FINAL TUNNEL LINING OF PLAIN CONCRETE

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Abstract

This paper is devoted to the design structures of plain concrete.

Keywords: Tunnel, final lining; construction of plain concrete.

1 Introduction

Reinforced concrete final liners of tunnel structures are required in the majority of designs in the Czech Republic. But this solution is associated with complications in terms of the consumption of time and high labour intensiveness in the reinforcement placement phase. Another unpleasant reality is the continually rising cost of reinforcing steel.

It is therefore necessary for the final lining designer to seek new approaches, such which would not involve the above-mentioned engineering complications and, at the same time, would reduce the construction costs.

The use of unreinforced concrete for the final lining belongs among such approaches adopted by designers and contractors. In the Czech Republic, this approach is today rather exceptional. Of late there are only two tunnels which have been constructed by this technique, i.e. the New Connection railway tunnels under Vítkov hill (Fig.1), Prague, and the Libouchec motorway tunnel.

Fig. 1 **The New Connection railway tunnel**

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A new standard for concrete structures [1] entered into force in the Czech Republic in December 2006. This standard defines, among other relationships, the basic relationships for the designing and assessment of unreinforced concrete structures.

2 Behaviour of plain concrete structures in the ultimate limit states (ULS)

In practice, an unreinforced concrete structure may fail in the two following ways:

1) as a result of crushing of concrete by a normal force acting:

a) concentrically or eccentrically, but still within the core of the section, usually without a bending crack developing within the concrete section (Fig.2a),

b) eccentrically, outside the core section, but still within the contour of the whole section, while the origination of a bending crack and unlimited extension of the crack is permissible (Fig.2b)

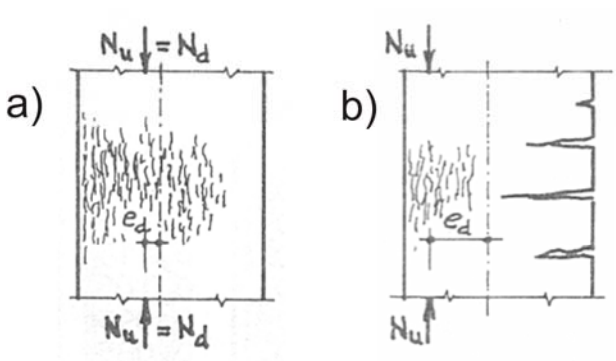

Fig. 2 **Compresive failure of an unreinforced concrete element**

2) as a result of a crack developing due to a normal force acting beyond the contour of the section; the loading capacity of the section depends on the tensile strength of concrete and the crack must not develop in the ULS (which is, at the same time, the limit state of cracking (LSC)) so that the equilibrium of internal forces can take place in the critical section (Fig. 3).

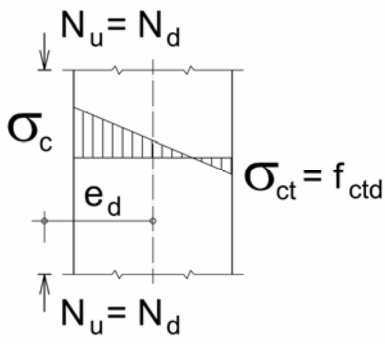

Fig. 3 **At the same time, the limit state of cracking is equal to the ULS**

3 PLOTTING OF THE INTERACTION DIAGRAM FOR A SYMMETRIC SECTION

The design strength of concrete in concentric compression f_{cd}

$$f_{cd} = \alpha_{cc,pl} \cdot \frac{f_{ck}}{\gamma_C}$$

$\alpha_{cc,pl}$ is a reduction coefficient allowing for the lower ductility of concrete which is plasticized under compression. The [1] recommends that the value $\alpha_{cc,pl} = 0{,}8$ be chosen in the case of the relative compression of concrete $\varepsilon_{cu} = 3{,}5$ ‰.

f_{ck} a characteristic value of concrete strength in concentric compression
γ_C a coefficient of concrete reliability

The design strength of concrete in concentric tension f_{ctd}

$$f_{ctd} = \alpha_{ct,pl} \cdot \frac{f_{ctk\,0,05}}{\gamma_C}$$

$\alpha_{ct,pl}$ is a reduction coefficient for the ductility of plasticized concrete in tension at the moment of cracking. According to the National Annexe, this coefficient is taken into consideration as follows:

