ANALYSIS OF SUPPORT DESIGN PRACTICE AT ELMALIK PORTALS OF BOLU TUNNEL

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ANALYSIS OF SUPPORT DESIGN PRACTICE AT ELMALIK PORTALS OF BOLU TUNNEL

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ABSTRACT

ANALYSIS OF SUPPORT DESIGN PRACTICE
AT ELMALIK PORTALS OF BOLU TUNNEL

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A completed part of the Bolu Tunnel at Elmalık side collapsed during the 1999 Düzce earthquake. In order to by-pass the collapsed section, a new tunnel route was determined. 474 meters of the new route, including two portals and double tubing, crossed through the weak to very weak rock units with intersecting fault gouge, excavated from Elmalık side. In this study, the characteristics of the rock masses and support classes are determined for new route of the Elmalik Side. Then, during the tunnel excavation, the deformations of temporary and permanent support systems were precisely measured and recorded. The support system properties as determined from NATM were analyzed by two dimensional convergence confinement method using the numerical RocSupport software.

As a result of this study, for weak ground tunneling, duration of primary support installation should be kept at minimum. Besides that, temporary support measures such as forepoling, face sealing and temporary invert have an important role in controlling deformations before the primary support installation. With the application of temporary supports, loading on the permanent support, and hence the final deformation of the excavation, was found to be reduced significantly. Application of rigid lining was found to be necessary in order to prevent long-term deformations in weak ground tunnels, even though it is contradictory to the NATM philosophy.

Keywords: Bolu Tunnels, tunneling, NATM, weak rock, RockSupport

ÖZ

BOLU TÜNELİ ELMALIK AĞZINDA
TAHKİMAT TASARIMI UYGULAMASININ ANALİZİ

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Bolu Tüneli'nin Elmalık tarafında tamamlanmış olan bölümü, 1999 Düzce depreminde göçmüştür. Göçen kısmı geçmek için, yeni bir güzergah belirlenmiştir. Yeni güzergahın zayıf - çok zayıf kayaçlardan geçen ve fay kili ile kesişen, iki portal ve çift tübü içeren, 474 metresi Elmalık tarafından kazılmıştır. Bu çalışmada yeni güzergahın Elmalık tarafı için kaya kütlelerinin özellikleri ve tahkimat sınıfları belirlenmiştir. Ardından, tünel kazısı sırasında geçici ve kalıcı tahkimat üzerinde oluşan deformasyonlar hassas biçimde ölçülmüş ve kaydedilmiştir. Yeni Avusturya Tünel Açma Yöntemi'ne (YATAY) göre belirlenmiş olan tahkimat özellikleri sayısal RockSupport programıyla iki boyutlu deformasyon sınırlama (convergence confinement) yöntemiyle incelenmiştir.

Bu çalışmanın sonucuna göre, zayıf zemin tünelciliğinde, birincil tahkimatın yerleştilme süresi olabildiğince kısa tutulmalıdır. Ayrıca; süren, ayna serpmesi ve geçici taban gibi geçici tahkimat önlemlerinin kalıcı tahkimat yerleştirilmesinden önce oluşan deformasyonların kontrolünde önemli bir rolü vardır. Geçici tahkimat kullanılarak kalıcı tahkimatın yüklenmesinin ve dolayısı ile kazının nihai deformasyonun, önemi oranda azaltılabileceği bulunmuştur. Her ne kadar YATAY felsefesi ile çelişiyor da olsa, zayıf zemin tünellerinde uzun vadeli deformasyonların önlenmesi için rijit tahkimat uygulanması gerekli bulunmuştur.

Anahtar Kelimeler: Bolu Tünelleri, tünelcilik, NATM, YATAY, zayıf kayaç

To my family

ACKNOWLEDGEMENT

Some of the material contained in Chapter IV and Appendix I (i.e. rock description and properties, in-situ and laboratory test results) is mainly from a geologic report provided to Astaldi S.p.A. by O. Şimşek in the name of IC Consulenten Ziviltechniker GmbH entitled "Bolu Tunnel Re-alignment At Stretch 2F Tunnel Section Geological and Geotechnical Report Volume-1" Permission from Astaldi S.p.A. to reproduce this material is gratefully acknowledged.

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LIST OF ABBREVIATIONS

BSxx..................... Concrete Class
c............................ Cohesion
E_m .... Modulus of Deformability
G........................... Shear modulus
GSI .... Geological Strength Index
H........................... Overburden thickness
NAFZ .... North Anatolian Fault Zone
NATM .... New Austrian Tunneling Method
p_i........................... Required skin resistance
p_0 .... Initial pressure condition, in-situ stress
R........................... Radius of protective zone
r_o........................... Tunnel radius
SDA .... Self Drilling Anchor
TDR .... Tunnel Designers Representative
φ .... Internal friction angle
γ............................ Density of the rock, unit weight
σ_c .... Uniaxial compressive strength
σ_{cm} .... rock mass strength
υ .... Poisson's ratio

Terminology:

Chainage .... the location along a road from a start or reference point (in m)
Re-profiling.......... re-excavation of previously supported sections in order to compensate deformations beyond design limits
Crown Area.......... Tunnel roof and upper portions of the sidewalls
Overbreak............. Space created when ground break outside the required excavation profile

CHAPTER I

INTRODUCTION

Tunnels are man made subsurface passageways through obstacles like mountains, water bodies or dense urban or industrial areas. Tunnels are used in mining industry to provide access to mineral deposits. There are also civil tunnels like railway, road or pedestrian traffic tunnels or water, electricity or sewage tunnels, as well. Additionally, large underground rock chambers, which shares many design commons with tunnels, are used as military arsenal or nuclear waste disposal and as turbine chamber of a hydro-electric power plant.

1.1 Design and Support of Tunnels

It is inevitable to utilize some kind of ground support for a tunnel or any other underground opening if long-term stability is important and certain dimensions should be kept. Support measures vary with parameters like ground conditions, geotechnical properties of the rock or soil, overburden thickness etc. Likewise, different tunnels of similar ground conditions can utilize different support systems according to tunnel designer's preference.

Tunnel supports can be classified in two main groups according to construction order and purpose.

a) Temporary ground support:

Where the ground has insufficient stand-up time, that is to say, when there is a high risk of severe deformation even closure or collapse of the cavity before the installation of permanent support, to allow the construction of primary lining some temporary support like steel sets and shotcrete or rock bolts can be installed. Temporary supports are removed during the tunnel excavation or permanent support installation.

b) Permanent Ground Support:

May be divided into two groups:

i) Primary lining:

All of the support measures that applied in conjunction with tunnel advance are called as primary lining. Loads and deformations that the ground may induce is carried by primary lining.

ii) Secondary (or final) lining:

Some tunnels require smooth inner profiles for usage (e.g. sewer and water tunnels) or aesthetic (e.g. traffic tunnels) purposes. Corrosion and erosion protection of primary lining is done by secondary lining. If required, water proofing of tunnel is provided by secondary lining. As another approach secondary lining can build to take the duty of primary lining over, if the primary lining is assumed to be worn out in time.

Supports of mining and civil tunnels are different also. Civil tunnels are generally driven under relatively shallow overburden. As the projected usage life is usually longer than 100 years and the tunnel is designed for public use, high safety factor (>2.0) is applied to ensure long-term stability. Additionally for civil tunnels water proofing, fine inner surface, illumination and ventilation is important. (Nielsen, 2002)

Mining tunnels provide access to working face. Except main access tunnels and shafts, life span of a mining tunnel is short. Average working life of a tunnel used for mineral exploitation is around 10 years. Acceptable deformation rates are higher and repair works are easier compared to civil tunnels, lover safety factors (1.2-1.5) are applied. (Whittaker & Frith, 1990)

1.2 NATM

NATM stands for New Austrian Tunnelling method. The method is developed by Austrian engineers (Rabcewicz et al.) in the 1960’s. According to NATM design philosophy surrounding rock or soil formations are integrated into support structure. NATM uses flexible primary support and allow surrounding strata to deform slightly to utilize self carrying capacity of the rock mass. To ensure this, behavior of the rock mass monitored during construction by means of deformation and strain, and stress records.

According to NATM, the rock mass is classified without a numerical quality rating. Ground conditions are described qualitatively. The main rock mass classes and behavior of rock masses for each rock mass group according to the ONORM B2203 are given in *Chapter III - Rock Classification and Construction Method in NATM*

The New Austrian Tunnelling Method constitutes a design where the surrounding rock- or soil formations of a tunnel are integrated into an overall ring like support structure. Thus the formations will themselves be part of this support structure.

With the excavation of a tunnel the primary stress field in the rock mass is changed into a more unfavourable secondary stress field. According to the NATM theory, by the effect of stress rearrangement following the excavation, a *rock arch* formed above tunnel axis and all of the plastic and elastic behaviour is observed below the *rock arch*.

In NATM main objective of the support measurements are to *activate* the so called *rock arch*. NATM supports are to be light and flexible to allow but limit deformations by yielding under load. Therefore, by letting closure of the cavity within pre-determined deformation limits, the rock arch is activated. Activated rock arch does actual load carrying and the secondary stress field developed in a favourable way.

1.3 Bolu Tunnels

Bolu Mountain Crossing and Bolu Tunnels is the most challenging portion of the motorway between Istanbul and Ankara. As a result of the extremely difficult geological and seismic characteristics of the project area, natural disasters (flooding and earthquakes) and contractual disagreements completion of the 25.5 kilometers long section took more than 13 years.

During the November 1999 Düzce earthquake approximately 250 m of the tunnel section have been collapsed. Consequently, the works have been stopped in both tunnels at Elmalik (Ankara) side until August 2004. At that time, as a result of the re-assessment study, Elmalik side tunnel excavation started again from new Portals. This new tunnel alignment by-passed collapsed section. Beside emergency crossings (cross adits) and electromechanically works, approximately 1100 m tunnel work is completed in a year. Left tube excavations faces met at 26^{th} of August 2005. And right tube excavations met at 4^{th} of September 2005.

Both tunnels opened to traffic at 5^{th} of May 2007, after completion of emergency crossings and electro mechanic works.

1.4 Scope of the Study

Aim of this study is to analyze the support design practice at Elmalik Portals of Bolu Tunnels which is based on New Austrian Tunneling Method and the experience gained from the old design of the abandoned section.

For the analysis relation between rock-mass properties, support design and deformation behavior has been observed during the excavation of Bolu Tunnels New Alignment Elmalik Portals. (Right tube chainages 53+747 and 53+961 and left tube chainages between 63+779 and 64+141.00)

In order to achieve this objective:

i. Rock masses have been characterized,
ii. Support classes have been determined according to NATM ,
iii. Deformations on primary supports were monitored and recorded,
iv. The design has been justified by field deformation measurement,
v. The design also verified by convergence-confinement method.

CHAPTER II

BASICS OF TUNNEL DESIGN AND NEW AUSTRIAN TUNNELING METHOD

2.1 Basics of Tunnel Design

Various precautions are taken to ensure long-term stability and/or to maintain sufficient dimensions of the tunnel excavation. Those support measures vary with parameters like ground conditions, geotechnical properties of the rock or soil, overburden thickness etc. Likewise, different tunnels of similar ground conditions can utilize different support systems according to tunnel designer's preference.

Tunnel supports can be classified in two main groups according to construction order and purpose.

a) Temporary ground support:

Where the ground has insufficient stand-up time, to allow the construction of primary lining some temporary support like steel sets and shotcrete or rock bolts can be installed. Temporary supports are removed during the tunnel excavation or primary support installation.

b) Permanent ground support:

Support measures which are not removed and take place in final tunnel section, are called as permanent ground support. There are two types of permanent ground support, primary and secondary lining.

All of the support measures that applied in conjunction with tunnel advance are called as primary lining. Loads and deformations that the ground may induce is carried by primary lining.

Some tunnels require smooth inner profiles for usage (e.g. sewer and water tunnels) or aesthetics (e.g. traffic tunnels). Corrosion and erosion protection of primary lining is done by secondary lining. If required, water proofing of tunnel is provided by secondary lining. As another approach secondary lining can build to take the duty of primary lining over, if the primary lining is assumed to be worn out in time.

Supports of mining and civil tunnels are different. Mining tunnels are to provide access to working face. Except main access tunnels and shafts, life span of a mining tunnel is short. Average working life of a tunnel used for mineral exploitation is 10 years. As acceptable deformation rates are higher and repair works are easier compared to civil tunnels, lower safety factors (1.2-1.5) are applied. (Whittaker and Frith, 1990; pp 260)

Civil tunnels are generally driven under relatively shallow overburden. As the projected usage life is usually longer than 100 years and the tunnel is designed for public use, high safety factor (>2.0) is applied to ensure long-term stability. Additionally for civil tunnels water proofing, fine inner surface, illumination and ventilation is important.

2.2 Types of Tunnel Supports

There are different ways to support an excavation. Those may be used alone or in groups. Basic support measures are steel rib, concrete lining, and rock bolt.

2.2.1 Steel rib

Rolled steel beam supports are applied where rock mass conditions are likely to create large deformations and fracturing. Steels arcs are very popular due to their high mechanical strength, post yield behavior and bearing capacity. (Steel arcs continue to support tunnel extrados even after yielding point.) They are usually used as first support measure to be applied after excavation. Initial deformations are absorbed by the steel arc. Then primary lining is completed by shotcrete or in-situ or precast concrete.

Generally steel arcs are formed from two or more beam segments and set up in excavation face by means of connecting (fish) plates. Instead of an arc shaped profile, a closed ring should be preferred when resistance against side pressure and floor uplift is required. Ring closure can be obtained either by using steel ribs connected to form circular shape or curved concrete or shotcrete at floor. *Figure 2.1* and *2.2* shows example steel ribs configurations and steel rib connection types.

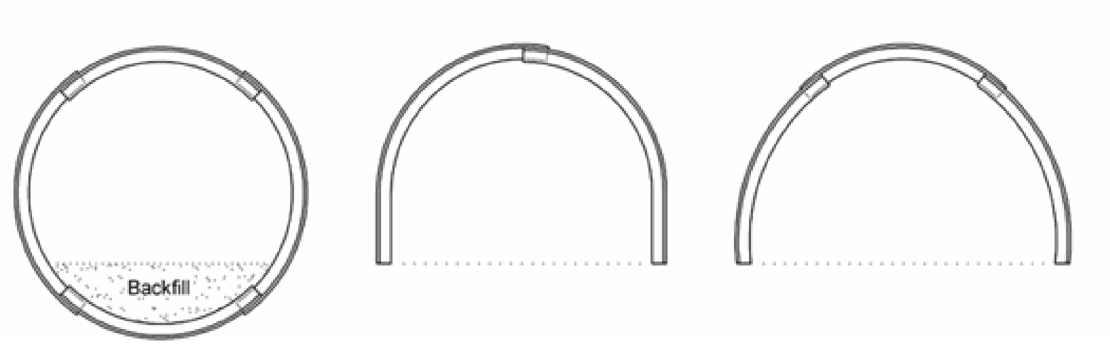


Figure 2.1: Example Steel Rib Support Configurations

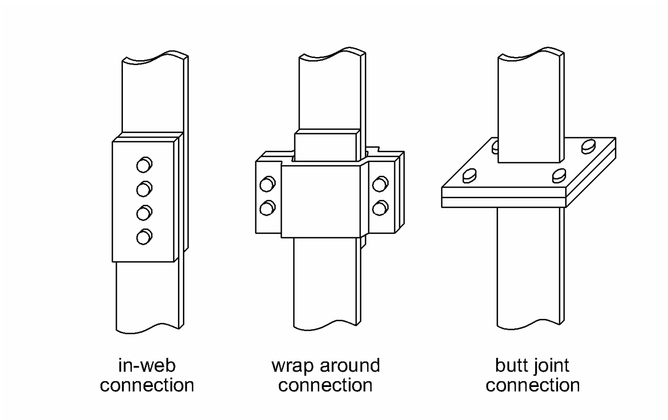
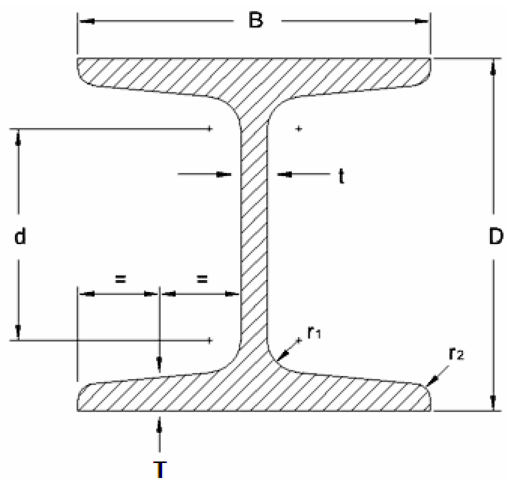


Figure 2.2: Steel Rib Connection Types

Tie-bars (or struts) are installed between arcs to provide regular spacing and strength in tunnel axis. This is very important if H section ribs are used, because H section supports are inherently weak in this direction. Foot blocks are used to prevent penetration of the support legs to the tunnel floor. Those can be concrete blocks, steel plates or wood blocks. Wire mesh layers can be used behind the steel arches to prevent detachment of smaller particles.

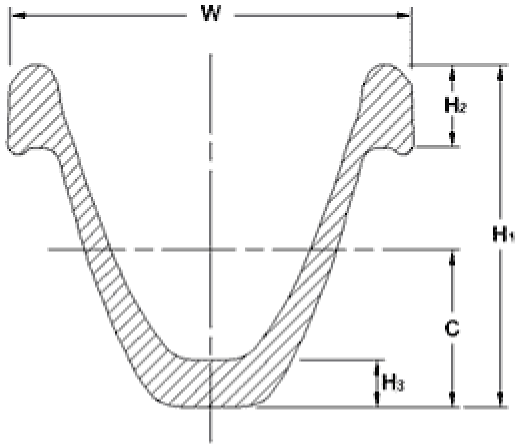
Two different types of steel beams are used in tunneling. H section beams and V section beams. H section steels are weak in YY-direction. V-section steel ribs are developed to overcome significant deformation and eccentric support loading encountered in mining tunnels. Main properties of the H and V sections beams are given in *Tables 2.1* and *2.2*.

Table 2.1 Dimensions and Properties of 'H' Sections Used for Supports (Whittaker and Frith, 1990)



		89x89	102x102	114x114	127x114	152x127
	Nominal Size	89x89 19.35 kg/m	102x102 23.07 kg/m	114x114 26.79 kg/m	127x114 29.76 kg/m	152x127 37.20 kg/m
D	Depth of section (mm)	88.9	101.6	114.3	127.0	152.4
B	Width of section (mm)	88.9	101.6	114.3	114.3	127.0
	Mass of section (kg/m)	19.35	23.07	26.79	29.76	37.20
t	Web thickness (mm)	9.5	9.5	9.5	10.2	10.4
T	Flange thickness (mm)	9.9	10.3	10.7	11.5	13.2
	Flange angle	98°	98°	98°	98°	98°
r_1	Root radius (mm)	11.0	11.2	14.2	9.9	13.5
r_2	Toe radius, inner (mm)	3.2	3.2	3.2	4.8	6.6
d	Depth between filets (mm)	45.2	54.0	61.1	79.5	94.5
	Cross section area (cm^2)	24.9	29.4	34.4	37.3	47.5
Moment of inertia (cm^4)	I_{xx}	306.7	286.1	735.4	979.0	1818.0
	I_{yy}	101.1	154.4	223.1	241.9	378.8
Elastic modulus (cm^3)	Z_{xx}	69.04	95.72	128.60	154.20	238.70
	Z_{yy}	22.78	30.32	39.00	42.32	59.65
Plastic modulus (cm^3)	S_{xx}	82.7	113.4	151.2	180.9	278.6
	S_{yy}	38.03	50.70	65.63	70.85	99.85
Radius of gyration (cm)	R_{xx}	3.51	4.06	4.62	5.12	6.20
	R_{yy}	2.01	2.29	2.54	2.55	2.82

Table 2.2 Dimensions and Properties of ‘V’ Sections Used for Supports (Whittaker and Frith, 1990)



		25	29	36
	Section Kg/m	25	29	36
H1	Height 1 (mm)	118	124	138
H2	Height 2 (mm)	28.5	30.0	33.5
H3	Height 3 (mm)	15.0	16.0	17.0
W	Width (mm)	137.0	150.5	171.0
C	Centroid	56.5	57.8	66.0
A	Area (cm^2)	32.2	37.4	46.4
Moment of inertia (cm^4)	I_{xx} I_{yy}	465 566	591 792	933 1284
Elastic modulus (cm^3)	Z_{xx} Z_{yy}	75.7 82.6	89.2 105.3	129.7 150.2

Equalization of strength in the x and y directions is achieved by V section steels. Only drawback of V section is high support weight, therefore, increased steel usage and heavier machinery to handle ribs.

2.2.2 Concrete Linings

Concrete has three distinctive usages in tunneling. Poured concrete, namely shotcrete, is very popular especially in NATM tunnels. Sprayed through a nozzle by a high pressure pump, shotcrete exactly match the tunnel profile. It is used in conjunction with wire mesh and rock bolts or alone. Shotcrete is used to control or even prevent rock deformations at the freshly excavated rock near tunnel face. It is also used to seal the rock mass to prevent deterioration. Its ability to control rock deformations in a fast and effective way made the shotcrete almost synonymous to NATM.

There are dry and wet applications of shotcrete. In dry shotcrete application, dry shotcrete mixture poured into hopper of the pump and sent to the nozzle by compressed air power. Water is added at the nozzle. In wet shotcrete application, shotcrete mixture including water is prepared at the batching plant and delivered to the site by trans-mixer trucks. In this case only additives are introduced in the pump. In wet application, diversion between the theoretical mix design and the application is less than dry application. And it is more suitable to continuous application. On the other hand, dry shotcrete equipments are cheap and more compact. As the water introduced at the face, transportation of material is easier. That’s way if there is no need for continuous application dry shotcrete is preferred.

Micro silica is a popular additive. Added in 8-13% by weight of cement, it reacts with the calcium hydroxide produced during cement hydration and increase the compressive strength of the shotcrete two or three times. It is also reduces rebound and increase stickiness (Whittaker and Frith, 1990).

Under poor rock conditions wire mesh is used as a primary support and to prevent detachments before shotcrete application; to provide grabbing surface to shotcrete or as a shotcrete reinforcement to increase ductility. Steel or PVC fibers are developed as replacement for wire mesh reinforcement. Fibers give ductility to the shotcrete and gives ability to withstand larger deformations even under un-uniform loading. Fiber reinforcement is expensive but can be preferred as it is easier to apply fiber than wire mesh reinforcement. The drawback of fiber reinforcement is, it is not as strong as wire mesh reinforcement against bending moments.

Cast in-situ or precast concrete lining is almost a must for public usage tunnels due to its aesthetic and ventilation properties. Concrete has high compressive strength and modulus of elasticity. However it is weak against tension and bending. If such loading is expected steel reinforcement is to be applied. Another drawback of cast in-situ concrete is that fresh concrete should be protected against loading during cure period. Otherwise, it cannot take its full strength. To prevent loading of fresh concrete, other support measures can be employed previously as temporary support.

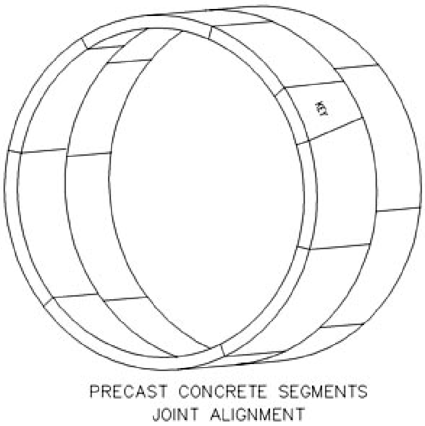


Figure 2.3 Installations of Precast Concrete Segments

As they are cast in separate place, precast concrete segments can be installed with little interruption to other underground works, and can be placed directly under load.

2.2.3 Rock bolts

Rock bolts are linear steel elements introduced into the rock body to bind jointed parts of the rock mass to each other and to shotcrete or concrete shell if they exist. Rock reinforcements increase the tensile strength of the rock, makes surrounding rocks to incorporate and work against deformations. It also can be used to prevent detachment of blocks and therefore wedge failures in heavily jointed but strong rock. There are mechanically anchored, resin anchored, grouted and swellex type of rock bolts.

Mechanical anchors are fixed in place by an expansion shell at the one end and a plate/nut system at the other end (*Figure 2.4*). Mechanical anchors work well in hard rock but in closely jointed or soft rock they are not very effective due to deformation and failure of the rock in contact with the grips of the anchor. Therefore in weak rock resin cartridge anchors are used. When using resin cartridge (or chemical) anchors, separate resin and catalyst cartridges pushed in the hole ahead of the bolt rod. At the end of the hole cartridges are break up. The chemicals, free of cartridges, are mixed and in few minutes resin is set.

If the rock bolt is meant to be used in long term in order to prevent rusting and corrosion the gap between drill hole and the bolt is filled with grouting. With chemical anchors special slow setting grout cartridges can be used behind fast setting anchor cartridge to fill the hole.

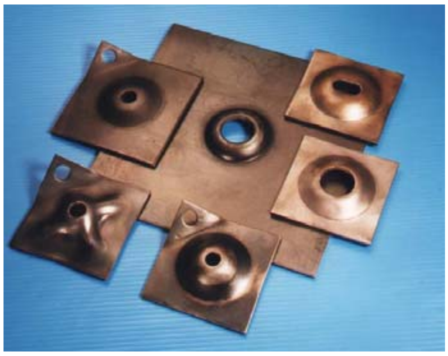
Sometimes rock bolts are installed for safety purposes only. That is to prevent rock falls due to detachments caused by joint planes. This type of 'safety bolts' are not carry much load. If a rock bolt will carry significant load as a rule of thumb it should be pretensioned up to 70% of the its capacity (*Hoek, 2000)*. Required pretension is applied by tightening the nut by a pneumatic torque wrench.

If the rock bolts are to be applied close to the excavation face, therefore stress development will occur after the installation of the bolt. So that, pre-tensioning is not required and simple 'dowel' type bolts can be used. Un-tensioned dowels work against movement of the rock. For this reason they are also called as passive rock bolts. There are three types of dowels grouted dowels, friction dowels and swellex.

Grout dowel is actually a steel rebar put into a grout filled drill hole (*Figure 2.4 d*). The nut is tightened after the grout is set. When more flexible support is required instead of rebar, steel cables can be used.

Friction dowel is a 'C' section tube (*Figure 2.4 e*) when pushed into a slightly undersized hole it generates radial spring force that bounds the bolt to the rock. They are especially good against rock bursts. Swellex system has similarities to the friction dowel. Swellex bolt is a larger diameter steel tube which is factory folded into a smaller diameter. After placement it is filled by pressured water. This way swellex bolt expands and fills the drill hole. This system is proved to be effective in jointed rock where grouting can be problematic. Neither friction dowel nor swellex bolt is recommended to the long term usage due to the corrosion problem.

(a) Expansion shell



(b) plates

(c) nuts and spherical seats

(d) grout dowel

(e) friction dowels

Pictures are taken from Steeledale SCS (Pty) Limited. Manufacturer of Rock Bolts, Studs & Mechanical Anchors, Grinaker-LTA Mining Products, web site: “http://www.steeledalescs.co.za

Figure 2.4 Rock Bolts and Related Equipments

2.3 New Austrian Tunneling Method (NATM)

New Austrian Tunnelling Method (NATM) has evolved as a result of experiences and innovation achieved in Austrian Alpine tunnelling conditions between 1957 and 1965. (Whittaker and Frith, 1990) In essence, NATM is an approach or philosophy integrating the principles of the behavior of rock masses under load and monitoring the performance of underground excavations during construction. The NATM is not a set of specific excavation and support techniques. It involves a combination of many established ways of excavation and tunneling, but the difference is the continual monitoring of the rock movement and the revision of support to obtain the most stable and economical lining. (Bieniawski, 1989)

New Austrian Tunneling Method can briefly described as utilization of the observational method to reduce support requirements in tunnel excavation. For that reason, to understand the logic of NATM one should know what the observational method is. The principles of the observational method were first described in detail by Peck (1969). But it has always been applied in construction projects that include excavation (Powerham, 1996).

According to Powerham (1996) the essentials of the observational method are:

a) Description of the general properties of the subject area,
b) the estimation of most probable conditions and the most unfavorable probable deviations from these conditions,
c) the establishment of the design based on the most probable conditions,
d) determination of the quantities to be observed as construction proceeds and the calculation of their expected values and values under the most unfavorable conditions,
e) determination of design modification for every significant deviation of the predictions in advance of the project,
f) the measurement of quantities to be observed and the evaluation of actual conditions,
g) and the modification of design to suit actual conditions.

In 1948 Rabcewicz invented dual lining supports expressing the concept of allowing the rock to deform before the application of the final lining so that the loads on lining are reduced. *Primary lining (initial support)* of dual lining system as proposed by Rabcewicz is made of systematically anchored rock arch with surface protection by shotcrete and possible steel rib reinforcement, and closed by invert. The second one *(final support)* is an inner arch consisting of concrete and is generally carried out before the outer arch reached equilibrium. Dual lining supports have become one of the key items of NATM later on. Kovari (1994), however, considers the idea behind this concept as Engensser's arching action published in 1882. (Karakuş and Fowell, 2004)

Later in 1964, Rabcewicz explains the New Austrian Tunnelling Method as:

".. A new method consisting of a thin sprayed concrete lining, closed at the earliest possible moment by an invert to a complete ring –called an auxiliary arch- the deformation of which is measured as a function of time until equilibrium is obtained."

Müller (1978) considers the NATM as a concept that observes certain principles. Although he has listed no less than 22 principles, there are seven most important features on which the NATM based (Bieniawski, 1989):

1. Mobilization of the Strength of the Rock Mass:

Surrounding rock mass is the main component of the tunnel support. Its self supporting capacity should be conserved. In order to enable the rock to support itself primary support must have suitable load deformation characteristics and be placed at the correct time.

2. Shotcrete Protection:

Excessive deformations and loosening must be minimized to preserve the load carrying capacity o the rock mass. For this reason, thin layer of shotcrete with or without rock bolting is installed immediately after face advance.

3. Measurements:

In order to obtain information on tunnel stability, monitoring of the deformations of the excavation and the loading of the supports are essential. This requires installation of instrumentation equipment at the time the initial shotcrete lining is placed.

4. Flexible Support

Versatile and elastic support is characteristic of the NATM. Rigid support is to be avoided. Strengthening is done by a flexible combination of rock bolts, wire mesh, and steel ribs instead of just thicker concrete lining. The primary support partly or completely represents the total support required. Capacity of secondary support is determined according to the monitoring results.

5. Closing of Invert

Installation of the invert is vital to form a load-bearing ring of the rock mass. In soft-ground tunneling, invert should be closed as quickly as possible. However, for tunnels in rock, support should not be installed too early since the load-bearing capability of the rock mass would not be fully mobilized.

6. Contractual Arrangements:

NATM can be successfully applied only if contractual system permit changes in support and construction methods during construction.

7. Support Measures & Payment:

Rock Mass Classification Determines Support Measures. Payment for support is based on rock mass classification.

There are two approaches among tunnel engineers for NATM. A group of engineers consider NATM as a philosophy where tunnel walls allowed to deform for certain extend to decrease ultimate support load. Other group considers that NATM as only a collection of tunneling methods like sequential excavation, sprayed concrete and passive rock bolts.

In 1980, to solve literature conflicts, New Austrian Tunneling Method (NATM) concept has been redefined by "Austrian National Committee on Underground Construction of the International Tunneling Association (ITA)". (Kovari, 1994) ITA' new explanation was:

"The New Austrian Tunneling Method (NATM) is based on a concept whereby the ground (rock or soil) surrounding an underground opening becomes a load bearing structural component through activation of a ring like body of supporting ground".

Sauer (1988) made another definition of NATM which describes the method in a more general fashion.

"A method of producing underground space by using all available means to develop the maximum self-supporting capacity of the rock or soil itself to provide the stability of the ground."

Rock mass classification in NATM is done without a numerical quality rating; ground conditions are described qualitatively. The Austrian standard ONORM B2203 of October 1994 is based on the suggestions by Rabcewicz et al. (1964). The main rock mass classes and behaviour of rock masses for each rock mass group according to the ONORM B2203 are given in *Chapter III - Rock Classification and Construction Method in NATM*

According to Rabcewicz (1964), to be able to design load bearing capacity of the support, the relationship between the disturbed ground around the cavity (*protective zone*), and the bearing capacity of the support (*skin resistance*) is required to be established. Mathematical representation of this relation is described by Kastner as: (Karakuş and Fowell, 2004)

$$p_i = -c\cot\phi + p_0\left[\cot\phi + (1-\sin\phi)\right]\frac{r}{R}^{\frac{2\sin\phi}{1-\sin\phi}} \tag{2.1}$$

Omitting the cohesion, the Equation 1 yields to:

$$p_i = p_0(1-\sin\phi)\frac{r}{R}^{\frac{2\sin\phi}{1-\sin\phi}} = np_0 \tag{2.2}$$

c: cohesion, φ: internal friction angle, R: radius of protective zone, r: radius of cavity, p_i: required skin resistance, p_0: initial pressure condition (γH), n: function of p_0 and φ.

In NATM, the design of the support system should be integrated to the deformation characteristics of the ground. Load bearing capacity of the media and the support system can be best understood by the rock support interaction diagram (*figure 2.6*). (Rabcewicz, 1965; Vavrovsky, 1995).

Rabcewicz (1973) stressed that NATM support should be neither too stiff nor too flexible. If the right support is applied at the right time, the support pressure takes the minimum value at equilibrium point. He used the ground response curve to explain his idea. When a stiffer support (support 2) is chosen, it will carry a larger load because the rock mass around the opening has not deformed enough to bring stresses into equilibrium. After point C, ground behavior becomes non-linear. If the support 1 is installed after a certain displacement has taken place (point A), then the system reaches equilibrium with a lower load on the support. But, after the point B "detrimental loosening" starts and the required support pressure to stop the loosening increases greatly. Details of ground support interaction explained in "*Section 2.4 - Ground Support Interaction in NATM*" of this manuscript.

In 1996, Institution of Civil Engineers have been published a redefinition of NATM for soft ground applications as Sprayed Concrete Lining (SCL) method. According to Institution of Civil Engineers there are two principal measures of NATM for soft ground applications. Excavation must be done in short time and as small steps. Completion of primary support ring must not be delayed. This claim also states that: "Sprayed concrete lining usage in soft ground in urban areas does not employ NATM philosophy. Rather it is the use of NATM related construction techniques." (Karakuş and Fowell, 2004)

A critical analysis of the principles of the complete New Austrian Tunneling Method (NATM) has been published by Kovari (1994). The author claimed that the NATM is based on erroneous concepts. The most recently published paper by Kovari (2004) traces the history of rock bolts and the NATM from its beginnings and shows how it developed. This paper describes numerous examples of civil engineering work worldwide with early application of rock bolting. In concluding, it is demonstrated that NATM is in many respects borrowed and has created much confusion amongst professional engineers. (Kovari, 2004)

2.4 Ground - Support Interaction in NATM

New Austrian Tunnelling Method relies on "*ground-support interaction*" or "*convergence confinement analysis*" for support design of an underground excavation. According to the Hoek (1980):

"The principal objective in the design of underground excavation support is to help the rock mass to support itself."(Hoek, Brown, 1980)

To explain how to help rock mass, Hoek determines rock-support interaction in 5 stages, namely virgin rock phase, free phase, early support phase, deforming support phase and equilibrium. (*Figure 2.5*)

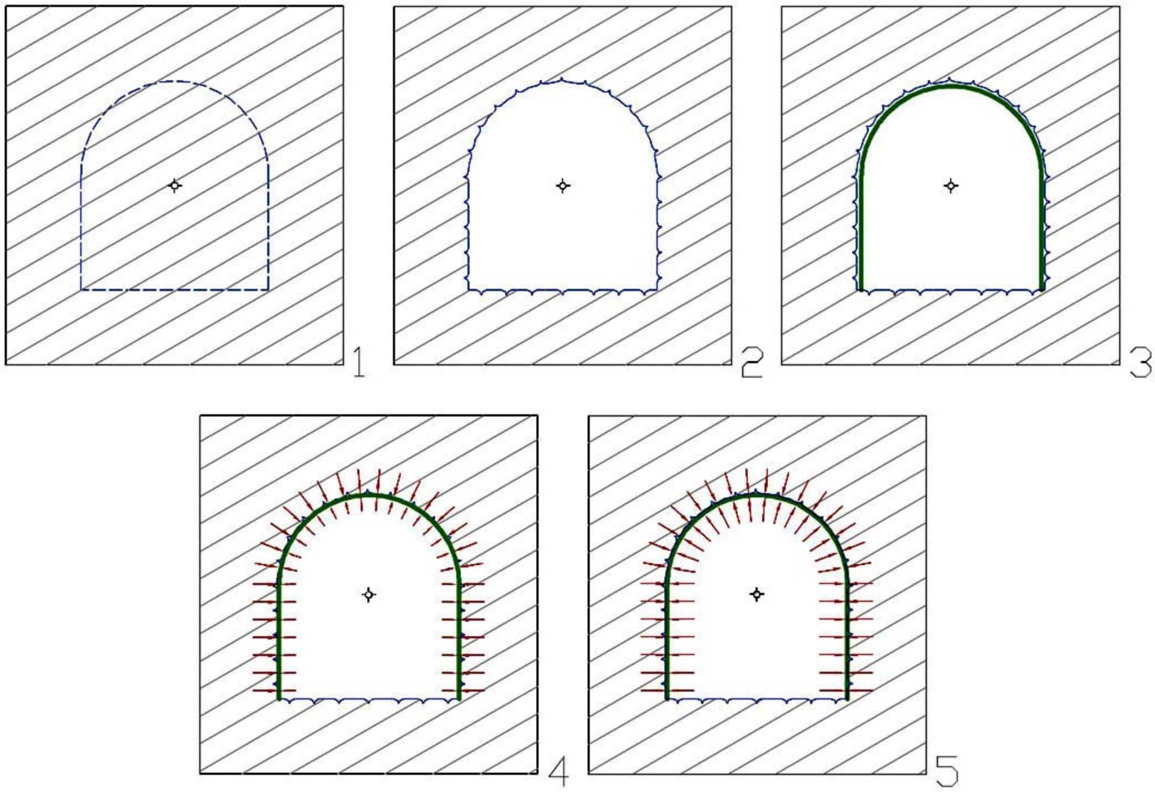


Figure 2.5 Rock - Support Interaction Stages according to Hoek (1980)

Stage 1 - Virgin rock:

In that stage rock body is unexcavated and stable in natural equilibrium. Rock mass inside the proposed tunnel profile can be considered to provide initial support pressure (P_i) equal to in-situ stress (P_o).

Stage 2 – Free phase:

After excavation, as P_i reduced to zero while rock pressure increased by the mass of loosened rock above the tunnel, unsupported rock body is free to intrude into tunnel. But the tunnel will not collapse because radial deformation is limited by the proximity of the tunnel face. If the tunnel face is not capable to provide that support temporary support applications are required.

Stage 3 – Early support phase:

In that phase a support arch is installed but, there is space between the support arch and tunnel perimeter. Therefore rock is free to deform but at the end it will touch to the support.

Stage 4 – Deforming support phase:

Now rock pressure is carried by support system but support pressure is not high enough yet to limit rock deformations. Supports provide pressure as response to deformation. Therefore while supports are yielding under rock pressure, P_i increases.

On the other hand, as the rock mass develop deformations, pressure required to limit deformation is decreasing.

Stage 5 – Equilibrium phase:

In that phase rock pressure and support pressure comes into equilibrium. There will be no rock deformation furthermore, and tunnel is stable.

Therefore, by allowing deformations within certain limits support requirement might be reduced. If this limit exceeded rock starts to loose its self supporting ability. Consequently support requirement decrease. In order to determine support requirement ground-support interaction analysis is done on a rock-pressure vs. deformation plot. As an example, rock-pressure vs. deformation curve of a simple full-face driven drill and blast tunnel are plotted in *Figure 2.6*.

The blue line on the plot shows the ideal ground-support interaction curve. As it is seen from the plot, between points the ‘A’ and ‘B’, the surrounding rock deforms freely and *pressure required to limit deformation* decreases due to stress release. At point be support system starts to work. After that point support pressure increases with deformation. At point ‘C’ the system comes to equilibrium.

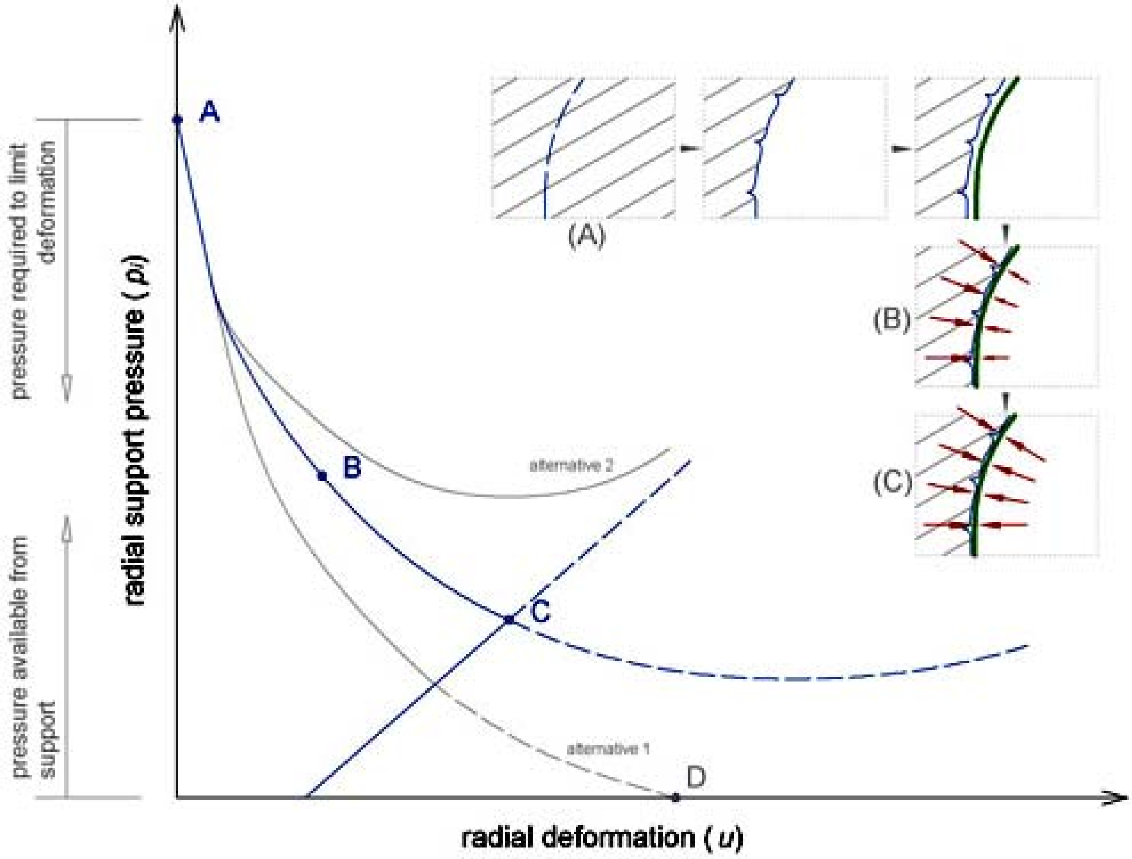


Figure 2.6 Rock-Pressure Deformation Curve of a Simple Full-Face Driven Drill and Blast Tunnel

Beyond point ‘C’ see you can see the dashed line, representing the deformation curve that would be if the support doesn’t exist. Here you can see the curve turning upward. This is the point where surrounding rock looses its ability support itself. Hereafter, required support pressure increases with deformation.

There are also two gray lines standing for alternative rock behavior. *Alternative 1* shows a rock mass behavior where support application is not required at all. Here you can see the dashed line (if the support doesn’t exist) drops to the zero line. It means at that point rock will support itself without any help from support system. On the contrary *alternative 2* shows deformation curve when the rock loosening so fast that, applied support couldn’t help to stop deformations.

Consequently, as you can see in *Figure 2.6*, if you install supports immediately after excavation support requirement will be higher. To reduce the required support pressure you should allow deformations to certain limit. When this limit exceeded rock starts to loose its self supporting ability, therefore support requirement decrease.

2.5 Rock Classification in NATM

According to NATM, the rock mass is classified without a numerical quality rating. Ground conditions are described qualitatively. Rock classes are determined according to the appearance of the rock at the excavation face of the tunnel. In case of a top heading, bench, invert excavation, the rock conditions of the top heading drivage govern the classification. In case of a drivage sequence with side galleries, each drivage regarded separately and classified accordingly.

NATM rock classification system is based on the suggestions by Rabcewicz et al. (1964) The rock classification system and its description is presented in Table 3.1 (Ayaydın, 1986) (quoted from Coşar, 2004) Later on, this classification system is modified and standardized as "*Austrian Standard ONORM 2203".* Final Revision of ONORM 2203 was done in 1994. Comparison of rock classification systems used with NATM and the main rock mass classes and behavior of rock masses for each rock mass group given in *Table 2.3*. Latest rock mass classification used in NATM, *Austrian Standard ONORM 2203 after October 1994*, is suggested and preferred in this study.

Table 2.3 Rock Classification System,

according to Pacher and Rabcewicz (Ayaydın, 1986)

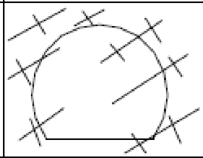
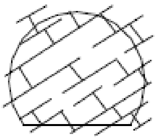
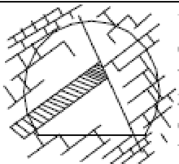
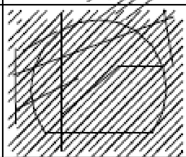
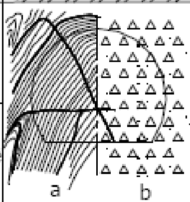
ROCK MASS CLASSIFICATION IN TUNNELING				
CLASS No	CLASS NAME		GEOMECHANICAL BEHAVIOUR	WATER EFFECT
I	STRONG get slightly fragile by time	 Dense, uncertain discontinuity traces	Uniaxial compressive strength of rock is higher than the tangential stress on the opening wall No I: continuously stable (precautions to rock burst) No II: continuously stable with the support of the bench	None
II	FRAGILE get very fragile by time	 Certain discontinuities due to the bedding and jointing, locally clayey joint fillings		Not significant
III	FRIABLE	 Wide and effective crushing, fucturing, mylonite zones in all directions, clayey fillings	Tangential stress on the excavation wall is higher than or equal to resistance of the rock. Open or close load bearing arch is necessary	Very effective on joint fillings
IV	SQUEEZING	 Crushed, folded thick mylonite zones, very well squeezed, cohessive soil	Due to tangential stresses on the excavation wall is higher than the bearing capacity of the rock, rock behaves plastically. Deforming towards opening No IV : slow and minor rate No V: fast and effective rate Horizontal stress and floor heaving are expected. Load bearing close arch that should be installed immediately after excavation is necessary	Highly effective on joint fillings and rock mass quality
V a	VERY SQUEEZING potential to swelling	 Completely crushed, mylonitized and doughed rock		Very high, softening
V b	SQUEEZING Low cohessive soil	Low cohessive weathered soil		
VI	Special Type (Flowing)	Non-cohessive, flowing soil	Similar with V and special precautions are necessary	Very much

Table 2.4 compares the three rock classification systems used in NATM. In Bolu Tunnels, "AUSTRIAN STANDARD ÖNORM B 2203 (after Oct. 1994)" has been utilized for rock classification and class description. Then support classes are determined accordingly.

Table 2.4 Comparison of Rock Classification Used with NATM (Geoconsult, 1994)

AUSTRIAN STANDARD ÖNORM B 2203 (after October 1994)		AUSTRIAN STANDARD ÖNORM B 2203 (before October 1994)		CLASSIFICATION AFTER RABCEWICZ - PACHER	
A1	Stable	1	Stable	i	Stable rock, slightly afterbraking
A2	Slightly afterbraking	2	Afterbraking		
B1	Fragile	3	Slightly friable	ii	Friable
B2	Heavily fragile	4	Friable of slightly pressure exerting		
				iii	Heavily friable
C1	Pressure exerting	5	Heavily friable or pressure exerting		
				iv	Pressure exerting
C2	Heavily pressure exerting	6	Heavily pressure exerting	v	Heavy pressure exerting or flowing
L1	Loose ground highly cohesive	7	Flowing	vi	Special conditions
L2	Loose ground low cohesive				

2.6 Support Classes and Properties

NATM support classes are described in detail as follows:

CLASS A - Stable to Slightly Overbraking Rock Masses

Support class A1 corresponds to stable rock; rock mass that behaves elastically. Deformations are small and decrease rapidly. There is no tendency of overbraking. The rock mass is permanently stable without support. There is no influence of water.

In case of large excavation profiles excavation can be divided as top heading and bench. Drill and blast technique is required for excavation. Round length is determined by the practical construction reasons. Systematical support does not require but local support elements might used.

Support class A2 characterize slightly overbraking rock; rock mass that behaves elastically. Deformations are small and decrease rapidly. A slight tendency of shallow overbreaks in the crown area (tunnel roof and upper portions of the side walls), caused by discontinuities and the dead weight of the rock mass, exists. Effect of water is insignificant.

In case of large excavation profiles excavation can be divided as top heading and bench. Drill and blast technique is required for excavation. Round length is, determined by the practical construction reasons, around 2.5 to 3.5 meters in top heading and to 4.0 meters in benching.

Crown area should be supported. Rock bolts are required and should be installed not later than one round behind the face. Bolting direction should be chosen in accordance with the orientation of the discontinuities.

CLASS B - Fragile Rock Masses

Support class B1 stands for friable rock mass; major parts of the rock mass behave elastically. Rapidly decreasing small deformations are expected. Stan-up time is limited. If supports are not installed in time, overbreaks and loosening of the rock strata in tunnel roof and upper sidewalls may occur. Effect of water presence is generally insignificant.

The cross section of excavation should be divided into top heading and bench. Round length is between 2.0 and 3.0 meters in top heading and maximum 4.0 meters in bench. Drill & blast operation is needed. A systematic support pattern is required in top heading and in bench. The support should be installed not later than one round behind the face.

Support class B2 represents heavily friable rock mass; this type of rock mass is characterized by large areas of non-elastic zones extending far into the surrounding rock mass. Stand-up time of unsupported span is short. Although, immediate installation of the tunnel support ensures deformations can be kept small and cease rapidly. Delayed installation may cause deep loosening and loading of the initial support. The potential of deep and sudden failures from roof sidewalls and face is high. Large water inflows in weathered or disintegrated rock masses have considerable influence on the strength of rock.

The subdivision into top heading and bench is required. Rock masses sensitive to vibration may require mechanical excavation. Round lengths are between 1.5 to 2.5 meters in the top heading and maximum 3.5 meters in the bench. Concrete invert arch should be necessary. When required, invert distance from face should not be more than 150 m. Tunnel roof and side walls require systematic support which to be installed after excavation before any further advance. Forepoling may be required locally.

CLASS C - Pressure Exerting Rock Masses

Designation ‘C’ refers to a rock mass in which the stresses following the redistribution process of rock pressure and/or restraints are higher than the strength of the rock mass. Plasticity and viscosity of the rock mass leads to a distinct time depending deformation behavior. Considerable loosening pressure and activation of self weight loads of rock only occur when too large deformations are allowed, which due to detrimental loosening and disintegration of the rock mass, cause an extensive reduction of the rock’s strength. Rock masses tending to rock burst or rock fall and rock masses with swelling components are associated to this group.

Support class C1 corresponds to pressure exerting rock mass. This kind of rock mass is characterized by plastic zones extending far into the surrounding rock mass and failure mechanisms such as spalling buckling, shearing and rupture of the rock structure, by squeezing behavior or by tendency of rock burst. It shows a moderate, but distinct time depending squeezing behavior. Deformations at the cavity boundary are moderate and calm down slowly except in case of rock bursts. Seepage and concentrated water inflows have a considerable influence on the behavior of the rock mass.

The subdivision between top heading and bench is imperative. Invert excavation is required. Round lengths are in a range between 1.0 and 1.5 meters in the top heading and up to 2.0 meters in the bench. Excavation may be done by smooth blasting or by a tunnel excavator. All tunnel support including shotcrete sealing is applied systematically and before any further advance. Forepoling is required over the whole roof section. Invert arch is required and to be installed within 100 to 150 meters behind the top heading face.

Support class C2 represents heavily pressure exerting rock mass. This kind of rock mass is characterized by the development of deep failure zones and rapid and significant movement of the rock mass into the cavity and deformations which decrease very slowly. Support elements may frequently be overstressed. Seepage and concentrated water inflows have a considerable influence on the behavior of the rock mass.

A subdivision into top heading, bench and invert is imperative. Maximum round length is 1.2 meters in the top heading and 2.0 meters in the bench. Excavation may be done by smooth blasting or by tunnel excavator. Shotcrete sealing is required immediately. A dense support pattern at all exposed surfaces are required. Special features such as deformation slots in the shotcrete or highly deformable support elements might be required. The support elements should maintain the triaxial stress state of the rock mass. All tunnel supports are applied systematically and immediately after excavation. The ring closure of the invert arch may be required as short as 25 to 50 meters behind the top heading face. Forepoling is required over the whole roof section. According to the amount of the observed deformations shortening of the round lengths, increase in forepoling length, large central support body at the top heading face, further subdivision of the face, widening of the lining foot, bolting and grouting of the abutment shotcrete zone or temporary invert arches may become necessary.

CLASS L - Loose ground

This group contains rock masses such as disintegrated or decomposed rock, loose grounds and organic soils. Low properties of the rock mass yield to elastic or plastic overstressing. The behavior of the rock is time dependent; i.e. rock mass quality decreases within the free span if no support is installed within a reasonable time. Accordingly the maximum length of a round which can be excavated and supported in time is a criterion of rock quality and therefore taken into account for evaluation of the support class.

Support class L1 described as highly cohesive short-term stable rock mass. For L1 class rock mass, seepage is generally limited due to dense materials. But, seepage's and concentrated inflows have a considerable influence on the behavior and properties of the exposed ground. Swelling may be initiated.

A subdivision into top heading, bench and invert is imperative. A support body for the top heading face is required in most of the cases. Length of rounds should not exceed 1.5 meters in the top heading and 3.0 meters in the bench. Excavation should be done by tunnel excavator. Top heading and bench supports are required to be installed immediately after excavation. Shotcrete sealing should be done simultaneously with the excavation of subdivided section. Invert arch is required and should not be more then 100 to 150 meters behind the top heading face. Forepoling or lagging is required.

Support Class L2 describes short term stable rock mass with low cohesion. This type ground has no stand up time without support. Seepage's and concentrated inflows may occur and may have a considerable influence on the behavior of the exposed ground.

Subdivision into side galleries is required. Excavation should be done by tunnel excavator. Top heading and bench supports are required to be installed immediately after excavation. Shotcrete sealing should be done simultaneously with the excavation. Length of rounds should not exceed 1.2 meters in the top heading and 2.0 meters in the bench. Top heading and bench supports are required to be installed immediately after excavation. Shotcrete sealing should be done simultaneously with the excavation of subdivided section. Temporary ring closures of the subdivided cross sections may become necessary. The ring closure of the invert arch may be required as short as 25 to 50 meters behind the top heading face.

The corresponding support classes are summarized and their properties are explained in Table 2.5.

Table 2.5 Support Classes Summary (Geoconsult, 1994)

Support Class	A1	A2	B1	B2
Description	Stable rock	Slightly overbraking rock	Friable rock	Heavily friable rock
Behavior	Elastic	Elastic	Mostly elastic	large areas of non-elastic zone extending far into the surrounding rock
Deformation	Small and decrease rapidly	Small and decrease rapidly	Small and decrease rapidly	Immediate support is required to stop deformations and cease rapidly
Overbreak	there is no tendency	slight tendency of shallow overbreaks		Potential of deep and sudden failures. Forepoling may be required locally
Stand up time	Permanently stable	Permanently stable	Limited	Short
Influence of water	No	Insignificant	generally insignificant	considerable influence on weathered or disintegrated rock masses
Excavation	Full face or T/H and bench	Full face or T/H and bench	Top heading and bench	Top Heading, bench and invert
Drill and Blast	X	X	X	(smooth blasting)
Roadheader	-	-	-	X
Tunnel excavator	-	-	-	-
Round length (m)	Limited by construction reasons and smooth blasting requirements	Up to 2.5-3.5 meters at top heading and 4.0 m at bench.	T/H: 2.0 - 3.0 Bench: 3.0	T/H: 1.5 - 2.5 Bench: 3.5
Support	Local support elements when necessary	Supports are required in the crown area	Systematic support at top heading and bench	Tunnel roof and side walls require systematic support installed at the face. Cast in place invert arch may be required. (max 150m behind the bench face)

Table 2.5 Contn'd

Support Class	C1	C2
Description	Pressure exerting rock	Heavily pressure exerting rock
Behavior	Plastic zones extending far into the surrounding rock mass and failure mechanisms like spalling, buckling, shearing and rupture by squeezing behavior or tendency of rock burst	Deep failure zones around cavity
Deformation	Moderate time dependent deformations calm down slowly	Rapid and significant deformations that decrease very slowly.
Overbreak	Immediate shotcrete sealing is required after excavation. Forepoling is compulsory.	Immediate shotcrete sealing is required after excavation. Forepoling is compulsory.
Stand up time	Short	Short
Influence of water	Seepage and concentrated water inflows have a considerable influence	Seepage and concentrated water inflows have considerable influence
Excavation	Top Heading, bench and invert	Top Heading, bench and invert; if required further subdivision of top heading face
Drill and Blast	Smooth blasting	(smooth blasting)
Roadheader	X	X
Tunnel excavator	X	X
Round length (m)	T/H: 1.0 - 1.5 Bench: 2.0	T/H: 1.2 Bench: 2.0
Support	Tunnel roof and side walls require systematic support installed at the face. Cast in place invert arch may be required. (100 – 150m behind the bench face)	A support body for top heading face is required. Highly deformable support elements which are capable to handle triaxial stress state are required to prevent support overstressing. Cast in place invert 25 – 50 m behind the bench face.

Table 2.5 Contn'd

Support Class	L1	L2
Description	Highly cohesive short-term stable rock	Short time stable rock mass with low cohesion
Behavior	Remains stable for a limited time	This type of ground has no cohesion and requires simultaneous forepoling and support with excavation.
Deformation	High	High
Overbreak	Immediate shotcrete sealing is required after excavation. Forepoling is compulsory.	Immediate shotcrete sealing is required after excavation. Forepoling is compulsory.
Stand up time	Very short	No
Influence of water	Seepage is limited due to dense materials but swelling may be initiated.	Seepage and concentrated water inflows may occur and have a considerable influence
Excavation	Top Heading, bench and invert; if required further subdivision of top heading face	Sub division into side galleries may be required.
Drill and Blast	-	-
Roadheader	-	-
Tunnel excavator	X	X
Round length (m)	T/H: 1.5 Bench: 3.0	T/H: 1.2 Bench: 2.0
Support	All tunnel support with the exception of the inner layer of wire mesh is required before any further advance at the top heading and bench. The invert arch is required to be installed 100 to 150 meters behind the top heading face.	All tunnel support with the exception of the inner layer of wire mesh is required before any further advance at the face of top heading and bench. The invert arch is required to be installed 25 to 50 meters behind the top heading face.

2.7 NATM Construction Method

When talking about NATM, there are contradictory opinions in the literature such as, some of the engineers refer it as a special technique others as a sort of philosophy. Many refer to NATM when NATM *construction method* (shotcrete, rock bolts etc.) is applied, without considering whether NATM *design philosophy* is utilized or not.

Things are getting more complicated for soft-ground tunnels. Romero (2002) states that: *"The deformation of the soil is not easily 'controlled'. Therefore it can be concluded that the excavation and support planned for sequentially excavated, shotcrete-lined tunnels utilizes NATM* <u>*construction*</u> *methods but not necessarily NATM* <u>*design*</u> *methods."*

For the reason, it would be a better idea to consider NATM construction and design methods separately. Although there are plenty of variations in NATM construction method, there are some commons that describes NATM tunnel. Those can be listed as:

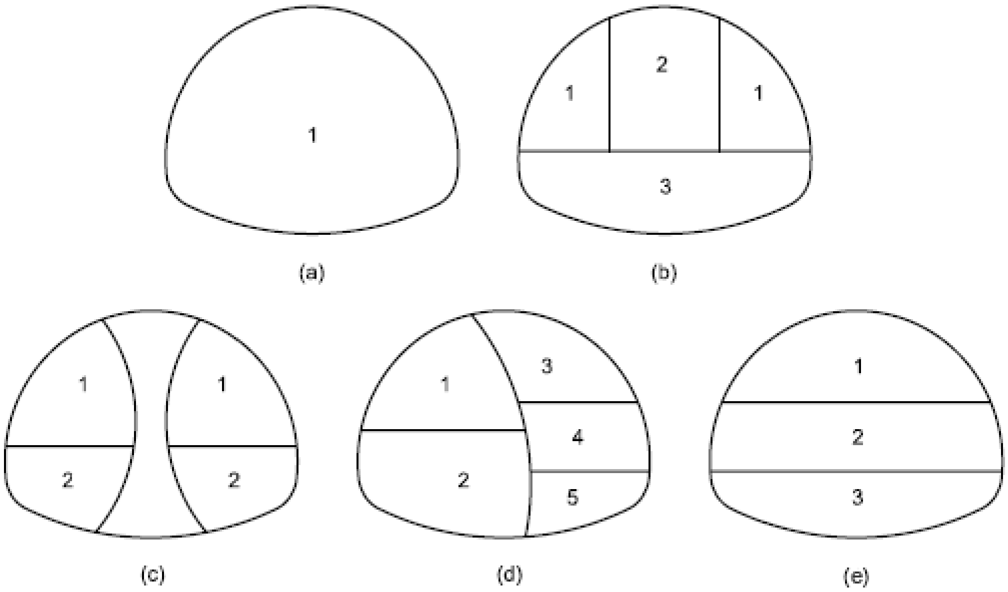
- The initial ground support is provided by shotcrete in combination with fibre or welded-wire (wire mesh) fabric reinforcement.
- Steel arches, usually lattice girders, and sometimes ground reinforcement, e.g. rock bolts, spiling, utilized.
- Ring like support structure created.
- The final support is usually, but not always, a cast-in-place concrete lining.

2.7.1 Excavation Sequences

Sharing same common construction techniques, variation of NATM applications are in excavation sequences. NATM tunnel may be without sequences, *full-face* or *open-face* tunnel, or with sequences according to Cross Diaphragm Method, Upper Half Vertical Subdivision method or Top Heading and Bench Method. (*Figure 2.7*)

Sequences of *Top Heading and Bench* method is herewith explained in detail since this method is utilized in Bolu Tunnels. In Top Heading and Bench approach tunnel advance is divided into three sequences. Upper half of the tunnel is called as Top Heading. Below top heading level there is a section called as bench. Below bench level there may be a third section called invert, if rock properties requires it (e.g. weak ground conditions). See *Figure 2.7* for three sections of Top Heading and Bench method.

Top heading (T/H) excavation leads to the tunnel advance. There is certain distance between T/H and bench excavation faces. Similarly invert excavation follows bench face from a certain distance. *Figure 2.8* shows the Excavation Sequence of T/H and Bench Tunnel.



(a) Full Face excavation, (b) Upper Half Vertical Subdivision Method, (c) Cross Diaphragm Method, (d) Top Heading and Bench Method, (e) Top Heading and Bench Method with Staggering.

Figure 2.7 Variations of Excavation Sequences in NATM

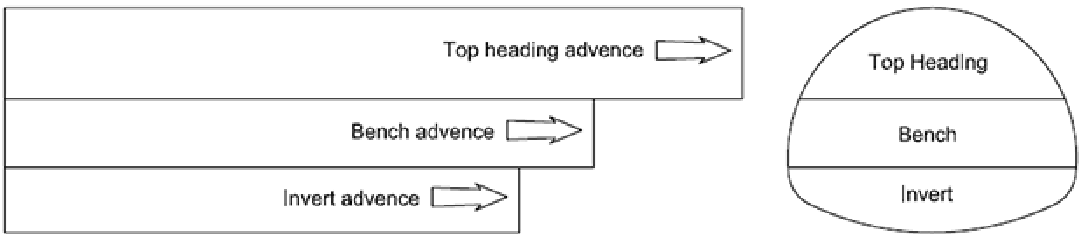


Figure 2.8 Excavation Sequences of Top Heading and Bench Method

The longitudinal distances between the 3 stages of the excavation - namely interline distances - (from Top Heading face to bench face and from top heading to invert face) were defined according to the different geological conditions. During excavation, geological maps have been prepared to provide documentation of rock and rock mass conditions encountered by the site geologist.

Excavation Works

Excavation activity has been either performed by means of a hydraulic excavator or drilling and blasting operations according to the defined construction sequence and by several work activities. The excavation phases of each stage were performed in different time to avoid any uncontrollable structural stress distribution due to the rock loading pressure and tectonic systems.

The excavation profile refers to the theoretical profile called as T-line. An appropriate enlargement of the theoretical excavation profile has been made in order to provide enough space for radial deformations and construction tolerances. (*Figure 2.9*)

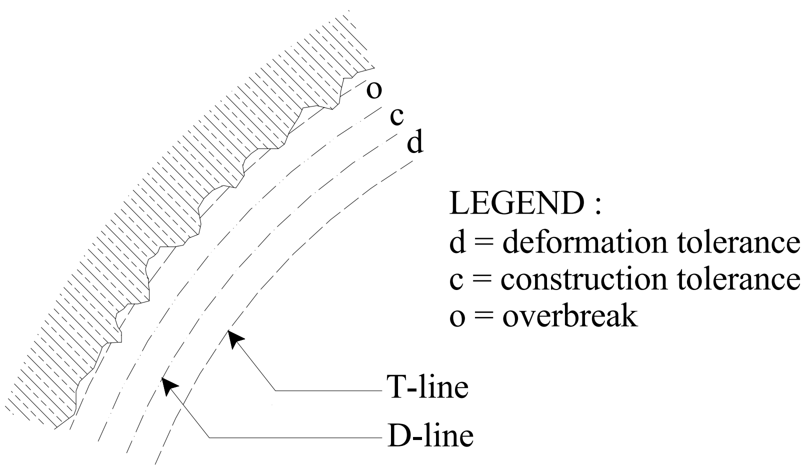


Figure 2.9 Excavation Tolerances (Astaldi, 2004)

The D-Line *(Figure 2.9)* describes radial deformation tolerances for the certain rock classes. The deformation values given on relevant drawings may be adjusted to suit extraordinary large deformations. The D-line represents the minimum profile to be excavated.

The space created when the ground breaks outside the profile calculated for the support elements required is called over break. The over break may be caused by improper and careless excavation (avoidable over break) or purely by geological reasons (unavoidable over break). In case of excessive over break extraordinary remedial works (e.g. grouting) has to be carried out.

2.7.2 Support Measures Applied in NATM

The commonly used temporary supports in NATM Tunnels are: face sealing, face bolts, forepoling, intermediate shotcrete invert, bolted shotcrete footing plates

Permanent supports are: steel ribs, shotcrete lining, rock bolts, monolithic concrete invert, intermediate (also called Bernold) lining, and inner lining concrete

2.7.2.1 Face sealing

The Shotcrete sealing is a support measure to stabilize and to minimize the risk of collapses of the excavated tunnel face and round for work safety. When the excavation is completed the excavated round is sealed by shotcrete. This operation consists in covering the whole exposed surface by a steel fiber reinforced (if required) shotcrete layer using sodium-silicate shotcrete. Thickness may change 5 to 10 cm.

2.7.2.2 Steel Rib

TH profile steel ribs have been commonly installed immediately after excavation. In Bolu tunnels, Mining originated, Omega TH29 steel ribs were used. TH29 profiles are hot rolled and have bell shaped cross section. Connections of the rib sections are done with an overlap of the profiles fitting into each other (with 30-60 cm overlap) and bond with clamps. (*Figure 2.10*). This connection is arranged in order to compensate large deformations due to friction in the clamp connections.

Steel ribs performs a primary support immediately after excavation and shall subsequently act as reinforcement and load distributing part inside of the shotcrete lining.

Steel ribs are assembled on the ground and then lifted to the final position. In order to improve the structure of the primary support, the steel ribs should be positioned as close as possible to the excavation line. For that reason concrete foot blocks are used to set the steel ribs to the required elevation. The connection between the previous ribs is performed by steel bars (namely tie bars). Those steel bars are fixed at the steel ribs by welding.

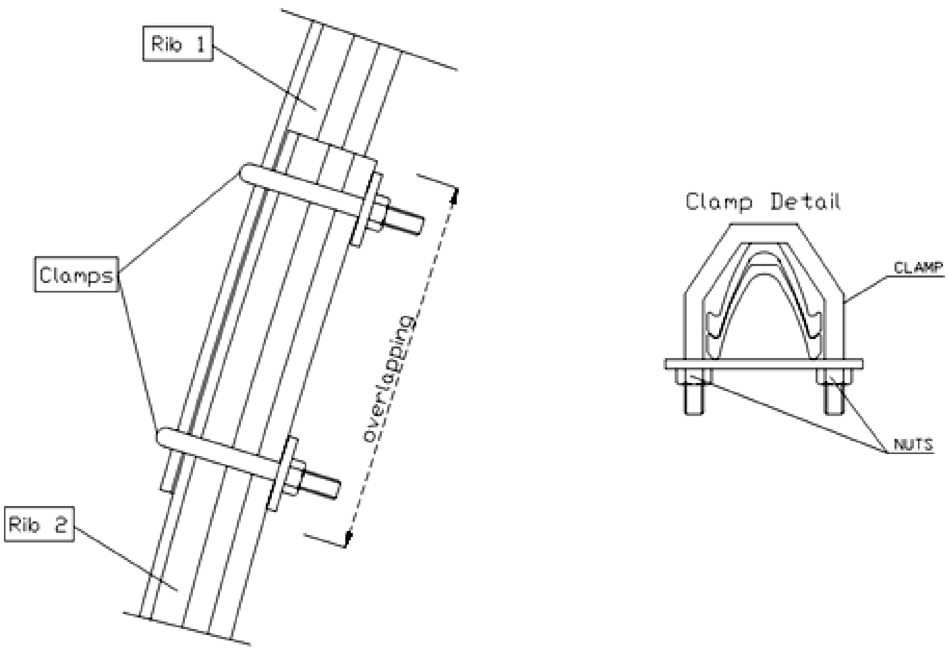


Figure 2.10 Omega TH 29 Steel Ribs (Technical Drawings, 45.110/15766)

2.7.2.3 Shotcrete Lining

Shotcrete or sprayed concrete lining is the major primary tunnel support. Shotcrete is applied as wet mix Shotcrete. Thickness depends on applied support class. Shotcrete lining is applied in one or two layers. When two layers of Shotcrete are applied, one layer of wire mesh is installed between the two Shotcrete layers. Sometimes, additives and steel fibers may be used as reinforcement.

