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Seismic verification of foundation blocks in bridge structures

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ABSTRACT: the present paper studies a particular problem of soil-structure interaction which makes it possible to check the foundation block for bridge structures. This is made up of two phases: the first provides a method for calculating the springer reaction domain by analyzing the loads distribution under the assumption of a hinged arch; the second calculates the failure multiplier for the block-soil set by means of plasticity theorems under the assumption that the soil has a standard behaviour according to the Ziegler and Drucker's hypothesis.

The same algorithms allow the failure multiplier to be calculated during a seism, providing significant results which confirm the reliability of the results obtained by other authors.

1 INTRODUCTION

In the case of arch-structures, verification of foundation blocks is of considerable importance as these are responsible for discharging stresses acting on the structure to the soil. It is, therefore, essential to know the worst load conditions and hence determine the real safety coefficient. The springer reaction domain can be defined using the components R_x and R_y, then the crisis multiplier is calculated as the smallest allowed kinematic multiplier.

Let us, therefore, consider the parabolic arch hinged at the springers ,and let h(z) be the height of the generic section and I(z) the relative moment of inertia (fig. 1).

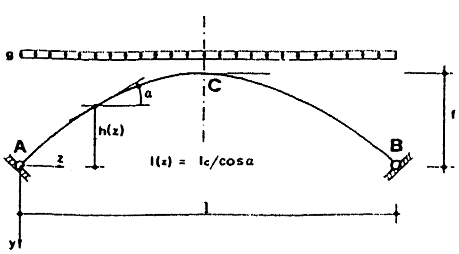

Fig. 1: Arch and springer reactions

In analytical terms:

$$h(z) = 4 f z(l - z) / l^2 \qquad I(z) = I_c / \cos(\alpha z) \quad (1)$$

where a(z) is the slope of the tangent axis and I_c is the moment of inertia of the cross section.

Below it is essential to know the line of influence of the drift for vertical travelling forces given by:

$$h^f(z) = 5\{z^4 - 2 l z^3 + l^3 z\} / (8 f l^3) \quad (2)$$

which is an expression calculated using the generalized Betti's principle.

From (2) it is possible to obtain its own weight reaction BD=R_g of components:

$$V_g = g\, l / 2 \qquad H_g = g\, l^2 / 8 f \quad (3)$$

Whereas, the seismic action generates in B a reaction R_{Br}, if coming from the left to the right, and a reaction R_{Bl}, in the opposite case. Given the hemisymmetrical nature of the load these reactions behaves isostatically.

Any heat variations generate a horizontal reaction R_{Bt}. If, Δt is the heat variation acting on the structure, by applying the virtual works principle, we get:

$$R_{Bt} = 15\, E\, I_c\, \alpha \Delta t / 8 f^2 \quad (4)$$

In order to determine the domain, the accidental load a must be also taken into account. For the latter an evenly distributed load in the most unfavourable position is assumed.

2 THE DOMAIN D_a

For the accidental load acting on the whole span the reaction value is $R_g a/g$, which is obtained by neglecting the thrust drop. Any two complementary load conditions 1 and 2 give rise to two reactions R_1 and R_2, such that their resultant is PQ (fig. 2).

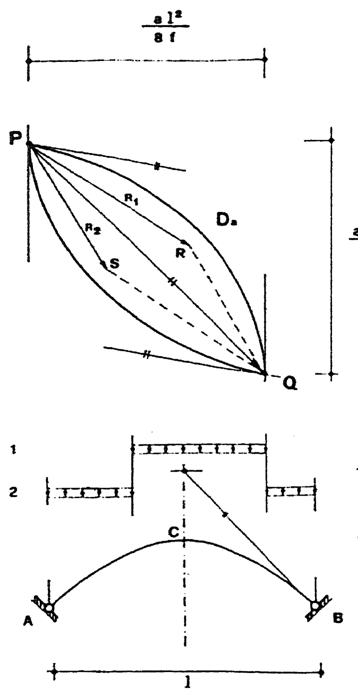

fig. 2: the domain Da

A load adz at the abscissa z causes in B a dR with components:

$$dV = a\,dz\,z/l \qquad dH = a\,dz\,h^f/l \tag{5}$$

and sloping on the horizontal by

$$\tan\beta = -\,z/(l\,h^f(z)) \tag{6}$$

From (6) it can be seen how for z = 1, it is tg ß = - a, ß = 90°. Whereas for z=0 is

$$\lim_{z\to 0}\ \tan\beta = -\,1/\left(l\,\frac{dh^f}{dz}\right) \tag{7}$$

Therefore for the accidental load extended over the whole span, a load variation of -adz must occur together with a slope of dR included in the interval 2

$$-\infty \le \tan\beta \le -\,1/(l\,\tan\varphi_A) \tag{8}$$

where φ_A is the slope in A of $h^f(z)$.

