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Walls over compressible soils and unstable slopes. Examples

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ABSTRACT: It is well known that earth reinforcement structures are the best solution when a compressible soil with a poor quality is present at the foundation. Choosing an earth reinforcement wall instead of other solutions allows us to solve the geotechnical problem; this is one of the most common applications for this kind of structures. But, there are far more advanced applications, such as. . .

When there is a risk of overall slope stability failure, the use of an earth reinforcement structure instead of classical solutions allows for an increase of the stability safety factor at both short and long terms.

1 INTRODUCTION

Traditionally, the MSE walls have been the solution adopted for structures laying over foundation soils having a too low bearing capacity.

The characteristic flexibility of this kind of structures is what gives its proper behavior. But, what happens when we push up to its limit the bearing capacity of the foundation soil? How good is the behavior of this kind of walls for situations in which other kind of structural solutions are discarded?

In many cases MSE walls are showing adequate performance even over soils where other structures have been declared as both technically and economically useless. For these cases, the MSE structures show themselves as a clear alternative when dealing with inadequate, too soft and compressible subsoils.

This is a compilation of recent samples of MSE structures (with metallic reinforcement) designed and erected in Spain. After the analysis of these samples, we will be able to extract some conclusions quite generic about the factors that condition not only the design, but also the construction and service performance of these structures. Some of the walls samples herein exposed were designed and built knowing in advance the bad subsoil conditions, while for other samples the problem was exposed during the service life of the structure.

2 EXAMPLES

2.1 *Icod de los vinos (Tenerife)*

This is a peculiar case because the project was split in different phases. Initially, an 11 m height wall was designed at the bottom of an embankment; a highway passing over the embankment needed to be widened. So, in order to achieve this goal, the above mentioned wall was built; allowance for a width increase would be possible due to the addition of backfill material over the embankment, being the backfill to be withstood by the wall at the base.

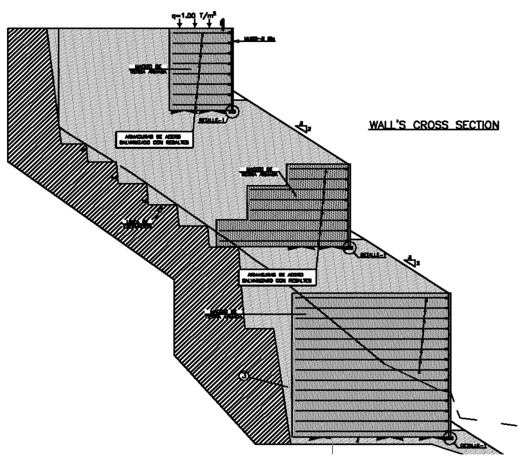

Figure 1. Caption of a typical cross section. *Icod de los Vinos* (Tenerife, Spain).

After having designed and erected the structure taking into consideration the imposed design requirements, it was found that there was a potential risk of lack of slope stability due to the self weight of the planned backfill at the top of the first (the lower one) MSE wall designed and the critical geometrical configuration of the original cross section .

Consequently, after the first wall was built, it was necessary to design (after conducting a global stability

Figure 2. Tiered walls at Icod de los Vinos (Tenerife, Spain).

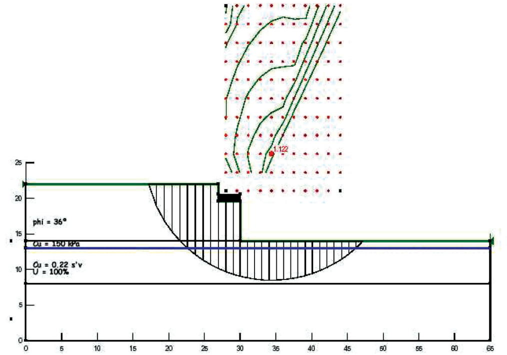

Figure 3. Monzon's detour abutment's long term circular slip stability analysis. (1.5 Security coefficient not reached).

analysis using TALREN, a Bishop Method's commercial software) a group of retaining walls inserted in the new slope in order to stabilize it.

It was decided to place two MSE walls which allowed for an increase of the global stability safety coefficients at the short and the long term for the analysed failure surfaces.

The contribution of the two walls to the increase of stability was not equal: it was the intermediate one who more effectively contributed to the slope stabilization. That wall was designed using a trapezoidal cross section, but the longer reinforcements were placed at the lower block instead of on the upper, as these walls are typically projected. (Figs. 1–2)

The length of the steel reinforcement strips (and also the reinforcement density) were increased section by section as demanded in order to 'interrupt' the most critical failure circles and achieve the imposed security coefficients.

