A bridge foundation resisting sliding soil mass

La fondation du pont résistant la masse du sol glissant

A. Szavits Nossan & V. Szavits Nossan
University of Zagreb, Croatia

B. Stanić
Opusgeo, Zagreb, Croatia

I. Mihaljević
Geokon, Zagreb, Croatia

ABSTRACT
Two viaducts for a new motorway in northwestern Croatia were constructed over a very gently sloping valley. Soil investigations indicated a long landslide consisting of a medium to highly plastic clay layer slowly sliding over very gently sloping and stable marl. The foundation of each viaduct pier was designed as a parallel pair of reinforced concrete diaphragm panels extending through the sliding mass into the stable marl bedrock to resist the thrust of the sliding soil. During the construction of the viaduct foundations, several instruments were installed and readings taken on a regular basis. Almost two years after the viaduct completion, no measurable movements of foundation piers were recorded, although the slope did not yet reach its equilibrium. A three dimensional soil-structure interaction analysis was performed, using a simple elastic-plastic soil model, to verify the stability of the deep foundation. The results of the analysis generally comply with recorded measurements.

RÉSUMÉ
Deux viaducs pour la nouvelle autoroute en nord-ouest de la Croatie ont été construits au dessus d'une vallée doucement inclinée. Les reconnaissances du sol ont indiqué un long glissement qui consistait d'une couche d'argile de moyenne à forte plasticité qui glissait lentement sur la marne stable et très doucement inclinée. La fondation de chaque pile des viaducs a été calculée comme un pair parallèle de panneaux moulés en béton armé qui s'étendaient à travers la masse glissante dans le rocher stable de marne pour résister les poussées du sol glissant. Pendant la construction des fondations des viaducs plusieurs instruments étaient installés et les mesurages ont été pris régulièrement. Presque deux ans après la construction des viaducs, il n'y a pas de mouvement des piles de fondation mesurables même que la pente n'ait pas encore atteint son équilibre. Une analyse numérique trois-dimensionnelle d'interaction entre la structure et le sol, avec un modèle du sol simple elasto-plastique, a vérifié la stabilité de la fondation profonde. Les résultats de cette analyse sont généralement en accord avec les mesurages.

Keywords : deep foundation, barrette, bridge, landslide, soil movements, soil pressure, numerical analysis

1 INTRODUCTION

A stretch of a new motorway in northwest Croatia had to cross a gently sloping valley from 3 to 12 m above the ground level. Ground investigations indicated an unstable clay layer, up to 14 m thick, underlain by stiff marl extending to greater depths. Early inclinometer readings registered a down slope movement of the clay layer with a rate of about 2 to 3 cm per year. An embankment was abandoned as a solution early in the design stage due to uncertainties related to the stabilization of the slide. The designer chose two parallel viaducts instead, each of them with two lanes and seven spans carried by piers.

Each pier foundation consists of two parallel reinforced concrete diaphragm panels (barrettes) of rectangular cross-section, 0.8 m thick and 5.75 m wide, embedded from 12 up to 24 m into the stiff marl, depending on the thickness of the sliding soil mass. The longer cross-sectional axis of each barrette was approximately aligned with the anticipated horizontal direction of the soil mass movement. The clearance between two adjacent barrettes was 1.65 m. The two barrettes were connected at their top by a reinforced cap bearing the viaduct pier. The barrettes were designed to sustain the lateral thrust of the sliding soil mass approximately equal to three times the theoretical passive earth force. The factor of three was chosen by the designer in anticipation of the arching effect in the sliding soil around the deep foundation.

Prior to the start of the construction, a series of inclinometers, surface and viaduct bench-marks and piezometers were installed into the ground. Three vertical deformeters, measuring vertical soil deformations, were also installed adjacent to one of the deep foundations. Several tilt meters were installed on viaduct piers as soon as they were constructed.

The viaducts have been in use for almost two years without any measurable displacement or tilting of the piers, while the upper part of the sliding mass is still active.

This paper compares the results of recorded soil displacements prior and after the construction of foundation barrettes with the results of a three-dimensional finite element analysis of the interaction between the sliding soil mass and the foundation structure.

