavation of Drive 3. It was considered that sandy andesitic gravels on the alignment of Drive 1 and 2 caused the excessive wear.

**Excavation performance of EPB machine**

The distributions of the average process duration are shown in Figure 4. The intermediate demobilization periods are not considered and only the time intervals from the beginning to the end of the excavation are analysed.

As shown in Figure 4, unproductive time (waiting and breakdowns) made up 37% of the tunnel excavation period and only 49% of it was directly related with production activities, which can be considered as "machine efficiency". In the beginning, the mentioned value was in a range of 30% and then increased to the level of 69% at the last drive of the excavation. This is totally related to the increasing harmony between the crews, equipment and the mastery of the craft. On the other hand, another indication showing the increasing mastery of the craft is the shortening in the preparation period.

Same results were also noticed while considering the duration for the placement of the rings. Placement of a ring was reduced from 267 minutes at the first drive to 88 minutes at the last drive and the average was 161 minutes.

**Settlements and face stability**

Before starting the excavation, measurement stations were designed and three to five settlement points were installed on ground and in the buildings. The distance between the measurement stations was 30 to 100 m. Totally, 85 units were surface points while 317 units were installed at buildings. In addition vertical inclinometers were installed at eight measurement stations to measure lateral displacements. On the other hand, the existing condition of all buildings located in the area affected by the tunnel was determined photographically and were recorded with sketches before the excavation. Crack meters also were installed on two important historical buildings. Totally, 5,040 deformation readings were carried out during the excavation. Maximum settlement value was recorded as 21 mm.

Volume loss of ground (K) for unit advance of tunnel caused by surface settlement can be expressed with following formula (12, 10):

\[
K = \frac{\Delta V}{V} = \frac{2.5 \cdot i \cdot S_{\text{max}}}{\pi \cdot \frac{D^2}{4}}
\]

where

- \(i\) distance between inflexion point of settlement curve and tunnel axis. It is possible to...
determine empirically for silty clays and clays \( i = 0.5 \cdot (H + D/2) \) (13, 10).

\[ S_{\text{max}} \]  
maximum settlement value at the axis of tunnel,

\( D \)  
Diameter of tunnel, and

\( H \)  
Depth of tunnel axis.

Volume loss of ground for the maximum value can be calculated as:

\[ i = 0.5 \cdot (8.4 \text{ m } + \frac{6.5 \text{ m}}{2}) = 5.8 \text{ m} \]

\[ K = \frac{2.5 \cdot 5.8 \text{ m} \cdot 0.021 \text{ m}}{\frac{3.14}{4} \cdot (6.5)^2} = 0.0092 = 0.9\% \]

Considering average values \( (S_{\text{max}} = 7 \text{ mm}, \ H = 9.62 \text{ m}) \) volume loss can be calculated as:

\[ K = \frac{2.5 \cdot 8.6 \text{ m} \cdot 0.007 \text{ m}}{\frac{3.14}{4} \cdot (6.5)^2} = 0.0045 = 0.45\% \]

It is clear that the ground loss value \( (K) \) is extremely related to the applied tunnel technology. The mentioned values were between 0.77 % and 1.32 % at Cairo Metro in Nile alluvial fine sands, where a 9.45 m shield diameter EPB machine was used (14).

Soil stability number, \( N \) as a criterion of face stability is given below (10, 12)

\[ N = \frac{\sigma_s + \gamma \cdot Z_o - \sigma_F}{C_u} \]

where

\( \sigma_s \)  
additional loads caused by traffic or buildings; it was assumed as average \( \sigma_s = 10 \text{ kPa} \),

\( \gamma \)  
Unit weight of soil, from the laboratory tests it was between \( \gamma = 17 \text{ to } 21 \text{ kN/m}^3 \), assumed average as \( 18 \text{ kN/m}^3 \) (15),

\( Z_o \)  
depth of tunnel axis, \( Z_o = (H+D/2) \),

\( \sigma_F \)  
Face pressure at the tunnel axis level; it was applied between \( \sigma_F = 160 \text{ to } 240 \text{ kPa} \) depending on soil type and tunnel depth (11),

\( C_u \)  
Undrained cohesion of soils; it was found between \( C_u = 30 \text{ to } 80 \text{ kPa} \) in laboratory tests (15).

