„OVIDIUS„ UNIVERSITY OF CONSTANȚA

FACULTY OF CONSTRUCTION

SUMMARY OF DOCTORAL THESIS

THE CALCULUS AND BEHAVIOR OF THE REINFORCED AND PRESTRESSED CONCRETE ELEMENTS UNDER THE SHEAR FORCE

PhD Student: Eng. CECILIA PÉNZES (BARTÓK)

Scientific Coordinator

Prof. PhD. Eng. AUGUSTIN POPAESCU

Constanța, 2013

ACKNOWLEDGEMENTS

After having finished the long process of elaborating the Ph. D. thesis, I especially want to render thanks to the scientific leader of the thesis, prof. univ. dr. eng. Augustin Popaescu for the permanent guiding and encouragement. The professor was present in my life with a competitive and exigent leading, being my professor at University Transilvania in Brasov, at the Faculty of Constructions, the scientific leader of my dissertation at the end of university , later of the MA dissertation: The Rehabilitation of Old Buildings.

For the technical support I would like to express my thanks to the firm Popaescu&CO S.R.L., lead by prof. dr. ing. Popaescu Augustin and to PREFA S.A. Brașov for the possibility of trying the bridge girder L=24m.

I would also like to express my thanks to prof. dr. eng. Tudor Clipii from the Faculty of Constructions and Architecture, Timisoara for the help offered in learning and using program modules pStress and pTruss, used int he Ph. D. thesis.

I also render my thanks to prof. dr. eng. Virgil Breabăn, dean of the Faculty of Constructions at University Ovidius, Constanta for the support he offered during the Ph. D. studies.

I would also like to thank my husband, Lóránd, the moral support he offered me during the elaboration of the thesis.

Finally I would like to thank to my father the perseverance and professional exigence he showed me during several years. Thanks to him I have achieved this point in my professional career.

Author,
Eng. Cecilia Pénzes

Key words: reinforced and prestessed concrete beam,limit shear strength, strut and tie method, Strut and Tie programme, beams with special detailing, testing the first series, shear strengthening of the reinforced concrete beams.

CONTENTS

THE OBJECTS AND STRUCTURE OF THE THESIS

The PhD thesis entitled „The behavior and calculus of the reinforced and prestressed concrete elements under the shear force”, has the following objectives:

-analysis of calculus and behavior under the shear force of the reinforced and prestressed concrete beams in the national standard 10107/0-90: *Civil and industrial buildings. Design and detailing of concrete*, *reinforced concrete, and prestressed concrete structural members* and by Eurocode2: *Design of concrete structures EN, Part 1: General rules and rules for buildings*(EC2);

-analysis of the strut and tie method;

-analysis of two testing on stand up to failure of precast prestressed and reinforced concrete beams;

-to realize a synthesis of the methods (general methods in shear strengthening as well as achieve in international area in order to reduce the strengthening costs) used to increase the shear resistance of reinforced concrete beams.

The PhD thesis is composed of 6 chapters, ordered logically and as part of each one is presented the specify problems, subordinated to realize the thesis objects.

The **first chapter** begins with a presentation of calculus and behavior of the reinforced concrete elements under the shear force as depicted in the old and new calculation standards. The standard 10107/0-90 uses the method of the inclined-sections limit equilibrium. The design model considers a failure mechanism with one degree of freedom composed of two stiff bodies (the beam segments separated by the inclined breaking crack) which rotate relatively.

In EC 2 the shear design is based on the strut and tie model (STM).

This chapter also contains some conclusions which are drawn from comparing the test results with other formulations: the truss model with concrete contribution of the old EC2 (ENV 1992-1-1: 1991); the equation 11-3 of the ACI 318-02 Code, the draft for public comment of the CSA (CSA Committee A23.3, 2003) and a semi-analytical method proposed by Cladera and Mari.

In **chapter 2** are presented some D-regions of liniar members (statical discontinuities from concentrated forces or reactions, geometric discontinuities, such as abrupt changes in cross section, etc.). STM is an approach used to design discontinuity regions in reinforced and prestressed concrete structures and elements. Members or regions of members may be designed by idealizing the concrete and reinforcement as an assembly of axially loaded members, interconnected at nodes. The following steps must be made: identify D-region, determine boundary condition of D-region, sketch the flow of forces; develope a truss model; calculate the forces in struts and ties, select steel area for ties; check stress level in struts and detail reinforcement based on calculation results in harmony with the technical prescription in force.

There is presented a soft program, which is very useful because it briefs space of calculus time, so the number of the arithmetical operation can be made quickly for dimension the members with STM.

Chapter 3 contains the testing on stand up to failure of precast prestressed and reinforced concrete beams.