The average security coefficients (from the preliminary client's studies) before designing the definitive solution (two tiered MSE walls placed at the top of the lower one) had a value of 1.2. At this point, it was compulsory to act section by section, up to reach a 1.5 security coefficient (at least) by increasing the length and the reinforcement quantities.

Such a configuration, composed of three tiered MSE walls allowed for the stabilization of a slope that initially was deemed as unstable; this was achieved without altering the design of the first wall, which was not the optimum within the scope of the more complex final solution.

The technical specifications (calculation cases, security coefficients, method, etc. . .) took into account during the design time was the *French Technical Specs for MSE Structures*: NF P 94-220-0.

2.2 *Monzon's detour (Zaragoza, Spain)*

A detailed analysis was accomplished during the construction of the Monzon's detour (Zaragoza, Spain); this analysis took into account the possibility of having some global stability problems (Fig. 3) in some of the backfills, slopes and even some of the MSE abutments due to the possibility of finding inadequate substratum soil. That is, it would be necessary to modify the design of some structures some of them already partially built, to ensure their stability.

More specifically, there was a problem with two MSE abutments partially erected which didn't have the necessary reinforcement dimensions to have a stable structure. After reaching this construction stage, the possibility of dismantling the structure and redesigning a new one was discarded. In order to solve the problem quickly, it was necessary to use the elements which had been already pre-cast and used to partially build the structure, (reinforcement strips and pre-cast panels). After analysing several possibilities, the following solution was adopted: additional reinforcement was placed as needed beginning from the highest level already built, in order to allow for an increase of the structure's global stability safety coefficient. Again, it was necessary to 'interrupt' the most critical failure surfaces.

As a result of the application of these design criteria, two additional reinforcement levels had to be placed, each one having a density of 3.5 strips per linear meter (metallic strips, 45 mm × 5 mm rectangular section) and 20 m length. It wasn't possible to link those additional reinforcements to the facing, which was already pre-cast; in addition, such a linkage was not necessary at all, given the projected strip lengths and the way the reinforcement would work (Fig. 4).

Such a configuration allowed for an adequate stabilization of the structure, increasing as already mentioned the global stability safety coefficient mentioned the global stability safety coefficient.

2.3 *Wall #19 (River Ballonti Area)*

This case corresponds to a wall laying over extremely soft and compressible soils. A badly planned and

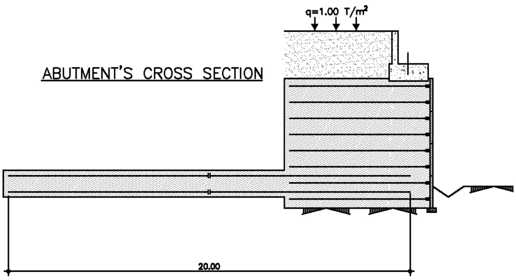

Figure 4. Caption of an abutment's cross section. Focus on two (20 m. long) metallic reinforcement layers added.

Figure 5. Huge settlements (0.5 m–0.75 m) suffered along the wall's facing with no cracked panels reported at all. See the horizontal joint's deflection.

inadequate site characterisation campaign prevented from knowing in advance the real characteristics of the substratum under the structure. Later, the real soils foundation characteristics were discovered.

The structure partially laid over the Ballonti's actual river bed, while its ends laid over anthropic soil. The average thickness for these anthropic deposits was around 3.0 m. Under that layer, there was a superficial stratum of alluvial-colluvial clayey sediments having a 0.70 m average thickness. Bellow it, there was a 7 m layer of alluvial material, mainly silts & mud, showing an allowable bearing pressure of 0.1 MPa approximately.

The results of the in-situ & laboratory tests gave a friction angle from 14° to 16° and cohesion values varying from 0.014 to 0.03 MPa.

Because of this, big settlements were observed; although some settlement was already expected, it was not in the same amount as it was observed. The wall suffered from a maximum settlement of nearly 0.75 m (Fig. 5). In spite of this settlement, the structure showed no disruption along its length. This behaviour was to be expected and, as the wall was built in several construction stages, some added measures were considered.

Figure 6. Repairing jobs. Elastic joint of expanded polystyrene placed at the footing.

There were some zones of the structure in which there was a possibility of interaction between the MSE wall and the footings of some piles which were cast in place close to the facing (0.5 m at most). In order to minimize the effect of the above mentioned footing over the MSE wall, some vertical joints were placed at adequate locations of the facing. But, due to a mistake in the construction of the piles' footing, the wall facing was allowed to lie over a corner of the footing, while the backfill had no restraint for its vertical movement.