The stability number can be calculated from the average values as follows:

\[ N = \frac{10 \text{ kPa} + 18 \text{ kN/m}^3 \cdot 12.8 \text{ m} - 200 \text{ kPa}}{50 \text{ kPa}} = 0.8 \]

This value is indicating “small movements – elastic conditions” (10).

An inverse correlation was found between face pressure values and settlements (Figure 5). Above calculations and mentioned correlations show that there is a harmony between the design pressures and the applied values, and the settle-
ments were successfully controlled.

**Cut and cover tunnels**

Cut and cover tunnels, one of the most significant structures in the Izmir Metro system, were designed to meet very unstable and heterogeneous soil conditions. No particularly new construction methods of constructing these tunnels or of handling the material were used, but some features of the work were of interest considering the difficult and uncertain nature of the encountered geological conditions. A mixed formation that included a wide variety of layers like gravelly sands to sandy silts to silty clays and partially disintegrated material, tending to cause slips and slides were encountered. Near the Konak and Çankaya stations where a lot of land was claimed from the sea through the centuries, the level of artificial fill was very deep, ranging up to 6 m (16, 17). Making an open cut through this kind of material under a street carrying heavy traffic, and with important buildings (two to nine story buildings were as close as 2 m to the diaphragm walls) close to the work on both sides, required the most careful excavation methods.

**Summary of Case Histories**

The structures built in open cut sections reasonably could be subdivided into the track tunnels, the station tunnels and siding and turning units. In these connections the great variety of soils, the high ground water level, the consideration of existing compulsory points and the safety of the built up area made high demands on the realisation. The total length of the Izmir Metro alignment between Konak and Basmane in the cut and cover tunnels is approximately 1 250 m.

In the downtown area, three underground stations Konak, Çankaya and Basmane were excavated using the open cut and cover method. The depth of the excavation was ranging from 15 to 18 m in Konak and Çankaya to 12 to 16 m in Basmane. Due to the very high ground water level and very soft ground conditions, various types of structural systems for the stations were considered for the optimization of the design and construction. Both the conventional reinforced concrete inner framing and diaphragm walls were adopted in the final design. A typical structural system chosen for these three stations is shown in Figure 6. Diaphragm walls with a thickness of 0.8 to 1.2 m and a depth ranging from 25 to 33 m were built first as barriers to fight the ground water. The excavation was carried out in three or four stages to allow placing of steel braces. The tunnel inner frames built in the dry work area consisted of a cast in-place concrete U-structure with prestressed precast beams forming the ceiling. In order to prevent a probable water flow through the complete construction, transgression grooves were provided in the top floor of the stations, which allowed a

![Fig. 6 Typical cross-section of cut and cover structures. Bild 6 Regelquerschnitt für die offene Bauweise.](image-url)
transgression in case of high water.

**Evaluation of design assumptions and site measurements**

The initial design of the internally supported diaphragm wall system was based on the assumption of triangular distribution of horizontal earth pressure up to the second excavation stage (Rankine), and an equivalent uniform earth pressure distribution was accepted in further excavation stages where the strut rows increased (18). Additionally the hydrostatic water pressure and the effects of surcharge loads resulting from buildings, traffic or other surface loads which were located above the active sliding plane (active wedge) were taken into consideration. Structural analysis was carried out for every construction stage, such as excavation, pouring concrete to inner frame, strut placing and removing (19).

In each station, four chosen panels of braced diaphragm wall, supported by internal pipe struts were instrumented so that both surface and internal displacements and loads could be monitored during and after excavation (20). These panels were instrumented with load cells and slope inclinometers. In addition piezometers and well points for observation of the water pressure on the wall were installed. The inclinometer readings provided a direct measure of the slope of the wall at any elevation and load cells readings provided a direct measure of axial force of the steel struts.

