Repairing sinking of the bridge over Guadalquiver River in Alcala del Rio, Seville, Spain

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Abstract: The bridge over Guadalquivir river, near Alcalá del Rio in Seville (Spain), has nine spans of 30 m with a total length of 270 m. In 1993 two of the central piers located in the centre of the river sank by 140mm and 60mm. The superstructure is isostatic and is formed of six prefabricated concrete T-beams 1500mm high. The foundations for each support are formed of two circular pier-piles with a diameter of 1.5m.

The bridge was constructed between July 1990 and October 1992. In the first week of April 1993 it was observed that the handrails were at a vertical angle in the centre of the bridge, which indicated a sinking of piers P-4 and P-5 in the middle of the river. Following topographic observations showed vertical movements of 139mm and 54mm in the two piers.

After studying different solutions it was concluded that the best technical and economic remedial option was to use Jet Grouting injections. By using two concentrical circular crowns of nine Jet Grouting injections, with the most interior vertical and the most exterior inclined to the axe of the pier, a kind of superpile of soil-cement was fomed greatly increasing the factor of safety and the most importantly lifting pillars P-4 and P-5 10 cm and 2.5 cm respectively, reducing almost all the angular distortion of the deck. The remedial works were undertaken in 1993.

Résumé: Le pont sur la rivière Guadalquivir, près Alcalá del Río dans le Séville (l'Espagne), qui a neuf embrasures de 30 m chacun avec une longueur totale de 270 m, a subissent en 1993 une écroulement de 14 cm et 6 cm sur deux des piliers centraux placés dans le centre de la rivière. La superstructure est isostatique et est formée par six T-poutres en béton armé préfabriqués 150 cm. hauteur. La fondation est constituée par deux piles circulaires ot 1,50 m de diamètre dans chaque appui.

Le pont a été construit entre juillet 1990 et octobre 1992. Dans le première semaine d'avril 1993 il a été observé que les rampes ont formé un angle vertical dans le centre du pont, qui a indiqué une embrasures des piliers P-4 et P-5 juste au milieu de la rivière. L'observation topographique suivante a montré les mouvements verticaux de 13,9 cm et 5,4 cm dans chaque pilier.

Après l'étude de solutions différentes on l'a conclu que le mieux techniquement et économiquement, était pour utiliser Jet-Grouting injections. En utilisant deux couronnes circulaires concentriques de neuf Jet-Grouting, la d'intérieur vertical et le plus extérieur incliné à l´axe du pilier, il a été formé une sorte de superpilier de ciment de sol levant radicalement le coefficient de sécurité et le levage le plus important les piliers P-4 et P-5, 10 cm et 2,5 cm respectivement, la réduction presque toute l'altération angulaire du pont. Les travaux de réparation ont été faits en 1993.

Keywords: Bearing capacity, bridges, cavities, deformation, density, foundations.

INTRODUCTION

The JUNTA DE ANDALUCIA Administration constructed the Variant of Alcalá of the Rio, which includes a nine span bridge over the Guadalquivir river, between July 1990 and October 1992.

The bridge consists of nine equal spans of 30m, which amounts to a total length of 270m. The deck consists of 10m of pavement plus two sidewalks of 1m, forming a total width of 12 m.

The bridge was designed with deep foundations for all supports, giving the existing ground in the zone. The bridge foundations are formed of two circular pier-piles with a diameter of 1.50 m and a length of 25 m. Of the 25m for the piers 2m are embedded in gravels, 12 m. (8 diameters) in blue marls with the remaining 11m corresponding to the exempt part of the pier, of which the lowest 2 m. corresponds to the average depth of the Guadalquivir river (Figure 1 and 2).

Figure 1. Bridge in phase of construction

Figure 2. Finished bridge

DETECTED PATHOLOGY

In the first week of April of 1993, a vertical angle was observed in the metal vehicle barriers and in the handrails of the bridge, which indicated the possibility of a lowering of the deck at the level of piers P-4 and P-5 (Figure 3).

The level of these piers was verified, which observed average settlements in piers P-4 and P-5 of 139mm and 54mm respectively. It was decided to carry out a follow-up of the settlements by using topography, by installing fixed levelling bases on both sides of the bridge. From 4th April the follow-up was carried out, which was observed that up to 17th May 1993 both abutments and the rest of the piers did not have any settlement, but piers P-4 and P-5 continued settling reaching a total settlement of 146mm and 60mm, respectively. The settlement of both pillars was similar.

