A PROCEDURE FOR THE DESIGN OF BRIDGE RAIL ANCHORAGE FOR THE INSTALLATION ON EXISTING STRUCTURES

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ABSTRACT

According to European standards a bridge rail can be installed on a public road only if it has successfully overcome a full scale crash test. The behavior of the restraint system is strongly affected by the characteristics of the anchoring system which, in the case of bridge rails, is generally realized by connecting the steel posts to the edge beam of the bridge deck by means of a given number of steel anchors.

The resistance of the anchoring system, on the other hand, is strongly affected by the bridge deck concrete slab characteristics. Therefore the Italian regulations require the designers to check if the barrier can be installed as it was in the crash test, given the specific concrete slab available on site. If not, the designer is required to modify the anchorage system to allow the connecting system to offer a resistance equal or higher than the one offered in the crash test. This problem becomes extremely important when a new bridge rail is installed on an existing structure, the structural and geometric characteristics of which can differ considerably from the ones of the structure used for the crash test.

This paper describes a procedure which has been applied to check the compatibility between the existing supporting structure and the bridge rail anchors, as used in the crash test configuration. A number of different design interventions which can be considered when the support doesn't allow the installation of the bridge rails as crashed are also described.

Keywords: safety barriers, bridge rail, anchorage design, on site installation

1. INTRODUCTION

After over 15 years from the publication of the first mandatory National regulation on the design and use of safety barriers on public roads (MinLLPP, 1992), dated February 1992, only in the very recent years this regulation has become really applicable after the approval, at the Ministry level, of a number of restraint systems.

According to the regulation, which has been updated several times since the first edition with the latest revision (MinIT, 2004) dated June 2004, a safety barrier can be installed on a public road only if it has successfully undergone a set of crash tests according to the European Standards EN1317 parts 1 and 2 (CEN, 1998 and CEN 1998a).

Since the first edition of the regulation it was clearly stated that there was the need for a “design of the installation” which had to define how the safety barriers had to be installed on site, given the specific support available. This is due to the well known fact that crash tests are conventional tests which are conducted under very peculiar conditions which might not reflect the conditions available on site.

In terms of bridge rails the major effects are related to the reduced width of the bridge edge beam as compared with the support slabs used for the crash test, and to its lower concrete strength.

The evaluation of the effect of these differences on the overall behavior of the barrier can be made by means of numerically simulating the crash in the two configurations (the one adopted in performing the crash tests and the one existing on site) and evaluating the differences (Biagini et al, 2007 and Bonin et al, 2004), but this type of application, very useful for research purposes, could be difficult to apply in practice for general design purposes.

There is therefore the need for a close form design procedure. This might be derived from the design methods usually applied in the reinforced concrete structures and anchoring systems fields.

2. THE DESIGN PROCEDURE

The design procedure proposed in this paper can be synthesized as follows:

A. defining the maximum resistance of the anchoring system adopted in the crash test configuration on the crash test support;
B. defining the maximum resistance of the anchoring system adopted in the crash test configuration on the existing support;
C. comparing the resistances defined above: if the maximum resistance on site is lower than the maximum resistance in the crash test, the anchoring system has to be changed in order to allow it to offer the same resistance as the tested solution or, if no suitable solutions can be designed, the reinforced concrete support structure has to be replaced with a new one.

The different parts of the procedure are described hereafter in details together with a sensitivity analysis to show the influence of the key parameters on the resistance offered by the system.

3. EVALUATION OF THE ANCHORING SYSTEM RESISTANCE

A procedure has been developed in order to quantify the resistance of a specific anchoring system realized with steel anchors. This is based on the EOTA "Guideline for European technical approval of metal anchors for use in concrete" (ETAG 001) with specific reference to Annex C "Design methods for anchorages" (EOTA, 2001).

For the specific application of the method to the evaluation of the resistance of the bridge rail anchoring system the loading action has to be defined. This has been considered as a combination of:

- a bending moment (M) which produces a tensile stress (N) in the front anchors (the ones installed towards the traffic);
- a shear action (V), acting on the outer anchors;

as shown in Figure 1

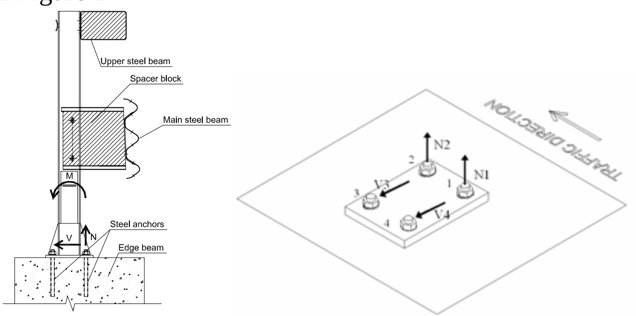


Figure 1 - Schematization of the actions on the bridge rail anchors

In the structural analysis of the anchoring system the following assumptions have been made:

- the shear resistance acts in a direction perpendicular to the traffic flow, as shown in Figure 1. This assumption is based on the observation of the crash test results on several bridge rails. Even though the direction of the vehicle just before the impact is at an angle with the traffic flow relatively low (typically 20°) the effect of the steel beam and of the spacer block is that the action on the post is almost perpendicular to the traffic flow, as can be seen by observing the deformation of the post after the crash;
- the analysis can be conducted with the ETAG001 approach, which is valid for static actions, even though the applied forces are in a dynamic domain. This assumption is considered applicable in the specific proposed procedure as the main aim is a comparison between the crash test resistance and the resistance of the anchoring system on site. Therefore the effect of a "dynamic amplification factor" should be smoothed out in making a comparison between the two resistances (both referred to the same dynamic loading condition).