2 GENERAL LAYOUT AND THE SOIL PROFILE

Figure 1 shows the general layout of the sliding area, the position and orientation of the barrettes and the positions of measuring devices. The recorded horizontal displacement vectors at the ground surface are plotted to indicate the directions of movements.

Figure 2 presents a vertical cross section down the sloping valley, positioned approximately along the greatest clay layer thickness. The position of the viaduct is close to the slide toe. The ground surface of that area has a gentle slope of about 11^0, whereas the slope approaches 20^0 at the head of the slide. The contact surface between the sliding mass and the underlying marl is generally steeper than the ground surface. Several small springs in the upper part of the sliding mass indicate a high phreatic surface of the ground water. Piezometers close to the viaduct area confirmed a small artesian pressure.

Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering
M. Hamza et al. (Eds.)

doi:10.3233/978-1-60750-031-5-1495

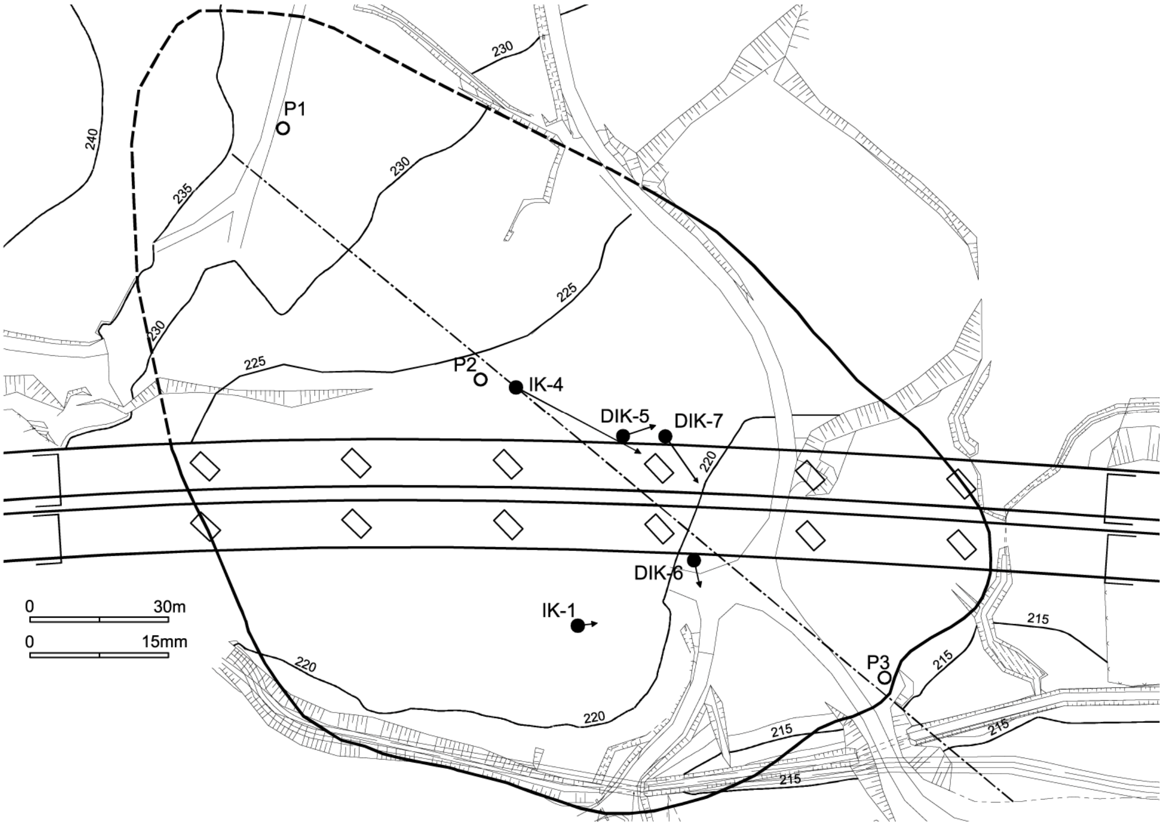

Figure 1. Viaducts, pier foundations, contours of the landslide, positions of inclinometers (IK), deformeters + inklinometers (DIK). piezometers (P) and total horizontal displacement vectors at ground surface measured by inklinometers up to February 2009