The structural system behaviour was understood more clearly with the comparison of the measured values with the calculated ones. A significant difference was found between the measured and calculated strut forces. Although greater forces were expected at the lower layers, the measurements showed that greater forces occurred at the upper rows. These observations led to a reconsideration of the assumptions made at the design stage concerning the structural behaviour of the support system. The related evaluations are summarised below.

According to the standard analysis procedures used, each excavation stage was considered as a separate loading case during analysis and design. After analysing the system for each excavation stage, the most unsuitable strut forces are taken as design forces. In this practice, greater internal forces are obtained (as the applied external forces are the determining factor) at the lower strut layers.

When the behaviour of structural system is observed, it can be seen that, treating each excavation stage as an independent load case does not represent the true behaviour. Fundamentally, every strut supported excavation stage can be achieved by excavating the next level from the previous one. It can be assumed that the recently placed strut can resist almost all of the additional excavation loads. Displacements and strut
forces should be calculated by this manner for every excavation stage, and then should be superimposed. This analysis procedure can reflect the excavation and strut placing stages more realistically. The analysis sequences and the results found by this modified method are shown in Figure 7. Measured values for Konak Station struts show an accordance with results calculated by this modified method, which considers consecutive stages. Estimated results are given in Table 3.

**Discussion**

Should these evaluations, concerning the calculations of strut forces, suggest that standard calculation methods provide unreliable results? Strut forces are related to soil deformations. According to these deformations, there should be a “redistribution” of the strut forces over the long term and this would cause much greater loads at lower struts progressively. In case of cut and cover construction, casting the base slab immediately after reaching the excavation base, fixes lateral deformations. As a result of this no load increase it observed at lower layer struts.

Another result, reached by comparison of calculated and measured strut forces, is a risk of designing on the unsafe side of the upper struts, by using forces obtained from the standard method. It will be useful to consider this point in design tolerances (21).

Both the design and the construction stages of the cut and cover structures within the scope of the Izmir Metro System involved rich experience from the civil engineering point of view. Progress of the design and construction stages in parallel and following the structural behaviour with continuous observations and measurements enabled the evaluation and verification of the analysis. On the other hand, solving both unexpected problems arising from difficult soil conditions and construction detail problems encountered at the site let the engineers on the site and in the design office gain important experiences.

**Conclusions**

Tunnelling through dense urban settings is undoubtedly difficult. When complex and changing soil conditions, archaeological and various legal/political issues are also present, the challenges are almost insurmountable. The Izmir Metro Project was realised under the most difficult conditions and a wide range of techniques had to be applied. Due to the design-build nature of the contract, design and construction were carried out almost simultaneously and valuable experience was gained. Without excessive and costly delays in overall delivery of the metro system.

**References**

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**Authors**


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### Table 1 Parameters of Nenehatun Tunnel.

**Tabelle 1 Daten des Nenehatun Tunnels.**

<table>
<thead>
<tr>
<th>Tunnel Type</th>
<th>Dimensions (L x H) [m]</th>
<th>Excavated Area, [m²]</th>
<th>Length [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double Track</td>
<td>9.7 x 7.10</td>
<td>65</td>
<td>1.384.7</td>
</tr>
<tr>
<td>Enlarged Section</td>
<td>16.5 x 9.7</td>
<td>146</td>
<td>141.58</td>
</tr>
<tr>
<td>(Switch Tunnel I, II)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Station Tunnel</td>
<td>16.09 x 9.13</td>
<td>139</td>
<td>149</td>
</tr>
<tr>
<td>Buca Connection Tunnels (Single Track)</td>
<td>6.9 x 6.2</td>
<td>39</td>
<td>76.9</td>
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<tr>
<td>Escalator Tunnels</td>
<td>9.29 x 6.55</td>
<td>57</td>
<td>357.68</td>
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<tr>
<td>Total</td>
<td>-</td>
<td>-</td>
<td>2107.96</td>
</tr>
</tbody>
</table>