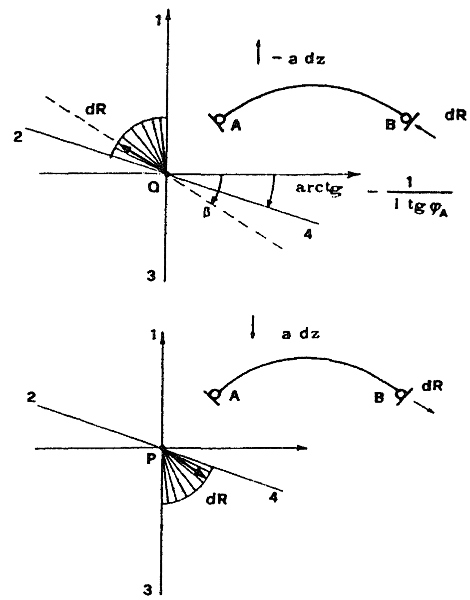

fig. 3: Variability field of the springer reaction

Eq. (8) corresponds, (fig. 3), to the two angles 1Q2 and 3Q4. However, as H < 0, dR in Q cannot be included in the angle 1Q2. A null accidental load corresponds to a null reaction and therefore a variation can only be a +adz. Repeating the above argument, we find that dR in P must be included in the angle 3P4. The tangents P and Q at the boundary of the domain D_a are thus defined.

Let us now consider the load condition in fig. 4a. A variation of this load may be either an elementary load -adz acting on the section (0,z), or a +adz in the section (a,1). The load -adz in (0,z) causes a dR contained in the angle (tUz). For a load -adz applied in z=0 and $z=z_a$ the dR slopes by the amount

$$\arctan(2z) \le -\,1/(l\,h(z)) \tag{10}$$

A load adz in (z,1) causes a dR contained in the angle (t'U3) in which t is the half line opposite t, if adz is applied in z = z_a, dR is directed according to the vertical line.

Any set of variations ±adz thus causes a dR moving towards the left of the half line t; it can be concluded that if PU is the reaction induced by the extended load from z = 0 to z = z_a, the varied reaction has its extreme at the left of the tangent equiversal to t through U.

Another load condition that gives a reaction directed according to r_1 can only associate a dR_1 with resultant US in the opposite direction s of the PU. This can be seen in fig. 4a considering that r_1 is outside the angle (tUz) and outside the angle (3Ut'). Therefore, PU is the maximum value that can obtain a reaction directed according to r_1.

In light of the above, it follows that the point U lies on the boundary of D_a and that the parallel to t through U is the tangent in U at D_a.

With analogous processes to those above, it can be concluded that the upper branch is related to the load distributions with its end in z = 0 while the lower branch is related to the load distributions

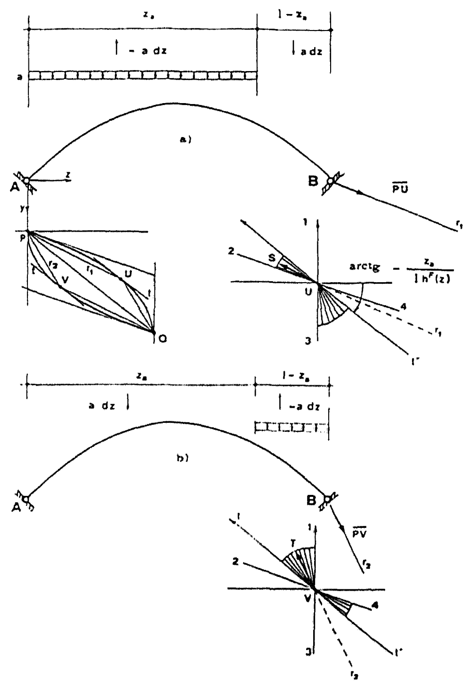

fig. 4 complementary load distributions

with its end in z = 0 while the lower branch is related to the load distributions with its end in z=1 (fig. 4b). Two complementary load distributions, such as those in fig. 4, correspond to two vectors PU and PV whose resultant PQ is the reaction due to the load extended over the whole span.

It is interesting to observe how the results reached can also be applied to clamped arches. It is well-known how for .non-smamm f/l ratios the springer reaction eccentricity are.negligible.

3 FAILURE MULTIPLIER

Once the springer reaction domain is known, the foundation block subject to the springer reaction can be studied by using the limit design methods for calculating the failure multiplier relative only to the increase in accidental loads. (Chen 1975, Franciosi 1979, Franciosi 1985). Let us consider the foundation block in standard soil (Drucker 1954), whose intrinsic curve is reduced to bilateral.

