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A complex segmental concrete block retaining wall structure for the reconstruction of a historical bridge

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ABSTRACT: A historical bridge, made up of sandstone masonry, crossing a river in Northern Italy, suffered heavy damage to the wing walls of the abutment on the right side of the river, due to a large upslope landslide. The two wing walls, 7.0 m high, had the shape of a quarter circle and the face was made up of sandstone blocks. The design of the new wing walls was based on the technique of segmental concrete blocks for the face and high tenacity polyester geogrids for soil reinforcement. The concrete blocks were designed approximately with the same dimensions of the original sandstone blocks, with split face and yellow sandstone color to preserve the original external finish. To support the excavation of the unstable sand – lime soil, a micro-pile diaphragm wall, tied by steel tendons driven into the backfill, was designed. Both the design principles and the construction methods used for the reconstruction of the wing walls are described.

1 INTRODUCTION

The "Ponte Prugneto" historical bridge (Fig. 1), originally built around 1850, is located near Maranello, at 400 m above sea level, on the Appennine mountain in the south of Modena in northern Italy, and crosses the Scoltenna river along the Provincial Road SP30. The bridge is made up of sandstone masonry, with three arches of approximately 16 m span each resting on 7 m high masonry abutments and piers, with 4 m high masonry foundations on the river bed. Destroyed by a bomb during World War II, the bridge was rebuilt immediately after the end of the conflict. In 1958 a landslide hit the slope on the right side of the river and damaged the first arch of the bridge. The arch was rebuilt and a connection was created between the foundations of the abutment and the pier, by means of a massive concrete beam which acts as a bank for the river flow as well. In 1980 a new landslide hit the bridge on the right side but the reinforced structure remained intact, showing only a little uplift at the top of the first arch and a thin horizontal lesion. But the wing walls broke off and fall into the river (Fig. 1).

Figure 1. View of the bridge with the failed upstream wing wall on the right.

2 PROJECT

The main objective of the project was to guarantee the road conditions, i.e. the bridge stability. Considering that the foundations-abutments-arch structure survived so well in 1980, it was decided to rebuild only the wing walls. In fact, they support the lateral masonry walls of the bridge abutment and protect the foundations of the abutment from water erosion. The two wing walls, 7.0 m high, had the shape of a quarter circle and the face was made up of sandstone blocks, approximately 0.5 m × 0.2 m × 0.3 m each. The wing walls supported a mass of fine sand and lime soil, made unstable by the landslide movement.

The bridge is the only crossing point of the river within kilometers, hence the wing walls had to be repaired without interrupting the traffic.

The bridge was under historical protection, hence any repair work had to preserve the original shape and original external appearance.

3 CHOICE OF DESIGN SOLUTION

Starting from the idea to restore the bridge to its original shape, the profile of the wing walls were drawn exactly symmetrical to the existing walls on the left side of the river, although different from the sketches of the 1958 project. Such geometry required construction of embankments with a conical form, sustained by 7.0 m high wing walls. Such height, together with the very poor geotechnical properties of the soil, produces a very strong thrust. The traditional solution of a heavy concrete wall with deep foundations and steel tendons was massive and introduced technical complications due to the convergence of the tendons toward a single point, because of the half-circular shape of the structure. Moreover the external face of the concrete wall had to be covered with natural stone blocks, for aesthetical reasons, in order to match the historical appearance. However natural sandstone blocks are very expensive and difficult to find. Therefore the Technical Committee of the Province of Modena Administration, in charge of the project, opted for a new solution: the design of the new wing walls would be based on the technique of segmental concrete blocks for the face and high tenacity polyester geogrids for soil reinforcement. The concrete blocks were designed with approximately the same dimensions and colour of the original sandstone blocks, and with a split face to preserve the original external finish.

The final solution considered past experience, and avoid a massive excavation of the slope. Hence to support the excavation of the unstable sand – lime soil, a micro-pile diaphragm wall, tied by steel tendons driven into the backfill, was designed. The reinforcing geogrids were connected to the diaphragm wall through specially designed connections. The radius of curvature of the wing walls was as narrow as 13 m, thus controlling the radial and tangential reinforcement layout, need to carry the two components of the driving forces. The toe of the new walls was protected against scouring with a embankment stone, cemented with concrete. The geogrid reinforced segmental walls required a foundation on the soft saturated soil at the river bed level. In order to avoid further excavation which would have intercepted the bridge foundations, a concrete mat with a deep front key was designed. In this way important results could be obtained: reduce the excavation volumes, avoid damaging the existing bridge structure, guarantee the safety of traffic and labour during the execution of the works and stiffen the structure to resist future landslides.

In comparison to the traditional solution initially considered, this solution was less invasive, more environmentally friendly, consistent with the architectural context, and simple and quick to build (which means safe working conditions as well).