To better investigate the latter aspect in order also to allow the use of the model to define the actual dynamic resistance of the anchoring system (and not only a comparison between the two different configurations) a specific research program is currently active. In this project single posts with different support conditions are being tested at the University of Florence full scale impact test facility. The results of this research project should allow to define a set of "calibration factors" to be applied to the ETAG Method to account for the effect of the dynamic loading.

The calculations that have to be made according to the ETAG method for agroup of anchors are:

- steel failure resistance;
- pull-out failure resistance;
- concrete cone failure resistance;
- splitting failure resistance.

In this specific application pull out failure has not been considered as for chemical anchors, as the ones considered in this study and mostly used for bridge rails, the concrete cone failure resistance is more severe, as stated also by the HILTI anchor's design guide (HILTI, 2004).

Splitting failure has not been considered as the ETAG procedure indicates that this can be omitted if the distance between the anchors (DL, Dt) is lower than the effective depth of the anchors (h_{ef}), as usually occurs for bridge rail anchoring systems and will always be the case in the applications shown hereafter.

Given a geometric configuration of the anchoring system, the type and size of steel anchors and the concrete slab geometry and resistance (as in the example shown in Figure 2), the tensile and shear design resistances (Nsd and Vsd) can be defined as described in the following sections based on the steel and concrete failure resistances determined by means of ETAG Method.

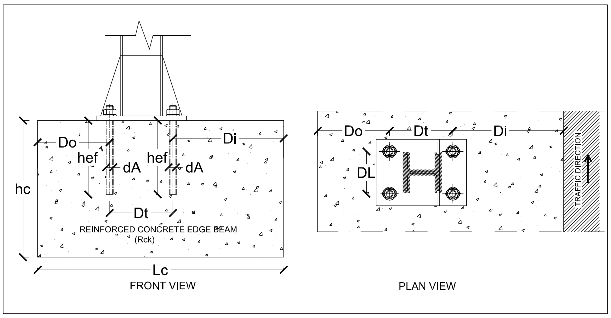


Figure 2 - Geometric description of the anchoring system and concrete support

3.1 Evaluation of the tensile resistance

The characteristic <u>tensile resistance</u> of a group of anchors can be determined as:

$$N_{Sd} = \min \begin{cases} \dfrac{\sum_{i=1}^{n_N} N_{Rk,s,i}}{\gamma_{Ms}} \\ \dfrac{N_{Rk,c}}{\gamma_{Mc}} \end{cases} \tag{Eq. 1}$$

where:

$N_{Rk,s}$ is the characteristic tensile resistance of a single anchor in the case of steel failure;

n_N is the number of tensioned anchors

$N_{Rk,c}$ is the characteristic tensile resistance of the group of anchors in the case of concrete cone failure;

γ_{Ms}, γ_{Mc} are safety factors with respect to steel failure which has been set equal to 1.12 and 1.6 respectively according to the Italian regulation on the design of reinforced concrete and steel structures.

$$N_{Rk,s} = A_s \cdot f_{uk} \quad [N] \qquad \text{(Eq. 2)}$$

where:

A_s is the area of the steel anchor which is characteristic of each given anchor used;

f_{uk} is the characteristic steel ultimate tensile strength (nominal value).

$$N_{Rk,c} = N^0_{Rk,c} \cdot \frac{A_{c,N}}{A^0_{c,N}} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{ec,N} \cdot \psi_{ucr,N} \quad [N] \qquad \text{(Eq. 3)}$$

where:

$$N^0_{Rk,c} \;(\text{characteristic resistance of a single anchor in a cracked concrete}) = = 7.2 \cdot \sqrt{Rck} \cdot h_{ef}^{1.5} \quad [N] \qquad \text{(Eq. 4)}$$

where:

h_{ef} is the effective anchorage depth [mm];

Rck is the characteristic concrete compression strength [N/mm^2].

$A^0_{c,N}$ (influence area of a single anchor in an indefinite slab) = $(2 \cdot c_{crN})^2$ (Eq. 5)

For typical steel anchors c_{crn} can be assumed as $1 \cdot h_{ef}$.

$A_{c,N}$ (area of influence of the group of anchors).