2.7.2.4 Rock Bolts

During the tunnel construction rock bolts are used for face bolting, to increase face stability, or radial rock bolting, as preventive action against deformations.

Radial Rock Bolts:

Radial Rock bolts are part of primary support. Activating the composite action between the surrounding rock and the shotcrete, rock bolts contributes the load bearing capacity of the primary tunnel lining.

Rock bolts were applied at each second or third shotcreted top heading round behind actual excavated top heading face. The lengths and position of radial rock bolts are defined according to the rock types. Generally 3 meter long bolts were used. During the drilling operation, 3 meter rods are attached to each other by couplings to achieve entire length of rock bolt.

Sometimes, SDA bolts (self drilling anchor) may be used. This is because the bolt itself is used as the drill rod. Rod and bit remain in the hole after drilling operation as a rock bolt, which is grouted through the flushing hole. In case of collapsing boreholes, this system still enables the installation of rock bolts.

Face Bolts:

If required, to ensure the face stability, up to 15 m long rock bolts are installed to the excavation face. The face bolts at top heading are drilled each 8th or 9th top heading round according to the round lengths. The numbers of the rock bolts are defined depending on the conditions. *Figure 2.11* shows a typical face bolt pattern.

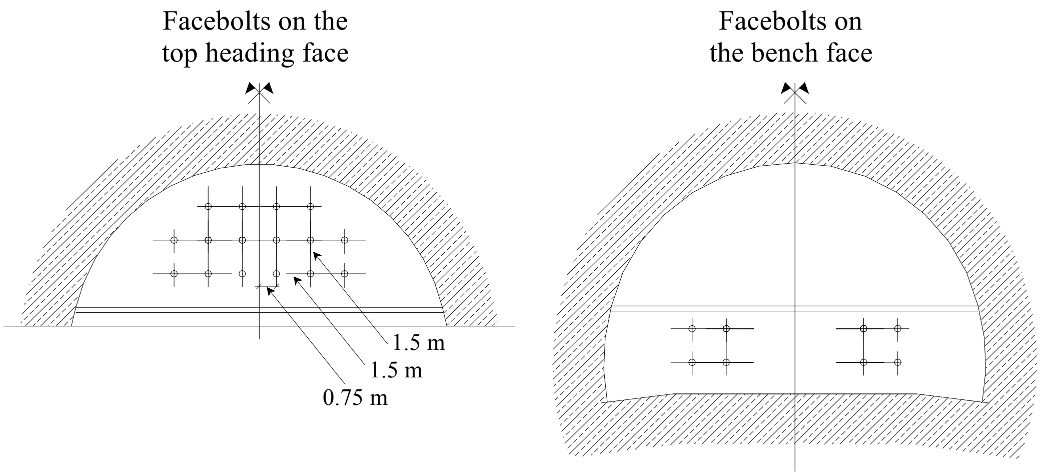


Figure 2.11 Typical Face Bolt Pattern (Astaldi, 2004)

Rock bolts are installed by three steps: drilling, grouting, and tightening. Rock bolt drillings are performed by a jumbo drill rig and using self drilling anchor (SDA). When drilling is finished, rod and bit remain in the hole. Bolts are grouted after completion of drilling operation by mortar pomp. It takes 8-12 hours for the grout to get cured. After then, the bolts are tightened with anchor plates and nuts.

2.7.2.5 Forepoling:

Forepoling is a pre-excavation support element required for the tunnel excavation works in rock and soil conditions, which tend to produce overbreak, collapses or material inflows immediately following excavation. Forepoling may be applied locally or systematically according to the circumstances required for the safety of the works and to prevent overbreak which may occur during top heading excavation.

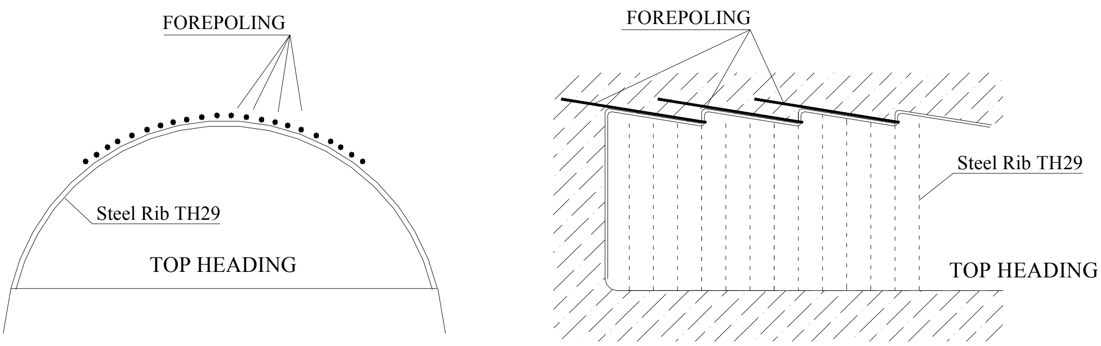


Figure 2.12 Forepolings (Astaldi, 2004)

Prior to every forepoling installation, the tunnel profile at top heading has to be stepped up by using four different sizes of steel ribs. The forepoling is applied in the roof of the tunnel above the steel rib in sub-horizontal (3 to 5 degrees upward inclination) position with gradient as flat as possible. (*Figure 1.12)*

According to the ground conditions 10~20 meters long grouted pipes or rock bolts are installed as forepoling. The former one known as pipe roof umbrella system and provides better protection against overbreaks.

2.7.2.6 Intermediate Shotcrete Invert

In order to decrease the high deformation rates, when required, intermediate shotcrete invert is installed. The final scope of intermediate invert is to slow down or to stop deformations of the Top Heading Shotcrete shell and to decrease the settlement rates of the top heading footings. (*Figure 2.13*)

For intermediate Shotcrete Invert ground must be excavated to certain level in an arch shape afterwards three layers of Shotcrete are sprayed and two layers of reinforcement mats are installed inside the Shotcrete.

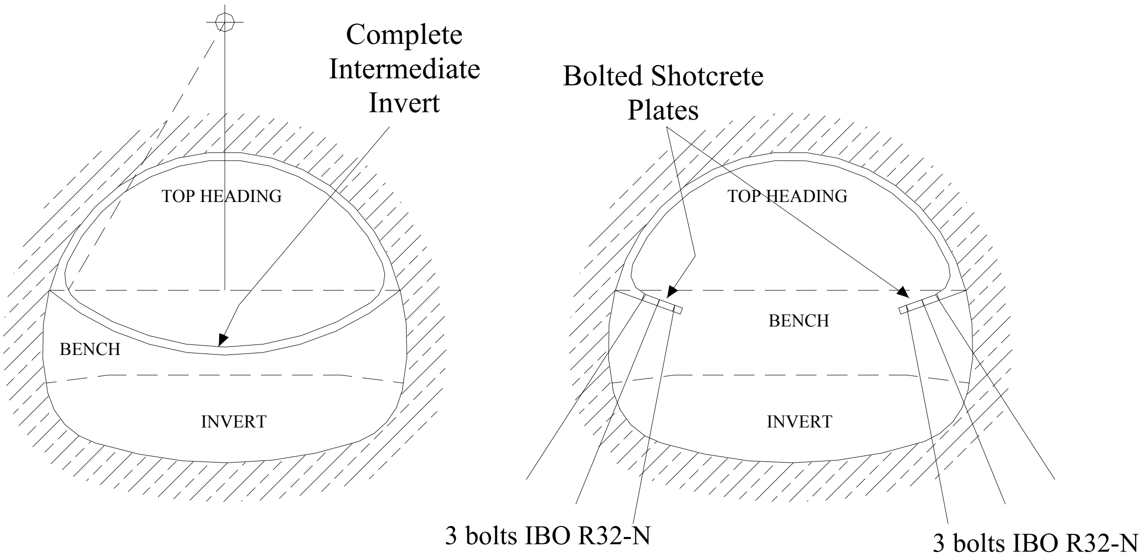


Figure 2.13 Complete Intermediate Invert and Bolted Shotcrete Plates (Astaldi, 2004)

2.7.2.7 Bolted Shotcrete (Footing) Plates

Instead of full intermediate invert, sometimes bolted shotcrete plates may be used. The bolted shotcrete plates (*Figure 2.13*) are utilized to decrease deformation rates at top heading footings. But they were mostly not used during the construction of new alignment.

2.7.2.8 Monolithic Concrete Invert

Monolithic Invert is the lower part of the cross section and the last excavated part of the primary support system. The final scope of Invert is to achieve a “ring” closure. The invert was performed as a reinforced monolithic concrete arch.

Depth of invert excavation is determined according to the applied support class. In case of water presence at invert level a drainage pipe can be installed along the cross section. When the invert excavation completed, minimum 10 cm thick NaSi shotcrete sealing have been applied to fill over excavations and stop underground water running into the excavation. After the shotcrete sealing, a steel formwork is placed in front of the invert excavation. Then a steel rebar system is applied as reinforcement. Finally invert concrete is poured in several layers of thickness 50 cm.

2.7.2.9 Intermediate (Bernold) Lining

The intermediate lining (Bernold) is a reinforced lining system (*Figure 2.14*) which is installed only in Option 3 and Option 4 support classes. Long term creeping movements of shotcrete lining after tunnel ring closure, which was the main problem of the most sections of the Bolu Tunnels, are solved by Bernold lining.

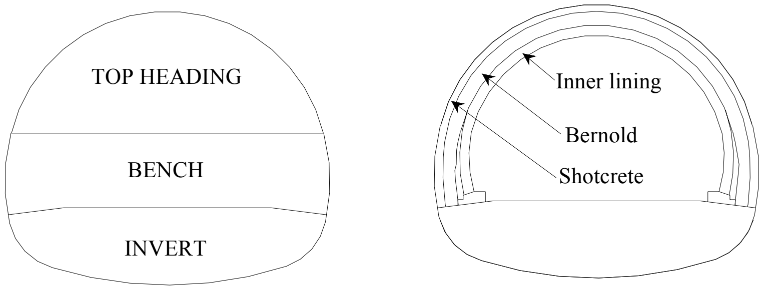


Figure 2.14 Intermediate (Bernold) Lining (Astaldi, 2004)

In Bolu Tunnels installation of intermediate lining at the previous sections were carried out at least at the third already hardened invert block and with a thickness of 60 cm.

For the Bernold lining, pre-cast panels laid on H profile steel ribs are commonly used as formwork. Those steel ribs are fixed in place by rock bolt extensions. Concrete is poured through apertures located along the precast elements. When the pouring concrete reaches its designed compressive strength, the steel ribs are removed but the precast panels remain embedded into the concrete. Then in order to fill possible voids behind the Bernold lining, contact grouting is applied at 2 bar pressure.

2.7.2.10 Foundation Beams

Foundation beams are longitudinal concrete beams. They perform as abutments for the Inner Lining. The foundation beams perform the connection link to the monolithic invert. Cable ducts and side walks are mostly founded on these longitudinal concrete beams.

At certain seismic active areas an additional seismic reinforcement is installed inside foundation beams. Seismic reinforcement is installed to attain larger strength in view of ground movements during earthquake event.

2.7.2.11 Inner Lining

The inner lining is a cast-in-situ concrete lining, increases the safety factor of the tunnel lining system, provides a uniform interior surface and improves the water tightness of tunnel lining. A smooth interior surface is required for airflow, aesthetic, lighting and maintenance reasons.

Inner lining used to be applied with steel reinforcement or un-reinforced. For inner lining installation a maximum monthly deformation rate of 3-5mm at Shotcrete lining is required.

To prevent leakage of groundwater into the tunnel and to avoid any contact between water and inner lining concrete is sealed by waterproofing membrane and protective felt. The waterproofing membrane provides the sealing function. The protective felt protects the waterproofing membrane against damage from contact with the shotcrete surface, to prevent interlocking between shotcrete support and final lining and to provide drainage into the longitudinal lateral drainages.

2.7.2.12 Further Activities

A) Groundwater Drainage System

The purpose of permanent groundwater drainage is to take away the groundwater collected by the waterproofing system. This drainage consists of perforated or slotted hard-PVC pipes embedded by Porous no-fines concrete. The perforated or slotted hard-PVC pipes are set down on mortar bed in the appropriate duct (covered by the protective felt) located at both feet sidewalls of the tunnels. The pipes are welded with the previous one by mean of waterproofing membrane.

Also if there is a water income in invert excavations, slotted elastic-PVC pipes are put in a trench on the excavation bottom and then the trench is filled by the porous no-fines concrete. Later, those slotted elastic PVC pipes are connected to main drainage system.

B) Seismic Joints

The scope of seismic joints is to absorb earthquake related ruptures and displacements without damaging the main structure. In inverts seismic joints were provided by foam concrete blocks. Several seismic joints are cutting (only in the Option 4 condition) the tunnel when crossing active faults.

C) Deformation Elements

Deformation elements were tried to be used to give deformation ability to thick (60 cm) shotcrete shell. Those were hollow boxes made of 4 mm thick steel plates. Placed along the round length, above the bench level those boxes were used to create space in shotcrete shell. Shotcrete shell were deforming freely without taking any damage by means of those deformation elements until temporary invert excavation. When second shotcrete shell was applied with temporary invert those deformation elements were removed and the remaining hole was filled by shotcrete.

D) Bench Pilot Tunnels

The BPT (bench Pilot Tunnel) pilots opened at bench level, in front of top heading advance, in Option 4 support class. Then backfilled by reinforced concrete to provide stiff support beam for top heading supports. It has its own steel ribs. Steel ribs may be used as full steel ring. These steel ribs were embedded in fiber reinforced shotcrete. (*Figure 2.15*)

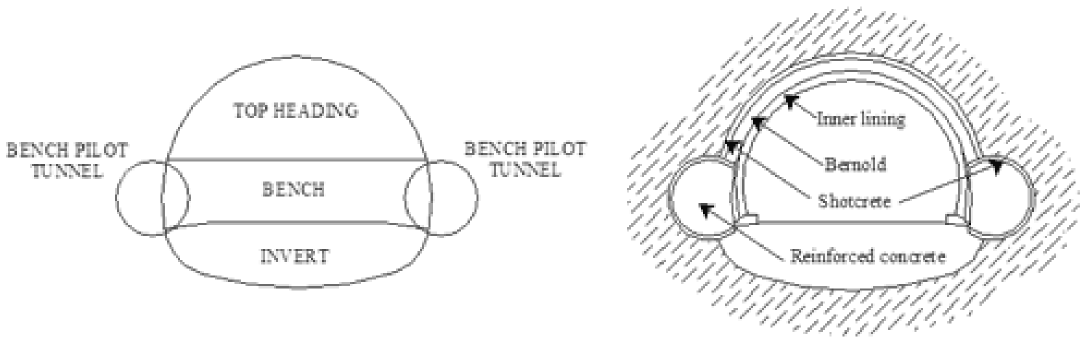


Figure 2.15 Bench Pilot Tunnels (Astaldi, 2004)

CHAPTER III

BOLU TUNNELS

3.1 Introduction

The Turkish section of the Trans-European Motorway runs from Edirne, near the Bulgarian/Greek border, to Istanbul and then on to the capital city Ankara is called as Anatolian Motorway. This motorway is a vital part of the Trans European Motorway project, which integrates a system of interconnecting roads in East Europe.

Gümüşova - Gerede Section (Section 2) of Anatolian Motorway (*Figure 3.1*) includes the Bolu Mountain Crossing (Stretch 2). The Bolu Mountain Crossing includes four viaducts of 2313, 1275, 358 and 664 meters and twin tunnels of 5800 meter total length. Total length of Stretch 2 is 25.5 Km. This is the most challenging portion of the motorway between Istanbul and Ankara. Construction of the section took more than 13 years due to the extremely difficult morphological, geological and seismic characteristics of the area as well as contractual disagreements and allocation problems.

At November 1999, Düzce earthquake stroke the area. The Project has been strongly affected from the disaster. Viaducts have some damage e.g. 6 pier foundations have been damaged, number of piers have small tilt or rotation and number of beams have been damaged. Approximately 250 m of the tunnel section have been collapsed in Elmalik (Ankara) side. After the earthquake a concrete bulkhead has been constructed in order to provide the stability inside the tunnel. Then, the works have been stopped in both tunnels at Elmalik side. The repairs of minor damages due to the earthquake at the Asarsuyu (Istanbul) side have been continued till November 2000.

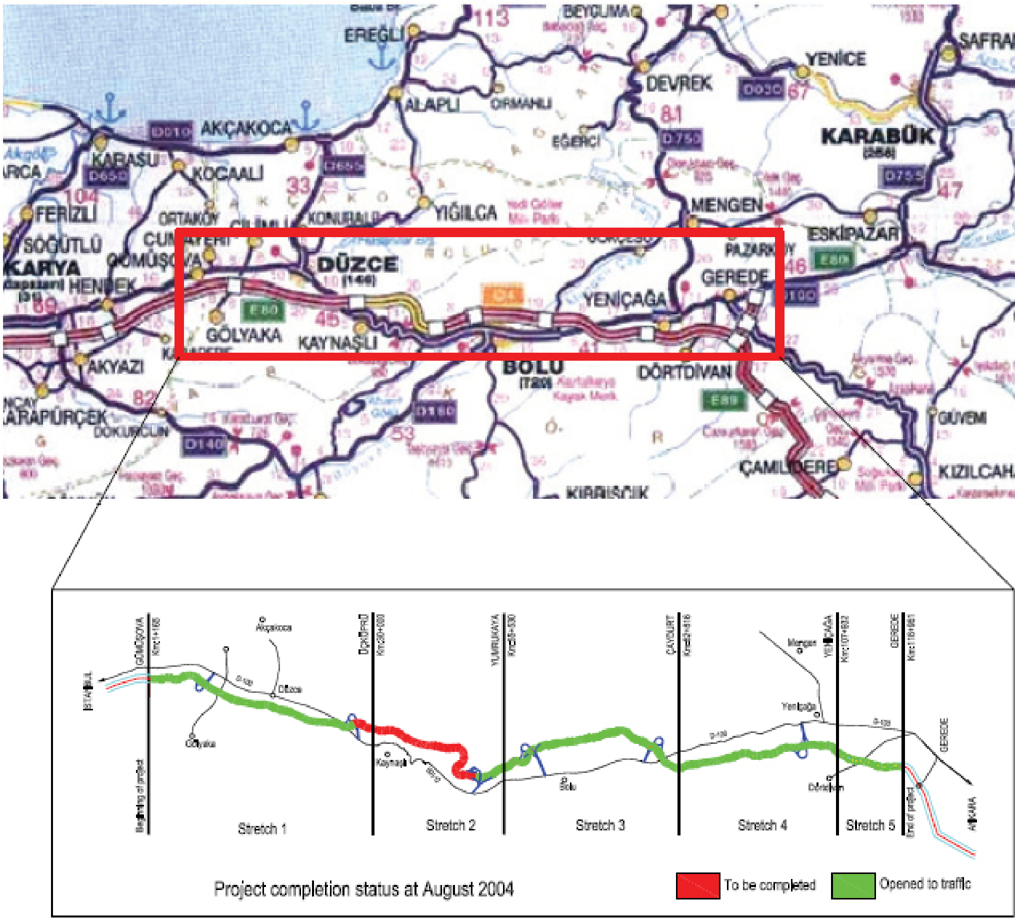


Figure 3.1 Map of the Site

Following the extraordinary event at 24.03.2000 the Employer (General Directorate of Highways) ordered to the Contractor to re-design the whole of the works according to new seismic design criteria in order to ensure the capacity to withstand without significant damages in case of a seismic event with 2000 years of recurrence period (instead of the 500 years previously adopted) and the possible flood hazard in the valley.

Since re-mining of collapsed tunnel section seemed time consuming and expensive, the employer requested to study of alternatives by-passing the collapsed section. At the end of 2000, alignment studies were finalized.

New tunnel alignment (*Figure 3.2*) mostly passing through weak rock, clay and fault gouge which is similar to the abandoned section. Therefore, support system has been revised according to the experience obtained from old section.

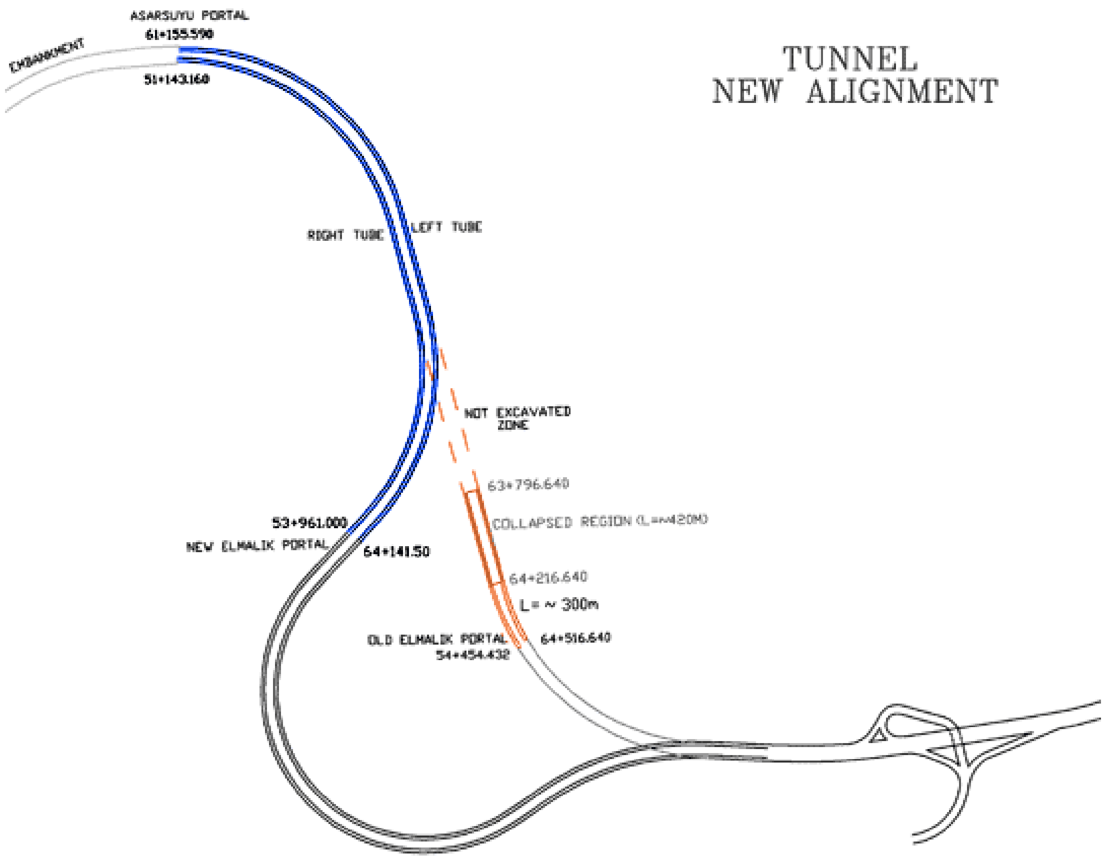


Figure 3.2 New Tunnel Alignment

Excavation of the new alignment started in August 2004. Approximately 1100 m tunnel work is completed in a year. Left tube excavations met at 26th of August 2005. Right tube excavation met at 4th of September 2005. After completion of emergency crossings and electro mechanic works, both tunnels opened to traffic at 5th of May 2007.

In this study, in order to justify the applied support system, relation between rock-mass properties, support design and deformation behavior has been observed during the excavation of Bolu Tunnels New Alignment Elmalik Portals. (Right tube chainages 53+747 and 53+961 and left tube chainages between 63+779 and 64+141.00) In order to accomplish this task, literature survey was carried out, followed by a comprehensive field study.

3.2 Characteristics of the tunnels

Bolu Tunnels crosses through the tectonically active area and faults which are in connection with North Anatolian Fault. The North Anatolian Fault is one of the most energetic earthquake zones in the world. Since the disastrous 1939 Erzincan earthquake, there have been several earthquakes related to the North Anatolian Fault, measuring over 7.0 on the Richter scale, occurred. (Wikipedia.org) November 12th, 1999 Düzce earthquake (M_w=7.1) caused substantial damage to the Bolu Tunnel and viaducts, which were under construction at the time of the earthquake. *Figure 3.3* shows the Geology of the Elmalik Side of the Tunnel.

Bolu Tunnels includes the two parallel tubes, located between Ch. 61+187 and Ch. 64+141 (Left Tube) and Ch. 51+172 and Ch. 53+961 (Right Tube). Each tube has 14.0 m with and 8.60 m height with a finished cross-section of 98 m^2 to provide 3-lane (3 x 3.75 m) wide motorway access. Even though practical tunnel lengths are longer due to the portal structures and cut and cover tunnels, lengths of NATM tunnels are 2963 m (left tube) and 2789 (right tube), with a slope of 2%.

In 1990 Astaldi contracted Geoconsult to guide the investigations and provide the design for the Bolu tunnels. In addition, a representative of the designer at the site was foreseen to consult the contractor for determination of rock classes and required support measures, interpretation of monitoring results, detail adaptation and modification of the design. Geoconsult prepared the Tunnel design according the NATM, following the Austrian Standard ÖN B2203, and requirements of the General Directorate of Highways (KGM, State Highway Authority).

According to the tunnel design, the excavation cross section varies between 170 m^2 and more than 250 m^2, depending on the support class. Axial distance between the two tubes is varying from 30 to 60 m according to the geometric design and rock conditions. Average distance is 55 m, leaving a pillar of 40 m between the two tunnels. Tubes have been excavated from 4 portals, or faces, in two locations. Those are Asarsuyu Valley (Istanbul side) and Elmalik Village (Ankara side). This study focused on the support design at Elmalik Portals. At Elmalik Portals driven parts variations of CM support class (CM35, CM45 and Option3) have been used. Typical cross section of CM support class is shown in *Figure 3.4.* For other design drawings you may refer to *Appendix II – Project Drawings.*

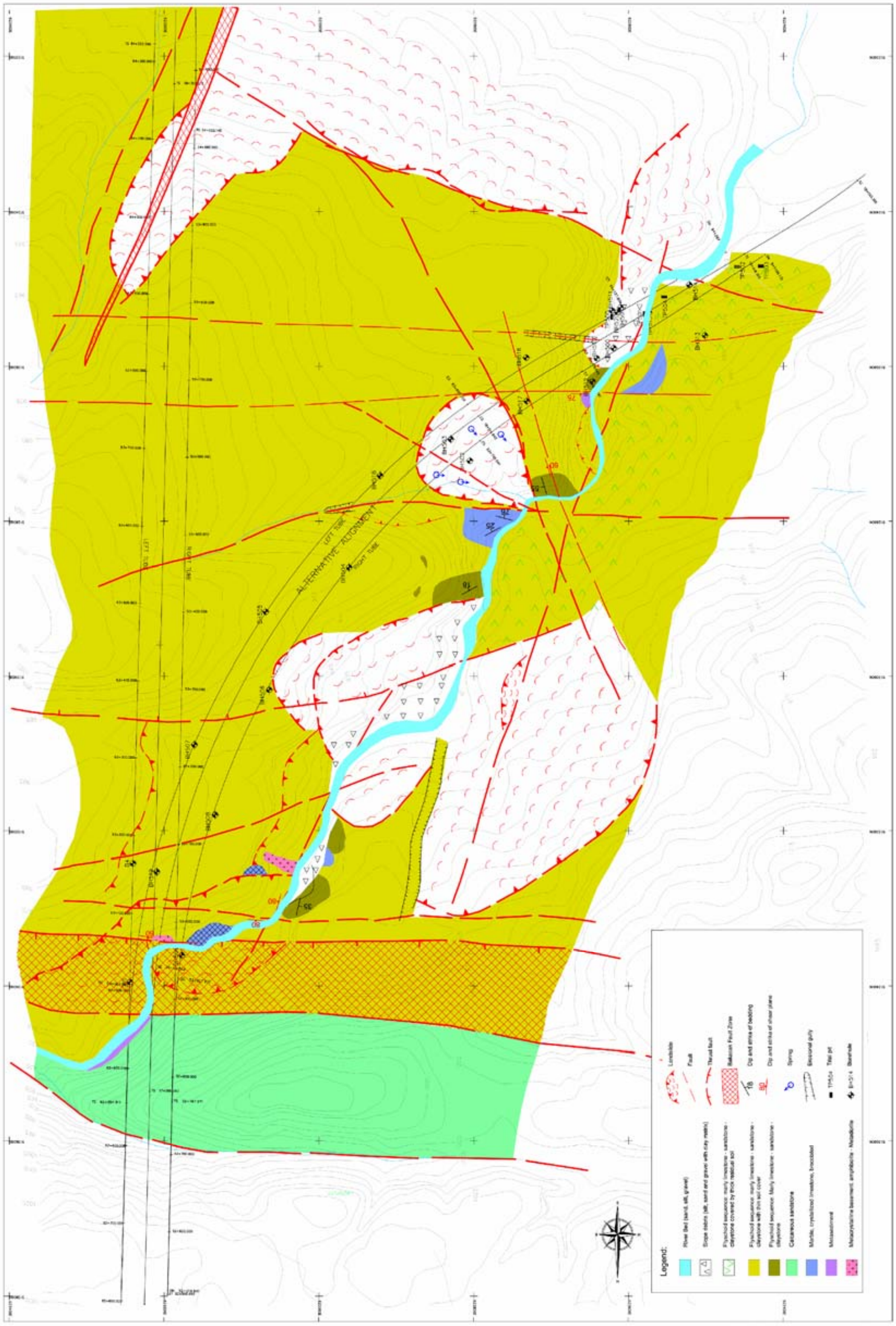


Figure 3.3 Bolu Tunnel Re-Alignment Geological Map (Technical Drawing 45.110/001)

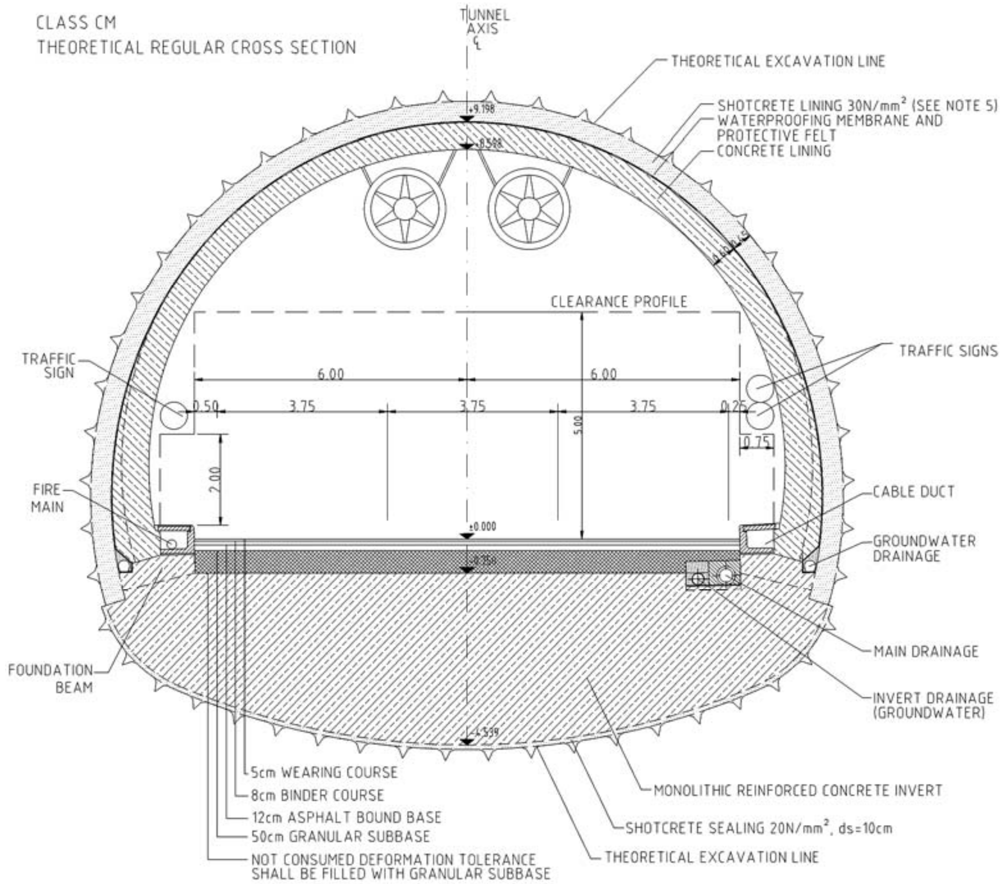


Figure 3.4: A Typical Cross Section of the Tunnel at Elmalik Portal for CM Class (Technical Drawing TN/TUG/D/LO/208 Rev.1)

3.3 History of Bolu Tunnels

Tunnel excavation started in 1993 at the Asarsuyu portal (*Figure 3.2*) in the west. The first 700 m of tunnel were excavated through the metacrystalline basement with brecciate, crushed fault material in patches. Expected range of designed rock classes ranging from A2 to C2 had been applied successfully, with only minor adjustments. However, in the following section of approximately 150 m, when tunnel excavation enters into severely faulted and fractured metasediment formation, large movements of 1 m and more occurred which led to severe damages to the primary lining and buckling of steel ribs accompanied with invert heave. In order to increase support capacity and obtain rapid support reaction to control unexpectedly high loads, rock bolting was increased to more than 370 m per round and heavy steel ribs were installed. However, the movements showed a more or less constant trend, which could only be stopped with ring closure. Consequently, the ring closure distance, that is the distance between top heading face and invert concrete, was reduced to 50 m. However, extensive re-profiling measures and replacement of heaved concrete invert were required.

In late 1995 tunnel advance had been stopped and the tunnel design was reconsidered. The shape of the invert was changed to a deeper invert, support measures were increased significantly in terms of capacity, number and type of rock bolts, and deformation elements were introduced. Tunnel excavation in Asarsuyu was resumed in September 1996.

Excavation at the east side (Elmalik portal) started in 1994. In the first 200 m, the tunnels were excavated within a series of tectonised siltstones and marls with intercalations of limestones. After about 200 m the tunnel encountered a flat transition zone between sediments in the hanging wall and metacrystalline basement rocks in the foot-wall. The transition zone was dominated by clayey fault gouges with residual shear parameters of $c = 10$ to 60 kPa and $\varphi = 10$ to 15°. With an overburden of approximately 70 m and a thickness of the fault gouge of up to 50 m the tunnel behavior of this section was characterized by excessive settlements of more than 1 m and longitudinal movements of up to 140 mm towards the portal. Special design solutions had to be developed to pass this fault gouge zone. Although this fault zone was finally passed successfully in 1998, similar problems related to low angle fault zones have continued to be encountered. Therefore it was decided to drive a pilot tunnel in the left tube from the Elmalik (Ankara) side towards Asarsuyu and to obtain geological and geotechnical date.

Based on the findings of this pilot tunnel and the experience from already excavated section, to take the highly squeezing nature of the ground and its insufficient self bearing capacity into account, two design solutions, called "Option 3" and "Option 4" have been developed.

In Option 3, the excavation is performed in a top heading-bench-invert sequence with 40 cm shotcrete and rock bolts. The ring closure distance between the top heading face and the heavy, 4.5 m thick reinforced monolithic concrete invert block has been minimized to approximately 25 m. Once the massive concrete invert is installed, an intermediate in situ cast concrete lining of class B40 is installed to finally stabilize the tunnel and to stop long term creep movements. This intermediate lining is installed behind precast concrete elements with a thickness of 4 cm in 4.4 m long stretches, which equals four rounds of advance. The precast panels are reinforced and are supported by heavy steel ribs. Simplified general layout of class Option 3 excavation and support is given in *Appendix II - Project Drawings*.

Option 4 is applicable to clayey fault gouges with only residual material parameters and has been applied the first time successfully in 1998 in passing the low angle fault zone encountered on the Elmalik side. The main concept of Option 4 is the precreation of stiff abutments for the top heading prior to top heading advance. For this reason, two 5 m diameter circular pilot tunnels are excavated at bench level prior to excavation of the main tunnel. After excavation of this two "bench pilot tunnels", they are backfilled with reinforced B40 concrete. The following top heading advance is supported by an initial shotcrete lining of 40 cm. Rock bolts are not foreseen as standard measure. Between 8 and 12m behind the top heading face, similar to Option 3 an intermediate concrete lining is installed on top of the bench pilot tunnels concrete fill. Simplified general layout of class Option 4 excavation and support can be seen in *Appendix II - Project Drawings*.

Both newly developed design solutions proved to be quite successful, however in 1999 two disastrous earthquakes struck Turkey.

3.3.1 Earthquakes

The first earthquake with a magnitude of M_w = 7.4, named Kocaeli (17 August 1999) quake with its epicenter close to İzmit, was located about 150 km west of the construction site. Whereas extensive damages occurred close to the epicenter (thousands of buildings fully or partly collapsed, more than 17000 people lost their lives), the tunnels and viaducts of the project suffered no damage. During this earthquake, structures at the construction site received accelerations between 0.2 and 0.3 g.

The second earthquake occurred on 12 November 1999 with its epicenter in Düzce with a magnitude of Mw = 7.1. Epicenter was at only about 20 km west of the site. In the vicinity of the site 0.6 to 0.8 g accelerations could be measured. This was exceeding the design consideration of 0.4 g. As a result of this earthquake, the sections of the Elmalik tunnels where only primary lining installed were collapsed. However, sections with completed inner lining concrete, there were only some longitudinal and circumferential cracks in the inner lining observed. Despite an overburden between 50 and 140 m, at the surface cracks and craters developed. A precise surface leveling showed that along the tunnel alignment, for about 250 m for each tube, there were settlements of more than 1 m. This indicates the collapse or severe damages.

On the Asarsuyu side of the tunnel, sections without inner lining suffered spalling and shear cracks at shotcrete and longitudinal cracks in the monolithic concrete invert. At the startup section of the first Option 4 application in Asarsuyu, where another low angle fault zone, the "Bakacak Fault", had to be crossed, the bench pilot tunnels were damaged extensively, which were already advancing through the fault zone, at the time of incident.

3.3.2 After the Earthquakes

As a consequence of the Düzce earthquake, a reassessment of the seismicity of the Bolu region was performed. According to the reassessment the design earthquake load was increased. Moreover it was found that the Bakacak fault in Asarsuyu is seismically active. Therefore in order to provide the required flexibility of the inner lining in case of another earthquake, the inner lining and the invert block length within the seismic active fault was reduced to 4.4 m and "seismic joints" are introduced. With this measure the shear resistance is reduced and, differential movements of the individual inner lining and invert segments in longitudinal and transversal direction are allowed.

Within the seismic joints the ground is only supported by the initial shotcrete lining. 50 cm clearance was left between inner lining blocks. In the invert, the seismic joints are filled with foam concrete bricks. As the seismic activity of the Bakacak fault (*Figure 3.3*) had not known before the earthquake, bench pilot tunnels passing seismically active zone had already been filled with concrete. Therefore, seismic joints in the bench pilot tunnel concrete were to be created by large diameter core drilling.

For the collapsed area of the Elmalik side of the Bolu tunnel, several design proposals for remedial measures were investigated, such as extensive grouting measures followed by re-excavation under application of Option 4. However, due to several uncertainties regarding the extent of the collapse, damages and movements to the massive concrete invert, it was decided to abandon the already excavated sections at the Elmalik portals and bypass them by realigning the Bolu tunnel.

The chosen realignment (*Figure 3.2*), starting with an approximately 270 m long cut and cover section, bypasses the Elmalik section by a wide curve and meets the original alignment in the Asarsuyu section before entering the Bakacak fault. The new alignment passes through quite similar ground conditions as already encountered along the old alignment.

Excavation of the new alignment started in August 2004. Excavation and support of the Left Tube main tunnels have been completed without any major problem in September 2005.

(Pictures taken during the various excavation and support works I Bolu Tunnels can be seen in *Appendix III – Site Pictures*.)

CHAPTER IV

ROCK PROPERTIES & DEFORMATION MEASUREMENTS

4.1 General Geology

The Bolu Tunnel re-alignment is located between the Asagibakacak and the Elmalık faults. (*Figure 4.1*) Those are active faults and create the structural and kinematic connection between the southern branches of North Anatolian Fault Zone (NAFZ) and the Düzce fault. The Asagibakacak fault forms the tectonic boundary between Asarsuyu and Elmalık domain. The rock formations encountered in Asarsuyu domain are included in Pontid tectonic block, whereas the rock formation encountered in Elmalık domain are included in Armutlu-Ovacik regional tectonic block. The Pontid tectonic block (Asarsuyu domain) thrust over the Armutlu-Ovacik (Elmalık domain) during late Cretaceous-Eocene. Also, several secondary thrust faults within the Elmalık domain and the contact fault (low angle), between the metacrystalline basement rocks and overlain sedimentary rocks are present in the study area. The several younger sub-vertical faults, which occurred parallel to NAFZ, have displaced the trust fault zones.

Two main lithological units were encountered in Asarsuyu domain. A longer portion of the Asarsuyu side was excavated in metacrystalline basement rocks, which consist of slightly to medium weathered metagranodiorite, metadiorite and amphibolite. The remaining part of the Asarsuyu side was excavated within the metasedimentary rocks. These rocks contain gray colored metasiltstone, metasandstone, crystallized limestone and quartzite. The brecciate marble and calcareous sandstone are encountered as huge shear bodies in the metasedimentary rocks. Due to the intensive faulting in the area the rock masses are in crushed and intensively fractured conditions. The contact between the metacrystalline basement and the metasedimentary rocks is formed by low angle fault.

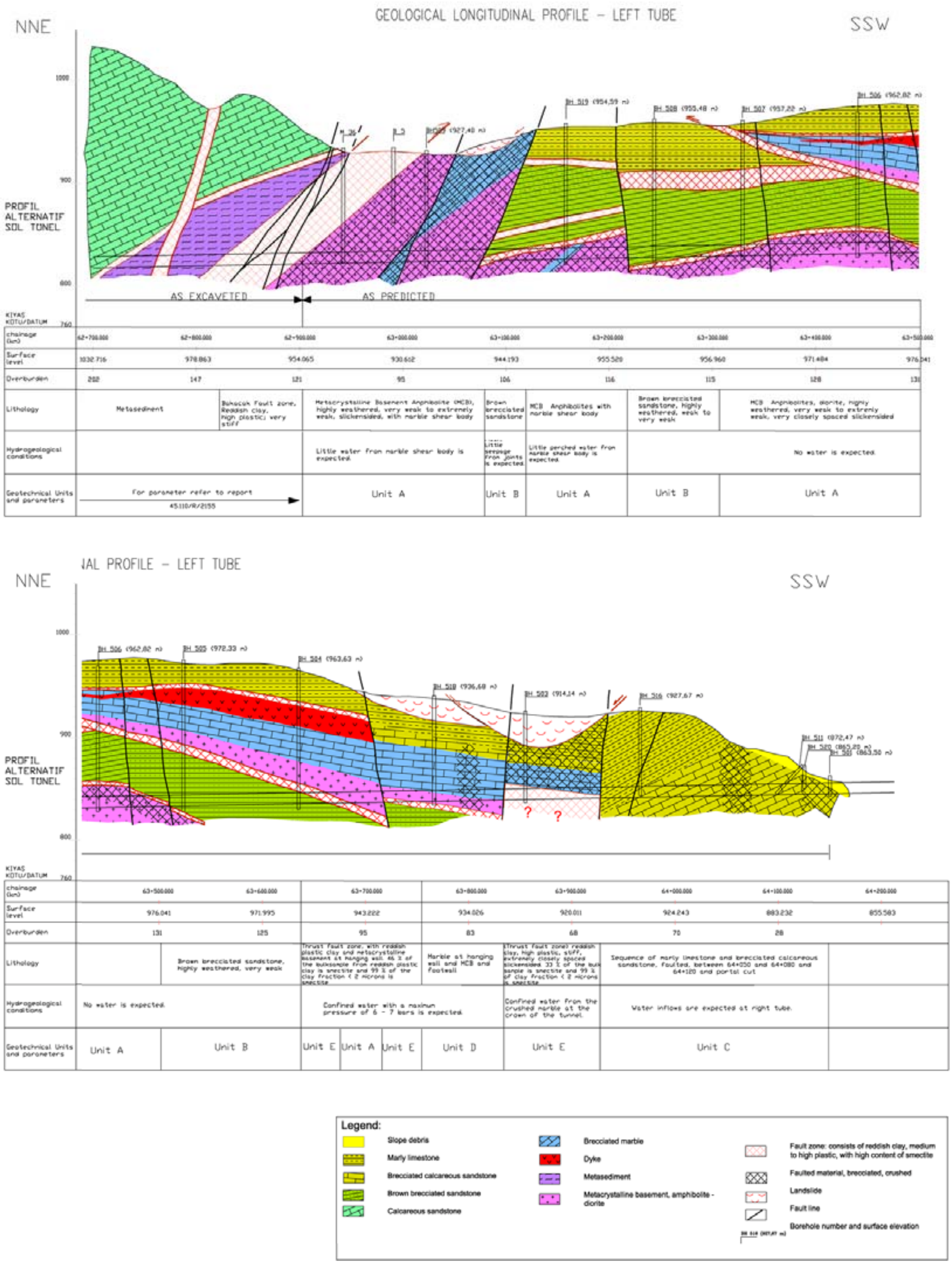


Figure 4.1 Bolu Tunnel Re-Alignment Geological Longitudinal Profile (Technical Drawing 45.110/002)

The rock masses have been encountered along new tunnel alignment in Elmalik domain consists of flyschoid sequences, marble shear bodies and metacrystalline basement. Flyschoid rock mass is constituted from top to down by following sequences:

- Brown to gray colored marly limestone-limestone, calcareous sandstone and clay stone.
- Gray colored limestone-crystallized limestone and marble
- Brown to gray and reddish brown colored brecciate sandstone. This unit covers the longer portion of the tunnel to be penetrated along new alignment.

The flyschoid rock mass overlies the metacrystalline basement (mainly amphibolites) by the tectonic contact. This zone contains medium to highly plastic fault gouge clay with few meters thickness.

The re-alignment starts for end of Asagibakacak fault zone in Asarsuyu side. This zone consists of red colored highly plastic clay and extremely slickensided discontinuity surfaces. Discontinuity analysis of the data collected during excavation indicates two different dip orientations of shear planes. The mean values of these 2 sets are 358/36 and 354/65.

The first one represents the general trend of the core zone and is most likely developed by the thrust fault between Pontid and Armutlu-Ovacik zone. The second one represents the Bakacak fault zone.

The sub-vertical faults encountered in the area strike mainly E-W, NE-SW and NNW-SSE with dip angle of approximately 80 degrees.

4.2 Geological Conditions along Tunnel Alignment

With respect the site investigation results, the rock masses crossing the tunnel profile have been characterized. Properties of the main geological units are as follows:

Bakacak Fault Zone

The Bakacak fault zone was encountered from chainage 62+840 to 62+900. The fault zone consists of reddish clay, high to medium plastic, with very closely to extremely closely spaced discontinuities. All discontinuity surfaces are silckensided. Two main discontinuity surfaces are recorded during excavation.

Metacrystalline Basement

The rock mass is composed of amphibolites with white colored quartzite and marble shear bodies. The amphibolites are brown to green and grey colored, medium to highly weathered, extremely weak to very weak. The size of quartzitic shear bodies range from several decimeters to several meters. The strength properties of the quartzite are the range of medium strong to very strong.

Amphibolites are intensively sheared and the discontinuity surfaces are characterized by undulated and slickensided surfaces with clay coating. The dip angles of discontinuities are mainly steep.

Marble shear bodies are fresh, medium strong to strong with closely spaced discontinuity. Confined water bodies are expected in marble and quartzitic shear bodies. These shear bodies and sub-vertical fault zones are likely in contact with Elmalik River. Carsifications typically in the range of some centimeters are developed along the fractures and they are filled with reddish clay. The proportion of reddish clay in brecciate marble is approximately 30 to 50%.

Brown Sandstone, Brecciate

The brecciate sandstone is medium to highly weathered, weak to very weak and the rock mass contains widely to very widely spaced discontinuities. Discontinuity surfaces are characterized by planar, undulating and silckensided surfaces with clay coating.

Thrust Fault Zone with Metacrystalline Basement

The metadiorite rocks are slightly weathered and weak. The discontinuities are very closely to closely spaced and discontinuity surfaces are characterized by undulated and silckensided surfaces with clay coating.

Brown colored thrust fault zone consists of highly plastic clay with low residual shear strength. It is extremely silckensided and is very stiff to hard. The bottom of thrust fault zone is made up of brown brecciate sandstone.

Grey Marble and Metacrystalline Basement

The metacrystalline basement consists of amphibolite and metadiorite and is slightly to medium weathered, very weak to medium strong. Discontinuities in metacrystalline basement are represented by shear joints, which are characterized by planar and silckensided surfaces with clay and limonite coating. Shear joints dip with nearly vertical angle and are moderately widely spaced.

The marble layer is grey, white and pink colored. It is slightly weathered to fresh and is medium to strong to strong. It mainly shows brecciate texture. Discontinuity surfaces are characterized by planar and smooth surfaces with clay coating. Joins dip with sub-vertical angle and are very closely to moderately closely spaced. The confined water with a maximum pressure of 6 – 7 bars was expected from marble layer, which is most likely connected to Elmalik River.

Reddish Brown Fault Gouge Clay

At the tunnel level, contact faults containing reddish brown clay have been encountered. It is high to medium plastic, stiff to hard and contains extremely silckensided surfaces with very low residual shear strength parameters.

Marly Limestone – Brecciate Calcareous Sandstone.

Elmalik portals driven part of tunnels have mainly been excavated through the typical flyschoid sequences which consist of marly limestone – calcareous sandstone and claystone. The mean orientation of this sequence is 080/48 (dip directions/dip angle). The marly limestone and brecciate calcareous sandstone are slightly to medium weathered, weak to medium strong, whereas claystone layers are highly weathered, very weak to extremely weak. Therefore, rock mass is formed as blocky material with fine matrix. Fine matrix is composed by clayey sandy silt.

The brown colored brecciate calcareous sandstone is the dominant unit in this section and is strongly affected by faulting parallel to Elmalik River at nearby portal area. The rock unit is disintegrated and crushed at the portal area. The rock unit consists of very closely to moderately spaced discontinuities, which are characterized by undulated and rough surfaces and are filled with mainly silty clay and limonite coating.

4.3 Hydrogeological Conditions

The Main water inflow has been expected from marble, marly limestone and its shear bodies. The marble shear bodies to be encountered within the metacrystalline basement bear mainly perched water with limited amount. However, the marble and marly limestone layers, which spread in large extent, bear the confined water table with considerable amount. These layers extend till Elmalik River that can create a water path to the tunnel excavation. It was expected that these layers to bear considerable amount of confined water with a pressure pf approximately 7 bars.

The artesian water encountered in the boreholes between 35 to 61 meters depth from ground surface confirms the connection of the marble layer and the River.

The artesian water raised more than 6 meters from ground level. The raising head permeability test was carried out between 0 and 2 meters depth and the result indicates the permeability value of $k = 3.99 \times 10^{-6}$ m/s.

It was therefore suggested to take special measures to control the confined water during excavation of tunnel and for long term of the tunnel life between chainage approximately 63+720 and 63+950.

In addition to this, since the tunnel level is below the Elmalik River, considerable amount of continuous water flow has been expected through the jointed and faulted marly limestone, calcareous sandstone sequences between chainage 63+950 and Elmalik Portal during excavation.

Water inflows will mainly effects the right tube. No considerable water inflow is expected from Eastern side of the alignment.

4.4 Geological and Geotechnical Model

The geological mapping and drilling results clearly revealed that the main part of the alternative tunnel alignment would be excavated in the brown to grey colored brecciate sandstone and amphibolites called as Metacrystalline Basement (MCB). A significant part of the remaining tunnel to be excavated has been estimated to driven in the flyshoid series. The reddish colored medium to highly plastic fault gouge clays, which are found at low angle thrust fault zone, would also be excavated around change 63+830 and 63+910. The marble shear bodies within the metacrystalline basement as well as its strata are also be encountered in the tunnel.

Details of the laboratory tests and field pressuremeter test results, which have been performed for each geotechnical unit, are given at following pages by the kind permission of Astaldi S.p.A. The summary of the results are given in *Tables 4.1, 4.2, 4.3, 4.4, and 4.5.* The entire in-situ & laboratory test results are in *Appendix II.*

UNIT A – Metacrystalline Basement (MCB)

Brown and green colored amphibolites with quartzite and grey marble shear bodies. Face instabilities and overbreak are expected where rock mass requires blasting. The brittle and squeezing rock mass conditions can be expected.

UNIT B - Brecciate Sandstone

Brown and Reddish brown colored brecciate sandstone with clay intercalations. The sandstone is in generally uniform conditions. The blasting will mainly be required for the excavation. The squeezing ground conditions will be encountered.

UNIT C – Marly Limestone – Brecciate Calcareous Sandstone

This rock mass is represented by sequences of brown colored calcareous sandstone, marly limestone and clay stone with clayey matrix. The intact rock sourced from marly limestone and calcareous sandstone is rather strong. However, the rock mass is in loosened blocky structural conditions due to the content of clayey matrix, which resulted by disintegration of claystone layers. The highly sheared and weathered part of the brecciate calcareous sandstone has been estimated to be encountered at the portal area and between the chainages 64+040 and 64+080.

UNIT D – Brecciate Marble

Grey, white and pink colored brecciate marble, mainly shows pore structures and is partly karstified along weakness zone. It has been estimated to encounter the marble as shear bodies in Metacrystalline Basement (MCB) and also around chainage 63+750 to 63+800.

Discontinuity controlled block failures can be expected during excavation. The friable rock mass conditions will likely be encountered.

Rock Parameters and Properties

Rock mass classification of main units and Geological strength index (GSI) values are given in *Table 4.1*. GSI values have been estimated from field observation of blockiness and discontinuity surface conditions according to Hoek and Brown (Hoek, 2000).

Table 4.1 Rock Mass Classification of Geotechnical Units
According to Hoek & Brown (Şimşek, 2001)

Geotechnical Unit		Structure	Surface Conditions	GSI
UNIT A	MCB	disintegrated	very poor	10 – 20
UNIT B	Brown brecciate sandstone	disintegrated	poor – very poor	15 – 25
UNIT C	Marly Limestone – Calcareous Sandstone	blocky / disturbed	poor – very poor	10 – 30
UNIT D	Marble	very blocky	fair - poor	30 – 40

Structure: blocky, very blocky, blocky/disturbed, disintegrated
Surface Conditions: very good, good, fair, poor, very poor

Rock mass properties of the four geologic units are presented in Table 4.2. The Uniaxial strength (UCS, σ_{ci}) values were extracted both from point load index (using *equation 4.1*) and uniaxial compressive strength tests. Detailed result of both tests can be found at *Appendix II* of this manuscript.

$$\mathrm{UCS} = 24 \times \mathrm{ls}(50) \tag{4.1}$$

Hoek Brown Constant m_i is calculated according to Brown failure criterion (Hoek, 2000) using *equation 4.2*. Where σ_1' and σ_3' are the maximum and minimum effective stresses at failure, $x = \sigma_3'$ and $y = (\sigma_1' - \sigma_3')^2$, for n specimens:

$$m_i = \frac{1}{\sigma_{ci}} \left[\frac{\sum xy - \left(\sum x \sum y / n \right)}{\sum x^2 - \left(\left(\sum x \right)^2 / n \right)} \right] \tag{4.2}$$

Table 4.2 Intact Rock Properties (Şimşek, 2001)

Geotechnical Unit		Density γ (kN/m^3)	σ_{ci} (MPa)	m_i
Metacrystalline Basement (MCB)	Min		1	5
	Average	22-23	4	7
	Max		12	9
Sandstone, Brown	Min		2	6
	Average	21-23	8	8
	Max		20	10
Marly Limestone - Calcareous Sandstone	Min		30	8
	Average	23-24	100	10
	Max		170	12
Marble	Min		17	8
	Average	24-26	55	10
	Max		124	12

σ_{ci}: Compressive strength of intact rock
m_i: Intact rock material constant

By using the intact rock properties given in *Table 4.2*, the corresponding rock mass properties, presented in *Table 4.3* have been determined using *equations 4.3* to *4.6*. (Hoek, 2000)

$$\sigma_1' = \sigma_{cm} + k\sigma_3' \tag{4.3}$$

Where σ_{cm} is the uniaxial compressive strength of the rock mass and k is the slope of the line relating σ_1' and σ_3'. Values of φ' (friction angle) and c' (cohesive strength) can be calculated from:

$$\sin\varphi' = \frac{k-1}{k+1} \tag{4.4}$$

$$c' = \frac{\sigma_{cm}(1-\sin\varphi')}{2\cos\varphi'} \tag{4.5}$$

Deformation modulus, E_m, have been found according to Serafim's and Pereira's (1983) equation (*equation 4.6*). This equation is modified to work with GSI instead of RMR by Hoek (2000)

$$E_m = \sqrt{\frac{\sigma_{ci}}{100}}10^{\left(\frac{GSI-10}{100}\right)} \tag{4.6}$$

Table 4.3 Rock Mass Properties (Şimşek, 2001)

Geotechnical Unit		φ (˚)	c (MPa)	Em (MPa)	σ_{cm} (MPa)	G (MPa)	GSI
Metacrystalline Basement (MCB)	Min	15	0,01	100	0,0	38	10
	Average	18	0,05	267	0,2	103	15
	Max	21	0,21	616	0,6	237	20
Sandstone, Brown	Min	17	0.02	188	0.07	72	15
	Average	21	0.15	533	0.4	193	20
	Max	25	0.44	1060	1.4	408	25
Marly Limestone - Calcareous Sandstone	Min	17	0.34	547	0.9	210	10
	Average	23	1.9	1778	5.5	684	20
	Max	28	5.0	3162	17.4	1216	30
Marble	Min	24	0.47	1303	1.5	501	30
	Average	29	1.9	4170	6.5	1604	40
	Max	33	5.5	10000	20	3846	50

φ: friction angle, c: cohesive strength, Em: deformation modulus, σ_{cm}: rock mass shear strength,
G: shear modulus, $G = E / (2 \times (1+\nu))$, $\nu = 0.3$

UNIT E - Contact Thrust Fault Zone / Reddish Brown Fault Gouge Clay

This zone contains mainly brown and reddish brown colored fault gouge clay. It is very stiff to hard and medium to high plastic. Fault gouge contains very closely spaced slickenside surfaces. The thickness of the zone ranges from 2 meters to more than 20 meters.

Plasticity index of reddish brown fault gouge range from 25% to 69% with an average value of 48%. Shear strength tests results are plotted on shear strength versus normal stress graphic on *Figure 4.1*.

The resulting trend line indicates the shear strength values as follows:

Peak Cohesion	$c_p = 190$ kPa	Residual cohesion	$c_r = 91$ kPa
Internal friction angle	$\varphi_p = 8^\circ$	Residual internal friction	$\varphi_r = 6^\circ$

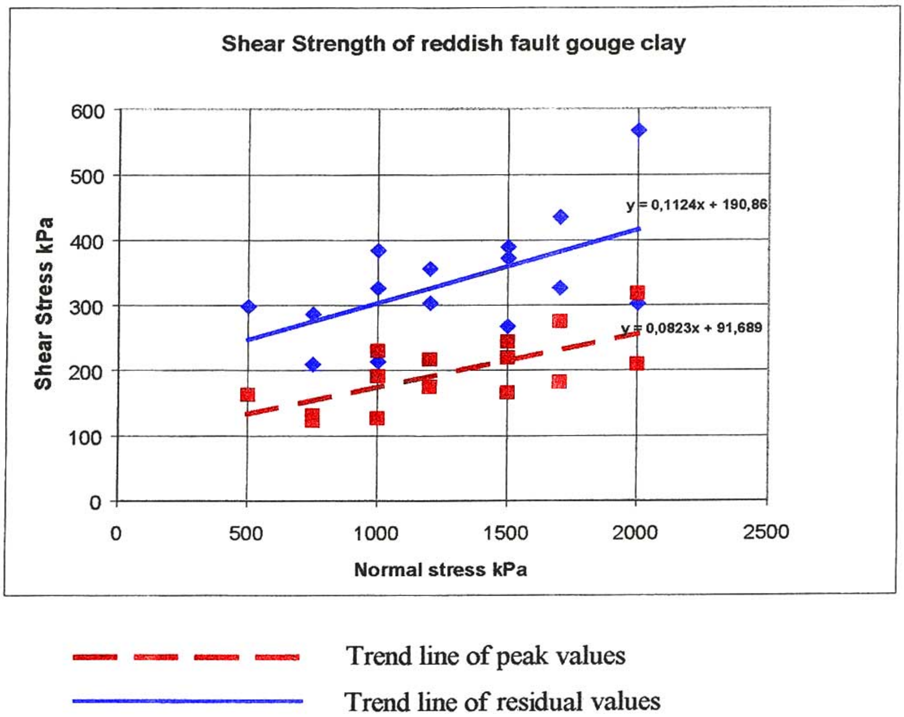


Figure 4.2 Shear Strength Values of Reddish Fault Gouge Clay (Şimşek, 2001)

Internal friction angle (peak and residual) are found rather low and cohesion is relatively high. It is assumed that this is due to quick shearing of high content of smecite in fault gouge clay mineral. However, since this fault gouge clay is very similar to one encountered at collapsed tunnel in main fault gouge zone same parameters are used which are:

Internal friction angle	$\varphi_r = 9 - 12^\circ$
Residual cohesion	$c_r = 90$ kPa
Stiffness	$E_m = 150 – 200$ MPa (drained)

Pressuremeter Test Results

The volume displacement system was considered for the calculations of shear stiffness. (*Equation 4.7*)

$$G = V (\Delta P / \Delta V) \tag{4.7}$$

Where V is the current volume at the start of the unlading curve and ΔP / ΔV is the unloading slope of the pressure-volume curve, from the stationary point to the current point of interest. Young modules are calculated from G (shear modules) considering $\upsilon = 0.3$. (*Equation 4.8*) Results are given in *Table 4.4*.

$$E = G (2(1 + \upsilon)) \tag{4.8}$$

As can be seen from *Table 4.4*, Young modules calculated for the crushed (poor) part of Unit C are found to be reasonable whereas, the values for Unit A and unit B is considerably lower than our expectation according to visual inspection and the calculation by Hoek and Brown failure criteria.

Table 4.4 Pressuremeter Test Results (Şimşek, 2001)

Borehole / Test No	Loading / Unlading Loops	Shear Stiffness (MPa)	Young Modules, E (MPa)	Geotechnical Unit
BH 520 Test#1	Loop 1 Loop 2 Loop 3 unloading	not reliable not reliable 194.0	not reliable not reliable 504.4	Unit C Reddish brown brecciate calcareous sandstone, crushed (poor)
BH 520 Test#2	Loop 1 Loop 2 unloading Loop 3 unloading	not reliable 96,8 228,5	not reliable 251.6 594.0	Unit C Reddish brown brecciate calcareous sandstone, crushed (poor)
BH 520 Tes#3	Loop 1 unloading Loop 2 unloading Loop 3 unloading	20.0 40.5 90.7	51.9 105.4 235.9	Unit C Reddish brown brecciate calcareous sandstone, crushed (poor)
BH 506 Test#1	Loop 1 Loop 2 Loop 3 unloading	not reliable not reliable 48.7	not reliable not reliable 126.6	Unit B Brecciate sandstone

4.4.1 Geotechnical Parameters Used in Design

With respect to the recent investigation results as well as previous experience in collapsed tunnel, following geotechnical parameters have been used in support design. (*Table 4.5*)

Table 4.5 Geotechnical Parameters, Used in Design (Şimşek, 2001)

Geotechnical Unit		Description	Density kN/m^3	Rockmass Shear Strength		Rockmass Stiffness, drained (MPa)
	In-Site Conditions			C (kPa)	φ (˚)	
A	poor	Amphibolite – diorite	22	100-200	15-18	400
	competent	(MCB)	23	200-300	18-21	600
B	poor	Brown Brecciate	21	100-250	17-20	500
	competent	Sandstone	23	250-450	21-25	500-1000
C	poor	Marl Limestone Brecciate	23	300	17-20	500-1000
	competent	Calcareous Sandstone	24	1500	23-28	1500-3000
D	poor	Marble	24	500-1000	24-27	1000-1500
	competent		26	2000-5000	29-33	4000-10000
E		Reddish Clay, Plastic – Fault Gouge	18-20	90	9-12	150-250

4.5 Deformation Measurements

As NATM requires continuous reconsideration and modifications of support measures to be applied as a response to the trend of deformation development, during the tunnel excavation of NATM tunnel, observation of rock behavior is very important.

Deformation readings are obtained by surveyors by measuring the 3D coordinates of target plates installed on shotcrete shell. Those deformation readings say little about rock pressure, rock stress and loads on supports elements. But it is enough for the purpose of strengthening support elements to stop deformations in tolerance limits.

In critical points to see the relation between deformation readings and support load, strain gauges were installed. But it was never required in Elmalik portal driven side of the tunnels except rod-extensometers. Rod-extensometers were installed to see the relation between deformation readings on the surface and displacement in rock mass around the tunnel opening.

Mentioned deformation readings are stored and processed in special software called "*Dedalos*". Output drawings from this software are analyzed by The Controller's Engineer (The Engineer) and Tunnel Designer's Representative (TDR) in daily basis. If there is a need for a modification in support system, it is decided by TDR and The Engineer according to the provided deformation readings. (*Figure 4.3*)

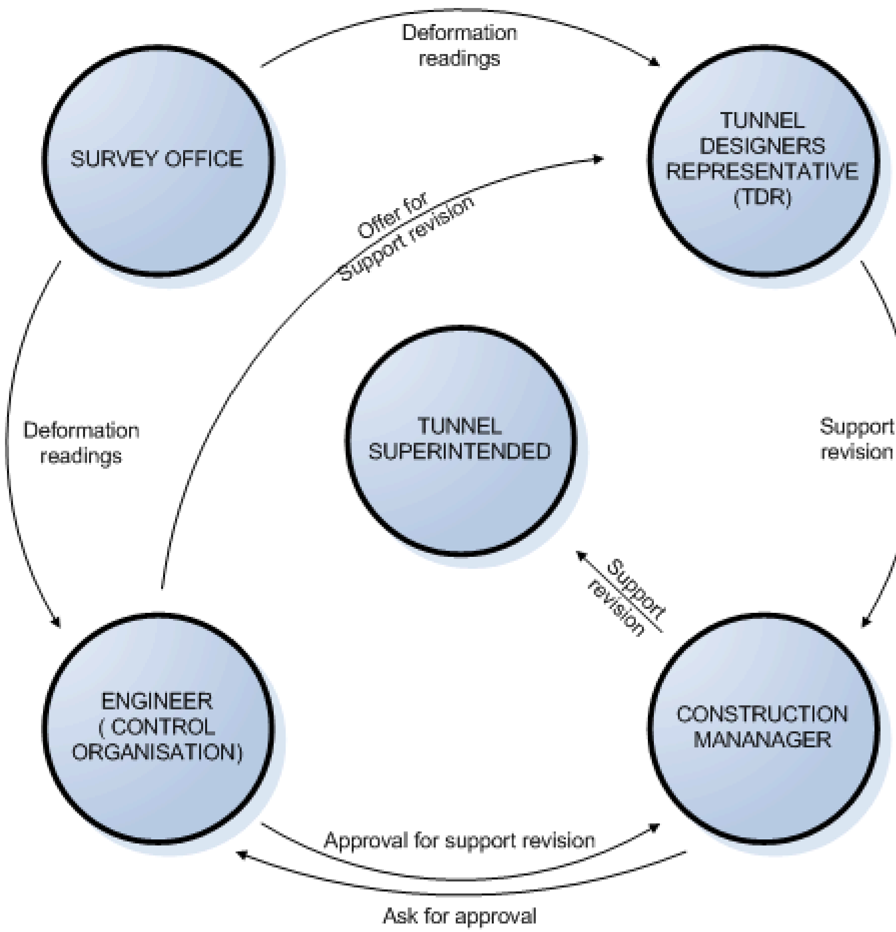


Figure 4.3 Decision Taking Procedure

4.5.1 Deformation reading procedure:

Theodolite laser total stations are used for deformation readings. This is a integrated electro-optical distance measuring device, allowing the measurement in one go of complete three-dimensional vectors.

Deformation stations with approximately 13.60 m clearance are installed by surveyors in tunnel. The idea is to place one deformation station for every inner lining block. Because, according to the design and project specification it is required to ensure low deformation rate (< 2 mm / month) to pour inner lining concrete.

Deformation stations are consisting of 7 optical targets. (*Figure 4.4)* Those targets are special reflective plates placed on a short steel bar (approximately 4 cm). Targets are installed on fresh shotcrete or bernold concrete (if exists). Reflective property makes it easier to read position of the target by using a theodolite. When a theodolite laser total station is used, it is fast and easy to determine relative position of the targets to a reference point. When targets are fixed, zero readings are done by surveyors. Thus, changes in the positions of the targets in time are measured and used to calculate deformations.

