Risks and responsibilities of geotechnics in highway-, bridge-, and slope engineering

Risque et responsabilité en géotechnique pour les travaux routiers, les ponts et les pentes

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ABSTRACT: Professional experience has shown again and again at numerous engineering projects that the geotechnical engineer's risk and responsibility are usually much higher than that of engineers involved in other fields of construction. This is demonstrated with some case histories in highway- and bridge engineering which also show the risks caused by insufficient site investigation and unstable slopes. The first two examples illustrate special geotechnical measures to save severely damaged highway bridges which were near collapse. From a static point of view, their safety factors were temporarily about F = 1. For buildings in unstable slopes or for the stabilization of sliding slopes, the semi-empirical design method based on monitoring and calculated risk (observational method) is recommended. For risk assessment, the residual shear strength has proved to be the most relevant geotechnical parameter. If Φ_r has to be assumed also as design parameter, comprehensive foundation and stabilizing measures are necessary as illustrated in this paper.

1 INSUFFICIENT SITE INVESTIGATION

1.1 *Project, ground properties*

The design for a 1100 m long highway bridge was changed shortly before construction work began. The alternative was a politically supported project which met certain environmental demands, and there was no time left for appropriate site investigation. Consequently, the soil (rock) behaviour at the new bridge pier positions had to be more or less interpolated. Detailed additional soil investigation was postponed until the beginning of the foundation work: The bridge piers were designed to rest on caissons, and when sinking the shafts, the subsoil characteristics were supposed to be clearly determined. The foundations should have been adapted to the local ground conditions by a proper excavation depth, by forming toe-bells of the caissons, or by grouting beneath the bases. Short exploration borings beneath its designed base should verify a proper bearing capacity for each caisson. Actually, these recommended detailed soil investigations were only partially performed, and an extremely heterogeneous underground finally caused excessive differential settlements of one twin pier.

The critical bridge pier is situated within a large-scale geological fault on toe of a slope where the soil conditions are extremely heterogeneous. Figure 1 shows a simplified sketch derived from exploration borings. In detail, weathered slope deposits (from silt to boulder) and fluviatile sandy gravel (silty) are interlayered near the surface, and they are locally underlain by clayey silt (red loam). This more compressible soil occurs rather irregularly, forming a significant eccentric lens just beneath the bridge pier. Beneath these deposits lie fine-grained mylonites, carstic zones, and completely decomposed „rock" (mechanically a „soil), and finally a more or less weathered phyllite. The mean ground water level lies some metres below the surface but varies greatly, depending on the seasons.

Laboratory tests confirmed the assumption that a lens of soft loam beneath the existing caissons caused the differential settlements. A highly compressible silt exhibited a residual shear angle of only $\Phi_r = 8°$.

The superstructure of the highway bridge was designed as a continuous girder with 9 spans and constructed by the free cantilevering method. The standard length of the spans is 130 m, and the cross section is a hollow box with a structural height of 3,0 to 7,8 m (prestressed reinforced concrete). The supports are either fixed or movable bearings. The bridge piers consist of two I-shaped members resting on two caissons each. Consequently, each bridge support, except the abutments, required the sinking of four caissons (Fig. 2).

The static system of the caisson group (4 caissons each), the twin piers, and the capping beam on top of the piers make up a frame which is rather sensitive towards differential settlements.

1.2 *Cause of failure*

During free cantilevering, one of the bridge supports began to settle more and more differentially (Fig. 2) : As the movements approached - and finally exceeded - the allowable limits considerably, the construction work was stopped. Eleven sections of the 130 m-span had been cast, but an ongoing free cantilevering was too risky (though only one section was left). In order to stop the differential movements, the site manager tried local underpinning by classical grouting (Fig. 1/measure 1). But already the drilling of some boreholes caused additional differential settlements of the critical pier footings No. 1R and 1L, and this tendency could not be stopped by any available underpinning or supporting method. Jet grouting or piling would have initially caused further settlements before stabilizing the movements. For static reasons this risk could not be taken: The structural safety of the frame had already decreased seriously and could not be proven any more by (classical) calculation. The maximum differential settlement had reached nearly Δs = 40 mm, whereas the „allowable" value was Δs = 10 mm for the originally assumed statically indeterminate frame system (Fig. 3). Increasing stress constraints, especially torsion, endangered the structure which had not yet failed: Its static system had changed automatically from an indeterminate one to a less sensitive, hinged frame when the foundation began to move (settlement and tilting) - Fig. 4. Furthermore, the creeping of the fresh concrete reduced the critical stress constraints, and the static system proved to be practically more flexible than theoretically assumed, because no capping slab existed on the head of the caisson group. Finally, the superstructure was not rigidly connected with this twin bridge pier. The loads were transferred by bearings which could withstand the differential displacements. Nevertheless, the free cantilevering equipment was completely removed and the hitherto constructed superstructure excentrically ballasted to unload the stronger settling foundation pier (Fig. 1/measure 2).

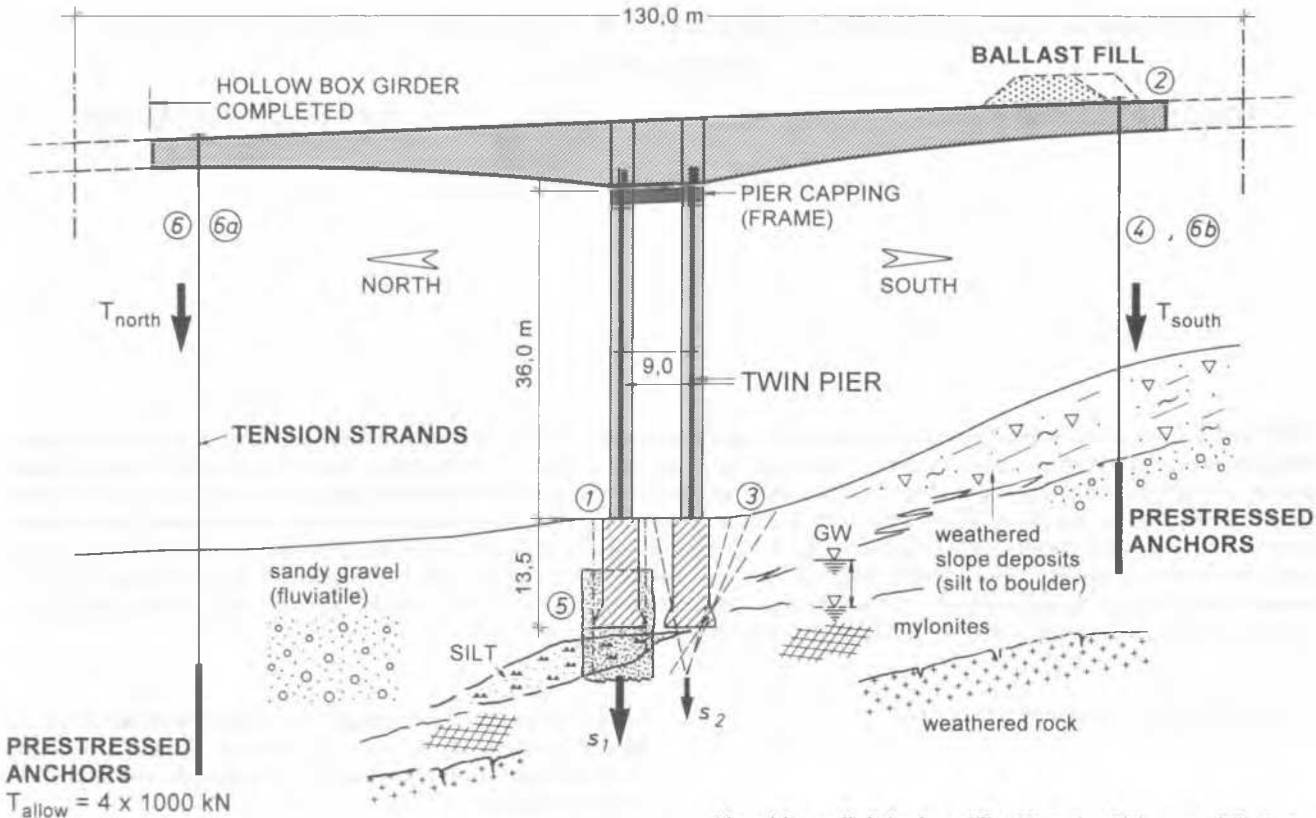


Fig. 1: Part of the longitudinal section of a 1.1 km long highway bridge. Differential settlement of the pier foundation (indicated by $s_1 > s_2$), and stabilizing measures:

1 = Conventional grouting (caused critical additional settlements during drilling)
2 = Eccentric loading of the superstructure by a temporary fill to reduce differential settlements
3 = Relief borings: drillings with step by step soil extraction beneath the southern foundation to force an increasing settlement of caissons 2R, 2L
4 = Pulling down the superstructure to reduce differential settlements
5 = Underpinning and partial re-levelling by soil fracturing
6 = Pulling down the superstructure on the other side to preload the underpinned pier foundation
6a, 6b = Cyclic loading and unloading by an alternating pulling down according to measure No. 4 or 6.

From a static point of view, the pier was in an initial state of collapsing. The reason that it did not give in, although the differential settlement had exceeded nearly four times the allowable value, was a favourable soil-structure interaction in the base zone of the caissons. Figure 4 shows that obviously a movable, hinged embedment must have developed instead of a rigid, fixed socket. Nevertheless, the bridge designer refused any further activity, and the insurance company cancelled the contract with the construction company. The demolition and reconstruction of the structure would have cost nearly 5 million US $ and would have heavily damaged the image of the civil engineering profession. So, geotechnics was challenged to save the project and take over the entire responsibility.