This has to be calculated according to the actual geometry of the anchoring system, accounting for the overlapping of the single anchors' area of influence, and the support slab size as shown in Figure 3. If the concrete slab is "indefinite" in width (part "a" of Figure 3) the area of influence can be defined as:

$$A_{c,N} = (2 \cdot c_{crN} + D_L) \cdot (2 \cdot c_{crN} + D_T) \qquad \text{(Eq. 6)}$$

where:

D_L is the longitudinal distance between the two lines of anchors, as shown in Figure 2;

D_T is the transversal distance between the two lines of anchors, as shown in Figure 2.

If the distance between the concrete slab edge and the anchors is narrower than c_{crn} on one side (part "b" of Figure 3) the area of influence becomes:

$$A_{c,N} = (2 \cdot c_{crN} + D_L) \cdot (c_{crN} + D_T + D_i) \quad \text{or} \quad A_{c,N} = (2 \cdot c_{crN} + D_L) \cdot (c_{crN} + D_T + D_o)$$

depending if the narrower edge is the inner (the distance of which from the anchors is D_i) or the outer (at a distance D_o from the anchors).

If the distance between the concrete slab edge and the anchors is narrower than c_{crn} on both sides (part "c" of Figure 3) it becomes $A_{c,N} = (2 \cdot c_{crN} + D_L) \cdot (D_T + D_i + D_o)$.

In eq. 3 the following symbols are also used:

$$\psi_{s,N} = 0.7 + 0.3 \cdot \frac{\min(D_i,\ D_o)}{c_{cr,N}} \ \leq\ 1 \qquad \text{(Eq. 6)}$$

$\psi_{re,N} = 0.5 + \frac{h_{ef}}{200} \leq 1$ The shell spalling factor, $\psi_{re,N}$, takes account of the effect of the reinforcement. If in the area of the anchorage there is a reinforcement with a spacing ≥150mm (any diameter) or with a diameter ≤10mm and a spacing ≥100mm then a shell spalling factor of $\psi_{re,N} = 1.0$ may be applied independently of the anchorage depth. For anchors with an effective depth not lower than 100 mm (which is usually the case for bridge rail anchors), independently of the actual reinforcement spacing, $\psi_{re,N} = 1.0$.

$\psi_{ec,N} = 1$ under the simplified assumption, proposed by the ETAG procedure, of considering all the tensioned anchors having the same resistance. The resistance of each tensioned anchor is therefore defined as:

$$N^{h}_{Rk,c} = \frac{N_{Rk,c}}{n_N} \qquad \text{(Eq. 7)}$$

$\psi_{ucr,N} = 1.4$ considering that the cement concrete is not cracked. If the concrete is cracked a value of 1 should be used.

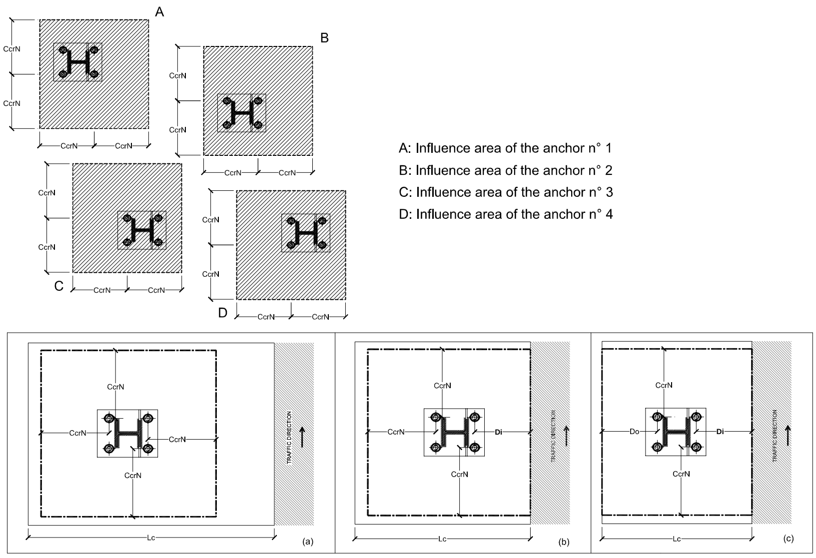


Figure 3 - Definition of the anchors group influence area for tensile strength

3.2 Evaluation of the shear resistance

The characteristic shear resistance of a group of anchors can be determined as:

$$V_{Sd} = \min \begin{cases} \dfrac{\sum_{i=1}^{n_V} V_{Rk,s,i}}{\gamma_{Ms}} \\ \dfrac{V_{Rk,c}}{\gamma_{Md}} \end{cases} \qquad \text{(Eq. 8)}$$

where:

$V_{Rk,s}$ is the characteristic shear resistance of a single anchor for steel failure;
n_V is the number of anchors that are devoted to bear shear actions
$V_{Rk,c}$ is the characteristic shear resistance of the group of anchors in the case of concrete cone failure.