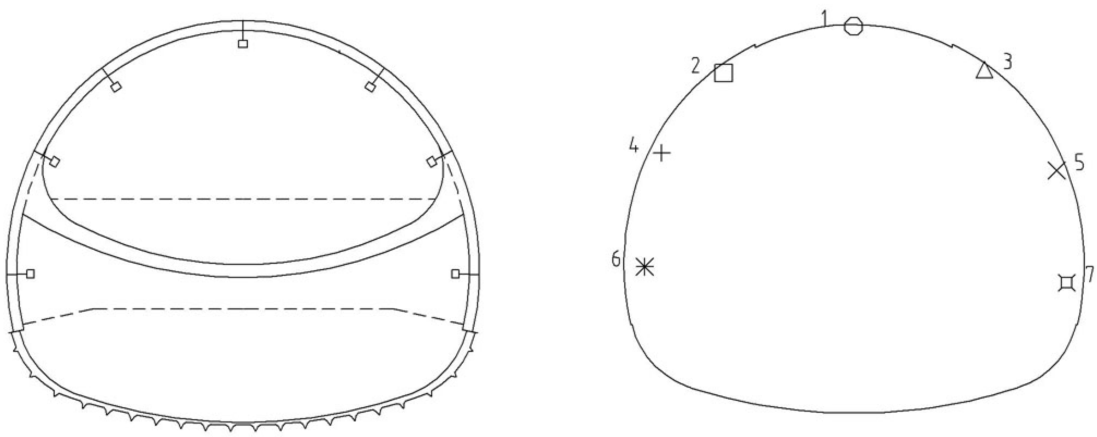


Figure 4.4 Location of Deformation Targets on Shotcrete Shell

Positions of the targets are read in every morning in a regular basis. If there is a special or risky situation, like a fracture in shotcrete shell, frequency of the readings can be increased. Readings are automatically recorded to a compact flash card then transferred to a computer. Within one and half hour deformation plots are ready including measurement process and necessary calculations. (Location of the deformation targets on shotcrete lining can be seen at *Figure 4.4*). Accuracy of the deformation measurement system is 0-3mm. As stated before, deformation plots are given to the TDR and The Engineer and to the shift engineer in daily basis.

4.5.2 Evaluation of deformation readings

Deformation readings are used to create deformation plots. As it is easier to follow deformation trends, negotiate and discuss on visual plots rather than numerical data, for the purpose of evaluation those plots are very useful.

In Bolu Mountain Crossing Tunnels mostly three types of plots were used for observation.

- S-H-L Diagram (or 3D monitoring): Deformation history for individual sections, displacement vs. time
- E Diagram: Settlement along tunnel axis, settlement vs. chainage.
- H-V Diagram: Horizontal and Vertical Displacement Vectors of individual sections vs. time

In the diagrams, settlement values show vertical displacement of a specific deformation target. It should not be though as total settlement of the tunnel section. First two drawings also include construction steps to allow reader to follow relation between deformation and tunnel advance.

A) 3D monitoring diagram (Displacement vs. Time)

3D monitoring diagram includes displacement vs. time plot. This diagram shows the deformation development for a single reading station. Daily report contains displacement vs. time plots for every active station. When deformation rate decreases under 3 mm / month rate, deformation readings are stopped. Then targets are cut out with rock bolt heads as preparation to inner lining works to provide smooth tunnel surface. An example to this report can be seen at *Figure 4.5*.

S-H-L diagram shows settlement, horizontal displacement and longitudinal deformation vs. time for a individual reading sections. Each paper consists of four rows. First three rows shows deformation in certain directions. 7 lines in diagram indicate 7 targets shown in the key drawing. Y axis indicates displacement and x axis indicates time.

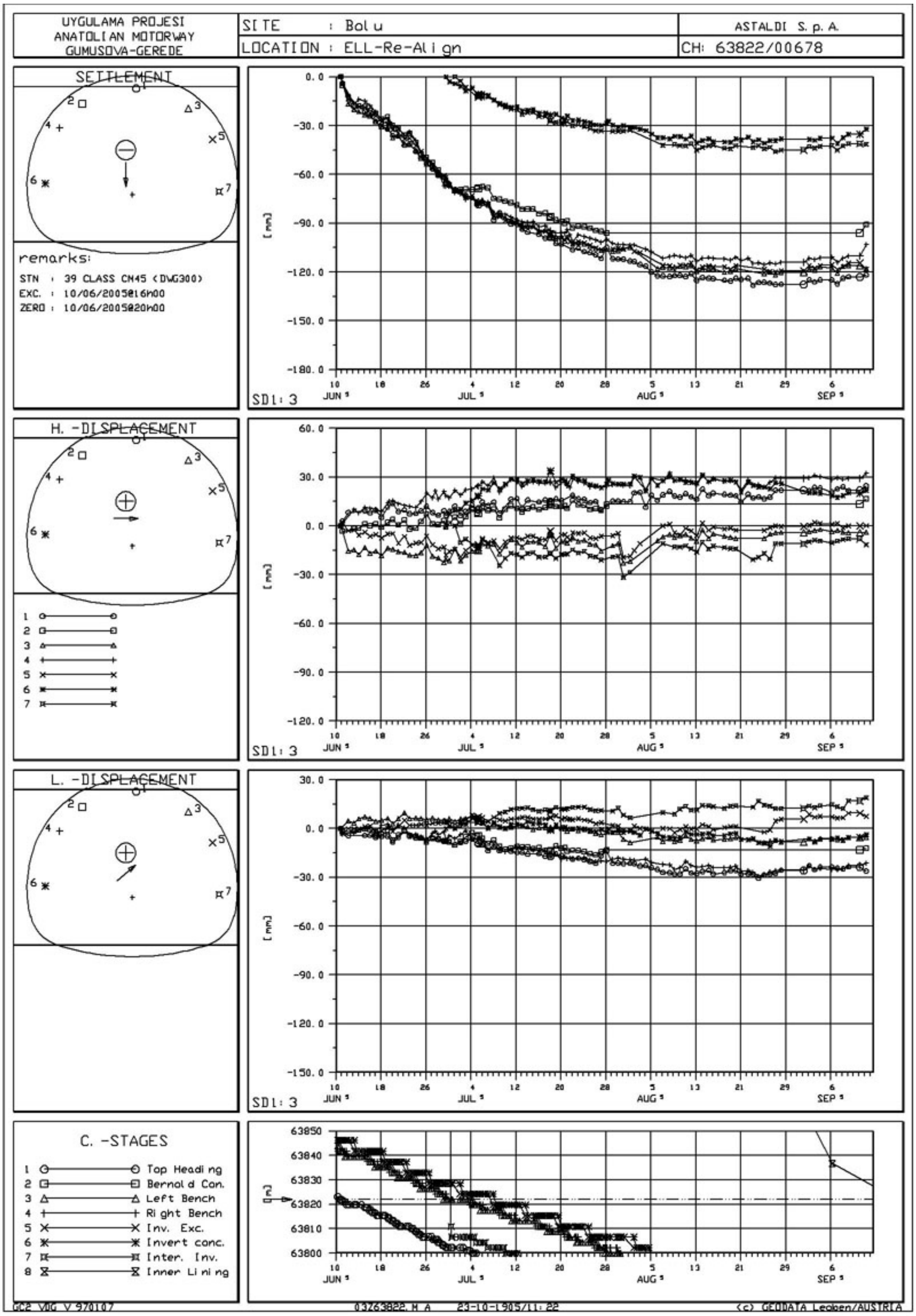


Figure 4.5 S-H-L Diagram Showing Displacements vs. Time (Astaldi S.p.A. Survey Office)

In order to show relation between target displacement and construction steps fourth row is added. In fourth row 8 line indicates 8 construction steps, y axis indicates chainage (or advance of a construction step) and x axis indicates time.

While x axis (time) is common in all rows. One can easily relate the four drawing.

As largest displacements are always occurs in the crown area of the top heading, the most important part of the graph is settlement diagram (top row). There is an expected deformation behavior on that plot (*Figure 4.11*)

B) E Diagram (Settlement vs. Chainage)

E diagram includes settlement (vertical displacement) vs. chainage plot that shows the deformation history through the tunnel axis. As previously mentioned, settlement is the largest and most important deformation type. That's way E diagram can be used to monitor overall deformation behavior of the tunnel. For horizontal and lateral displacements it is considered to follow displacement vs. time plots are enough. In any case it is possible to provide deformation vs. chainage plots through the software but this option was not used in a daily basis.

It is reported when asked by TDR (generally in a 7 – 14 day interval). A typical E diagram includes settlement vs. chainage plot including all active stations. Depending to the number of active stations this plot can be divided into pages. Example to this report can be seen at *Figure 4.6*

Lines on the diagram indicate displacement at certain date. Y axis represents settlement and x axis represents chainage. For comparison purposes construction stages are given in the bottom part (lines represents construction stages, y axis: chainage, x axis: date)

It is a little hard to understand the logic of E diagram but it is a very useful to see deformation trend through tunnel axis.

For example, if the lines are drawing near for a certain chainage this means that deformations are stopping there. If the drawing slopes down for certain point that means there is a weak zone around that point and it is sign for revision in support measures.

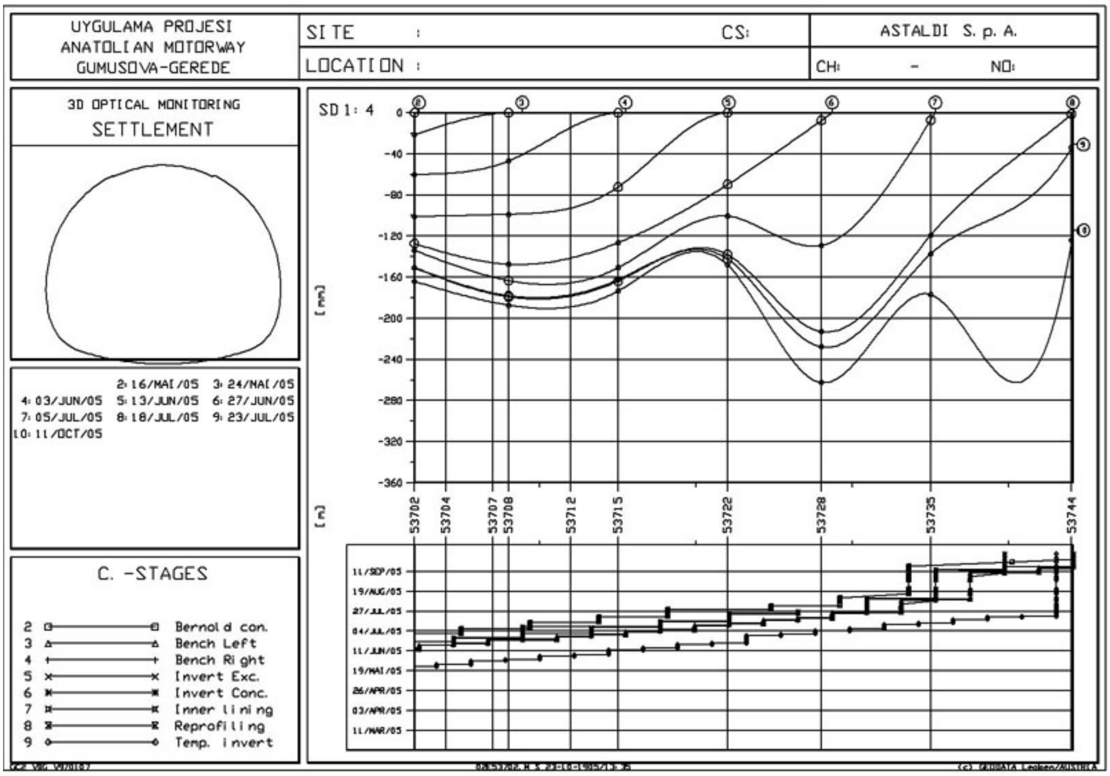


Figure 4.6 E Diagram Showing Settlements vs. Chainage (Astaldi S.p.A. Survey Office)

C) H-V Diagram (Displacement Vectors vs. Time)

This plot shows the position of the targets in the section after deformation. H-V diagram characterize the ‘shape’ of the deformation very well. So that one can easily decide on the direction of loading zone if the load (related to deformation) is coming from one part of tunnel.

An example to H-V diagram can be seen in *Figure 4.7*. In that diagram lines represents certain targets and numbers represents dates. Symmetrical behavior is expected if the ground around the cavity is homogenous.

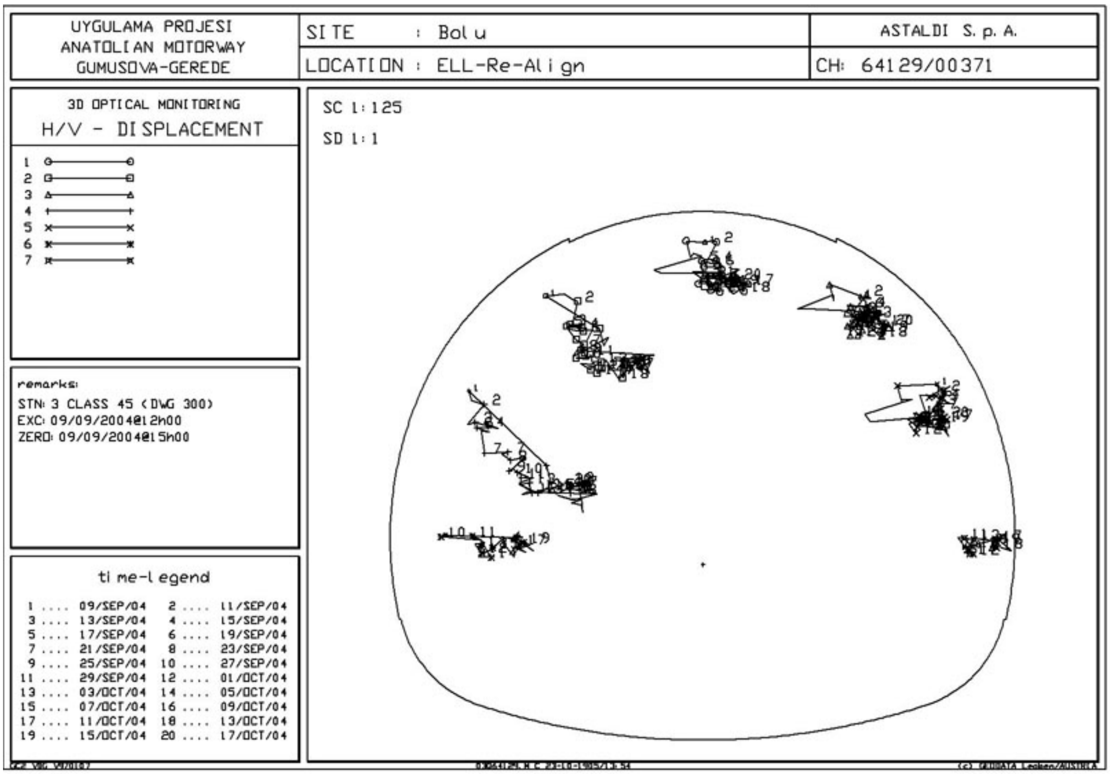


Figure 4.7 H-V Diagram Showing Displacement Vectors (Astaldi S.p.A. Survey Office)

In the example figure (*Figure 4.7*), H-V diagram of Elmalik Left Tube Station 3 is shown. Exaggerated displacement vectors for each target between 9 September and 17 October 2007 are plotted in *Figure 4.8*. From that figure (*Figure 4.8*) it is easily understood that there is relatively high deformation on left shoulder. This may caused by partially poor rock condition over right shoulder such as broken or disturbed rock body or increase in water content or tunnel is entering into weaker formation starting from right shoulder. For better understanding it is better to read H-V diagram with site geologists face drawings.

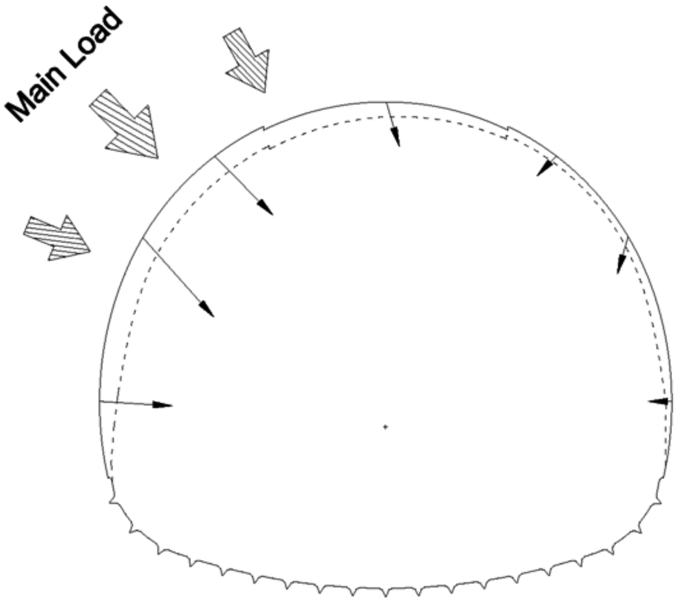


Figure 4.8 H-V Diagram Explanation

4.5.3 Expected Deformation Behavior

As stated before, there is an expected deformation behavior. It is expected for a newly excavated and supported top heading section to start deformation at highest rate and this rate should slowly decrease. Up to the point where bench excavations are started. Bench excavations will accelerate the deformations, but the rate is not expected to be as high as the initial rate of top heading. Then is should loose its acceleration again until the invert excavation. In CM class deformations are expected to stop after invert excavation. In Option 3 class it is required to apply bernold concrete to stop deformations.

For a clear understanding see the *Figure 4.9 and 4.10*, typical S diagrams.

In *Figure 4.9*, typical settlement plot for Option 3 class is shown. Note that deformation rate is increasing with the effect of temporary invert excavation, but it suddenly loose speed when temporary invert is completed but not stopped, because bench excavation is continuing for the previous rounds. When it's came to bench excavation for current round, deformation rate increases even more and this trend continues till the completion invert concrete. By the invert concrete ring is closed and deformation rate is suddenly drops, but not completely stop. To stop deformations (or drop the rate below 3 mm / month limit) Bernold concrete is required in Option 3 Class.

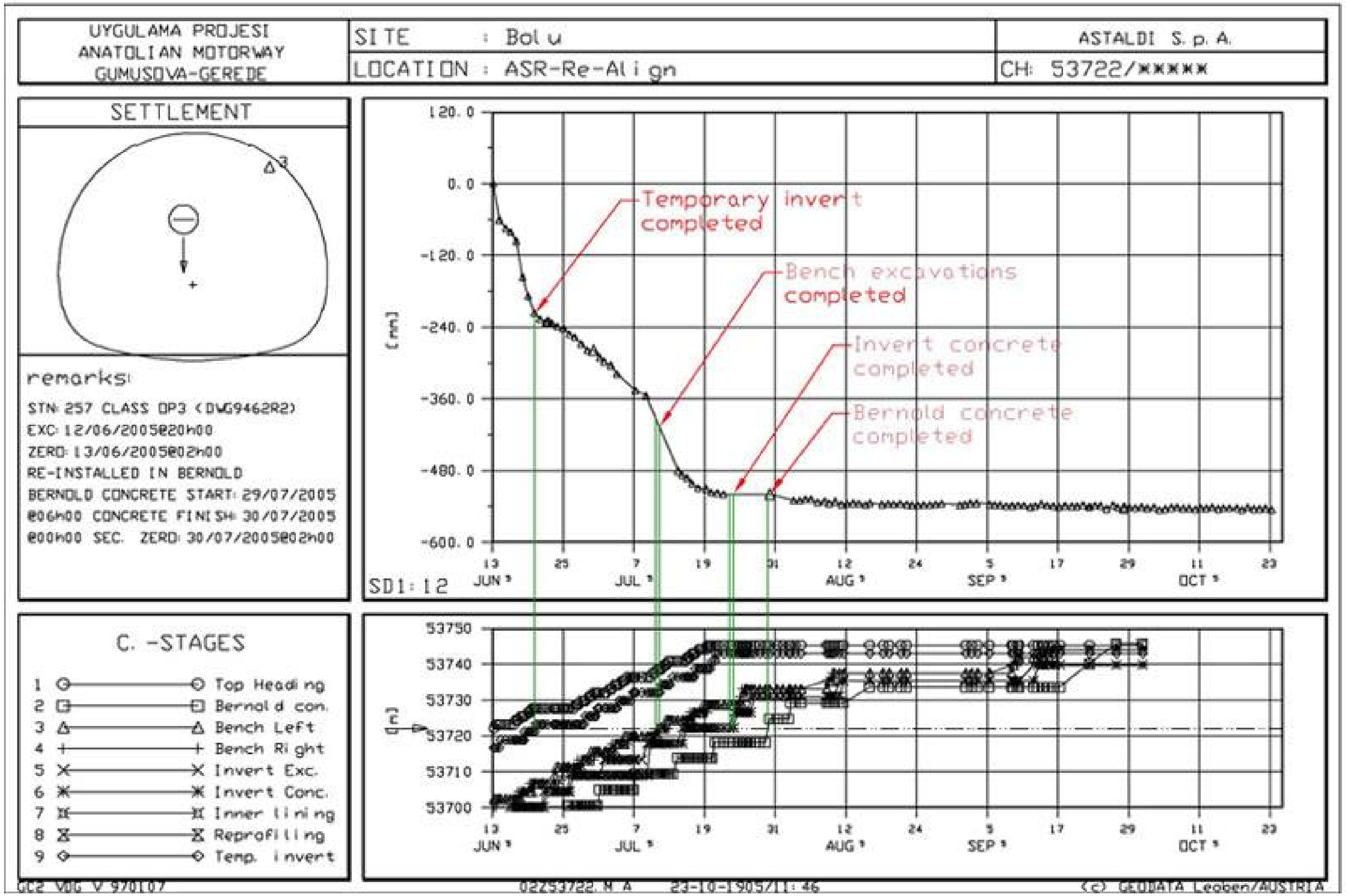


Figure 4.9 Typical Settlement Diagram for Option 3 Class (Astaldi S.p.A. Survey Office)

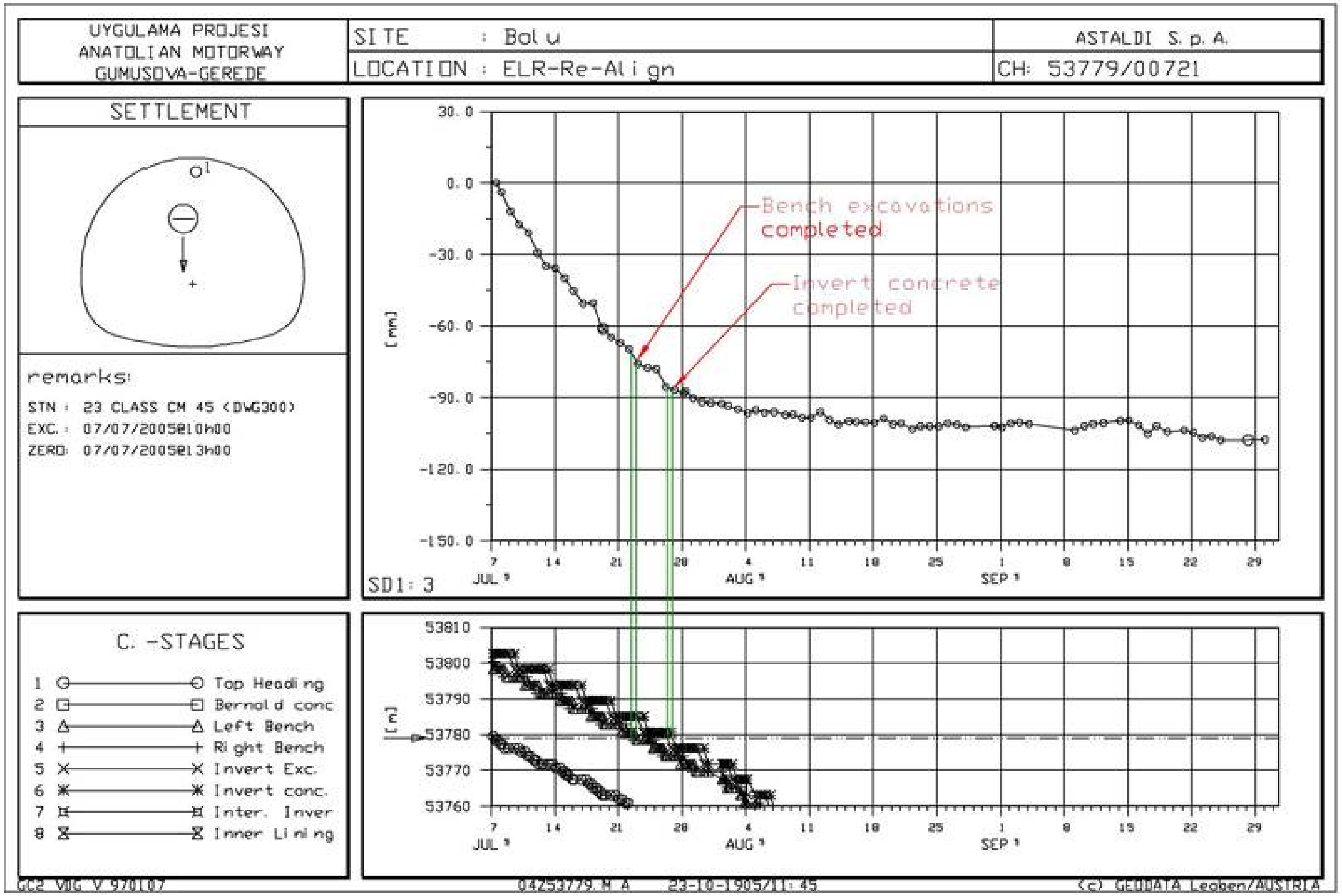


Figure 4.10 Typical Settlement Diagram for CM Class (Astaldi S.p.A. Survey Office)

In *Figure 4.10*, typical settlement plot for CM class is shown. In that diagram you can see that up to the completion of bench excavation, approximately 7.5 cm deformation occurs. When compared with figure 4.6 one can easily say deformations are low in top heading and there is no need for temporary invert. Then it is clearly seen that there is no accelerated deformation during invert excavation (note that invert excavation in CM is not as deep as invert excavation of Option3 and excavations duration is shorter.) And again it can be easily noticed that there is no need for bernold because deformation rates are decreased to stop in time.

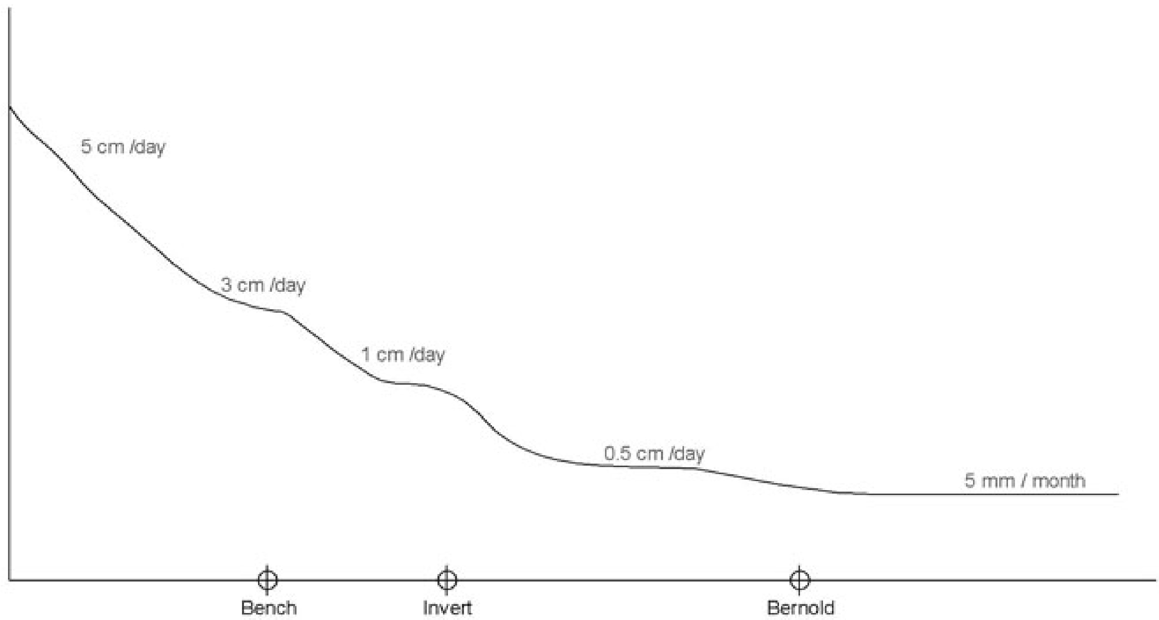


Figure 4.11 Expected Deformations Rates for Option 3

Finally in *Figure 4.11* expected (or desired) deformation rates are shown. As you see in the figure, 5cm/day deformation is expected after top heading excavation. Following the installation of each support element deformation rate is decreased. Finally it is expected from Bernold concrete to decrease deformation rate to 5 mm /month level. Increase in deformation rates during bench and invert excavations are normal.

CHAPTER V

ANALYSIS OF THE SUPPORT DESIGN

5.1 Support Design of Elmalik Portals

Before 1999 Earthquake, Elmalik driven part of the Bolu Tunnels were excavated and supported according to C2, CM, Option 3 and Option 4 support classes.

Before the earthquake of 1999, serious problems had been encountered during the application of C2 support class in Flyshoid, Metasediment, Siltsone series and especially fault gouge. Deformations exceeding design limits caused re-profiling excavations on already excavated and supported sections. Additionally, it takes too long for deformations to stop. Even though concrete thickness and number of rock bolts increased, it has been observed that deformation rates diminish in a certain rate but than, instead of stopping, continue at a small rate for a long time, sometimes for months, as a result of the loosening pressure, activation of the self weight loads and plastic zones extending far into the surrounding rock mass caused by large initial deformations. This hindered the installation of secondary support (as installation of secondary support requires deformation rates to decrease below 5mm/month) and causes 1000m separation between excavation face and lining concrete (secondary support).

As the new alignment passes through the same rock formations under C2 class it was inevitable to revise support design. As the flyshoid, metasediment and siltsone series are showing different deformation trends according to weathering grade, support class C2 has been divided into three subgroups. Those modified support classes are CM35, CM45 and Option 3. All revised subgroups have stronger support than original support. HEB100 profile steel ribs have been replaced by TH29 to improve deformability. SN type rock bolts have been replaced by self drilling IBO anchors to save time during primary support installation.

Hence, support ring completion time is decreased. Steel reinforcement has been added to inner lining concrete to compensate seismic loads during a possible earthquake.

Properties of modified rock classes are given respectively. Class CM35 (formerly class CM) has 35 cm shotcrete, 245 m per m of tunnel rock bolts and 35 cm deformation tolerance. Class CM45 has 45cm shotcrete, 245 m per m of tunnel rock bolts and 35 cm deformation tolerance. And Option 3 has 40 cm shotcrete; 216.8 m rock bolts per m of tunnel with 30 cm deformation tolerance and 60 cm concrete (called as bernold concrete) with 20 cm deformation tolerance.

Support class selections according to varying geologic units are given in *Table 5.1* and 5*.2*. Project drawings upon which support installations were done accordingly can be found in Appendix II of the manuscript. Key parameters of Basic Support Designs applied in Elmalik Portals driven part of the tunnel may be found in *Table 5.3.* Deformation plots (Displacement vs. Time; S-H-L) prepared during the construction of the new alignment Elmalık side are presented in Appendix IV.

Table 5.1 Support Classes vs. Geologic Units, Left Tube

Advence From Portal (and Chainage)		Lithological Description	Support Class
0 m	(64+141)	*Tunnel Portal* Flyshoid sequence	CM45 + 10cm shotcrete
3 m	(64+138)	Fault zone	
9 m	(64+132)		CM45
5 m	(64+136)	Flyshoid sequence	
35 m	(64+106)		CM35
79 m	(64+062)	Fault zone	
80 m	(64+061)	Flyshoid Sequence and Siltsone (Flyshoid Sequence)	
195 m	(63+946)	Metasediments	
196 m	(63+945)	Siltsone (Flyshoid Sequence)	
200 m	(63+941)	Metasediments	
205 m	(63+936)	Siltsone (Flyshoid Sequence)	
220 m	(63+921)	Metasediments	
236 m	(63+905)		CM45
337 m	(63+804)		Transition Zone
339 m	(63+802)	Main Fault Zone	
342 m	(63+799)		OPTION3
360 m	(63+781)	*Tunnel Breakthrough*	

Table 5.2 Support Classes vs. Geologic Units, Right Tube

Advance From Portal (and Chainage)	Lithological Description	Support Class
	Tunnel Portal	
0 m (53+961)	Flyshoid sequence	CM45 + 10cm shotcrete
8 m (53+953)	Siltstone (Flyshoid Sequence)	
9 m (53+952)		CM45
11 m (53+950)	Flyshoid sequence	
21 m (53+940)	Siltstone (Flyshoid Sequence)	
30 m (53+931)		CM35
117 m (53+844)	Metasediments	
121 m (53+840)		CM45
214 m (53+747)	*Tunnel Breakthrough*	

Table 5.3 Basic Support Designs

SUPPORT CLASS	C2	CM35	CM45
No. of basic design drawings	8382, 8405	9426, 9427	303, 304
TOP HEADING			
Excavation Area (m^2)	72.10 m^2	75.38 m^2	77.70 m^2
Sealing *(thickness, concrete class)*	3-5 cm, BS20	5 cm, BS20	5 cm, BS20
Steel Rib (type)	HEB100	TH29	TH29
Shotcrete Lining *(thickness, concrete class)*	25 cm, BS30	35 cm, BS30	45 cm, BS30
Deformation Tolerance	25 cm	35 cm	35 cm
Face Bolt (no, type, L)	-	10 x IBO R32N, 15m	10 x IBO R32N, 15 m
Radial Rock Bolt (no, type, L)	4 x 9 m + 9 x 6 m SN Bolt	6 x 9 m + 12 x 12 m IBO R32S	6 x 9 m + 12 x 12 m IBO R32S
Forepoling (no, type, L)	22 – 30 x 2.5 – 3.5 m Ø1.5” pipe	22 x 6 m, IBO R32N or 22 x 12 m, Ø3” pipe	22 x IBO R32N 6 m or 22 x 12 m, Ø3” pipe
Intermediate Invert *(thickness, concrete class)*	-	50 cm, BS20	50 cm, BS30
BENCH			
Excavation Area (m^2)	71.72 m^2	75.74 m^2	76.06 m^2
Maximum distance from Top Heading	35 m	35 m	27.5 m
Steel Rib (type)	HEB100	TH29	TH29
Shotcrete Lining *(thickness, concrete class)*	25 cm	35 cm, BS30	45cm, BS30
Deformation Tolerance	25 cm	35 cm	35 cm
Radial Rock Bolt (no, type, L)	-	8 x 9 m, IBOR32S	8 x 9 m, IBOR32S
INVERT			
Excavation Area (m^2)	30.34 m^2	45.86 m^2	49.55 m^2
Maximum distance from Top Heading	40 m	40 m	34.1 m
Sealing *(thickness, concrete class)*	-	10 cm, BS20	10 cm, BS20
Steel Reinforcement	-	✓	✓
Invert Concrete *(concrete class)*	50cm floor arch, BS30 concrete fill BS20	BS30	BS30
COVER SUPPORT			
Bernold Concrete *(thickness, concrete class)*	N/A	N/A	N/A
Foundation Beam *(concrete class, rein.)*	BS30 without reinforcement	BS40 with reinforcement	BS40 with reinforcement
Inner Lining *(thickness, concrete class)*	40 cm, BS30	40 cm, BS40	40 cm, BS40
Inner Lining Reinforcement	N/A	✓	✓

*BS20, BS30 and BS40 are concrete classes that represent 20N/mm^2, 30N/mm^2 and 40N/mm^2 respectively.

Table 5.3 Cont'd

SUPPORT CLASS No. of basic design drawings	OPTION 3 9464, 9465	OPTION 4 9432 A,B 9433	OPTION 4 / BENCH PILOT TUNNEL
TOP HEADING			
Excavation Area (m^2)	94,62 m^2	97.49 m^2	51.04
Sealing *(thickness, concrete class)*	5 cm, BS20	5 cm, BS20	
Steel Rib (type)	TH29	TH29	HEB100
Shotcrete Lining *(thickness, concrete class)*	40 cm, BS30	30 cm, BS30	30 cm, BS30
Deformation Tolerance	30 cm	30 cm	2 cm
Face Bolt (no, type, L)	10 x 15 m, IBO R32N	16 x 15 m, IBO R32N	4 x 15m, IBO R32N
Radial Rock Bolt (no, type, L)	8 x 9 m + 12 x 12 m IBO R32S	20 x 9 m, IBO R32S	
Forepoling (no, type, L)	22 x 16 m, IBO R32N or 22 x 12 Ø3'' pipe	22 x 16 m, IBO R32N or 22 x 12 Ø3'' pipe	
Intermediate Invert *(thickness, concrete class)*	50 cm, BS30	50 cm, BS30	
BENCH			
Excavation Area (m^2)	82,94 m^2	58.38 m^2	
Maximum distance from Top Heading	19,8	35.2	
Steel Rib (type)	TH29	TH29	
Shotcrete Lining *(thickness, concrete class)*	40 cm, BS30	30 cm, BS30	
Deformation Tolerance	30 cm	30 cm	
Radial Rock Bolt (no, type, L)	10 x 9 m IBO R32S	N/A	
INVERT			
Excavation Area (m^2)	65.82 m^2	66.53 m^2	
Maximum distance from Top Heading	24.2	26.4	
Sealing *(thickness, concrete class)*	10 cm, BS20	10 cm, BS20	
Steel Reinforcement	✓	✓	
Invert Concrete *(concrete class)*	B30	B30	
COVER SUPPORT			
Bernold Concrete *(thickness, concrete class)*	60 cm, BS40	60 cm, BS40	
Foundation Beam *(concrete class, rein.)*	BS40	BS40	
Inner Lining *(thickness, concrete class)*	60 cm, BS40	60 cm, BS40	
Inner Lining Reinforcement	✓	✓	

*BS20, BS30 and BS40 are concrete classes that represent 20N/mm^2, 30N/mm^2 and 40N/mm^2 respectively.

5.2 Analysis of The Support Design

In order to examine the support design and find out weakness, ground support interaction curves are drawn using "RocSupport" software of Rocscience Inc. This software estimates the deformation of circular tunnels in weak rock, and visualize the tunnel interaction with various support systems by rock-support interaction (or convergence-confinement) analysis. Rock-support interaction analysis method is based on the concept of a "ground reaction curve" or "characteristic line" obtained from the analytical solution for a circular tunnel in an elasto-plastic rock mass under a hydrostatic stress field. Therefore in rock support interaction analysis following assumptions are made:

- Tunnel is assumed as circular.
- In-situ stress field is assumed as hydrostatic. (i.e. equal stress in all directions)
- Rock mass is assumed as isotropic and homogeneous. Failure is not controlled by major structural discontinuities.
- Support response is assumed as elastic-perfectly plastic.
- Support is modelled as an equivalent uniform internal pressure around the entire circumference of the circular tunnel.

Due to the intensive faulting in the area and the crushed and fractured nature of the rock mass, hydrostatic stress field assumption and the isotropic and homogenous rock mass assumption are not fitting to our case; nevertheless the rock support interaction analysis have been carried out to get an idea about the relation between the support behavior and the observed deformations.

RocSupport software utilizes Duncan Fama or Carranza-Torres solution to plot ground - support interaction curve. Duncan Fama solution is based on the Mohr-Coulomb failure criterion of the rock while the Carranza-Torres solution is based on the Hoek-Brown criterion. In this study, as the available rock mass data is suitable to Mohr-Coulomb criterion, Duncan Fama solution is chosen.

The software computes support reaction by two parameters: maximum support pressure (MPa) and maximum average strain (%). It assumes that support system shall give maximum support pressure at maximum average strain and support pressure is linearly proportional to the strain. It is allowed to determine support installation timing according to the distance from face (m), according to the tunnel convergence (%) or according to the wall displacement (mm). Since the support system is modeled as an equivalent uniform internal pressure, bolt length is not taken as an analysis parameter. The software assumes the rock bolts are as long enough as to reach the outside of the plastic zone. (If the total length of the rock bolts falls in the plastic zone it would not act as a support.) At the end of analysis when the extent of plastic zone calculated, the software determines the bolt lengths by extending the bolts 2.0 m beyond the plastic zone.

RocSupport calculates the safety factor as a ratio of maximum available support pressure to the utilized support at equilibrium. Even though this approach seems correct, as every element of the support system would fail individually by elements. The supports system would never reach maximum available support pressure. Therefore if the support capacity is high enough, support design with unacceptably high deformations can give good safety factors.

5.2.1 Analysis 1

For the justification of C2 support class, deformation recordings of station 49 (chainage 63+035) of old alignment left tube have been used. When deformations stopped, 877mm vertical deformation was recorded at this station. Applied support system was modified C2 with HEB 140 steel rib, 45cm fiber shotcrete and 30 IBO R32S rock bolts of 392 meters per meter of tunnel. There were flyschoid sequences below bench level and metacrystalline above. (Aygar, 2000) Following rock parameters are selected from *Table 4.3*:

Rock parameters:

- Tunnel Radius $r_o = 8$ m
- In-Situ Stress $p_o = 100 \times 0.022 = 2.20$ MPa
- Young's Modulus $E = 533$ MPa
- Poisson Ratio $\upsilon = 0.3$
- Uniaxial Compressive Strength of Rock Mass $\sigma_{rm} = 0.23$ MPa
- Friction Angle $\varphi = 17.5^o$

Support Parameters:

- Steel Rib: HEB 140
- Shotcrete: 45cm fiber reinforced
- Rock Bolts: 30 pieces IBO R32S, 392 meters per meter of tunnel

Calculations:

Maximum support pressure of steel sets (HEB140) is assumed as P_{ssmax} = 0.067 MPa by interpolation from HEB 150 and HEB 203. Smiliarly stiffness k_s is assumed as 22 MPa

Maximum support pressure of rock bolts are calculated as follows:

$$P_{sb\max} = \frac{T_{bf}}{s_c \times s_l}$$

Where T_{bf} is the ultimate strength of bolt system from pull-out test,
s_c is the circumferential rock bolt spacing and
s_l is the longitudinal rock bolt spacing.

$$P_{sb\max} = \frac{0.280}{1.5x1.1} = 0.170MPa$$

Stiffness of rock bolt system:

$$\frac{1}{k_b} = \frac{s_c \times s_l}{r_i}\left(\frac{4L}{\pi d_b E_b} + Q\right)$$

Where L is the bolt length,
d is the bolt diameter (mm),
Q is the bolt extension parameter from pull-out test and
E_b is the deformation modulus of bolt.

$$\frac{1}{k_b} = \frac{1.5 \times 1.1}{8}\left(\frac{4 \times 12}{3.14 \times 32 \times 207} + 0.018\right) = 0.20625 \times \left(2.307 \times 10^{-3} + 0.018\right)$$

$$\frac{1}{k_b} = 4.188 \times 10^{-3} \quad \Rightarrow \quad k_b = 238.76 MPa$$

Maximum support pressure of shotcrete shell:

$$P_{sc\max} = \frac{1}{2}\sigma_{c(shotcrete)}\left[1 - \frac{(r_i - t_c)^2}{(r_i)^2}\right]$$

Where r_i is the tunnel diameter and

t_c si the shotcrete thickness.

$$P_{sc\max} = \frac{1}{2}\sigma_{c(shotcrete)}\left[1 - \frac{(r_i - t_c)^2}{(r_i)^2}\right]$$

$$P_{sc\max} = \frac{30}{2}\left[1 - \frac{(8 - 0.45)^2}{64}\right] = 1.64 MPa$$

Stiffness of shotcrete shell:

$$k_c = \frac{E_c\left(r_i^2 - (r_i - t_c)^2\right)}{(1 + \nu_c)\left((1 - 2\nu_c)r_i^2 + (r_i + t_c)^2\right)}$$

Where E_c is the deformation modulus of concrete.

$$k_c = \frac{21000\left(64 - (8 - 0.45)^2\right)}{(1 + 0.25)\left((1 - 2 \times 0.25) \times 8^2 + (8 + 0.45)^2\right)} = \frac{21000 \times 6.9975}{1.25 \times (0.5 \times 64 + 71.4025)}$$

$$k_c = 1136.897 \approx 1137 MPa$$

Maximum available support pressure P = 0.067 + 0.17 + 1.64 = 1.877 MPa

Stiffness of combined supports k = 22 + 238 + 1137 = 1397 MPa

Displacement capacity u = P / k = 1.877 / 1397 = 0.134 %

Input Parameters:

- Total maximum support pressure: 1.877 MPa
- Displacement Capacity: 0.134 %
- Distance from tunnel face: 2.2m (2 rounds)
- Tunnel Radius: 8 m
- In-stu Stress: 2.2 MPa (100 m x 0.022 MN/m^2)

Results:

As it is seen in *Figures 5.1* and *5.2*, radius of plastic zone is calculated as 49.92 m, with 9.52% tunnel convergence (equal to 761.82 mm wall displacement). Both by the analysis and the deformation measurements made, most of the deformations have been observed before support installation. Consequently, increasing support stiffness would not solve the problem. Immediate completion of the support ring is required. However, in the analysis 2.2 m tunnel face distance has been taken into consideration. This is the shortest practical support distance.

In order to stop deformations as in this example, Option 3 and Option 4 support classes are introduced. Option 3 has larger deformation tolerance (30 + 40 = 70 cm) and rigid cast-in situ concrete shell (intermediate lining) applied over shotcrete shell to rapidly stop deformations. Option 4 utilize bench pilot tunnels to improve load carrying capacity of the ground before face excavation.

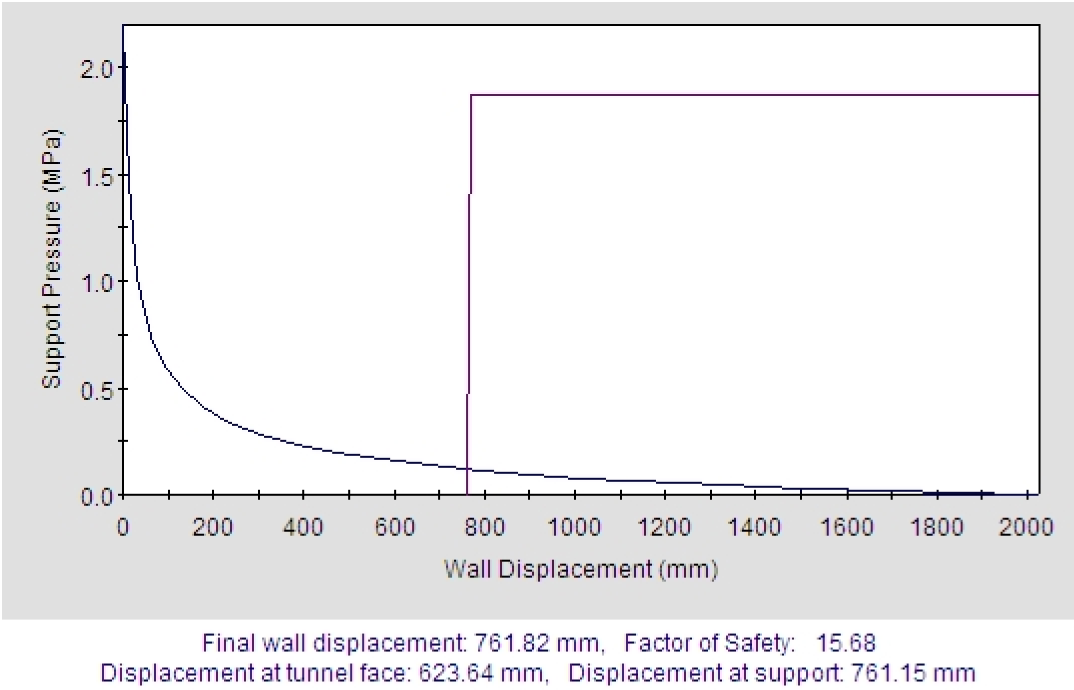


Figure 5.1 Ground-Support Interaction Curve of Class C2 Analysis

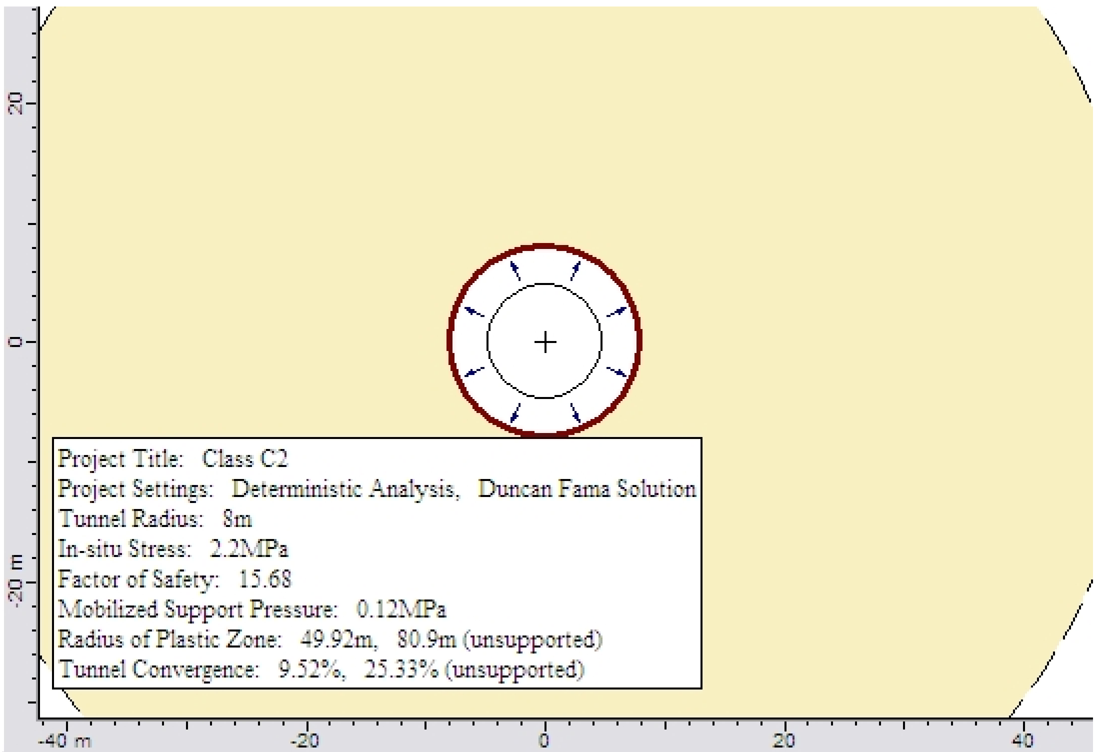


Figure 5.2 Tunnel Section View of Class C2 Analysis

5.2.2 Analysis 2

Analysis of CM35 application in relatively strong brown sandstone is a good example. TH29 steel rib, 35 cm fiber reinforced shotcrete and 18 IBO R32S rock bolts of 201 meters per meter of tunnel are used in CM35 support class. Following rock parameters are selected from *Table 4.3*:

Rock parameters:

- Tunnel Radius $r_o = 8$ m
- In-Situ Stress $p_o = 100 \times 0.022 = 2.20$ MPa
- Young's Modulus $E = 1000$ MPa
- Poisson Ratio $\upsilon = 0.3$
- Compressive Strength of Rock Mass $\sigma_{rm} = 1$ MPa
- Friction Angle $\varphi = 22^{o}$

Support Parameters:

- Steel Rib: TH 29
- Shotcrete: 35cm fiber reinforced
- Rock Bolts: 18 pieces IBO R32S, 201 meters per meter of tunnel

Calculations:

Maximum support pressure of TH29 steel is assumed as $P_{ssmax} = 0.156$ MPa by interpolation of maximum support pressures of TH38 and TH21 given in the software. Similarly stiffness k_s is assumed as 22 MPa

Maximum support pressure of rock bolts are calculated as follows:

$$P_{sb\max} = \frac{T_{bf}}{s_c \times s_l} = \frac{0.280}{2.x1.1} = 0.127 MPa$$

Stiffness of rock bolt system:

$$\frac{1}{k_b} = \frac{s_c \times s_l}{r_i}\left(\frac{4L}{\pi d_b E_b} + Q\right) = \frac{2 \times 1.1}{8}\left(\frac{4 \times 12}{3.14 \times 32 \times 207} + 0.018\right)$$

$$\frac{1}{k_b} = 0.275 \times (2.3078 \times 10^{-3} + 0.018) = 5.5847 \times 10^{-3} \quad \Rightarrow \quad k_b = 179.06 MPa$$

Maximum support pressure of shotcrete shell:

$$P_{sc\max} = \frac{1}{2}\sigma_{c(shotcrete)}\left[1-\frac{(r_i-t_c)^2}{(r_i)^2}\right] = \frac{30}{2}\left[1-\frac{(8-0.35)^2}{8^2}\right]$$

$$P_{sc\max} = 15\times\left[1-\frac{69.7225}{64}\right] = 1.2838 \approx 1.284 MPa$$

Stiffness of shotcrete shell:

$$k_c = \frac{E_c\left(r_i^2-(r_i-t_c)^2\right)}{(1+\nu_c)\left((1-2\nu_c)r_i^2+(r_i+t_c)^2\right)} = \frac{21000\left(64-(8-0.35)^2\right)}{(1+0.25)\left((1-2\times0.25)\times8^2+(8+0.35)^2\right)}$$

$$k_c = \frac{21000\times\left(64-7.65^2\right)}{1.25\times\left(0.5\times64+8.35^2\right)} = \frac{21000\times5.4775}{1.25\times101.7225} = 904.638 \approx 905 MPa$$

Maximum available support pressure P = 0.156 + 0.127 + 1.284 = 1.567 MPa

Stiffness of combined supports k = 22 +179 + 905 = 1106 MPa

Displacement capacity u = P / k = 1.567 / 905 = 0.173 %

Input Parameters:

- Total maximum support pressure: 1.567 Mpa
- Displacement Capacity: 0.173 %
- Distance from tunnel face: 2.2m (2 rounds)
- Tunnel Radius: 8 m
- In-stu Stress: 2.2 MPa (100 m x 0.022 MN/m^2)

Results:

As it is seen in *Figure 5.3* and *5.4*, radius of plastic zone found as 11.17 m, with 0.31% tunnel convergence (equal to 24.93 mm wall displacement). Although, CM35 support is weaker than Modified C2 support of previous example, deformations stay in limit (which is 35 cm) and the safety factor is found as 3.58 which is quite reasonable for a transportation tunnel.

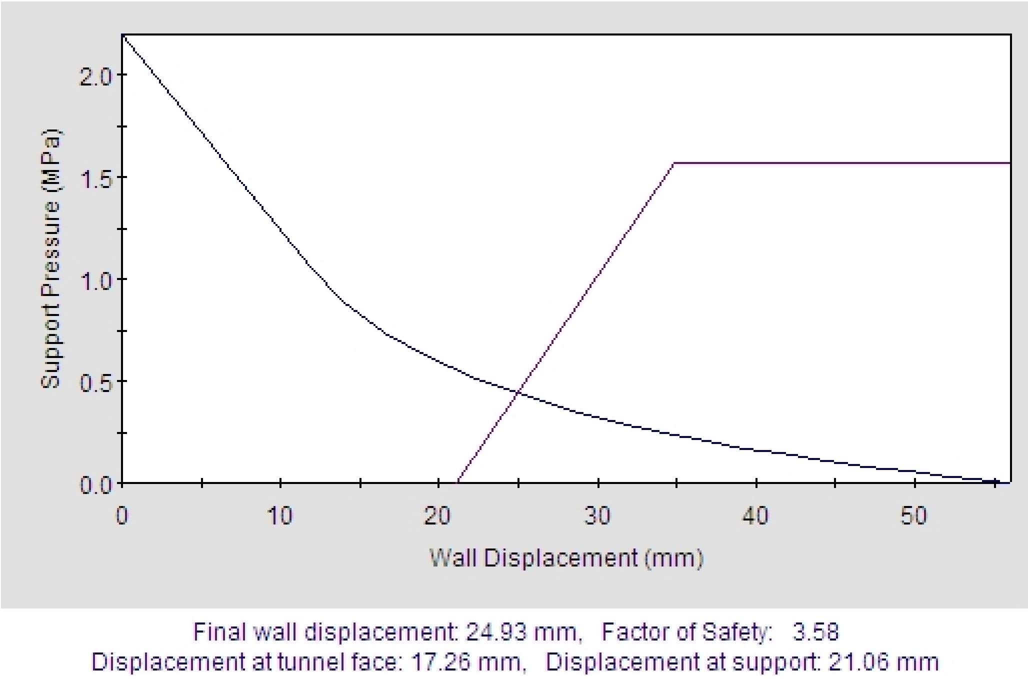


Figure 5.3 Ground-Support Interaction Curve of Class CM35 Analysis

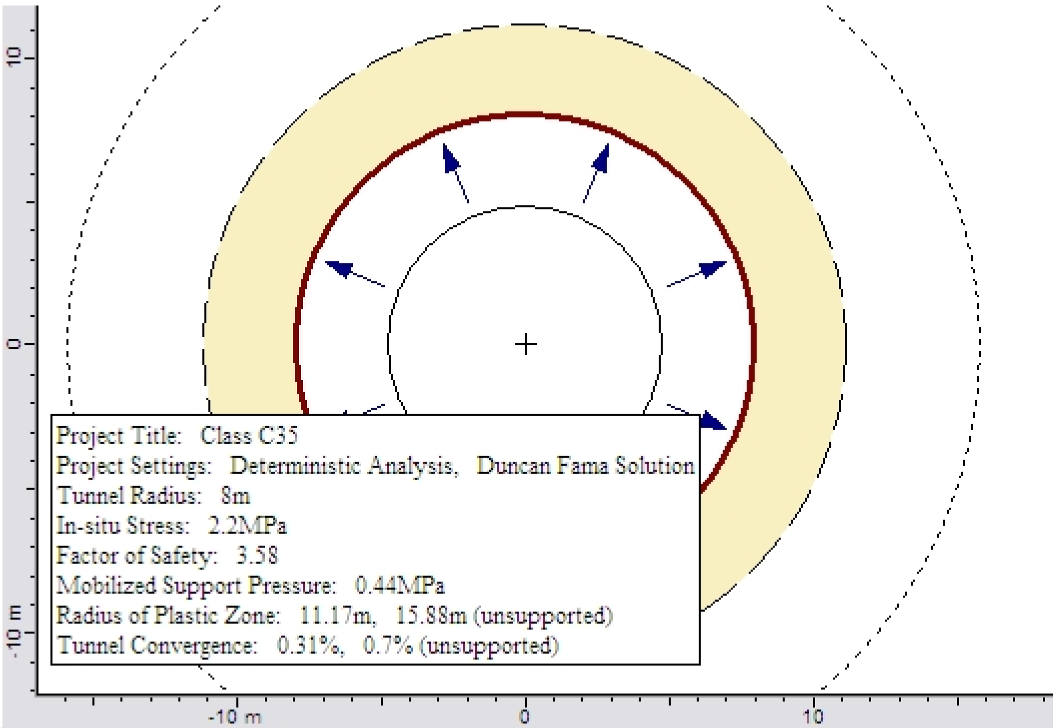


Figure 5.4 Tunnel Section View for Class CM35 Analysis

5.2.3 Analysis 3

When rock properties are not good as in this example (Analysis 2), C45 support class is utilized for the excavation and support of flyshoid series. Difference of CM45 from CM35 is thicker shotcrete shell (45 cm) which gives additional stiffness and increase safety factor. In order to see the difference between two support classes previous analysis has been repeated with CM45 support parameters. For the analysis following rock parameters are chosen from *Table 4.3*:

Rock parameters:

- Tunnel Radius r_o = 8 m
- In-Situ Stress p_o = 100 x 0.022 = 2.20 MPa
- Young's Modulus E = 1000 MPa
- Poisson Ratio υ = 0.3
- Compressive Strength of Rock Mass σ_{rm} = 1 MPa
- Friction Angle φ = 22^{o}

Support Parameters:

- Steel Rib: TH 29
- Shotcrete: 45cm fiber reinforced
- Rock Bolts: 18 pieces IBO R32S, 201 meters per meter of tunnel

Calculations:

Maximum support pressure of TH29 steel is assumed as P_{ssmax} = 0.156 MPa by interpolation of maximum support pressures of TH38 and TH21 given in the software. Similarly stiffness k_s is assumed as 22 MPa

Maximum support pressure of rock bolts are calculated as follows:

$$P_{sb\max} = \frac{T_{bf}}{s_c \times s_l} = \frac{0.280}{2.x1.1} = 0.127 MPa$$

Stiffness of rock bolt system:

$$\frac{1}{k_b} = \frac{s_c \times s_l}{r_i}\left(\frac{4L}{\pi d_b E_b} + Q\right) = \frac{2 \times 1.1}{8}\left(\frac{4 \times 12}{3.14 \times 32 \times 207} + 0.018\right)$$

$$\frac{1}{k_b} = 0.275 \times (2.3078 \times 10^{-3} + 0.018) = 5.5847 \times 10^{-3} \quad \Rightarrow \quad k_b = 179.06 MPa$$

Maximum support pressure of shotcrete shell:

$$P_{sc\max} = \frac{1}{2}\sigma_{c(shotcrete)}\left[1 - \frac{(r_i - t_c)^2}{(r_i)^2}\right] = \frac{30}{2}\left[1 - \frac{(8-0.45)^2}{64}\right] = 1.64 MPa$$

Stiffness of shotcrete shell:

$$k_c = \frac{E_c\left(r_i^2 - (r_i - t_c)^2\right)}{(1+\nu_c)\left((1-2\nu_c)r_i^2 + (r_i + t_c)^2\right)} = \frac{21000\left(64 - (8-0.45)^2\right)}{(1+0.25)\left((1-2\times0.25)\times 8^2 + (8+0.45)^2\right)}$$

$$k_c = \frac{21000 \times 6.9975}{1.25 \times (0.5 \times 64 + 71.4025)} = 1136.897 \approx 1137 MPa$$

Maximum available support pressure P = 0.156 + 0.127 + 1.64 = 1.923 MPa

Stiffness of combined supports k = 22 +179 +1137 = 1338 MPa

Displacement capacity u = P / k = 1.923 / 1338 = 0.144 %

Input Parameters:

- Total maximum support pressure: 1.923 MPa
- Displacement Capacity: 0.144%
- Distance from tunnel face: 2.2m (2 rounds)
- Tunnel Radius: 8 m
- In-stu Stress: 2.2 MPa (100 m x 0.022 MN/m^2)

Results

As it is seen in *Figures 5.5* and *5.6*, radius of plastic zone is calculated as 10.96 m, with 0.3% tunnel convergence (equal to 23.86 mm wall displacement).

Stiffness of the support systems are 1338 MPa and 1106 MPA for CM45 and CM35 respectively. Therefore CM45 support shell is not elastic as in CM35. Displacement capacity for CM45 is reduced to 0.114% from 0.173%. On the other hand, in practice shotcrete is applied in two layers with wire mesh between the layers to give ductility and increase elasticity.

Effect of shotcrete thickness can better be understood when deformation between support installation and the equilibrium point is compared. In CM35 3.87 mm deformation calculated after support installation (*Figure 5.3*). In CM45 this amount is approximately 2.80 mm. (*Figure 5.5*) Accordingly, even though CM45 support is significantly stronger it could not make a big difference as majority of the deformations (21.06 mm) occur before the support installation. This emphasizes the importance of support installation time and precautions to decrease initial deformation rate like immediate face sealing, forepoling etc.

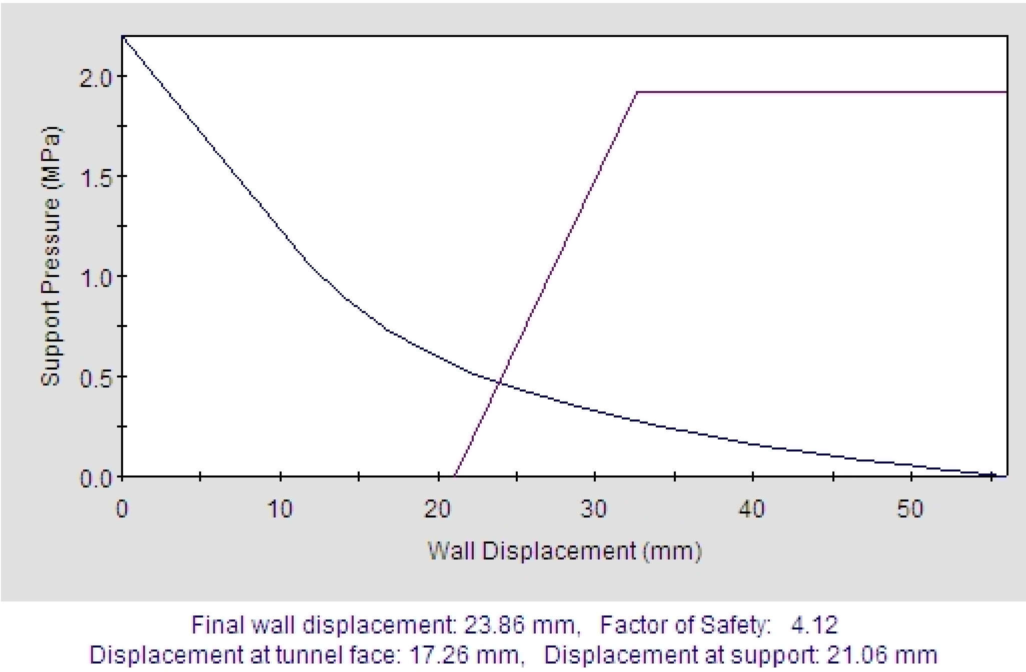


Figure 5.5 Ground-Support Interaction Curve of Class CM45 Analysis

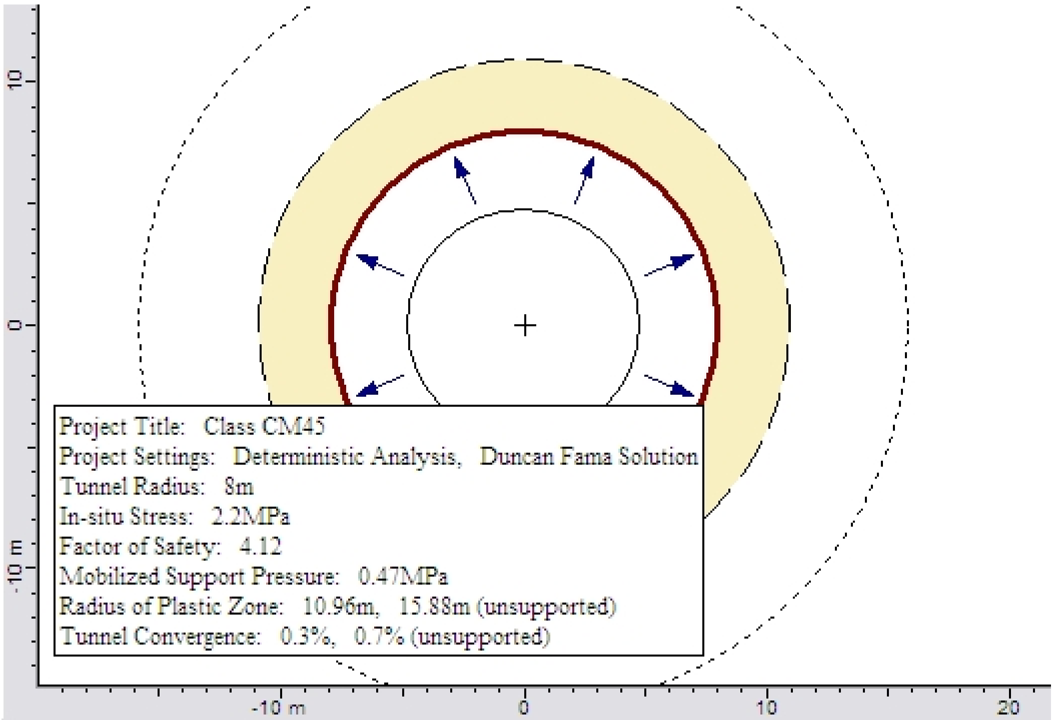


Figure 5.6 Tunnel Section View for Class CM45 Analysis

5.3 Discussion of the Results

One of the fundamental principles of NATM is to allow the rock to deform to a certain extent. It is assumed by definition of NATM that when the tunnel perimeter deforms, the surrounding rock mass starts to develop a load carrying (support) ring to stand-up. In order to activate self supporting ability of surrounding rock, elastic supports are used as primary support. In this manner, deformations can be controlled and reduced without totally preventing.

However in weak ground, deformations may increase support load. In Bolu it was found that if not stopped by rigid supports, long time deformations can reach tens of meters extents in weak ground (e.g. fault gouge and metasediments). Surely this will induce greater loads in support elements. Therefore immediate, continuous support around the tunnel perimeter (and, if required, also to the face) is required to minimize initial movement in the surrounding ground. It is also essential to structurally complete the supporting ring including invert concrete as quickly as possible.

Numerical convergence confinement analysis on RocSupport software also shown that, in weak ground, having low geotechnical parameters, most of the displacements took place before support installation. So that, it is critically important to install primary supports as soon as possible and as close as possible to the excavation face. Beside that remedial measures should be taken into account before or immediately after the face excavation. Those measures may be grouting of the top heading foot, forepoling, face bolts and immediate face sealing, drivage and supporting of side excavations ahead of the top heading. Application of temporary invert is very useful to obtain earlier ring completion and has a great effect.

For this reason in support classes Option 3 and Option 4, rigid support systems, bench pilot tunnels and Bernold lining were applied. This was contradictory to elastic primary support shell idea of NATM. But it was required to stop deformations. In 1999 Duzce Earthquake Tunnel took serious damage in the sections where primary lining had not been completed yet. These unlined sections were about 300 m in each tube. The reason of such long sections was left without final lining was long term deformations. After the collapse it was understood that instead of waiting long time (e.g for a year) for deformations to be stopped by elastic lining it was preferable to use rigid lining.

Therefore it can be said that for weak-ground, where self-supporting ability of the ground is very limited, elastic lining is not solution. Rigid lining is required to stop deformations immediately. Otherwise, long term deformations can cause higher costs by means of reprofiling works and time loss.

CHAPTER VI

CONCLUSIONS

Based on the results of this study following conclusions are drawn:

1. Deformations occurred after excavation should be monitored and guided in such a way that a protective support ring should be formed before excessive loosening of the rock mass occurs. That may decrease self supporting capacity.