There is presented the testing on stand up to failure of the bridge beam L=24m, passage DN1. The testing purpose was to check the requirements of the beam at it's ultimate limits in service, respective cracks, deformation and testing the behavior under loading to the up to failure, for legal quality certification of the production of these elements by testing the first series and obtaining the necessarry data for the legal certification of conformity of the prefabricated elements.

On the other hand is presented the testing up to failure of a reinforced concrete roof beam L=11.45m interex Vaslui. The testing of this roof beam, had as it's purpose the checking, by tests on the stand, of the behavior up to failure as well as the resistance of the beam for the ultimate limits in service, with special attention to the support points as well as that of the preponderant action of the shear force.

The **chapter 4** comes really and truly in continuation of the chapter 3, because contains the test results of the behavior up to failure of the prefabricated beam for bridge L=24 m passage DN1, and reinforced concrete roof beam l=11.45m interex Vaslui. The author compares the calculus of the roof beam according to the old and new calculation standards.

The calculus of the prestressed beam under the ultimate limit state is also given in this chapter.

Chapter 5 presents general methods used for shear strengthening of the reinforced concrete beams, such as strengthening with reinforced concrete, collar or composite materials known as Fibre Reinforced Polymer (FRP). As well as some testing achieves in international area, which exemplifies the shear strengthening of reinforced concrete T-beams by bonding external bidirectional Carbon Fibre Fabric (CFF) strips. This bi-directional CFF strip technique is not only improving the shear capacity of the strengthened beams but also reduces the quantity of material. The design shear strength of a concrete member strengthened with an FRP system is presented as depicted in different international calculation guides considered representative and compared by the author.

The **chapter 6** contains conclusions of the testing and research syinthesis performed in PhD thesis, related to the design of the shear force of the reinforced and prestressed concrete beams according to the old and new calculation standards; conclusions of analysis of the strut and tie method; conclusions of testing on stand up to failure of precast prestressed beams under the shear force; and conclusions formulated after the analysis of the design procedure and methods used for shear strengthening of the reinforced concrete beams. At the same time personal contributions of the thesis are presented.

CHAPTER 1
THE BEHAVIOR AND THE CALCULUS OF THE REINFORCED AND PRESTRESSED CONCRETE LINEAR ELEMENTS UNDER THE SHEAR FORCE AS DEPICTED IN THE CALCULATION STANDARDS

1.1. THE CALCULUS OF THE REINFORCED AND PRESTRESSED CONCRETE LINEAR ELEMENTS UNDER THE SHEAR FORCE ACCORDING TO STAS 10107/0-90

The behavior and the calculation of the reinforced concrete elements under the shear force have been studied in almost all of the major research centers in the world because the phenomenon itself is a most complex one. Multiple theoretical and experimental studies have been achieved in order to develop a satisfactory analytical model which both depicts the real-life behavior and is accessible for use in the design.

The bearing capacity under the shear force is determined by the shape and the proportions of the elements, the quantity and the position of the longitudinal reinforcement and the transverse reinforcement, the possible axial compressive or tensile stress, and the physical and mechanical properties of the concrete and the reinforcement.[1.1]

1.1.1. The shear-force behavior of the reinforced concrete elements without a transverse reinforcement

In a simplified model, we can consider that the loads applied to the reinforced concrete elements that have no transverse reinforcement, subjected to bending under a shear force, are taken over through two mechanisms – beam and arch (broken strut with cross-tie) – which coexist and determine each other.

The STAS 10107/0-90 stipulates that the limit under which we do not need to calculate the transverse reinforcement in the reinforced concrete linear elements is [1.1]:

$$Q = 0{,}5bh_0R_t$$

1.1.2. The shear-force behavior of the reinforced concrete elements with transverse reinforcement

To calculate the inclined sections under the shear force, the standard STAS 10107/0-90 uses the method of the inclined-sections limit equilibrium. The design model considers a failure mechanism with one degree of freedom composed of two stiff bodies (the beam segments separated by the inclined breaking crack) which rotate relatively. The limit-state equilibrium along the cracking with an inclined direction with respect to the axis of the elements can be described through an equation for a projection to the normal to the axis of the element and a momentum equation with respect to the application point of the resultant of the compression stresses inside the concrete, fig. 1.2. [1.1].