1.3 *Stabilization and rehabilitation*

An underpinning of the stronger settling caissons was too risky. Therefore, the differential settlements could be reduced only by forcing the other member of the twin-pier to settle more. This was achieved by „relief"-borings (∅ 140 mm) which were drilled step by step beneath the caissons No. 2 R and 2 L for local soil excavation (Fig. 5 and Fig. 1/ measure 3). After withdrawing the casing, the boreholes collapsed partially - thus inducing a controlled settlement. Only thin sleeve pipes were left in the boreholes to enable a later grouting as a permanent soil improvement. In total 45 relief borings were drilled to a maximum depth of 6 m beneath the caissons' base (Fig. 5). The local movements should occur gradually without a sudden collapse. This was achieved by varying the position, inclination, and depth of the boreholes and by also varying the drilling speed. Sometimes weak grout mixes were filled into the boreholes as a flexible support. The local soil excavation already reduced the maximum differential settlements of the caissons by 6 mm.

The re-tilting effect of the artificial voids in the subsoil beneath the caissons No. 2 R and 2 L could be intensified by pulling down the superstructure of the bridge excentrically with prestressed ground anchors (stabilizing measure No. 4 in Figs 1, 2). Four anchors were installed with an allowable working load of T_{allow} = 1000 kN, but only stressed by $T_w \leq 550$ kN each. These forces could be applied only after installing an additional (temporary) reinforcement within the bridge deck. As the last section of the free cantilevering had not yet been mounted, there was sufficient place for an auxiliary reinforcing of the hollow box girder. Due to the long distance of the anchors from the bridge pier, a great moment was transferred into the caisson group. It caused an unloading of the excessively settled caissons by ΔV = -6450 kN each and an additional loading of the opposite twin caissons by ΔV = +7750 kN each. Accordingly, the „stiffer" members of the twin-pier were forced to settle more.

During re-tilting of the caisson group (and the bridge structure respectively), the anchor forces had to be continuously readjusted according to their tendency to decrease.

The combined effects of these measures (No. 3 and 4 in Fig. 1) reduced the differential settlements nearly by one third to about $\Delta s \leq 30$ mm, and they led to a significant back-tilting of the leaning bridge pier. Consequently, the improved situation made a partial re-levelling and a definite underpinning with the soil fracturing method possible.

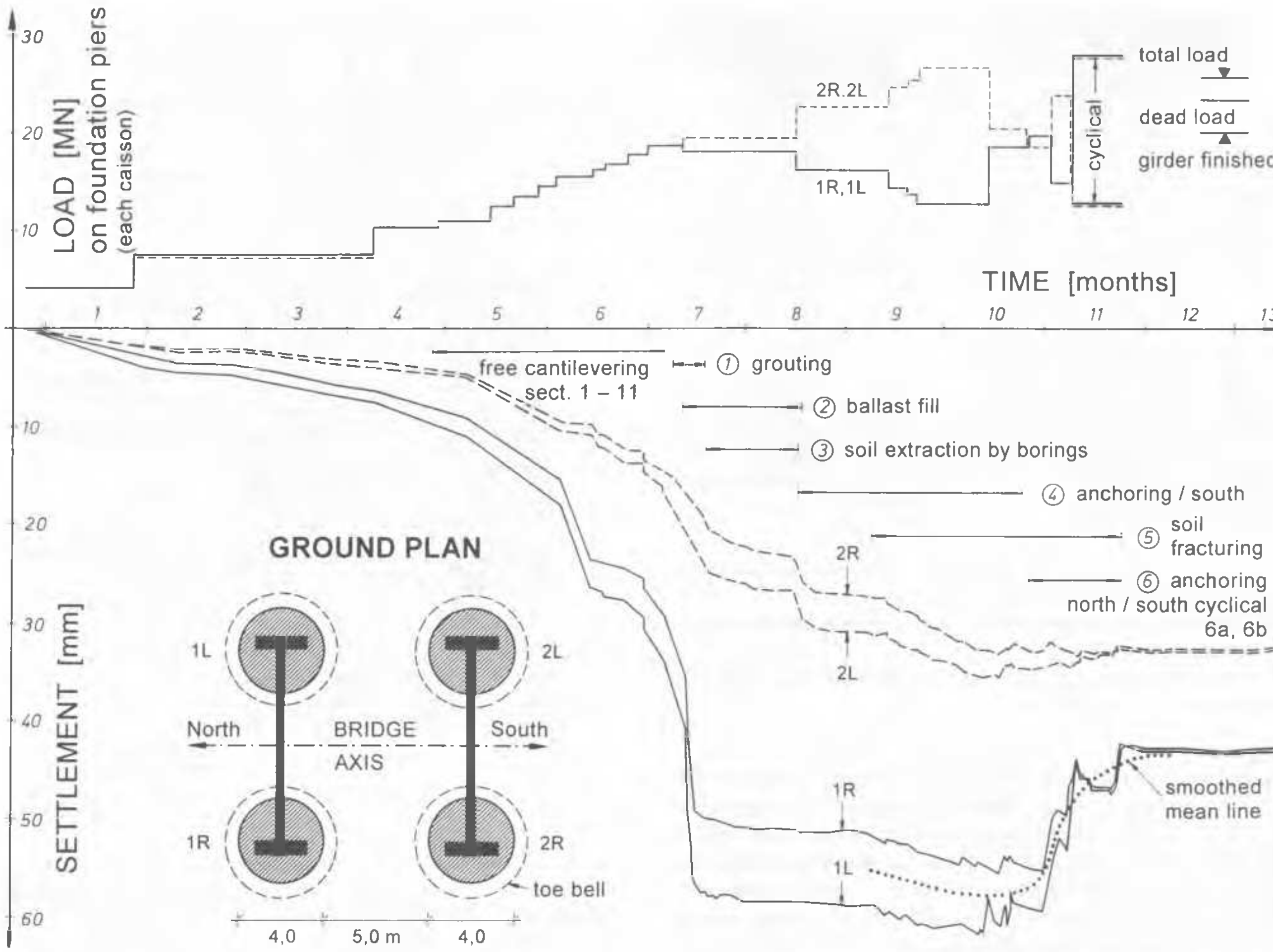


Fig. 2: Load-time and load-settlement curves of the four caissons. Construction and stabilizing measures (No. 1 to 6 and 6a/6b resp.).

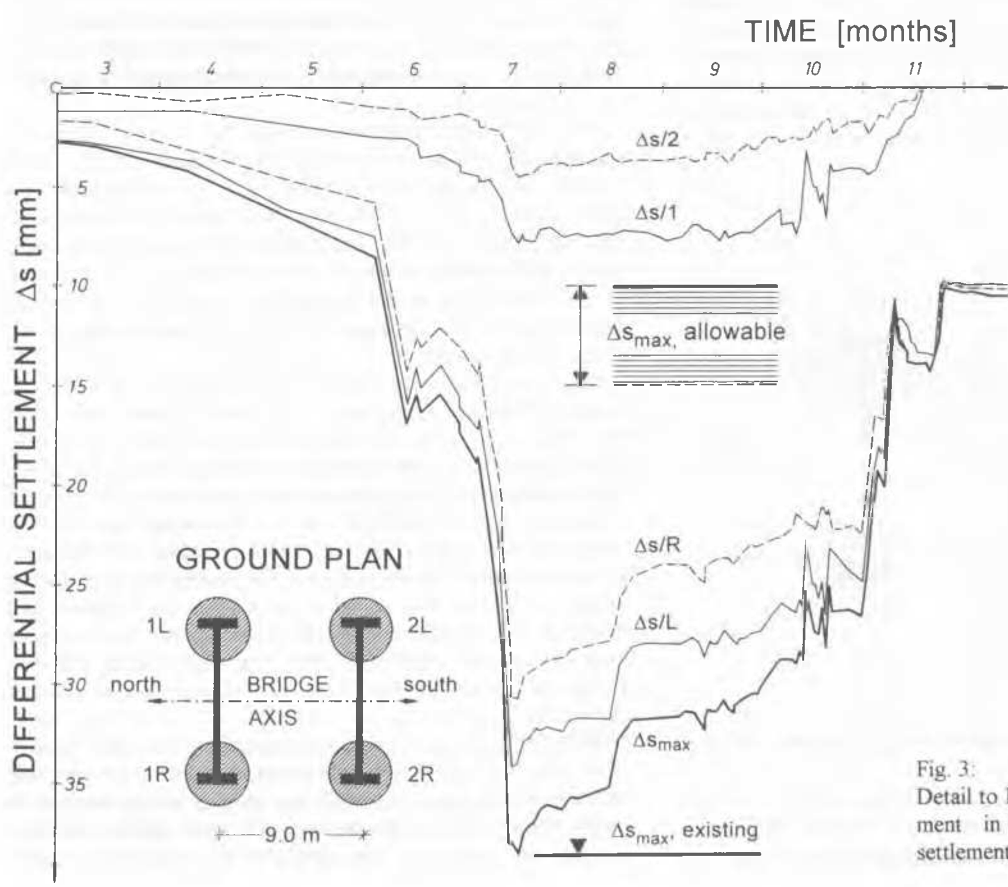


Fig. 3:
Detail to Fig. 2, showing the development in time of the differential settlements among the four caissons.

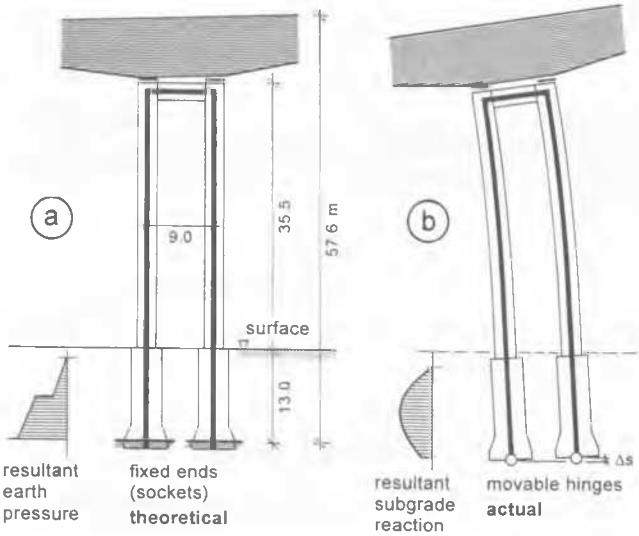


Fig. 4: Static system of the endangered twin-pier and horizontal sub-grade reaction (schematic).
a) Theoretical design assumption: Statically indeterminate frame with fixed sockets.
b) Actual situation due to large soil deformations: Statically determinate two-hinged frame

Soil fracturing was applied only beneath the pier foundation No. 1R and 1L (Figs. 1, 2, 5). Figure 6 illustrates the scheme of the soil improvement sequence and the stress state in the subsoil caused by repeated grouting. A primary grouting around the lower part of the pier shafts was performed at first to achieve a strengthened cover layer for controlled hydro-fracturing. After a turned-over pot of strengthened soil was formed, high pressure grouting followed beneath the base of the caissons (phase No. 4 in Fig. 6). Thus, the caissons were lifted like a piston. For splitting the soil, a grouting pressure of 30 bars was used. Subsequent grouting was carried out with 30 to more than 60 bars. In total, 42 grouting boreholes were drilled as indicated in Figure 5, and they reached 8 m beneath the caissons' bases (i.e. 21 m below working level). Within three months, about 215 m^3 of grouting mix was injected, requiring 864 m run of sleeve pipes (d = 2") with 0.5 m spacing of the sleeve valves.