2. In weak rock (C2, L1 and L2 classes), in order to reduce deformations occurring before the primary support installation temporary support elements; such as forepoling, face bolts, immediate face sealing, grouting of the top heading foot, excavation and support of side excavations ahead of the top heading; should be utilized before or immediately after the face excavation. It is also found essential to close the supporting ring structurally, including invert concrete, as quickly as possible. Otherwise deformations on freshly excavated sections may develop very fast and exceed design limits before the primary support installation. For this reason temporary invert installation in top heading which provides completion of the temporary support ring near the excavation face is found to be very useful.

3. Experiences show that for weak to very weak rock, rigid lining is required to stop deformations immediately. Otherwise, long time deformations cause higher costs by means of reprofiling works (re-excavation of previously supported sections in order to compensate deformations beyond the design limits) and time loss. Consequently, as 60 cm rigid concrete shell was found to be necessary, primary elastic support philosophy of New Austrian Tunneling Method failed in Bolu Mountain Tunnels.

4. In order to keep risk of earthquake damage at minimum it is essential to keep final lining as close as possible to the excavation face. This distance was approximately 100 meters in order to avoid the inference of excavation and primary support work activities.

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APPENDIX I

IN-SITU & LABORATORY TEST RESULTS

In-situ and laboratory test results presented in this appendix is taken from the geological report provided to Astaldi S.p.A. by Şimşek (2001) by the permission of Astaldi S.p.A.

A total number of 12 boreholes with a length of 1337 m have been drilled along the alternative tunnel alignment. Borehole depths were extended till 5 meters below the tunnel invert level. A list of boreholes which were carried out along the tunnel alignment is shown in *Table A1.1*.

Table A1.1 List of Drill Holes (Şimşek, 2001)

Drill no	Coordinates		Elevation (m)	Depth (m)	Location
	x	y			
BH 500	662619.63	512576.45	867.22	30	Portal
BH 501	662610.57	512523.77	863.50	36	Portal
BH 502	662805.24	512722.57	912.90	80	Tunnel
BH 503	662830.06	512694.67	914.14	80	Tunnel
BH 504	662961.11	512860.46	963.63	129	Tunnel
BH 505	623069.06	512917.33	972.33	145	Tunnel
BH 506	623064.37	513018.00	962.82	132	Tunnel
BH 507	623160.90	513088.85	957.22	135	Tunnel
BH 508	623137.23	513177.49	955.48	130	Tunnel
BH 509	623177.17	513360.47	927.40	106	Tunnel
BH 510	622641.13	512589.92	871.47	35.1	Portal
BH 511	622641.13	512589.92	872.47	33.7	Portal
BH 512	622610.57	512523.77	863.50	32.6	Portal
BH 516	622732.34	512588.90	927.67	80	Tunnel
BH 517	622731.64	512645.42	920.03	75	Tunnel
BH 518	622920.88	512741.09	936.68	105	Tunnel
BH 519	623209.39	513252.51	954.59	135	Tunnel
BH 520	622619.90	512536.60	865.20	18	Portal

Pressuremeter tests

Pressuremeter tests were carried out in four selected boreholes, which drilled through all geotechnical units to be encountered along tunnel alignment and portal area.

Assumed bulk weight:	20kN/m3
Required pressure at Point A:	50% of overburden
Required pressure at Point A1:	20% of overburden
Required pressure at Point B:	100% of overburden
Required pressure at Point B1:	50% of overburden
Required pressure at Point C:	failure*
Required pressure at Point C1:	0% of overburden

*Failure**: failure of soil or maximum pressure of pressuremeter or maximum expansion of pressuremeter.

Pressure steps are shown in *figure A1.1*. Borehole numbers, test depths and pressure steps are shown in *Table A1.2*.

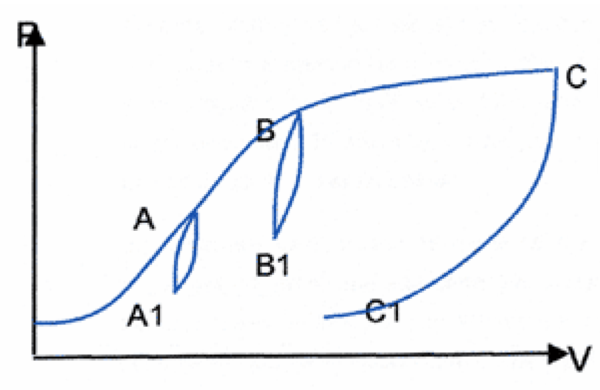


Figure A1.1: Guidance for loading/Unloading Cycle (Şimşek, 2001)

Table A1.2 Test Depth and Applied Pressure for Loading and Unloading (Şimşek, 2001)

Borehole No	Chainage	Test depth (m)	Pressures (bar)					
			A	A1	B	B1	C	C1
BH 502	53+716	38.6	4	1.7	8	4	50.1	0
		47.9	5	2	10	5	20	0
		61.2	6	failure at 6 bars				
BH 506	53+317	102.5	10	4	20	10	50	0
		112.5	11	4.4	22	11	50	0
BH 516	63+965	38.25	3.8	1.6	7.8	3.9	50	0
		49.3	5.2	2	10	5	50	0
		56.85	6	2.4	12	6	30	0
		67.85	7	2.8	14	7	50	0
		75.3	7.7	2.8	15	8	50	0
BH 520	64+122	6.2	3	1	6	3	20	0
		10	6	2	12	6	25	0
		15	8.9	3	18	9	25	0

Uniaxial Compressive Tests Results

Uniaxial test results are summarized on *Table A1.3*.

Table A1.3 Uniaxial Compressive Test and Density (Şimşek, 2001)

Borehole No	Depth (m)	Lithology	Diameter (cm)	Density (kN/m^3)	Failure Load (kg)	σ_c (Mpa)	Elastic module (Mpa)
BH 504	88.00-88.25	MCP	6.3	27.01	3515	11.35	
BH 505	101.10-102.00	Sandstone	6.3	21.39	3810	12.31	
BH 505	103.00-103/70	Sandstone	6.3	21.48	1715	5.49	
BH 505	120.00-120.30	Claystone	6.3	19.74	980	3.23	
BH 505	136.45-136.75	Sandstone	6.3	20.27	770	2.5	
BH 506	124.50-125.00	MCB	6.3	-	7750	24.6	10400
BH 506	124.50-125.00	MCB	6.3	-	6200	19.7	8600
BH 507	101.50-102.00	Sandstone	6.3	23.79	8100	26.08	
BH 507	105.20-105.80	Sandstone	6.3	21.07	2080	6.68	
BH 507	111.65-112.15	Sandstone	6.3	21.58	2130	6.68	4270
BH 507	116.55-117.00	Sandstone	6.3	20.70	905	2.9	
BH 507	126.15-126.50	MCB	6.3	23.42	550	1.77	2020
BH 508	95.77-96.14	Sandstone	6.3	20.89	1200	3.86	27000
BH 508	101.55-101.75	Sandstone	6.3	20.30	920	2.92	
BH 508	110.00-110.43	Sandstone	6.3	24.23	5100	16.37	93000
BH 508	113.00-113.40	Sandstone	6.3	19.51	710	2.227	2110
BH 508	118.90-119.40	Sandstone	6.3	20.28	1250	4.04	1830
BH 508	121.00-122.60	Sandstone	6.3	21.81	1350	4.35	16000
BH 518	58.70-59.00	Marble	6.3	26.59	8360	26.58	
BH 518	73.50-73.80	Marble	6.3	26.48	18350	58.34	
BH 518	78.90-79.15	Marble	6.3	26.39	14350	45.62	
BH 518	83.80-84.15	Marble	6.3	26.61	18350	58.52	
BH 518	95.05-95.35	MCB	6.3	25.43	958	3.09	
BH 519	82.00-82.20	Sandstone	6.3	22.51	2161	6.87	
BH 519	86.20-86.70	Sandstone	6.3	21.55	1640	5.18	
BH 519	92.25-92.50	Sandstone	6.3	20.53	1580	5.01	

Laboratory Tests on Reddish Clay

Shear strength, index tests and clay mineral analysis results are listed in following *Tables A1.4 and A1.5.*

Table A1.4 Shear Strength and Index Tests on Reddish Clay (Şimşek, 2001)

Borehole No	Depth (m)	Soil Type	C %	Density (kN/m^3)	Attaberg Limits			Class UCSC	Shear Box (CD)	
					LL%	PL%	PI%		C_p/C_r (kPa)	φ_p/φ_r
BH 502	61.15	Clay	74		100	31	69	CH		
BH 502	67.3	Clay	50		44	14	30	CL		
BH 502	72.65	Clay	43		41	16	25	CL		
BH 502	79.15	Clay	64		80	24	56	CH		
BH 504	110.6	Clay	53	17.95	100	33	67	CH	131/98	7/3
BH 504	115.3	Clay	43	-	92	42	41	MH		
BH 504	128.3	Clay	58	17.35	94	36	58	CH	168/12	9/9
BH 506	102.5	Clay	53	18.73	68	25	47	CH		
BH 507	122.2	Clay	38	22.17	78	32	44	ML	266/198	5/5
BH 518	100.1	Clay	36	16.89	81	32	49	CH	127/44	5/5
BH 519	100.7	Clay	64	17.35	82	42	40	MH	60/52	14/7

Notes: C% clay percentage
C_p/C_r cohesion peak/residual
φ_p/φ_r friction angle peak/residual

Table A1.5 Clay Mineral Analysis (Şimşek, 2001)

Borehole No	Depth (m)	Mineral content in clay fraction (<µm)	Partcile size distrubution	Percantage of smectite in bulk sample
BH 502	61.0-62.0	Smectite 99%	< 40µm = 72% <2 m = 33%	Sm_{eff} = 33%
BH 504	109.5-109.6	Smectite 99%	< 40µm = 72% <2 m = 33%	Sm_{eff} = 46%

Clay mineral analysis indicates that the fault gouge clay of low angle thrust fault zone contains extremely high smectite content.

Point Load Tests

Point load tests have been carried out on samples form boreholes BH 504, BH 505, BH 506, BH 507, BH 509, BH 516, BH 517, BH 518, BH 519 and BH 510. Results are listed below in *Tables A1.6 to A1.15*.

Table A1.6 Point Load Tests from BH 504 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $ls_{(50)}$ (Mpa)	UCS (ls*24) (Mpa)	Type of failure
61.35-61.50	Limestone	17	63.5	4.22	1.11	4.68	109.9	Through intact rock
62.50-62.65	Limestone	10.5	63.5	2.60	1.11	2.89	69.3	"
64.75-64.90	Mylonite	0.47	63.5	0.12	1.11	0.13	3.1	Through matrix
70.00-70.20	Limestone	14	63.5	3.47	1.11	3.85	92.4	Through intact rock
71.00-71.15	Mylonite	0.6	63.5	0.15	1.11	0.17	4	Along slickensided
88.70-88.80	MCB	1.95	63.5	0.48	1.11	0.54	12.9	Through intact rock
90.00-90.20	MCB	2.55	63.5	0.63	1.11	0.70	16.8	"
102.90-103.00	MCB	1	63.5	0.25	1.11	0.28	6.7	"
106.10-106.30	MCB	3.55	63.5	0.88	1.11	0.98	23.5	"

Table A1.7 Point Load Tests from BH 505 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $ls_{(50)}$ (Mpa)	UCS (ls*24) (Mpa)	Type of failure
87.5-87.7	sandstone	0.45	63.5	0.112	1.11	0.124	3	Through intact rock
97.7-97.85	sandstone	0.6	63.5	0.149	1.11	0.165	4.9	"
102.0-102.1	sandstone	1.65	63.5	0.409	1.11	0.454	10.9	"
105.25-105.45	sandstone	1.35	63.5	0.335	1.11	0.372	9	"
108.1-108.2	sandstone	0.75	63.5	0.186	1.11	0.206	4.9	"
112.4-112.5	sandstone	8	63.5	1.980	1.11	2.200	52.8	"
117.6-117.8	sandstone	0.95	63.5	0.236	1.11	0.262	6.2	"
119.1-119.25	sandstone	0.75	63.5	0.186	1.11	0.206	4.9	"
126.3-126.6	sandstone	0.8	63.5	0.198	1.11	0.220	5.2	"
129.4-129.55	sandstone	0.9	63.5	0.223	1.11	0.248	6	"
130.6-130.8	sandstone	1.35	63.5	0.335	1.11	0.372	9	"
136.3-136.45	sandstone	2.15	63.5	0.533	1.11	0.592	14.2	"
140.4-140.6	sandstone	1.4	63.5	0.347	1.11	0.385	9.2	"
143.1-143.3	sandstone	0.9	63.5	0.223	1.11	0.248	6	"

Table A1.8 Point Load Tests from BH 506 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $ls_{(50)}$ (Mpa)	UCS (ls*24) (Mpa)	Type of failure
83.8-84.0	sandstone	2.2	63.5	0.546	1.11	0.606	14.5	Through intact rock
84.6-85.0	sandstone	2.2	63.5	0.546	1.11	0.606	14.5	"
86.0-86.2	sandstone	1.7	63.5	0.68	1.11	0.755	18.1	"
87.7-87.9	sandstone	1.7	63.5	0.422	1.11	0.468	11.2	"
93.3-93.5	sandstone	1.4	63.5	0.347	1.11	0.385	9.2	"
94.5-94.7	sandstone	3.0	63.5	0.744	1.11	0.826	19.8	"
99.6-99.8	sandstone	1.2	63.5	0.298	1.11	0.33	7.9	"
106.6-106.8	MCB	1.0	63.5	0.248	1.11	0.275	6.6	"
108-108.1	MCB	3.3	63.5	0.818	1.11	0.908	21.8	"
116.0-116.3	marble	6.0	63.5	1.49	1.11	1.65	39.6	"
117.4-117.6	marble	10.5	63.5	2.60	1.11	2.89	69.4	"
125.0-125.15	MCB	4.3	63.5	1.07	1.11	1.18	28.3	"
127.25-127.5	MCB	2.35	63.5	0.583	1.11	0.647	15.5	Through joint
128.0-128.2	MCB	2.8	63.5	0.694	1.11	0.771	18.5	Through intact rock
129.6-129.75	MCB	1.5	63.5	0.372	1.11	0.413	9.9	"

Table A1.9 Point Load Tests from BH 507 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $ls_{(50)}$ (Mpa)	UCS (ls*24) (Mpa)	Type of failure
127.2-127.4	MCB	0.25	63.5	0.06	1.11	0.066	1.6	Through joint
128.0-128.15	MCB	1.80	63.5	0.45	1.11	0.49	11.8	Through intact rock
128.3-128.5	MCB	0.50	63.5	0.12	1.11	0.13	3.1	Through joint
129.2-129.35	MCB	0.20	63.5	0.05	1.11	0.055	13.3	"
130.5-130.7	MCB	0.35	63.5	0.09	1.11	0.099	2.4	"
130.5-130.7	MCB	0.40	63.5	0.10	1.11	0.11	2.6	"

Table A1.10 Point Load Test From BH 509 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $ls_{(50)}$ (Mpa)	UCS (ls*24) (Mpa)	Type of failure
84.40-84.70	MCB	0.1	63.5	0.025	1.11	0.028	0.6	Through intact rock
87.00-87.15	MCB	0.25	63.5	0.062	1.11	0.069	1.6	"
88.65-88.75	MCB	2.5	63.5	0.62	1.11	0.69	16	"
90.50-90.65	MCB	0.1	63.5	0.025	1.11	0.028	0.6	"
92.40-92.50	MCB	0.5	63.5	0.124	1.11	0.138	3.3	"
95.40-95.55	MCB	0.3	63.5	0.074	1.11	0.083	1.9	"
96.35-96.65	MCB	0.35	63.5	0.087	1.11	0.096	2.3	"
96.95-97.25	MCB	0.3/0.4	63.5	0.074 0.099	1.11	0.083 0.110	1.9 2.6	"
98.10-98.25	MCB	0.75	63.5	0.186	1.11	0.206	4.9	"
102.80-102.90	MCB	0.2	63.5	0.05	1.11	0.055	1.3	"
105.50-105.70	quartz-breccia	4.65	63.5	1.15	1.11	1.28	30.7	"
105.90-106.00	quartz-breccia	1.6	63.5	0.397	1.11	0.44	10.6	"

Table A1.11 Point Load Tests form BH 516 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $Is_{(50)}$ (Mpa)	UCS (Is*24) (Mpa)	Type of failure
18-18.15	sandstone	15.0	63.5	3.72	1.11	4.13	99.2	Through intact rock
22.5-22.8	brecciate limestone	19.0	63.5	4.71	1.11	5.23	125.5	"
26.5-26.8	calcerous sandstone	21.0	63.5	5.21	1.11	5.78	138.7	"
27.5-27.75	sandstone	21.0	63.5	5.21	1.11	5.78	138.7	"
31.0-31.2	calcerous sandstone	13.0	63.5	3.22	1.11	3.58	85.9	"
39.4-39.5	brecciate conglomer ate	6.0	63.5	1.49	1.11	1.65	39.6	"
39.5-39.6	brecciate conglomer ate	7.0	63.5	1.74	1.11	1.93	46.3	"
48.3-48.45	brecciate limestone	14.0	63.5	3.47	1.11	3.85	92.4	"
49.2-49.35	brecciate conglomer ate	14.0	63.5	3.47	1.11	3.85	92.4	"
51.5-51.7	"	11.0	63.5	2.73	1.11	3.03	72.7	"
52.5-52.7	"	10.0	63.5	2.48	1.11	2.75	63.6	"
58.0-58.2	limestone	23.0	63.5	5.70	1.11	6.33	151.9	"
64.5-64.9	calcerous sandstone	19.0	63.5	4.71	1.11	5.23	123.5	"
68.0-68.2	mylonite	0.3	63.5	0.07	1.11	0.08	1.92	"
78.0-78.15	calcerous sandstone	14.0	63.5	3.47	1.11	3.85	92.4	"
78.5-78.65	"	14.0	63.5	3.47	1.11	3.85	92.4	"

Table A1.12 Point Load Tests from BH 517 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $Is_{(50)}$ (Mpa)	UCS (Is*24) (Mpa)	Type of failure
29.30-29.40	marly limestone	16	63.5	3.97	1.11	4.40	106	Through intact rock
38.10-38.30	marly limestone	9	63.5	2.23	1.11	2.48	60	"
39.65-40.00	marly limestone	35	63.5	8.68	1.11	9.63	231	"
90.50-90.65	marly limestone	27	63.5	6.70	1.11	7.43	178	"
92.40-92.50	calcerous sandstone	15	63.5	3.72	1.11	4.13	99	"

Table A1.13 Point Load Tests from BH 518 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $Is_{(50)}$ (Mpa)	UCS (Is*24) (Mpa)	Type of failure
57.70-57.90	brecciate marble	1.20	63.5	0.298	1.11	0.330	7.9	Through slickensided
59.50-59.80	brecciate marble	8.50	63.5	2.108	1.11	2.340	56.0	Through intact rock
61.80-62.00	brecciate marble	3.50	63.5	0.868	1.11	0.963	23.0	"
62.80-63.00	brecciate marble	4.60	63.5	1.141	1.11	1.266	30.0	"
64.60-64.75	brecciate marble	4.60	63.5	1.141	1.11	1.266	30.0	"
65.50-65.75	brecciate marble	2.50	63.5	0.62	1.11	0.688	16.5	"
70.15-70.30	brecciate marble	4.20	63.5	1.04	1.11	1.156	27.7	"
73.10-73.25	brecciate marble	4.60	63.5	1.141	1.11	1.266	30.0	"
76.70-76.90	brecciate marble	15.00	63.5	3.72	1.11	4.129	99.0	"
77.50-77.95	brecciate marble	5.00	63.5	1.24	1.11	1.376	33.0	"
78.70-78.90	brecciate marble	11.50	63.5	2.852	1.11	3.166	76.0	"
79.45-79.60	brecciate marble	9.50	63.5	2.356	1.11	2.615	62.7	"

Table A1.13 Cont'd

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index ls(50) (Mpa)	UCS (ls*24) (Mpa)	Type of failure
80.60-80.75	brecciate marble	5.00	63.5	1.24	1.11	1.376	33.0	"
84.30-84.50	brecciate marble	3.00	63.5	0.744	1.11	0.826	19.8	"
86.00-86.20	brecciate marble	5.00	63.5	1.24	1.11	1.376	33.0	"
87.50-87.65	fault breccia	2.50	63.5	0.62	1.11	0.688	16.5	"
87.65-87.80	fault breccia	0.80	63.5	0.198	1.11	0.220	5.2	"
87.80-88.00	brecciate marble	14.00	63.5	3.472	1.11	3.854	92.5	"
88.80-89.00	brecciate marble	19.00	63.5	4.712	1.11	5.230	124.5	"
90.50-90.65	amphibolite	2.80	63.5	0.694	1.11	0.771	18.5	"
91.90-92.10	amphibolite	3.15	63.5	0.781	1.11	0.867	20.8	"
93.10-93.30	amphibolite	0.20	63.5	0.05	1.11	0.055	1.3	"
95.35-95.50	amphibolite	0.10	63.5	0.025	1.11	0.028	0.67	"
102.70-102.85	brecciate sandstone	11.50	63.5	2.852	1.11	3.166	75.9	"

Table A1.14 Point Load Tests from BH 519 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $ls_{(50)}$ (Mpa)	UCS (ls*24) (Mpa)	Type of failure
59.85-60.10	sandstone	2.25	63.5	0.55	1.11	0.61	14.6	Through intact rock
64.40-64.55	sandstone	1.20	63.5	0.30	1.11	0.33	7.9	"
69.70-69.80	sandstone	0.55	63.5	0.14	1.11	0.15	3.6	"
71.90-72.10	sandstone	1.75	63.5	0.43	1.11	0.48	11.5	"
75.40-75.60	sandstone	1.40	63.5	0.35	1.11	0.39	9.4	"
80.50-80.65	sandstone	1.20	63.5	0.30	1.11	0.33	7.9	Through slickenside
86.70-86.90	sandstone	2.00	63.5	0.50	1.11	0.55	13.2	Through intact rock
89.20-89.80	sandstone	0.90	63.5	0.22	1.11	0.25	6.0	"
91.60-91.80	sandstone	0.60	63.5	0.15	1.11	0.17	4.1	"
94.30-94.50	brecciate sandstone	2.00	63.5	0.50	1.11	0.55	13.2	"
97.60-97.80	sandstone	2.20	63.5	0.55	1.11	0.61	14.6	"
104.5-104.7	sandstone	2.30	63.5	0.57	1.11	0.63	15.1	"
108.7-108.9	sandstone	3.00	63.5	0.74	1.11	0.82	20.0	"
117.6-117.8	sandstone	2.05	63.5	0.51	1.11	0.56	13.4	"
122.5-122.65	MCB	0.05	63.5	0.012	1.11	0.013	0.30	Through slickenside
124.0-124.4	MCB	0.05	63.5	0.012	1.11	0.013	0.30	"
125.0-125.2	marble	3.00	63.5	0.74	1.11	0.82	20.0	"
126.5-126.7	marble	45	63.5	11.16	1.11	12.4	297	Through intact rock
127.0-127.3	marble	2.5	63.5	0.62	1.11	0.69	16.6	Through joint

Table A1.14 Cont'd

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index ls(50) (Mpa)	UCS (ls*24) (Mpa)	Type of failure
127.3-127.5	marble	40	63.5	9.90	1.11	11.0	264	Through intact rock
128.2-128.5	marble	16	63.5	3.90	1.11	4.30	103	"
129.5-129.7	marble	9.5	63.5	2.40	1.11	2.60	62.4	"
131.5-131.8	quartzite	13	63.5	3.20	1.11	3.60	86.4	Through joint
132.5-132.8	quartzite	5	63.5	1.20	1.11	1.37	32.8	"
134.4-134.6	amphibolite	1.8	63.5	0.44	1.11	0.50	12.0	Through intact rock
134.6-134.8	marble	1.4	63.5	0.35	1.11	0.39	9.4	"

Table A1.15 Point Load Tests from BH 520 (Şimşek, 2001)

Depth (m)	Lithology	Failure Load (kN)	Dia. De (mm)	P/De2 (Mpa)	Corr. Factor	Point load index $ls_{(50)}$ (Mpa)	UCS (ls*24) (Mpa)	Type of failure
5.30-5.50	brecciate sandstone	6.5	61.5	1.71	1.11	1.90	1.90	Through intact rock
13.20-13.50	calcerous sandstone	20.5	61.5	5.42	1.11	6.00	6.00	Through intact rock
15.10-15.30	calcerous sandstone	6.0	61.5	1.58	1.11	1.76	1.76	Through intact rock
17.50-17.70	calcerous sandstone	7.0	61.5	1.85	1.11	2.05	2.05	Through intact rock

APPENDIX II

PROJECT DRAWINGS

Simplified general excavation and support layouts of support classes C2, CM35, CM45, Option 3 and Option 4 are given in following pages. General layouts are prepared according to the following project drawings: (Astaldi S.p.A., 1993-2006)

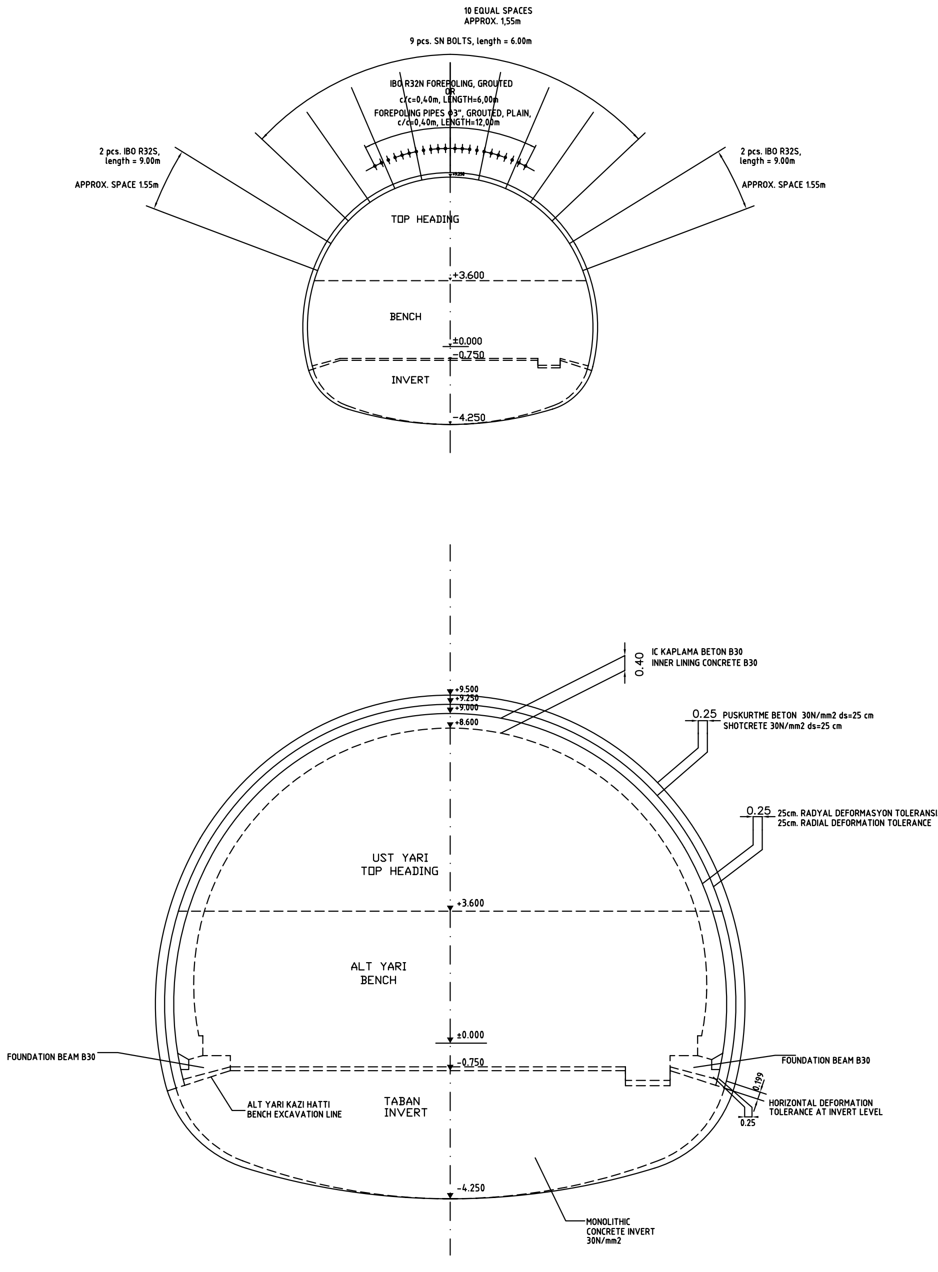
Class C2:
45.110/8382/2

Class CM/CM35:
45.110/9423 Rev.4, 45.110/9424 Rev.5, 45.110/9425 Rev.8, 45.110/9426 Rev.5

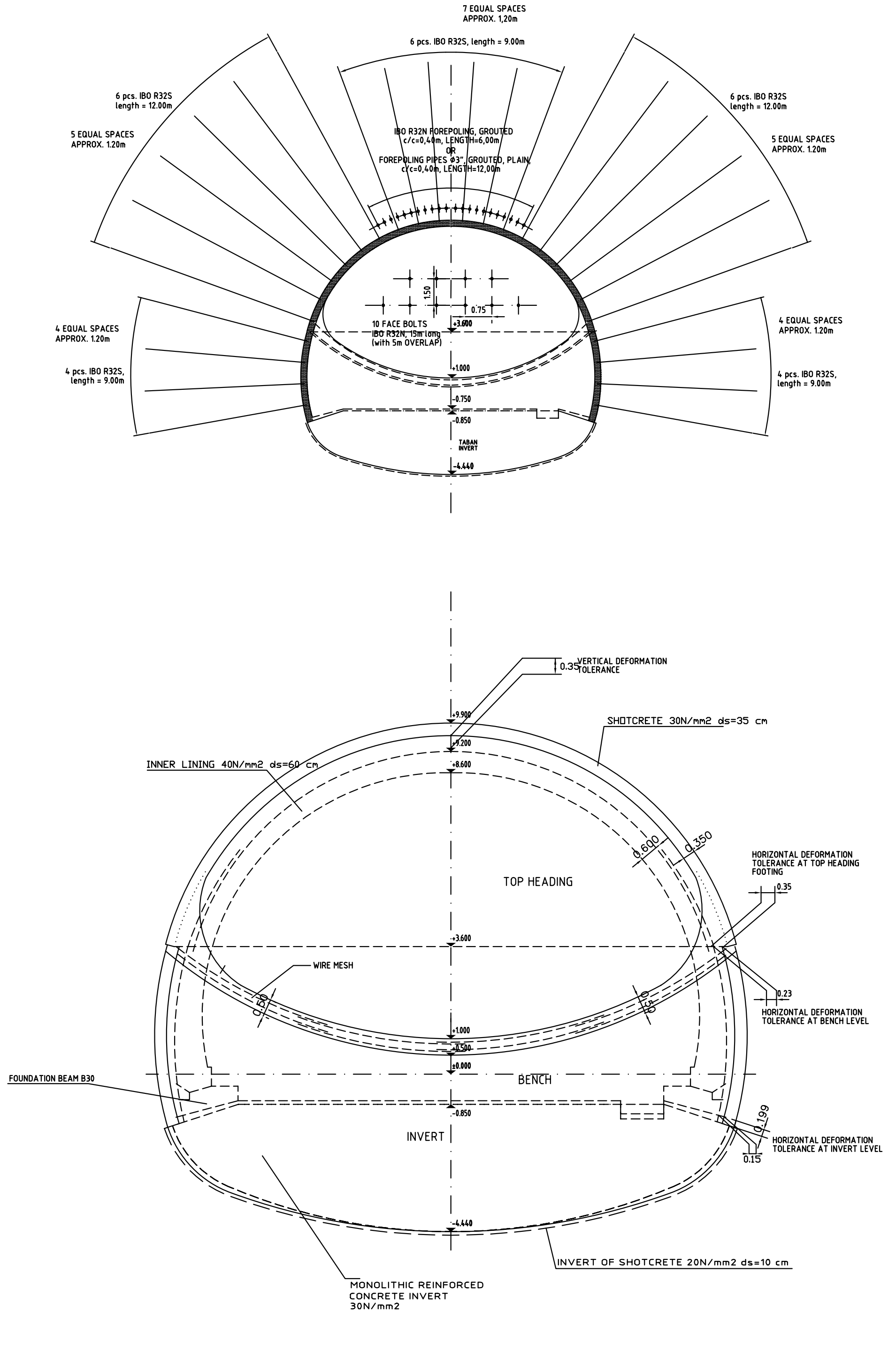
Class CM45:
45.110/TN/TUG/D/EX/300 Rev.1, 45.110/TN/TUG/D/EX/301 Rev.3,
45.110/TN/TUG/D/EX/302 Rev.3

Class Option 3:
45.110/9461 Rev.2, 45.110/9462 Rev.2, 45.110/9463-a, 45.110/9464 Rev.2

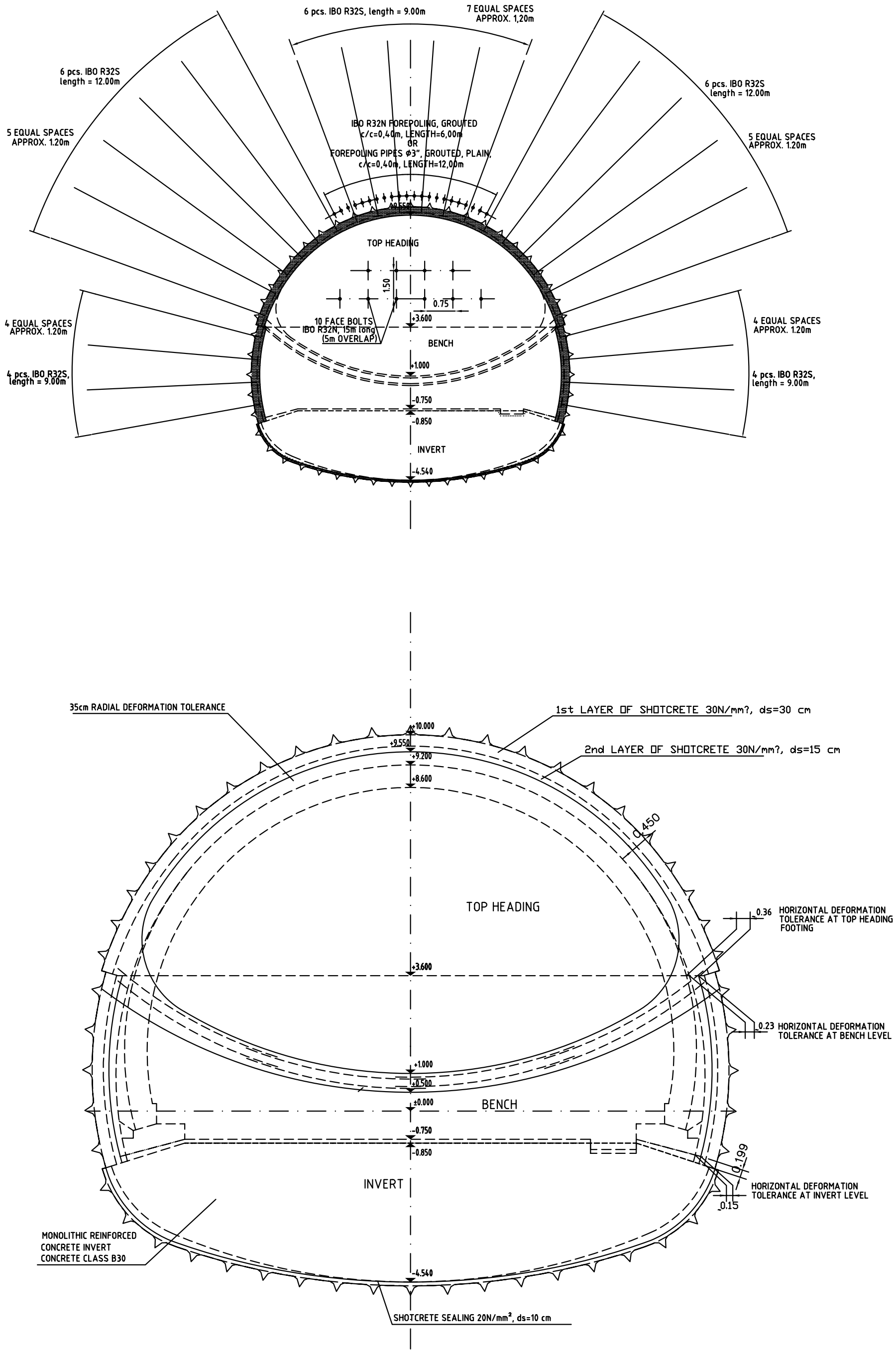
Class Option 4:
45.110/9430 Rev.2, 45.110/9432-B Rev.2, 45.110/9431 Rev.2



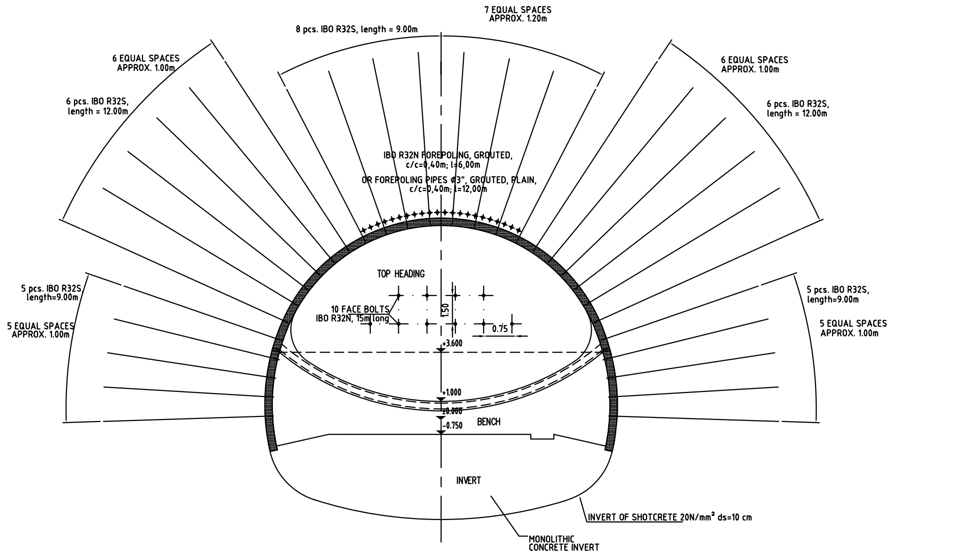
CLASS C2 SIMPLIFIED GENERAL LAYOUT	BASED ON PROJECT DRAWINGS: 45.110/8382/2 Rev.0

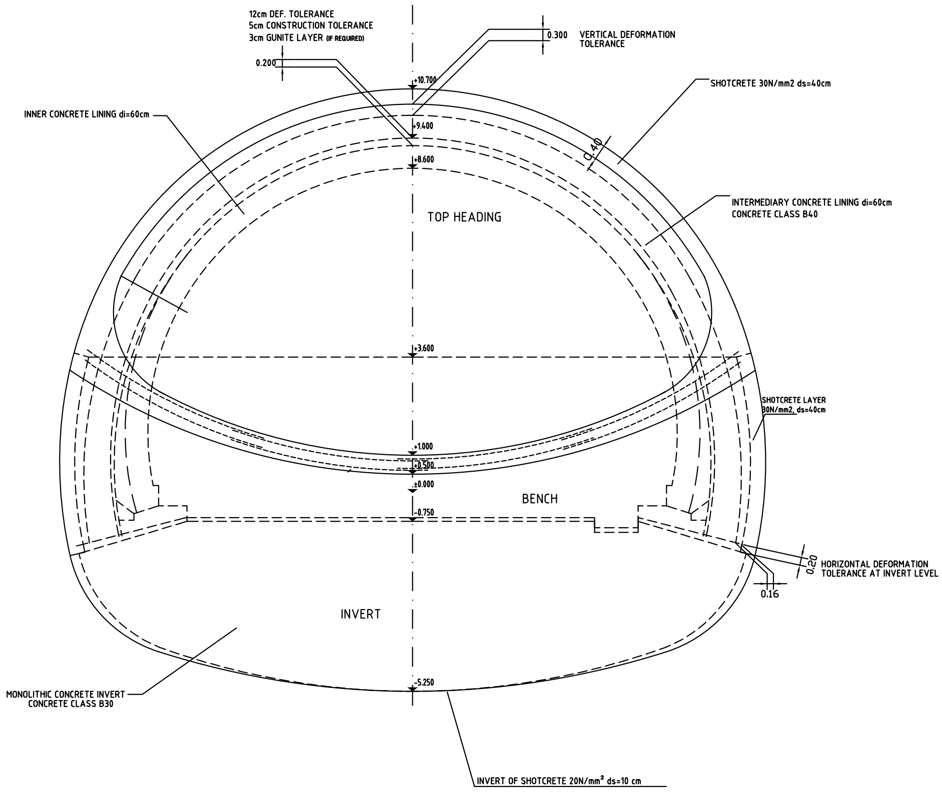


CLASS CM/CM35 SIMPLIFIED GENERAL LAYOUT	BASED ON PROJECT DRAWINGS: 45.110/9423 Rev-4 45.110/9424 Rev-5 45.110/9425 Rev-8 45.110/9426 Rev-5

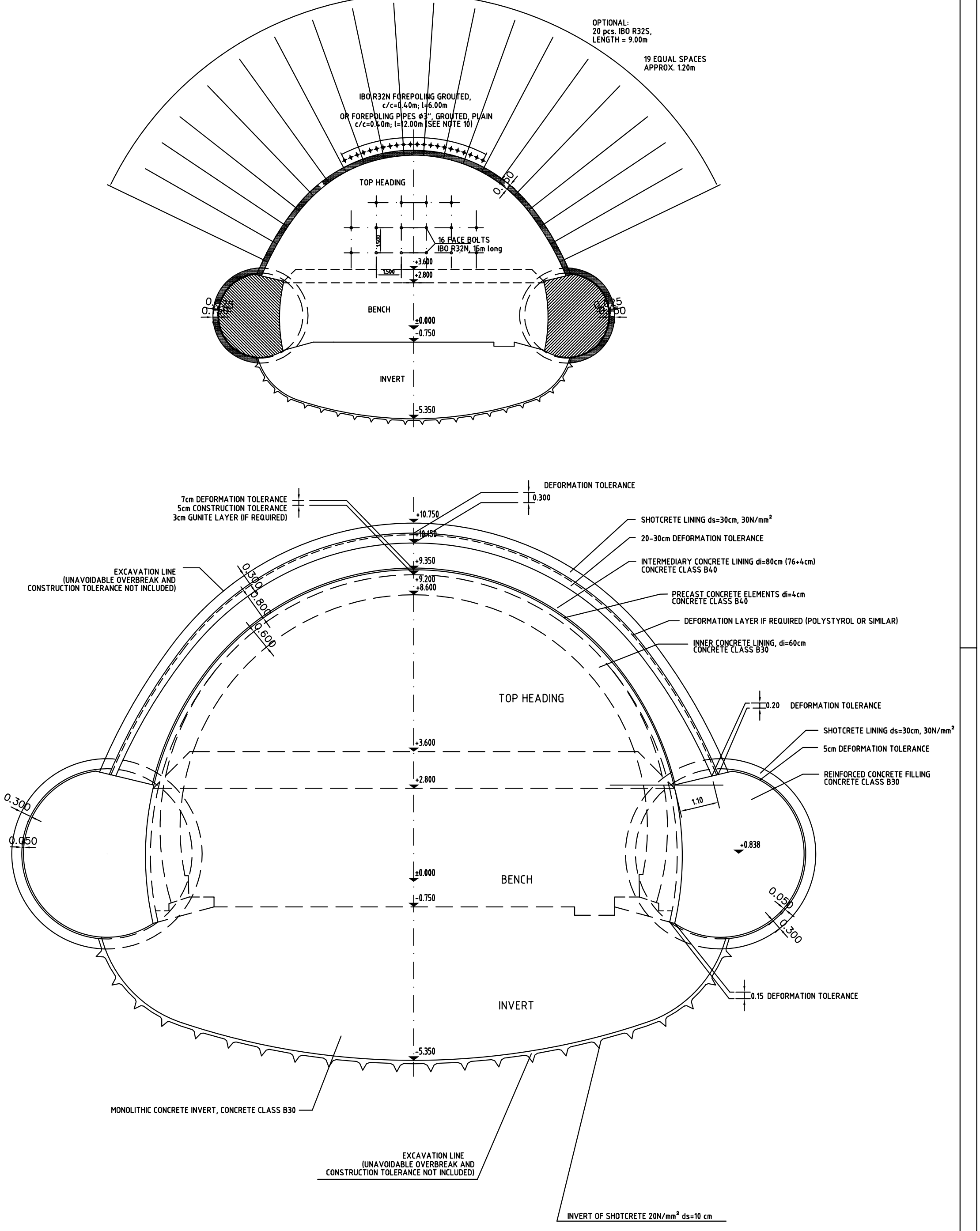


CLASS CM45 SIMPLIFIED GENERAL LAYOUT	BASED ON PROJECT DRAWINGS: 45.110/TN/TUG/D/EX/300 Rev.1 45.110/TN/TUG/D/EX/301 Rev.3 45.110/TN/TUG/D/EX/302 Rev.3





CLASS OPTION 3 SIMPLIFIED GENERAL LAYOUT	BASED ON PROJECT DRAWINGS: 45.110/9461 Rev.2 45.110/9463-a 45.110/9464 Rev.2 45.110/9462 Rev.2



CLASS OPTION 4 SIMPLIFIED GENERAL LAYOUT	BASED ON PROJECT DRAWINGS: 45.110/9430 Rev.2 45.110/9432-B Rev.2 45.110/9431 Rev.2

APPENDIX III

SITE PICTURES

Picture A3.1 Top Heading Excavation

You may see on above picture (A3.1) very beginning of the top heading excavation carried by jackhammer. Please note that shotcrete sealing of the tunnel face. Jackhammer will first remove that sealing.

Picture A3.2 Face Sealing Application

And here on picture A3.2 you may see the application of sealing shotcrete on freshly excavated face. This will protect tunnel face until next top heading excavation. Face sealing reduces lateral deformations and increases work safety.

Picture A3.3 Omega TH 29 Steel Ribs

Mining originated, Omega TH29 profile steel ribs have been installed immediately after excavation. The connections of the rib sections are done with an overlap of the profiles fitting into each other and are connected with clamps (overlapping 30-60 cm. according to approved design drawings). These types of connections are arranged in order to compensate large deformations due to friction in the clamp connections.

Picture A3.3 Welding of “Tie Bars”

The connection between the previous ribs is performed by five steel bars (namely tie bars). Those steel bars are fixed at the steel ribs by welding.

Picture A3.4 Bench Excavation

Picture A3.4 shows completed bench excavation with access ramp on left side. Note that surveyor team taking deformation readings on top heading.

Picture A3.5 Jumbo Drill Rig Installing Round Bolts

In Picture A3.5, above, you can see the Installation of Rock Bolts by Jumbo drill. Below in Picture A3.6 face bolt application are shown.

Picture A3.6 Jumbo Drill Rig Installing Face Bolts

Picture A3.7 Invert Excavation

Monolithic Invert is the lower part of the cross section and the last excavated part of the primary support system. The final scope of invert is to achieve a "ring" closure and an arch-effect. The invert was performed as a reinforced monolithic concrete arch with a length of 4.4 m. (Pictures A3.7 and A3.8)

Picture A3.8 Invert Reinforcement Installation

Picture A3.9 Bernold Form

For the Bernold lining, pre-cast panels laid on H profile steel ribs are used as formwork. Those steel ribs are fixed in place by rock bolt extensions. Concrete is poured trough apertures located along the precast elements. Those precast panels fixed to the rib with steel wire remain embedded into the concrete cast permanently. The steel ribs are removed after concrete is hardened. (Pictures A3.9 and A3.10)

Figure A3.10 Bernold Concrete Pouring

Picture A3.11 Waterproof Membrane and Protective Felt of Inner Lining Concrete

Here with Picture A311 details of Inner Lining water sealing is shown. In order to prevent leakage of groundwater into the tunnel and to avoid any contact between water and inner lining concrete is sealed by waterproofing membrane and protective felt. The protective felt is a continuous filament non-woven polypropylene geotextile of uniform thickness and surface texture as require by the approved design drawings. The waterproofing membrane provides the sealing function. The protective felt protects the waterproofing membrane against damage from contact with the shotcrete surface, to prevent interlocking between shotcrete support and final lining and to provide drainage into the longitudinal lateral drainages.

Picture A3.12 Inner Lining Form

For the installation of inner lining a special formwork, which is moving on a rail system, is used. When the formwork is placed, workers install the steel reinforcement than the formwork lifted up to its exact position by its own hydraulics. Before the concrete pouring starts the formwork should be supported by wooden planks to take concrete loads away from hydraulic system. (Picture A3.12)

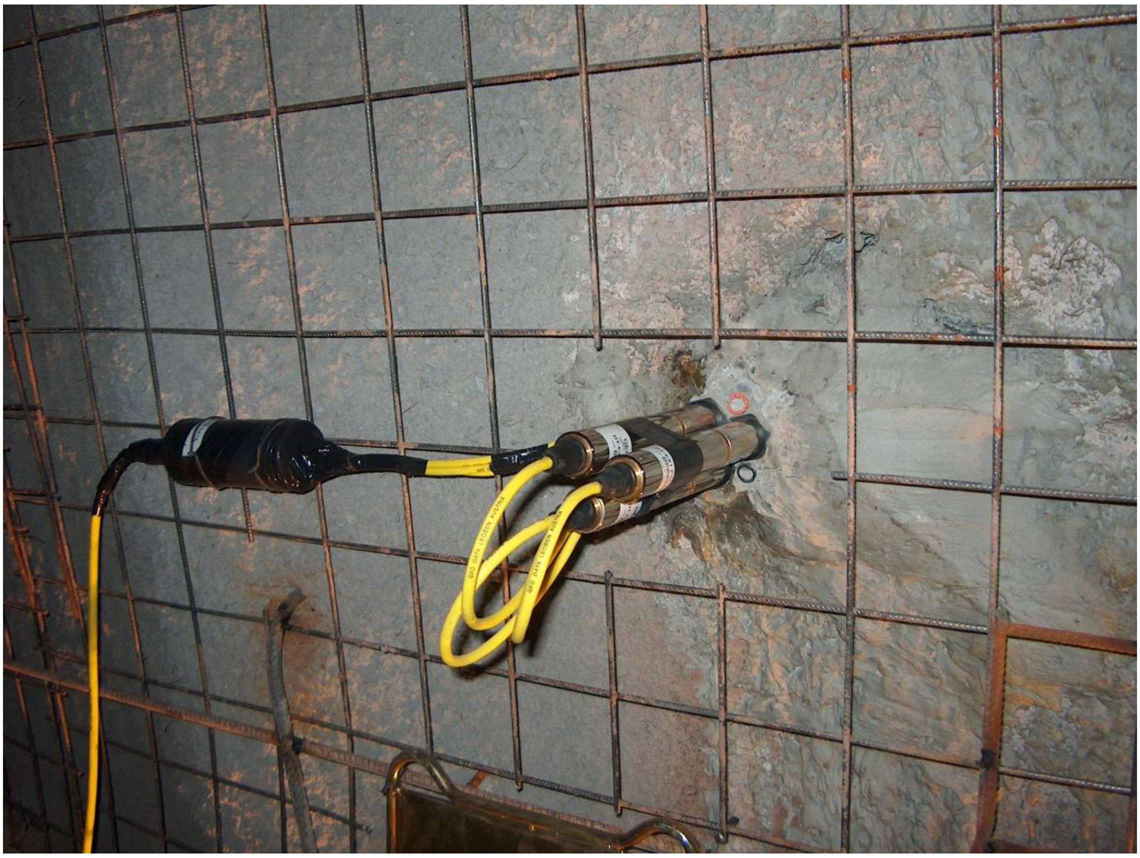
The formwork has steel panels as equipment to close the lateral spaces, but they cannot cover the whole free surface. Wood panels (together polystyrene panels) or wire mesh is used at the stop ends of the formwork. Final linings are cast as 13.6 meter long sections.

Picture A3.13 Seismic Joints

Several seismic joints are cutting (only in the Option 4 condition) the tunnel when crossing active faults. Those were 50 cm wide interruptions in the concrete lining. The scope of seismic joints is to absorb earthquake related ruptures and displacements without damaging the main structure. In inverts seismic joints were provided by foam concrete blocks. (Picture A3.13)

Picture A3.14 Pressure Cells and Strain Gauges

Pressure cells and strain gauges installed in inner lining is shown in Picture A3.14. Picture A3.15 display head of the rod extensometer left outside of the shotcrete.



Picture A3.15 Extensometer Head

APPENDIX IV

DEFORMATION READINGS

Deformation plots (Displacement vs. Time; S-H-L) prepared during the construction of Bolu Tunnel New Alignment Elmalık Side are presented in this appendix.

Locations of deformation reading stations are given in tables *A4.1* and *A4.2*

Table A4.1 Deformation Targets of Elmalık Left Tube

Station	Chainage		Station	Chainage		Station	Chainage
STN-01	64+141		STN-16	64+025		STB-31	63+893
STN-02	64+135		STN-17	64+016		STN-32	63+884
STN-03	64+129		STN-18	64+007		STN-33	63+875
STN-04	64+124		STN-19	63+998		STN-34	63+866
STN-05	64+118		STN-20	63+990		STN-35	63+857
STN-06	64+113		STN-21	63+981		STN-36	63+849
STN-07	64+103		STN-22	63+972		STN-37	63+840
STN-08	64+094		STN-23	63+963		STN-38	63+831
STN-09	64+085		STN-24	63+954		STN-39	63+822
STN-10	64+077		STN-25	63+946		STN-40	63+815
STN-11	64+068		STN-26	63+937		STN-41	63+806
STN-12	64+059		STN-27	63+928		STN-42	63+796
STN-13	64+050		STN-28	63+919		STN-43	63+788
STN-14	64+042		STN-29	63+911		STN-44	63+781
STN-15	64+034		STN-30	63+902			

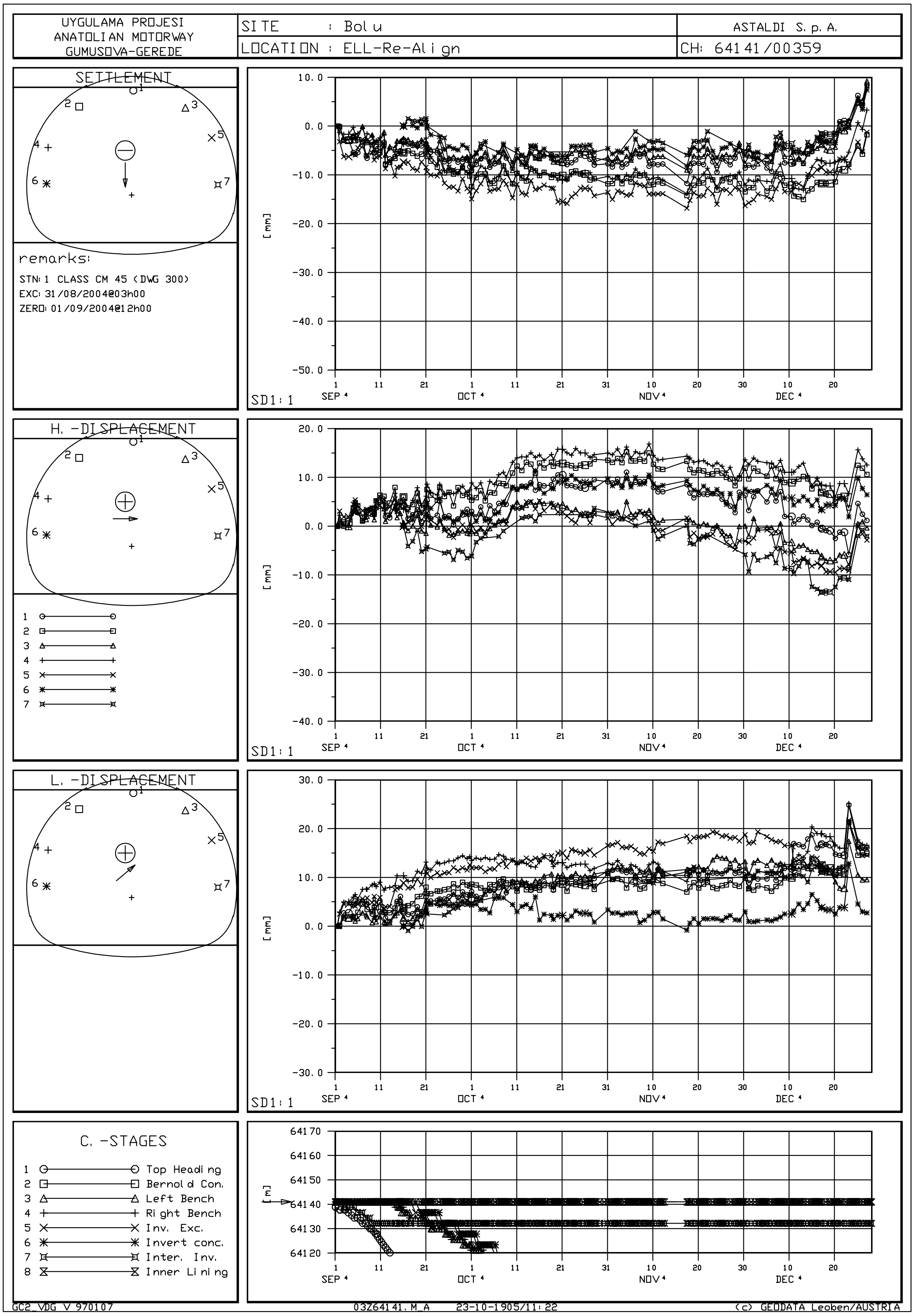
Table A4.2 Deformation Targets of Elmalık Right Tube

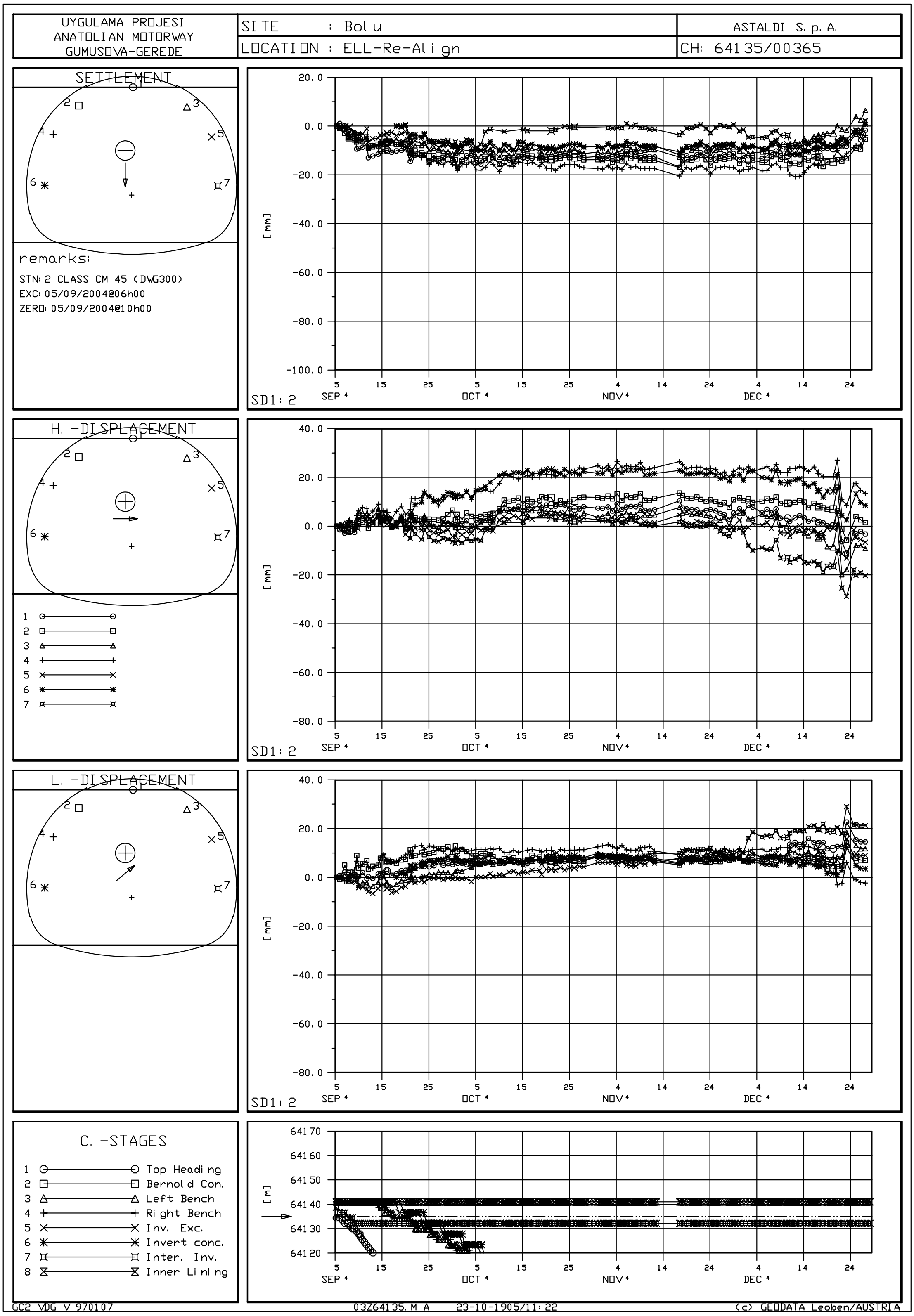
Station	Chainage		Station	Chainage			
STN-01	53+960		STN-08	53+911		STN-15	53+849
STN-02	53+955		STN-09	53+902		STN-16	53+840
STN-03	53+950		STN-10	53+893		STN-17	53+831
STN-04	53+945		STN-11	53+885		STN-18	53+823
STN-05	53+937		STN-12	53+876		STN-19	53+814
STN-06	53+928		STN-13	53+867		STN-20	53+805
STN-07	53+920		STN-14	53+858		STN-21	53+796

Left Tube

3D Monitoring (S-H-L) Diagrams

(Displacement vs. Time)


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 64141/00359
SETTLEMENT
remarks:
STN: 1 CLASS CM 45 (DWG 300)
EXC: 31/08/2004@03h00
ZERO: 01/09/2004@12h00
[mm]
SEP 4
OCT 4
NOV 4
DEC 4
SD1:1
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64141.M_A
23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA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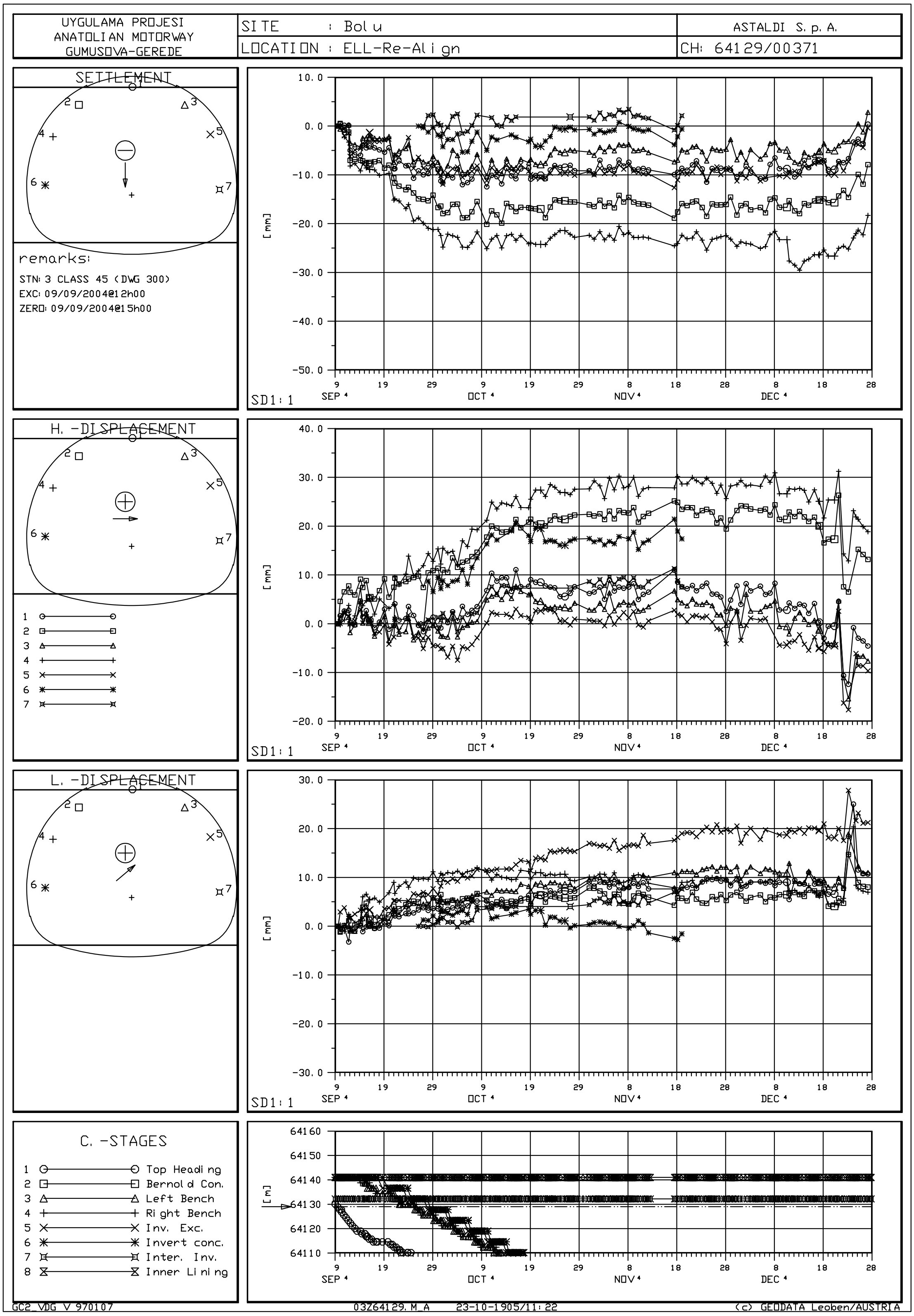
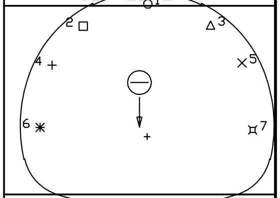
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 64135/00365
SETTLEMENT
remarks:
STN: 2 CLASS CM 45 (DWG300)
EXC: 05/09/2004@06h00
ZERO: 05/09/2004@10h00
[mm]
SD1: 2
SEP 4
OCT 4
NOV 4
DEC 4
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64135.M_A
23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 64129/00371
SETTLEMENT
remarks:
STN: 3 CLASS 45 (DWG 300)
EXC: 09/09/2004@12h00
ZERO: 09/09/2004@15h00
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[mm]
[m]
SD1:1
SEP 4
OCT 4
NOV 4
DEC 4
GC2_VDG V 970107
03Z64129.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 64124/00376

SETTLEMENT



remarks:

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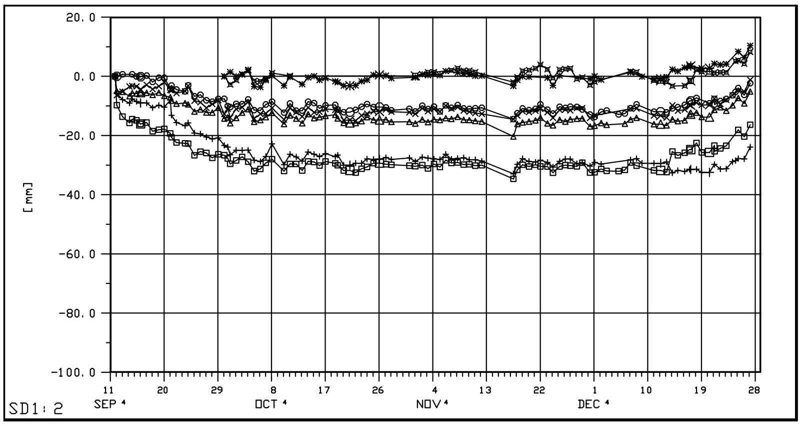

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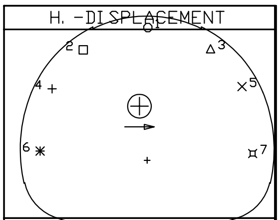
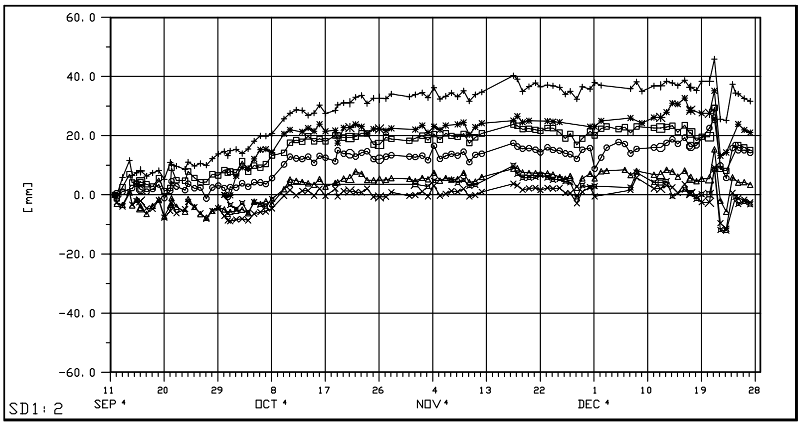