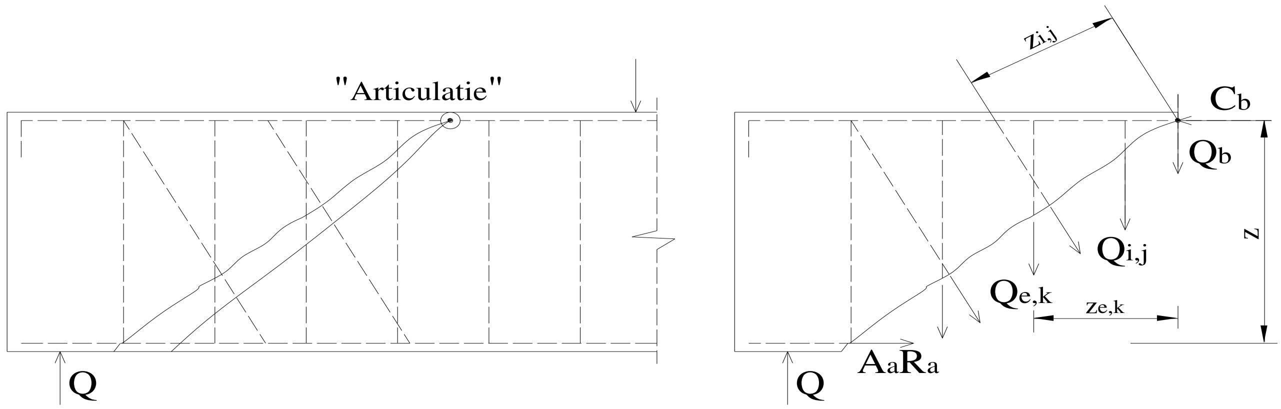


Fig.1.2. Design model[1.1]

The shear-force calculation of the constant-section elements can be made using [1.4]:

$$Q \le Q_{cap} = Q_b + \sum A_{at} R_a \sin\alpha + \sum n_e A_e R_{at}$$

1.1.3. The behavior and the calculus of the capable shear strength for the prestressed concrete elements according to the standard STAS 10107/0-90

The calculation of the capable strength for both the bending moment and the shear force is made in inclined cracked sections [1.4].

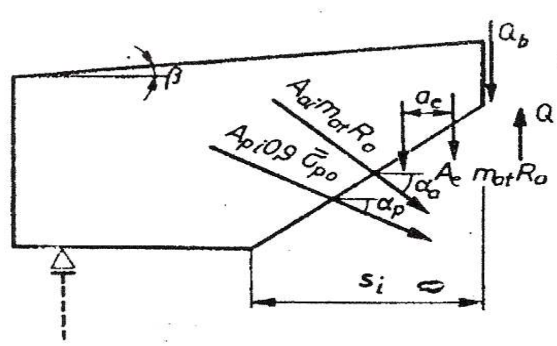


Fig. 1.9. Inclined cracked section. The limit strength calculation for the shear force. [1.4]

The equilibrium equation in an inclined cracked section is:

$$Q_{cap} = \sum 0{,}9\bar{\sigma}_{po} A_{pi} \sin\alpha_p + \sum A_{ai} m_{at} R_a \sin\alpha_a + \sum A_{et} m_{at} R_a + Q_b$$

1.2. THE SHEAR STRENGTH PROCEDURE IN EUROCOD2

1.2.1. Design stages in EC2

The steps for the shear force calculation are [1.7.]:

- determination of the diagram for the design shear force, V_{Ed};
- correction of the diagram with possible reductions;
- calculation of the bearing capacity of the concrete segment, $V_{Ed,c}$;
- verification whether:

$V_{Ed,red} \le V_{Ed,c}$ if it holds, the transverse reinforcements will be chosen in a constructive manner.

- if $V_{Ed} > V_{Rd,c}$, calculate the bearing capacity of the compression concrete diagonals $V_{Rd,\max}$ in accordance with the type of the transverse reinforcement used (vertical or inclined) and check whether: $V_{Ed} \le V_{Rd,\max}$.

If it is not true, increase the dimensions of the concrete segment;

- calculate the bearing capacity of the transverse reinforcements $V_{Rd,s}$ by choosing a diameter and a A_{sw}/s distance between the bars in accordance with the diagram V_{Ed}.

1.2.2. Members not requiring design shear reinforcement

The design value for the shear resistance $V_{Rd,c}$ is given by [1.6] :

$$V_{Rd,c} = \left[C_{Rd,c}k(100\rho_l f_{ck})^{1/3} + k_1\sigma_{cp}\right]b_w d \tag{1.2.b.1}$$

with a minimum of:

$$V_{Rd,c} = (v_{\min} + k_1\sigma_{cp})b_w d$$

1.2.3. Members requiring design shear reinforcement

The design of members with shear reinforcement is based on a truss model [1.6] (fig. 1.13).

