LOCAL EFFECTS OF COUPLING OF PRESTRESSING TENDONS IN STRUCTURAL JOINTS

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Abstract: *The increasing propensity for building continuous p.c. structures leads to the need of using in an extensive way prestressing tendons couplers and/or anchorage devices, which are quite often located on construction joints. The presence, in those regions, of high compressive stresses, localized in narrow areas and variable during the evolution stages, gives rise to non planar deformation states on the coupling surfaces.*
These deformation systems also change in time because of differential creep due to the high spatial variability of the stress state.
It is well known that in such conditions wide cracks are likely to rise along construction joints deeply sapping the structure durability, leaving the prestressing tendons exposed to aggressive agents attacks too.
A satisfactory performance level in these zones is achieved by means of a careful evaluation of the stresses evolution and the design of a proper ordinary reinforcement to control cracks width. A second ruling variable can be the total compression acting on the joint by effect of continuous tendons .
The aim of the numerical analysis carried out is the investigation of the stress and strain fields in the zones near the anchorages and the couplers in a two-dimensional field.
A non-linear finite element model able to predict cracking will be illustrated and design aids will be given regarding the placing of ordinary reinforcement, both for quantity and geometry.

1. STATE OF THE ART

Modern building techniques of prestressed bridges ask for the use of tendons couplers and/or the splitting of prestressing in regions inside the structure.

High stresses localizations, bound to evolve in time according to the construction sequence and to concrete creep, arise within the structure as a consequence of this construction procedure. In the same regions take place imposed deformation systems that can lead, combined with the other external actions, to the opening of the joints.

We will analyze some typical cases underlining the effect of mechanic and geometric variables that, with the building sequence, significantly influence the structural response.

Theoretical and experimental analysis of the stress and strain state nearby tendons couplers has been approached in the past by a relevant number of researchers, such as Stone & Breen ([1] & [2]), leading to the definition of consolidated design rules, quoted in the most recognized design codes such as the Model Code 90 [3], the A.A.S.H.O. specification for highway bridges [4], the A.C.I. code for buildings [5] and the Eurocode 2 [6].

On the other hand, the studies of the effects induced on the structures by tendons couplers and anchorages, and the following relapses in the world of structural design, even if empirically started in the past by Leonhardt [7], still find a open debate in the scientific field [8], [9], [10], [11], [12].

The importance of the subject was also remarked by the inspection results on the bridges with coupled tendons carried out in Germany and presented in a report of the German Infrastructures Govern Department [13]. In fact a frequent presence of wide cracks in regions nearby couplers, often followed by corrosion and damage of prestressing tendons, was reported in this document.

The current lack of a throughout study able to describe the phenomenon complexity has brought to the indication of merely empirical rules to control the problem, even in the most developed international design codes like Eurocode 2-2 [14].

The screening of the bibliography has highlighted the following weaknesses in the previous researches:

- Disregard of real construction methods, related to a clearly identifiable temporary sequence in a reference time interval.
- A not systematic analysis of the effects of concrete creep, which, in general, in presence of imposed deformations can damp the peak values of stresses evaluated by means of linear elastic analysis. In the papers of Stone ([1] & [2]) creep is taken into account, but only with regard to the local effect of prestressing loss between the tendon and the coupler, whose magnitude, numerically evaluated, turned out to be sensibly reduced in reality.

2. ANALYSIS LINEOUT

This paper aligns itself with the work done by the same authors [15] & [16], where a simple structure was studied in the hypothesis of linear elasticity and linear creep.

The same elementary model is now studied, made of the combination of two concrete blocks, 3m long, 2m high and 0.45m thick each, as shown in figure 1.

The tendon anchorages or the coupling devices are placed on the interface between the two blocks. The presence of mechanical non-linearity of both concrete and steel and combination of creep and shrinkage is now taken into account.
The choice of such a simple model is grounded on the idea of examining the case of tendons coupled or anchored in the webs of a box shaped bridge beam; for easiness of work, the analysis affects only a portion of the web, as it could be completely disjointed from the structure.
In the real structure stresses and strains depends on the whole cross-section geometry. Nevertheless, as the aim of the work is to study in detail the effects of the presence of coupling devices, we consider only the actions induced by the prestressing forces, supposed to be constant in time and not subjected to the classical losses.
We also neglect the effects of other permanent and variable actions. A justification of such starting hypothesis is the advisability of analyzing in a separate way the phenomenon of interest avoiding mixtures with other well known ones and the localization of couplers in regions where the internal actions induced by permanent actions have little magnitude.