- 1 ○
- 2 □
- 3 △
- 4 +
- 5 ×
- 6 ✳
- 7 ⌑



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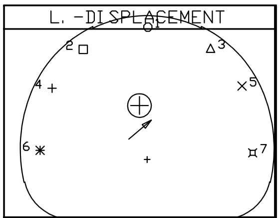
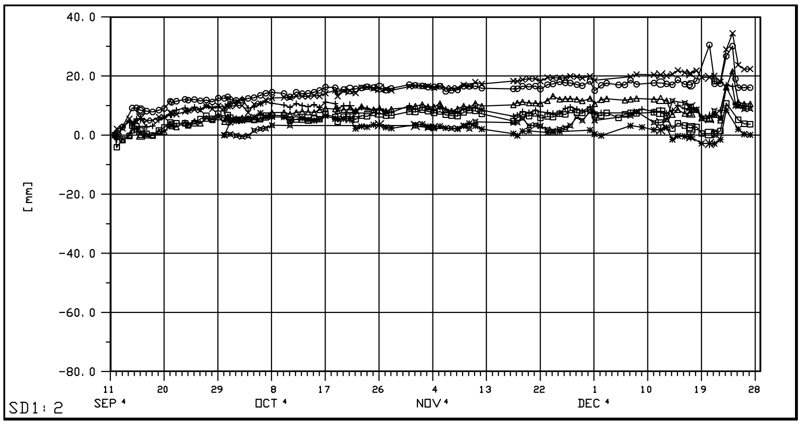
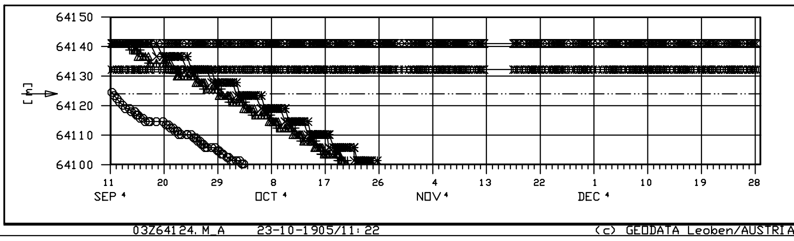


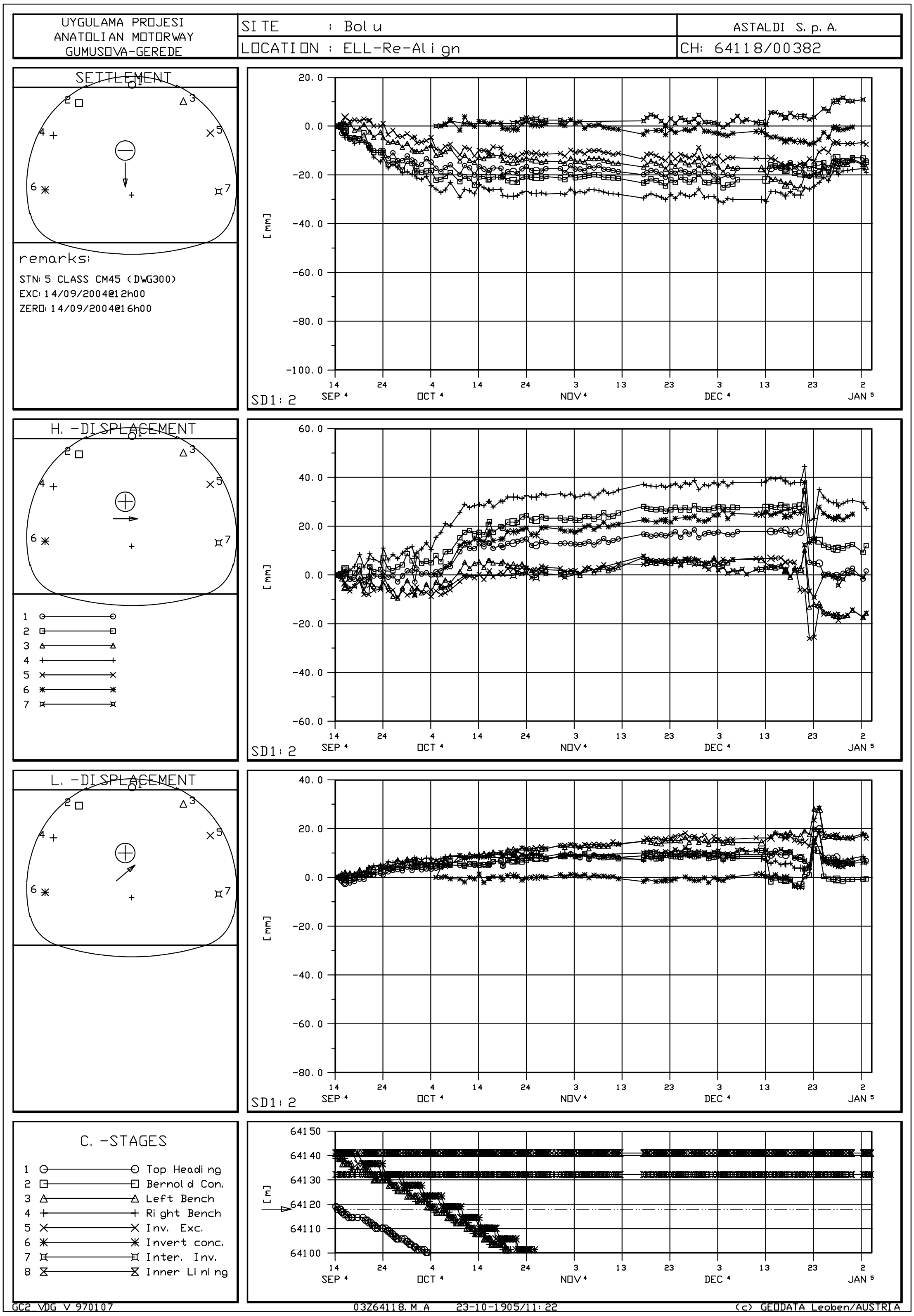


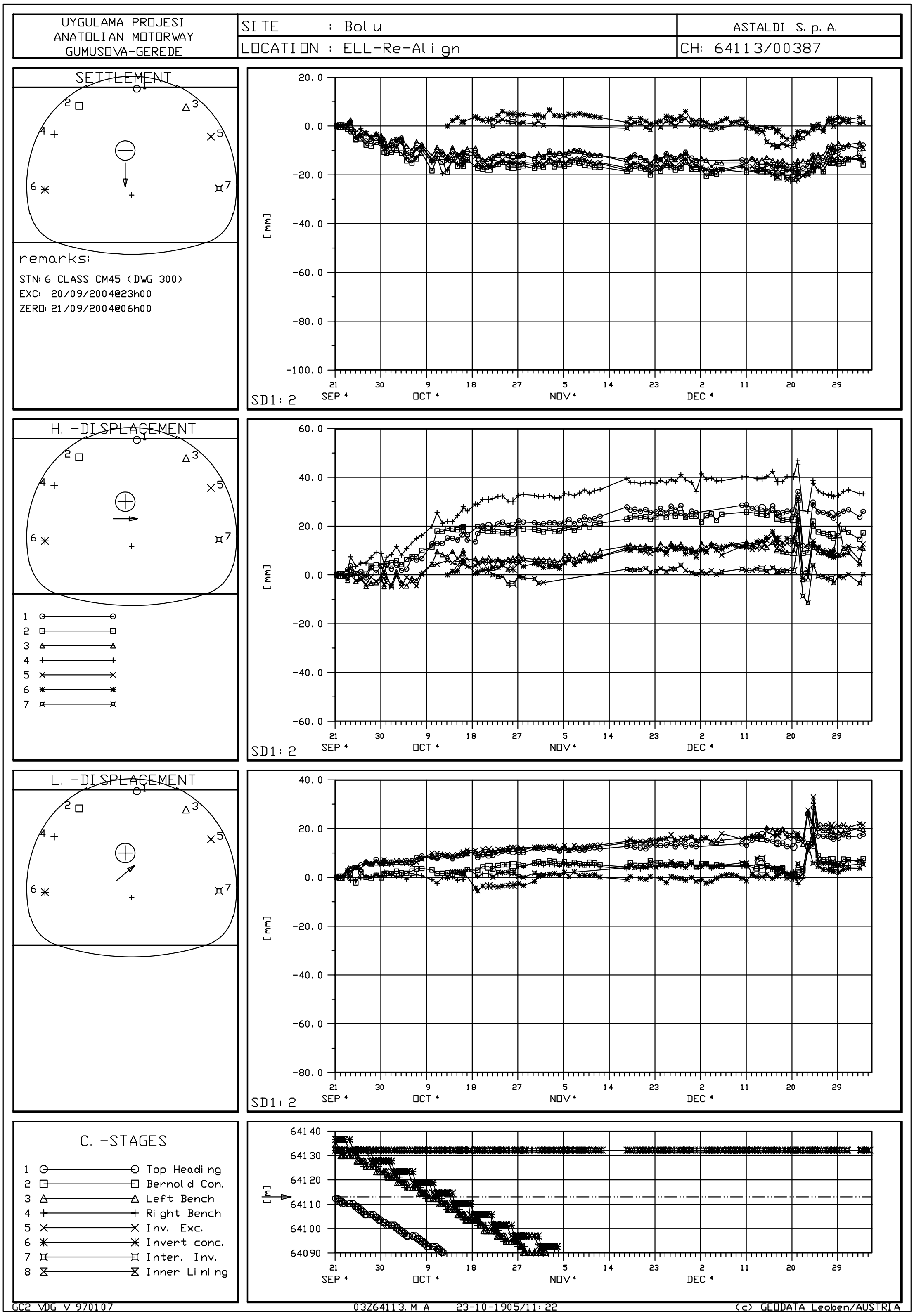
C.-STAGES

- 1 ○ Top Heading
- 2 □ Bernold Con.
- 3 △ Left Bench
- 4 + Right Bench
- 5 × Inv. Exc.
- 6 ✳ Invert conc.
- 7 ⌑ Inter. Inv.
- 8 ⧗ Inner Lining



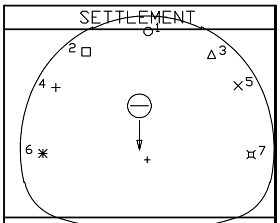
GC2_VDG V 970107 03Z64124.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 64118/00382
SETTLEMENT
remarks:
STN: 5 CLASS CM45 (DWG300)
EXC: 14/09/2004@12h00
ZERO: 14/09/2004@16h00
[mm]
SD1: 2
SEP 4
OCT 4
NOV 4
DEC 4
JAN 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64118. M_A 23-10-1905/11: 22
(c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 64113/00387
SETTLEMENT
remarks:
STN: 6 CLASS CM45 (DWG 300)
EXC: 20/09/2004@23h00
ZERO: 21/09/2004@06h00
[mm]
SD1: 2
SEP
OCT
NOV
DEC
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64113.M_A
23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 64103/00397

SETTLEMENT



remarks:

STN: 7 CLASS CM35 (DWG9424)
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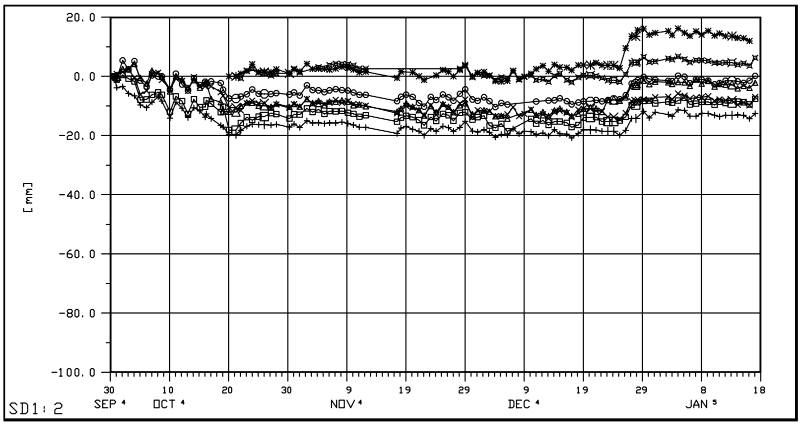
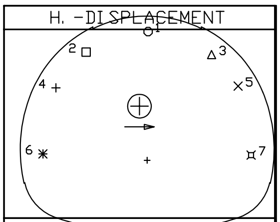
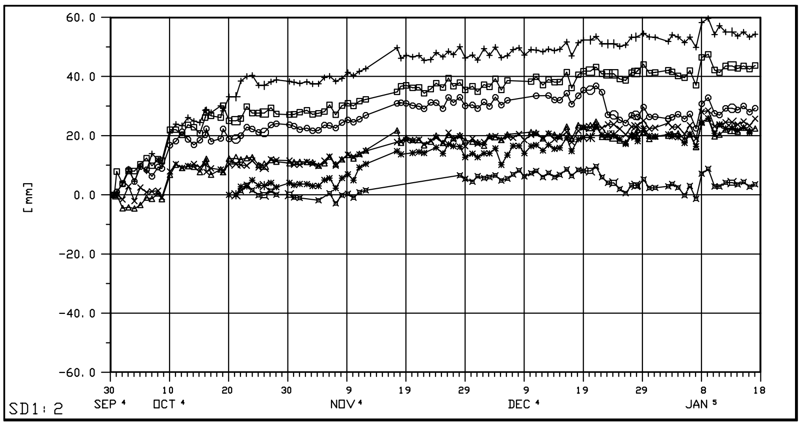
ZERO: 30/09/2004@16h00



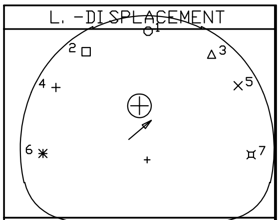
H.-DISPLACEMENT

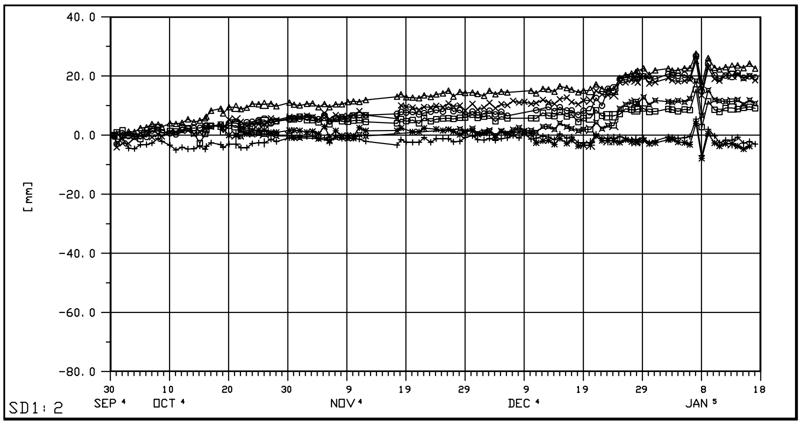


1 ○———○
2 □———□
3 △———△
4 +———+
5 ×———×
6 ✳———✳
7 ⌑———⌑



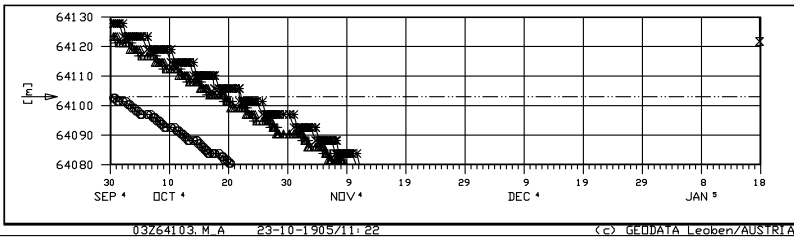
L.-DISPLACEMENT



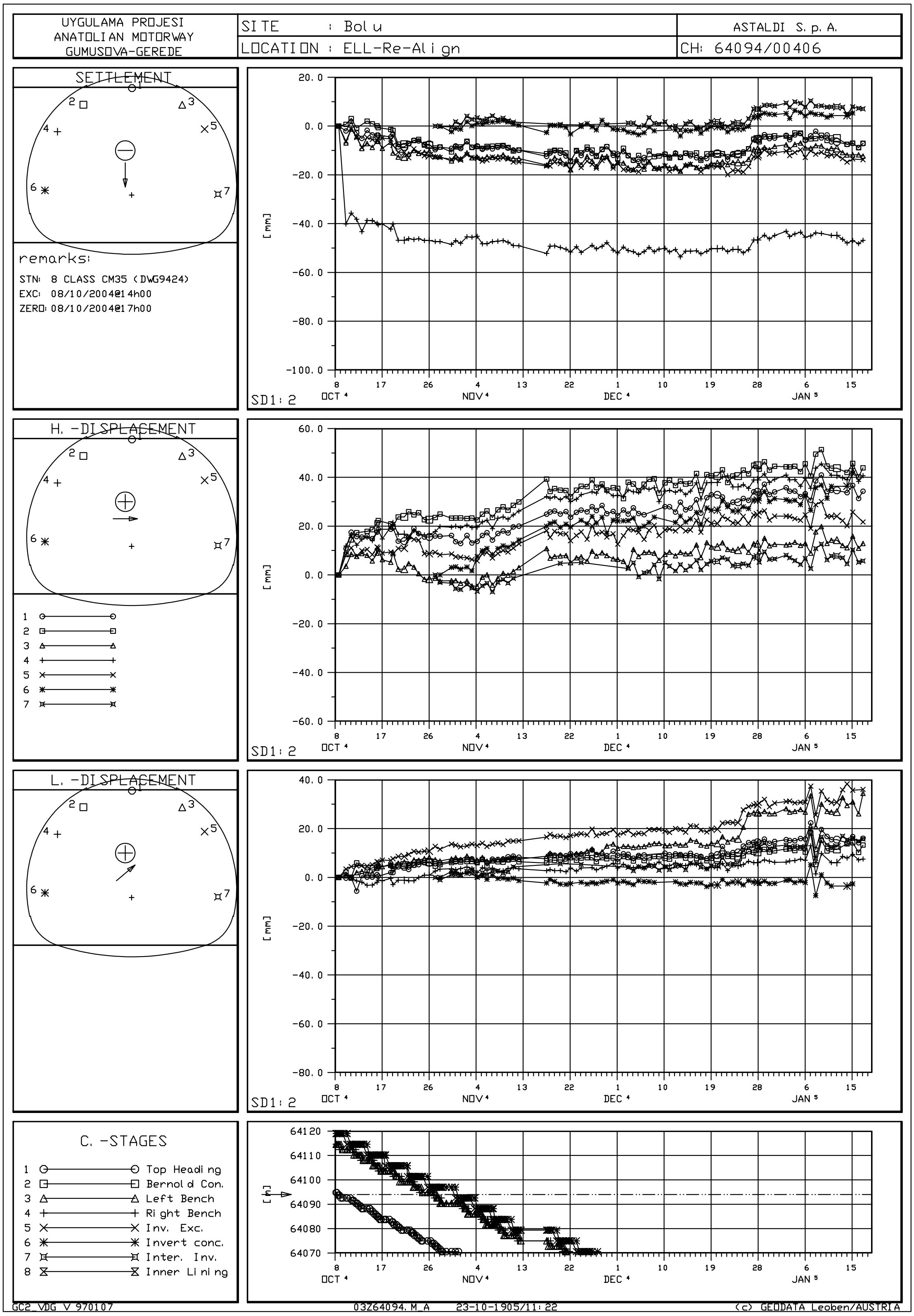


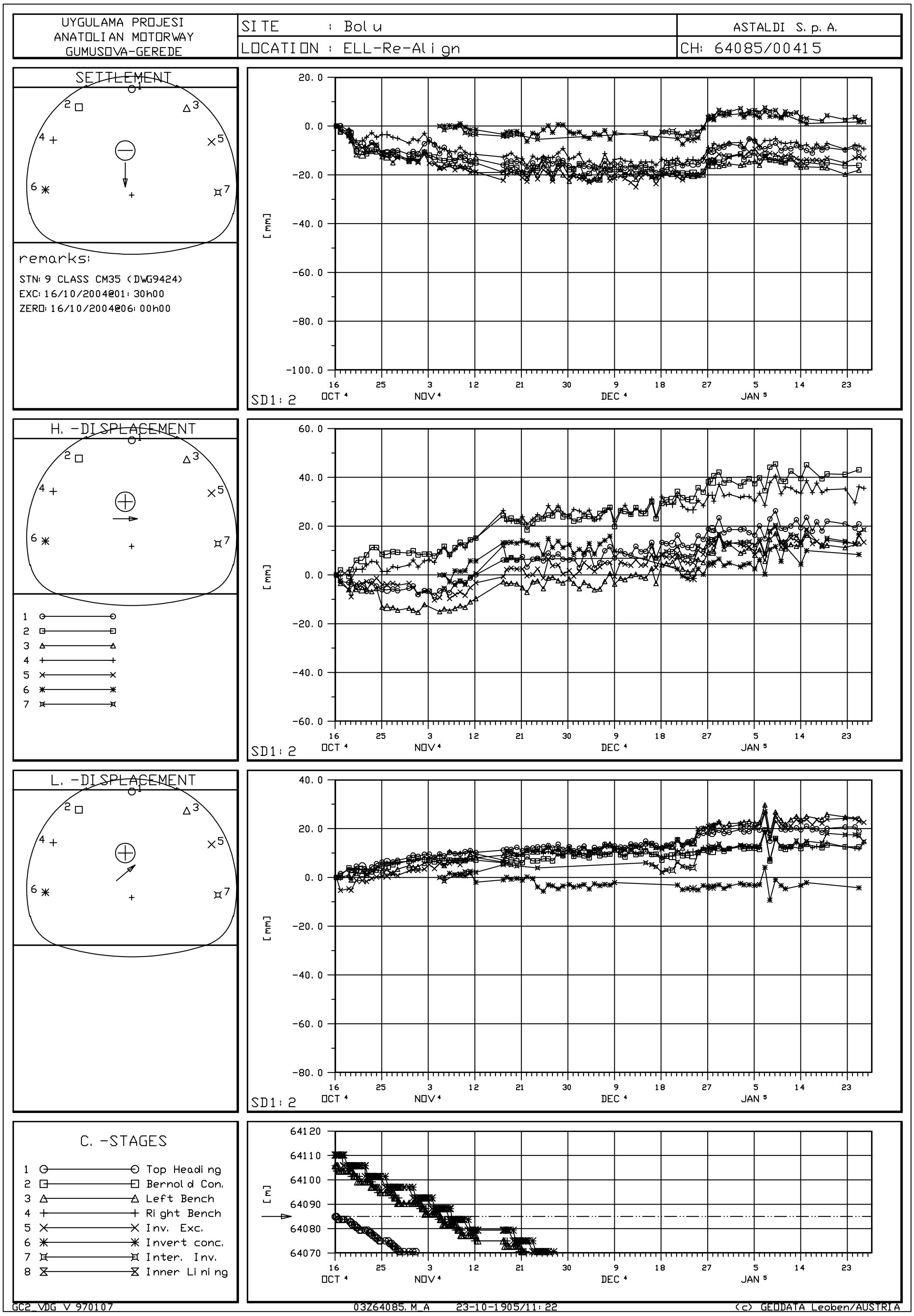
C.-STAGES

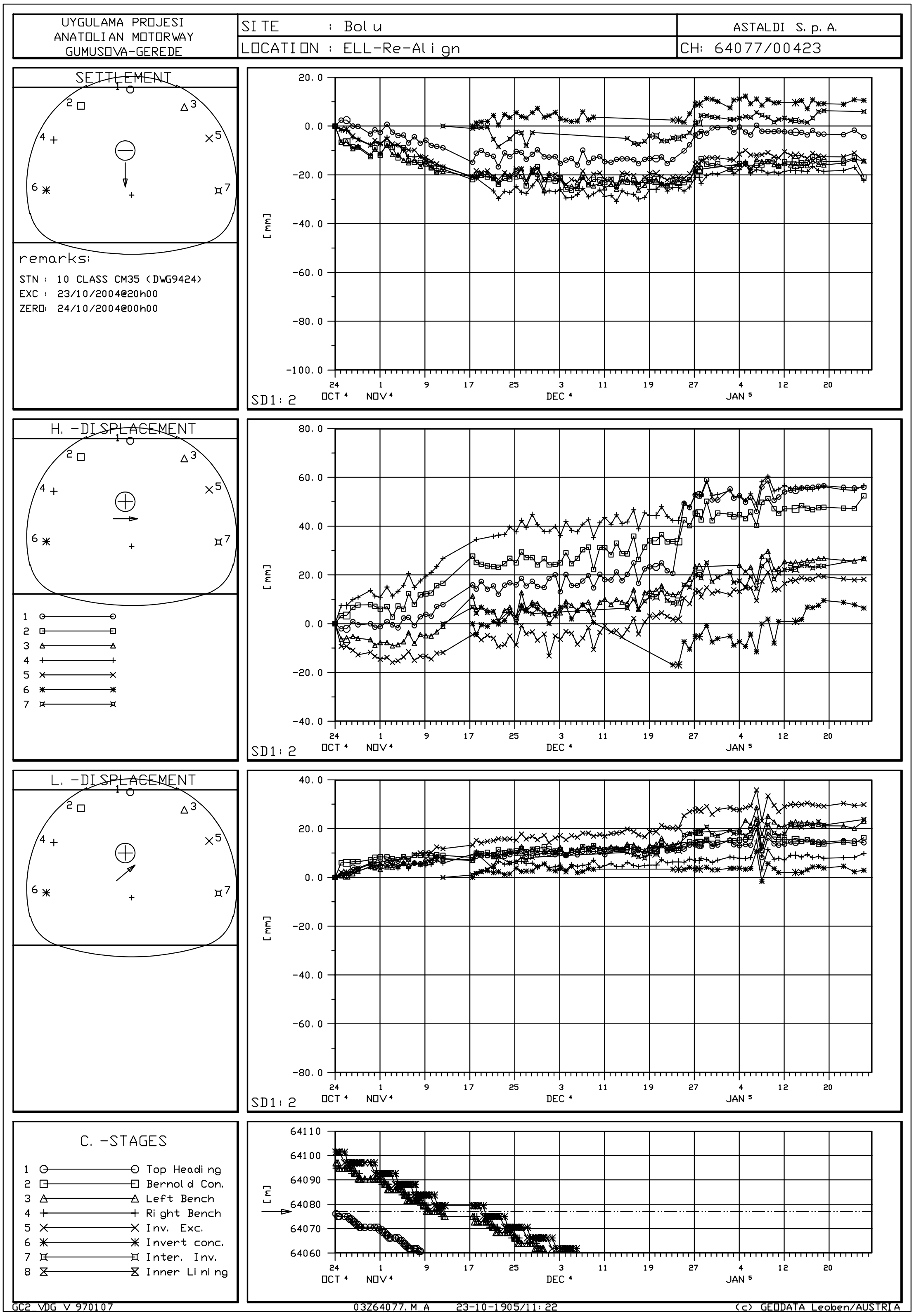
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining

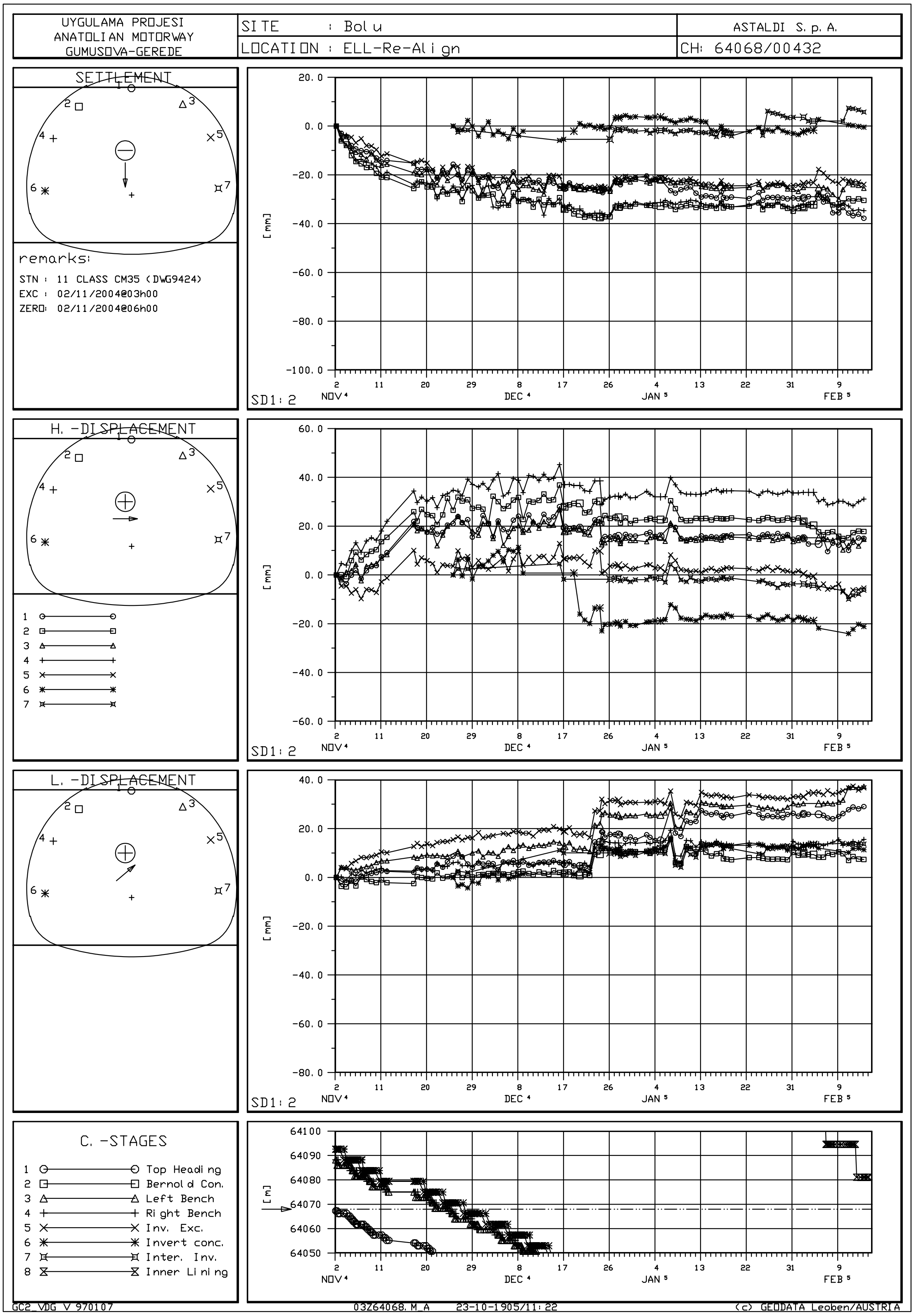


GC2_VDG V 970107 03Z64103.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 64094/00406
SETTLEMENT
remarks:
STN: 8 CLASS CM35 (DWG9424)
EXC: 08/10/2004@14h00
ZERO: 08/10/2004@17h00
H.-DISPLACEMENT
L.-DISPLACEMENT
C.-STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[mm]
SD1:2
OCT 4
NOV 4
DEC 4
JAN 5
GC2_VDG V 970107
03Z64094.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

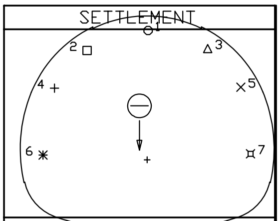

UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 64085/00415
SETTLEMENT
remarks:
STN: 9 CLASS CM35 (DWG9424)
EXC: 16/10/2004@01:30h00
ZERO: 16/10/2004@06:00h00
[mm]
SD1: 2
OCT 4
NOV 4
DEC 4
JAN 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64085.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

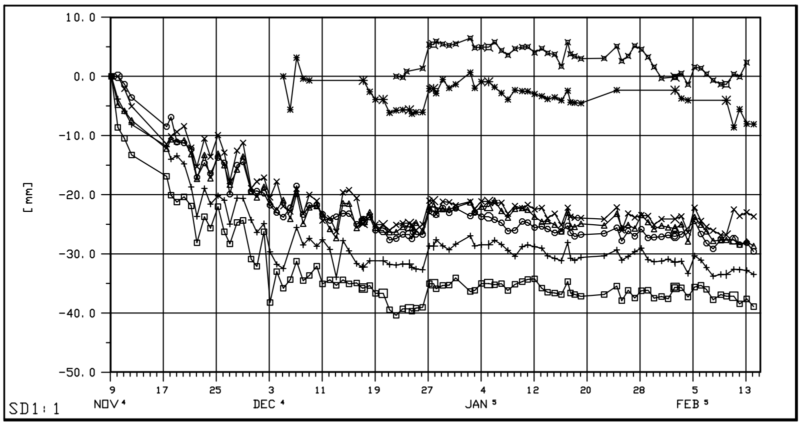

UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 64077/00423
SETTLEMENT
remarks:
STN : 10 CLASS CM35 (DWG9424)
EXC : 23/10/2004@20h00
ZERO: 24/10/2004@00h00
[mm]
SD1: 2
OCT 4 NOV 4 DEC 4 JAN 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64077. M_A 23-10-1905/11: 22
(c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 64068/00432
SETTLEMENT
remarks:
STN : 11 CLASS CM35 (DWG9424)
EXC : 02/11/2004@03h00
ZERO: 02/11/2004@06h00
[mm]
SD1: 2
NOV 4
DEC 4
JAN 5
FEB 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64068.M_A
23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S. p. A. CH: 64059/00441

SETTLEMENT

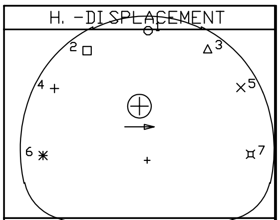


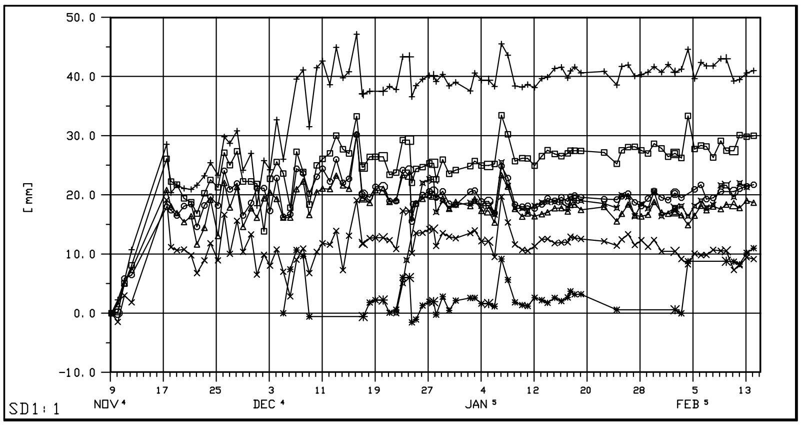


remarks:

STN : 12 CLASS CM35 (DWG9424)
EXC : 09/11/2004@00h00
ZERO: 09/11/2004@04h00

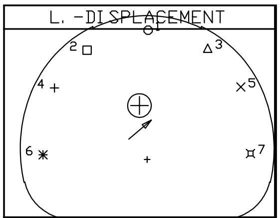
H. -DISPLACEMENT

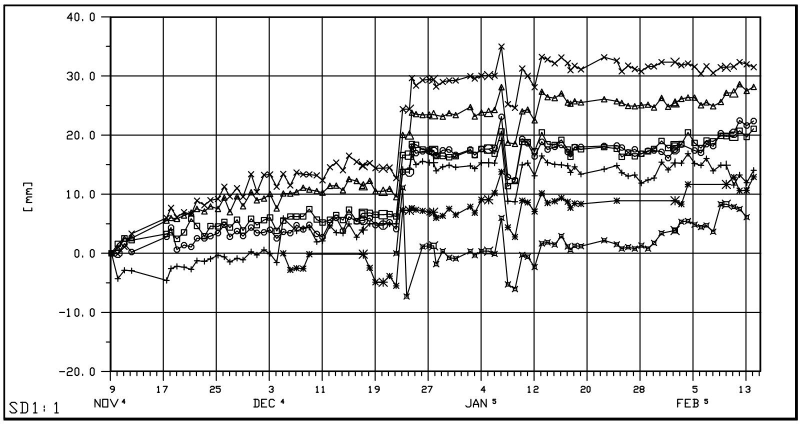




1 ○
2 ⊡
3 △
4 +
5 ×
6 ✳
7 ⌘

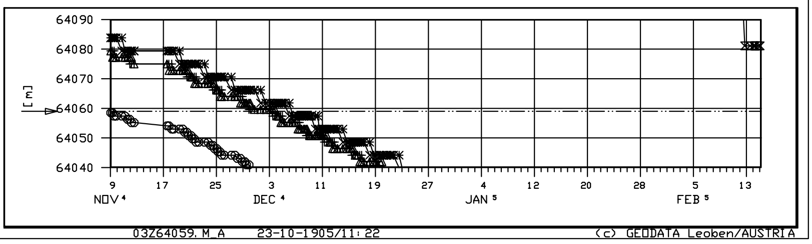
L. -DISPLACEMENT



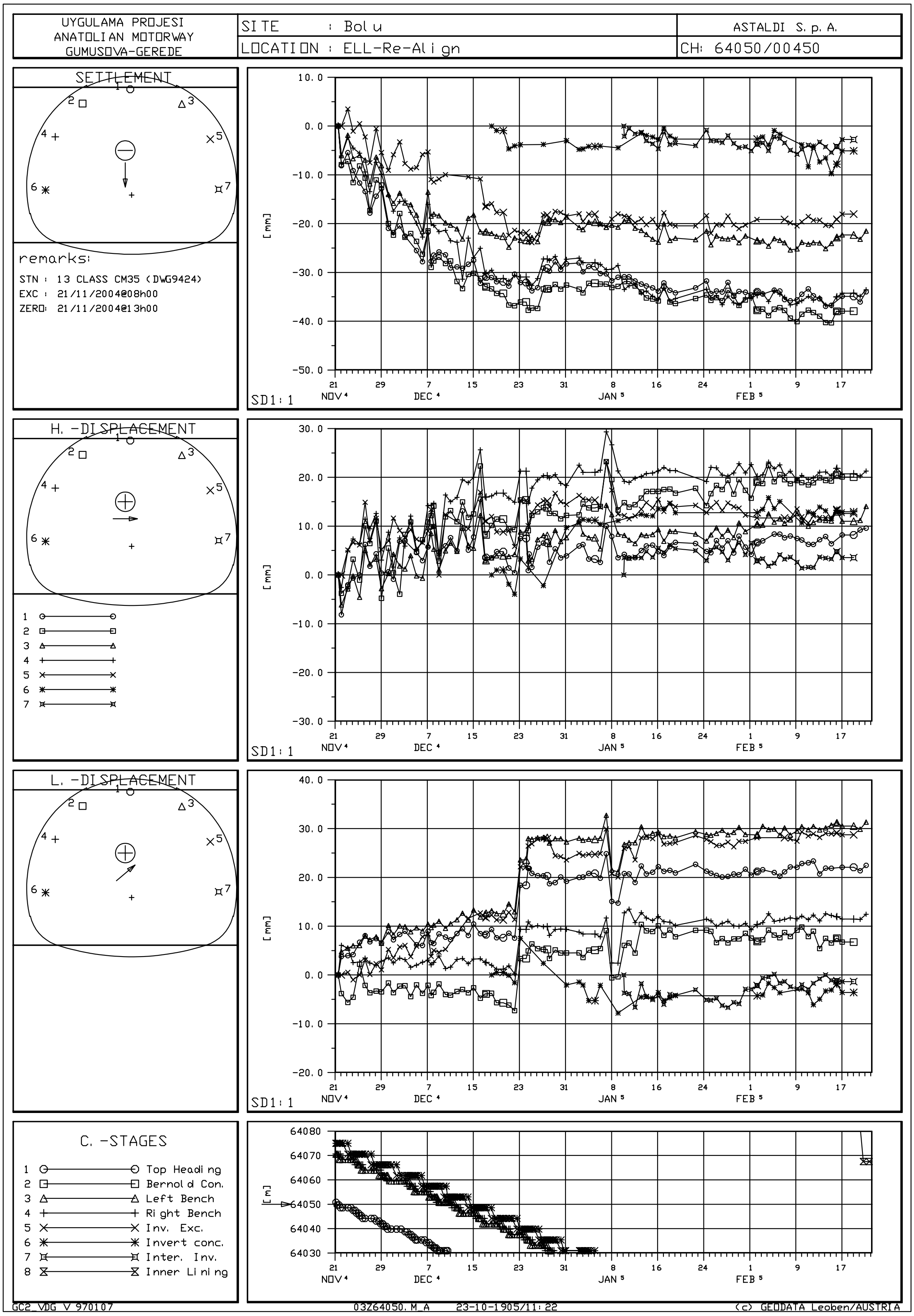


C. -STAGES

1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining

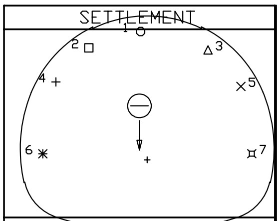


GC2_VDG V 970107 03Z64059.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 64050/00450
SETTLEMENT
remarks:
STN : 13 CLASS CM35 (DWG9424)
EXC : 21/11/2004@08h00
ZERO: 21/11/2004@13h00
[mm]
SD1:1
NOV 4
DEC 4
JAN 5
FEB 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64050.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S.p.A. CH: 64042/00458

SETTLEMENT



remarks:

STN : 14 CLASS CM35(DWG9424)
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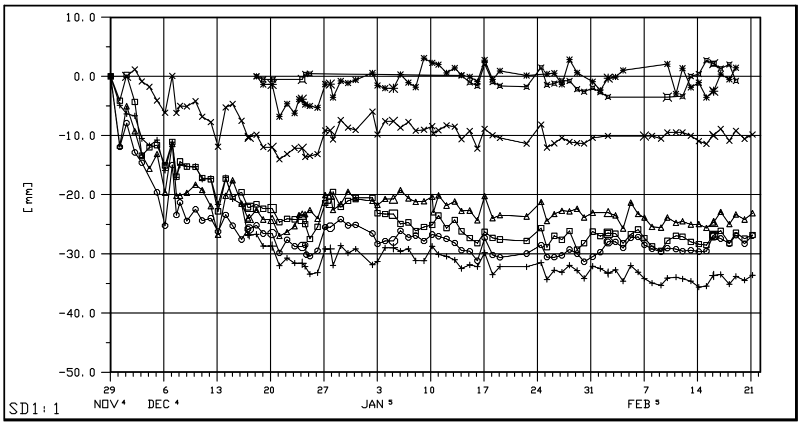
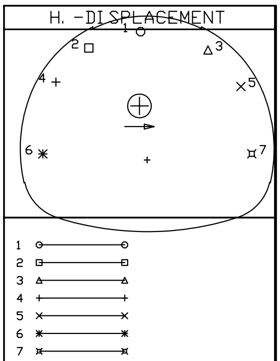
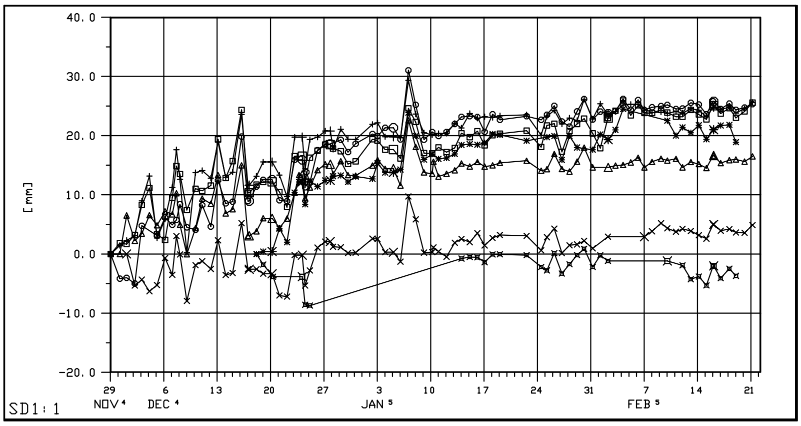
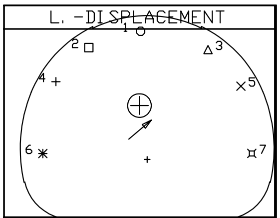


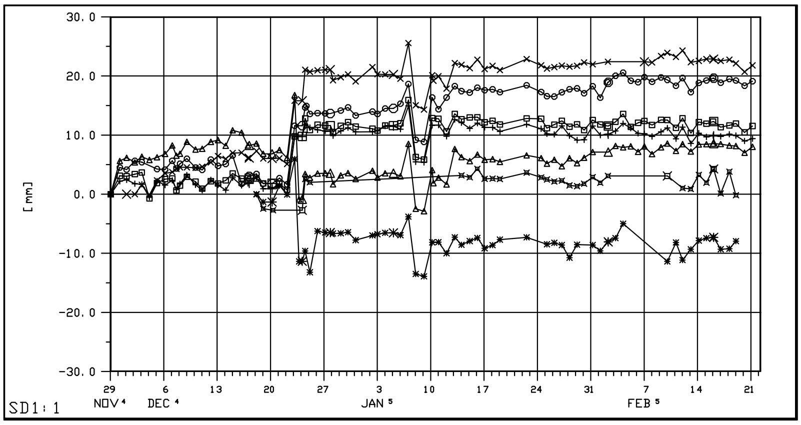
H.-DISPLACEMENT





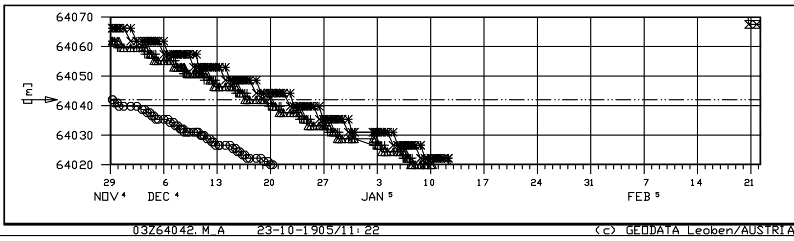
L.-DISPLACEMENT



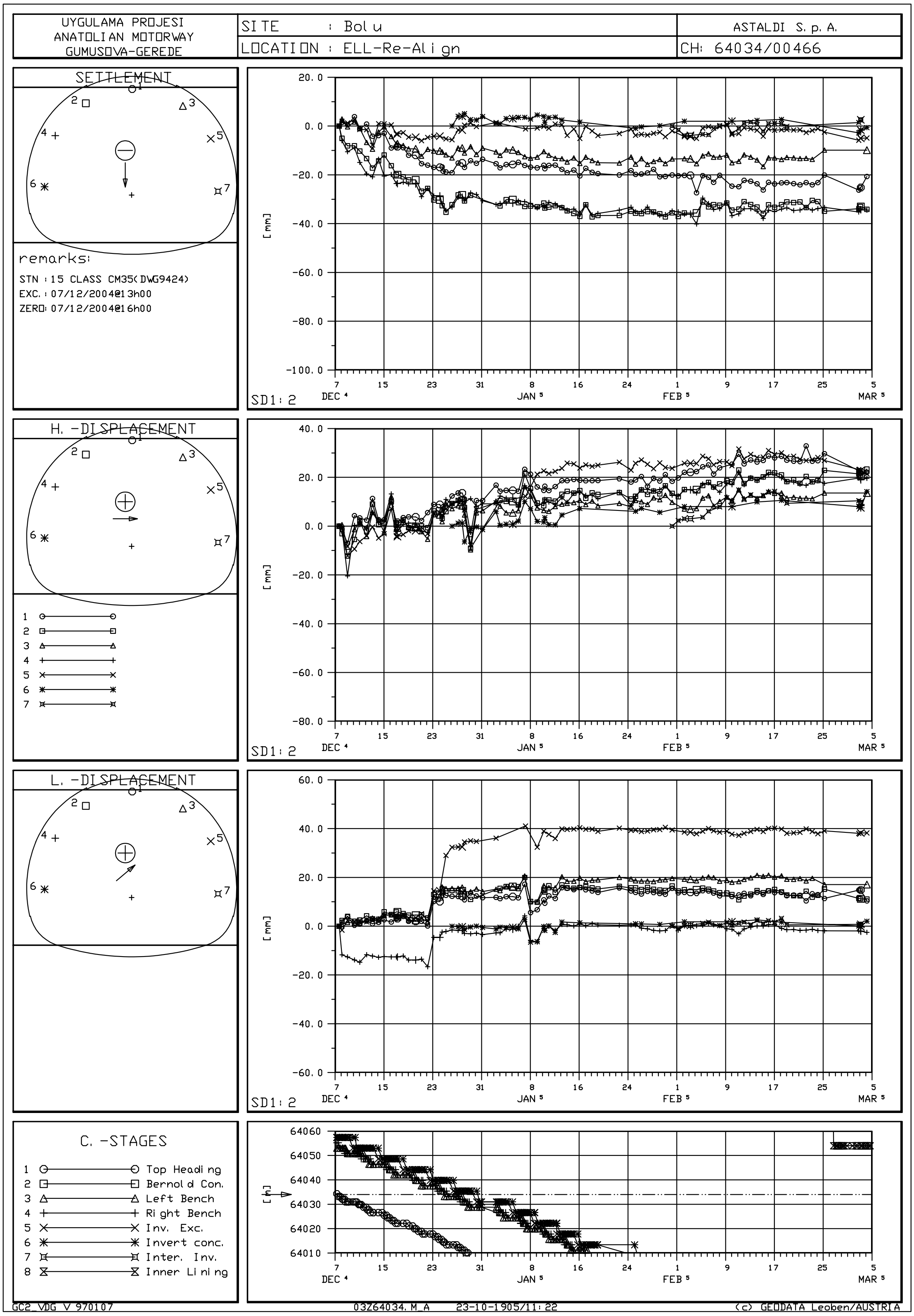


C.-STAGES

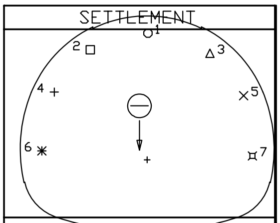
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining

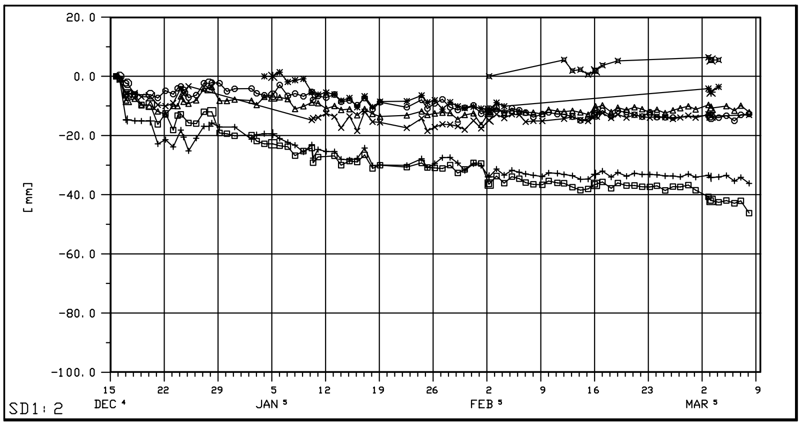


GC2_VDG V 970107 03Z64042.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 64034/00466
SETTLEMENT
remarks:
STN : 15 CLASS CM35(DWG9424)
EXC. : 07/12/2004@13h00
ZERO: 07/12/2004@16h00
H.-DISPLACEMENT
L.-DISPLACEMENT
C.-STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[mm]
[m]
SD1:2
DEC 4
JAN 5
FEB 5
MAR 5
GC2_VDG V 970107
03Z64034.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

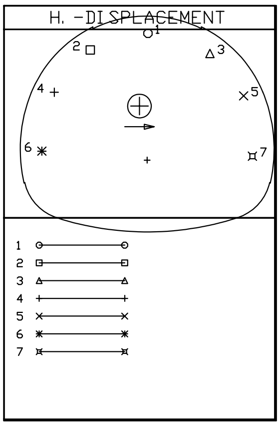
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S. p. A. CH: 64025/00475

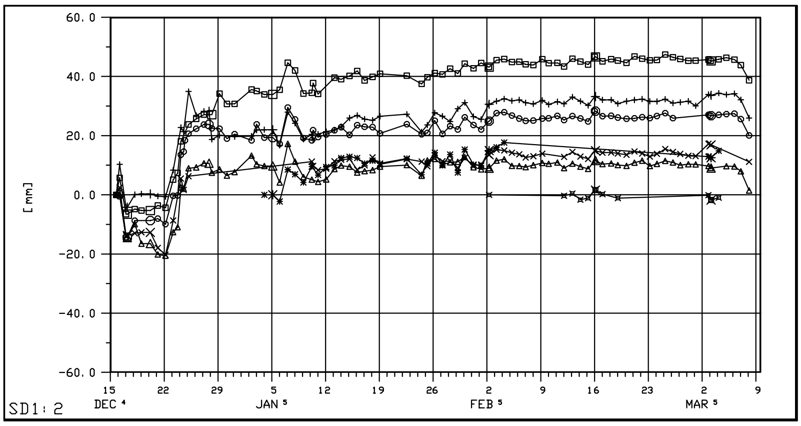


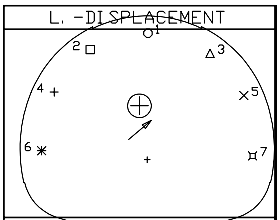


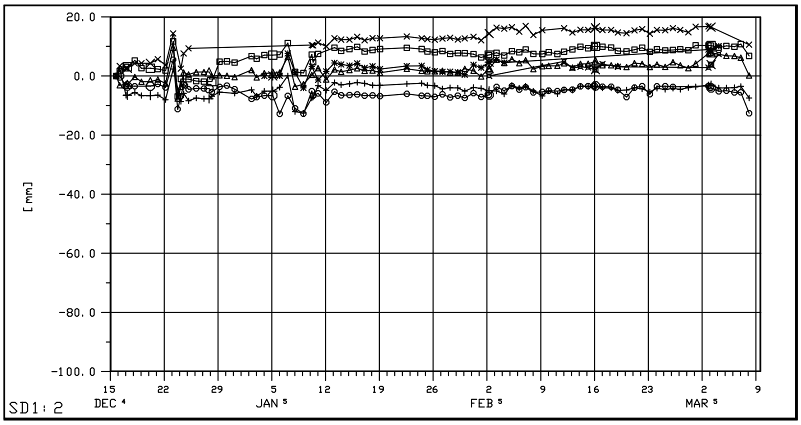
remarks:

STN: 16 CLASS CM 35 (DWG9424)
EXC: 15/12/2004@16h00
ZERO: 15/12/2004@18h00



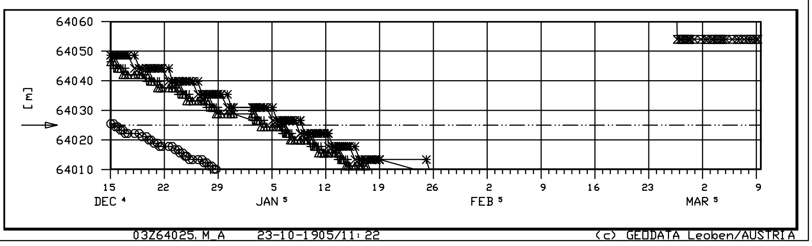




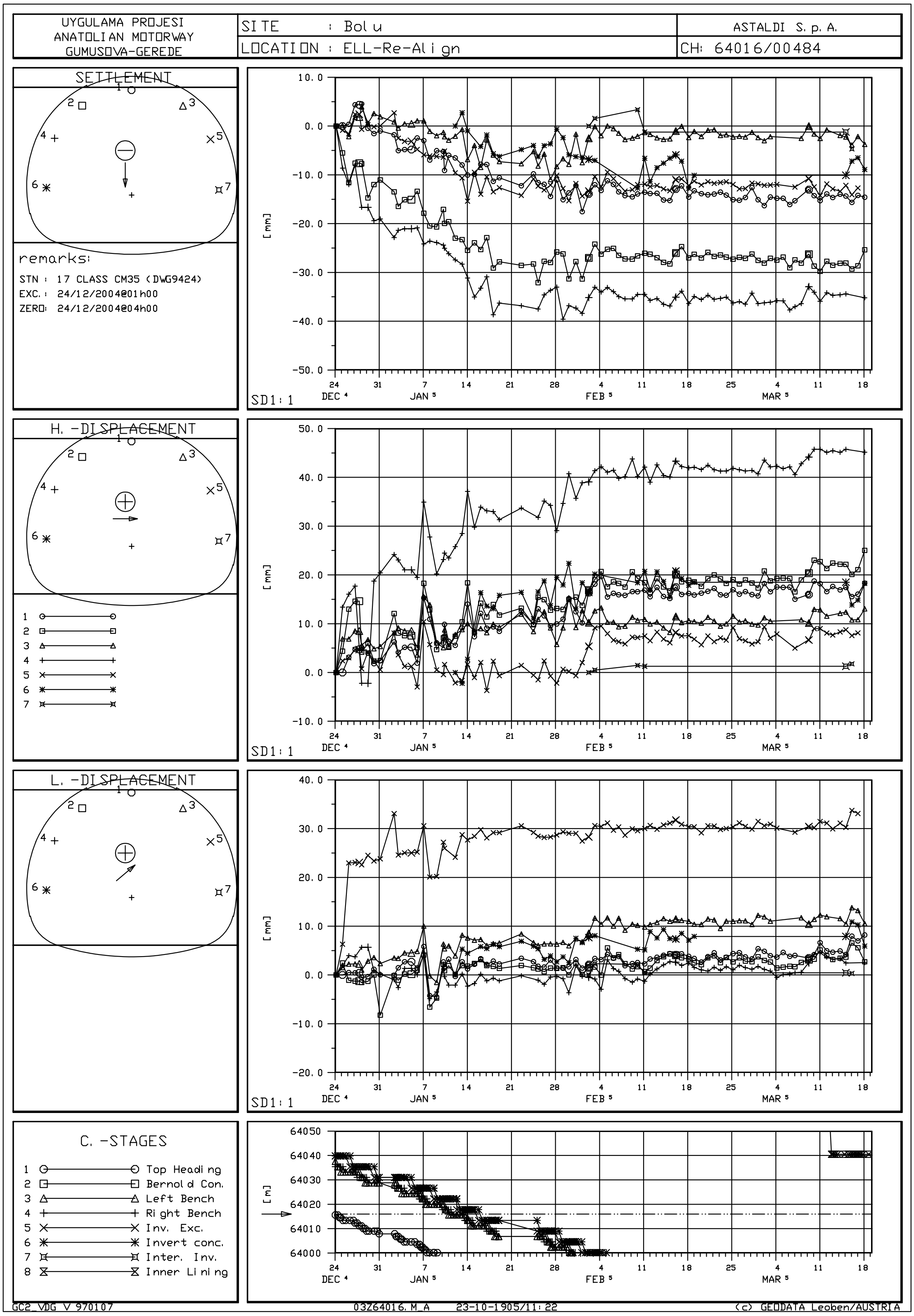


C. -STAGES

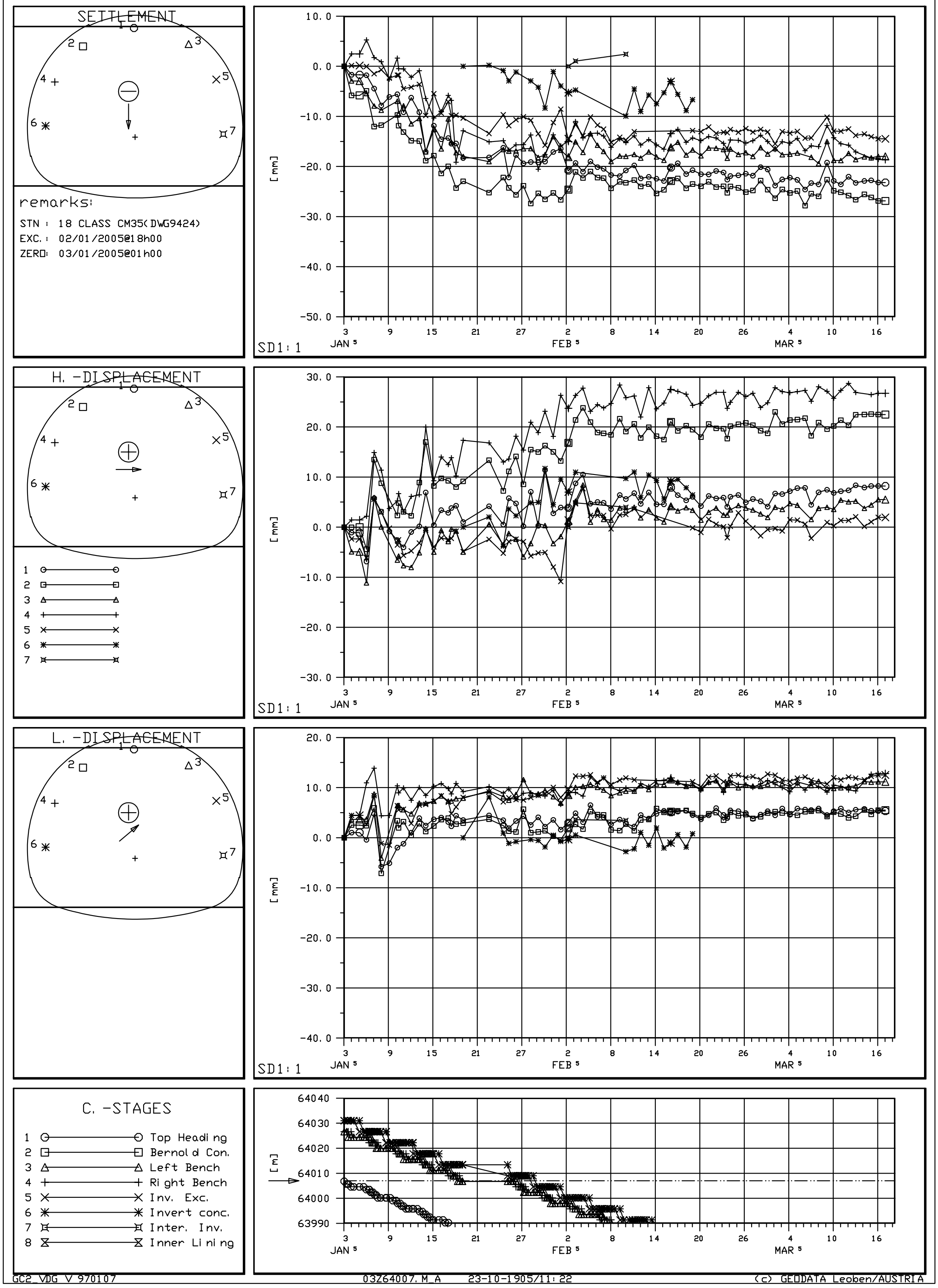
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining



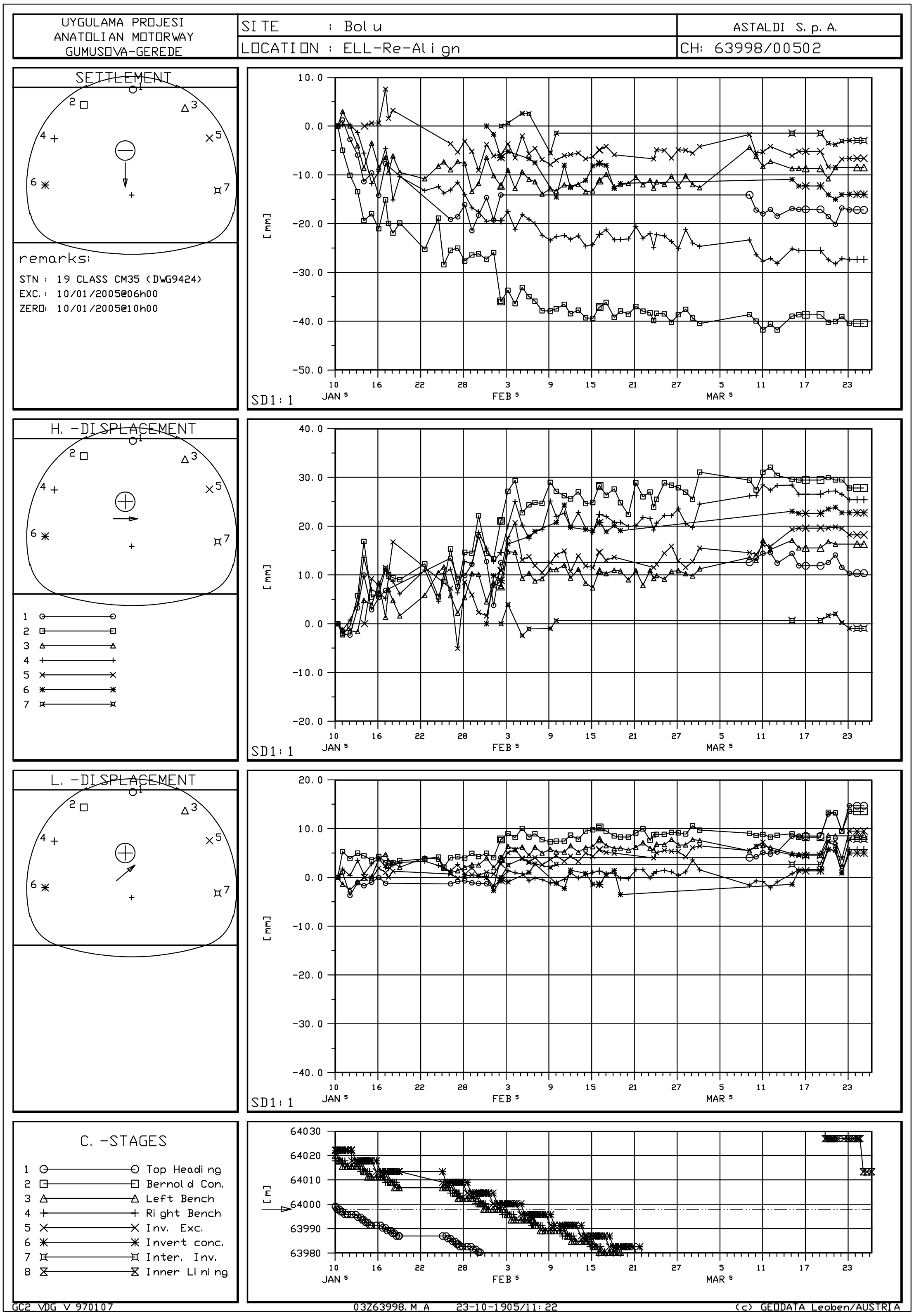
GC2_VDG V 970107 03Z64025.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 64016/00484
SETTLEMENT
remarks:
STN : 17 CLASS CM35 (DWG9424)
EXC. : 24/12/2004@01h00
ZERO: 24/12/2004@04h00
[mm]
SD1:1
DEC 4
JAN 5
FEB 5
MAR 5
H.-DISPLACEMENT
L.-DISPLACEMENT
C.-STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z64016.M_A
23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

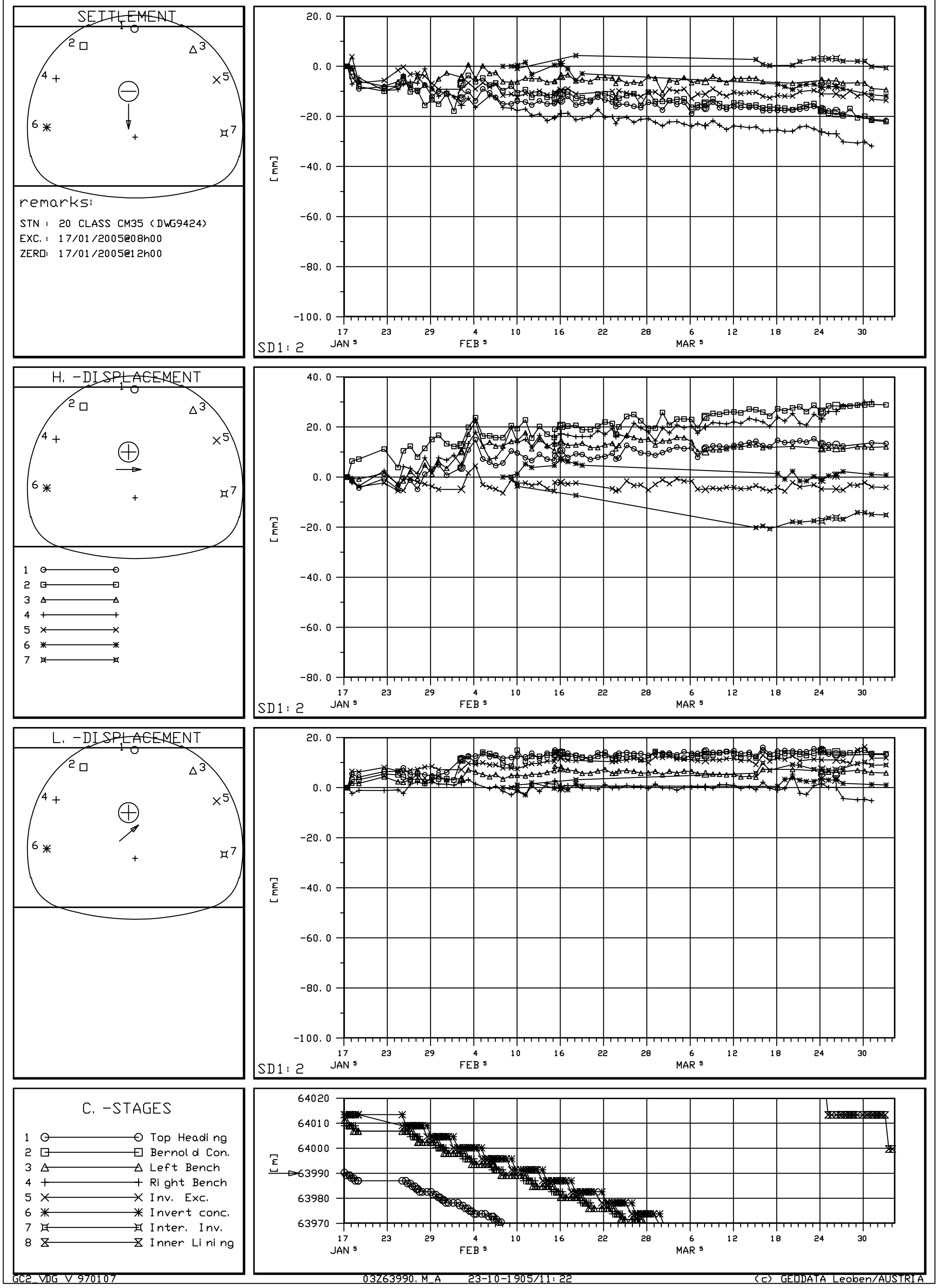
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S. p. A. CH: 64007/00493