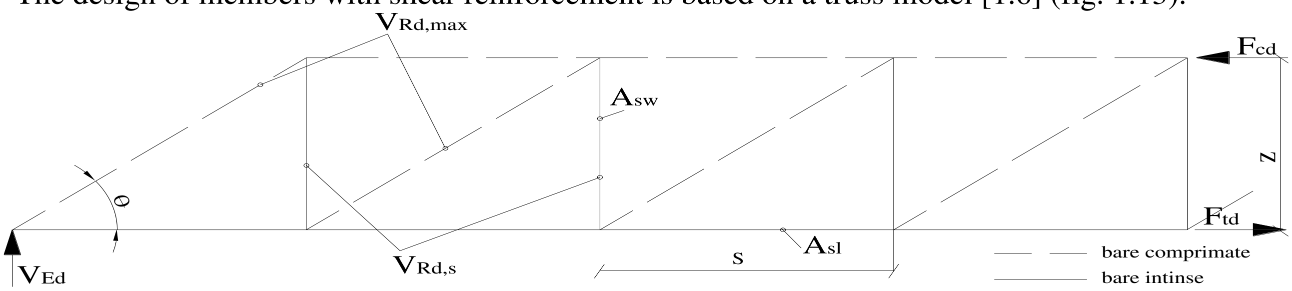


Fig. 1.13. - Calculation model under the shear force with stirrups[1.7.]

For members with vertical shear reinforcement, the shear resistance $V_{Rd,s}$ is the smaller value of:

$$V_{Rd,s} = \frac{A_{SW}}{s} \cdot z \cdot f_{ywd} \cdot ctg\,\theta \tag{1.2.h}$$

$$V_{Rd,\max} = \alpha_{cw} b_w z v_1 f_{cd} /(ctg\,\theta + tg\,\theta) \tag{1.2.i}$$

For members with inclined shear reinforcement, the shear resistance is the smaller value of [1.6] :

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} (ctg\,\theta + ctg\,\alpha) \sin\alpha \tag{1.2.m}$$

$$V_{Rd,\max} = \alpha_{cw} b_w z \nu_1 f_{cd} (ctg\,\theta + ctg\,\alpha)(1 + ctg^2\theta) \tag{1.2.n}$$

1.2.4. Determination of the design shear force V_{Ed} for the simply supported or continuous beams

The value of the angle θ must be within: $21{,}8^\circ \leq \theta \leq 45^\circ$, ($1 \leq ctg\theta \leq 2{,}5$).

Various inclinations θ will result in different quantities for the transverse and the longitudinal reinforcement because of the different loads inside the compression diagonals and in the tensile bars respectively (both in the stirrups and in the longitudinal bars). The role of the design is to optimize $ctg\theta$ in order to lead to a total minimum quantity of reinforcement and workmanship.[1.7]

1.2.5. Comparison of the EC-2 with other formulations for the shear strength procedure

The shear strength of reinforced concrete beams with stirrups has been a highly controversial matter since Ritter and Mörsh proposed the first truss models. Since then, different analytical models have been discussed, such as truss models with concrete contribution, shear/compression theories, truss models with variable angle of inclination, and compression field theories. In the paper from Cladera A. și Mari R.-*Shear strength in the new Eurocode2. A step Forward?, Structural Concrete.2007. 8.No2.* [1.8], in order to evaluate the EC-2 shear procedure for reinforced concrete members with web reinforcement, a database with 122 beams was developed. All the beams were simply supported and loaded with one or two point loads.

The empirical shear strength of the beams specimens of the database has been compared with four other formulations: the truss model with concrete contribution of the old EC-2 (ENV 1992-1-1:1991), the equation 11-3 of the ACI 318-02 Code (American Concrete Institute. *ACI Building Code Requiments for Reinforced Concrete*. ACI, Farmington Hills), the draft for public comment of the CSA (CSA Commitee A23.3. *Design of Concrete Structures*. Public rewiew draft, Canadian Standards Association, Rexdale, ON, September 2003,p.233), and a semi-analytical method proposed by Cladera and Mari. [1.8]

The results indicate that for reinforced concrete members with web reinforcement the average of the V_{fail}/V_{pred} (shear force causing failure in the empirical test/predicted shear resistance by different compared formulations) ratio is equal to 1.19 for the old EC-2 shear procedure, 1.38 for the ACI 318-02 shear procedure, 1.13 for the CSA formulation and 1.06 for the proposed method by Cladera and Mari. It can be seen that even the very simple equation 11-3 of the ACI 318-02 correlates better with the empirical tests than the new EC-2 does. The old EC-2 formulation of 1991 also offers a better correlation with the test results than the new EC-2 shear procedure. [1.8]

CHAPTER 2
THE ANALYSIS OF THE REINFORCED AND PRECAST CONCRETE BEAMS WITH DISCONTINUITY REGIONS UNDER THE SHEAR FORCE AND SOME SPECIFIC ASPECTS OF THIS BEAMS WITH SPECIAL DETAILING

2.1. BEAMS WITH SPECIAL DETAILING

2.1.1. Short corbels

2.1.2. Support beam ends with reduced cross sections

The trimming of the ends of the elements may feature various shapes and dimensions. Below is shown an example of a beam trimmed for support alongside a model for the strut and tie.