$V_{Rk,s} = 0.5 \cdot A_s \cdot f_{uk}$ $[N]$ (for typical bridge rail anchors which have rupture elongation > 8%) (Eq. 9)

$$V_{Rk,c} = V^0_{Rk,c} \cdot \frac{A_{c,V}}{A^0_{c,V}} \cdot \psi_{s,V} \cdot \psi_{h,V} \cdot \psi_{\alpha,V} \cdot \psi_{ec,V} \cdot \psi_{ucr,V} \; [N] \qquad \text{(Eq. 10)}$$

$$V^0_{Rk,c} \; (\text{characteristic resistance of a single anchor in a cracked concrete}) = \qquad \text{(Eq. 11)}$$
$$= 0.45 \cdot \sqrt{d_{nom}} \cdot \left(\frac{h_{ef}}{d_{nom}}\right)^{0.2} \cdot \sqrt{Rck} \cdot D_o^{1.5} \quad [N]$$

where:

d_{nom} is the nominal diameter of the steel anchor
$A^0_{c,V}$ (influence area of a single anchor) = $4.5 \cdot (D_o)^2$, defined considering a 53,3° distribution of the tensions from the anchors to the slab edge.

$A_{c,V}$ (area of influence of the group of anchors). Considering that, according to the ETAG method, only the outer anchors should be considered as resistant to shear actions (as described earlier), the area of influence can be defined as (Figure 4):

$A_{c,V} = \left(2 \cdot (1.5 \cdot D_o) + D_L\right) \cdot 1.5 \cdot D_o$, if the slab thickness (h_c) is greater than 1.5 D_o,

or

$A_{c,V} = \left(2 \cdot (1.5 \cdot D_o) + D_L\right) \cdot h_c$, for thinner slabs.

These equations are valid in the general case when 2 anchors are placed in the outer alignment. If only one anchor is used instead the single anchor equation has to be used.

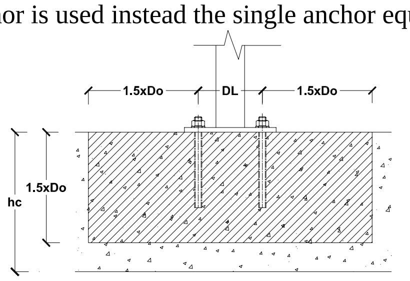


Figure 4 - Definition of the anchors group influence area for shear strength

$\psi_{s,V} = 1$ if anchors are not placed in the slab corners (at a distance lower than $1.5 \cdot D_o$ from the lateral edge of the slab);

$\psi_{h,V} = 1$ if the concrete slab thickness (h_c) is greater than $1.5 \cdot D_o$. For thinner slabs the following equation should be used: $\psi_{h,V} = \left(\frac{1.5 \cdot D_o}{h_c} \right)^{\frac{1}{3}}$

$\psi_{\alpha,V} = 1$ under the assumption, discussed earlier, that the shear action can be considered perpendicular to the traffic direction (which means perpendicular to the outer concrete slab edge);

$\psi_{ec,V} = 1$ under the simplified assumption, proposed by the ETAG procedure, of considering all the sheared anchors having the same resistance. The resistance of each sheared anchor is therefore defined as: $V^h_{Rk,c} = \frac{V_{Rk,c}}{n_V}$.

$\psi_{ucr,N} = 1.4$ considering that the cement concrete is not cracked and the slab is provided with edge steel reinforcement and closely spaced stirrups. If the concrete is cracked a value of 1 should be used;

Considering the distribution of the loading actions shown in Figure 1 each anchor has a specialized function with the inner ones resisting to the tensile actions and the outer ones devoted to resist to the shear actions. There is therefore no need to check for the resistance of a given anchor to combined tensile and shear actions. This assumption, based on the ETAG schematization of loading, is valid in the typical bridge rail anchoring configurations where the anchors hole in the steel plate connected to the steel post are either all slotted or not slotted at all. If the outer holes are slotted and the inner are not this assumption is not valid anymore as the inner anchors bear both the shear and tensile stress.

4. SENSITIVITY ANALYSIS OF THE INFLUENCE OF THE DIFFERENT PARAMETERS ON TENSILE AND SHEAR STRENGTH

As discussed earlier the anchoring system configuration on site can be considerably different from the one adopted during the crash test. The main differences are usually:

- a different concrete characteristic resistance of the reinforced concrete slab (Rck);
- a different distance between the outer alignment of anchors and the outer edge of the concrete slab (D_o);
- a different distance between the inner alignment of anchors and the inner edge of the concrete slab (D_i).

A sensitivity analysis has therefore been conducted to show the influence of these three parameters on the tensile and shear strength. Figure 5 shows the influence of changing D_o and D_i for different Rck values on the design tensile resistance while Figure 6 shows the influence of D_o on the shear strength resistance.