### Table 2 Daily advance rates of tunnels.

**Tabelle 2 Vortriebsleistungen.**

<table>
<thead>
<tr>
<th>Tunnel Type</th>
<th>Location</th>
<th>Average (working days) [m/d·face]</th>
<th>Average (calendar days) [m/d·face]</th>
<th>Maximum [m/d·face]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Double Track</td>
<td>Üçol Side</td>
<td>2.62</td>
<td>1.72</td>
<td>5.2</td>
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<tr>
<td></td>
<td>Bahribaba Side</td>
<td>2.6</td>
<td>1.89</td>
<td>5.2</td>
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<tr>
<td>Enlarged Sections</td>
<td>1st Switch Tunnel</td>
<td>0.78</td>
<td>0.58</td>
<td>-</td>
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<tr>
<td></td>
<td>2nd Switch Tunnel</td>
<td>1.07</td>
<td>0.87</td>
<td>2.4</td>
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<td></td>
<td>Station Tunnel</td>
<td>1.22</td>
<td>0.97</td>
<td>2.4</td>
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<td>Single Track</td>
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<td>0.98</td>
<td>0.61</td>
<td>1.3</td>
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<td></td>
<td>Buca Connection left</td>
<td>1.46</td>
<td>1.17</td>
<td>2.6</td>
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</table>

### Table 3 Comparison of measured and calculated loads.

**Tabelle 3 Vergleich von gemessenen und berechneten Lasten.**

<table>
<thead>
<tr>
<th>Strut Row</th>
<th>Calculated Loads by Standard Method</th>
<th>Estimated Loads by Modified Method</th>
<th>In-situ Measured Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>225</td>
<td>145</td>
<td>120</td>
</tr>
<tr>
<td>Second</td>
<td>320</td>
<td>137</td>
<td>138</td>
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<tr>
<td>Third</td>
<td>405</td>
<td>38</td>
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### Table: Stations Dimensions

<table>
<thead>
<tr>
<th>STATIONS</th>
<th>A (cm)</th>
<th>B (cm)</th>
<th>Bshaft</th>
<th>H (cm)</th>
<th>d1 (cm)</th>
<th>d2 (cm)</th>
<th>t1 (cm)</th>
<th>t2 (cm)</th>
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</thead>
<tbody>
<tr>
<td>KONAK</td>
<td>1600</td>
<td>1670</td>
<td>2030</td>
<td>2400</td>
<td>100</td>
<td>80</td>
<td>85</td>
<td>125</td>
</tr>
<tr>
<td>CANKAYA</td>
<td>1800</td>
<td>1670</td>
<td>1975</td>
<td>2500</td>
<td>120</td>
<td>100</td>
<td>125</td>
<td>145</td>
</tr>
<tr>
<td>BASMANE</td>
<td>1600</td>
<td>1675</td>
<td>2030</td>
<td>2850</td>
<td>100</td>
<td>80</td>
<td>75</td>
<td>120</td>
</tr>
</tbody>
</table>

### Diagram: Existing Street

- Protective Concrete
- Insulation
- Cast-in-place Slab
- Precast Beam
- Invert
- Insulation
- Base Slab

### Excavation Stages

**Excavation Stage-I**

**Excavation Stage-II**

**Excavation Stage-III**

**Excavation Stage-IV**

**Excavation Stage-V**

### Estimated Final Strut Forces per Unit Length:

- First Row Permanent Strut Force: \( \Sigma R_1 = R_1^u + R_1^v + R_1^w = 36.30 \text{ ton/m} \)
- Second Row Permanent Strut Force: \( \Sigma R_2 = R_2^u + R_2^v = 34.25 \text{ ton/m} \)
- Third Row Permanent Strut Force: \( \Sigma R_3 = 9.50 \text{ ton/m} \)