Ground investigations in the area generally indicate a clay layer (under the top soil) of medium to high plasticity and medium to firm stiffness (consistency index varied between 0.83 and 1.25, unconfined compressive strength mostly between 115 and 280 kPa but occasionally as low as 30 kPa, and SPT *N* values between 5 and 24, whereas the minimum measured effective friction angle was 18^0 and the effective cohesion 6 kPa). This layer reaches a depth from 4 to 15 m from the ground surface. A softer and more permeable transition layer, up to 1 m thick, was detected under the clay layer. No undisturbed samples could be recovered from this layer. It consists of clayey gravel, crushed sandstone and crushed siltstone. It is underlain by a deep layer of hard marl with the unconfined compression strength between 4 and 9 MPa.

Figure 3 shows Atterberg limits, natural water content and unconfined strength data from soil samples taken from a 28 m deep borehole adjacent to pier S3. These data are more or less typical for the whole sliding area.

3 MEASUREMENTS OF THE SLOPE MOVEMENT

Slope movements were measured form the beginning of the construction at the site and were continued up to the present, almost two years after the completion of the structure. The general history of the movements may be followed in Figure 4. This figure presents the down slope horizontal displacement time history of the ground surface measured by several inclinometers in the vicinity of pier S3: IK-1 on the down slope side, IK-4 and IK-7 on the upslope side. The following may be observed from Figure 4:

- The horizontal displacement rate in the down slope direction was about 15 to 20 mm/year during construction activities;
- After the completion of the structure, the displacement rates remained generally unaltered or even slightly increased for another six months;
- After the six month period upon the completion of the structure, the displacement rates slow down to about 7 mm/year on the upslope side, and to 1 mm/year on the down slope side, indicating a much more pronounced slow down of displacement rates on the down slope side than on the upslope side.

It all implies that the main driving force of the sliding mass originated in the upper slope region, and that deep viaduct foundations gradually balanced its magnitude. Tilt meters, installed on viaduct piers did not indicate any rotation up to the present time.

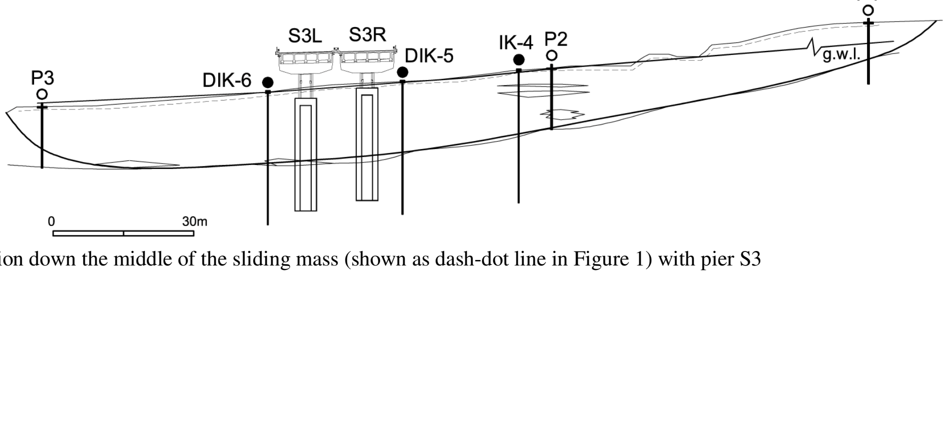

Figure 2. Cross-section down the middle of the sliding mass (shown as dash-dot line in Figure 1) with pier S3