For this application the anchoring system is realized with 4 chemical anchors type M24, the properties of which can be found in HILTI, 2004, placed at a transverse distance (D_T) of 180 mm between them and a longitudinal distance (D_L) of 140 mm. In the same figures the design resistance of the crash test configuration “”reference condition”) is indicated with an “A”. The reference condition has the following characteristics:

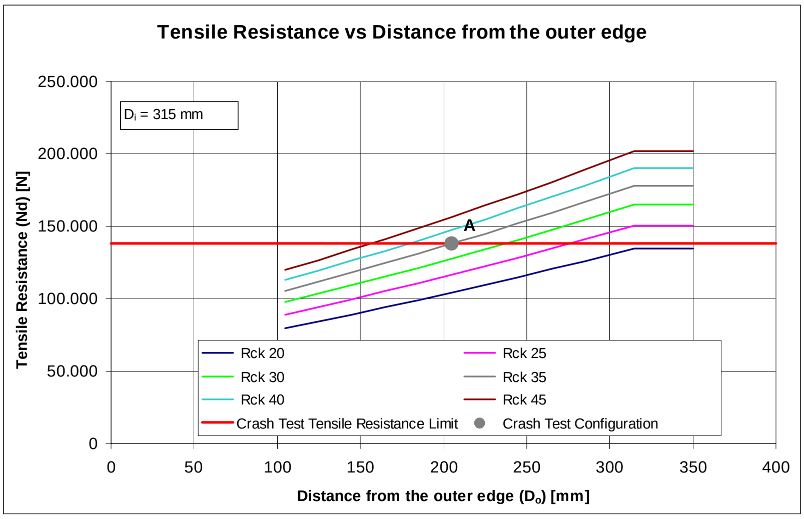
- Rck = 35 N/mm^2;
- D_o = 205 mm;
- D_i = 315 mm;
- total width of the concrete slab = 700 mm.

For this application the minimum distance between the anchors and the concrete slab edges has been fixed to 105 mm, as required by the HILTI anchors design manual (HILTI, 2004) for type M24 anchors. The minimum concrete characteristic resistance (Rck) has been set to 20 N/mm^2, as recommended by the ETAG procedure (EOTA, 2001) and the maximum to 45 N/mm^2. Note that in Figure 5 and Figure 6 the total width of the supporting concrete edge beam has to increase when the considered variable (D_o or D_i) increases.

Based on the results shown in Figure 5 and Figure 6 the following preliminary conclusions can be drawn:

- the minimum distance required by the HILTI method for a specific type of anchors (105 mm in the specific case) might be insufficient to install the bridge rail anchors even if the concrete characteristic resistance is increased as compared to the one used in the crash test, due to a lack in tensile strength;
- if the characteristics resistance of the concrete of the existing bridge deck is lower than the one used in the crash test the same anchoring system can be applied if a wider distance from the outer edge (D_o) is provided (see Figure 5, left). In the given example a concrete with an Rck of 25 N/mm^2 can still be used if a value of D_o of at least 280 mm is provided, given the other parameters remain as they were in the crash test;
- the influence of the edges is limited to a distance from the anchors equal to c_{crN} which means, in the given example, 315 mm. This leads to the conclusions that if the distances from the edges used in the crash test are wider than this value these can be reduced to the value of c_{crN} without affecting the structural behavior of the anchors;
- the influence of the concrete characteristics is extremely limited on the shear resistance of the anchors which, on the other hand, is strongly affected by the distance from the outer edge. According to the ETAG procedure the check for the concrete resistance to shear actions may be omitted for distances to the outer edge wider than $10 \cdot h_{ef}$, which means, in the given example, 2100 mm.

As a matter of fact, for the typical configurations of bridge rail anchors, the effect of the distance to the outer edge becomes almost non relevant much before the value proposed by ETAG. As it can be seen in Figure 6 for a value of D_o greater than approx. 300 mm the shear increases suddenly. It should be kept in mind, anyhow, that the calculations are made assuming the slab thickness used in the crash test (1200 mm). If the slab thickness is lower than 1.5·D_o the shear resistance should be calculated considering the effect of the slab thickness as shown in the procedure described above.



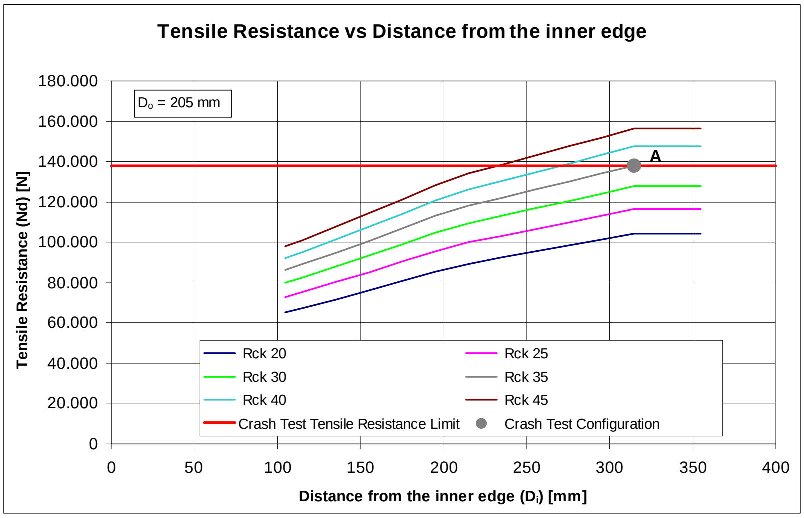


Figure 5 - Influence of Rck and D_o (upper) and D_i (lower) on tensile resistance