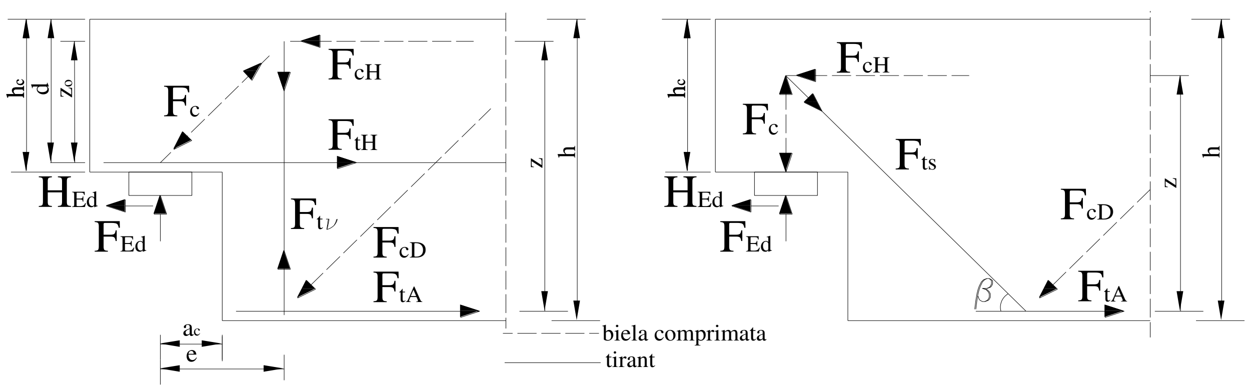


Fig. 2.4. Strut and tie model for a support beam end with reduced cross section

a) horizontal reinforcement and vertical stirrups b) horizontal and inclined reinforcement

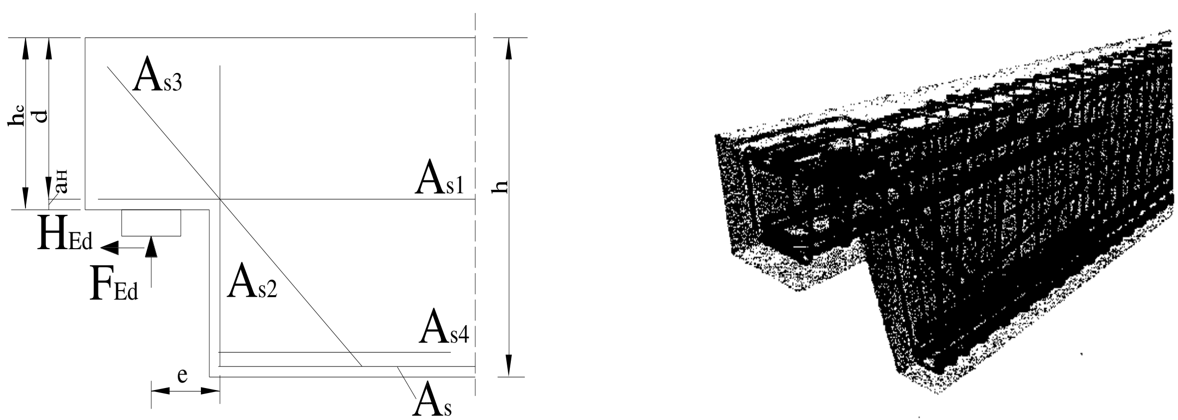


Fig.2.4.c) Characteristic reinforcements for a support of the beam end

More explicitly, here is presented the design of the trimmed or simply supported beams through a combination of the two models of struts and ties (Fig. 2.5. and fig.2.6.).