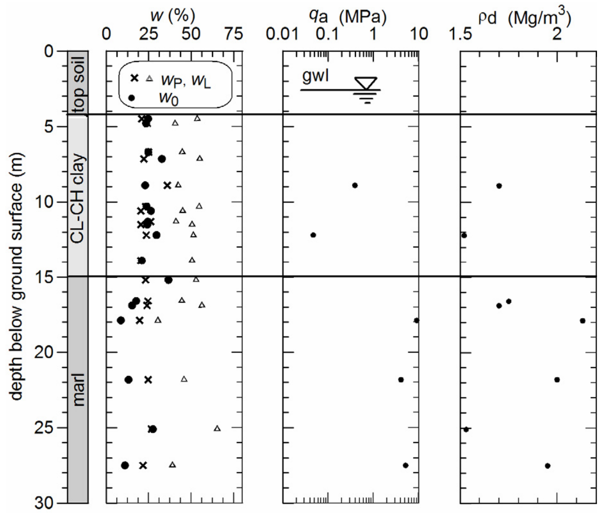

Figure 3. Atterberg limits, natural water content, unconfined compression strength and dry density measured from samples adjacent to the deep foundation

Figure 5 shows horizontal displacements down the slope for several inclinometers and vertical displacements along vertical lines measured by two sliding deformeters (Kovári & Amstad 1982) in front of the deep foundation. Inclinometer measurements indicate the depth of the sliding mass, while deformeter measurements indicate the bulging of the sliding body in front of the deep foundation.

4 THE ANALYSIS

A three dimensional finite element analysis of the soil structure interaction for the sliding mass, firm base and the deep foundation of pier S3 was conducted. The principal aim of the analysis was to verify the stability of the deep foundation under the earth pressure from the sliding soil mass. The complex geometry of the slide and the foundation were simplified, as shown in Figure 6. The width of the sliding mass of 25 m corresponds to the projected length of the viaduct span onto the anticipated direction of sliding. A thin inclined soft soil layer, separating the upper clay sliding mass from the stiff and firm marl base was included in the model. The phreatic surface was at the ground surface.

The modelled slice of the sliding soil mass without the embedded foundation was first brought to plastic equilibrium by phi-c reduction procedure to model the sliding conditions with the factor of safety of one (Brinkgreve & Swolfs 2007). The resulting material parameters for the simple elastic-plastic "Mohr-Coulomb" soil model used in the subsequent analyses are shown in Table 1.

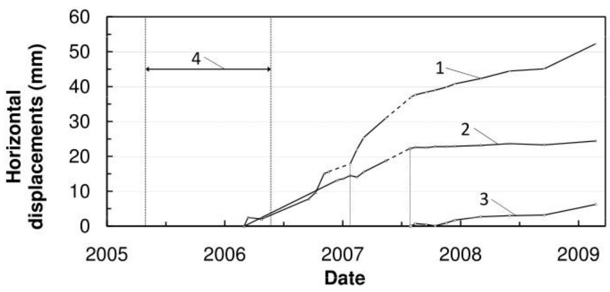

Figure 4. Time history of horizontal displacements of the ground surface as measured and interpreted by inclinometer measurements: inclinometer IK-4 (1), IK-1 (2) and DIK-7 (3); (4): barrettes construction period

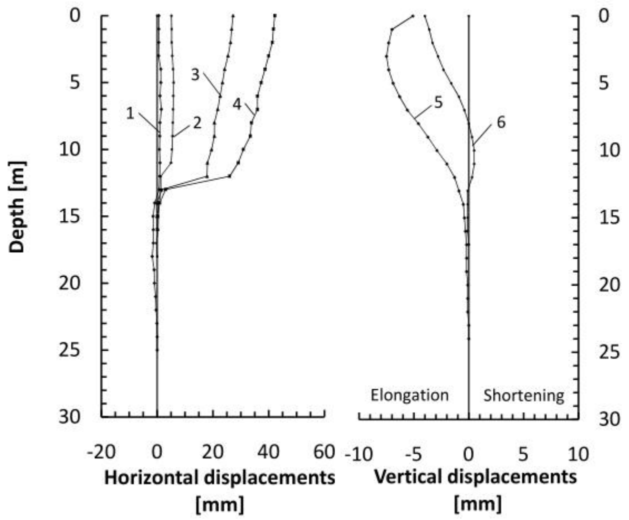

Figure 5. Horizontal displacements vs. depth below ground surface: inclinometers IK-5 (1) and DIK-7 (2) from Aug 2007 to Feb 2009, IK-4 (3) from Mar 2006 to May 2007 and IK-4 (4) from Mar 2006 to Feb 2009; vertical displacements along deformeter axis: DIK-5 (5) and DIK-7 (6) from Aug 2007 to Feb 2009.