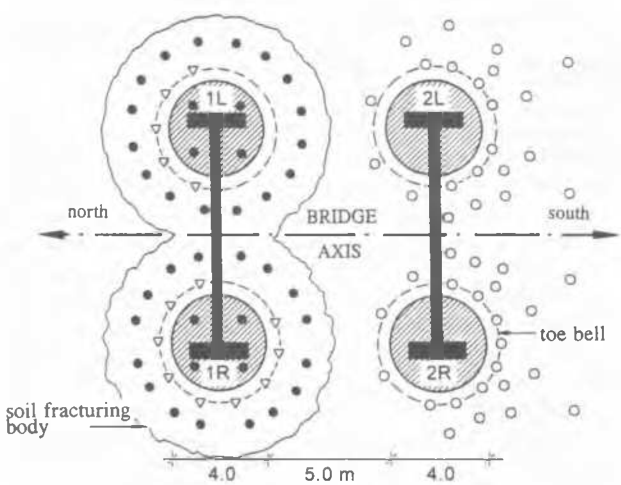


Fig. 5: Ground plan of the boreholes drilled during the stabilizing measures No. 1,3, and 5 (see Fig. 1):
- ∇ first trial grouting (classical), i.e. measure No. 1.
- o relief borings with soil extraction (measure No. 3).
- • grout borings for soil fracturing (measure No.5).

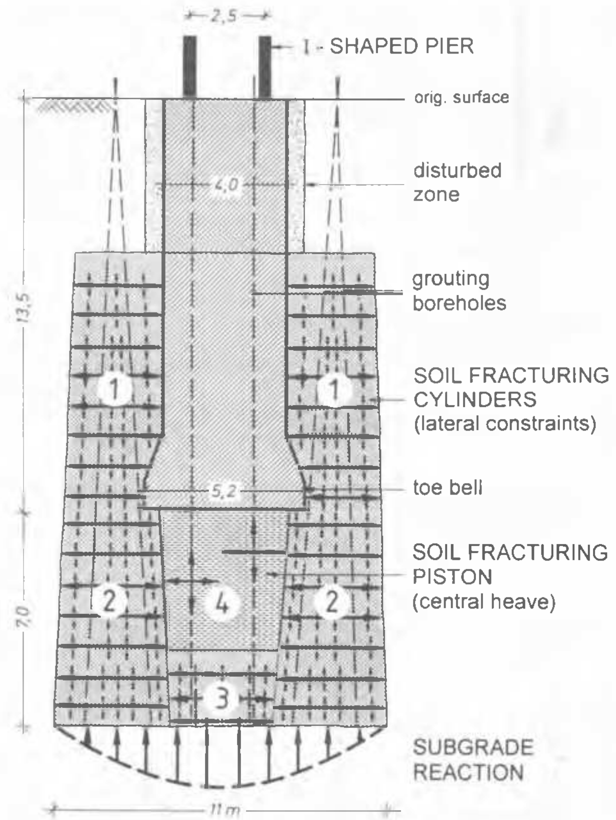


Fig. 6: Scheme for re-lifting the inclined bridge pier by means of soil fracturing in steps (phases 1 to 4). Model of the stress state in the subsoil caused by repeated grouting.

Soil fracturing provided a lifting of the northern caissons even under base pressures of σ_v = 900 to 1350 kN/m^2. The maximum σ_v values resulted from cyclical loading. In order to achieve a deep and intensive soil improvement by compaction/compression, soil fracturing was also executed under full anchor forces T_w (measures 6a and 6b in Fig. 1).

The maximum settlement decreased by 20 mm. The main objective was to reach the allowable differential settlement of $\Delta s \leq$ 10 mm, and this was fully achieved (Figs 2,3). Accordingly, the bridge bearings on top of the piers had neither to be exchanged nor to even be re-levelled. The maximum horizontal displacement on top of the leaning bridge pier decreased from Δx = 150 mm to Δx = 45 mm (only in the longitudinal direction). This residual deviation could be compensated by the free cantilevering of the bridge superstructure.

After completion of the stabilizing measures No. 1 to 5 (Fig. 1), further differential settlements could not be ruled out when increasing the loads by casting the last section of the superstructure and after opening the highway bridge for heavy traffic. Furthermore, a load test seemed to be suitable to check the degree of soil improvement by soil fracturing especially in connection with seismic activities which are possible in this area. These aims could be achieved by a cyclic loading and unloading of the bridge piers by prestressed anchors, when the superstructure still acted as a statically determinate balance beam. The hysteresis loops were run till a quasi-stationary state was reached, and they anticipated further settlements under full load and in case of earthquake.

According to Figure 1, four vertical soil anchors with $T_{allow.}$ = 1000 kN, T_w = 550 kN each were installed on either side (south/north) of the twin pier. These anchors were connected by tensile strands with the bridge deck. The load test for the lifted caissons No. 1R and 1L was performed by pulling the northern

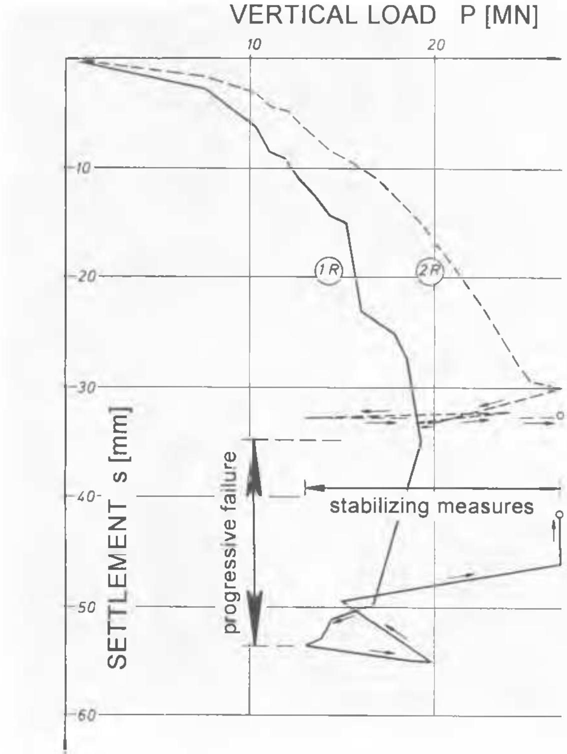


Fig. 7: Load-settlement behaviour of the right caissons No. 1R and 2R (see Fig. 2): Influence of stabilizing measures clearly visible.

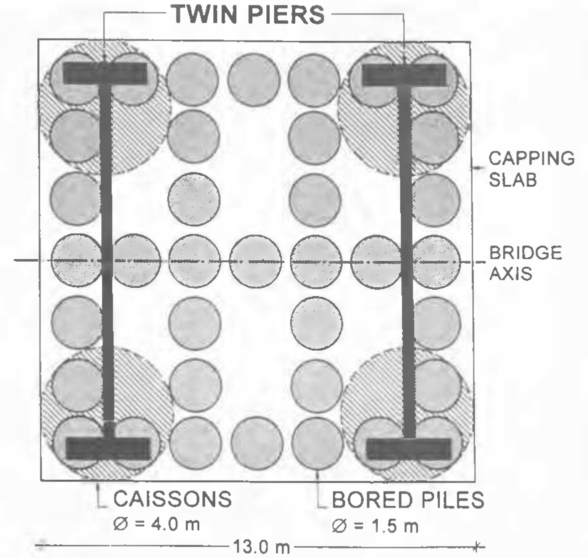


Fig. 8: Ground plan of the foundation of another twin pier of the 1100m long highway bridge.
1st design: 4 caissons.
2nd design: box-shaped pile foundation

members (measure 6a in Fig. 1) Afterwards the superstructure (fitted with proper auxiliary reinforcement) was pulled down cyclically by activating either the southern or the northern tension equipment: 6b or 6a in Figure 1, whereby the measure No. 6b corresponds to the previous measure No. 4. The range of the total vertical load varied between V_{min} = 24.5 MN to V_{max} = 54.5 MN for the northern or southern pair of caissons (in the longitudinal direction of the bridge). Thus, the load oscillation was $\Delta V \leq 15$ MN for each caisson (Fig. 2), corresponding to a variation in the base stress of $\Delta\sigma$ = 750 kN/m^2 (± 375 kN/m^2 respectively). The maximum value exceeded the future maximum total load (after bridge completion and traffic opening) significantly, so that further settlements were already anticipated before closing the free cantilevering elements to a statically sensitive continuous girder.

After cyclical loading, the soil around the shaft of the caissons No. 1R and 1L was also grouted near the surface to increase skin friction (and lateral earth resistance) which obviously had got lost previously as a result of soil loosening during the sinking of the shafts.