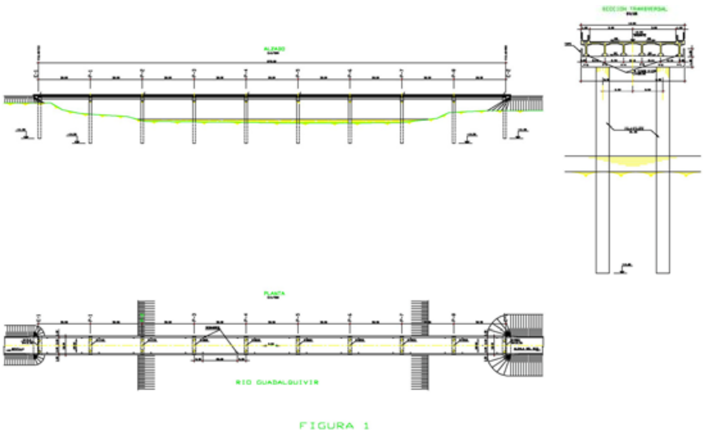

Figure 3. General drawings of the bridge

ACTIONS ON THE PIER-PILE

A review of the bridge design was conducted to verify its validity. The pier-piles were calculated considering the following loads:

- Dead loads.
- Live loads - in accordance with the Spanish Instruction of Bridges, considering diverse positions of the same load. These comprise a uniform load of 0,40 Ton/m2 and a carload of 60 Ton.

The maximum axial load characteristic, which transmits from the piers to the ground (assuming both pillars were working equally), without considering wind and the earthquake loading, was calculated as 503,02 Ton. per pile. In case of performance of the wind and the earthquake the maximum and minimal loads were:

- $Q_{máx}$: 554,22, Ton.
- $Q_{mín}$: 451,82, Ton.

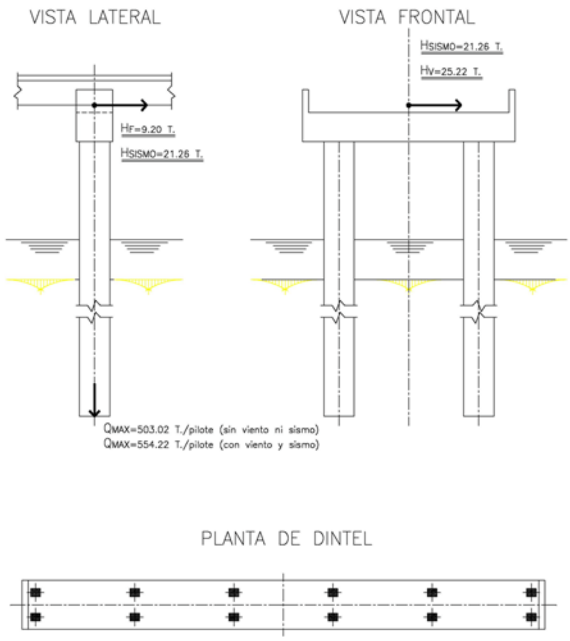

Figure 4. General drawings of the bridge pier piles

The scheme of maximum horizontal and vertical actions in the piers were:

C.P.(T) + S.C.V.(T)	66,82 + 20,16	79,02 + 25,92	79,02 + 79,27	79,02 + 79,27	79,02 + 25,92	66,82 + 20,16
TOTAL(T)	86,98	104,94	158,29	158,29	104,94	86,98

* SUMA C.V. TABLERO=	700,42 T.	SUMA=1.006,04 T.=503,02 T/pilote
* PESO DINTEL+TAPES=	72,00 T.	
* PESO PILAS - PILOTE=	233,62 T.	

A ground investigation was undertaken for the bridge design which indicated several metres of alluvial deposits resting on the mioceno substrate of marls to unlimited thickness. None of the boreholes were conducted at the location of piers P-4 and P-5. The alluvial was proved to have an average compactness and the marl had a compression resistance of q_u=6,3 kp/cm^2. The S.P.T. tests gave an average value of N=62.