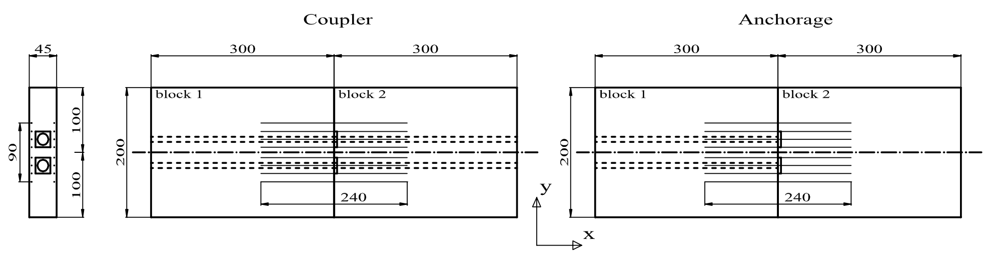
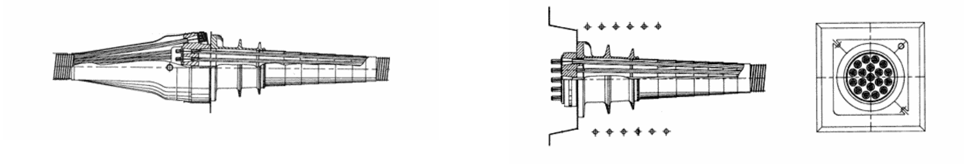


Figure 1: Model geometry

The structure is realized with a f_{ck} 35 MPa concrete and is supposed to be prestressed with two baricentric tendons made of fifteen 0.6” strands stressed to 1350 MPa for a total of 2815 KN per tendon, that would generate an average compressive stress in concrete of 6.25 MPa with no ordinary reinforcement.
Anchorages and couplers are both modelled as squared 245x245mm steel plates 50mm thick. A vertical distance of 160mm is granted between the plates. Such scheme is a good approximation of the geometric dimensions and layout provided by tendons producers. Prestressing load is seen as a uniform distributed pressure on the iron plates.
The following simplificative hypothesis are also made:

1. The presence of the cables inside the walls is neglected. No effects of the tendon sheets and/or the grouting are taken into account.
2. The problem is analyzed in plane stress, so the effects of the dispersion of the prestressing force in the thickness of the web and the confinement provided by the confinement reinforcement is neglected.
3. Concrete is assumed to have a linear behaviour in compression. This is verified almost everywhere apart from a small region behind the iron plates. This assumption may lead to a stiffer behaviour of the structure but, as confinement along Z axis is also neglected, the two effects may balance themselves.
4. Creep is assumed linear even where concrete stress is above the conventional limit of $0.45f_{ck}$. Right under the anchor plates a theoretical compressive stress of 47 MPa may

be reached, but in reality the flanges that can be seen in figure 2 help to noteworthy reduce this value. In a section 100mm away from the plates the theoretical stress decreases to 24MPa by the only effect of dispersion, without taking into account the actual geometry of the anchorage or coupling device.



a) coupler b) anchorage

Figure 2: Coupling and anchorage devices

Three scenarios are studied:

1. Anchorage: both tendons are anchored in the section, and the second block is jointed to the first one only by ordinary reinforcement.
2. Half coupling: both tendons are coupled, but only half of the prestressing force is transferred to the second block.
3. Full coupling: both tendons are coupled, and the full prestressing force is transferred to the second block.

A web of ordinary reinforcement, made of two layers of ϕ14/20cm, both in X and Y direction, is placed in each block. These bars do not cross the joint and are placed to simulate a minimum geometrical reinforcement ratio of 0.34%.

Three different dispositions of ordinary reinforcement lying across the joints are studied:

1. No ordinary reinforcement across the joint
2. Sixteen ϕ14/14cm bars, 240cm long each (120cm in each block) roughly equispaced in a zone spreading from 150mm below the lower iron plate to 150mm above the upper plate and providing an additional 0.66% geometrical reinforcement ratio in the controlled zone.
3. Sixteen ϕ16/14cm bars, placed in the same way and providing an additional 0.75% geometrical reinforcement ratio.