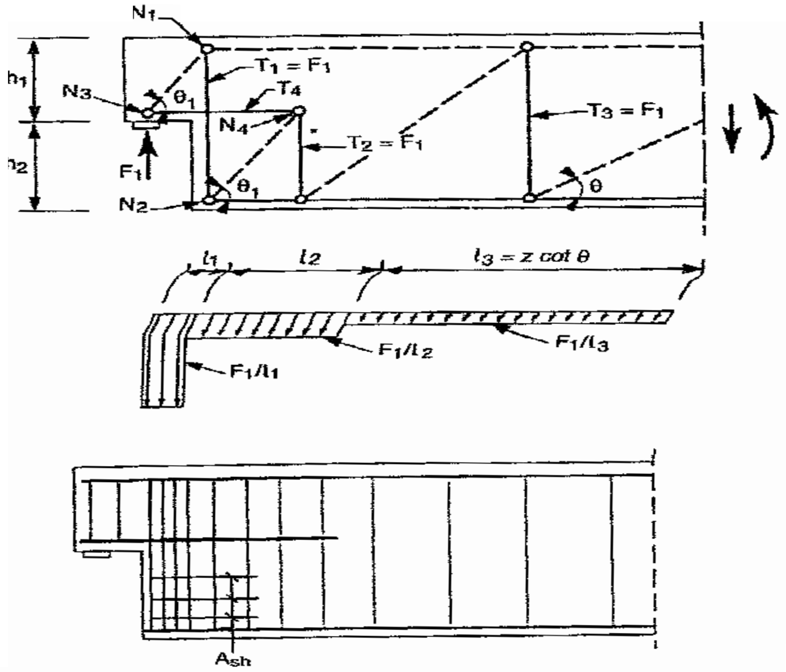


Fig.2.5. Model with horizontal reinforcement at the support and reinforcement style[2.5]

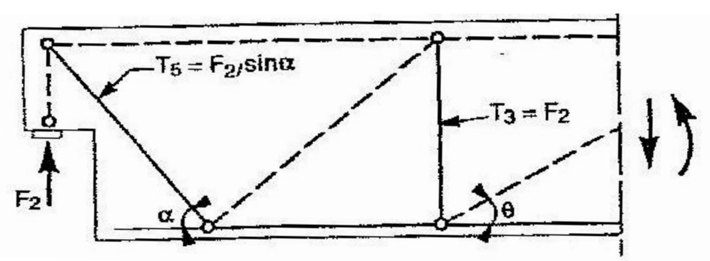


Fig.2.6. Model with inclined reinforcement at the support[2.5]

2.1.3. Beams with web openings

2.2. MODELING OF THE DISTURBED OR DISCONTINUITY REGIONS

2.2.1.Theoretical analysis

Concrete structural members can be divided into two general regions: flexural regions (Bernoulli or B-regions) and regions near discontinuities (Disturbed or D-regions). In the case of the B-regions, it is accurate to assume that planes remain planes after loading, and the plane section assumption of flexural theory can be applied. In the case of B-regions, the load path of applied forces is of little interest. In general, any portion of a structural member outside of a B-region is a D-region. A discontinuity in the stress distribution occurs at an abrupt change in the geometry of a structural element (geometric discontinuities), at a concentrated load or reaction (loading or statical discontinuities), or a combination of the two (loading and geometric discontinuities).

2.2.2. Historical development of strut and tie modeling

The beginnings of STM date back to the infancy of reinforced concrete design. In 1899, Wilhelm Ritter developed a truss mechanism to explain the role of transverse reinforcement in the shear strength of a beam. Previously it was believed that transverse steel bars provided dowel action that resisted horizontal shear deformations. Based on Ritter's truss model, it is clear that the stirrups are in tension, and dowel action is not the primary shear resistance mechanism. Ritter's model was later refined by Mörsch in 1902. Mörsch believed that the discrete diagonal forces that Ritter had used in his truss would be better represented by a continuous field of diagonal compression. In 1927, Richart proposed a method of shear design in which the concrete and steel contributions to shear strength were calculated independently then summed to determine the overall shear strength. In this method the concrete contribution to shear strength V_c was based on empirical observations of beams failing in shear and the steel contribution was based on a truss model whose concrete compression field was at an angle of 45 degrees from the longitudinal reinforcement. Revival in the use of STM began in the United States in 1980. At that time STM was first applied to concrete members subjected to a combination of shear and torsion. [2.2].

In particular, in Europe, Swiss and Scandinavian engineers utilised paralell or fan-shaped stress fields to explain the real load baring capacity of plane reinforced concrete structures by the theory of plasticity[Muttoni/Schwartz/Thuerlimann(1997)]. Stimulated by J. Schlaich, the strut and tie method was generalised and improved to be applicable to all kinds of reinforced concrete members and to entire structures [Schlaich/Weischede(1982), Schlaich/Schaefer/Jennewein(1987), Schlaich/Schaefer(1991), Schlaich/Schaefer (1998)] [2.9].

2.2.3. Analysis with strut and tie method

Elements of a Strut and Tie model[2.3] are:

-strut: compression field member

-tie: the reinforcement

-node:where struts and ties intersect (meeting point).

2.2.3.1.Strut and tie modeling

The steps for strut-and-tie modeling are:

1. after the D region identification-stress analysis and the graphical representation of the elastic stress trajectories for the given loading case is ;
2. the developing of the strut-and-tie model based on the graphical representation of the stress;
3. the static analysis of the strut-and-tie model;
4. design of the reinforcements area using tensile forces in the elements of the model;
5. checking (node verification) of the concrete compression stresses;
6. element detailing based on the calculus results and according to the code rules.