Once the settlement of the foundation soil started, the backfill followed this movement, while the facing started cracking because of the vertical displacement restraint. As a consequence, some of the ties that linked the reinforcement to the facing panels broke. It was necessary to repair the facing and to make an EPS' elastic joint at the footing in order to reduce the remaining settlement (Fig. 6).

2.4 *Bilbao - behobia highway*

This is the case of two MSE abutments under an isostatic bridge (No intermediate piles, 25 m span, 2 decks 12 m width each one, and an average weight on beamseats of 300 kN/m) built over extremely compressible soils in which the amount of maximum settlement to be expected was known in advance; in this job the different design and construction stages were carefully planned.

A detailed site exploration including lab and on site tests was conducted in order to know the strength parameters of the subsoil and the maximum expected settlements.

In this case, after a standard stability analysis, the global stability wasn't considered as a critical matter, so no special study should be necessary (Fig. 7).

The abutments were monitored in order to have real time data during the construction about the settlement evolution, allowing for the possibility of making modifications on the design of the structure if

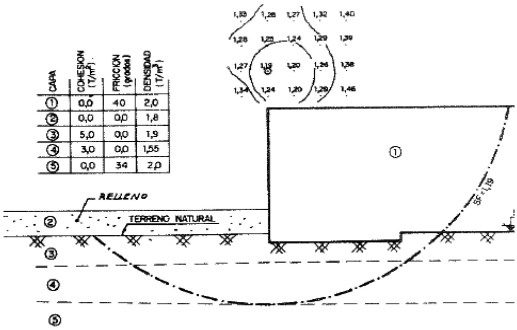

Figure 7. Bilbao – Behovia abutments. Standard circular slip stability analysis. No special study needed.

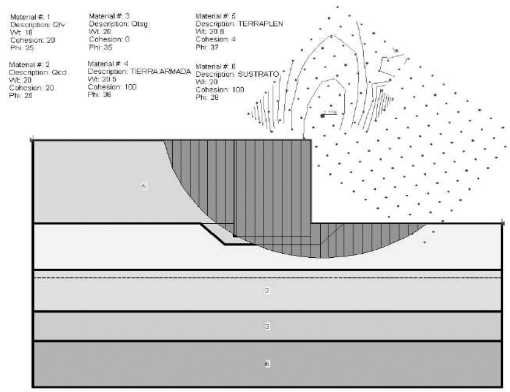

Figure 8. Abutment's global stability assured due to a foundation's soil replacement.

necessary, according to the so called Observational Method.

In order to limit the bridge settlements its construction was delayed up to two months alter finishing the underlying MSE abutments. The abutments' design height was increased in order to take into account the expected vertical settlement, around one meter.

In spite of keeping finally all this cautions, two months alter the construction of the bridge the mean value of the observed abutment settlements were approximately 0.2 m higher than expected; the bridge was not affected because it was an isostatic structure.

In spite of all the above mentioned, the observed differential settlement along the abutment facing was in the order of 1.8%. It should be mentioned that the behaviour of the MSE blocks has been excellent given this level of deformations; some panels showed just some cracking on the corners, which under no circumstances, compromised the proper functionality of the structure.

This structure was put on service without any problem and it is still open to traffic.

2.5 *Subsoil replacement under an abutment (Zaragoza)*

This example is quite representative of a fairly common solution within the MSE walls design area.

This case showed the typical problem of the presence of a substratum layer of very low bearing capacity. We should add to this the existence of a preliminary estimation of very high short and long term settlements (total and differential). As a consequence, the structure (a MSE abutment) would have suffered a series of vertical movements (non uniform) that were not compatible with the serviceability of the bridge itself. In addition, the global stability of the whole (abutment + foundation) was compromised, showing unacceptable values for the safety coefficients.

The adopted solution in this case was to replace the foundation material up to 2 m depth with a selected material which greatly increased the bearing capacity and showed out to conform a foundation soil for the MSE abutment. (Fig. 8)

3 CONCLUSIONS

The MSE structures are and will remain for a long time as the most adequate to be built over soft soil with low bearing capacity. The flexibility is the main quality of this kind of structures when comparing them to any other "rigid" solution that is discarded at first sight when these geotechnical problems are present, because technically and also economically there are no valid competitors against the MSE solutions.

The above mentioned consideration should not be used as a valid excuse to forget about the need of conducting a proper site exploration and testing, with a detailed analysis of the foundation soil characteristics.

When dealing with a global stability problem, the MSE solutions answer to the problem by themselves, without the need of resorting to external solutions, that is, the structure itself due to its basic conception (backfill + reinforcements) together with an adequate design is capable giving an adequate answer to the stability problem.

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