For what concerns the tensioning sequence the following hypothesis are taken (see figure 3): named as t=0 the casting time of the first block of the web, it is tensed at time t_1=14days; the second block of the web is cast at time $t_2 = t_1$+14days = 28days, whereas it is tensed at time $t_3 = t_2$ + 14days = 42days if we are in the coupling scenario.

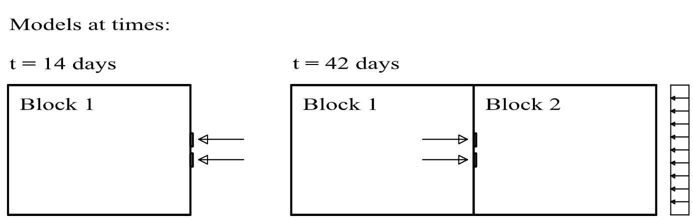


Figure 3: Coupling tensioning scheme

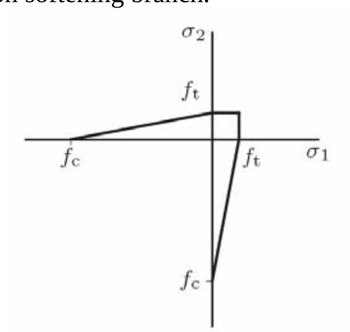
3. MODEL DESCRIPTION

3.1 Mesh and elements description

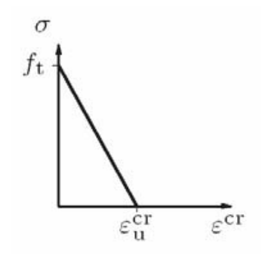
The two web portions are modelled with eight nodes quadratic serendipity plane stress elements with a regular mesh spacing of approximately 120mm.
The grid of ϕ14/20cm bars is introduced as embedded reinforcement, that is to say a virtual stiffness contribution to the concrete elements along the bars direction.
An interface surface, characterized by suitable contact properties and meshed with isoparametric three nodes quadratic line elements, is interposed between the two concrete blocks. Reinforcing bars crossing this joint are meshed using truss elements and are rigidly linked to the concrete elements in the edge nodes.

3.2 Materials properties

Each material is described with its average mechanical characteristics.
Concrete is assumed to be an isotropic elastic material in the uncracked state and to have a linear elastic behaviour in compression after cracking. The smeared crack approach is followed. Figure 4(a) shows the material failure surface in tension and figure 4(b) the linear tension softening branch.



a) failure curve



b) tension softening

Figure 4: Concrete behaviour in tension

Concrete compressive strength, tensile strength, and modulus of elasticity at 28 days are taken in accordance to the Model Code 90: f_{ck} = 35 MPa, f_{cm} = 43 MPa, f_{ctm} = 3.23 MPa, E_c=31100 MPa, ε_{cr}=1.04E-4. Time effect on each parameter is taken into account according to MC90 as figured in equation (1).

$$f_{cm}(t) = \beta_{cc}(t)\cdot f_{cm}(28d) \quad f_{ctm}(t) = \beta_{cc}(t)\cdot f_{ctm}(28d) \quad E_c(t) = \beta_E(t)\cdot E_c(28d) \tag{1}$$

The ultimate deformation at which no residual tensile stress is available, ε_u^{cr}, is set equal to eight times the elastic deformation at cracking ε_{cr}. The shear stiffness in cracked state is reduced by a shear retention factor β_g = 0.2 as shown in equation 2.

$$G^{cr} = \frac{\beta_g}{1-\beta_g} G^{el} \tag{2}$$

Creep and shrinkage are also formulated in accordance to MC90.
The constitutive equations of the interface surface are shown in expression (3)

$$t_n = D_n \cdot \Delta u_n \qquad t_s = D_s \cdot \Delta u_s \tag{3}$$

where the subscripts *n* and *s* identify the directions normal and tangential to the surface; t_n and t_s are the forces per unit length and Δu_n and Δu_s are the displacements in the directions *n* and *s* of the nodes that were coincident at the beginning of the analysis.
The surface is modelled as rigid-perfectly-brittle in direction *n* and rigid-elastic in direction *s*. The value at which cracking occurs is the cohesion *c* between the two castings ad is evaluated (see equation (4)) in accordance to the design prescriptions of MC90 for shear joints.

$$c = \beta_c f_{ctm} \qquad where \quad 0.2 \le \beta_c \le 0.4 \tag{4}$$

The stiffness D_n and D_s can then be expressed as follows:

$$\begin{aligned} &D_n, D_s \to \infty && if\ \sigma_n < c \\ &D_n = 0, D_s = G^{cr} \cdot t && if\ \sigma_n > c \end{aligned} \tag{5}$$

Steel is assumed elastic-plastic with isotropic hardening with modulus of elasticity *Es* = 2.06E+06 MPa, yielding strength *fym* = 550 MPa, failure strength *ft* = 1.25 *fy* and ultimate deformation ε_u = 0.075.