Among this stages the most laborious ones are the first three, because they require a lot of work and several repetitions of some of the actions, which can lead to a long period of time for the calculus [2.7].

2.3. ANALYSIS OF THE BRIDGE BEAM END

This subchapter discusses the analysis of the prefabricated bridge beam end. The author obtained the directions of the main unit stresses by running the pStress program module and the statistical calculus of the bars was made using the pTruss program module.

The prefabricated reinforced concrete beam has the shape shown in fig. 2.27. and undergoes a uniformly distributed force of *p*=20.51 kNm. We can identify a single disturbed region generated by the support.

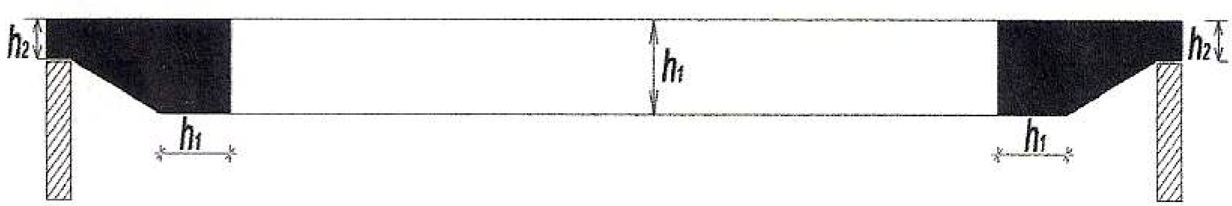


Fig.2.35. Identification of the disturbed zone

In a first stage, we took the built-in section at h1 = 90cm.

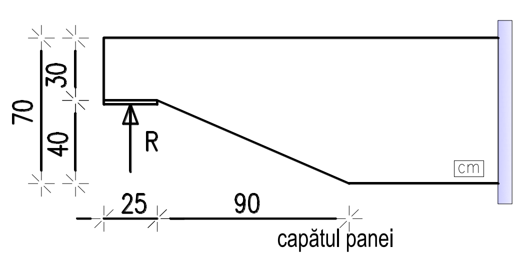


Fig.2.36.Built-in section considered in the program

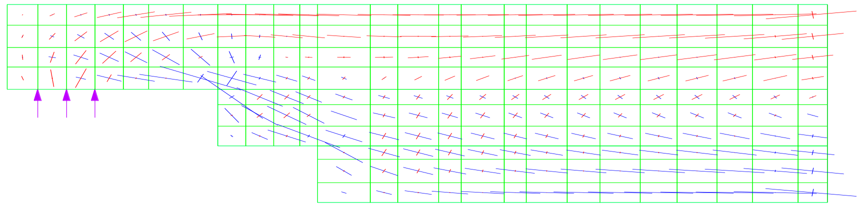
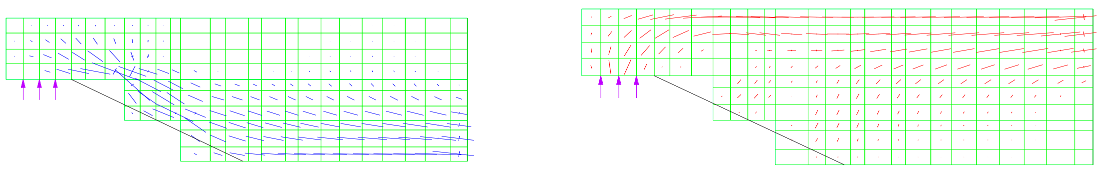


Fig.2.37. a) Result obtained by the pStress module



b) Principle tensile stresses c) Principle compressive stresses

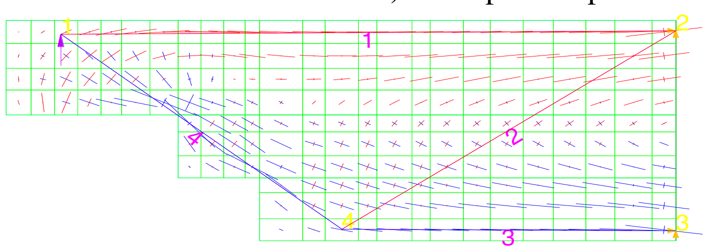


Fig.2.40. Strut and Tie model for the end of the beam

The values of the axial forces in the model's bars are given in the table 2.1.