In practical applications it should be noted that the concrete edge beams on which the barriers have to be installed have a fixed width. Therefore the increase in the value of the distance to the outer edge (D_o), that allow for an increased tensile strength, leads to a decrease in the distance to the inner edge (D_i) that, on the other hand, reduce the tensile strength. The diagram of Figure 7 shows that, for the given support width of 700 mm, there is an optimal position in terms of distance to the outer edge that leads to the maximum tensile strength and only a limited set of D_o values, depending on the value of Rck, that lead to tensile resistances greater or equal to the crash test tensile resistance.

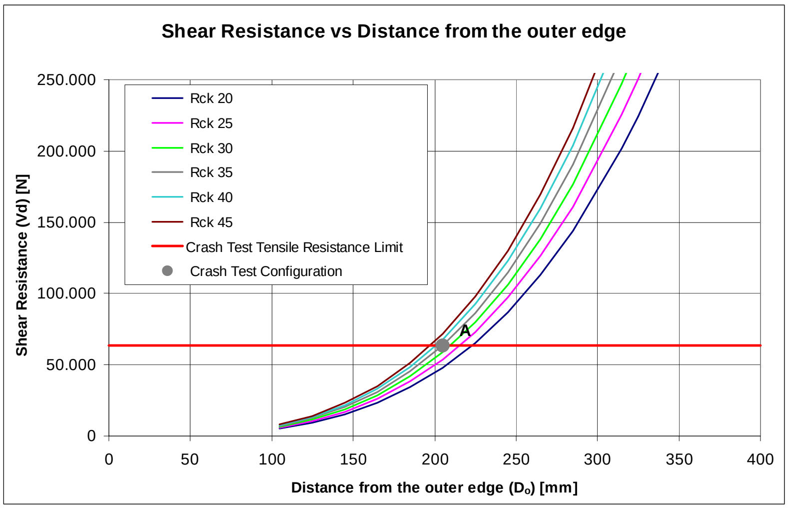


Figure 6 - Influence of Rck and D_o on shear resistance

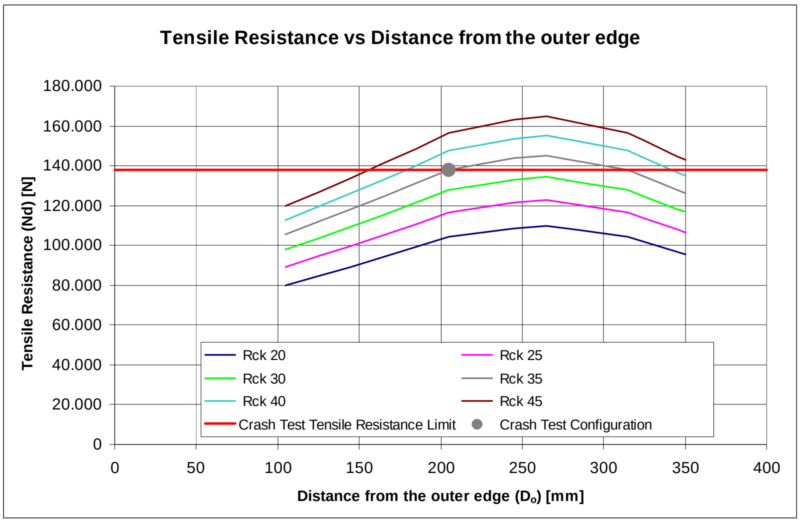


Figure 7 - Influence of Rck and Do on tensile resistance (support width=700 mm)

5. DESIGN INTERVENTIONS ON EXISTING BRIDGE DECKS

If an existing bridge deck has an edge beam where the barrier has to be installed that do not allow the installation of the barrier as it was in the crash test there are a number of possible design interventions that the designer can consider before replacing the whole bridge deck edge beam.

Among the different possible solutions the following are analyzed below:

- modifying the position of the barrier with respect to the support width;
- increasing the size of the anchors;
- increasing the number of anchors.

The following applications are referred to the same anchoring system used for the sensitivity analysis but installed on a concrete support width a fixed width of 500 mm (instead than the 700 m used for the crash test).

5.1 Modifying the position of the barrier with respect to the support width

The first thing that the designer of the barriers installations can do is to check which is the optimal position of the barrier that provide the highest resistance to shear and tensile actions. This type of intervention can be applied by the designer after the following design checks:

- the shear strength for the D_o value adopted has to be at least equal to the one calculated for the crash test configuration (see Figure 6);
- the position of the barrier with respect to the traffic flow has to be compatible with the minimum acceptable width of the adjacent shoulder.

For the specific application discussed in this section (with an edge slab of 500 mm) the analysis of the tensile resistance (Figure 8) shows that the given support is not suitable for anchoring the specific bridge rail as it is independently of the type of concrete used. There is no position of the outer anchors that provide a tensile strength equal to the crash test configuration tensile resistance.