Figure 7 shows the load settlement curves of the right pair of the caisson indicating an approaching ground failure before stabilization work began. Especially in case of caisson No. 1R the

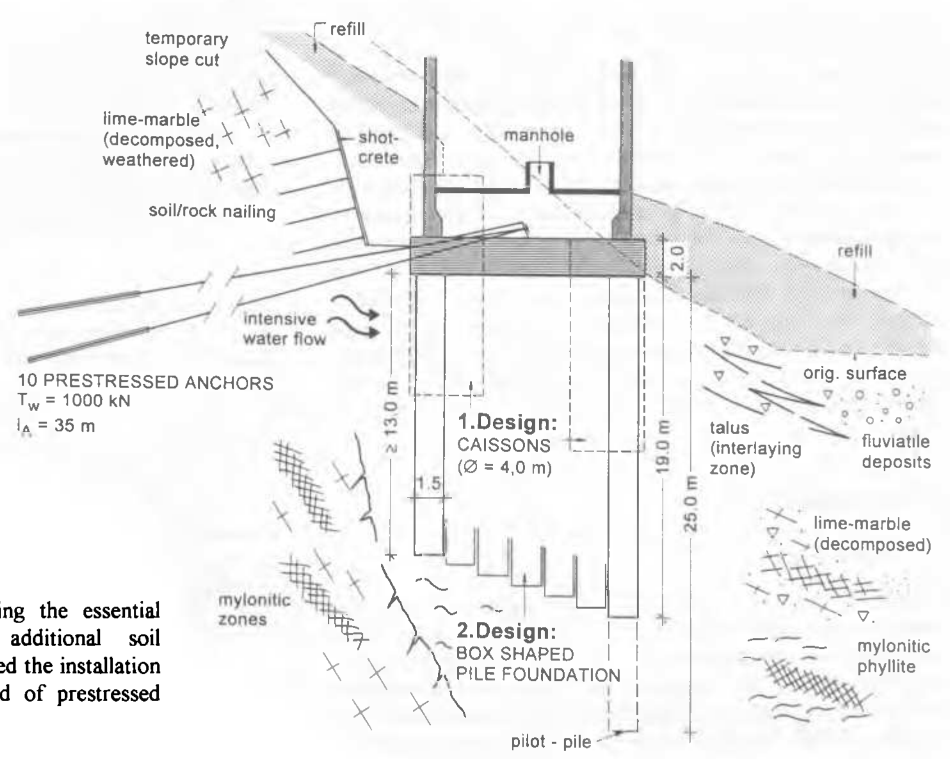


Fig. 9:
Cross section to Fig. 8, illustrating the essential change of the design after additional soil investigation. 2nd design also involved the installation of a deep-seated capping slab and of prestressed anchors against slope movements.

Fig. 10: Partial view of an Alpine highway in Austria, running on steep, slide-prone slopes in a seismic area (up to 10° MS).

failure was already in a progressive state due to the local soil lens with a very low residual shear strength. The movements went on in spite of an unloading. Consequently, the recorded settlements were caused not only by a vertical compression of the subsoil but also by considerable shear deformations.

Remediation reduced the longitudinal differential settlement within the caisson group beneath the allowable value of $\Delta s_l = 10$ mm and the transversal deformation to $\Delta s_q = 0$ (Fig. 3). The long term settlement since opening the bridge in the year 1991 are negligible, and earthquakes have not affected the structure either.

1.4 *Redesign of another bridge pier*

Near the other end of the 1100m long bridge, the above described problem could be avoided to a great extent by performing timely additional site investigations instead of a mere interpolation between the borings of the previous bridge concept. Figures 8, 9 show the first design which involved the sinking of four caissons for a twin pier. When excavating the first caisson, an unexpected high groundwater level and slide-prone clayey interlayers were found. Additional core borings disclosed such complex local ground conditions that the foundation procedure was immediately changed from caissons to a box-shaped arrangement of large diameter bored piles (i.e. second design in Fig. 9). Furthermore, the pier toe was lowered, and the whole structure had to be tied back with prestressed anchors (Fig. 9).

2 UNSTABLE SLOPES

2.1 *Calculated risk*

Most of the highways in the mountainous region of Austria are running along unstable slopes (Fig. 10). Especially in case of high bridge piers and statically sensitive superstructures, the foundation and structure resp. are very sensible to differential movements. Usually, soil and rock parameters exhibit such a wide scatter that the safety of such slopes cannot be proved by theoretical methods only. Therefore, calculated risks have to be accepted, and monitoring according to the observational method is essential.

In mountainous regions, the ground parameters frequently exhibit wide variation (even within a small area) to such an extent that geotechnical design procedures seem to provide not more than border values and serve for reference only. Figure 11 illustrates this - somewhat schematically - along the slide surface in an unstable slope consisting of extremely heterogeneous, weathered schistose talus with sandy to clayey mylonitic zones. Such small scale - mixed ground conditions are much worse than a multi-layered subsoil with clearly differing geotechnical parameters. The mean design value can only be a „most probable" value and has to be validated by the observational method. Due to the steeply inclined slopes, there is also the problem of the seepage flow, and, moreover, seismic aspects have to be considered. The results of evaluating slope stability or the lateral pressure on retaining structures are less influenced by the method of calculation than by the assumption of relevant soil/rock properties, seepage flow conditions, and seismic parameters. This is the reason why sophisticated design methods are not warranted, but parametric studies with certain boundaries are very important.

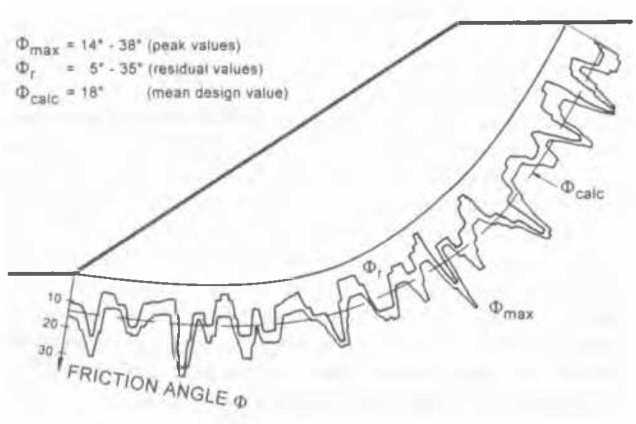


Fig. 11: Scatter of internal friction along the slide surface in an extremely heterogeneous ground. Schematic.

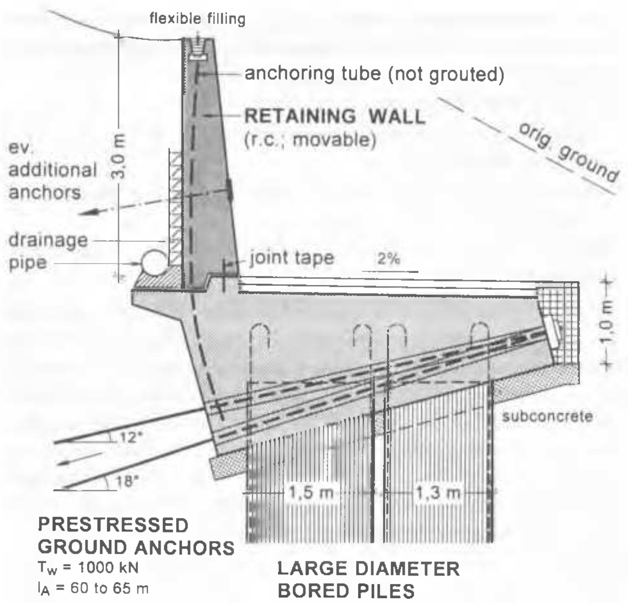


Fig. 12: Top of a twin-pile wall in an unstable slope (deep seated failure planes and creep zone near the surface); heavily weathered and decomposed schists. Capping beam of the pile wall with movable reinforced concrete wall for lateral pressure reduction; precautionary features include the possibility of installing additional prestressed anchors. Capping beam used as permanent access way for slope monitoring.

The optimal solution for slide stabilization and retaining structures can frequently be achieved only step by step in connection with taking in situ measurements. It would be economically unjustifiable to construct most expensive protective structures, whilst throughout assuming and superposing the most unfavourable parameters.

„Calculated risks" are to be accepted in the design of roads and expressways through valleys in mountainous areas where hillsides with a slide potential extend over a distance of several kilometres. Risk assessment has to distinguish between the possibility of local slides and the stability against general failure. In order to reduce construction costs as well as to save time, the application of supplementary construction methods (mainly anchors) should be considered. Soil measures are – even in connection with local remedial works - less costly than an "absolutely safe", fully engineered design which seeks to avoid the possibility of additional measures taken at a later time. Finally, one should bear in mind that „absolute safety" cannot be provided under such extreme topographical and geotechnical conditions.

In such cases, flexible retaining structures have proved successful. They are adaptable step by step, both technologically as well as economically, to the locally prevailing slope pressures, slope movements, and ground conditions. This practical approach is based on continuous measurements and observations of the retaining structure, the surface and the subsoil/rock surface during the entire construction period (e.g. by geodetic survey, extensometers, and inclinometers, monitoring anchors, earth/rock pressure cells). After completion of construction, subsequent random monitoring is recommended. Calculations and theoretical considerations are only the basis for the first design and for interpreting the obtained measurement results. This „semi-empirical" design method has stood up under most difficult conditions for more than 25 years.

Figure 12 shows the top zone of a restraining structure which stabilizes a sliding slope along a highway. The large diameter bored piles dowel deep seated slide zones, and the above retaining wall is flexibly mounted on the capping beam (with a hinge) in order to minimise the surface-near creeping pressure. A prestressing and anchoring of this movable superstructure is possible if long-term monitoring were to disclose a gradual increase of lateral forces. Another combination between rigid and flexible members within a retaining structure is illustrated in Figure 13. Also indicated is the outstanding importance of (deep) drainage of unstable slopes and the possibility to install additional anchors if long-term monitoring required a strengthening of the system (e.g. tying back of the crib wall in Fig. 13).

Foundations in such critical areas highly depend on the static system of the building. If the structure is not too sensitive towards horizontal and/or vertical movements, a flexible system should be preferred. In case of sensitive buildings (e.g. slope bridges with continuos girder superstructures), the foundation requires a high resisting movement (e.g. large diameter caissons, sometimes with multiple anchorage). That means that rather rigid (and deep) footings have to be designed. Nevertheless, even such buildings should be protected - in addition - at the hillside by a flexible retaining structure which simultaneously acts as a first barrier (=„primary" retaining system) against excessive slope pressures. As the latter may change with time, a long-term monitoring of sensitive structures in slide-prone, deeply inclined slopes is unavoidable.