GC2_VDG V 970107 03Z64007.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 63998/00502
SETTLEMENT
remarks:
STN : 19 CLASS CM35 (DWG9424)
EXC. : 10/01/2005@06h00
ZERO: 10/01/2005@10h00
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[mm]
[m]
SD1:1
JAN 5
FEB 5
MAR 5
GC2_VDG V 970107
03Z63998.M_A
23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

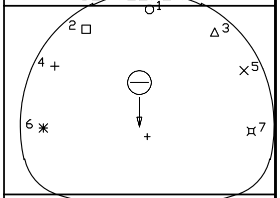
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 63990/00510



GC2_VDG V 970107 03Z63990.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA

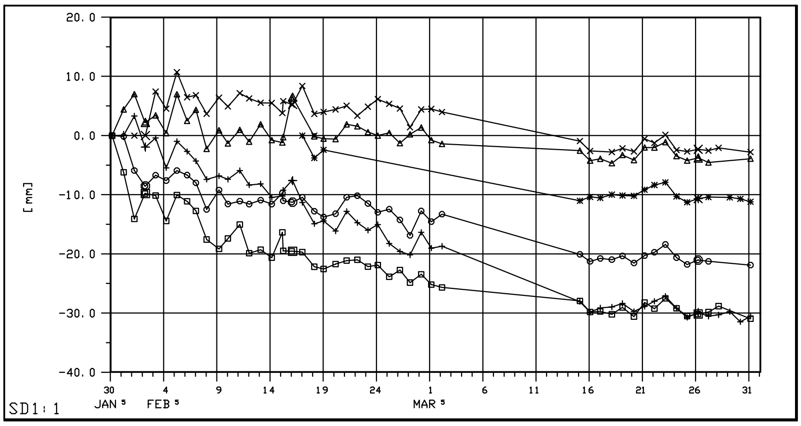
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELL-Re-Align	CH: 63981/00519

SETTLEMENT

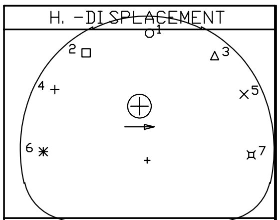


remarks:

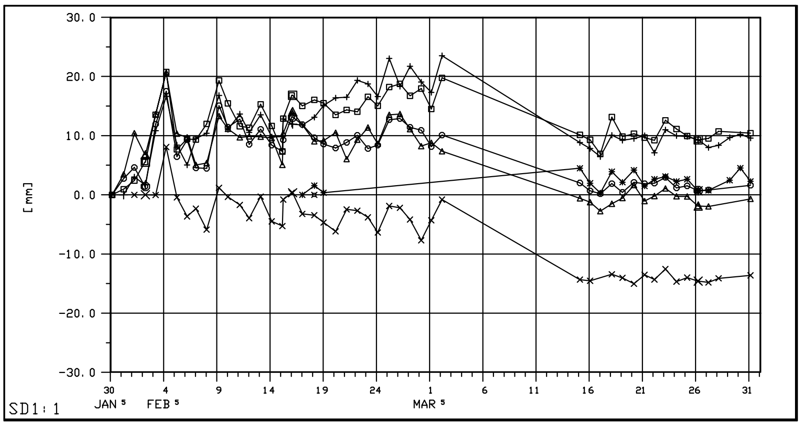
STN : 21 CLASS CM35 (DWG9424)
EXC. : 30/01/2005@01h30
ZERO: 30/01/2005@04h00



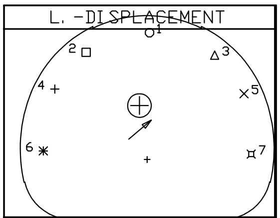
H. -DISPLACEMENT

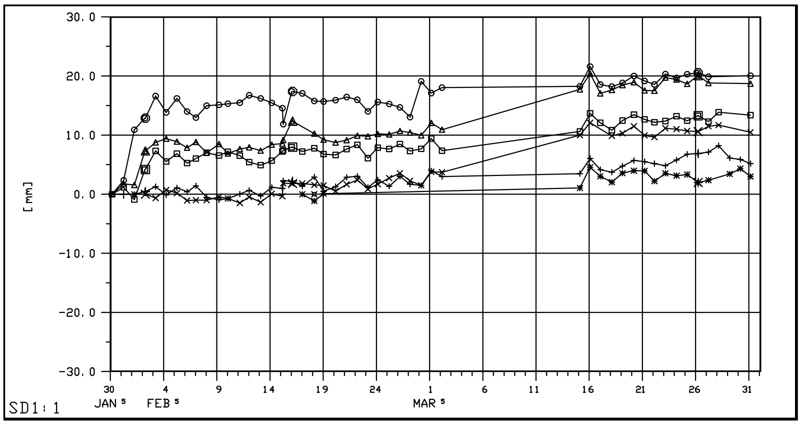


1 —○—
2 —□—
3 —△—
4 —+—
5 —×—
6 —✳—
7 —⌑—



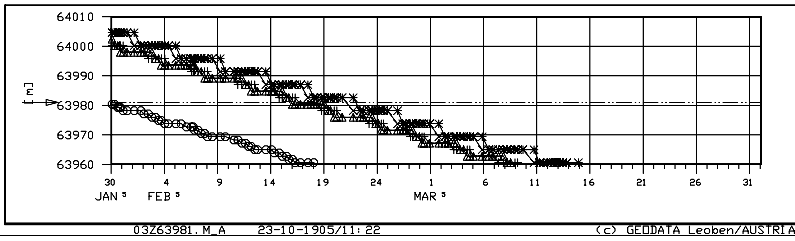
L. -DISPLACEMENT



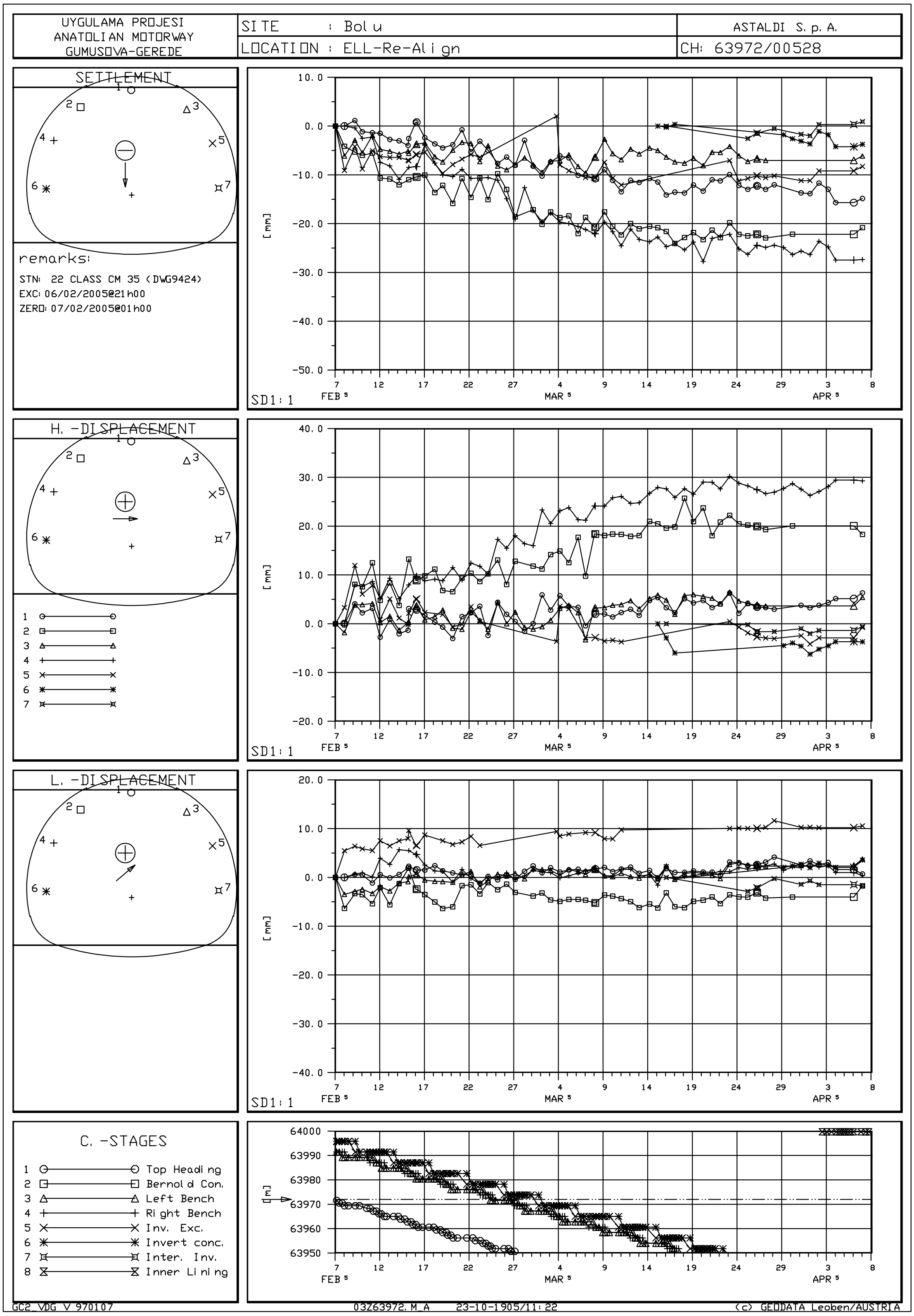


C. -STAGES

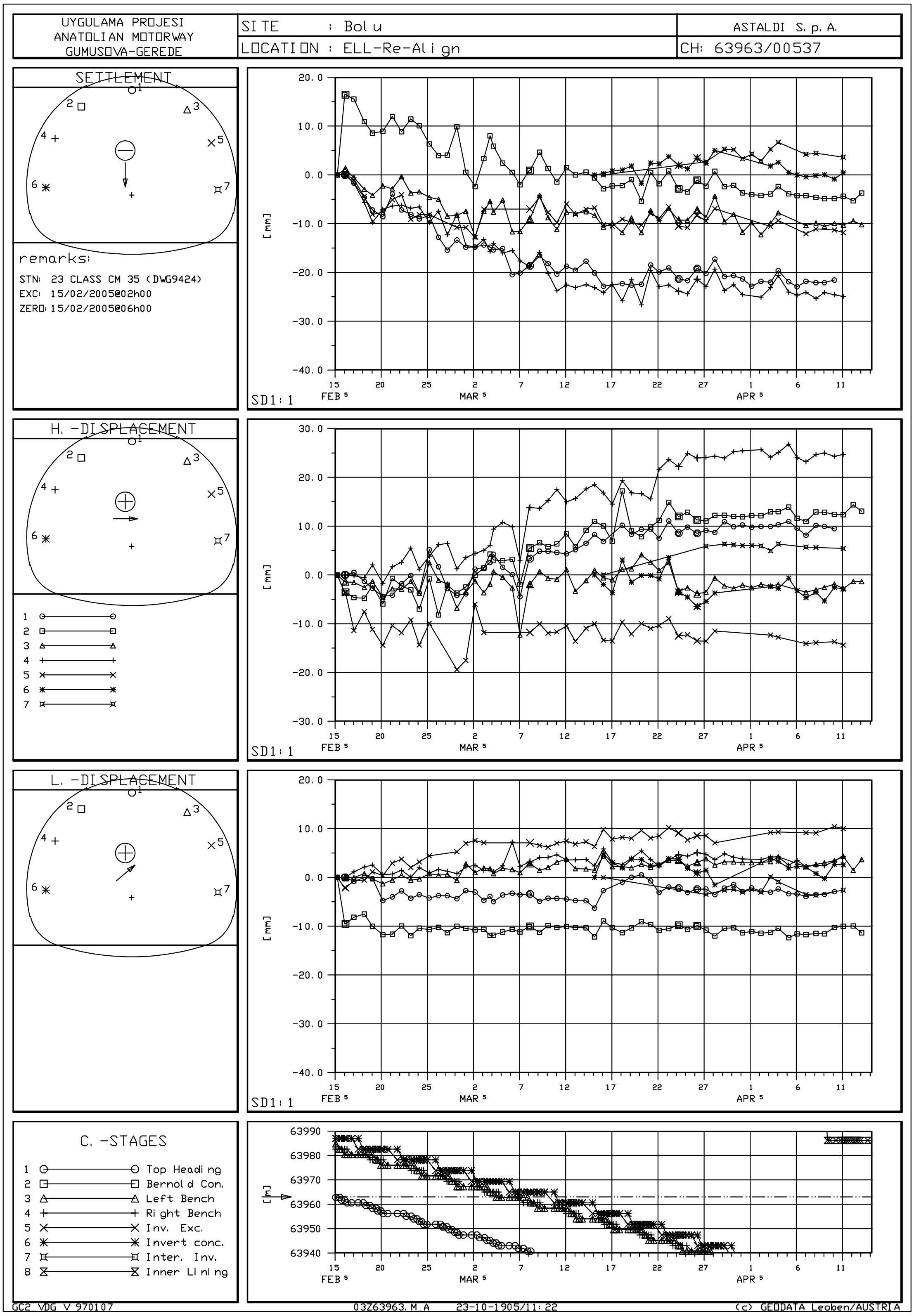
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining

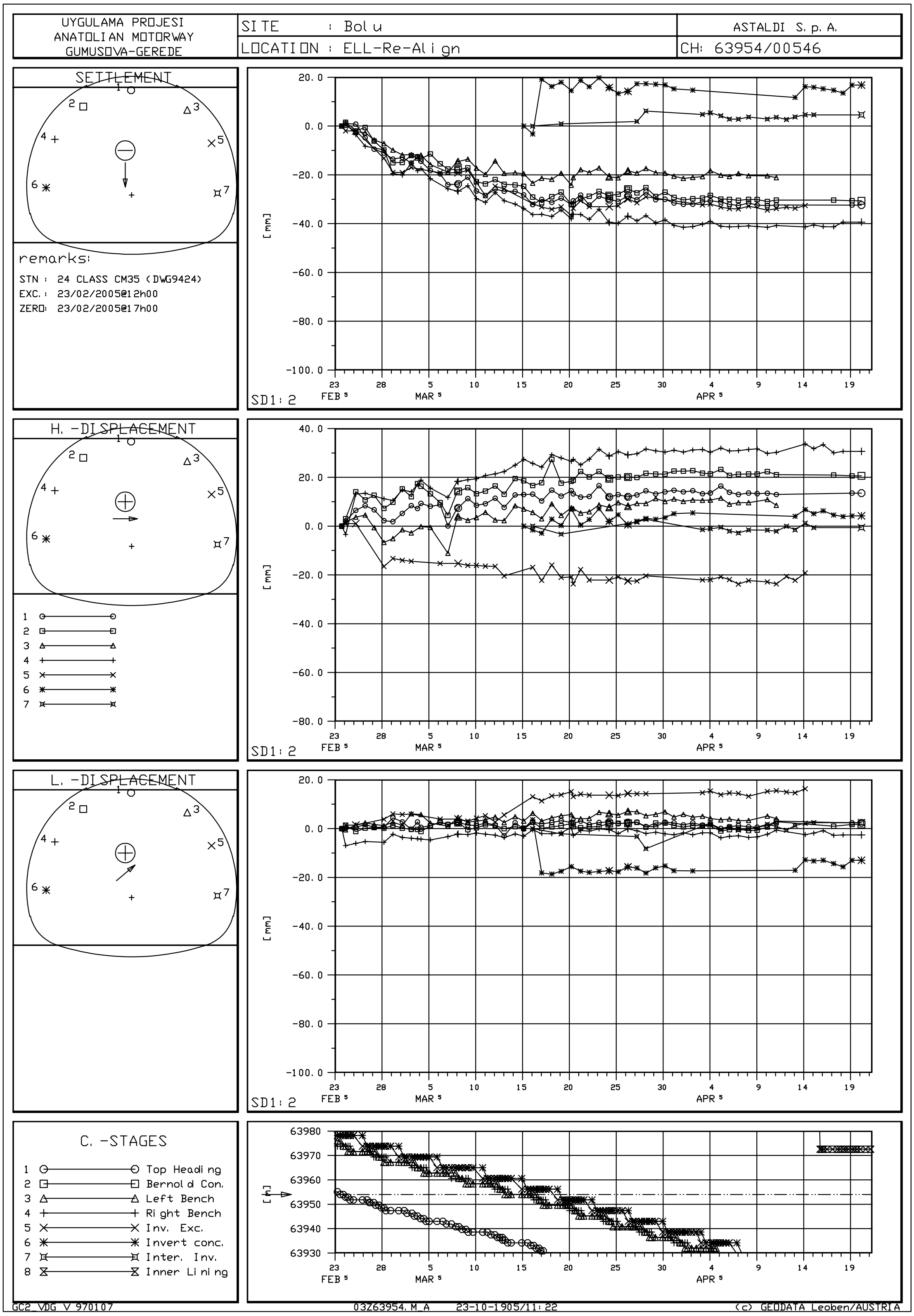


GC2_VDG V 970107 03Z63981.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA



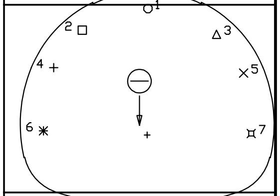
UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 63972/00528
SETTLEMENT
remarks:
STN: 22 CLASS CM 35 (DWG9424)
EXC: 06/02/2005@21h00
ZERO: 07/02/2005@01h00
[mm]
SD1: 1
FEB 5
MAR 5
APR 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z63972. M_A 23-10-1905/11: 22
(c) GEODATA Leoben/AUSTRIA

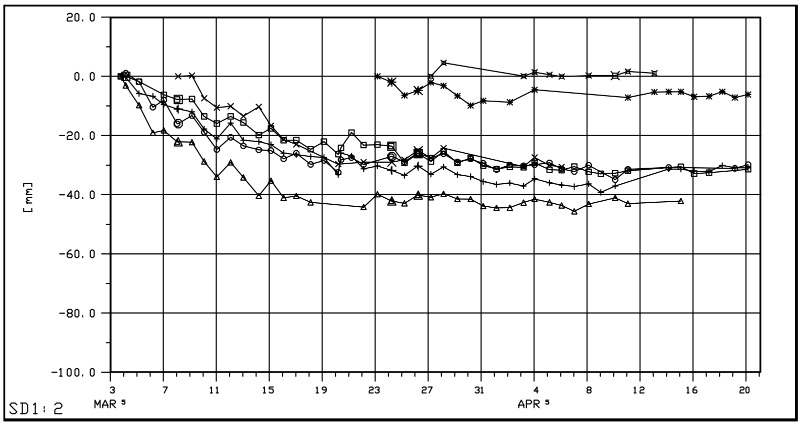

UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 63963/00537
SETTLEMENT
remarks:
STN: 23 CLASS CM 35 (DWG9424)
EXC: 15/02/2005@02h00
ZERO: 15/02/2005@06h00
SD1: 1
[mm]
FEB 5
MAR 5
APR 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z63963. M_A 23-10-1905/11: 22
(c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 63954/00546
SETTLEMENT
remarks:
STN : 24 CLASS CM35 (DWG9424)
EXC. : 23/02/2005@12h00
ZERO: 23/02/2005@17h00
[mm]
SD1: 2
FEB 5
MAR 5
APR 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z63954.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S.p.A. CH: 63946/00554

SETTLEMENT

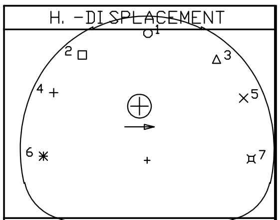


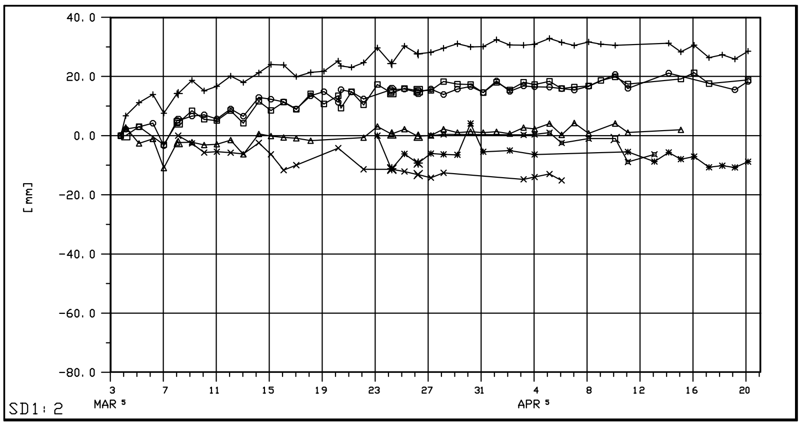


remarks:

STN : 25 CM CLASS35 (DWG9424)
EXC. : 03/03/2005@11h00
ZERO: 03/03/2005@19h00

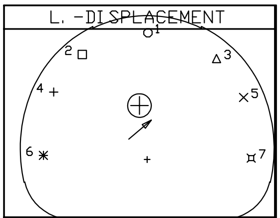
H.-DISPLACEMENT

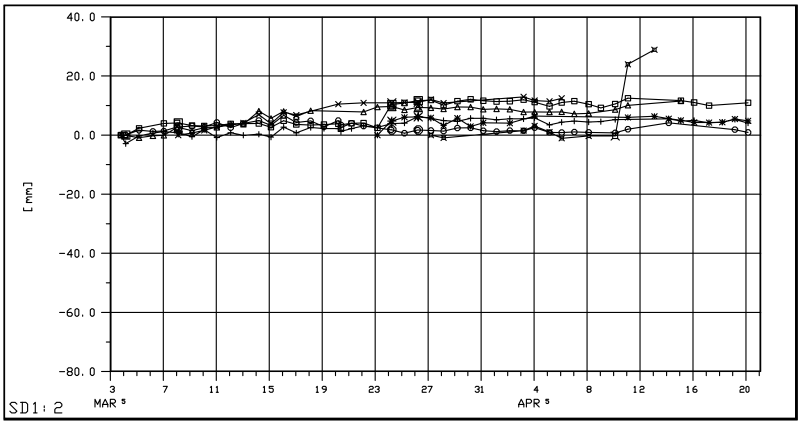




1
2
3
4
5
6
7

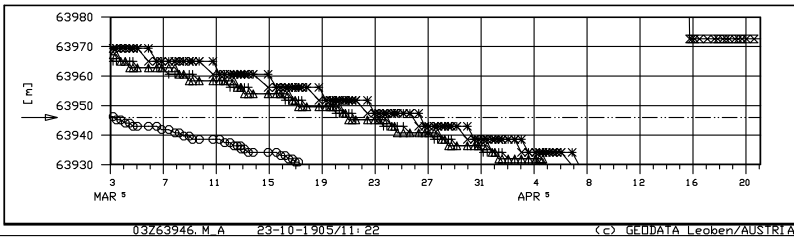
L.-DISPLACEMENT



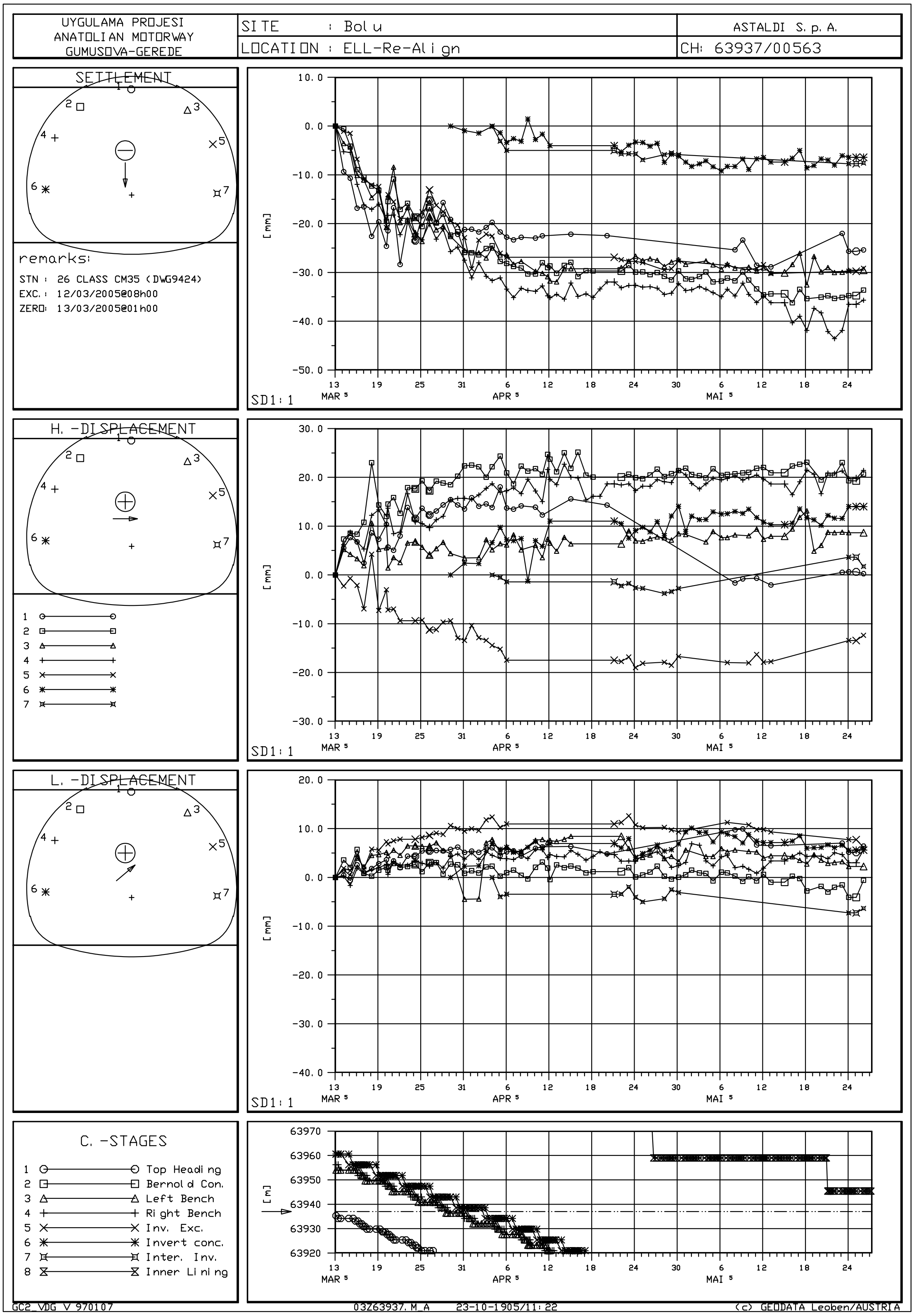


C.-STAGES

1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining

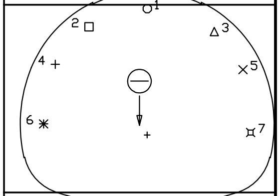


GC2_VDG V 970107 03Z63946.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S. p. A.
CH: 63937/00563
SETTLEMENT
remarks:
STN : 26 CLASS CM35 (DWG9424)
EXC. : 12/03/2005@08h00
ZERO: 13/03/2005@01h00
[mm]
SD1:1
MAR 5
APR 5
MAI 5
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z63937.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

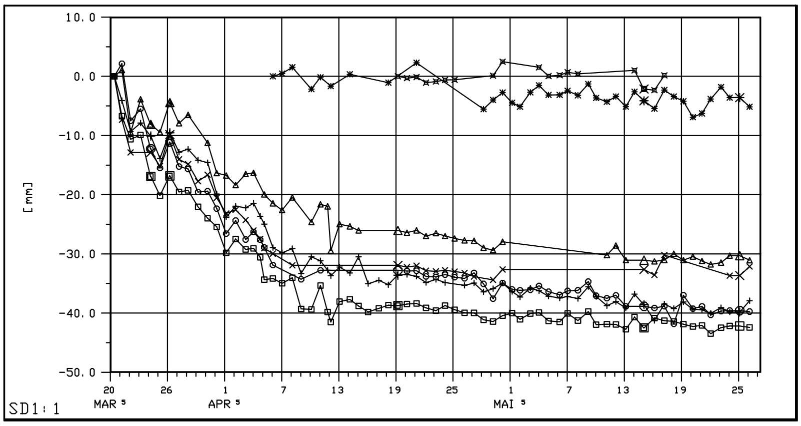
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 63928/00572

SETTLEMENT

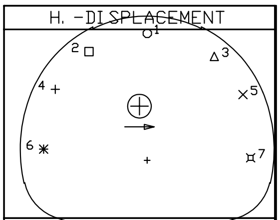


remarks:

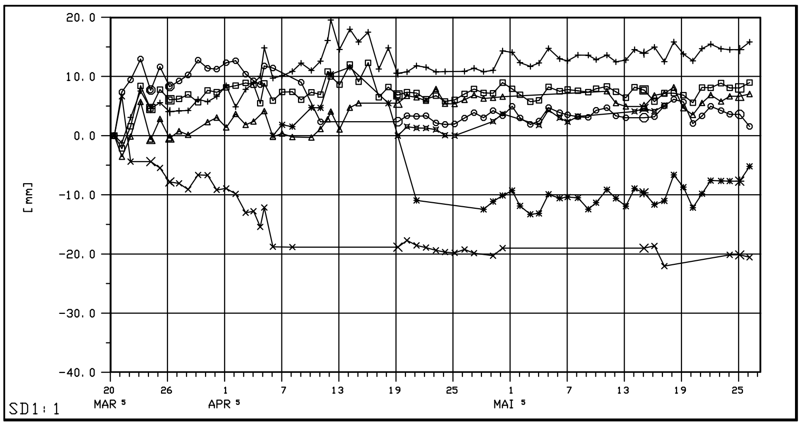
STN : 27 CLASS CM35 (DWG9424)
EXC. : 20/03/2005@07h00
ZERO: 20/03/2005@10h00



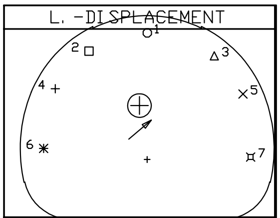
H. -DISPLACEMENT

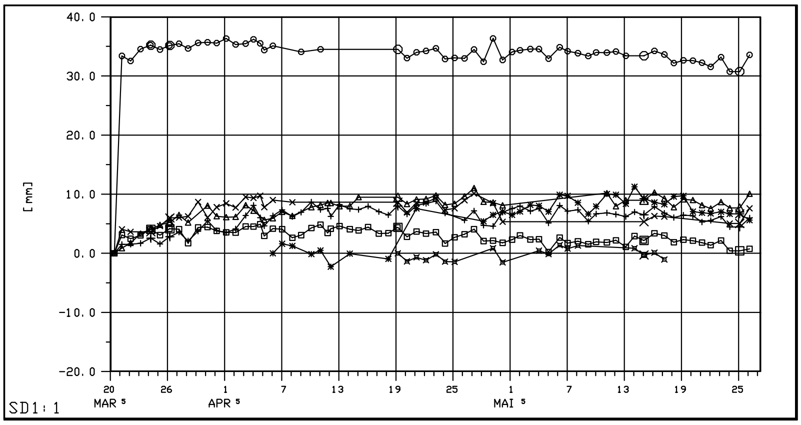


1 ○———○
2 □———□
3 △———△
4 +———+
5 ×———×
6 ✳———✳
7 ⌑———⌑



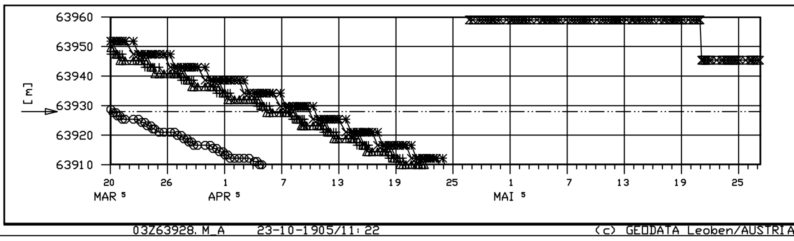
L. -DISPLACEMENT





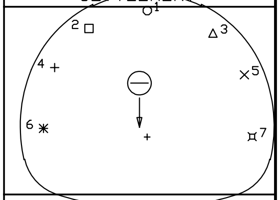
C. -STAGES

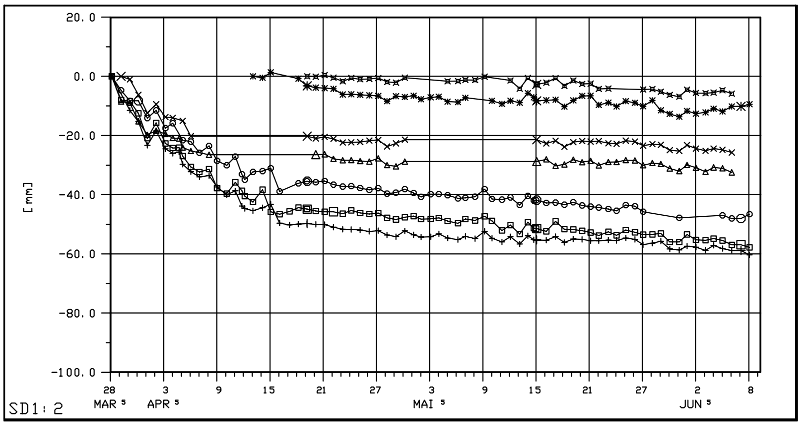
1 ○———○ Top Heading
2 □———□ Bernold Con.
3 △———△ Left Bench
4 +———+ Right Bench
5 ×———× Inv. Exc.
6 ✳———✳ Invert conc.
7 ⌑———⌑ Inter. Inv.
8 ⧖———⧖ Inner Lining



UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 63919/00581

SETTLEMENT

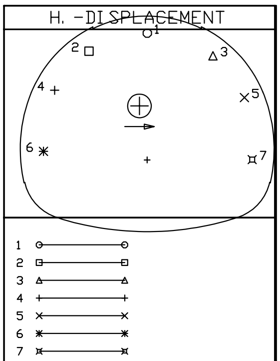


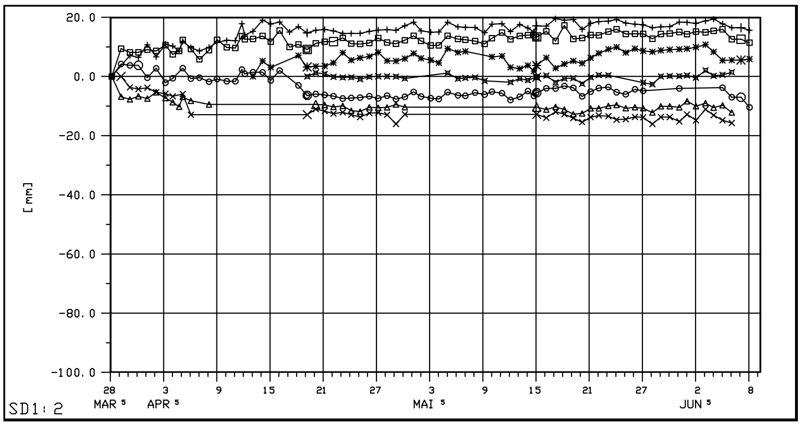


remarks:

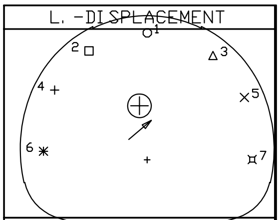
STN : 28 CLASS CM35 (DWG9424)
EXC. : 27/03/2005@18h00
ZERO: 28/03/2005@04h00

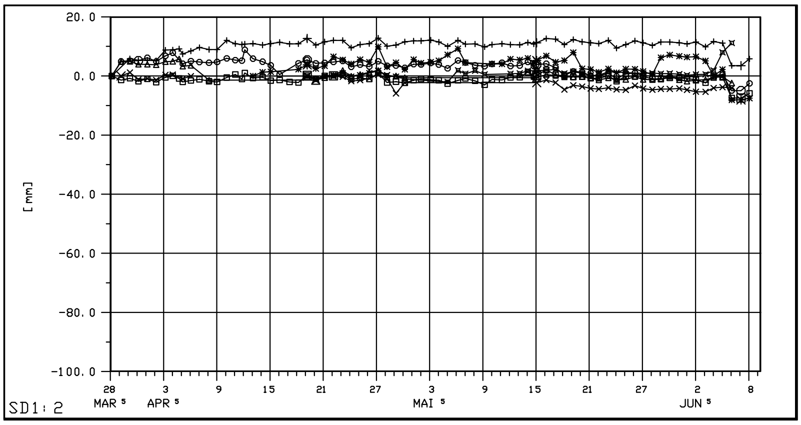
H.-DISPLACEMENT





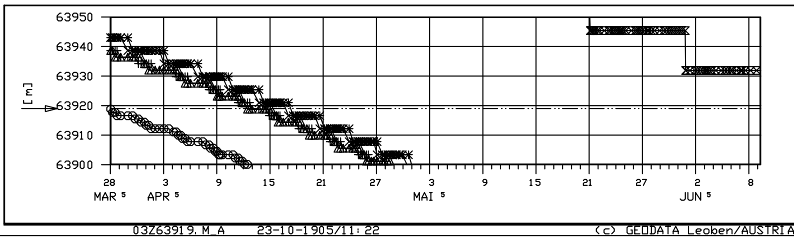
L.-DISPLACEMENT





C.-STAGES

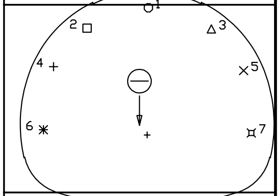
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining



GC2_VDG V 970107 03Z63919.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA

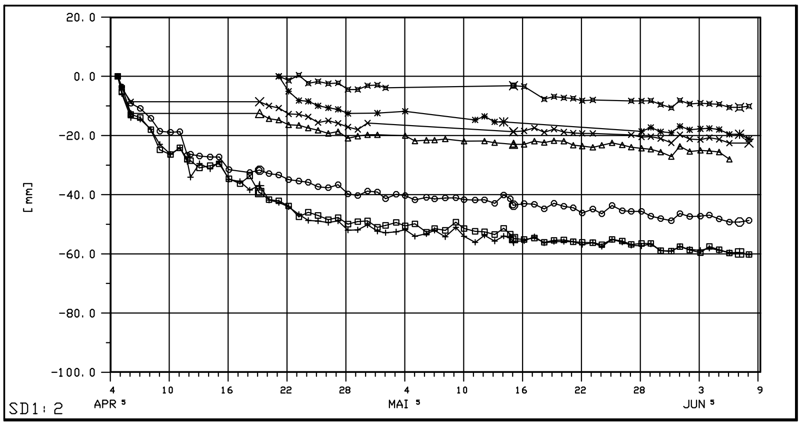
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S. p. A. CH: 63911/00589

SETTLEMENT

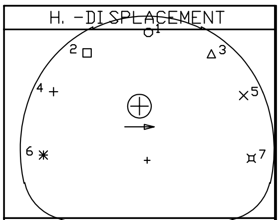


remarks:

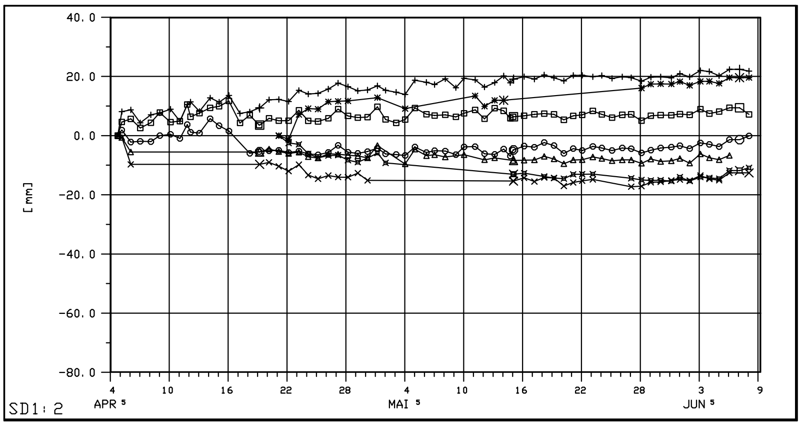
STN : 29 CLASS CM 35 (DWG9424)
EXC. : 04/04/2005@15h00
ZERO: 04/04/2005@19h00



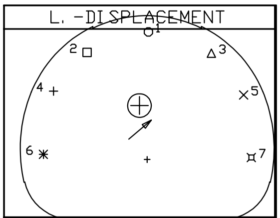
H. -DISPLACEMENT

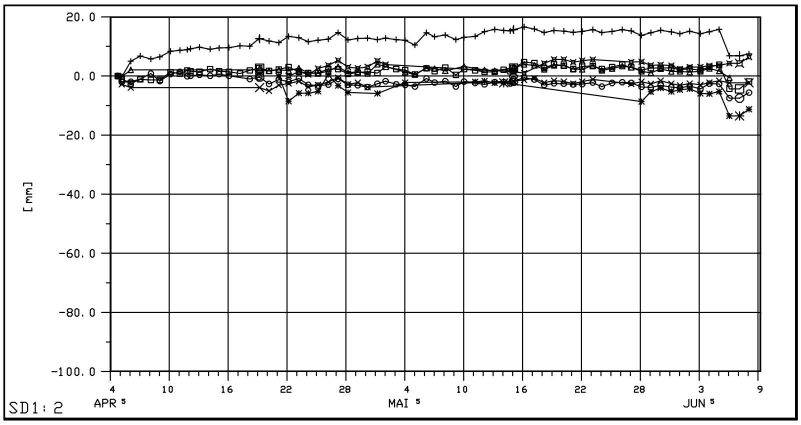


1 ○
2 □
3 △
4 +
5 ×
6 ✳
7 ⌑



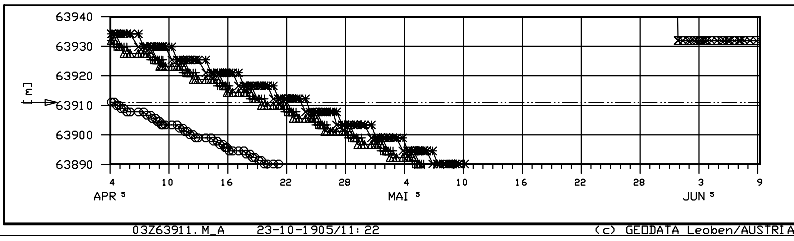
L. -DISPLACEMENT





C. -STAGES

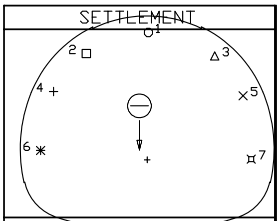
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining



GC2_VDG V 970107 03Z63911.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA

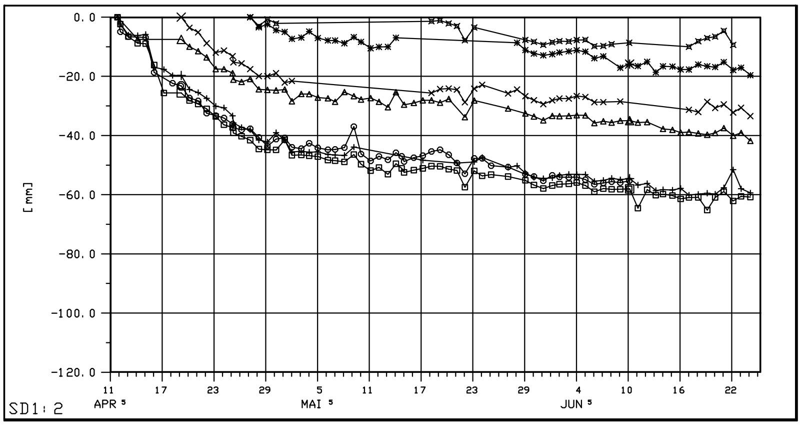
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S. p. A. CH: 63902/00598

SETTLEMENT

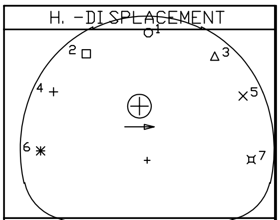


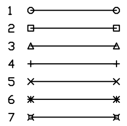
remarks:

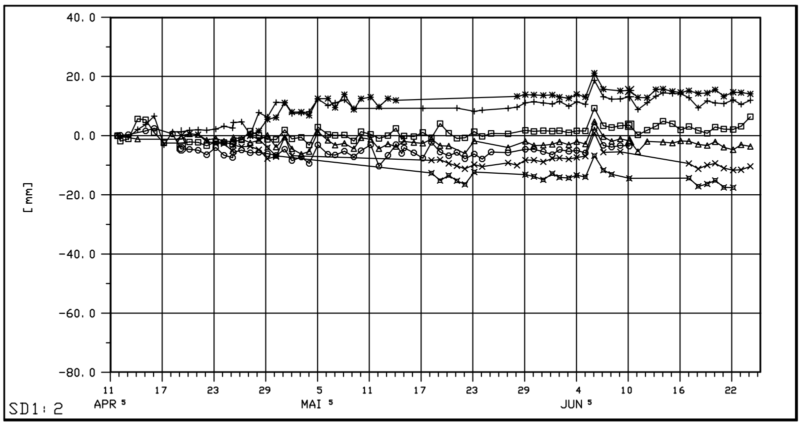
STN : 30 CLASS CM45 (DWG300)
EXC. : 11/04/2005@16h00
ZERO: 11/04/2005@20h00



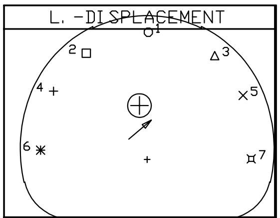
H. -DISPLACEMENT

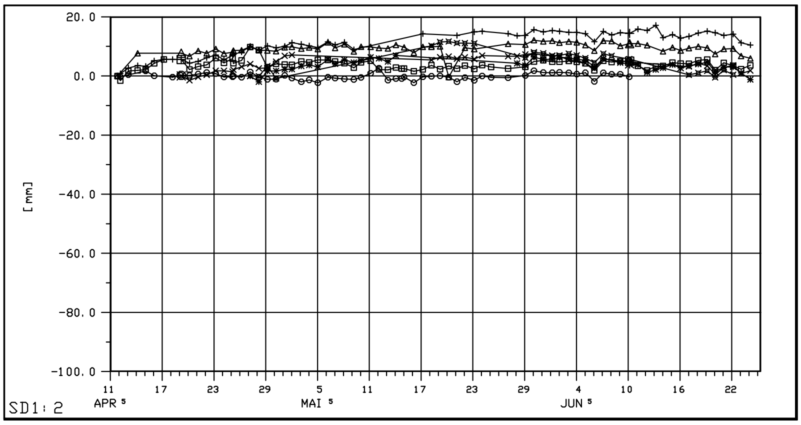






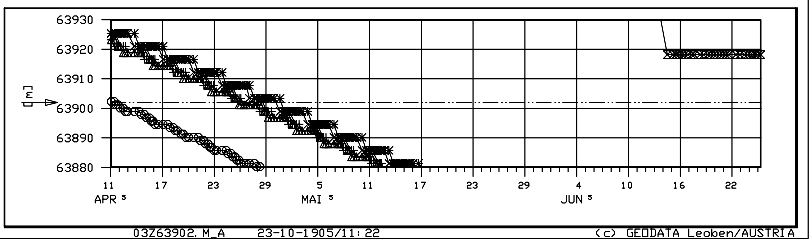
L. -DISPLACEMENT





C. -STAGES

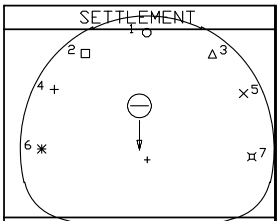
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining

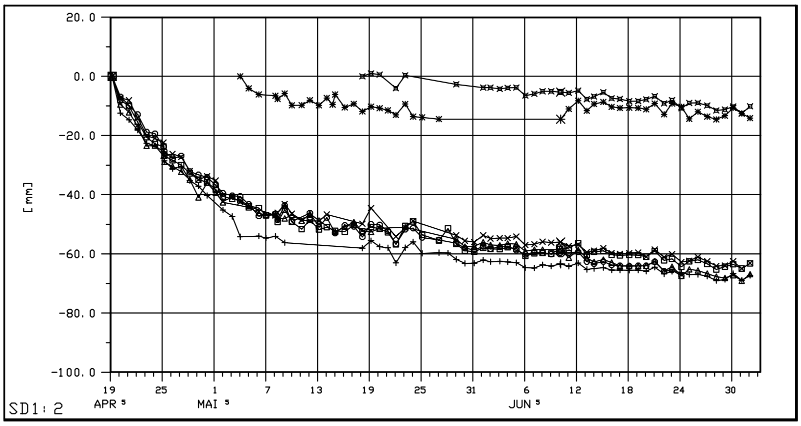


GC2_VDG V 970107 03Z63902.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELL-Re-Align	CH: 63893/00607

SETTLEMENT

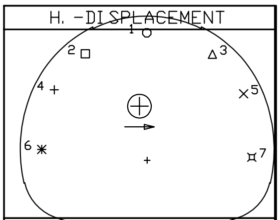


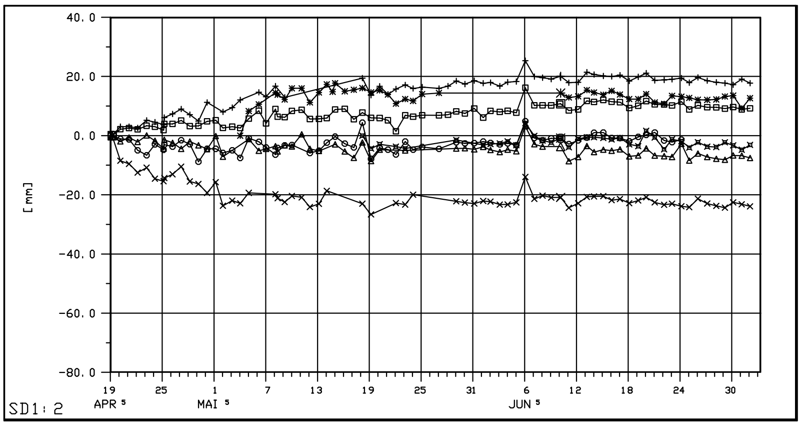


remarks:

STN : 31 CLASS CM45 (DWG300)
EXC. : 18/04/2005@12h00
ZERO: 19/04/2005@04h00

H. -DISPLACEMENT





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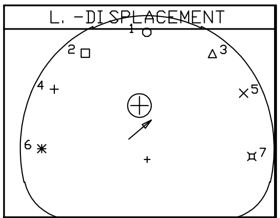
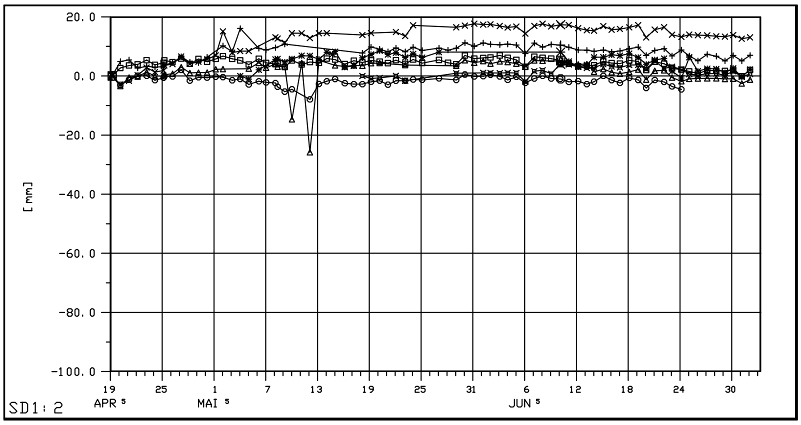
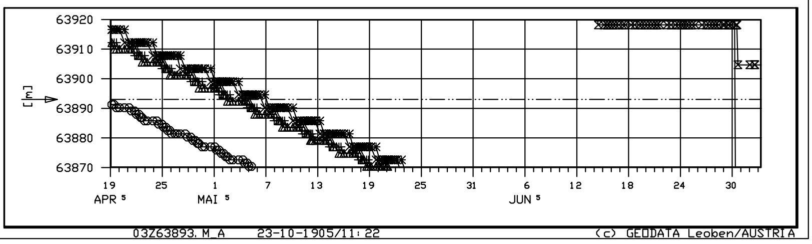
L. -DISPLACEMENT





C. -STAGES

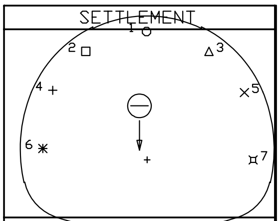
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining



GC2_VDG V 970107 03Z63893. M_A 23-10-1905/11: 22 (c) GEODATA Leoben/AUSTRIA

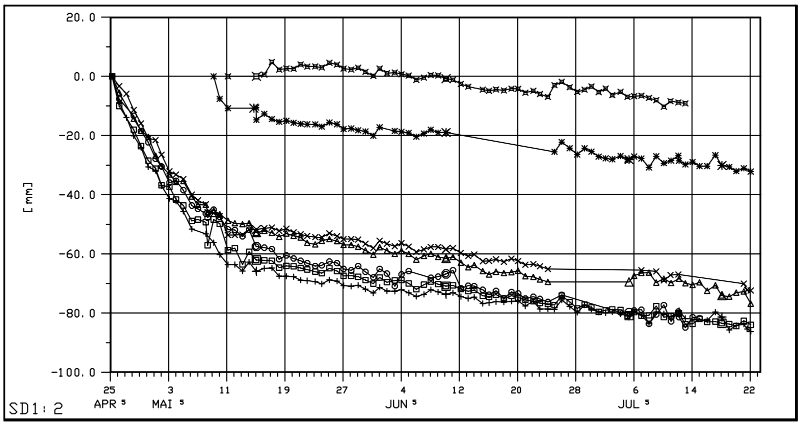
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S. p. A. CH: 63884/00616

SETTLEMENT

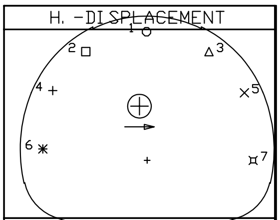


remarks:

STN : 32 CLASS CM45 (DWG300)
EXC. : 25/04/2005@03h00
ZERO: 25/04/2005@06h00



H. -DISPLACEMENT



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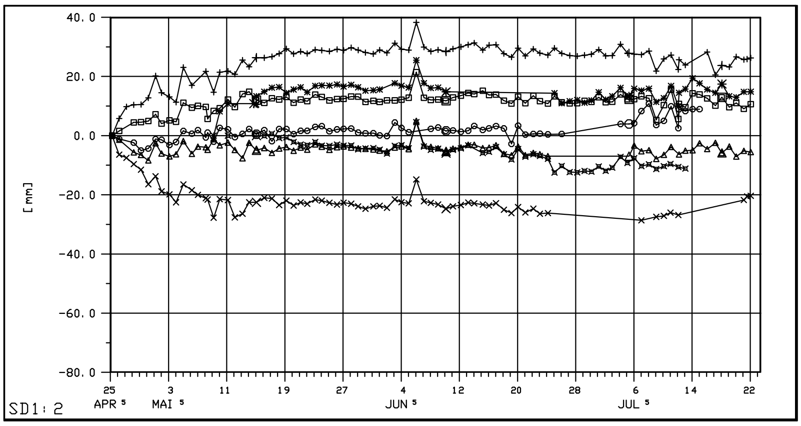
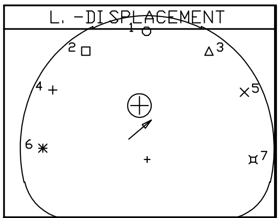
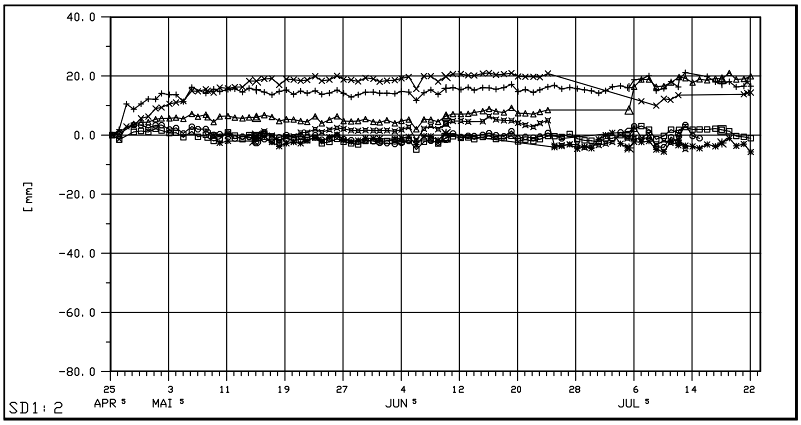


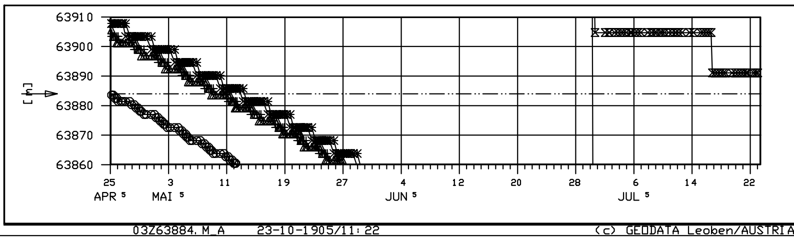
L. -DISPLACEMENT





C. -STAGES

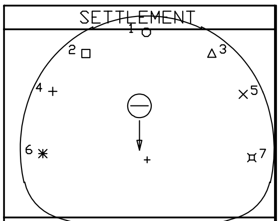
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining

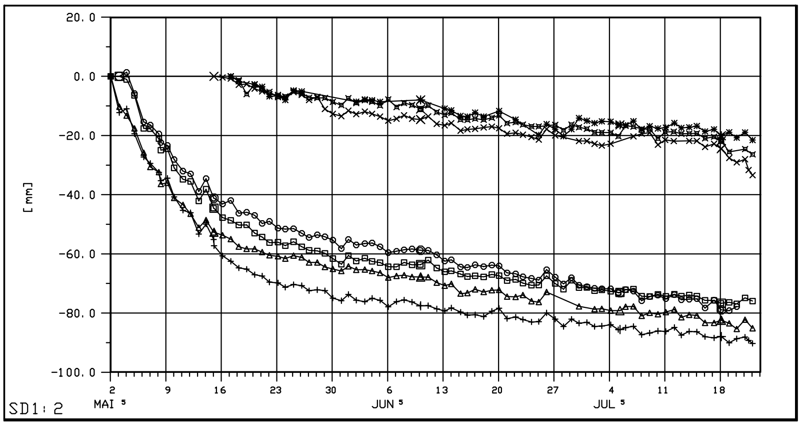


GC2_VDG V 970107 03Z63884.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 63875/00625

SETTLEMENT

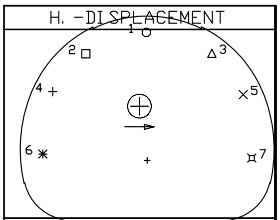


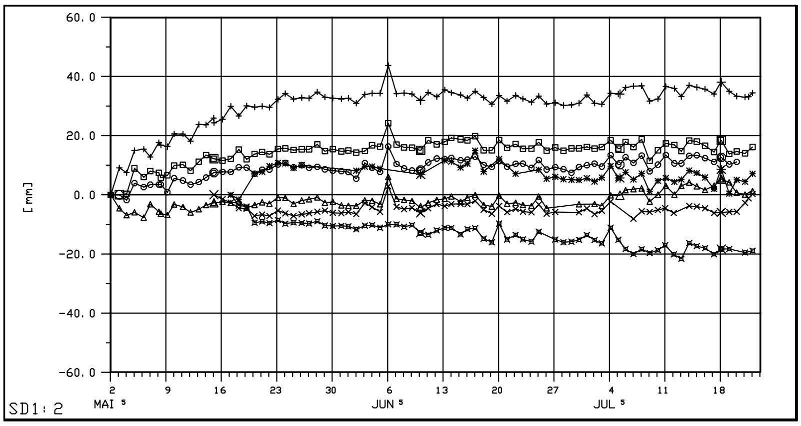


remarks:

STN : 33 CLASS CM45 (DWG300)
EXC. : 01/05/2005@21h00
ZERO : 02/05/2005@01h00

H.-DISPLACEMENT





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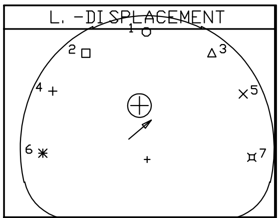
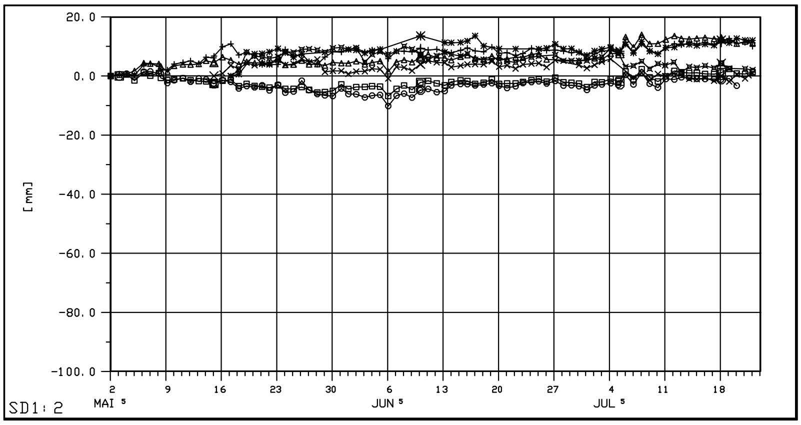
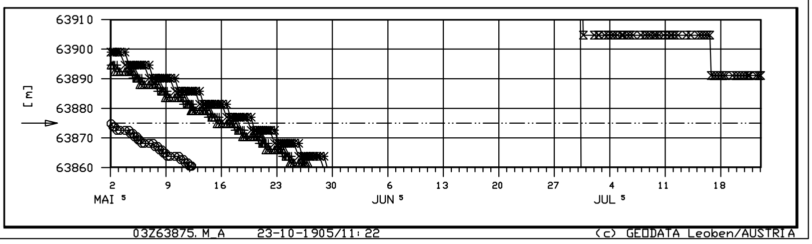
L.-DISPLACEMENT





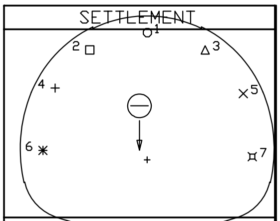
C.-STAGES

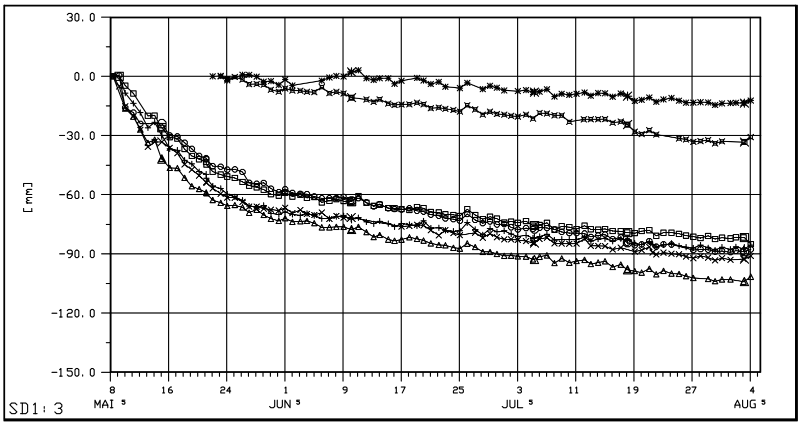
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining



UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 63866/00634

SETTLEMENT

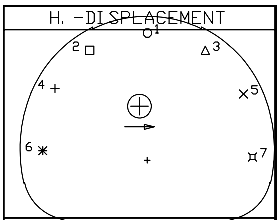


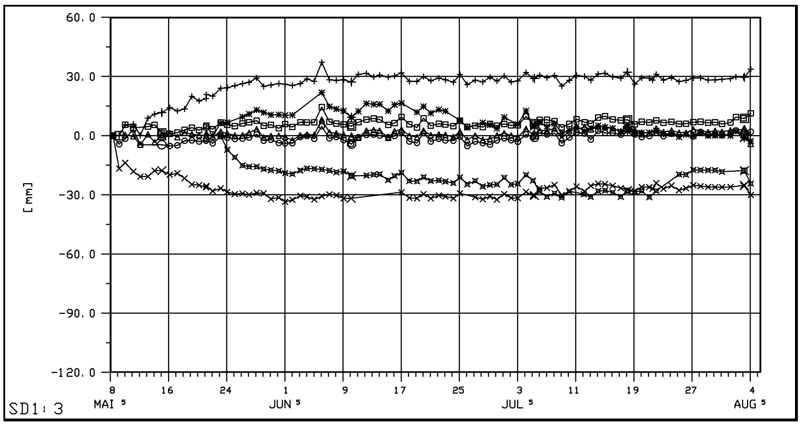


remarks:

STN : 34 CLASS CM45 (DWG300)
EXC. : 08/05/2005@04h00
ZERO: 08/05/2005@09h00

H.-DISPLACEMENT





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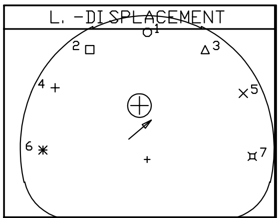
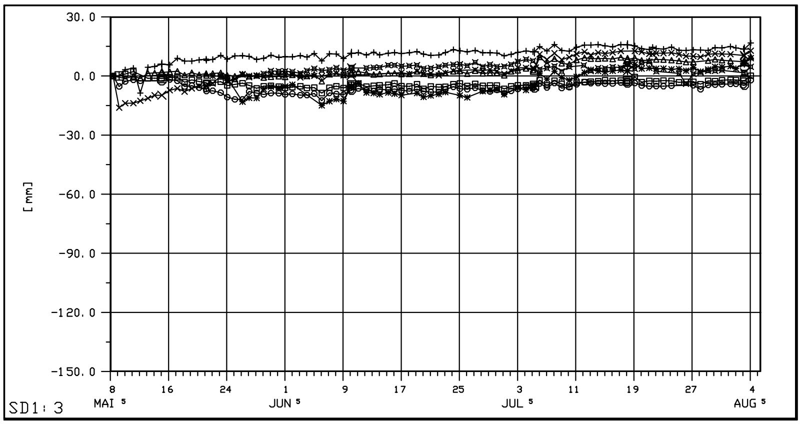
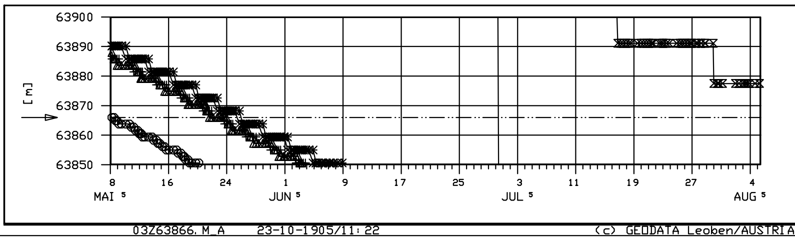
L.-DISPLACEMENT