The project was approved by all the competent authorities. In particular it received the "Nihil obstat" authorization of the Superintendence to the Cultural and Architectural Goods, who are typically adverse to accept innovative materials and technologies for historical preservation works. The works were then awarded to a local Contractor through a public tender procedure.

4 DESIGN

The final design (see Fig. 2) included a micro-pile diaphragm wall, tied by 3 orders of steel tendons contrasted by horizontal concrete beams, to support the excavation and to contrast the upslope movements; in front of this one a geogrid reinforced segmental wall was designed. The stability analyses were carried out based on a 5 layers geotechnical model (see Fig. 3), resulting from geological survey, boreholes and geotechnical lab testing:

- layer 1: reinforced soil, $\gamma = 18$ kN/m^3, $\varphi' = 30°$, k $> 1.09 \times 10^{-4}$ m/s (permeability)
- layer 2: landslide debris, $\gamma = 17$ kN/m^3, $\varphi' = 18°$;
- layer 3: surficial clay, $\gamma = 20$ kN/m^3, $\varphi' = 28°$;
- layer 4: foundation concrete mat, scour stones and micro-piles, $\gamma = 25$ kN/m^3, $\varphi' = 45°$, c′ = 50 kPa;
- layer 5: clay subgrade, $\gamma = 20$ kN/m^3, $\varphi' = 34°$. Extensive global stability analyses were performed, both for static and seismic conditions, considering the worst situation of rapid water draw-down after river flooding. Hydrological analyses identified the maximum 100-year return flood level 4.5 m

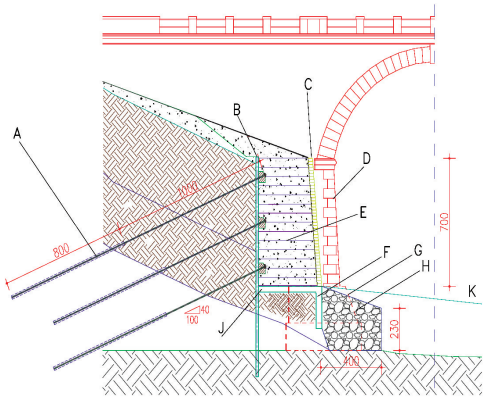

Figure 2. Final layout: (A) steel tendons; (B) micro-piles; (C) concrete block face; (D) bridge abutment; (E) geogrids; (F) front concrete key; (G) scour stone embankment (H) abutment foundation; (J) foundation concrete mat; (K) river bed.

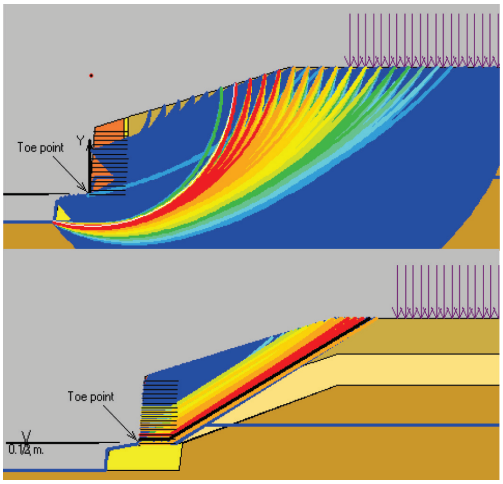

Figure 3. The geotechnical model and typical results of global stability analyses.

above the river bed. The geotechnical model for global stability analyses, with typical rotational and translational results, are shown in Fig. 3.

Since the bridge is located in a Class III seismic area, according to Italian code, the horizontal acceleration a = 0.09 g was applied.

The wall face stability was analyzed based on NCMA recommendation. Laboratory tests provided the values of the connection strength between geogrids and segmental concrete units (NCMA SRWU – 1) and block-block and block–geogrid shearing resistance (NCMA SRWU-2).

The final layout of the reinforced soil retaining walls, 7.0 m high, with a 87° face inclination from the horizontal, required 14 layers of ARTER 200-150 woven polyester geogrids, supplied by Alpe Adria Textil, 6.0 m long and spaced at 500 mm vertical centres. The final cross- section is showed in Fig. 2. For the facing concrete blocks, the Steingrid system, developed by La Cementifera, was selected. This patent pending system includes concrete blocks with a split face, plastic pins for block to block connection, HDPE bars for geogrid to block connections inserted into a groove running along the top side of the blocks. A special production with sandstone yellow colour was developed.

Full testing of the block – geogrid system was performed, including block – block shear, block – geogrid shear, and block – geogrid pull-out, in order to get all the parameters required for the stability analysis of the face. Excellent interaction coefficients and connection strengths were obtained from laboratory tests, allowing stability of the face both in terms of forces and deformations.