As an example, Figure 14 shows the time-displacements of two slopes which exhibit typical long-term creeping. From such curves, a creeping factor k_{cr} can be deduced which makes a long-term prognosis of future displacements possible. This parameter has proved fairly successful in risk assessment of creeping slopes, but

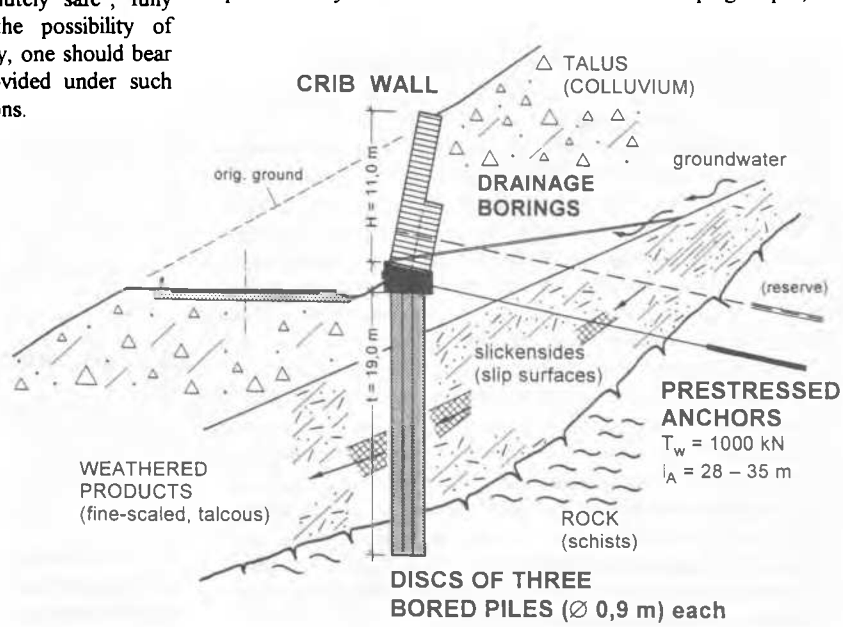


Fig. 13:
Combination of rigid and flexible retaining structures in a sliding slope: Pile wall (of disc-shaped elements), crib wall, and prestressed anchors.

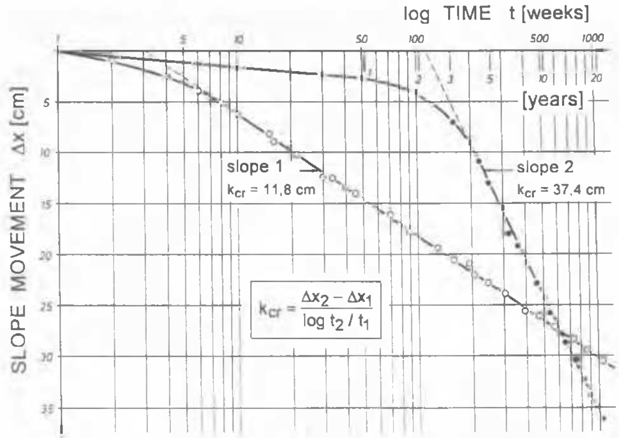


Fig. 14: Regular creeping behaviour of two slope sections. Definition of creeping factor, k_{cr}.

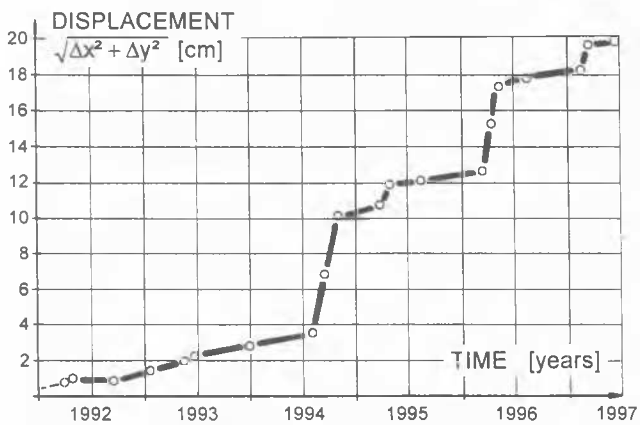


Fig. 15: Irregular movements of an unstable slope with a low residual shear strength. Increasing risk of sudden slope failure.

should always be considered in connection with the residual shear strength of the subsoil. If the Φ_r - value is very low, the creeping velocity can gradually increase due to progressive failure. Experience has shown that this effect may occur with a great time difference, thus providing „delayed" slides. In many cases creeping does not occur more or less gradually, as indicated in Figure 14, but in (seasonal) phases, Figure 15. This is much more critical because creeping may abruptly change to sliding and slope failure - especially in zones with a low residual shear strength.

The protective flexible structure should be designed in such a way that strengthening is possible at any time - especially during the construction period and the first two to five years after completion. Generally, this is the most critical phase. Thus, excessive sliding pressures can be taken over by the primary retaining structure without threatening the sensitive building itself (e.g. slope bridges, television towers, head masts of aerial rope-ways, etc.). Geotechnical anchoring has proved especially suitable for such measures. Usually, long, prestressed anchors are superior to rock or soil nailing with short reinforcing elements. Combinations of several anchor and nailing systems are, of course, possible.

Experience has shown that subsequent strengthening of retaining structures may be necessary due to critical construction stages, hazardous events (heavy rainfalls, earthquake), long-term creep of slopes, long-term deterioration of rock masses or the retaining structure itself, and alteration of external loads. Hence, the possibility of strengthening retaining structures at any time after their construction should be taken into account in the design and calculation as well as during the construction.

The semi-empirical design of retaining structures and foundations in unstable slopes requires the following prerequisites:

- Comprehensive ground investigations.
- Calculations with parametric studies.
- Design of possible supplementary measures.
- Reliable monitoring.
- Practical experience, proper engineering judgement, and intuitive feeling for the subject.
- Joint willingness of all involved persons, clients, and contractors to take over a calculated risk.

In many cases of ground engineering under difficult conditions this philosophy provides the only technical solution - not to mention the cost savings. A „fully engineered" design, i.e. a design which requires no further modification following detailed design is hardly possible. The potential to make modifications during construction and to strengthen the structure at any time, also after construction, is a fundamental requirement of the observational method and the semi-empirical design method. It involves the concepts of the most probable and most unfavourable conditions.

2.2 *Residual shear strength*

Slope stability analyses and designing retaining structures or foundations in unstable slopes require primarily the knowledge of the residual shear strength of the ground. Figure 16 shows - as an example - that the residual value widely depends on the normal stress and on the degree of water saturation. Hence, deep-seated slide surfaces are especially critical. Numerous test series have revealed that there is usually a linear correlation between the internal friction and the maximum grain size of a soil or of a heavily decomposed rock - provided the grain size distribution has no intermittent shape (Fig. 17). The determination of the residual shear strength cannot be based on such a correlation and therefore requires more tests, which are essential for a proper risk analysis.

2.3 *Case histories*

Figure 18 shows an example where the shear strength of mylonites dropped to a minimum residual value of about $\Phi_r = 5$ ° during the construction period. This nearly caused the collapse of a 3-span bridge crossing a highway just one month before opening. Instead of demolishing and rebuilding the bridge nearby, a step by step stabilization was performed with numerous measures based on the observational method:

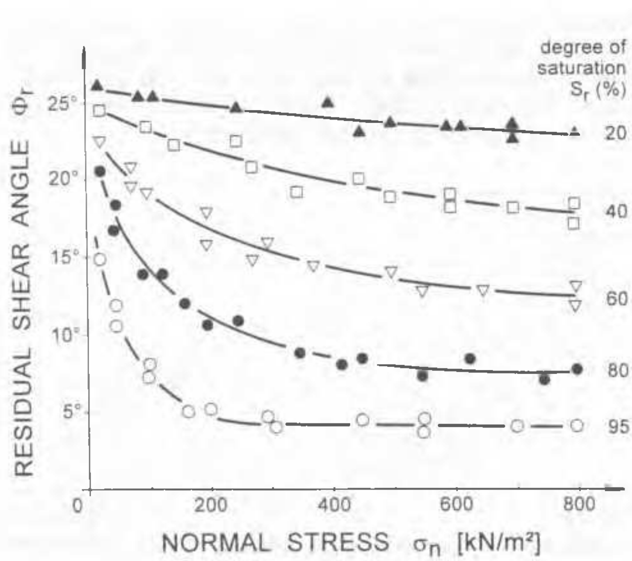


Fig. 16: Residual shear angle, Φ_r, versus effective normal stress, σ_n. Degree of saturation, S_r, as parameter. Results of direct shear tests with silty clay.

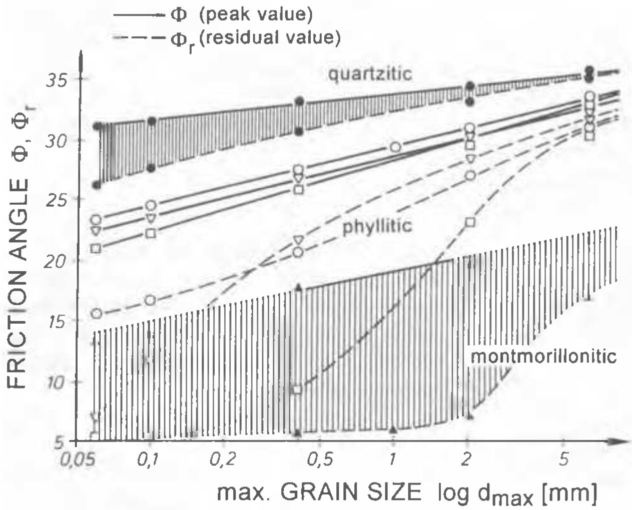


Fig. 17: Angle of internal friction Φ (full lines) and residual shear angle, $Φ_r$ (broken lines), versus maximum grain size, d_{max}. Three different types of mylonites.
Loss of shear strength due to particle re-orientation indicated by hatched areas for quartzitic and montmorillonitic mylonites. In between three examples for phyllitic mylonites.