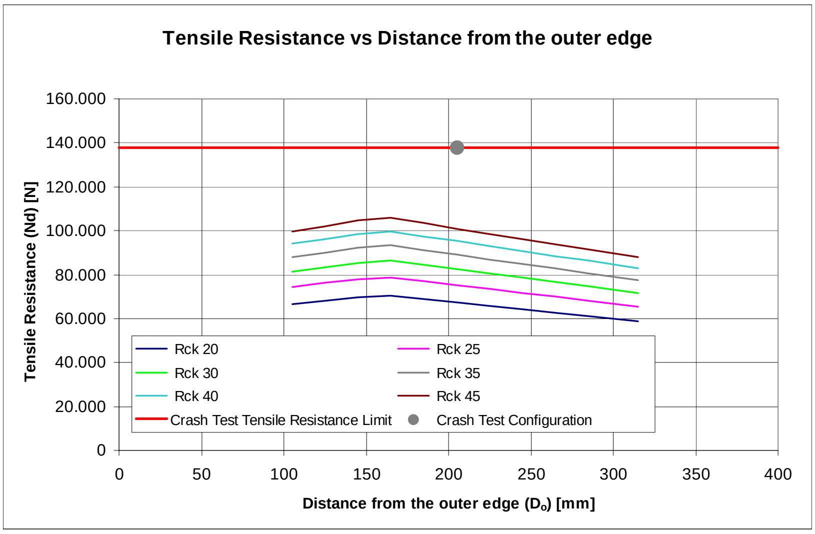


Figure 8 - Influence of Rck and D_o on tensile resistance (support width=500 mm)

5.2 Increasing the size of the anchors

The use of increased anchors with respect to the ones used in the crash test can be considered only if the thickness of the edge beam of the bridge deck is compatible with the new anchors.

The minimum concrete thickness depends on the length and diameter of the anchor considered and is defined for each specific type of anchor (as a possible reference the HILTI anchors design manual can be considered to set the minimum slab thickness).

It should be noted, anyhow, that the influence of increasing the anchors size is rather limited and tends to reduce considerably reducing the concrete slab width. Figure 9 shows the variation in the tensile strength when changing the anchors' type from the M24 used in the crash test (in solid lines) to an M30 (in dashed lines) on the 500 mm wide concrete slab. The variation in strength appears really limited.

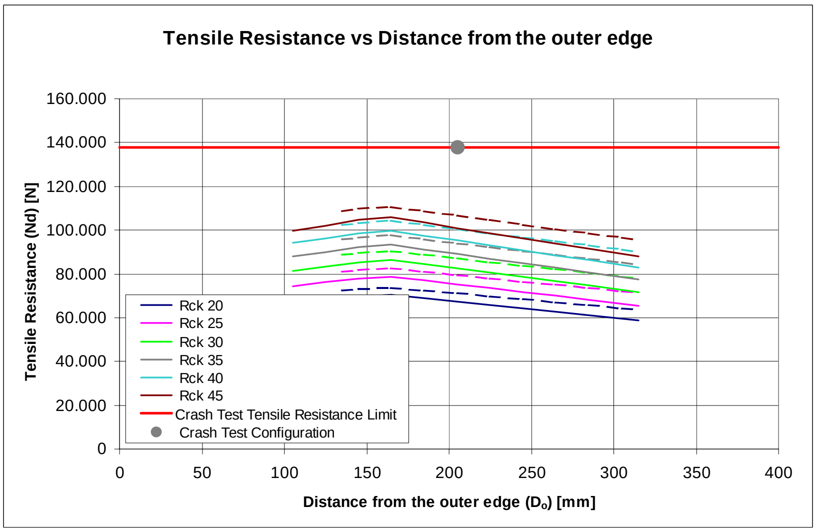


Figure 9 - Influence of Rck and D_o on tensile resistance (support width=500 mm) with anchor types M24 (solid lines) and M30 (dashed lines).

5.3 Increasing the number of anchors

Another option for increasing the anchoring system strength is to increase the number of anchors. This type of intervention is more complicated as it requires to increase the size of the steel plate linked to the steel post of the bridge rail, as shown in Figure 10, where an 8 anchors configuration is considered with the new set of anchors placed on the same alignment of the original ones.

In this configuration the same equations described earlier can be used to determine the tensile and shear resistance by substituting the value of DL with DL+2·DL1, where DL1 is the distance between the added anchors and the ones used in the crash test.

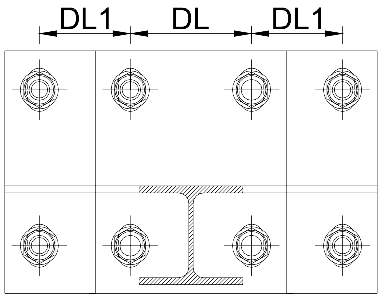


Figure 10 - Anchoring system with increased number of anchors (from 4 to 8).