The required length of geogrids was equal to 6.0 m, but the diaphragm walls are only 3.8 m from the face, next to the bridge. Hence a connection system between the micro-piles and the geogrids was designed, in order to anchor the geogrids where the available length was less than the design length. Such a connection, shown in Fig. 4, includes a 20 mm diameter steel hook either placed in the connecting concrete beams or welded to the micropile casings, to which a folded steel mesh is then connected; geogrids are wrapped around the steel mesh and fixed into the previously compacted soil layer. In this way geogrids are connected all along the horizontal bar of the steel mesh and not through the apertures, thus avoiding any concentrated stresses and brittle rupture. The connections shall have the same design strength of geogrids. In-situ tensile testing of connections during construction confirmed the design assumptions.

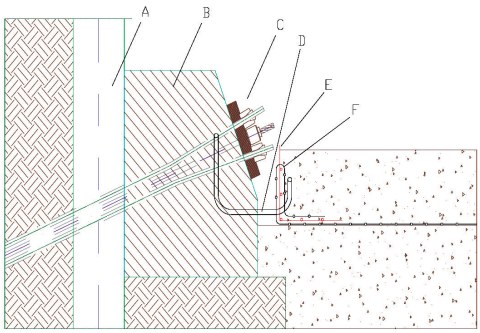

Figure 4. Geogrid – micro-pile connection: (A) micro-pile; (B) concrete beam; (C) steel tendon; (D) steel hook; (E) welded wire mesh; (F) geogrid.

Since the wing walls make a quarter circle in plan view, with a 13 m radius, the static load scheme of a horizontal cross-section is a reverse arch with soil pressure from inside (see Fig. 5). This means that tangential tensile forces need to be resisted by the geogrid reinforcement, in order to avoid the potential failure of the walls by breaking vertically and opening up.

The tangential reinforcement has been designed according to the method proposed by Rimoldi et al.

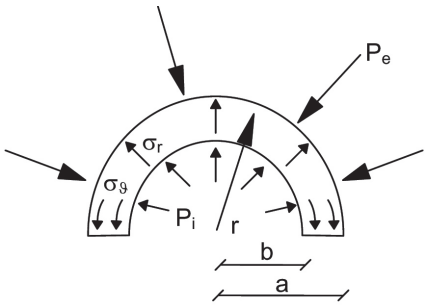

Figure 5. Scheme of forces and stresses on the horizontal crosssection of a circular wall (Rimoldi et al, 1989).

(1989), which convert the reverse arch to an equivalent thick horizontal plate subjected to external radial stresses:

$$pe = 0,\ pi = \sigma h = K\gamma Z + q \quad (1)$$

where K is the coefficient of horizontal soil pressure, γ is the soil unit weight, Z is the depth under the wall crest and q is the uniform surcharge.

The tangential horizontal stress in the soil is then:

$$\sigma_\theta = [a^2b^2\ (p_i - p_e)\ r^{-2} + (p_ib^2 - p_ea^2)]/(a^2 - b^2) \quad (2)$$

Figure 6. Construction works: top view with the micro-pile diaphragm walls and geogrids being connected to concrete blocks (top); geocomposite drainage strips (centre); initial placing of stones for the scour embankment (bottom).

where a and b are defined in Fig. 5.

Considering constant the stress σ_θ over a soil layer of thickness S_v equal to the geogrid vertical spacing, the tangential force which each geogrid layer has to carry is:

$$F_\theta = \sigma_\theta \times S_v \quad (3)$$

The calculation resulted in a tensile strength of 150 kN/m in the tangential direction. In order to avoid laying one layer of geogrid in the radial direction and another layer in the tangential direction, bidirectional geogrids were specified, with 200 kN/m tensile strength in the longitudinal direction and 150 kN/m in the transversal direction. The geogrid was aligned at the face and overlapped toward the back.

5 CONSTRUCTION

The carrying out of the works was rapid. The design foresaw the use of a soil with friction angle $\varphi = 30°$ and permeability $k = 1.10 \times 10^{-4}$ m/sec. Soils were sampled and examined from different pits in the province. The samples coming from a pit very close to the bridge gave good results, but for the permeability: $\varphi' = 37.8°$; $k = 3.40 \times 10^{-8}$ m/s. This soil was approved with the requirement to introduce drainage strips, 300 mm wide at 1.0 m horizontal spacing, laid radially at each half centre between geogrid layers. The strips were cut from a tri-planar geocomposite supplied by Tenax. For the construction of the key of the foundation mat, it was decided to pour the concrete directly inside the excavated trench. The result was a tooth with wedge cross-section, with maximum width of 1.10 m at top, hence larger than the original design. Considering that such a thicker and deeper key considerably reduced the risk of scouring of the foundation, it was decided to reduce the dimensions of the stone scour embankment, aligning its front with the bridge abutment: in this way the light of the bridge within the abutment and the pier remains unchanged, therefore facilitating the water flow. The works were completed without any further expense to the approved budget of 450,000.00 €.

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