Table 1. Soil parameters after the first phi-c reduction

Parameter	Unit	Clay layer	Soft layer	Stiff marl
Density	Mg/m^3	1.85	1.85	2.0
Effective cohesion*	kPa	0.93	0.93	18.7
Effective friction*	degree	15	11.8	28.35
Young's modulus	MPa	10	5	120
Poisson's ratio	-	0.3	0.3	0.25

(*) reduced by phi-c reduction analysis to comply with the factor of safety of one for the slope without the embedded foundation

The next analysis included embedded elastic barrettes and design loads from the viaduct. The individual barrettes were modelled as solid elastic pile elements of rectangular cross sections. The last analysis was a second phi-c reduction procedure. This analysis should correspond to the design situation with maximum possible thrust of the soil mass on the deep foundation.

The top part of Figure 7 shows the original finite element mesh in the vertical plane along the analysed slope. The middle part of Figure 7 shows the deformed mesh after the first phi-c reduction, which corresponds to plastic equilibrium of the sliding mass prior to foundation embedment. The sliding of the upper clay layer and the bulging of the sliding mass at the toe of the slope are apparent. The bottom part of Figure 7 shows the results of the second phi-c reduction analysis, which corresponds to sliding resisted by embedded barrettes. The bulging of the sliding soil upslope of the barrettes is now apparent. It is compatible with extension deformations measured by sliding deformeters.

Figure 8 shows the development of horizontal displacements of the top of two adjacent barrettes vs. phi-c reduction factor during the iteration process of this analysis. The plastic equilibrium is reached with an increase of the factor of safety (phi-c reduction factor) of only 4.5 %. Such a small increase of the factor of safety of the sliding soil mass might explain the very gradual and slow decrease of the rate of soil movement measured by inclinometers. Nevertheless, this analysis should provide the maximum lateral soil thrust on the foundation barrettes against which they should be designed. The corresponding bending moments in foundation barrettes calculated for this design situation are shown in Figure 9.

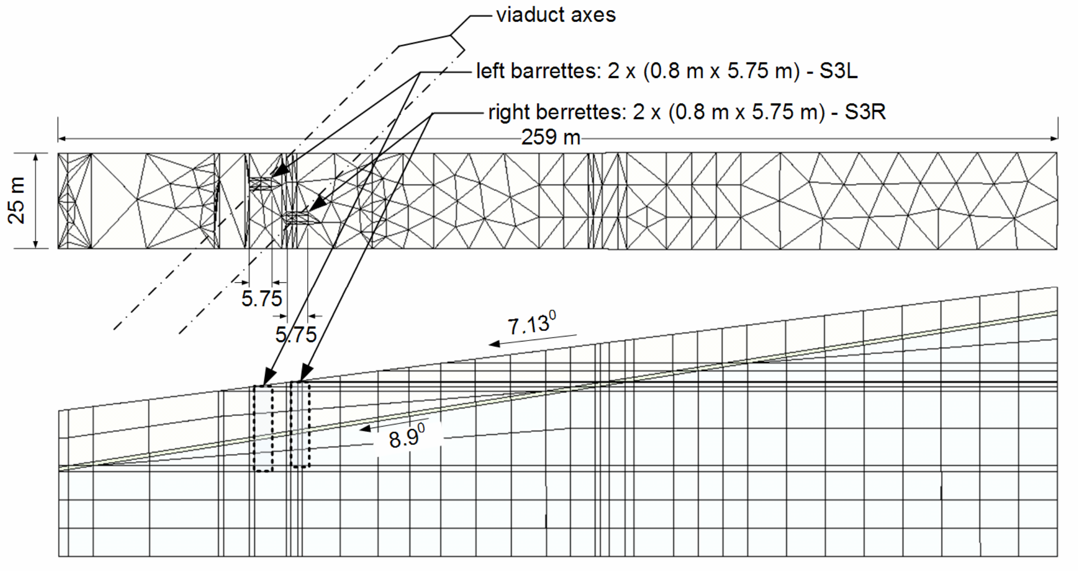

Figure 6. Three-dimensional finite element mesh used in the analysis and the position of foundation barrettes