The effect of this type of intervention is relevant compared to the previous one, as shown in Figure 11. It is referred to the condition where the distance DL1 is set equal to the value of DL (= 140 mm). The effect of this intervention increases when increasing the value of DL1 and the minimum value of this parameter should therefore be set based on:

- the value of tensile and shear resistance that has to be obtained (Nd, Vd);
- the optimal position of the anchoring system within the available width of the supporting concrete beam;
- the characteristic resistance of the concrete (Rck).

In the example shown, the DL1 value of 140 mm is the minimum allowable value for a supporting beam having a width of 500 mm, provided a concrete mix with a characteristic strength of Rck 45 is used and a D_o value between 180 and 200 mm is adopted.

Infact the latter range of D_o values allow to have a tensile resistance of at least 140 kN and, simultaneously, a shear resistance of at least 63 kN.

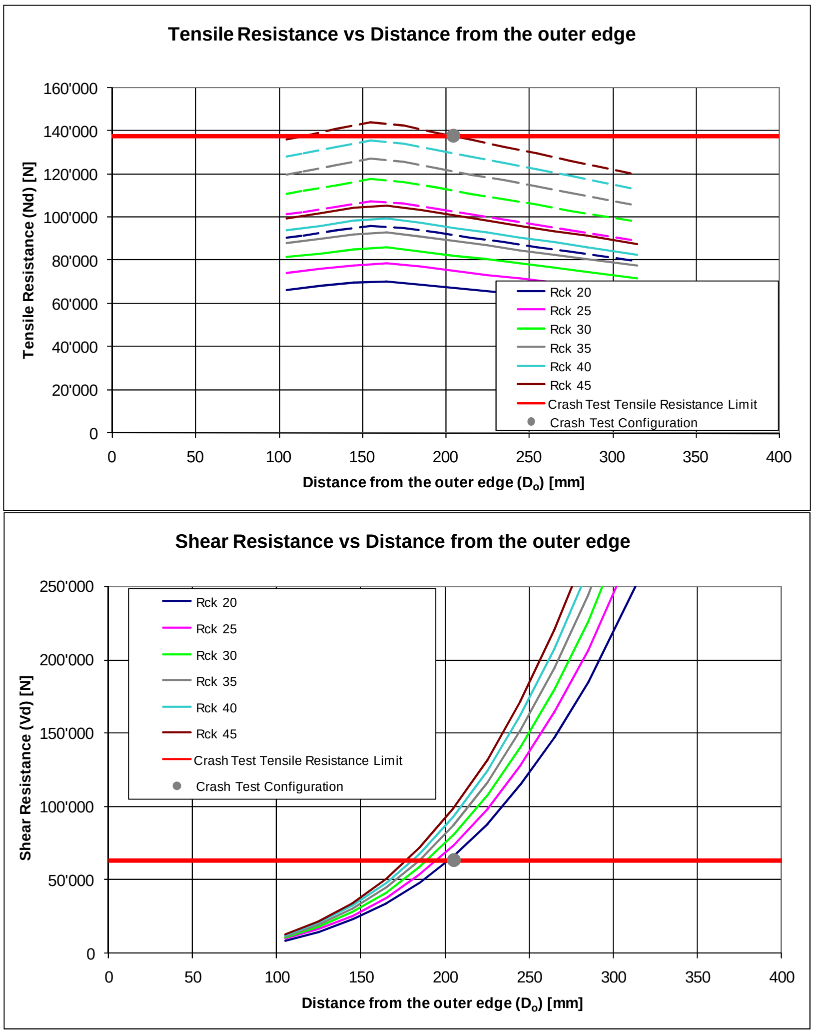


Figure 11 - Influence of Rck and D_o on tensile resistance (upper) and shear strength (lower) (support width=500 mm) with 4 anchors (solid lines) and 8 anchors (dashed lines)

6. CONCLUSIONS

The installation of bridge rails on existing bridge structures requires the designers to check if the anchoring system used in the crash test can provide, on site, a resistance equal or higher than the one provided by the anchoring configuration used during the crash test.

A close form procedure has been proposed to allow the designer to perform these checks and to design interventions to be applied to the anchoring configuration if the available resistance is lower than the required one.

The sensitivity analysis conducted on a specific bridge rail tested on a concrete slab support 700 mm wide and made with a concrete with a characteristic strength Rck of 35 N/mm^2 has shown that the most important parameter for both the tension and shear resistance is the distance of the outer anchors to the outer edge of the concrete slab.

The concrete strength has a considerable influence on the tensile strength while it is almost non relevant for the shear strength.

As far as the design of different type of interventions to increase the strength of the anchoring system is considered it has been shown that the increase in the distance to the outer edge might not be sufficient if the total width of the support slab is limited. An increase in the size of the anchors has a very limited effect on the tensile strength while the increase in the number of anchors has a considerable effect that increases for wider distances between the additional anchors and the ones used in the crash test. This distance should therefore be designed based on the actual slab size, the distance to the outer edge and the concrete characteristics in order to minimize the size of the extension to the steel plate supporting the steel post.

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