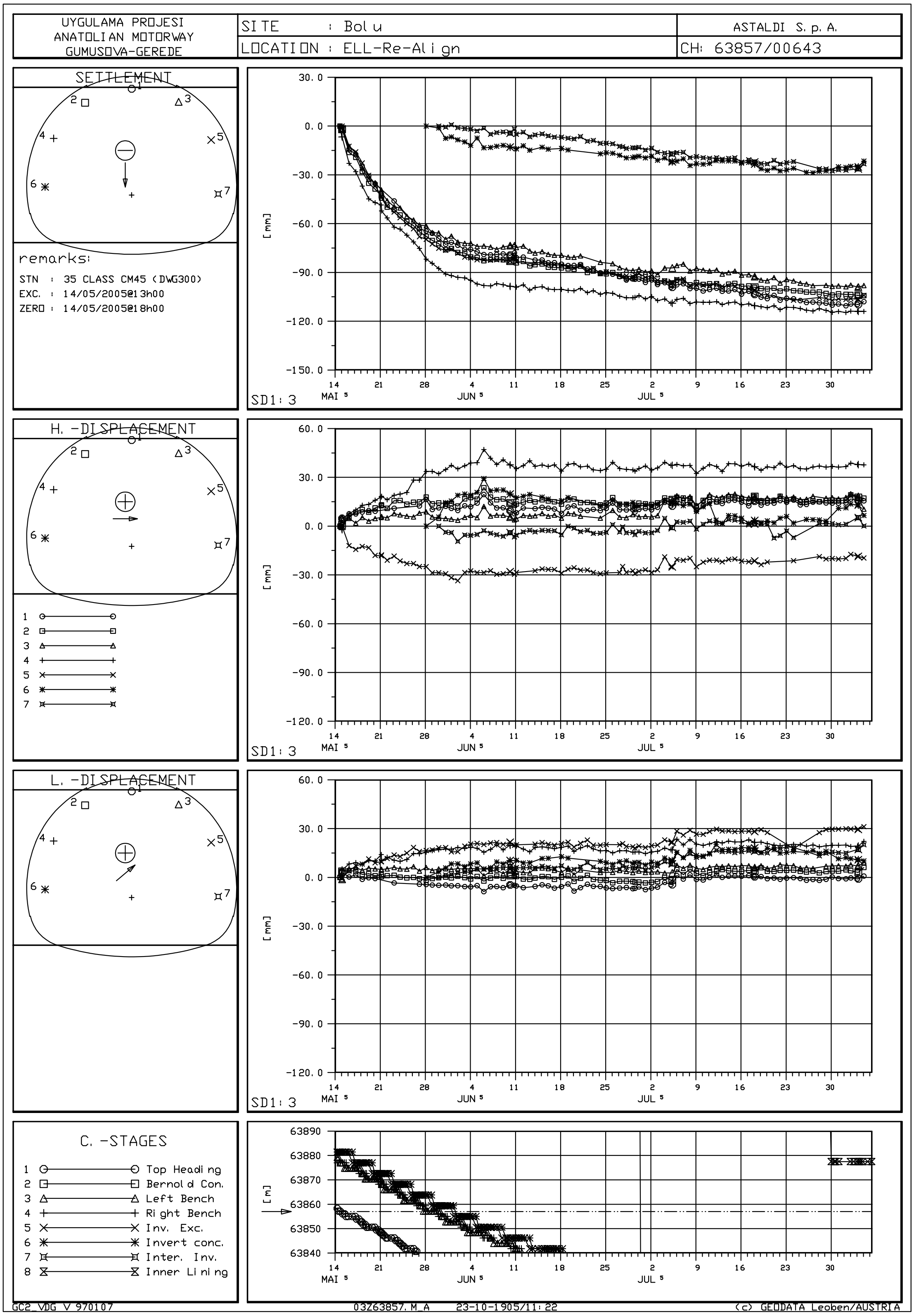


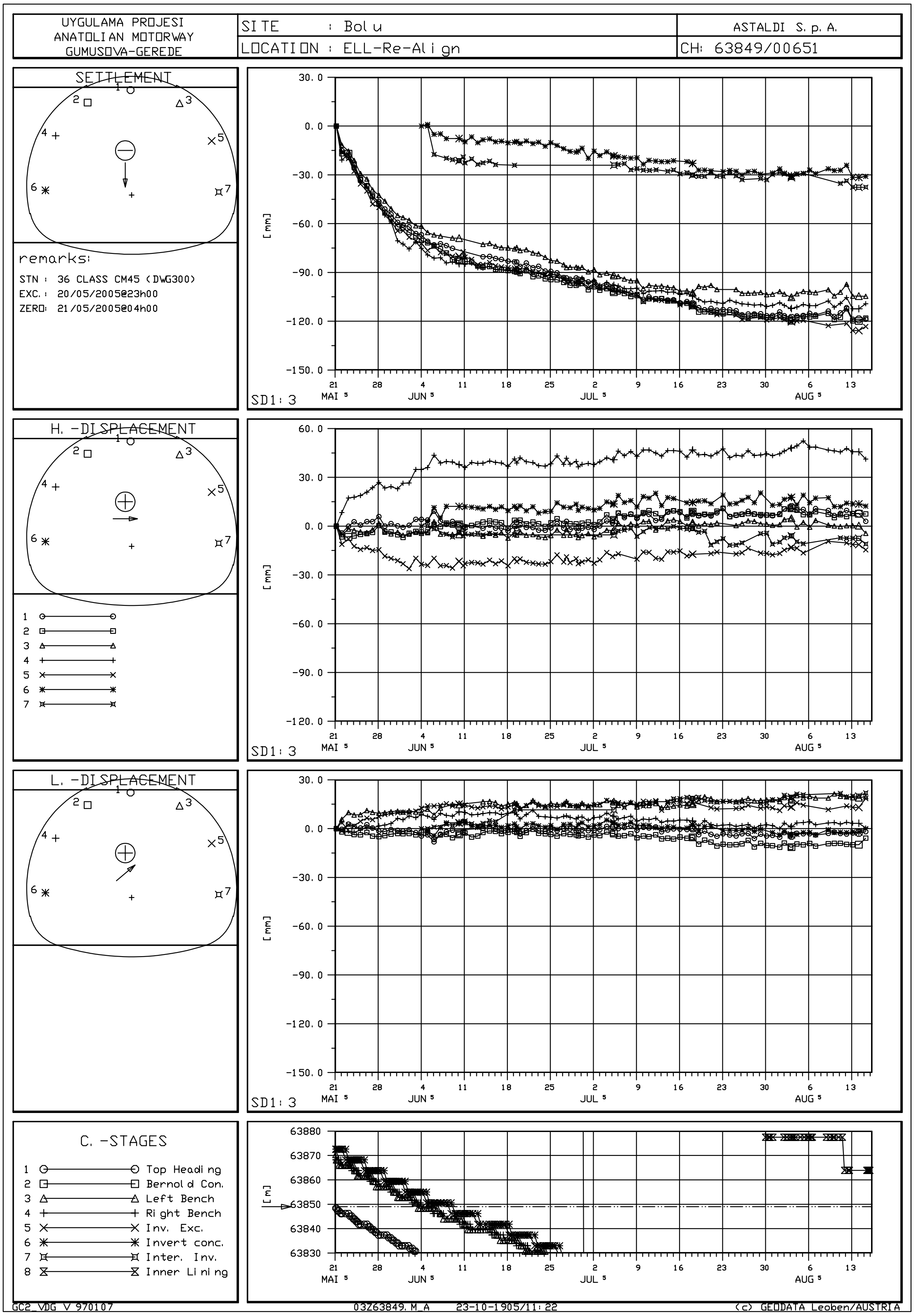
C.-STAGES

1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining



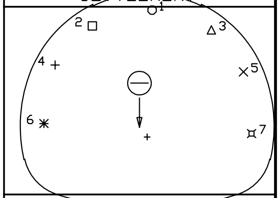
GC2_VDG V 970107 03Z63866.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA

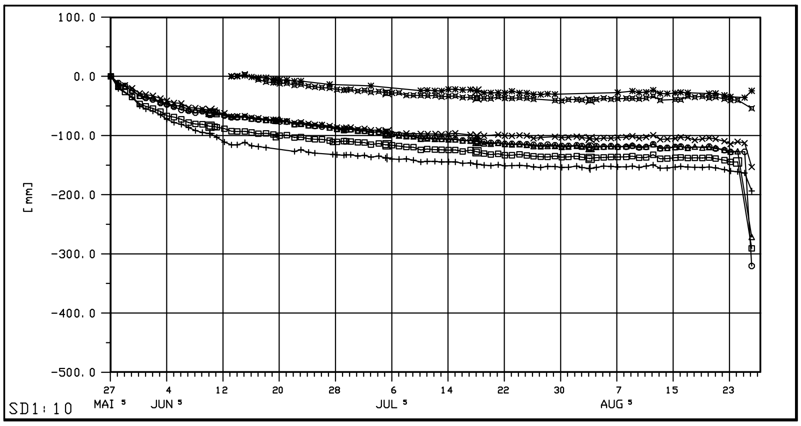

UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 63857/00643
SETTLEMENT
remarks:
STN : 35 CLASS CM45 (DWG300)
EXC. : 14/05/2005@13h00
ZERO : 14/05/2005@18h00
[mm]
SD1:3
H.-DISPLACEMENT
L.-DISPLACEMENT
C.-STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
MAI 5
JUN 5
JUL 5
GC2_VDG V 970107
03Z63857.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 63849/00651
SETTLEMENT
remarks:
STN : 36 CLASS CM45 (DWG300)
EXC. : 20/05/2005@23h00
ZERO: 21/05/2005@04h00
[mm]
SD1:3
MAI 5
JUN 5
JUL 5
AUG 5
H.-DISPLACEMENT
L.-DISPLACEMENT
C.-STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z63849.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 63840/00660

SETTLEMENT

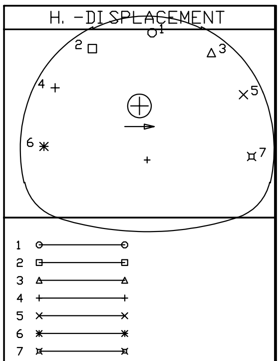


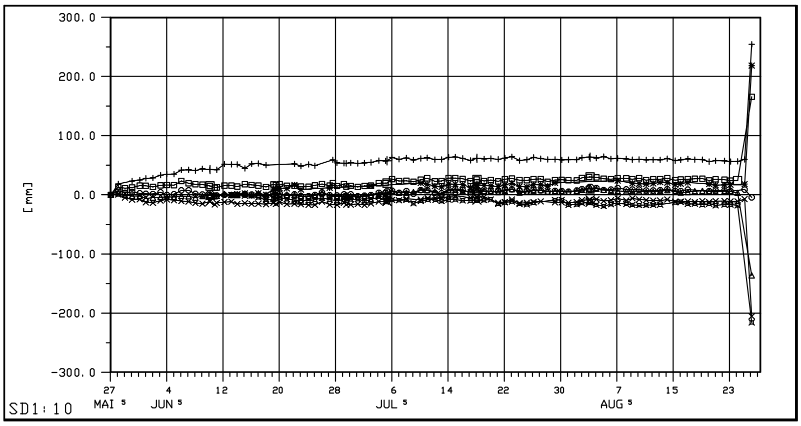


remarks:

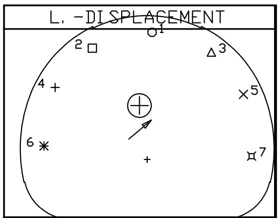
STN : 37 CLASS CM45 (DWG300)
EXC.: 26/05/2005@22h00
ZERO: 27/05/2005@02h00

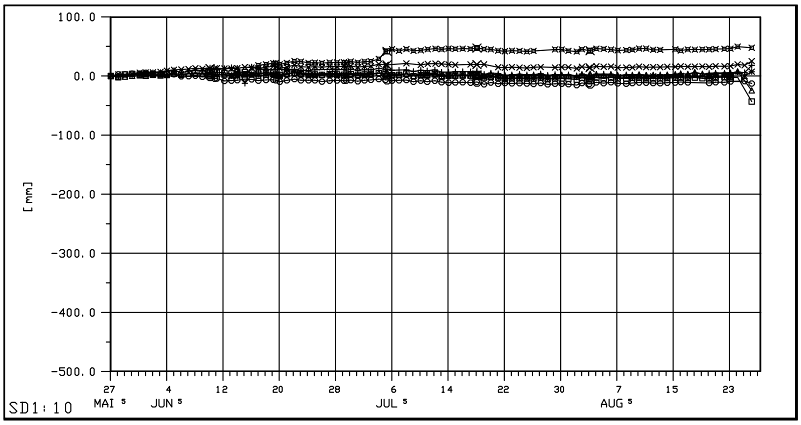
H.-DISPLACEMENT





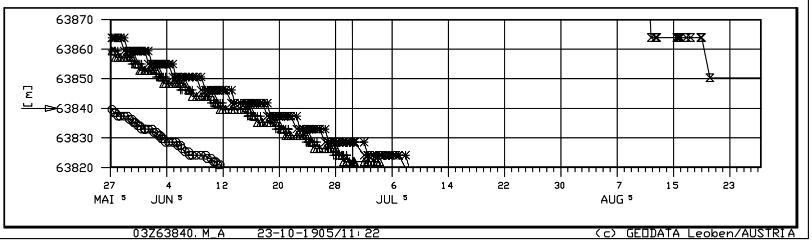
L.-DISPLACEMENT



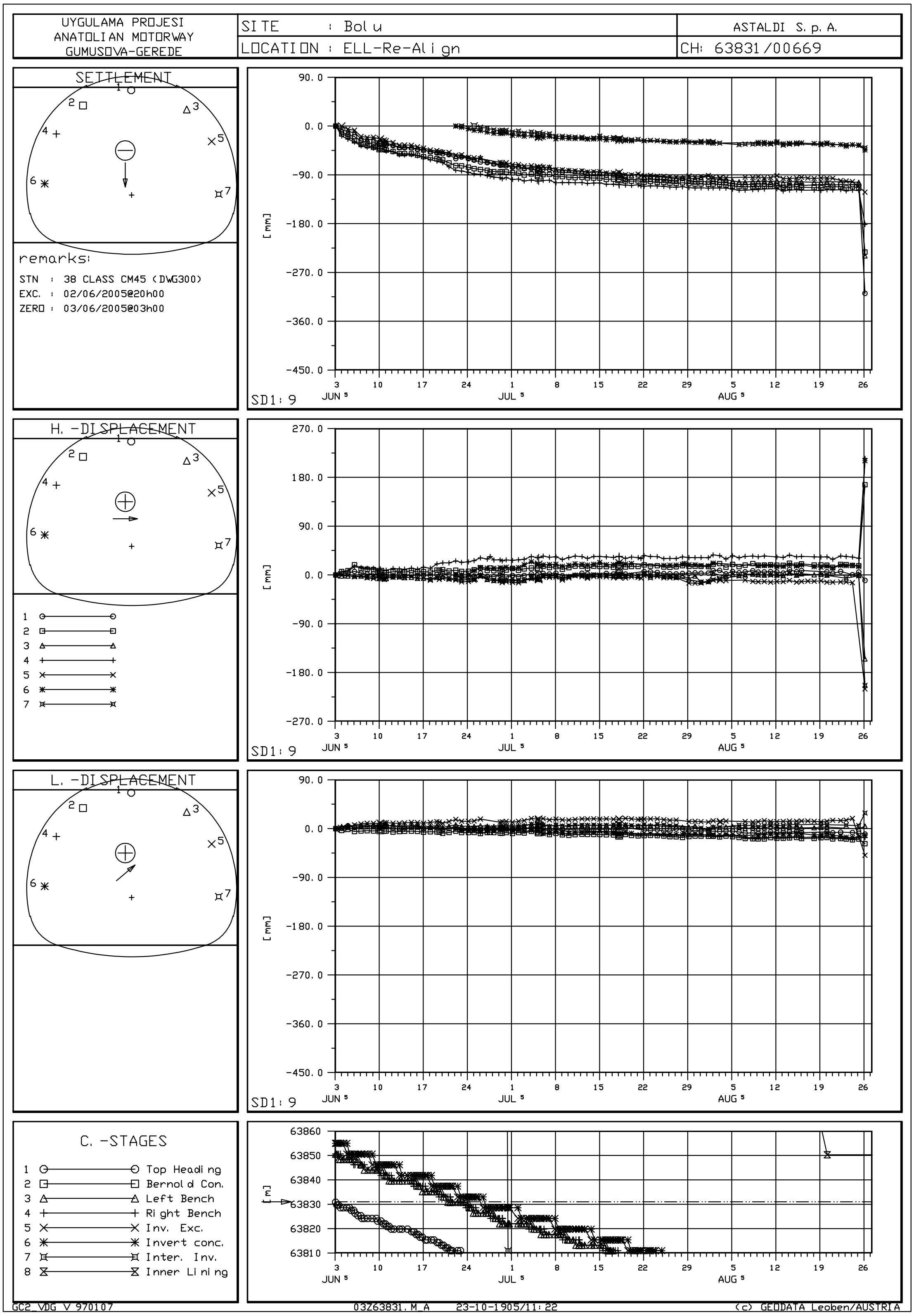


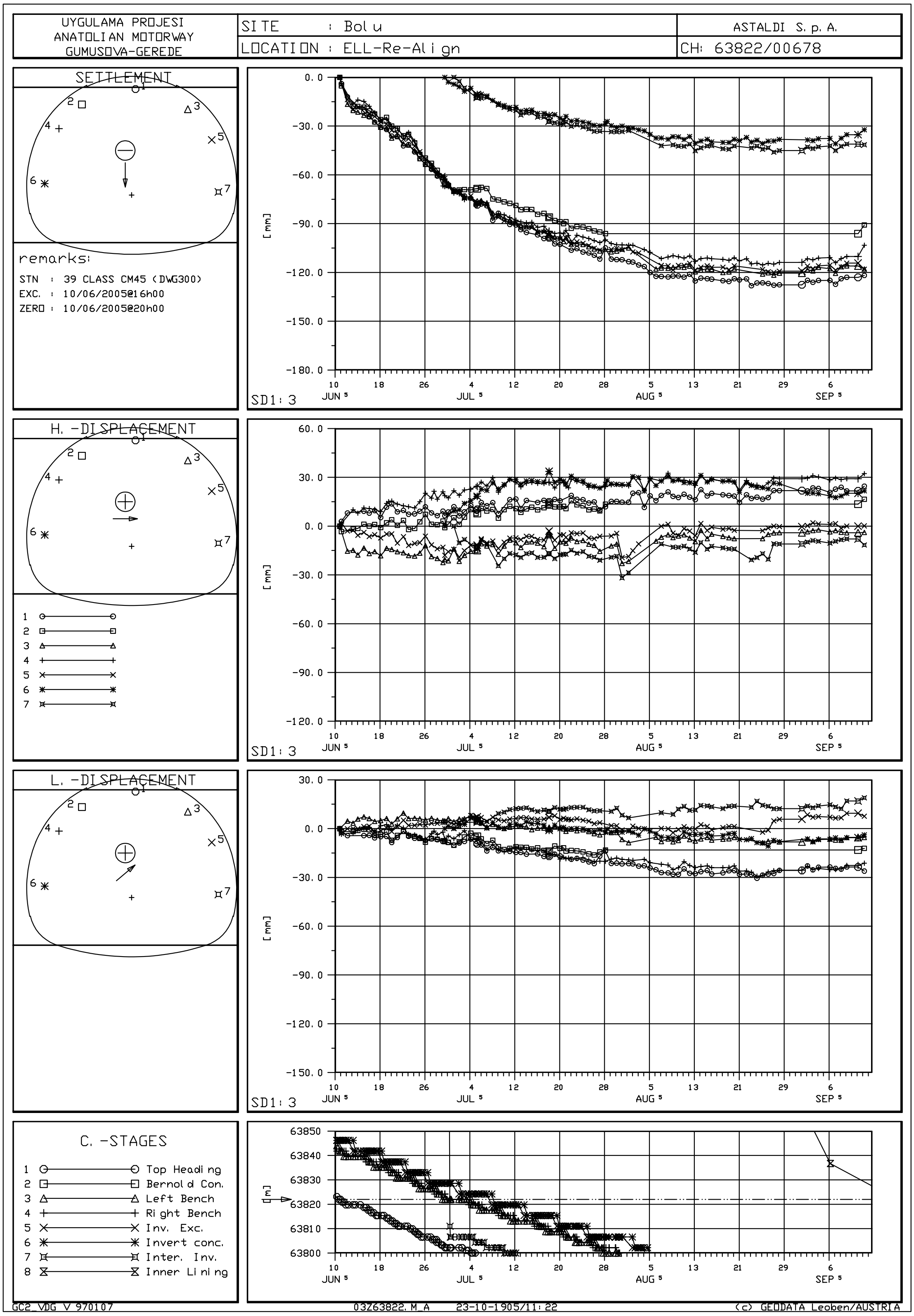
C.-STAGES

1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining



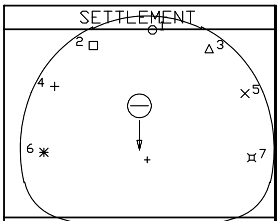
GC2_VDG V 970107 03Z63840.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 63831/00669
SETTLEMENT
remarks:
STN : 38 CLASS CM45 (DWG300)
EXC. : 02/06/2005@20h00
ZERO : 03/06/2005@03h00
[mm]
SD1:9
H.-DISPLACEMENT
L.-DISPLACEMENT
C.-STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
JUN 5
JUL 5
AUG 5
GC2_VDG V 970107
03Z63831.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 63822/00678
SETTLEMENT
remarks:
STN : 39 CLASS CM45 (DWG300)
EXC. : 10/06/2005@16h00
ZERO : 10/06/2005@20h00
[mm]
SD1:3
JUN 5
JUL 5
AUG 5
SEP 5
H.-DISPLACEMENT
L.-DISPLACEMENT
C.-STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z63822.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

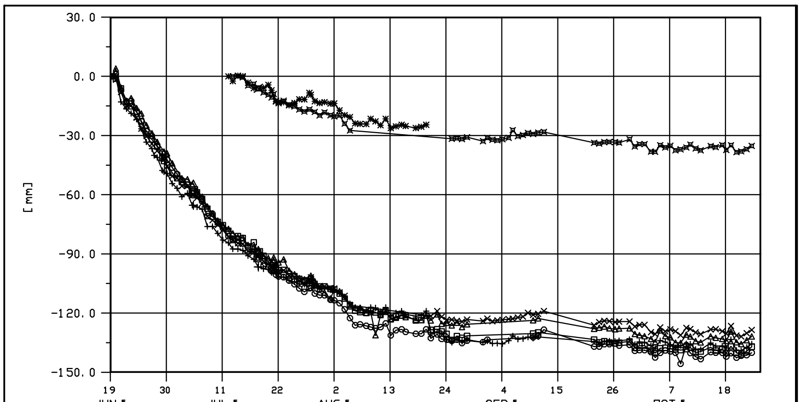
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELL-Re-Align	CH: 63815/00685

SETTLEMENT



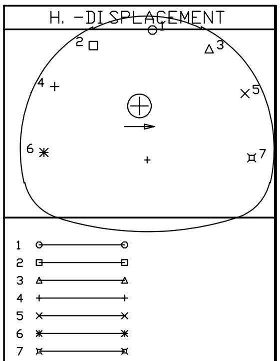
remarks:

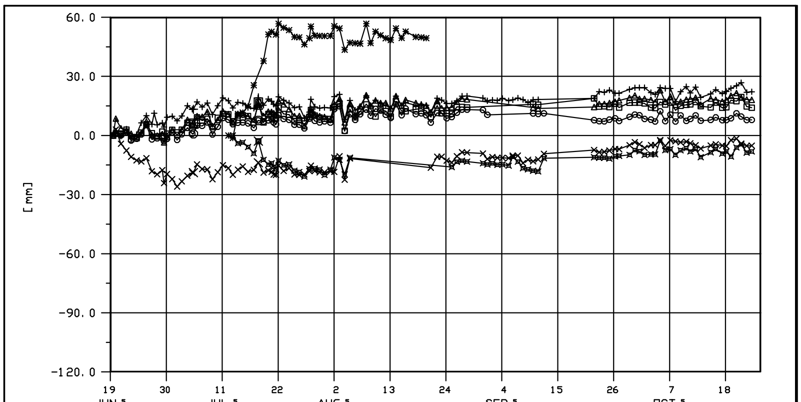
STN : 40 CLASS CM45 (DWG300)
EXC. : 19/06/2005@06h00
ZERO : 19/06/2005@13h00



SD1:3

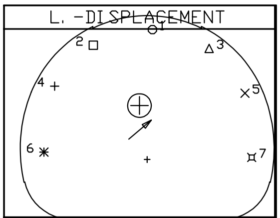
H.-DISPLACEMENT

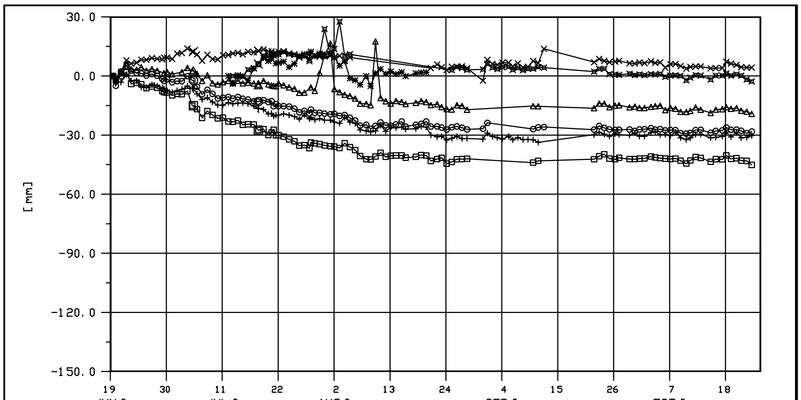




SD1:3

L.-DISPLACEMENT

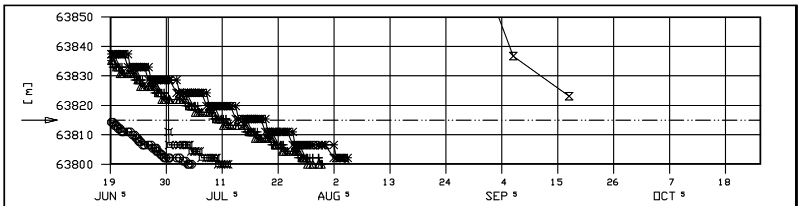




SD1:3

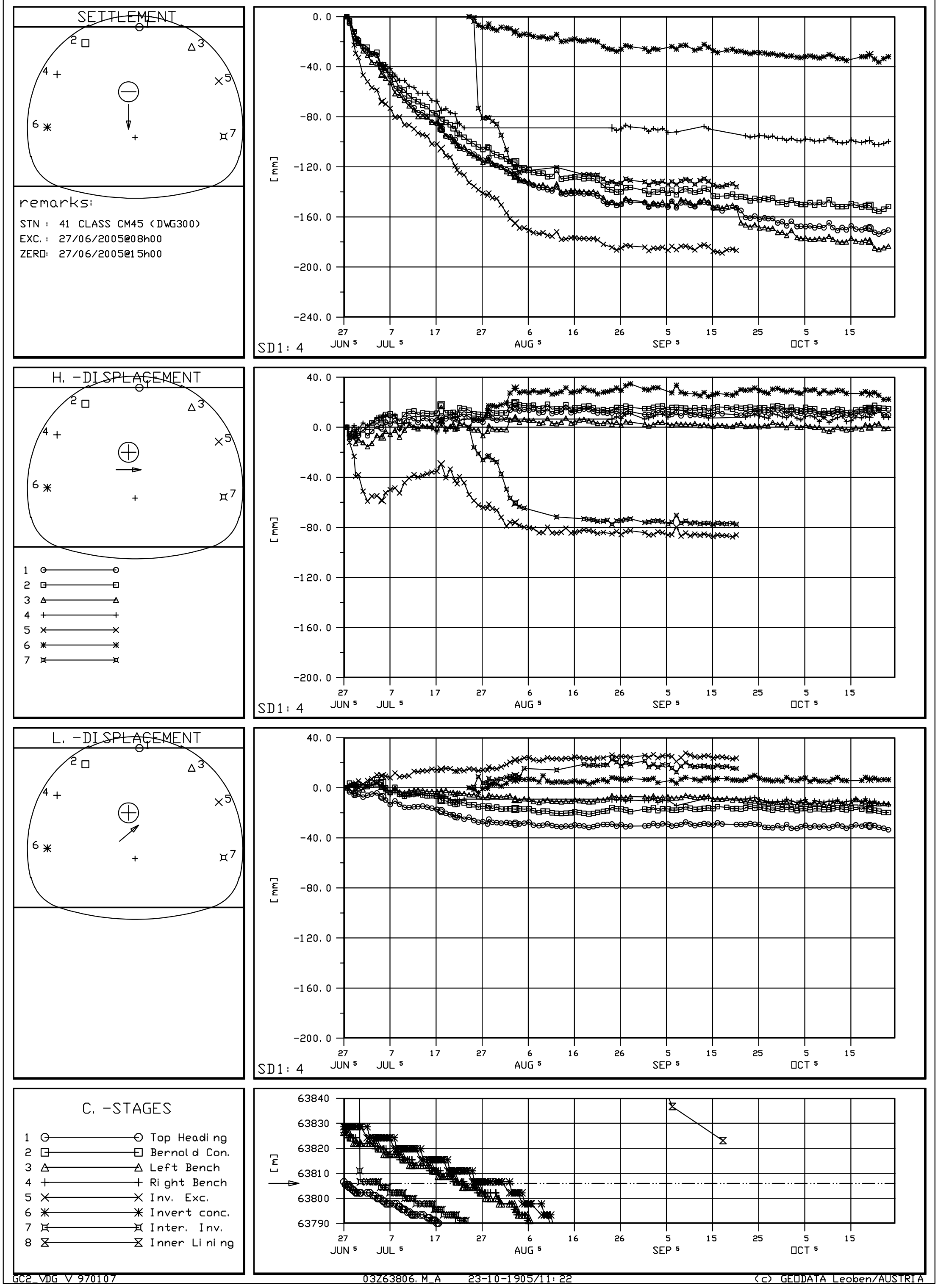
C.-STAGES

1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining

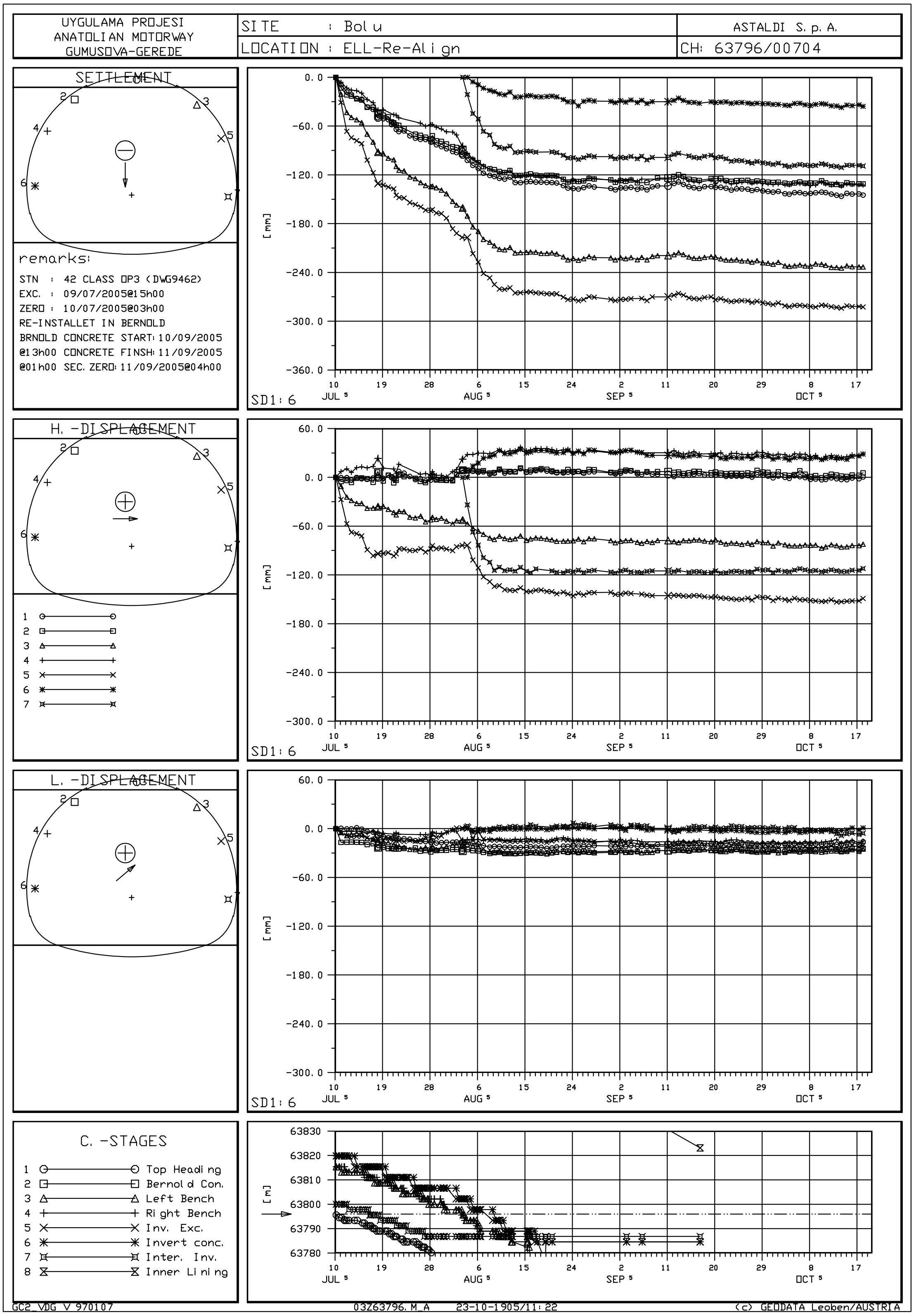


GC2_VDG V 970107 03Z63815.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S.p.A. CH: 63806/00694



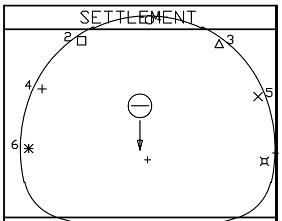
GC2_VDG V 970107 03Z63806.M_A 23-10-1905/11:22 (c) GEODATA Leoben/AUSTRIA

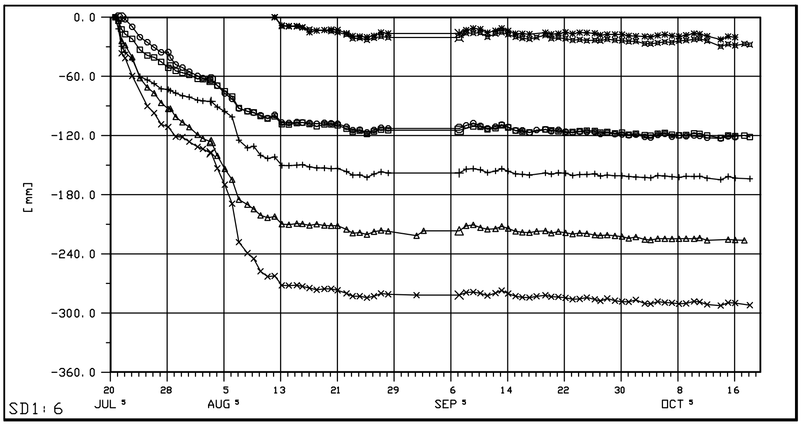


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 63796/00704
SETTLEMENT
remarks:
STN : 42 CLASS DP3 (DWG9462)
EXC. : 09/07/2005@15h00
ZERO : 10/07/2005@03h00
RE-INSTALLET IN BERNOLD
BRNOLD CONCRETE START: 10/09/2005
@13h00 CONCRETE FINSH: 11/09/2005
@01h00 SEC. ZERO: 11/09/2005@04h00
[mm]
SD1: 6
JUL 5
AUG 5
SEP 5
OCT 5
H.-DISPLACEMENT
L.-DISPLACEMENT
C.-STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z63796.M_A 23-10-1905/11:22
(c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELL-Re-Align	ASTALDI S. p. A. CH: 63788/00712

SETTLEMENT

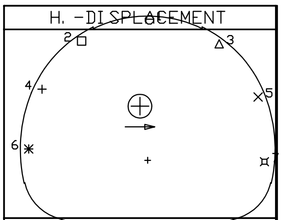




remarks:

STN : 43 CLASS OP3 (DWG9462)
EXC. : 20/07/2005@10h00
ZERO : 21/07/2005@03h00
RE-INSTALLET IN BERNOLD
BRNOLD CONCRETE START: 05/09/2005
@12h00 CONCRETE FINSH: 06/09/2005
@10h30 SEC. ZERO: 07/09/2005@04h00

H. -DISPLACEMENT



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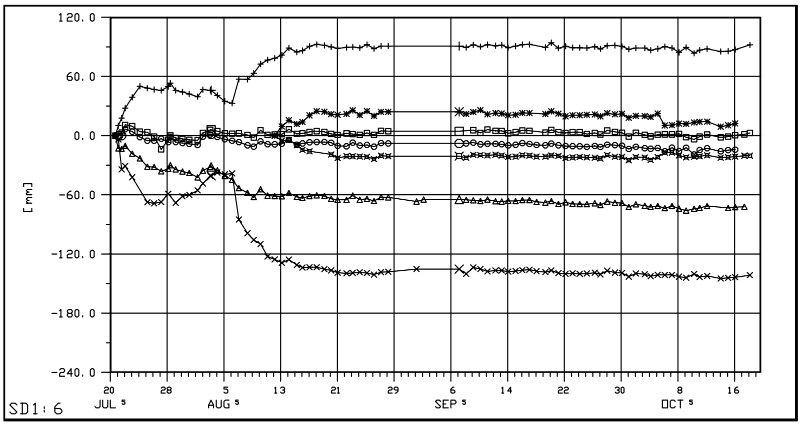
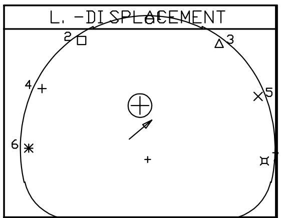
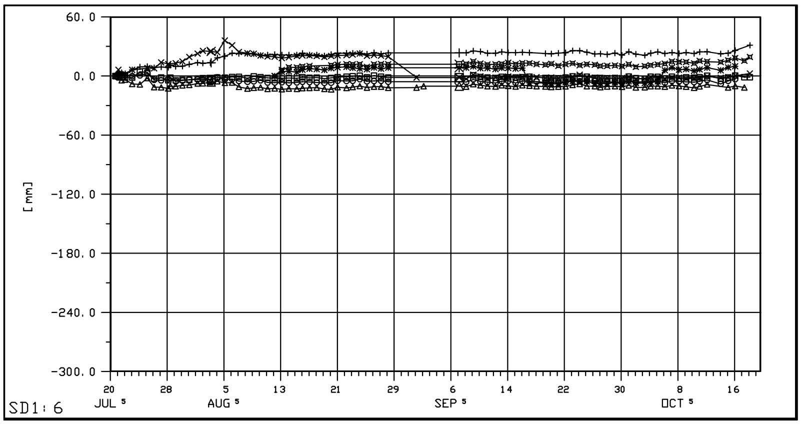


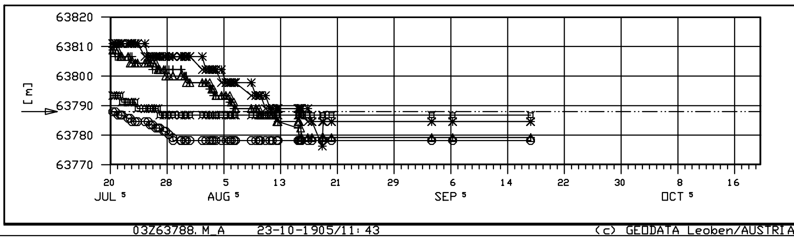
L. -DISPLACEMENT

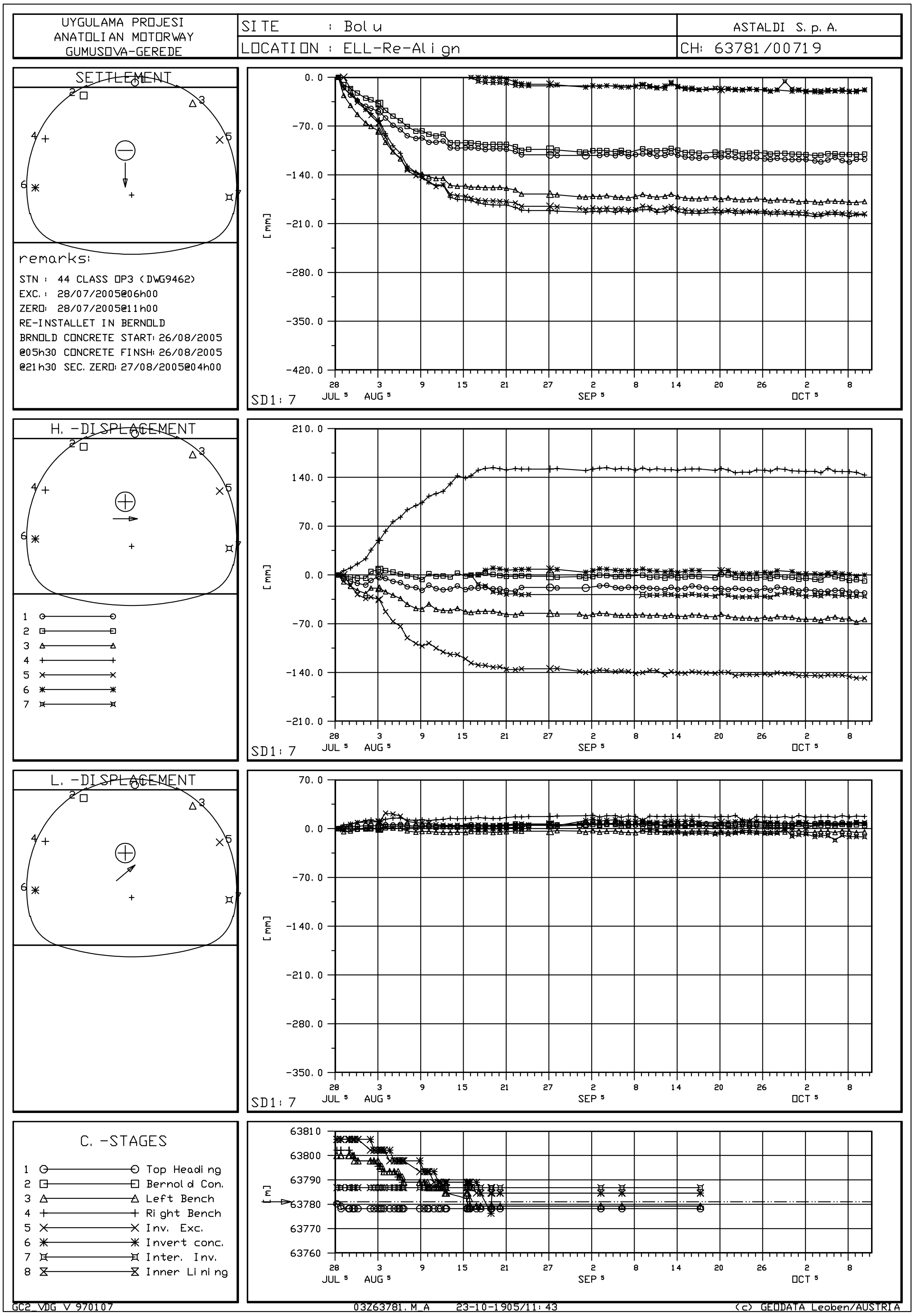




C. -STAGES

1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining




UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELL-Re-Align
ASTALDI S.p.A.
CH: 63781/00719
SETTLEMENT
remarks:
STN : 44 CLASS DP3 (DWG9462)
EXC. : 28/07/2005@06h00
ZERO: 28/07/2005@11h00
RE-INSTALLET IN BERNOLD
BRNOLD CONCRETE START: 26/08/2005
@05h30 CONCRETE FINSH: 26/08/2005
@21h30 SEC. ZERO: 27/08/2005@04h00
SD1: 7
H. -DISPLACEMENT
L. -DISPLACEMENT
[mm]
JUL 5
AUG 5
SEP 5
OCT 5
C. -STAGES
1 Top Heading
2 Bernold Con.
3 Left Bench
4 Right Bench
5 Inv. Exc.
6 Invert conc.
7 Inter. Inv.
8 Inner Lining
[m]
GC2_VDG V 970107
03Z63781.M_A 23-10-1905/11:43
(c) GEODATA Leoben/AUSTRIA

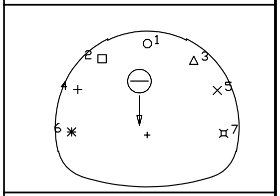
Right Tube

3D Monitoring (S-H-L) Diagrams

(Displacement vs. Time

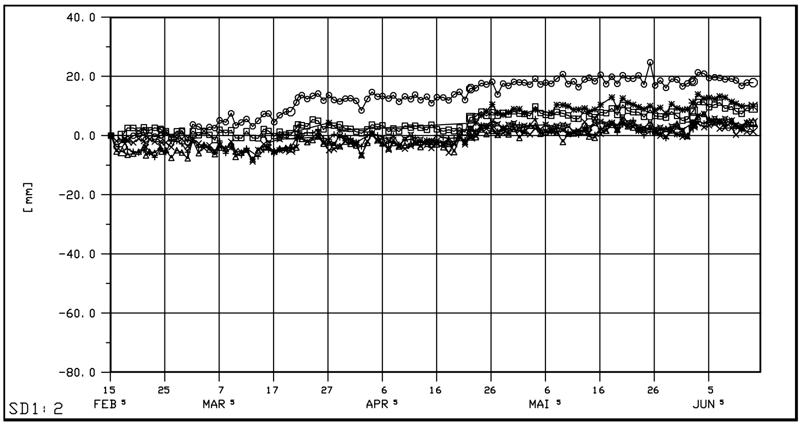
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELR-Re-Align	CH: 53960/00540

SETTLEMENT

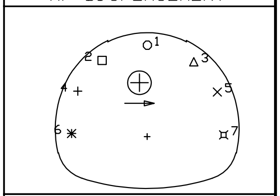


remarks:

STN : 1 CLASS CM 45 (DWG300)
EXC. : 14/02/2005@22h00
ZERO: 15/02/2005@03h00



H. -DISPLACEMENT



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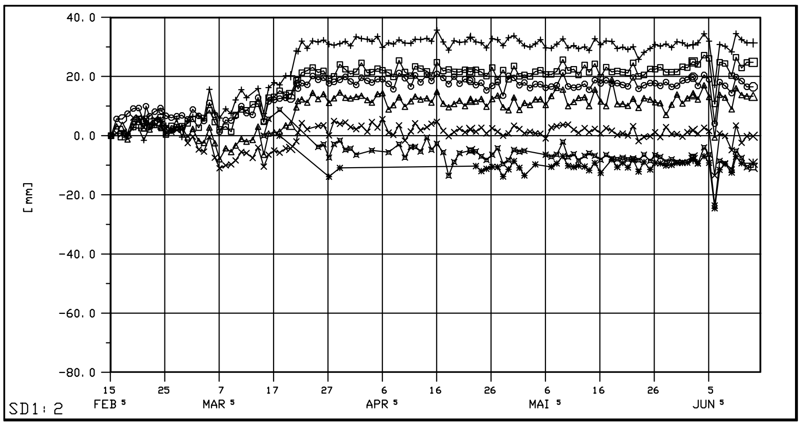
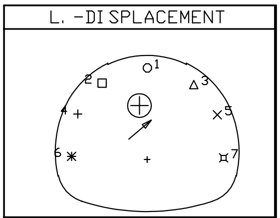
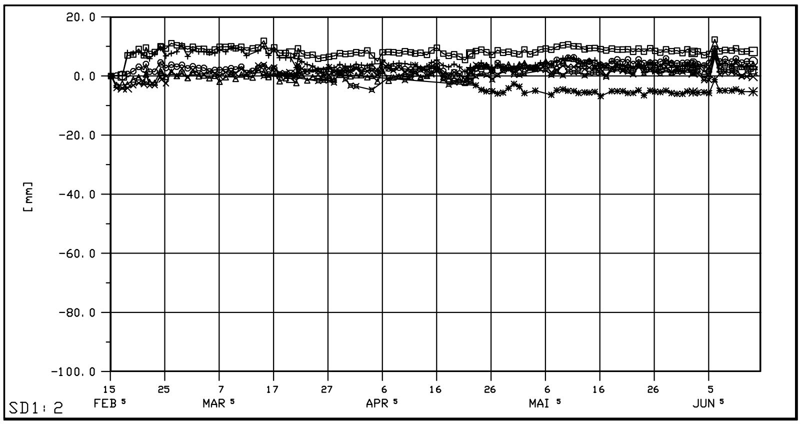


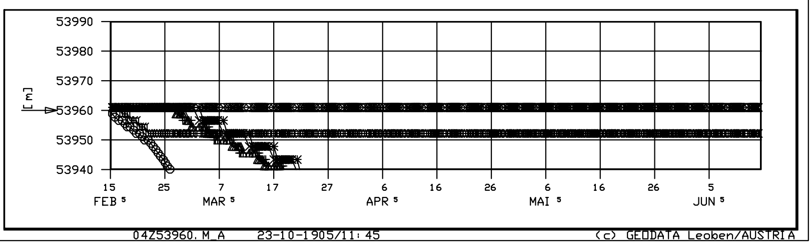
L. -DISPLACEMENT





C. -STAGES

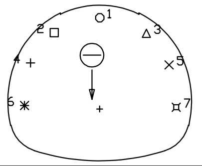
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



GC2_VDG V 970107 04Z53960.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA

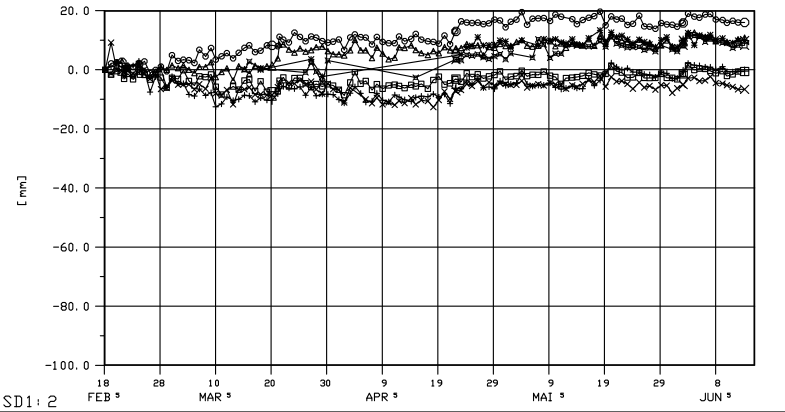
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELR-Re-Align	CH: 53955/00545

SETTLEMENT



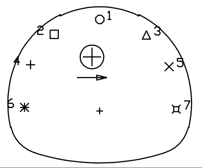
remarks:

STN : 2 CLASS CM 45 (DWG300)
EXC. : 17/02/2005@16h00
ZERO: 18/02/2005@03h00



SD1: 2

H.-DISPLACEMENT



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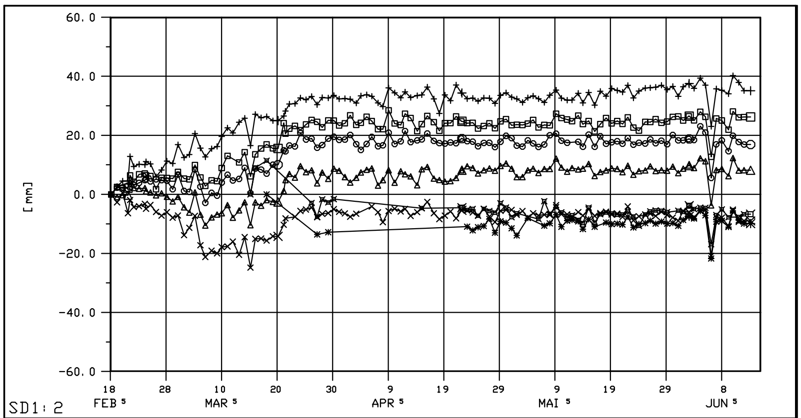
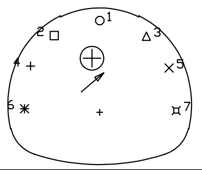
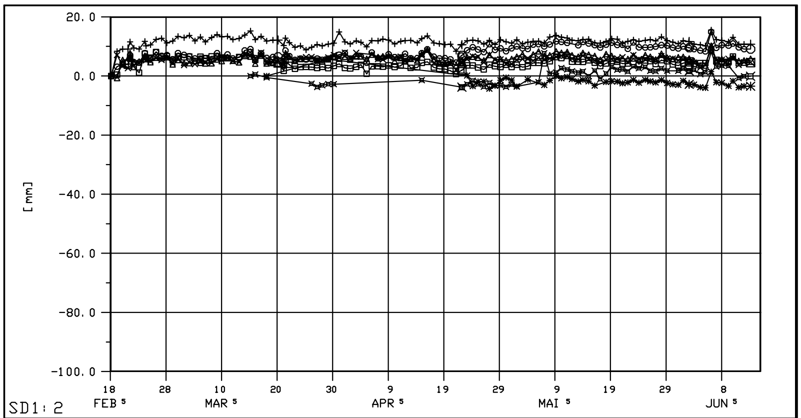


SD1: 2

L.-DISPLACEMENT

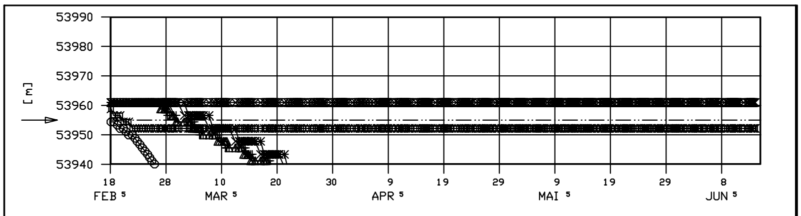




SD1: 2

C.-STAGES

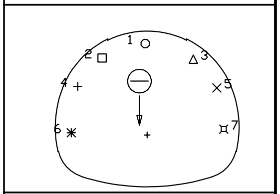
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining

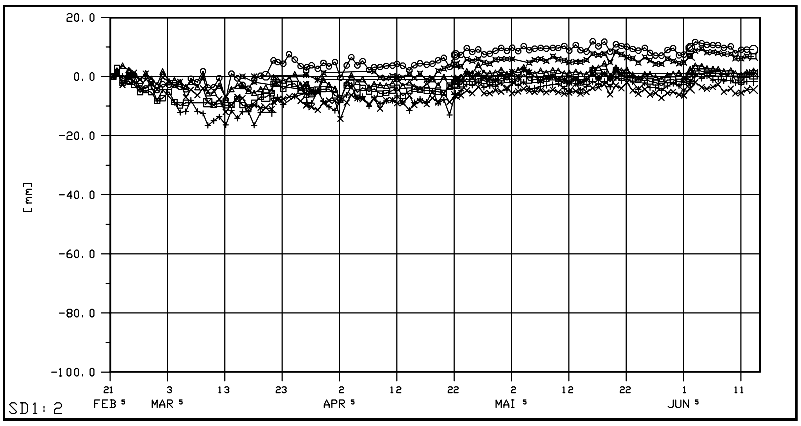


GC2_VDG V 970107 04Z53955.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELR-Re-Align	ASTALDI S. p. A. CH: 53950/00550

SETTLEMENT

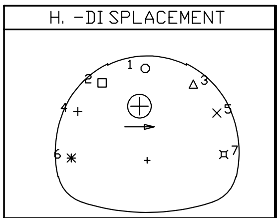


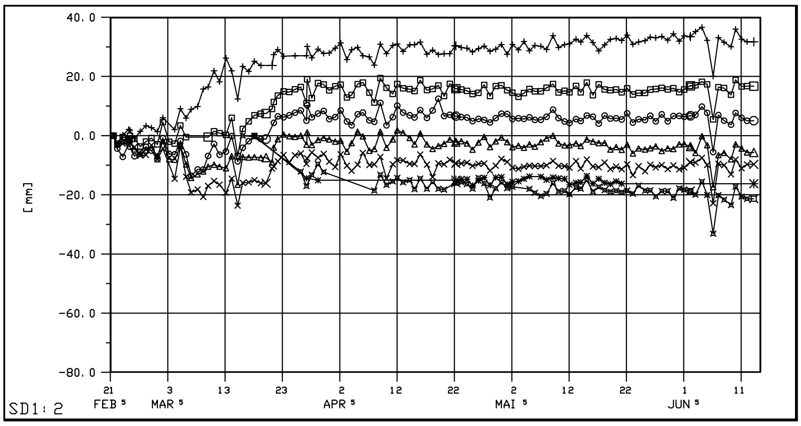


remarks:

STN : 3 CLASS CM 45 (DWG300)
EXC. : 21/02/2005@08h00
ZERO: 21/02/2005@13h00

H. -DISPLACEMENT





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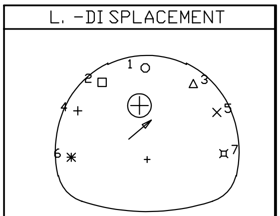
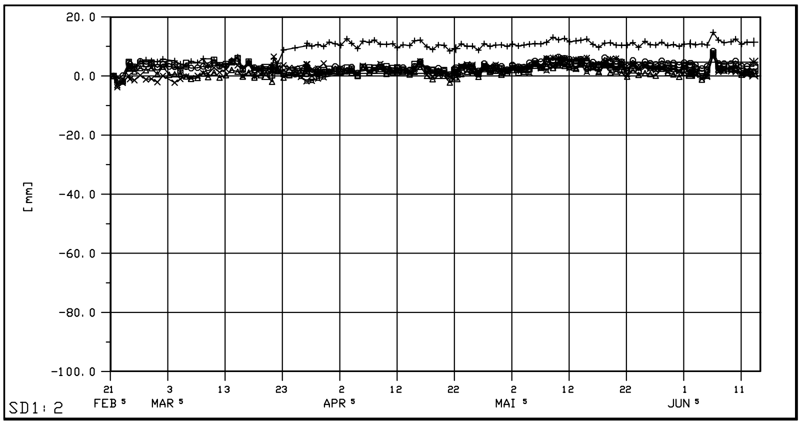
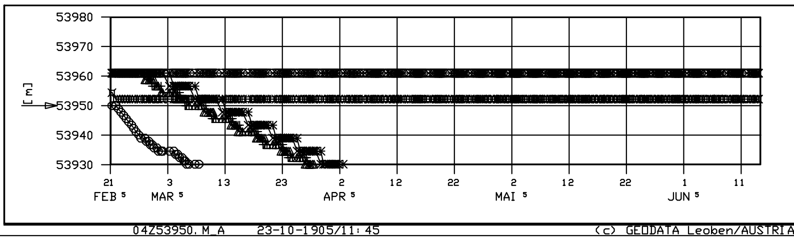
L. -DISPLACEMENT





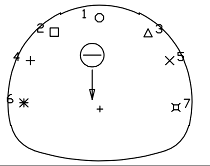
C. -STAGES

1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



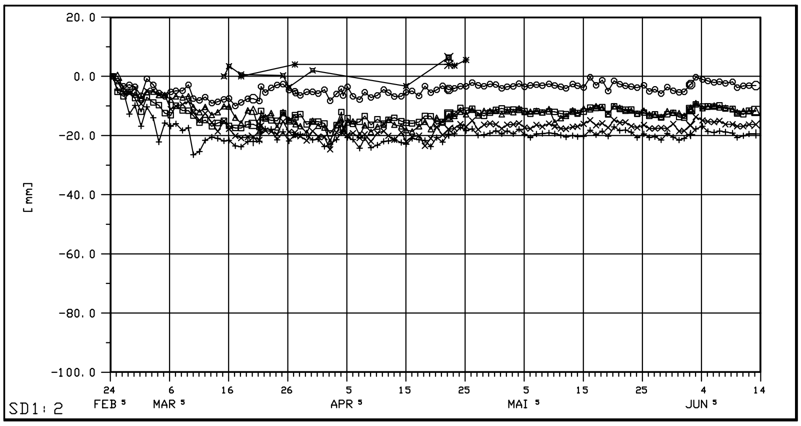
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELR-Re-Align	CH: 53945/00555

SETTLEMENT

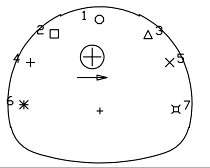


remarks:

STN : 4 CLASS CM 45 (DWG300)
EXC. : 24/02/2005@06h00
ZERO: 24/02/2005@10h00



H. -DISPLACEMENT



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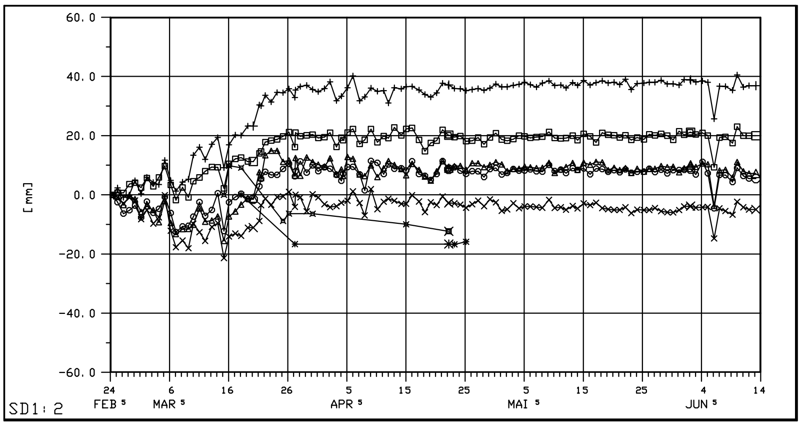
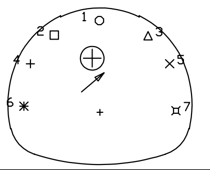
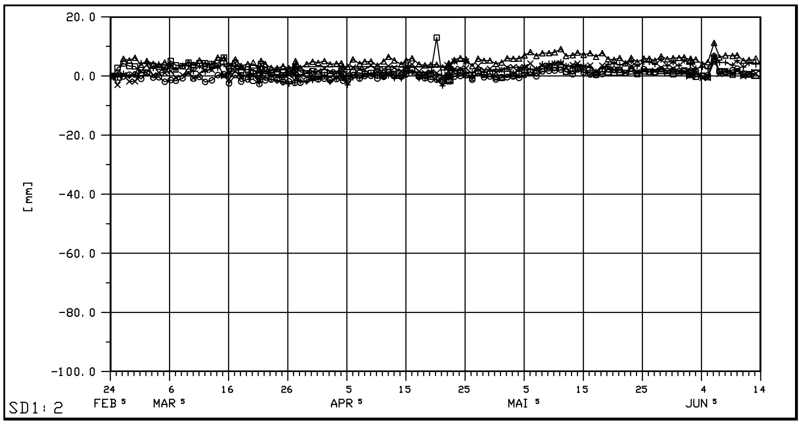


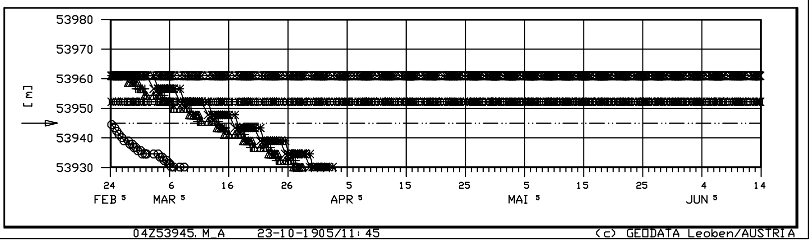
L. -DISPLACEMENT





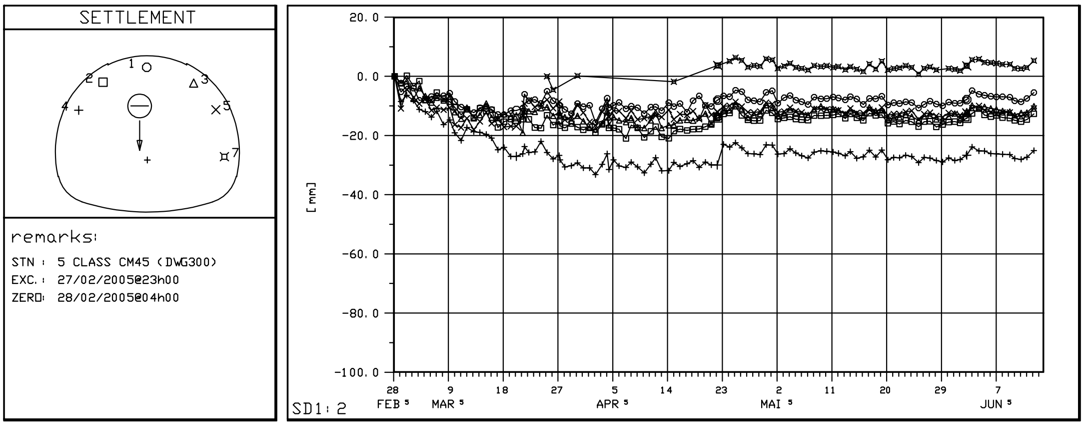
C. -STAGES

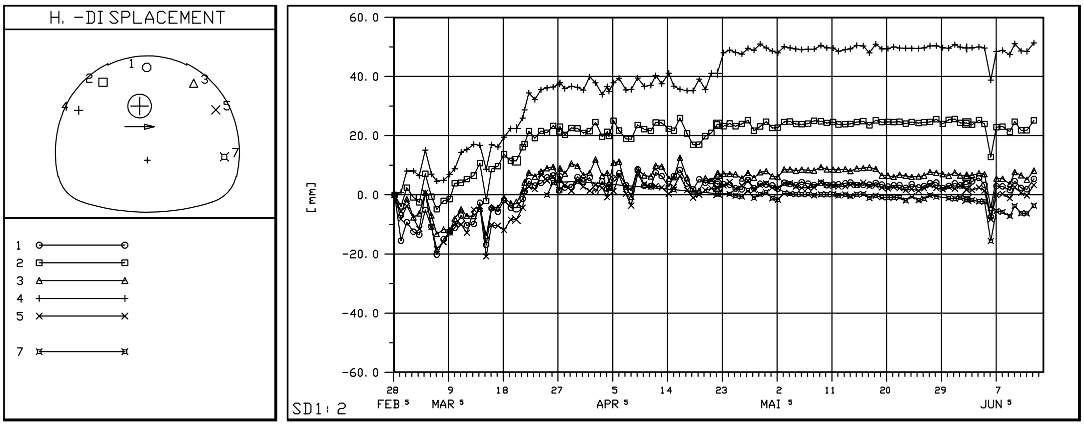
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining

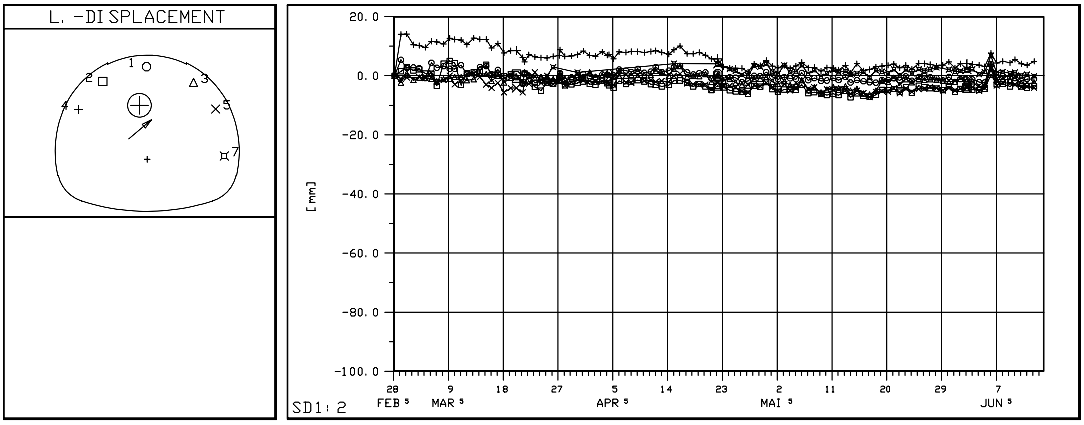


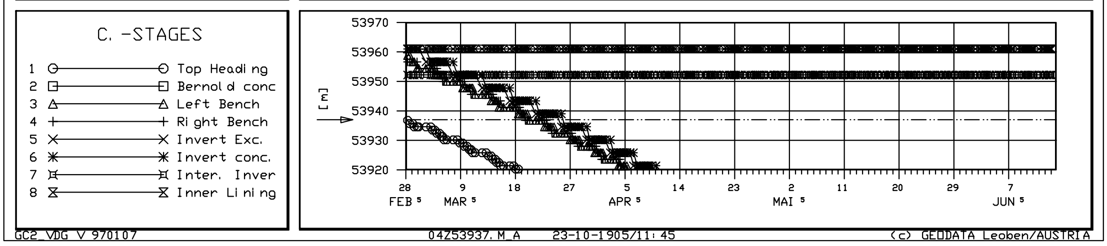
GC2_VDG V 970107 04Z53945.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELR-Re-Align	CH: 53937/00563

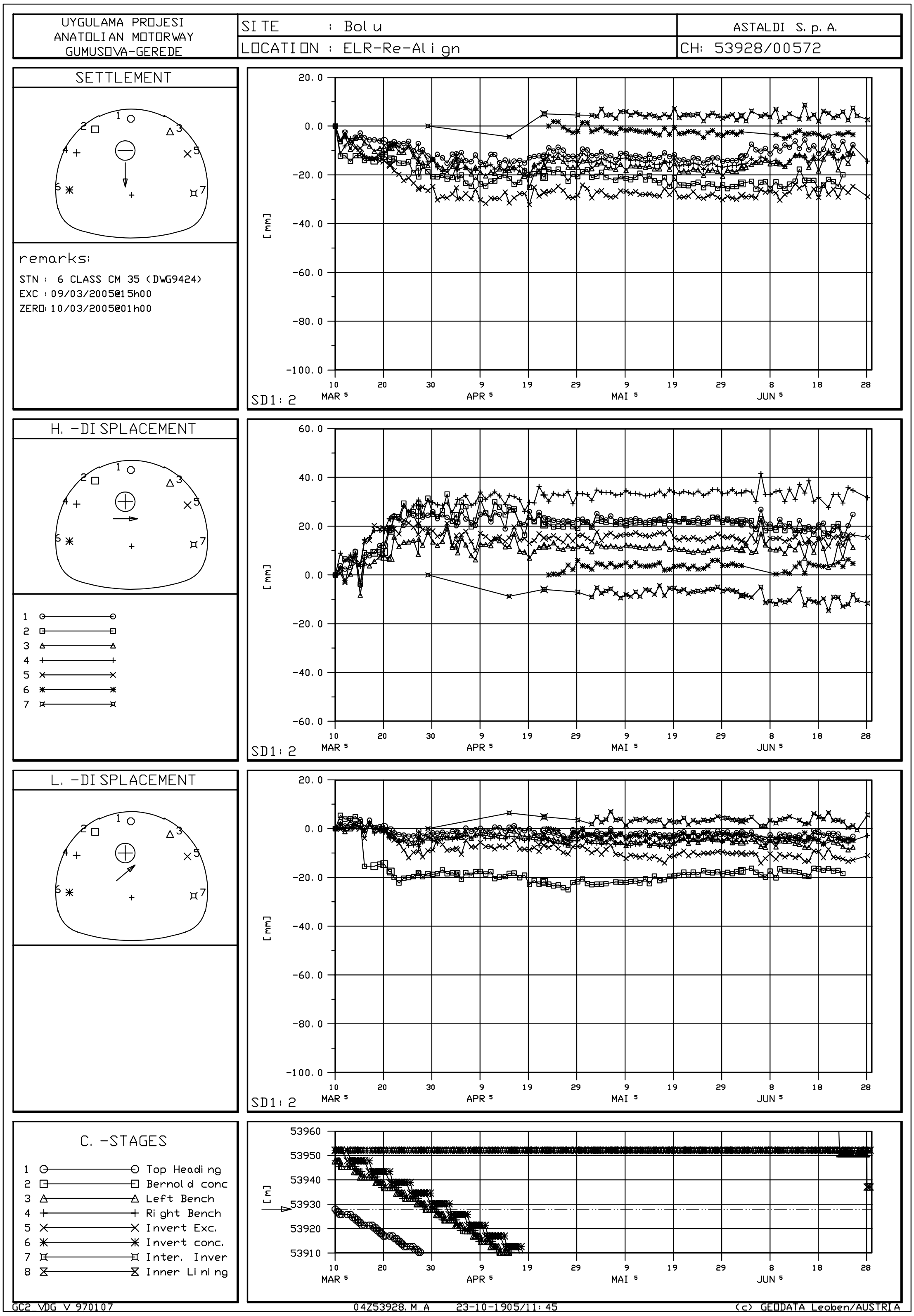






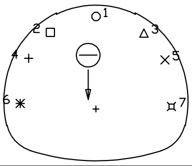


GC2_VDG V 970107 04Z53937.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELR-Re-Align
ASTALDI S. p. A.
CH: 53928/00572
SETTLEMENT
remarks:
STN : 6 CLASS CM 35 (DWG9424)
EXC : 09/03/2005@15h00
ZERO: 10/03/2005@01h00
[mm]
SD1: 2
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining
[m]
MAR 5
APR 5
MAI 5
JUN 5
GC2_VDG V 970107
04Z53928.M_A 23-10-1905/11:45
(c) GEODATA Leoben/AUSTRIA