- Slope flattening.
- Tying back the bridge abutment and the endangered pier by prestressed anchors (90 m long).
- Installing reinforced concrete beams to strut the bridge piers against each other beneath the highway surface; this required the installation of additional caissons on the toe of the opposite slope (Fig.19)
- Dowelling the slope with large diameter bored piles.
- Excavation of the crest of the slope and extension of the bridge by another field consisting of a hollow box.
- Drainage borings (up to 150m length).
- Reinforced concrete ribs with prestressed anchors.

This was a failure history with especially unfavourable conditions despite the relatively flat slope. In most cases the residual shear strength is not the relevant design parameter but certainly is the predominant factor required for risk assessment.

If sliding has already occurred, the residual shear angle should increasingly be consistent for the design of stabilization measures and retaining structures. This is shown in Figure 20 where a 40 m deep excavation in an unstable slope caused progressive failure. Several retaining measures based on the peak value of the shear strength within this geological fault proved to be insufficient: slope-flattening, rip-rap, pile dowelling, tying back with prestressed anchors. The decrease of shear strength exceeded the effects of the stabilizing measures until the cut was widely refilled again, thus forcing the highway in a tunnel - according to the cut and cover method.

When dowelling an unstable slope with bored piles, sliding masses generally should not be considered a quasi monolithic body in nature, though calculations may be based on this idealised theory. Actually, such moving masses contain more or less criss-crossed discontinuities or plastified zones, and secondary fractures may develop within them. Figure 21 demonstrates this in a case where large diameter piles were concreted only close to the primary slip surface according to the conventional dowelling theory. The upper part was refilled with soil (to save money) and was later on sheared off by a secondary slide.

Furthermore, the geometry of the surface and soil layers of that place where the dowels are installed has a strong influence on the design. Fig. 22 illustrates this for slopes which either steepen or flatten below the restraining piles. The first case requires more reinforcement and a deeper socket in the stable ground.

Risk assessment in connection with creeping and progressive failure of slopes is especially critical if statically sensitive bridges have to be constructed there. Monitoring should begin as early as possible before starting construction. Numerous measurements over a period of 25 years have revealed that a creeping pressure acts on retaining structures and foundations in such unstable zones. This pressure exceeds widely the earth pressure at rest but hardly approaches the passive boundary value (Brandl 1979,1987,1993). Figure 23 shows how many retaining measures are required to take over these lateral forces for a bridge abutment on top of a valley in an unstable slope. The structure had to be tied back in the longitudinal and transversal direction with prestressed grout anchors up to 55 m length. Figure 24 illustrates this for a bridge pier in an unstable steep slope. The reinforced concrete beam on top of each pair of caissons allows the structural possibility to install additional long anchors if long-term monitoring indicated the requirement of subsequent strengthening: Sleeves resp. tubes in the concrete which contains proper steel reinforcement for additional anchors.

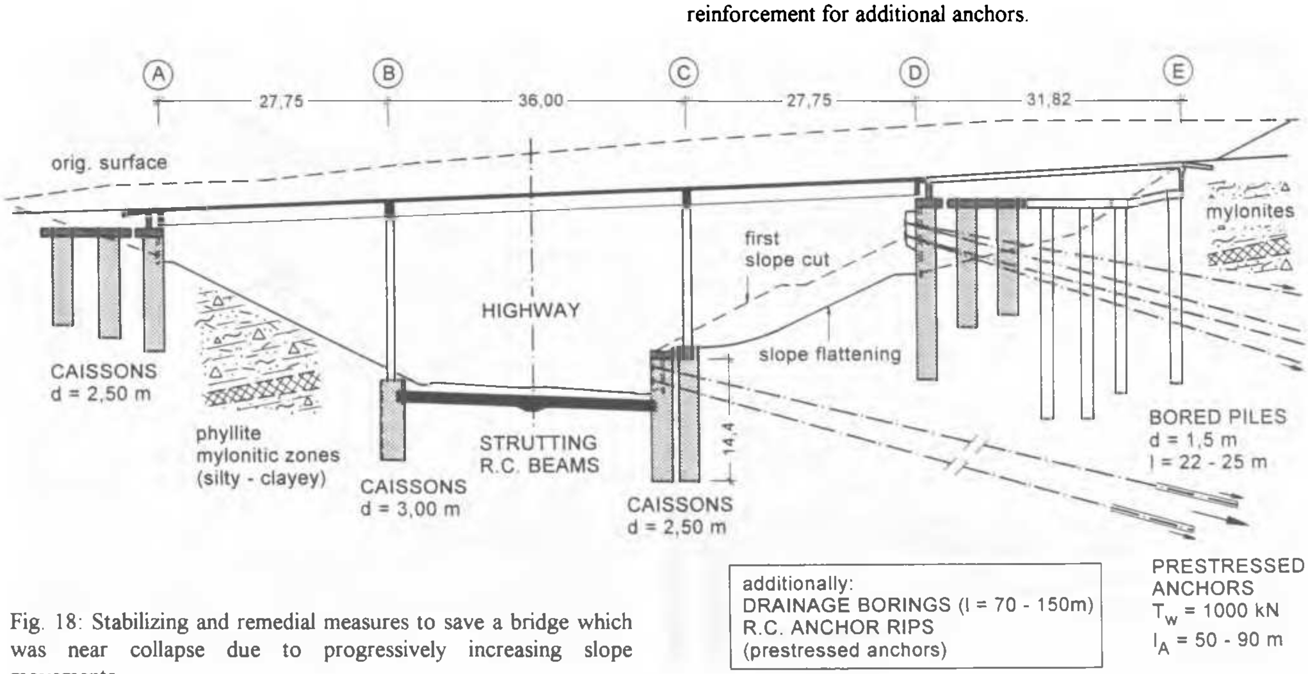


Fig. 18: Stabilizing and remedial measures to save a bridge which was near collapse due to progressively increasing slope movements.

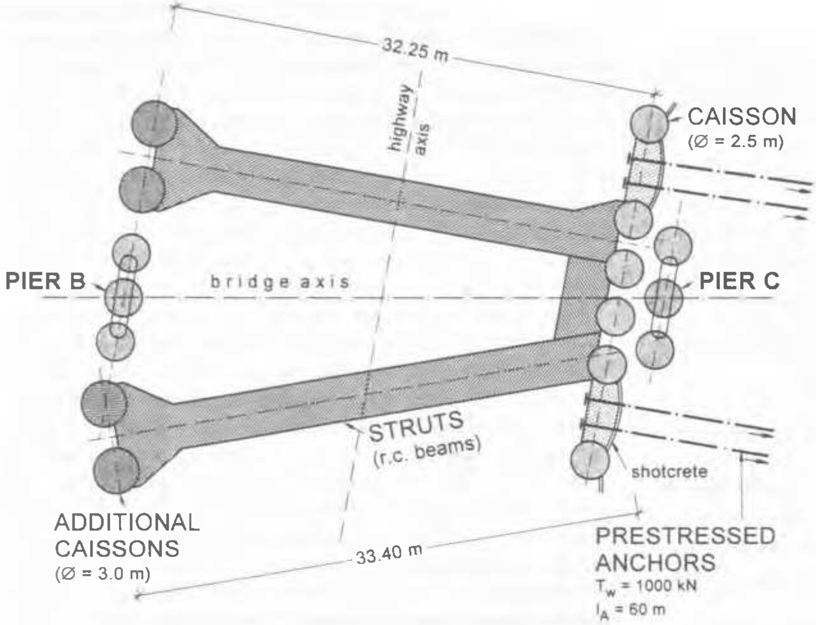


Fig. 19: Partial view of the ground plan to Fig. 18 showing the reinforced concrete beams strutting the caissons against each other on the right and left toe zone of the slope cut, as well as the additional caissons and the prestressed anchors.

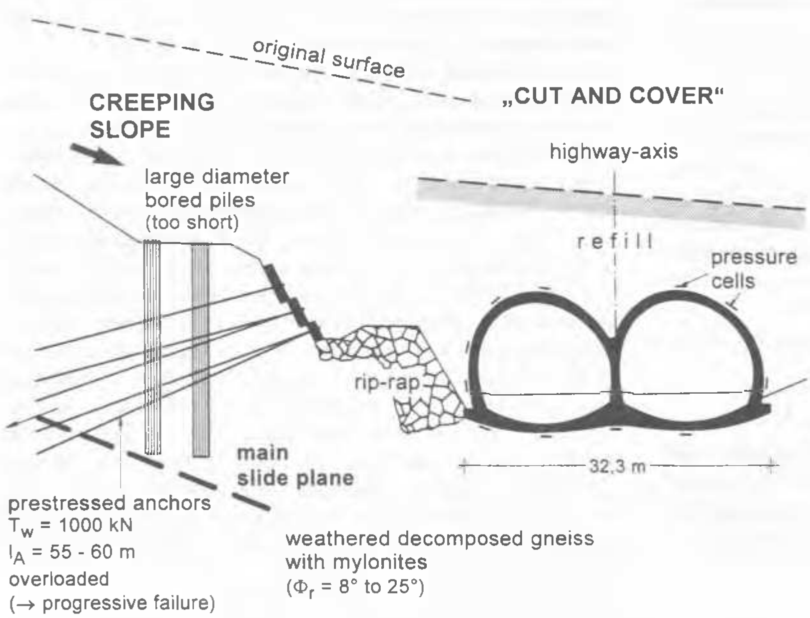


Fig. 20: Insufficient step by step measures to stabilize a creeping slope with a low residual shear strength. Progressive failure finally caused a refilling of the slope cut with a tunnel for the endangered highway.