With these values the ultimate load capacity for pile was:

$Q_h = Q_p + Q_f = A_p \cdot r_p + A_f \cdot r_f$

A_p, A_f = Cross section and perimeter of pile

r_p, r_f= unitary ground resistance of tip and on lateral friction

$q_u=6{,}3k_p/cm^2=63Ton/m^2$

The project adopted:

$r_p=5\ q_u= 315\ Ton/m^2 \quad r_f=0{,}25\ q_u =15{,}75\ Ton/m^2$

The ultimate load, not including the the resistance of shaft friction of the gravels was:

$Q_h = 1{,}766 \cdot 315 + 56{,}52 \cdot 15{,}75 = 1.446\ Ton$

The safety coefficient to the maximum load of service without magnifying was:

$$F = \frac{Q_h}{N_{\max}} = \frac{1.446Ton}{503Ton} = 2,88$$

This was considered to be acceptable. If loadings from earthquakes and the transverse wind was included, the coefficient of safety was reduced to F=2,51 in the most loaded pier; this value was also considered sufficient.

RECOGNITIONS REALIZED AND RESULT OBTAINED

In view of the settlement, it was decided to undertake two boreholes on the basis of the following:

Hypothesis

Due to the similar settlement behaviour in the pair of pier-piles for piers P-4 and P-5, an hypothesis was made that the marl could not be homogeneous and contained some soft nodules or a cavity under the pile tip. The theory that the piles were badly installed or that there was an imperfection on the pile was discarded as it was considered unlikely that this could have occurred to all 4 piles. In addition, by the controls taken during the construction heterogeneities along the pillar were also discarded.

Realised recognition

The investigation consisted of the drilling of 2 boreholes, one between the piles of the pier P-4 and other one between those of the pier P-5. The boreholes were undertaken on a boat as the piers were placed in the centre of the river. In situ standard penetration tests (S.P.T.) were undertaken every 2.00 m in each borehole. In addition, soil samples were recovered for laboratory tests.

Result obtained

The soil profiles obtained are as follows:

BORING	PIER	MOUTH ELEV.	DEPTH (m)	TIPE OF SOIL
S-1	P-4	-1,98	From 0 to 0,25	Lime
			From 0,25 to 2,25	Gravel
			From 2,25 to 28,50	Blue Marl
S-2	P-5	-1,93	From 0 to 1,30	Fine Gravel
			From 1,30 to 21,30	Blue Marl

No hollows or soft zones were detected below the piles tips; nevertheless, the identification of the softer marl was important. The original geotechnical investigation identified that the marl had a N(SPT)=62 and a resistance to compression of at least 6,3 kg/cm^2. The S.P.T. test in the additional investigation registered blows less than half the original N value along the pile and under the pile tip. The average N blow count turned out to be of 36 (in S-1) and of 35 (in S-2). All of these values are lower than the N=62 from the original project investigation, which was carried out in the zones of the abutments. In fact, in the additional investigation no SPT values of 60 or greater were obtained over the 20 to 23 meters depth of the borehole. By applying the relationship between the S.P.T. test and the resistance of compression (Bowles, 1996 & Xanthakos, 1995), of q_u = N/7,5, values as low as q_u=21/7,5=2,8 kg/cm^2 were obtained, which is much lower than the value of 6,3 kg/cm^2 given in the initial geotechnical report of the base project. The simple compression tests performed on samples gave low values, between 3,40 and 4,91 kg/cm^2, in the first 12 meters, except a value of 10,08 kg/cm^2. Major values of 6 kg/cm^2 were found below depths of 16 m, which corresponded to N values greater than50.

As conclusions the following points were indicated:

a) The existence of large soft zones of soil or caves below the pile tips were not confirmed.

b) Nevertheless, the first 16m of marl was found to be much less competent than given in the original geotechnical report.

c) Due to the minor geomechanic resistance of the soil, it was decided to verify the safety coefficients of the piers P-4 and P-5 with new geomechanic parameters.

d) For the calculations that are developed later it was supposed that the resistance to simple compression along the pile and at the tip was of 5 kg/cm^2, which is equivalent to a blow of N = 38 (q_u = 38/7,5 = 5 kg/cm^2).

ANALYSIS OF THE FOUNDATION AND POSSIBLE REASONS OF THE PATHOLOGY

Piers 4 and 5

The projects foundation design was checked considering the caps of gravels and existing blue marls in the area of piers 4 and 5 and considering the soil thicknesses as identified in the boreholes. The admissible load of the pile can be calculated from the ultimate pile capacity. The ultimate load was calculated as the sum of the resistance by tip and the shaft friction resistance.