$\alpha_{ct,pl} = 0{,}8$ when the two following conditions are met (Fig.4, straight line A):
- the indirect design loads resulting from volumetric changes in the concrete structure are appositely determined
- the characteristic tensile strength of concrete $f_{ctk,\,0.05}$ is guaranteed through preliminary testing of the concrete to be used

$\alpha_{ct,pl} = 0{,}6$ when at least one of the above two conditions is met (Fig.4, straight line B):
$\alpha_{ct,pl} = 0{,}4$ when neither of the above two conditions is met (Fig.4, straight line C):
$f_{ctk\,0,05}$ a characteristic value of concrete strength in concentric tension
γ_C a coefficient of concrete reliability

3.1 POINT 0 – Theoretical concentric compression

In the case of the limit design approach where the possibility of buckling of the section can be excluded, we can determine the normal force in the ULS using the relationship

$$N_{Rd0} = b \cdot h \cdot f_{cd} \tag{1}$$

where b is the section width and h is the section depth and the failure moment:

$$M_{Rd0} = 0 \tag{2}$$

However, the loading capacity of an element under concentric compression cannot be practically taken into consideration because [1] introduces at least the so-called minimum eccentricity into the calculation.

3.2 POINT 1 – The effect of geometrical imperfection

The required minimum eccentricities are given by the following relationships:

$$e_{d,min} = \frac{h}{30} \tag{3}$$

$$e_{d,min} = 20\ mm \tag{4}$$

The above eccentricity covers the effect of geometrical imperfections; we take the higher of the two values into consideration for the calculation purpose.

When the section is under eccentric compression and the cracking is permitted, the loading capacity of the section can be determined generally using the limit design approach and the relationship

$$N_{Rd1} = f_{cd} \cdot b \cdot h \cdot \left(1 - \frac{2 \cdot e_d}{h}\right) \tag{5}$$

$$M_{Rd1} = N_{Rd1} \cdot e_d \tag{6}$$

This relationship is applicable with a theoretical limitation of the design eccentricity

$$0 \le e_d \le \frac{h}{2} \tag{7}$$

3.3 POINT 2 – Boundaries of the core of the section

When the value of the normal force eccentricity $e_d = h/6$ and, in the case of elastic behaviour, the entire section is under compression, a crack will develop in the ULS after the effective compressive zone gets plasticized and the limit design relationships (5) and (6) will again be applicable for $e_d = h/6$, i.e.

$$N_{Rd2} = f_{cd} \cdot b \cdot h \cdot \left(1 - \frac{2 \cdot \frac{h}{6}}{h}\right) \tag{8}$$

$$M_{Rd2} = N_{Rd2} \cdot \frac{h}{6} \tag{9}$$

3.4 POINT 3 – Maximum value of the failure bending moment in the case of eccentric compression

The maximum value of the failure moment in the ULS can be achieved at $e_d = h/4$.

$$N_{Rd3} = f_{cd} \cdot b \cdot h \cdot \left(1 - \frac{2 \cdot \frac{h}{4}}{h}\right) \tag{10}$$

$$M_{Rd3} = N_{Rd3} \cdot \frac{h}{4} \tag{11}$$

3.5 POINT 4 – Limit eccentricity in terms of the permissible crack extension

The limit eccentricity $e_{d,lim}$ = 0,4h derived for a permitted crack length is (h - x) = 0,75h. The co-ordinates of the point 4 are then given by the relationship

$$N_{Rd4} = f_{cd} \cdot b \cdot h \cdot \left(1 - \frac{2 \cdot 0{,}4 \cdot h}{h}\right) \tag{12}$$

$$M_{Rd4} = N_{Rd4} \cdot 0{,}4 \cdot h \tag{13}$$

The loading capacity of the section determined by the limit design approach is limited from the point 4 to the origin of co-ordinates by a straight line expressing the ratio M_{Rd} / N_{Rd} = 0,4h. The effect of the line comes to an end at the latest in point 5C (Fig. 4).