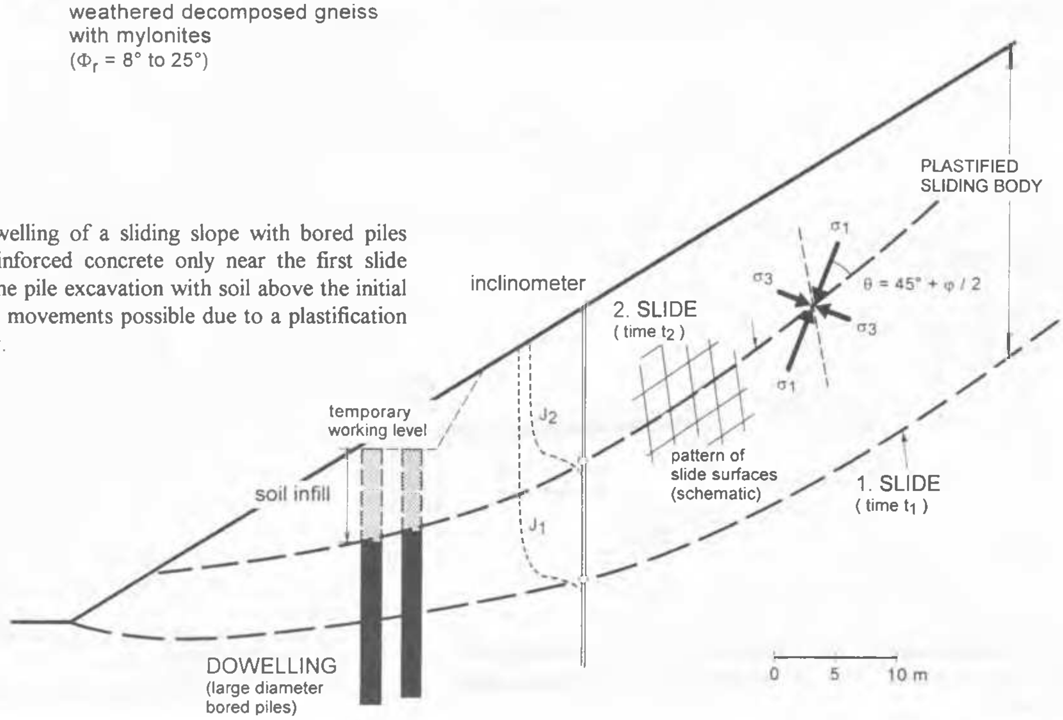


Fig. 21: Improper dowelling of a sliding slope with bored piles which consisted of reinforced concrete only near the first slide zone. The refilling of the pile excavation with soil above the initial slip made further slope movements possible due to a plastification of the whole slide body.

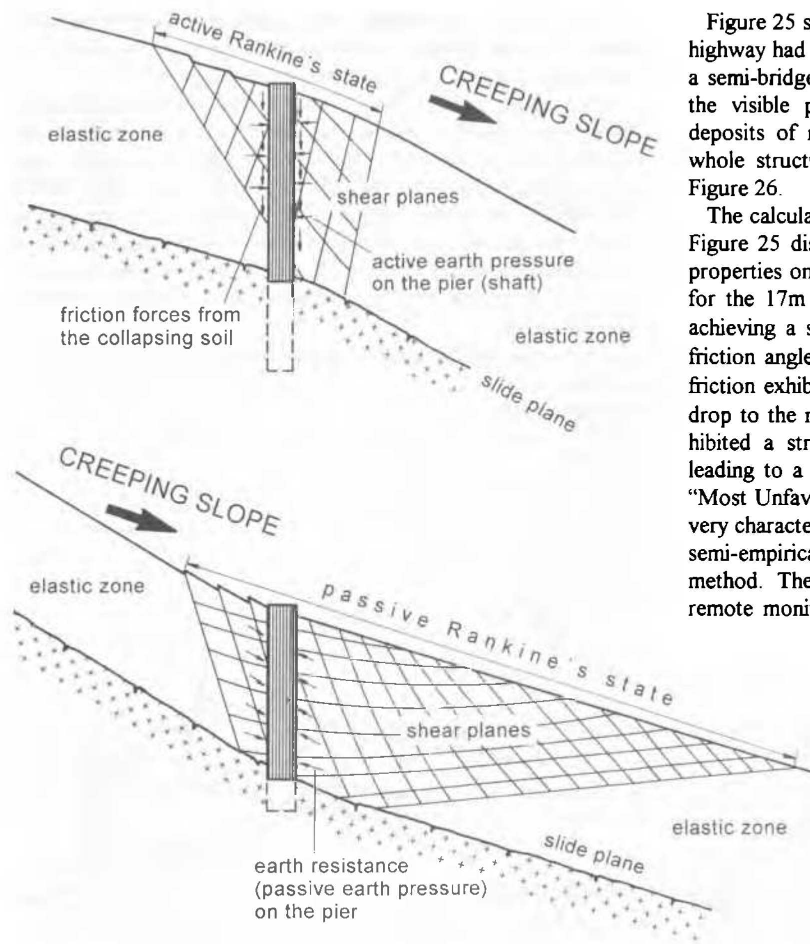
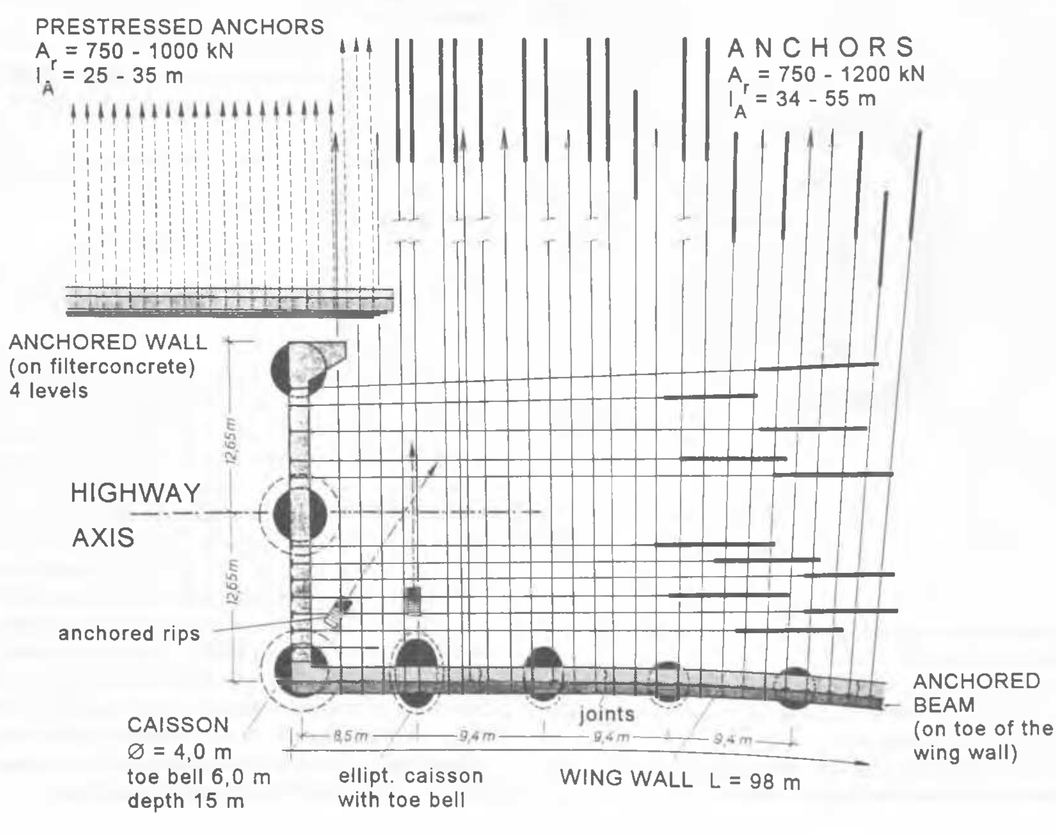


Figure 25 shows the cross section of a slide-prone slope where a highway had to be constructed. In order to minimise the slope cut, a semi-bridge was designed. Its foundation is by far deeper than the visible part above ground surface. Moreover, silty slope deposits of mica schist required an intensive anchorage of the whole structure. Details of monitoring installation are given in Figure 26.

The calculation of the 17 to 22 m high anchored element wall in Figure 25 disclosed the extreme influence of the ground's shear properties on the required anchor forces. Figure 27 illustrates this for the 17m high wall section. The necessary anchor forces for achieving a safety factor F = 1 varied by ΔA = 1000 kN if the friction angle varied by only $\Delta\Phi = 1°$. But actually, the internal friction exhibited a scatter of $\Delta\Phi = 15°$, and, moreover, it could drop to the residual shear value $\Phi_r \ll \Phi$. The cohesion also exhibited a strong influence on the results of calculation, hence leading to a great difference between "Most Probable"(MP) and "Most Unfavourable" (MU) conditions. This example is therefore very characteristic of the advantage of the observational method or semi-empirical design resp. over the fully engineered design method. The half-bridge exhibits multi-anchored caissons with remote monitoring of the anchor forces. In the top zone of the

Fig. 22: Influence of the slope inclination and soil/rock layers on the earth pressure on caissons or large diameter bored piles for slope stabilization.

Fig. 23: Ground plan of a highway bridge abutment in a steeply inclined unstable slope. Foundation on caissons and geotechnical anchoring in both directions. Retaining walls also intensively tied back with prestressed anchors. Only anchors of one of a total of four levels are indicated. $A_r = T_w$ = working load.



caissons and in the retaining walls, tubes have been installed to make - in case of danger - subsequent strengthening possible, fast and at all times.

Along many sections of highways in the Austrian mountains, these comprehensive ground engineering measures account for roughly 80 % of the total construction costs, i.e. the visible parts of the bridges represent only 20 %. Monitoring of the ground structures is less costly than that of superstructures (bridge decks) but is, nevertheless, essential for proving the long-term stability.

Accordingly, extensive monitoring has been performed since 25 years on all critical construction sites, comprising the following measurements:

- Movements and stresses of ground and structures.
- Anchor forces.
- Water levels, etc.