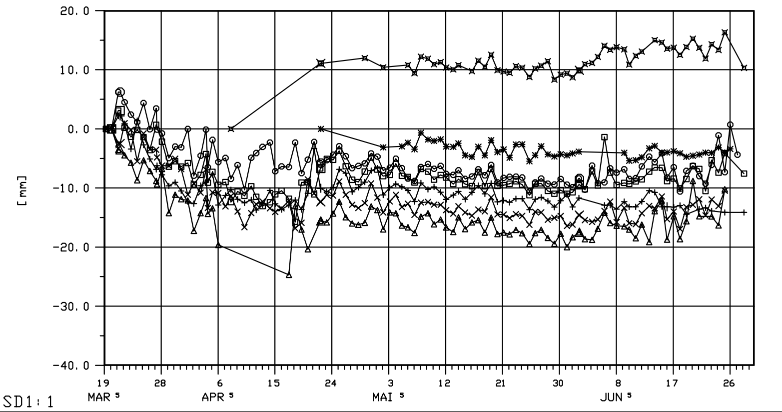
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELR-Re-Align	CH: 53920/00580

SETTLEMENT

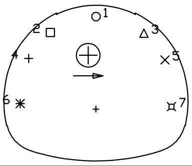


remarks:

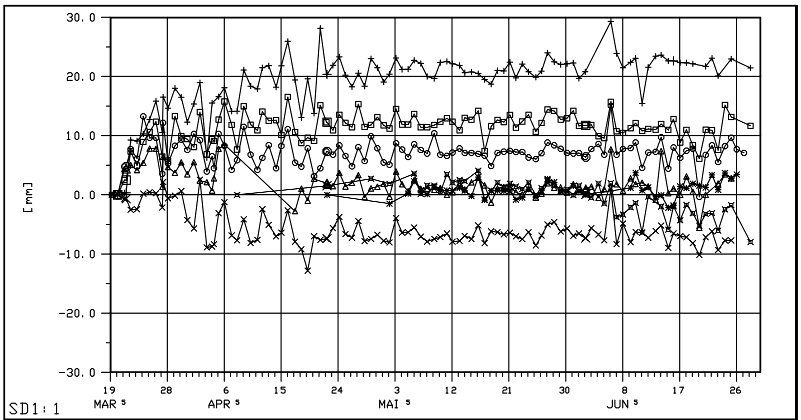
STN : 7 CLASS CM35 (DWG9424)
EXC.: 18/03/2005@17h00
ZERO: 19/09/2005@03h00



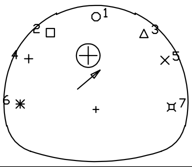
H.-DISPLACEMENT

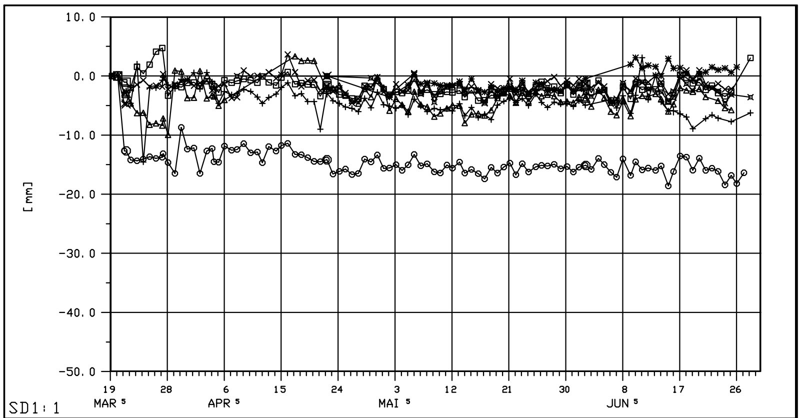


1 ○
2 □
3 △
4 +
5 ×
6 ✳
7 ⌑



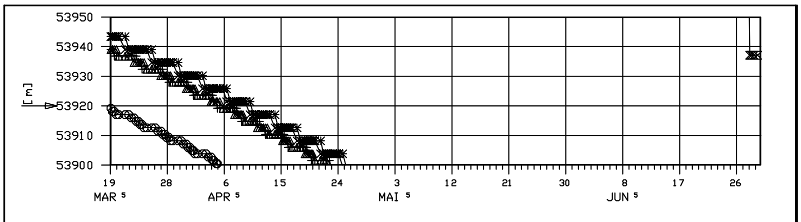
L.-DISPLACEMENT



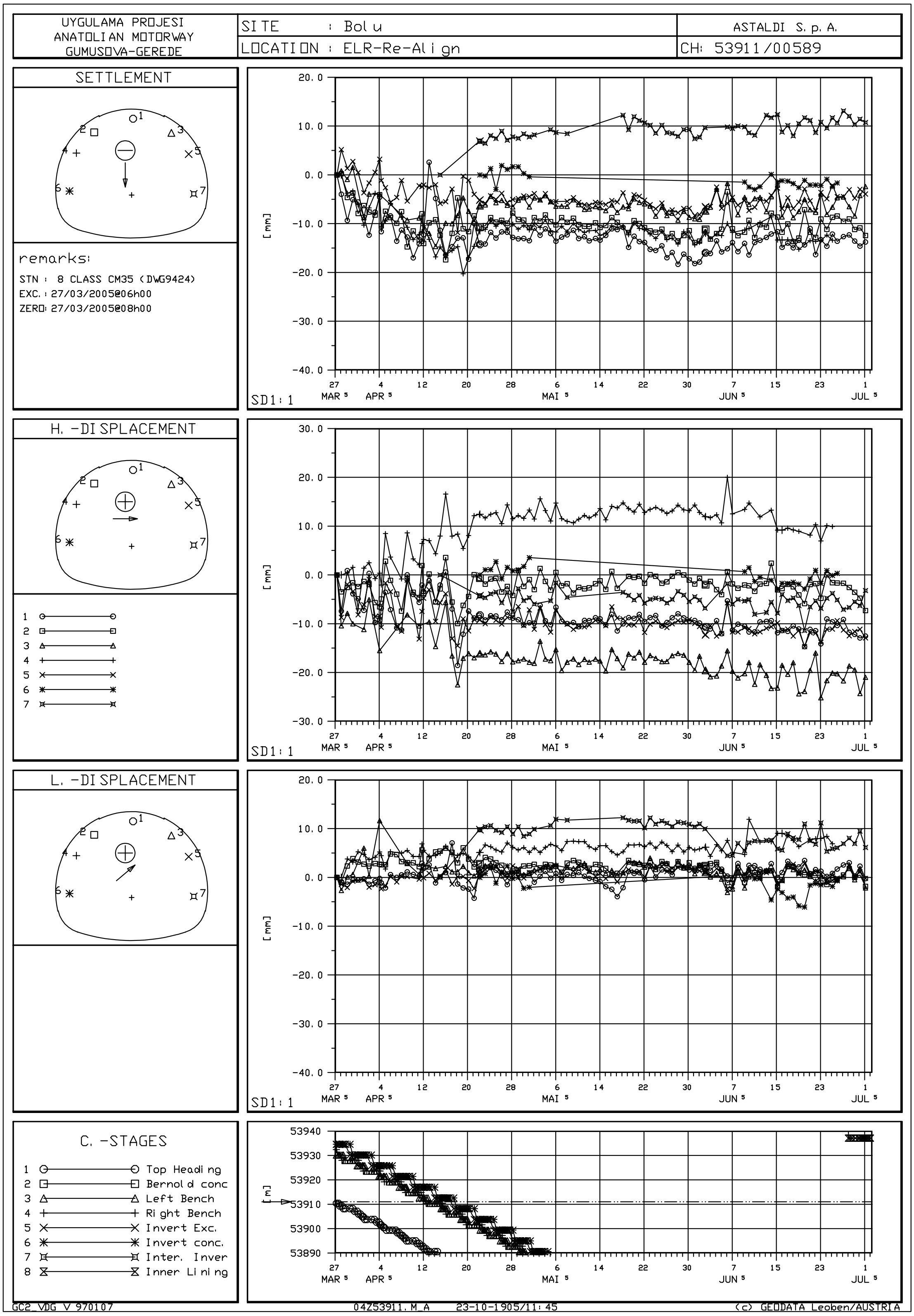


C.-STAGES

1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining

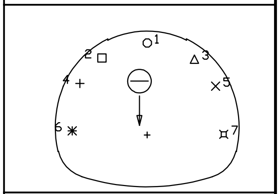


GC2_VDG V 970107 04Z53920.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELR-Re-Align
ASTALDI S. p. A.
CH: 53911/00589
SETTLEMENT
remarks:
STN : 8 CLASS CM35 (DWG9424)
EXC. : 27/03/2005@06h00
ZERO: 27/03/2005@08h00
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining
[mm]
[m]
SD1: 1
MAR 5
APR 5
MAI 5
JUN 5
JUL 5
GC2_VDG V 970107
04Z53911. M_A 23-10-1905/11: 45
(c) GEODATA Leoben/AUSTRIA

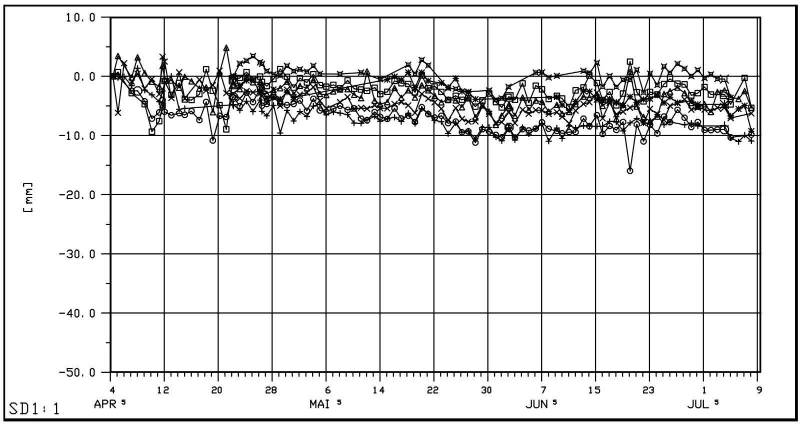
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELR-Re-Align	CH: 53902/00598

SETTLEMENT

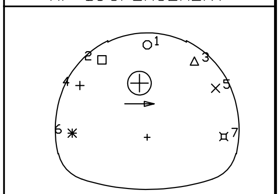


remarks:

STN: 9 CLASS CM35 (DWG 9424)
EXC: 04/04/2005@04h30
ZERO: 04/04/2005@09h00



H. -DISPLACEMENT



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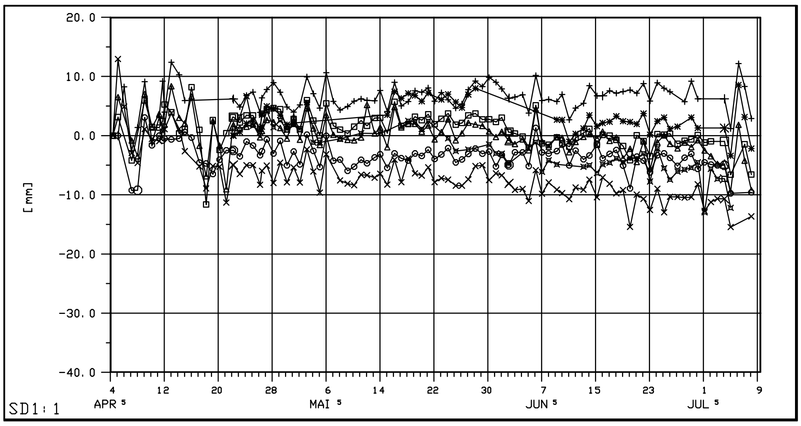
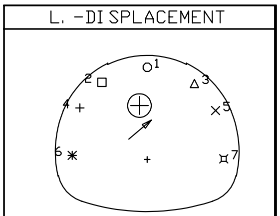
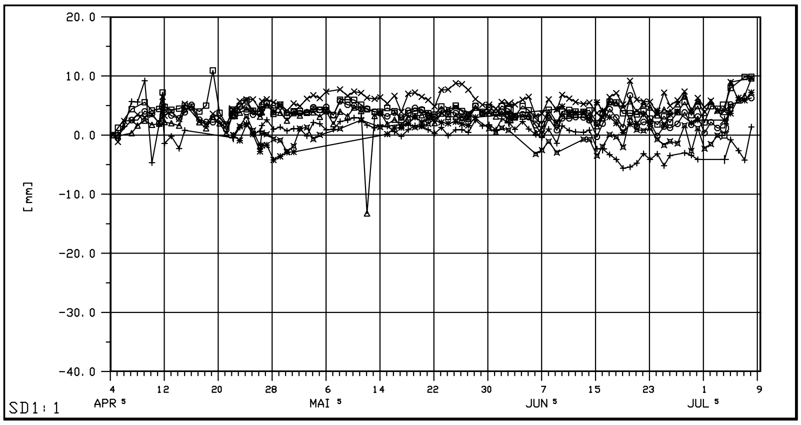


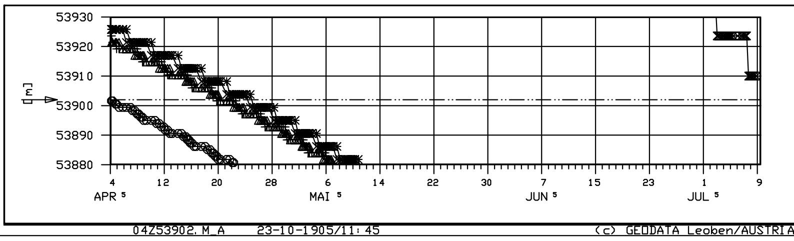
L. -DISPLACEMENT





C. -STAGES

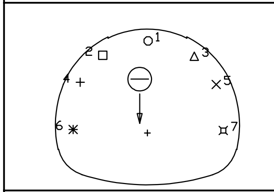
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



GC2_VDG V 970107 04Z53902.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA

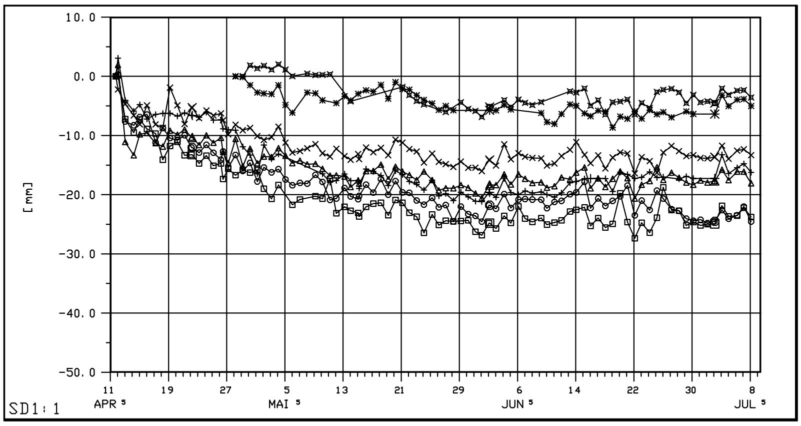
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELR-Re-Align	CH: 53893/00607

SETTLEMENT

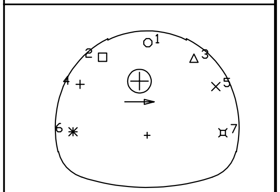


remarks:

STN : 10 CLASS CM35 (DWG9424)
EXC. : 11/04/2005@14h00
ZERO: 11/04/2005@18h00



H.-DISPLACEMENT



1 ○
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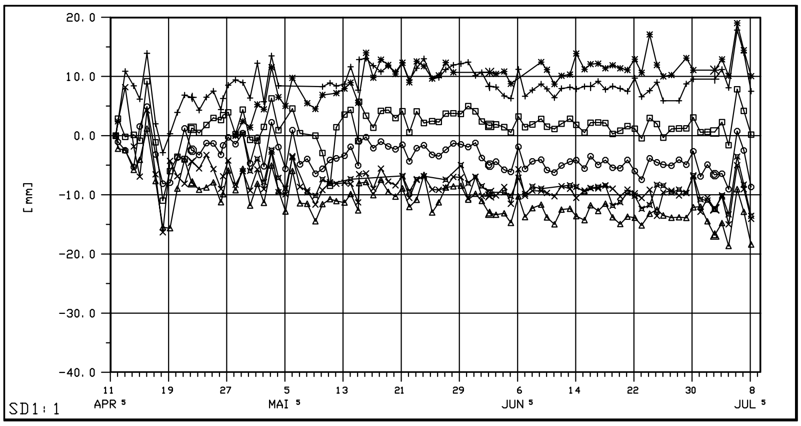
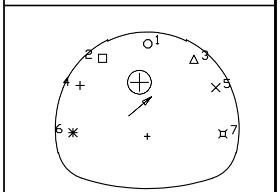
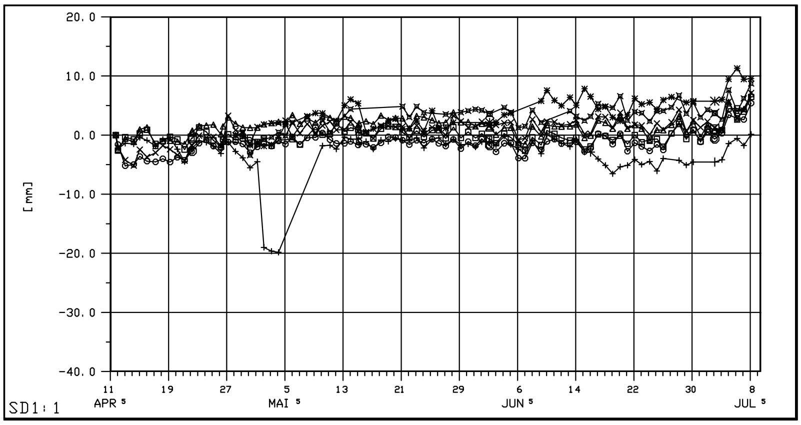
4 +
5 ×
6 ✱
7 ⌑



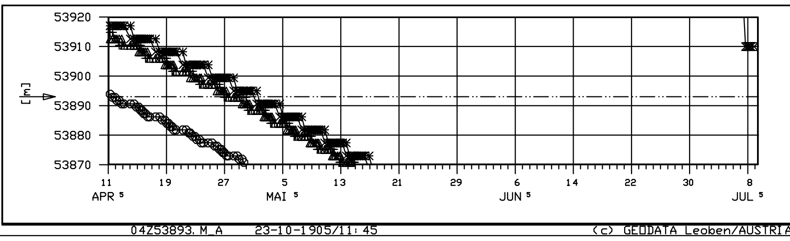
L.-DISPLACEMENT



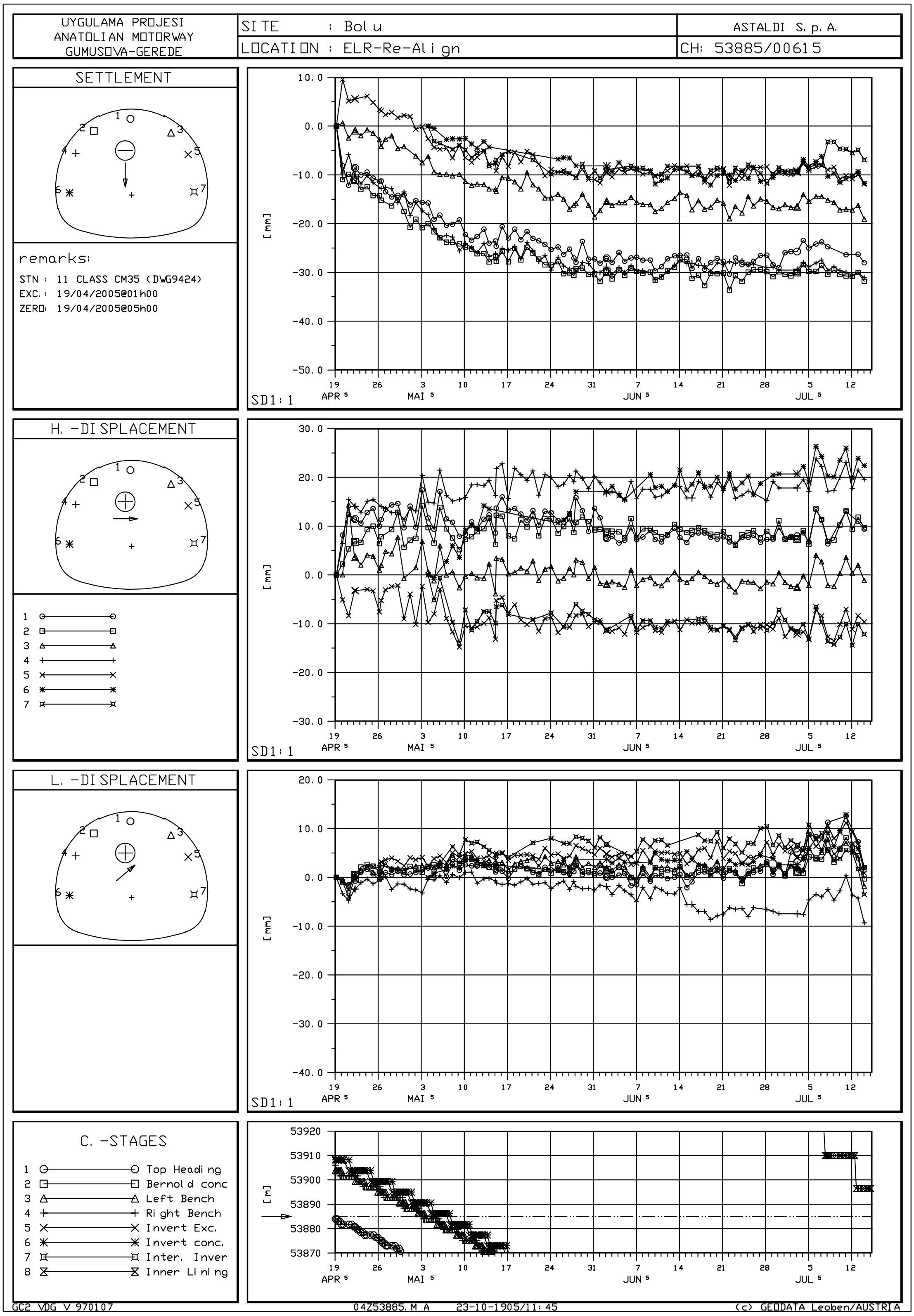


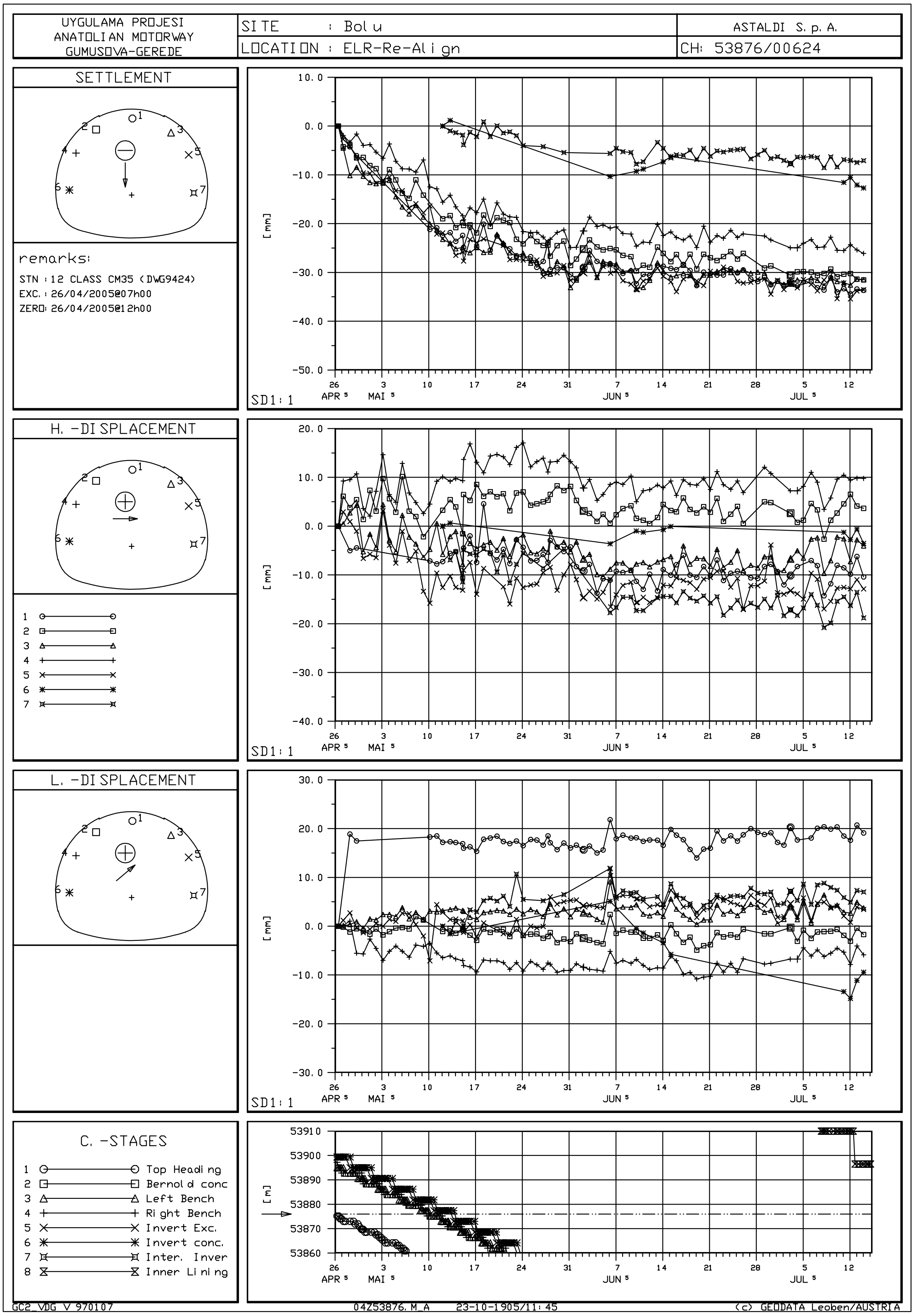
C.-STAGES

1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



GC2_VDG V 970107 04Z53893.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELR-Re-Align
ASTALDI S. p. A.
CH: 53885/00615
SETTLEMENT
remarks:
STN : 11 CLASS CM35 (DWG9424)
EXC. : 19/04/2005@01h00
ZERO: 19/04/2005@05h00
[mm]
SD1: 1
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining
[m]
APR 5
MAI 5
JUN 5
JUL 5
GC2_VDG V 970107
04Z53885.M_A
23-10-1905/11:45
(c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELR-Re-Align
ASTALDI S. p. A.
CH: 53876/00624
SETTLEMENT
remarks:
STN : 12 CLASS CM35 (DWG9424)
EXC. : 26/04/2005@07h00
ZERO: 26/04/2005@12h00
[mm]
SD1: 1
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining
[m]
26 APR 5
3 MAI 5
10
17
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7 JUN 5
14
21
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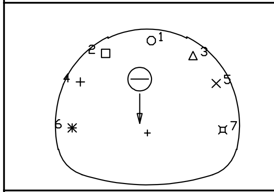
5 JUL 5
12
GC2_VDG V 970107
04Z53876. M_A 23-10-1905/11: 45
(c) GEODATA Leoben/AUSTRIA

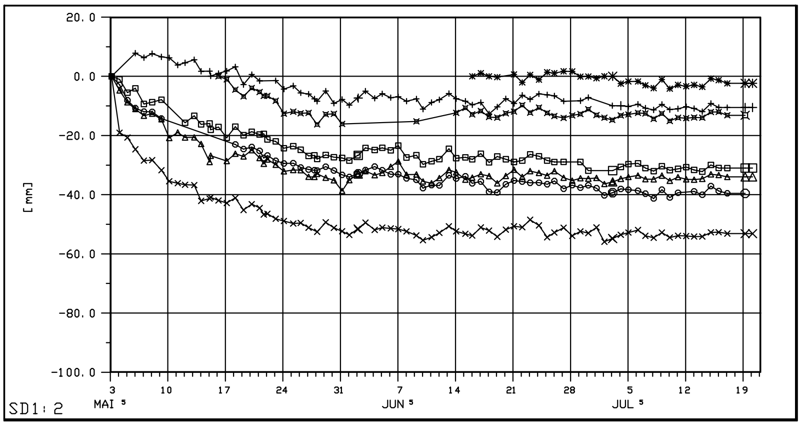
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELR-Re-Align	CH: 53867/00633

SETTLEMENT

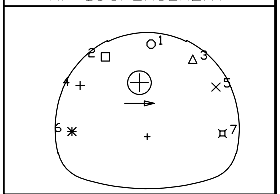


remarks:

STN : 13 CLASS CM35 (DWG9424)
EXC. : 02/05/2005@16h00
ZERO : 03/05/2005@01h00



H. -DISPLACEMENT



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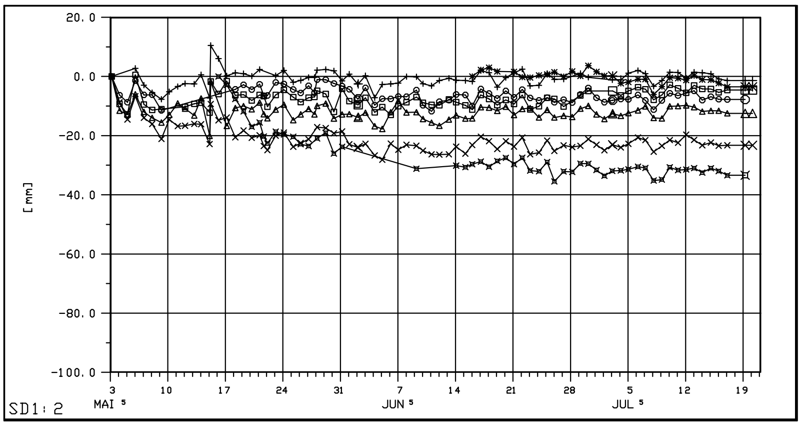
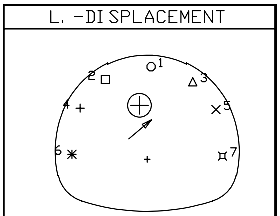
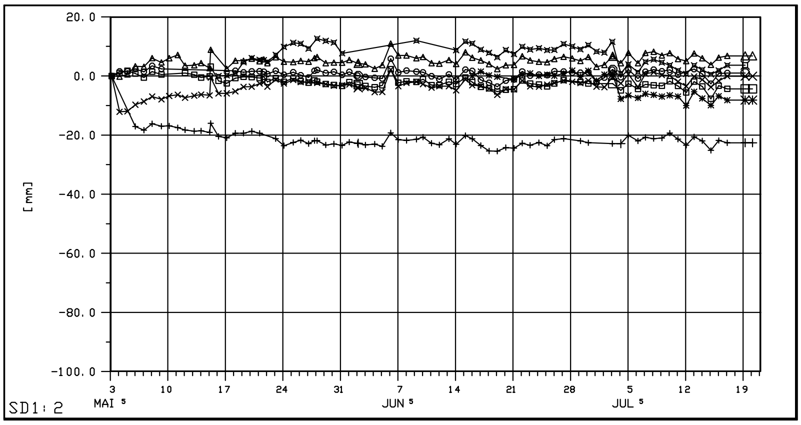


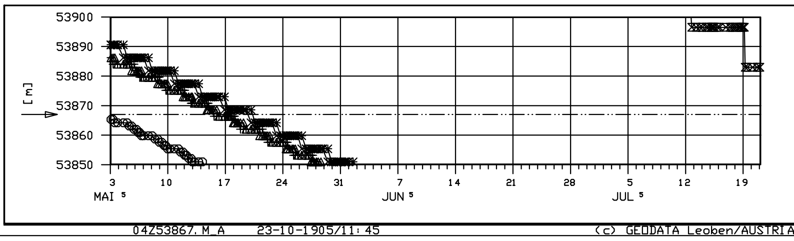
L. -DISPLACEMENT





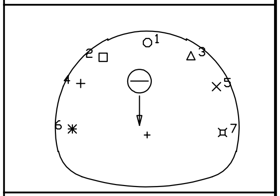
C. -STAGES

1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



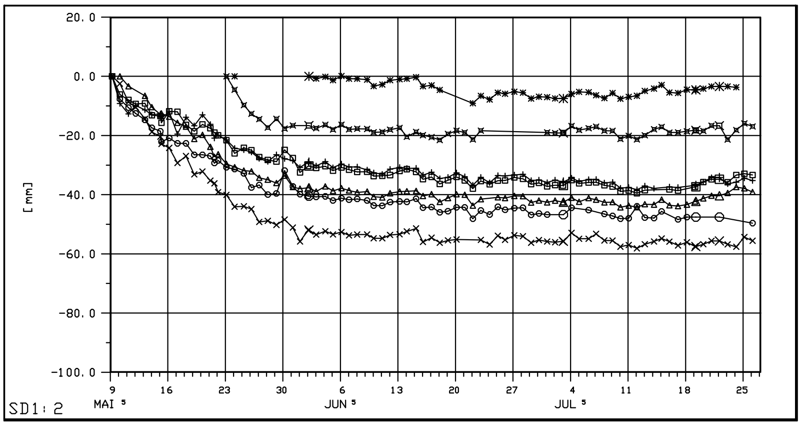
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S.p.A.
	LOCATION : ELR-Re-Align	CH: 53858/00642

SETTLEMENT

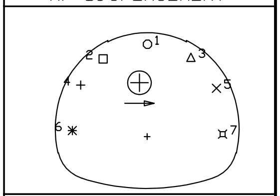


remarks:

STN : 14 CLASS CM35 (DWG9424)
EXC. : 08/05/2005@24h00
ZERO : 09/05/2005@04h00



H.-DISPLACEMENT



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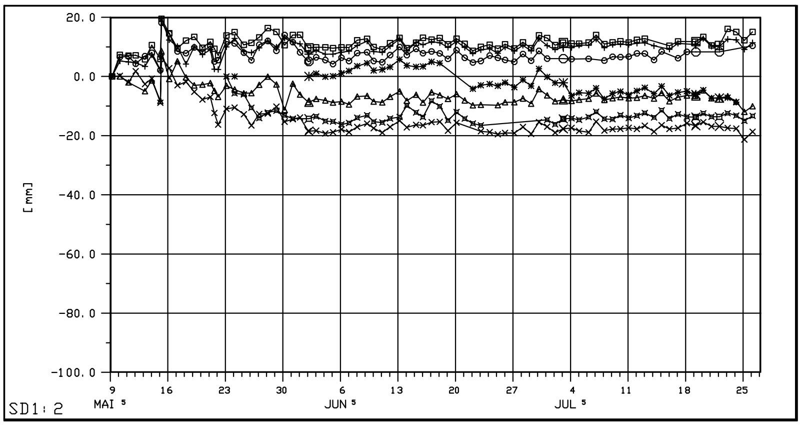
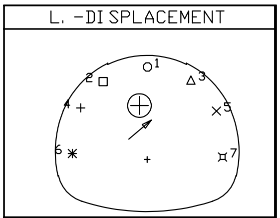
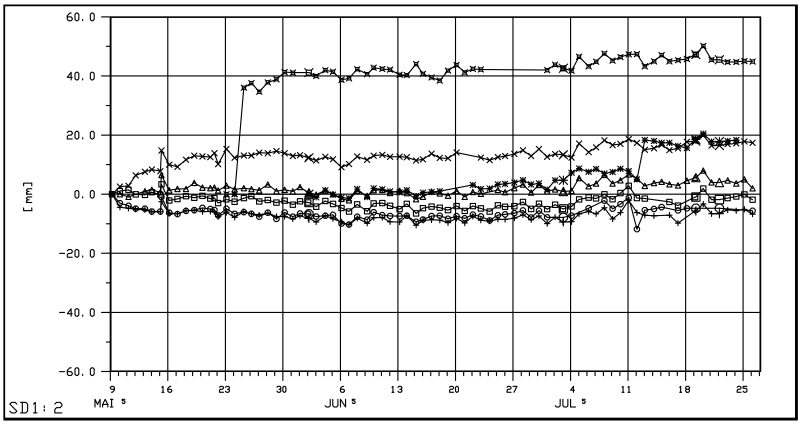


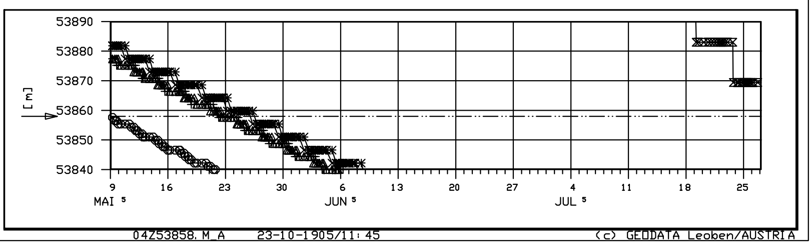
L.-DISPLACEMENT



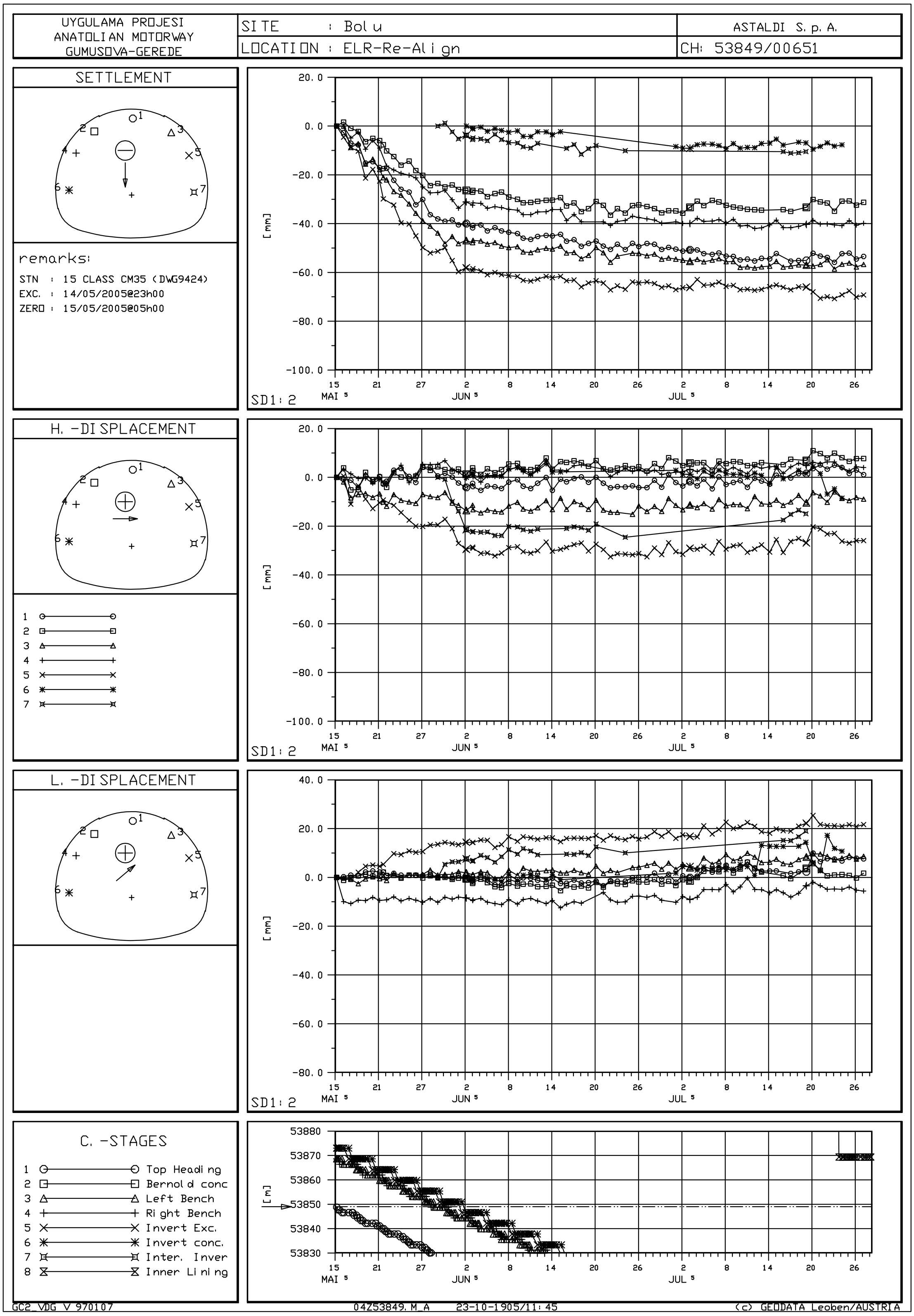


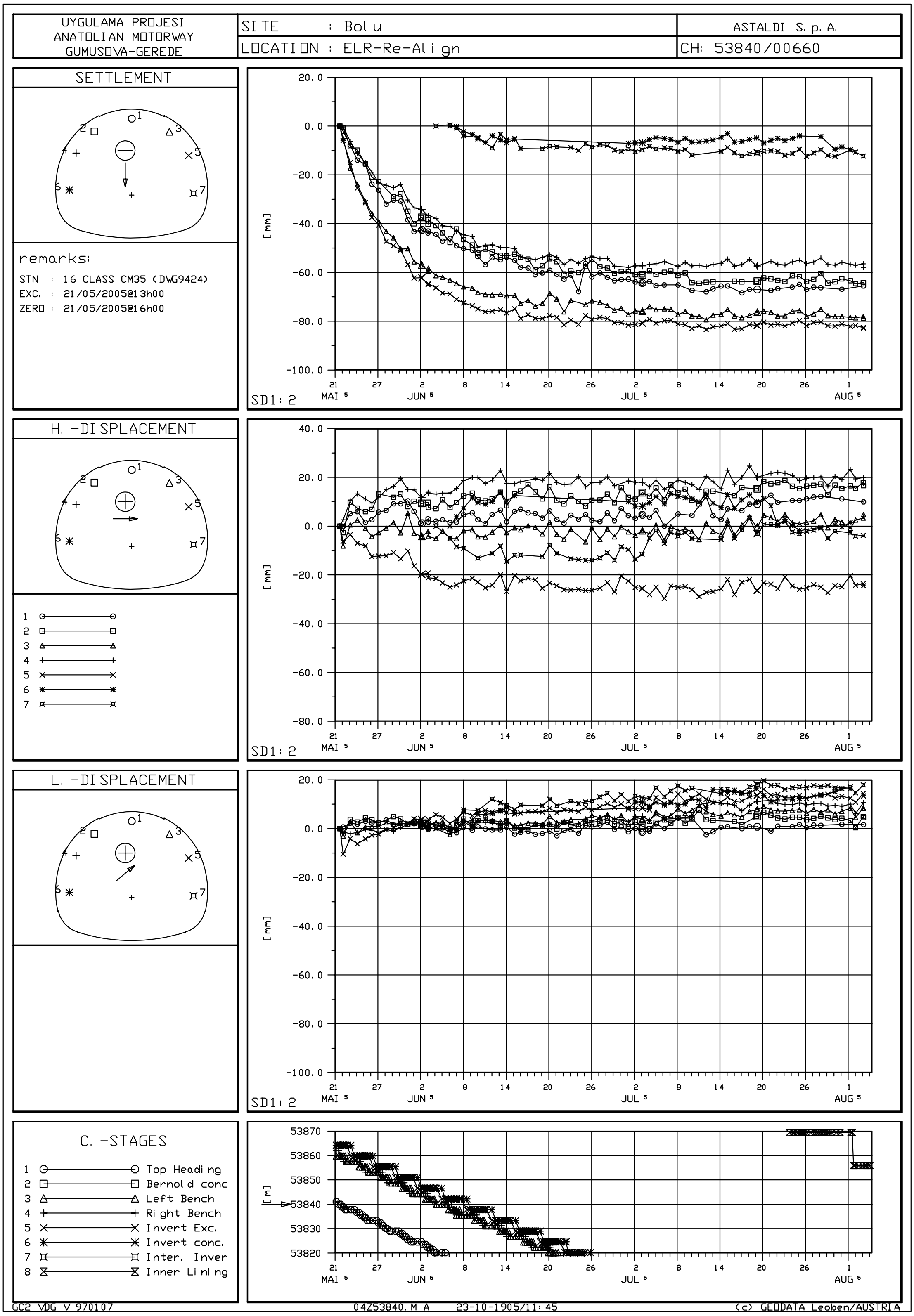
C.-STAGES

1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



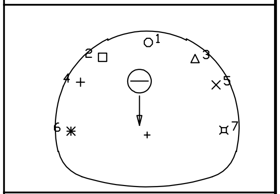
GC2_VDG V 970107 04Z53858.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELR-Re-Align
ASTALDI S. p. A.
CH: 53849/00651
SETTLEMENT
remarks:
STN : 15 CLASS CM35 (DWG9424)
EXC. : 14/05/2005@23h00
ZERO : 15/05/2005@05h00
[mm]
MAI 5
JUN 5
JUL 5
SD1: 2
H. -DISPLACEMENT
1
2
3
4
5
6
7
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining
[m]
GC2_VDG V 970107
04Z53849.M_A 23-10-1905/11:45
(c) GEODATA Leoben/AUSTRIA


UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELR-Re-Align
ASTALDI S. p. A.
CH: 53840/00660
SETTLEMENT
remarks:
STN : 16 CLASS CM35 (DWG9424)
EXC. : 21/05/2005@13h00
ZERO : 21/05/2005@16h00
[mm]
SD1: 2
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining
[m]
MAI 5
JUN 5
JUL 5
AUG 5
GC2_VDG V 970107
04Z53840. M_A 23-10-1905/11: 45
(c) GEODATA Leoben/AUSTRIA

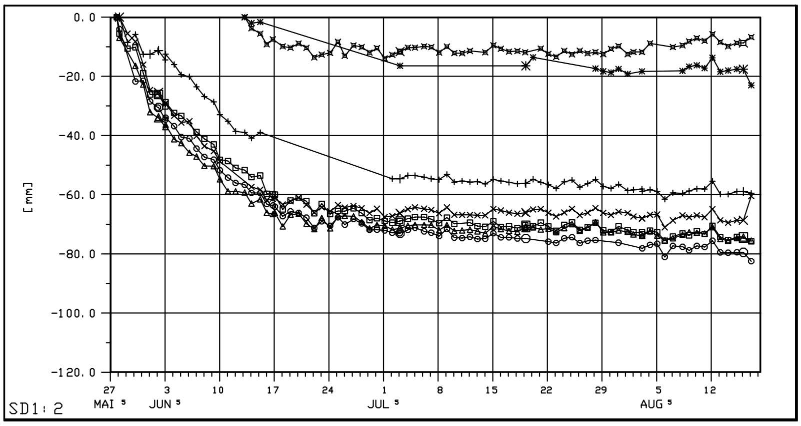
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELR-Re-Align	CH: 53831/00669

SETTLEMENT

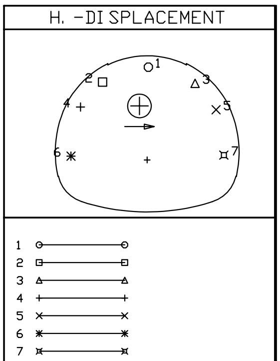


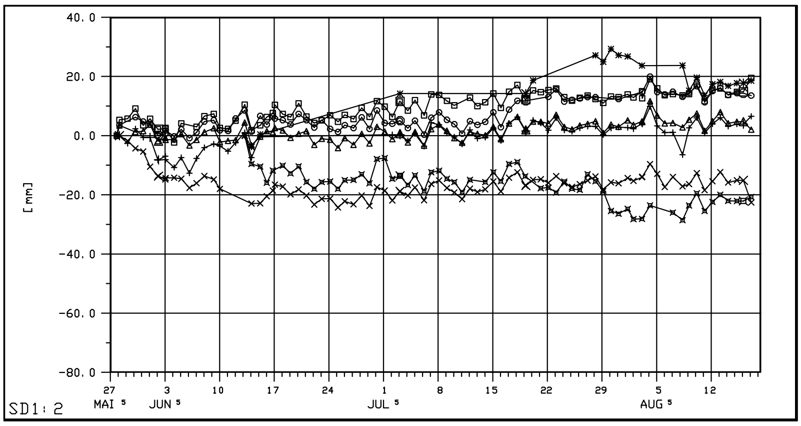
remarks:

STN : 17 CLASS CM45 (DWG300)
EXC. : 27/05/2005@16h00
ZERO : 27/05/2005@20h00

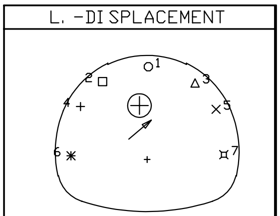


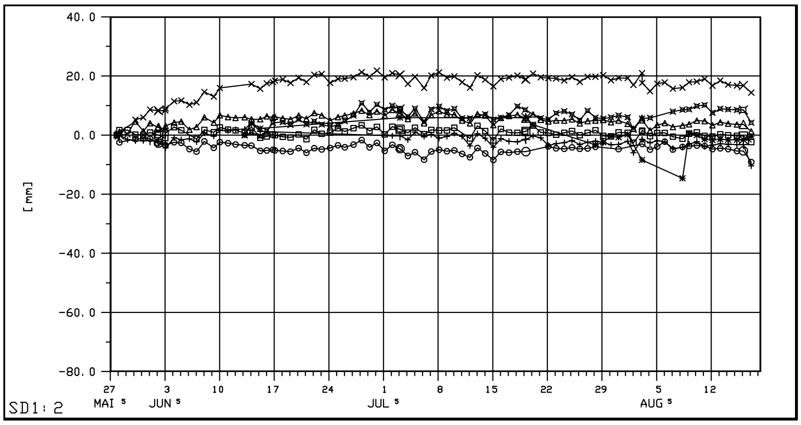
H.-DISPLACEMENT





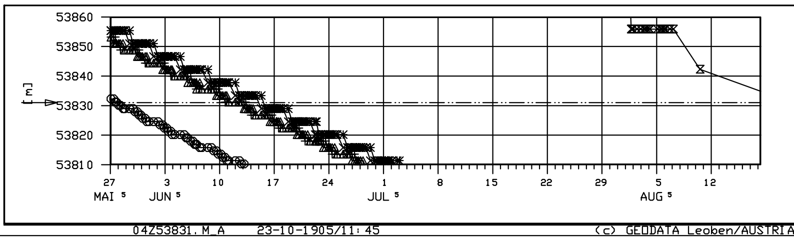
L.-DISPLACEMENT





C.-STAGES

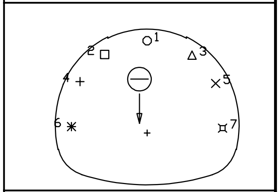
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



GC2_VDG V 970107 04Z53831.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA

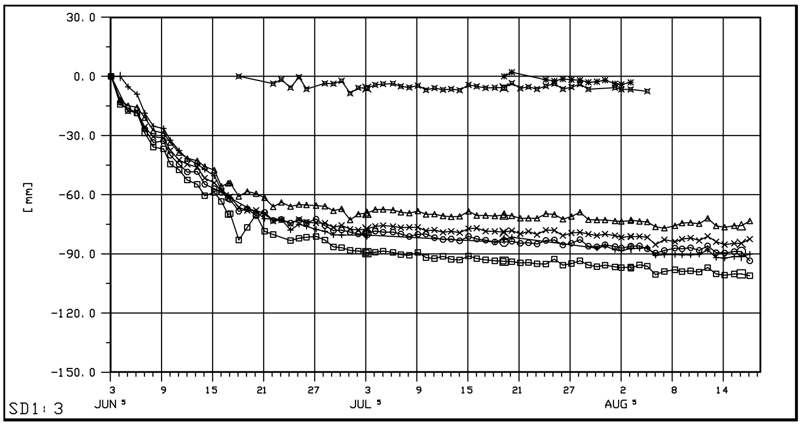
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELR-Re-Align	ASTALDI S. p. A. CH: 53823/00677

SETTLEMENT

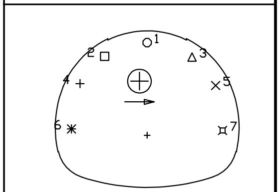


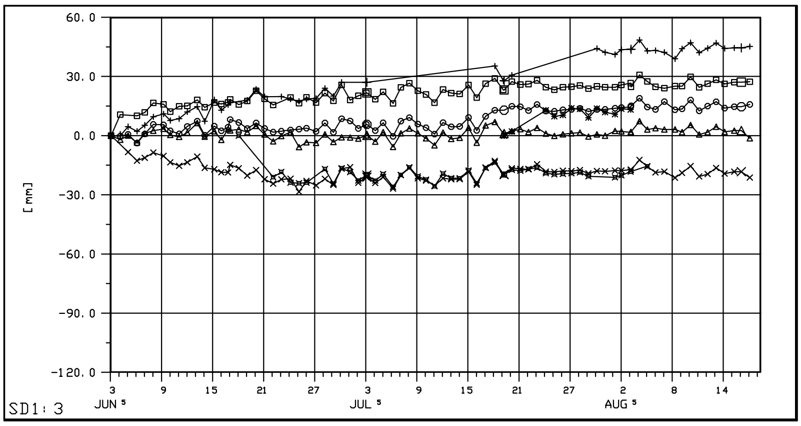
remarks:

STN : 18 CLASS CM45 (DWG300)
EXC. : 02/06/2005@22h00
ZERO: 03/06/2005@02h00

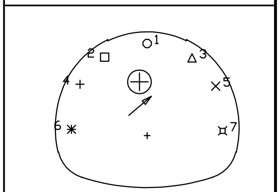


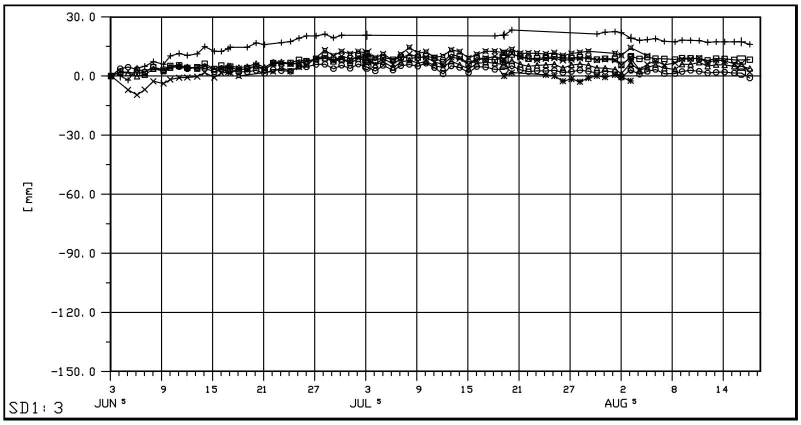
H. -DISPLACEMENT





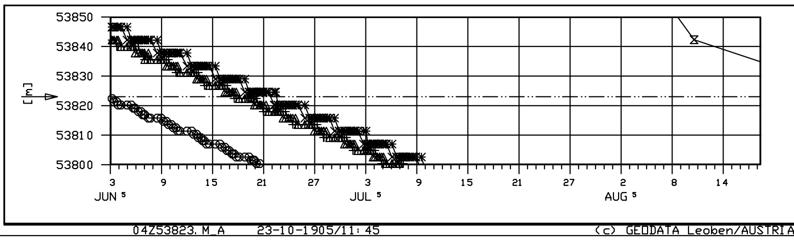
L. -DISPLACEMENT





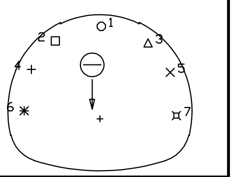
C. -STAGES

1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



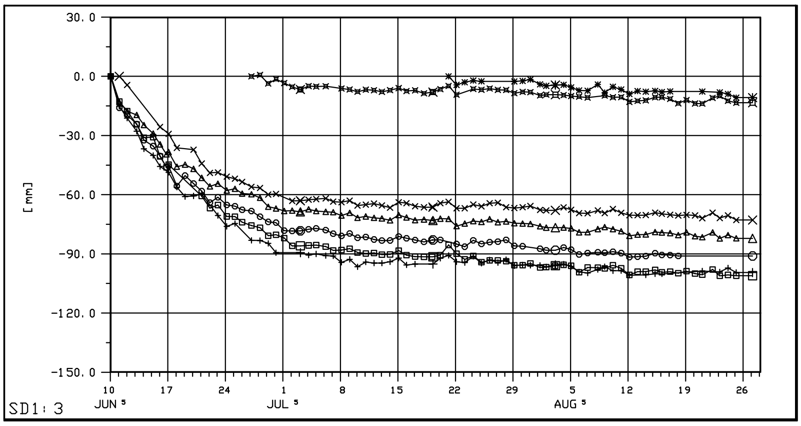
UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu LOCATION : ELR-Re-Align	ASTALDI S.p.A. CH: 53814/00686

SETTLEMENT

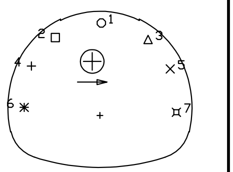


remarks:

STN : 19 CLASS CM45 (DWG300)
EXC. : 09/06/2005@21h00
ZERO : 10/06/2005@01h00



H.-DISPLACEMENT



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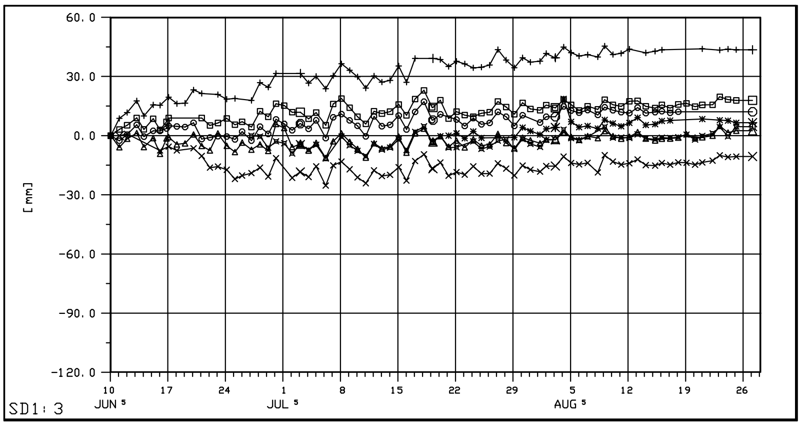
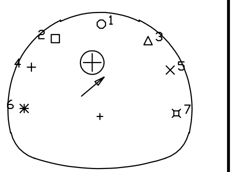
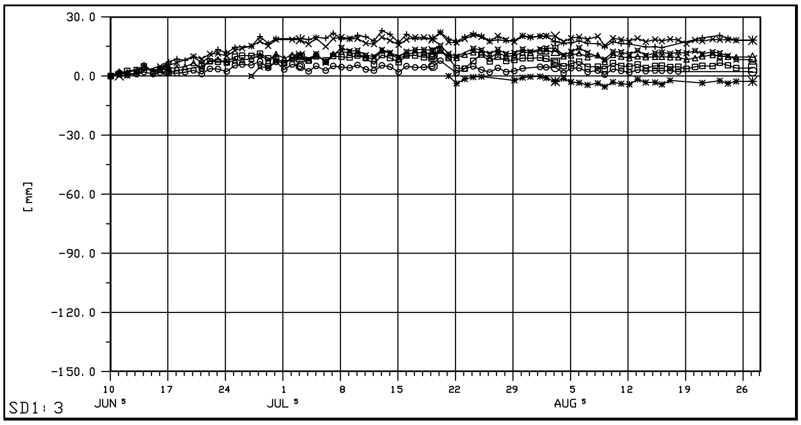


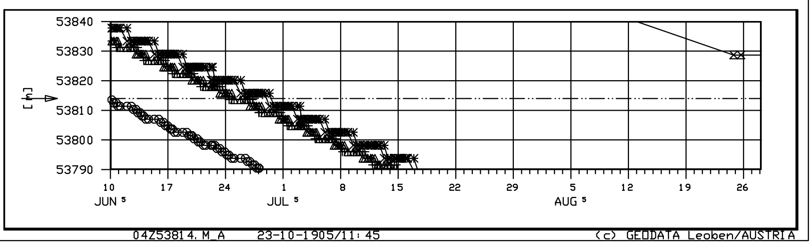
L.-DISPLACEMENT





C.-STAGES

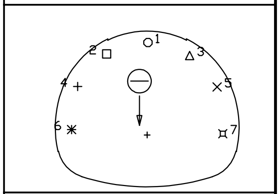
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining



GC2_VDG V 970107 04Z53814.M_A 23-10-1905/11:45 (c) GEODATA Leoben/AUSTRIA

UYGULAMA PROJESI ANATOLIAN MOTORWAY GUMUSOVA-GEREDE	SITE : Bolu	ASTALDI S. p. A.
	LOCATION : ELR-Re-Align	CH: 53805/00695

SETTLEMENT



remarks:

STN : 20 CLASS CM45 (DWG300)
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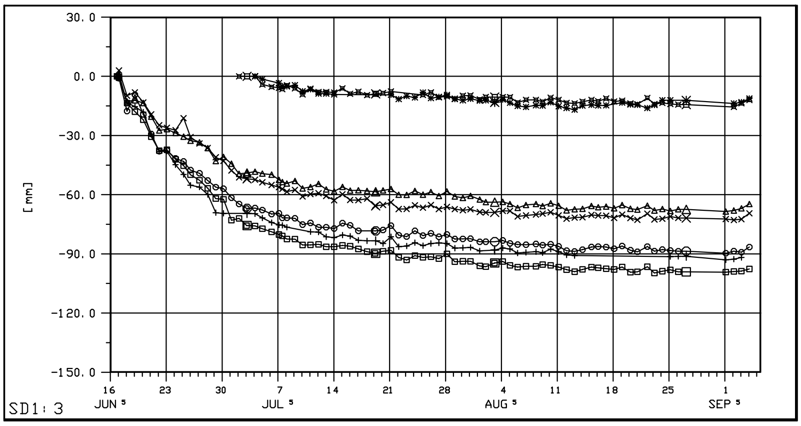
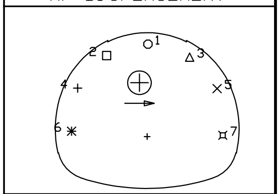
ZERO : 16/06/2005@20h00



SD1: 3

H. -DISPLACEMENT



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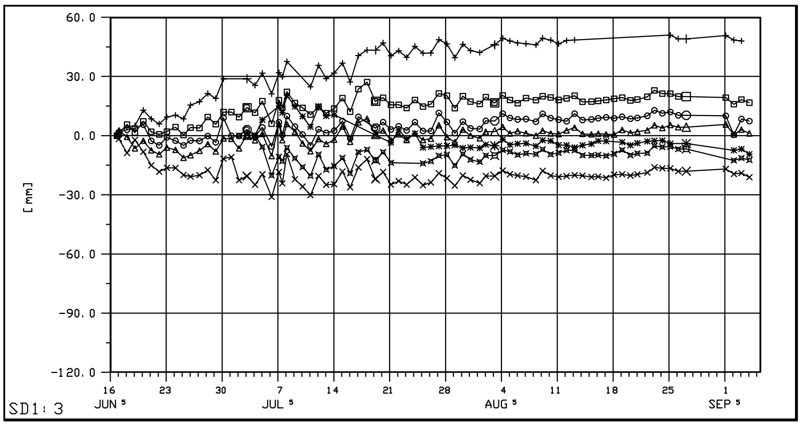
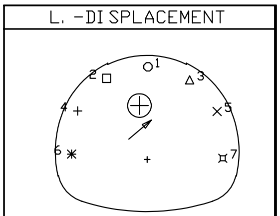
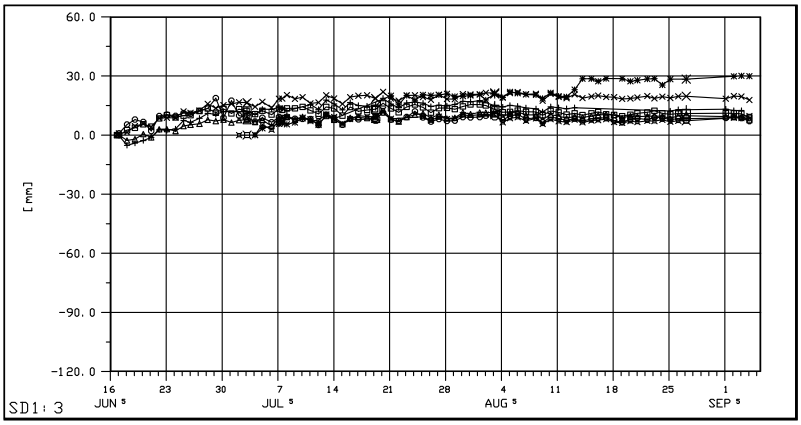


SD1: 3

L. -DISPLACEMENT

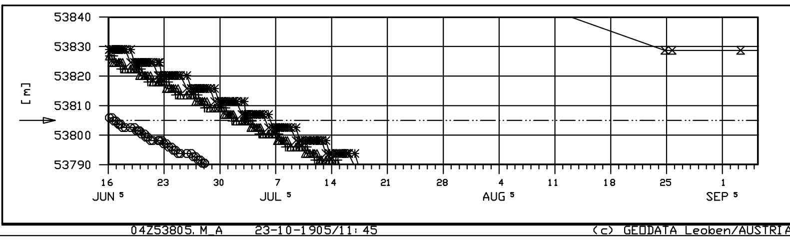


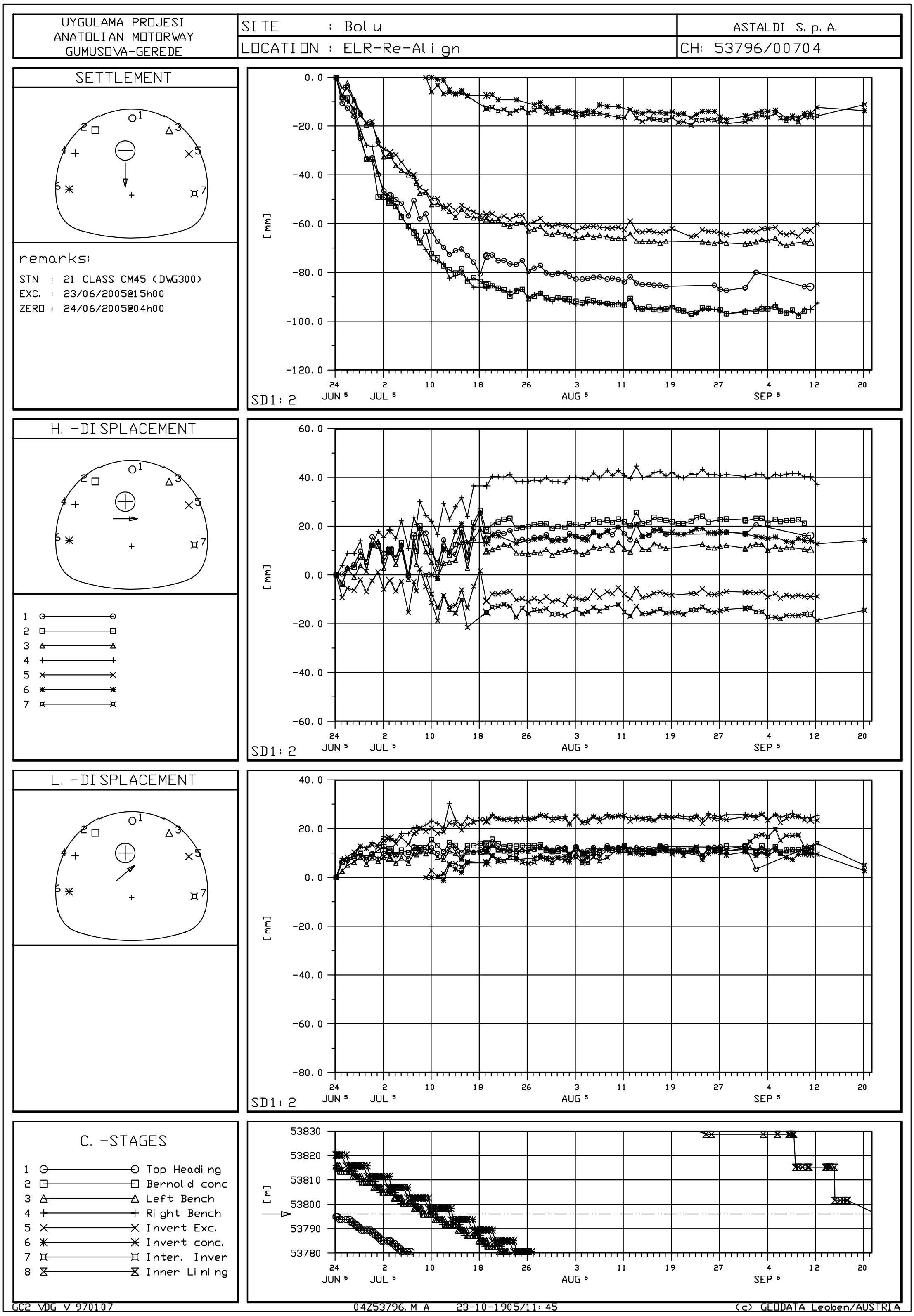


SD1: 3

C. -STAGES

1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining





UYGULAMA PROJESI
ANATOLIAN MOTORWAY
GUMUSOVA-GEREDE
SITE : Bolu
LOCATION : ELR-Re-Align
ASTALDI S. p. A.
CH: 53796/00704
SETTLEMENT
remarks:
STN : 21 CLASS CM45 (DWG300)
EXC. : 23/06/2005@15h00
ZERO : 24/06/2005@04h00
H. -DISPLACEMENT
L. -DISPLACEMENT
C. -STAGES
1 Top Heading
2 Bernold conc
3 Left Bench
4 Right Bench
5 Invert Exc.
6 Invert conc.
7 Inter. Inver
8 Inner Lining
[mm]
[m]
SD1: 2
24 JUN 5
2 JUL 5
10
18
26
3 AUG 5
11
19
27
4 SEP 5
12
20
GC2_VDG V 970107
04Z53796. M_A 23-10-1905/11: 45
(c) GEODATA Leoben/AUSTRIA