$Q_h = Q_p + Q_f = A_p\, R_p + \Sigma$ Perimeter x $L_i\, R_{fi}$

The admissible load Q_{adm} is equal to the load of collapse Q_h divided by the safety coefficient F. The value of the safety coefficient in the normal case is F=3. In the cases where there is sufficient knowledge of the site area and the resistant properties, some authors consider that the value of the coefficient might be reduced up to 2. The admissible load must be compared with the maximum characteristics axil corresponding to the maximum axil of service, without magnification.

In this case, the tip resistance was adopted as:

$R_p = 9C_u = 4{,}5\ q_u$ (marls)

The value for shaft friction in the marls was:

$R_f = \alpha\ C_u$ (marls)

According to Woodwaver mentioned by J.A.Jiménez Salas (1.972), for a value
$C_u = q_u/2 = 5/2 = 2{,}5$ kg/cm^2, $\alpha = 0{,}3$ from what it is obtained:

$R_f = 0{,}3.\ C_u$

$R_p = 4{,}5.\ 5{,}0 = 22{,}5$ kg/cm^2 = 225 Ton/m^2

$R_f = 0{,}3.\ 2{,}5$ kg/cm^2 = 0,75 kg/cm^2 = 7,5 Ton/m^2

- Resistance under the pile tip: pile Ø=1,50 meters

  $Q_p = A_p\ .\ R_p = 1{,}766\ .\ 225 = 397$ Ton

  $A_p = 1{,}5^2\ .\ \pi / 4 = 1{,}766\ m^2$

- Resistance for shaft friction: pile of 12 meters.
  - Resistance for shaft friction in marls:

  $R_f = 7{,}5$ Ton/m^2

  - Resistance for shaft friction in gravels:

  $R_f = 5$ Ton/m^2

  $Q_f = \pi\ .\ 1{,}5\ (2\ .\ 5 + 9{,}75\ .\ 7{,}5) = 4{,}712\ .\ 83{,}1 = 392$ *Ton (1)*

  $Q_f = \pi\ .\ 1{,}5\ .\ 9{,}75\ .\ 7{,}5 = 344$ *Ton (2)*

(1) Considering the contribution of the gravels.
(2) Not considering the contribution of the gravels.

- Ultimate loads at collapse.

  $Q_h = Q_p + Q_f = 397 + 392 = 789$ *Ton (1)*

  397 + 344 = 741 Ton (2)

Values lower than that obtained in the base project and for which the safety coefficient was turning out to be:

- Real coefficient of safety piers P-4 and P-5.

  F = Q_h /N = 789/503 = 1,57 *(1)*

  741/503 = 1,47 *(2)*

These coefficients of safety were considered to be totally insufficient. The working load was near the ultimate load and the soil was in elastoplastic phase, justifying the observed settlements, even more if it had been compared with the maximum design load (with wind and earthquake). With the previous parameters for r_p and r_f, supposing a module of deformation of the soil of E = 100 C_u it was estimated, by means of an elastoplastic program the settlement, being obtained as a value of 163mm., to be very similar to the observed ones. From this result it was proved, that the resistance by shaft friction was almost totally mobilised, being the pile also working for tip. This result can also be estimated seeing the values of maximum resistance for shaft friction Q_s = 344 to 392 Ton that is the first one that is mobilised.

OFFERS OF PERFORMANCE

Piers 4 and 5

In the previous paragraph it appeared that piers P-4 and P-5 had a very low coefficient of safety.

Therefore it was considered necessary to increase this coefficient of safety, and three possible procedures were studied for it.

a.) The first one comprised transmitting the load of the structure by means of a footing. This formed a footing of 5mx10 m. This solution presented difficulties with connecting these footings to the existing piers and for the construction under 2 meters of water in the middle of the river.

b.) The second solution comprised constructing a series of piles or micropiles that would join by means of a concrete pile cap, joining the piles and the existing pier-pile. Besides the possible limitations of vertical space for their construction, the same problems would exist as the construction previously mentioned for the concrete pile cap. In this case it would be necessary to have 5 piles Ø=450 mm. for each pier-pile.

c.) The third solution would be to improve the soil. The only suitable technology for this is jet grouting.

In this solution it was proposed that the soil was improved along the shaft of the pier-pile, and the superior active zone , the inferior and the security zone were injected, to obtain a " rigid element " of a diameter of approximately 3,50 m. Without considering the shaft friction, simply as tip resistance this element has a coefficient of safety larger than 3, with the solicitation of 503 Ton.

$$Q_p = 8\ c_u\ .\ A_p = 8\ .\ 25\ .\ \pi\ .\ 3{,}5^2/4 = 1920 Ton$$

F = 1920/503 = 3,8

The coefficient 8 was considered instead of 9, since in piles of such a great diameter it is recommended to reduce N_c. This solution had the advantage by which it was possible to implement it by floating means and it was not necessary to construct any joining element to the existing pier.