4. ANALYSIS RESULTS

All the analyses cover a time period of five years from the first casting.
The most critical scenario is of course the anchorage; it has then been run with three values for the cohesion coefficient $\beta_c = 0$, $\beta_c = 0.2$, $\beta_c = 0.4$ respectively.
A vertical crack along the interface between the blocks always occurs, but the quantity of crossing reinforcement placed in the joint deeply influences its width. No cracks occur inside the blocks, as the tensile strength of plain concrete is much bigger than the cohesion and the stresses transferred by reinforcement are not enough to cause cracking.
We focused our attention on four key parameters: the time when the first crack appears, the crack width, the crack vertical extension (Y direction), and the stress level in the bars crossing it and jointing the two blocks. For these last three parameters the value at five years is reported. The results obtained are shown in table 1.

Table 1: Results comparison for anchorage scenario

$\beta_c = 0$ Cohesion c = 0.001 MPa		Values at 5 years		
Crossing reinforcement	Time at cracking	Crack width	Vertical crack extension	Maximum bar stress
	[days]	[mm]	[cm]	[MPa]
0	0	0.31	192	--
16ϕ14	0	0.10	72	187
16ϕ16	0	0.07	72	156

β_c = 0.2 Cohesion c = 0.646 MPa		Values at 5 years		
Crossing reinforcement	Time at cracking	Crack width	Vertical crack extension	Maximum bar stress
	[days]	[mm]	[cm]	[MPa]
0	120	0.26	132	--
16ϕ14	180	0.09	72	181
16ϕ16	180	0.07	72	156

β_c = 0.4 Cohesion c = 1.292 MPa		Values at 5 years		
Crossing reinforcement	Time at cracking	Crack width	Vertical crack extension	Maximum bar stress
	[days]	[mm]	[cm]	[MPa]
0	300	0.22	92	--
16ϕ14	925	0.08	65	169
16ϕ16	1400	0.06	65	142

Grounding on these results, the “full coupling” scenario with cohesion c = 0.646÷1.292 MPa and ϕ14 or ϕ16 reinforcement crossing the joint is run.
No cracking phenomena arise because of the small time interval (14days) between casting of the second part and re-tensioning. During this period differential creep can not develop a differential deformation big enough to open the joint working against cohesion forces as a maximum tension of 0.15 MPa arises on the interface. After re-tensioning a compressive stress of 6.5÷4.5 MPa, that tends to the asymptotic value of 4÷5 MPa in five years, can be found behind the coupling zone in the second block.
The “half coupling” scenario is then run with the same values seen for the previous one. Again no cracking arises for the same reason explained in the “full coupling” example.
After re-tensioning a compressive stress of 3.5÷2.5 MPa, that tends to the asymptotic value of $\approx$ 2.0 MPa in five years, can be found behind the coupling zone in the second block.

5. CONCLUSIONS

The results of the analysis described in the paragraphs above can be synthesized as follows:

1. In the anchorage scenario a single crack localized on the joint surface takes place. This crack is due to differential creep between the two blocks, and in case of no reinforcement crossing the joint it can reach 0.25-0.30 mm after a few years of life of the structure. Placing a proper amount of reinforcement through the joint can sensibly reduce the crack width.
2. Despite its variation, the value of cohesion does not play a relevant role for crack width and stresses in reinforcement because of the perfectly brittle behaviour of the interface. It only determines the time at which the crack arises and a small reduction in the crack opening.
3. In case of full coupling cracks may arise in the time interval between the casting of the second block and the re-tensioning. The probability of crack formation increases with

elapsing time between these two moments. If re-tensioning is done within 14 days since the second casting, as reported in this study, no cracks are to be expected.

4. The same considerations can also be made for the half coupling scenario. Even if only one half of the tensioning force is taken away from the joint, the differential creep that may develop in 5 years only implies a small reduction in the compressive stress on the joint.

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