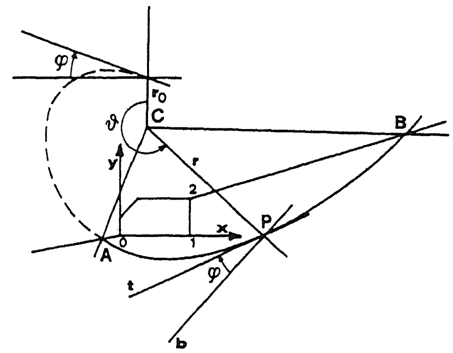

fig. 5: crisis curves

In these hypotheses, the slide curve is a logarithmic spiral with centre C and equation

$$r = r_o e^{\theta \tan\varphi} \qquad (10)$$

The shear mechanism motion is a rigid anti-clockwise rotation of the whole mass rotating around the centre C and is unilateral. The Lagrange equation for this can be written as follows:

$$\gamma_a L_a + L_p = -\int_A^B \tau_v \eta \, ds \qquad (11)$$

where L_a is the work performed by the R_a relative to the accidental loads and L_p is that of the permanent loads, including the weight of the mass and any loads acting on the outside surface, t_n represent the stresses acting on the slide curve with components (σ_n, τ_n). Eq.(11) comes from the rotation equilibrium equation around C of all the forces acting on the mass. in analytical terms:

$$\gamma_a M_a + M_p = M_s \qquad (12)$$

M_a and M_p represent the moments around C of R_a and the permanent loads. The moment M_a refers to the stresses t_n. For the latter we can write:

$$dM_s = (-\tau \cos\varphi + \sigma \sin\phi) r \, ds = -r\overline{OD} \, ds \qquad (13)$$

where $u_n = (\sin\varphi, \cos\varphi)^t$ is the versor of the normal to the bilateral in T'. Eq. (13) makes it possible to obtain M_s as a function of only the geometry of the problem:

$$M_s = -\int_{\theta_A}^{\theta_B} c\, r^2 d\theta = -c\, r_o^2 \int_{\theta_B}^{\theta_A} e^{2\theta \tan\varphi} d\theta \qquad (14)$$

and finally

$$M_s = \frac{c r^2}{2\tan\varphi}\left(e^{2\theta_B \tan\varphi} - e^{2\theta_A \tan\varphi}\right) \qquad (15)$$

with c being the soil cohesion. The γ_a taken from Eq. (11) refers to an arbitrary mechanism. In particular, it depends on the coordinates of c and the radius of r_o. The variables are reduced to 2 as a block vertex is certainly a point belonging to the crisis curve. Therefore, the problem is to calculate the lowest γ_a from all the possible m values by following the orthogonal gradient method (Ciarlet 1985), with the help of an automatic computation program.

4 SEISMIC EFFECTS

The failure multiplier of the block-soil set in the presence of a seism can be obtained in a static form by acting on the procedure of Mononabe (1929) and Okabe (1926) who hypothesized a flat slide surface. The passive resistance coefficient is obtained as the difference between the coefficient determined in the static phase and a decrease that takes the seismic action into account. Others, such as Jamiolkowsky (1986), substituted the static coefficient with the one calculated under the hypothesis of a logarithmic crisis curve. However, neither of the methods reach the real slide surface to which the minimum γ_a is associated.

The γ_a in the presence of a seism is calculated using the same procedure explained above while taking care to study the problem from a static point of view. All that is necessary is to take into account the acceleration due to the seism by means of a rotation of the soil-block equal to:

$$\alpha = \arctan[a_h / (g - a_v)] \qquad (16)$$

while the specific weights of the soil and the block become:

$$\gamma_t = \gamma_t / \cos\alpha \qquad \gamma_m = \gamma_m / \cos\alpha \qquad (17)$$

The rotation of the block is hypothesized with respect to the foundation vertex to which the lowest γ_a is associated.

5 NUMERICAL EXAMPLES

Let us consider the supporting arch of the Bloukrans bridge (S.A.) which has a span of 272 m, a parabolic rise of 62 m and is hinged on the springers. The section varies from 3.6 m at the crown to 5.6 m at the springers and the foundations are made up of independent truncated pyramid plinths. The reaction on the truncated pyramid is the sum of the constant aliquot due to the permanent loads (of components $H_p = 451$ t and $V_p = 457$ t), and the reaction corresponding to the accidental loads defined by the domain D_a. If the domain D_a and the geometric characteristics of the foundation block are assumed as input data, the failure multiplier $\acute{A}_a$ associated to every R_a can be calculated.

Tab. 1 shows the failure multipliers γ_a for different types of soil which highlight their variability as the internal friction angle varies.

tab.1

φ	c=10	c=2	c=0
0	1.15	-	-
5	2.06	-	-
10	4.94	-	-
15	7.72	2.52	1.36
20	11.96	5.45	3.78
25	17.65	10.60	8.78
30	-	18.93	16.66

tab.2

a_H	0	0.29g	0.5g
a_v	0	0.12g	0.25g
Caquot	8.17	-	-
Mononabe	10.14	7.24	3.69
S.G.I.	8.17	4.64	1.28
Authors	6.21	3.22	2.1

The same algorithms used above allow the anchorage block to be verified even in the presence of a seism as discussed in sect. 4. In tab. 2, for the anchorage block designed for the bridge over the strait of Messina (Jamiolkowski 1986) different values of γ_a are considered.

6 CONCLUSIONS

The aim of this paper is to provide a model for calculating the failure multiplier for some soil-structure interaction problems. In particular, the paper analyzes the passive resistance states developing behind the foundation blocks . This is limited to so-called standard soils with no stratum and, therefore, is one of the verification methods that must support direct investigation methods.

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