ADOPTED SOLUTION

From the economic analysis of all three solutions it was determined that the Jet Grouting solution had the least cost and the least problems to implement. In addition, with this solution a reduction in the settlements of piers P-4 and P-5 could be obtained. This solution consisted of the formation of 18 injection columns, which comprised nine vertical at a distance from the pile of 0.50-1.00 meter and the other nine inclined approximately 6,5 º degrees, as shown in figure 5. The injection produced a column of soil-cement of approximately 400-500mm. in diameter, consolidating and refilling the zones of the marls that surrounded the column. The recommended pressure for the injection pump was 350-400 kg/cm^2 and the cement injection was fixed to 250-300 kg of cement per linear meter for the treated zone. The dosing of the cement grout in weight was water/cement = 1/1, therefore the density of the mixture is 1,5 Ton/m^3.

The columns were formed alternatively, so that between 2 adjacent columns formed in the same day, there was at least 1,50 m., or 24 hours of consolidated cement of the adjacent columns, to what was being constructed at that time. The reasons for this gap in space and time was to avoid the possible fluidity of an important area during the injection of the mortar, as that could give rise to an unforeseeable increase in settlement. Also, during the jet grouting the traffic continued to use the bridge, although the lane that was over the pier-pile that was being treated was closed. The most sensitive part of the procedure was after an injection. Therefore, it was recommended to take frequent levels of the affected piers to control the raising of the pier and also reducing the speed of the vehicles on the bridge to avoid important dynamic loads. With the treatment, the settlement of the piers P-4 and P-5 was reversed obtaining a raising of the twin pillars of approximately 10 cm as maximum.

INITIAL SEAT	**FINAL SEAT**
P-4: 14,6 cm………………………………………………………………………..….	4,6 cm.
P-5: 6 cm…………………………………………………………………………………	3,5 cm.

The work was carried out during June and July of 1993. Figure 6 illustrates the initial and final settlements.

RESULT OF THE PERFORMANCE

The principal result was the total correction of the difficulties experienced in piers P-4 and P-5, restoring as minimum the same conditions of safety against foundation collapse as predicted in the initial project. In addition, it was possible to correct a great part of the settlements produced in the piers, with which, it was achieved to largely reduce the angular distortion of the deck. Although in an isostatic bridge of these characteristics it is possible to at first maintain the functionality of the structure, in spite of the fact that differential settlements take place between piers, with the adopted solution, many of the angular distortions diminished to minimal values, recovering again the bridge to an acceptable aspect from the aesthetic point of view.

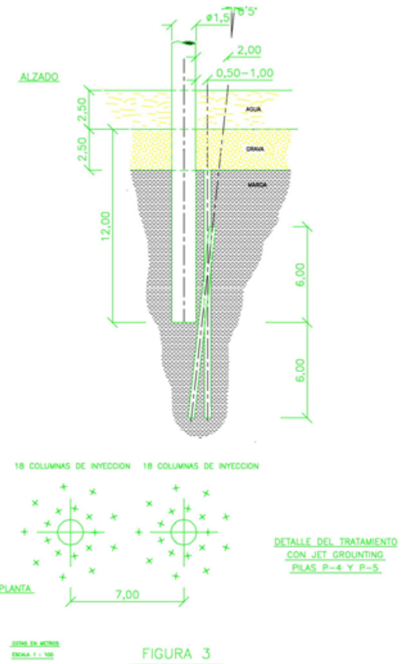

Figure 5. General drawing of the pier pile design

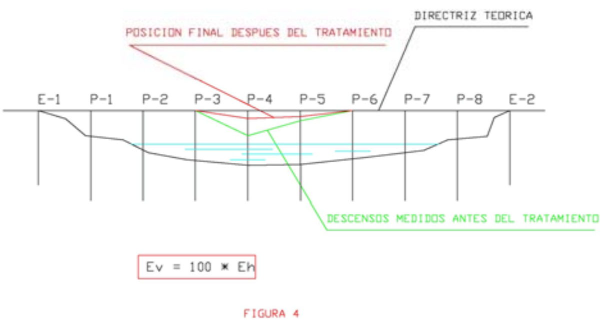

Figure 6. General drawing of the pier piles across the bridge crossing

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