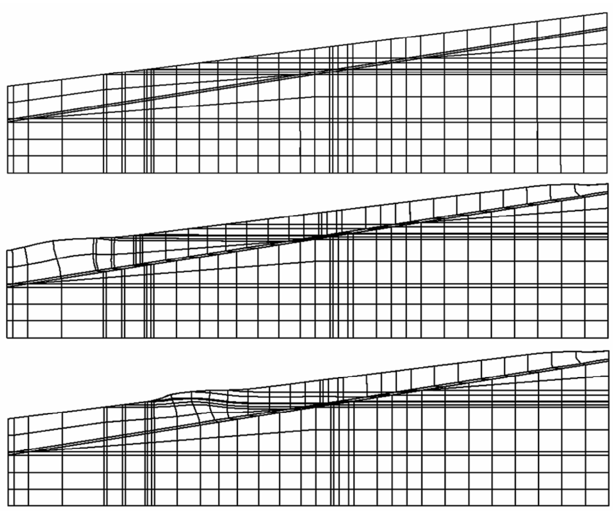

Figure 7. Finite element mesh (top); Deformed mesh - plastic equilibrium without barrettes (middle); Deformed mesh - plastic equilibrium with barrettes (bottom)

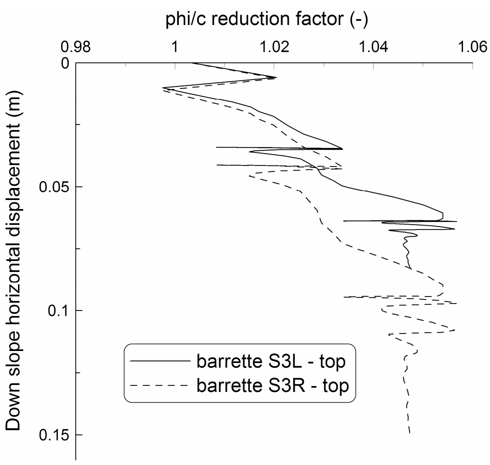

Figure 8. Phi-c reduction analyses with embedded barrettes

5 CONCLUSIONS

Deep foundations of two viaducts were designed to sustain lateral earth pressures from a large sliding soil mass. Extensive measurements of the soil and structure displacements were taken during the construction and almost two years after the successful completion of the structure. Three dimensional finite element analyses showed physically feasible phenomena at the construction site. The analyses included the simulation of the in situ sliding prior to construction and the soil structure interaction analysis, which complied with in situ measurements and demonstrated the general stability of the foundation.

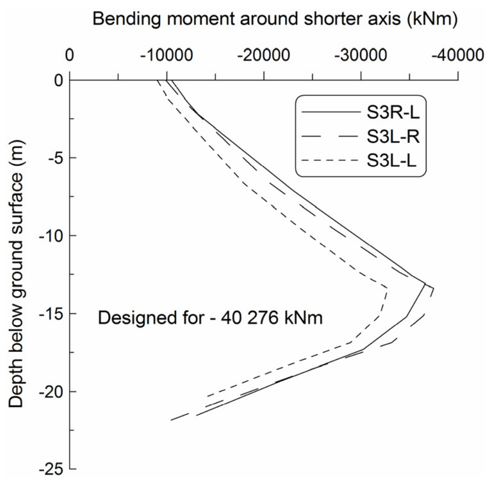

Figure 9. Bending moments in foundation barrettes for the case of plastic equilibrium of the sliding soil mass

AKNOWLEDGEMENT

The authors appreciate the support provided by Ms. Ivanka Brunetta from the Civil Engineering Institute of Croatia.

REFERENCES

Brinkgreve, R.B.J. & Swolfs, W.M., eds. 2007. *Plaxis 3D Foundation - Version 2*. Plaxis bv P.O. Box 572, 2600 An Delft, Netherlands.

Kovári, K. & Amstad, Ch. 1982. Technical note: A new method of measuring deformations in diaphragm walls and piles. *Géotechnique* 32(4): 402-406.