3.6 POINT 5 – Boundary between the elastic design and limit design approaches

Boundary point 5 must lie on a line which is permitted by the limit eccentricity (e.g. $e_{d,lim}$ = 0,4h). In cases of larger eccentricities it is no more possible to secure the equilibrium between external and internal forces within the critical section in other way than by means of the elastic design approach, using the design flexural tensile strength of concrete $f_{ctd,fl}$, which is determined according to the relationship:

$$f_{ctd,fl} = \alpha_h \cdot f_{ctd} \tag{14}$$

where α_h is a coefficient of the section width (h must be put into the relationship in mm)

$$\alpha_h = \left(1{,}6 - \frac{h}{1000}\right) \geq 1{,}0 \tag{15}$$

The elastic design approach, which expresses the linearly elastic behaviour of an unreinforced concrete element, is therefore applicable in the case of $e_d > e_{d,lim}$ = 0,4h.

The co-ordinates of points 5 (A through C) will be derived, using the elastic design approach, from the relationships:

$$N_{Rd5} = \alpha_h \cdot f_{ctd} \cdot \frac{1}{6} \cdot b \cdot h \cdot \left(\frac{0{,}4 \cdot h}{\frac{h}{6}} - 1\right) \tag{16}$$

$$M_{Rd5} = N_{Rd5} \cdot 0{,}4 \cdot h \tag{17}$$

The results will depend on the selected value of the design strength of concrete in concentric tension f_{ctd}, with the creep reduction coefficient $\alpha_{ct,pl}$ taken into consideration.

3.7 POINT 6 – Pure bending

In the case of linearly elastic behaviour of unreinforced concrete, the loading capacity of a rectangular section subjected to pure bending can be expressed simply by the following relationships:

$$N_{Rd6} = 0 \tag{18}$$

$$M_{Rd6} = \alpha_h \cdot f_{ctd} \cdot \frac{1}{6} \cdot b \cdot h^2 \tag{19}$$

3.8 POINT 7 – Simple concentric tension

When the design loading capacity of unreinforced concrete structures is, exceptionally, calculated with tensile normal forces taken into consideration, it is also possible to use the co-ordinates of point 7 for the concentric tension, which are given by the following relationships for the strength of concrete in concentric tension f_{ctd} (without coefficient α_h):

$$N_{Rd7} = b \cdot h \cdot f_{ctd} \tag{20}$$

$$M_{Rd7} = 0 \tag{22}$$

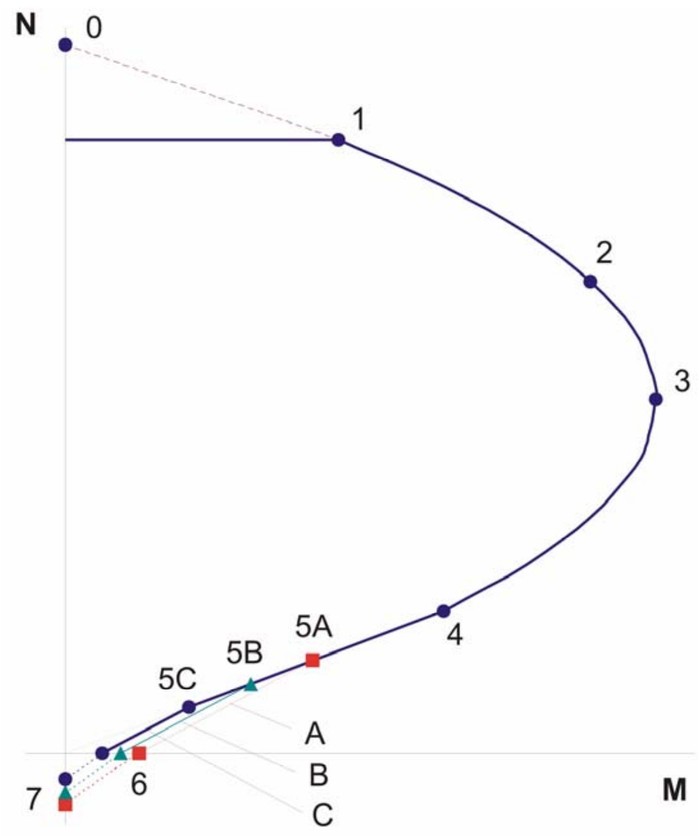

Fig. 4 **An interaction diagram for a rectangular unreinforced concrete section**

4 CONCLUSION

Considering the rapidly growing cost of reinforcing steel, the high consumption of time and labour intensiveness during the placement of reinforcing bars, we can expect that designers will be more and more forced to take unreinforced concrete design alternatives of supporting structures into consideration.

References

[1] ČSN EN 1992-1-1, Eurocode 2: *Design of concrete structures – Part 1-1: General rules and rules for buildings*, ČNI 2006