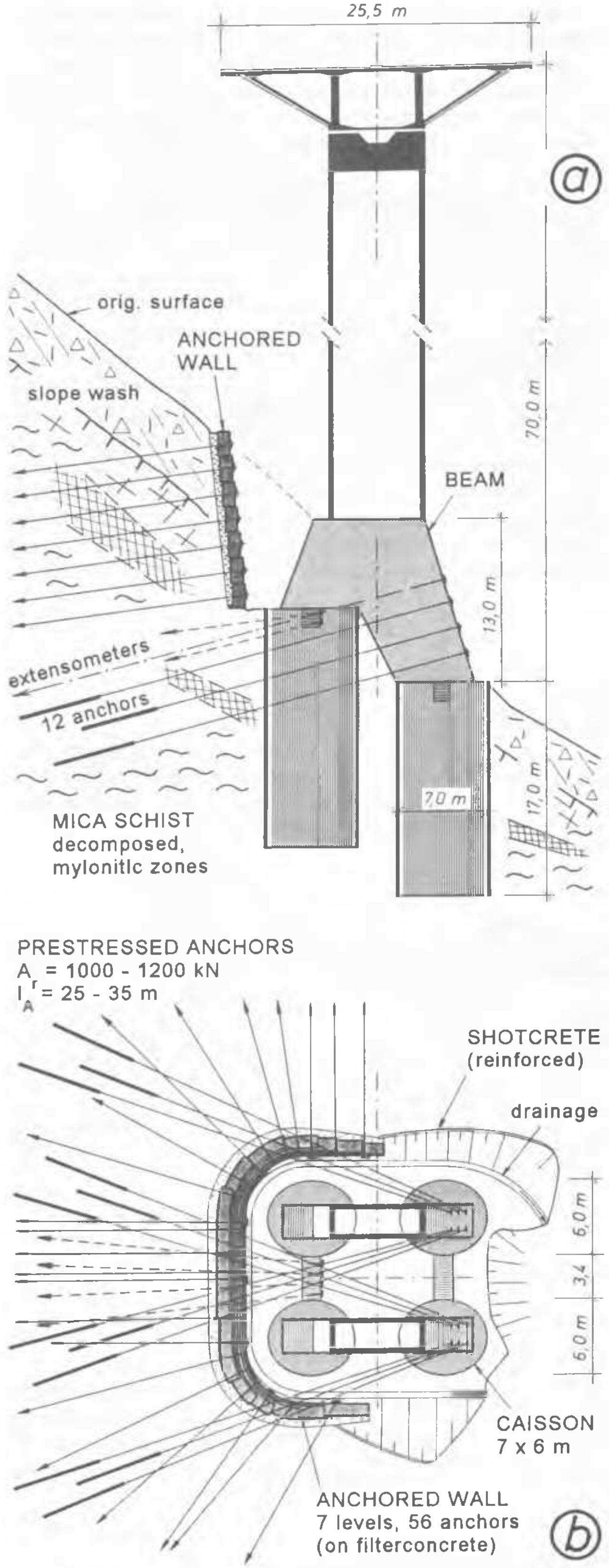


Fig. 24: Typical foundation and anchoring of a bridge pier in a steeply inclined,sliding slope.
Statically sensitive superstructure (2.6 km long bridge) completely separated from the slide-protection measures (tied back retaining structures) around each bridge pier.
Cross section (a) and ground plan (b). Prestressed anchors of the anchored element wall only indicated for one level.

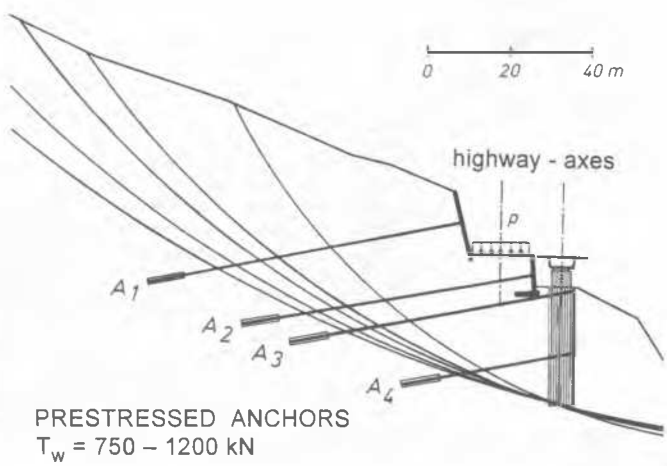


Fig. 25: Cross section of a semi-bridge on deep caissons in a slide-prone slope. Bridge, cantilever wall, and element wall tied back with prestressed anchors. Example for assessing the required anchor lengths from various slide surfaces, running through the caisson's base.
A_1 = resultant of anchor forces of the anchored element wall
A_2 = resultant of anchor forces of the tied back cantilever wall
A_3 = resultant of anchor forces on top of the caisson (remaining accessible)
A_4 = resultant of anchor forces within the caisson (only remote reading possible)
A_r = working load of anchors (=T_w)

3 CONCLUSION

In conclusion, it should be emphasised that building in unstable, heterogeneous, or soft soil and rock includes a significantly higher calculated risk than is experienced by other branches of civil engineering. Consequently, a proper design requires comprehensive experience and engineering intuition. In most cases, sophisticated theoretical models and calculations simply feign an accuracy which in practice does not exist. Statistical investigations, in the end, do not really solve the problem either. But, parametric studies are essential for a reliable risk assessment and to follow the concept of most probable and most unfavourable conditions. This involves design issues which need to be closed out during construction or even in the long-term according to the observational method. Unstable terrain requires a „semi-empirical" design method, based on comprehensive monitoring - and pre-planned safety measures which allow for future strengthening if the results of long-term measurements require such.

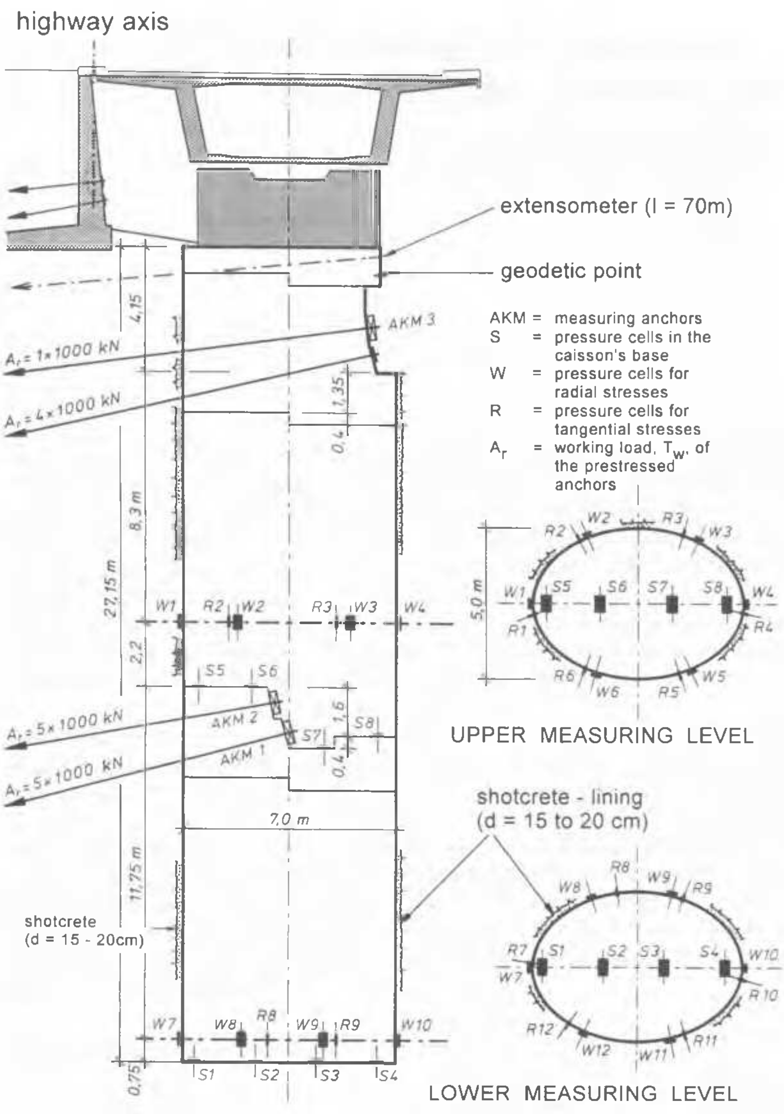


Fig. 26: Detail to Fig. 25 with monitoring instrumentation.

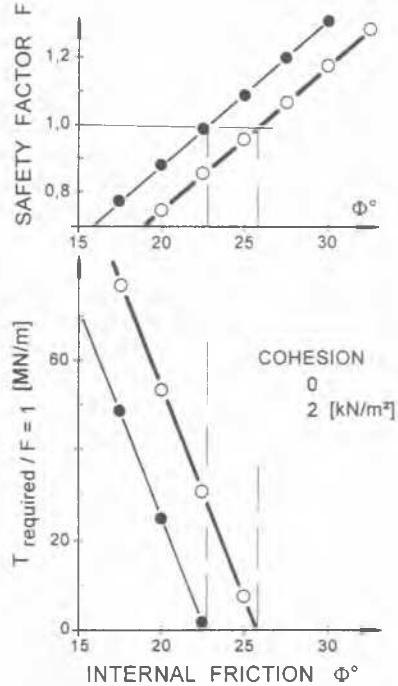


Fig. 27 : Influence of shear parameters on the safety factor against slope failure, F, and on the required anchor force T per meter run of the structure to achieve F = 1 for the anchored element wall above the semi-bridge in Fig 25.

REFERENCES

Brandl, H. 1979. Design of high, flexible retaining structures in steeply inclined, unstable slopes, Proc. 7th Eur. Conf. Soil Mech. Found. Engg, Brighton.

Brandl, H. 1987. Retaining walls and other restraining structures. In „Ground Engineer's Reference Book", F.G. Bell (ed.), Butterworths, London, 47/1-47/34.

Brandl, H. 1993. Installation, monitoring and design of caissons, Proc. of the 2nd Int. Geotechnical Seminar, Ghent, A.A. Balkema, pp. 3-20, Rotterdam, 1993

Brandl, H. 1998. Foundation strengthening and soil improvement for scour-dangered river bridges. 11th Danube-European Conference on Soil Mech. and Geotechn. Eng., Zagreb/Crotia. In "Geotechnical Hazards", B. Maric et.al (ed.), A.A. Balkema, pp. 3-28, Rotterdam 1998.

Mengis, R. 1977. Geotechnische Probleme bei tiefen Schächten in Kriechhängen. Mitteilungen Schweizer. Ges. für Boden- u. Felsmechanik, Nr. 96.