Table 2.1. Axial forces in the model's bars (kN)

Bar	1	2	3	4
Model	186,69	313,83	-1459,06	-203,36

The reinforcements for the two tensile bars are:

bar 1: $A_{a1} = N_1 / R_a = 186{,}69 \cdot 10^3 / 300 = 622{,}3 mm^2$

bar 2: $A_{a2} = N_2 / R_a = 313{,}83 \cdot 10^3 / 300 = 1043{,}33 mm^2$

Although the two reinforcement areas are different, we will use 3 reinforcements of Φ22 (11,40cm^2).

The reaction R is propagated through the concrete from the metallic support plate (Figure 2.28.) to the node 1 of the model and generates a transverse tensile force [2.4]:

Z = (1/3..1/4)R=(1/3..1/4)117,42≈35kN.

The necessary reinforcement area is given below and this reinforcements will be laid in a constructive manner: $A_a = Z / R_a = 35 \cdot 10^3 / 300 = 116{,}67 mm^2$.

The verification of the compressions in the node 2 is performed using the model in fig 2.33. where the compression force in the diagonal is transferred over 3Φ22 longitudinal reinforcement.

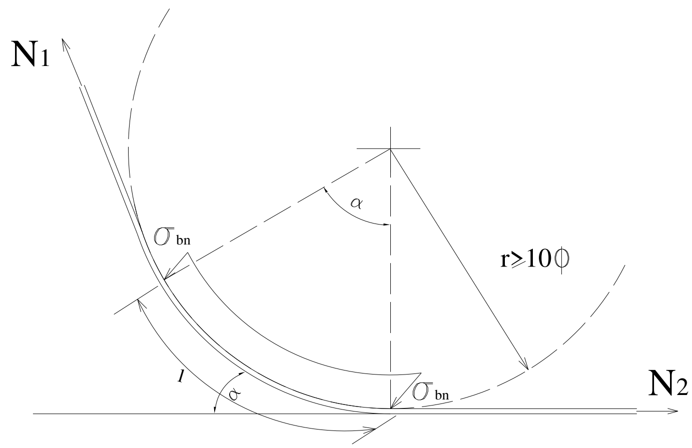


Fig.2.33. Node with reinforcement deviation

From the geometry of the model we got α=24°, that is 0,418 radians. Adopting the radius of the circle equal to 300mm, the arc length l=0,418r=125,4mm.

With the value presented above we got: $\sigma_{bn} = \frac{N_4}{l \cdot \sum \Phi} = \frac{203{,}36 \cdot 10^3}{125{,}4 \cdot 3 \cdot 22} = 24{,}57 N/mm^2$

Verification of the compressions is performed with the formula:

$\sigma_{bn} = 20{,}27 < 0{,}8R_c = 12N/mm^2$ does not hold.

The relation does not hold even if we use 4Φ25 reinforcements instead of 3Φ22,

$$\sigma_{bn} = \frac{N_4}{l \cdot \sum \Phi} = \frac{203{,}36 \cdot 10^3}{125{,}4 \cdot 4 \cdot 25} = 16{,}21 N/mm^2 > 0{,}8R_c$$

When using 6Φ25, we got:

$$\sigma_{bn} = \frac{N_4}{l \cdot \sum \Phi} = \frac{203{,}36 \cdot 10^3}{125{,}4 \cdot 6 \cdot 25} = 10{,}81 N/mm^2 < 0{,}8R_c$$

For the 6Φ25 longitudinal reinforcement is recommended to be laid continuously in the inferior zone of the beam without interruption at the inferior zone of the widening.

CHAPTER 3
TESTING ON STAND UP TO FAILURE OF PRECAST PRESTRESSED AND REINFORCED CONCRETE BEAMS

3.1. GENERAL ASPECTS WITH REGARD TO PERFORM THE TESTING OF THE PRECAST PRESTRESSED AND REINFORCED CONCRETE BEAMS

In this chapter are presented the experiments of two beams: bridge beam L=24m, passage DN1 and reinforced concrete roof beam L=11.45m interex Vaslui. The testing purpose of the 24m bridge beam was to check the requirements for obtaining the necessarry data for the legal certification of conformity of the element. The testing of the 12m roof beam had as it's purpose the checking, by tests on the stand, of the behaviour up to failure as well as the resistance of the beam for the ultimate limits in service, with special attention to the support points as well as that of the preponderant action of the shear force. [3.1], [3.2], [3.3]. Must be mentioned that both of the elements had reduced height of the cross section at support.

The tests were carried out under the guidance of prof.dr.eng. Augustin Popaescu in 2006 and 2007, by the POPAESCU & CO SRL firm, with the participation of Ph.D student Cecilia Pénzes (Bartók).

At the test was applied guide C181-88 *Guide for first series testing methodology of prefabricated elements in statical loading* [3.4].

3.2.BRIDGE BEAM L=24 M PASSAGE DN1

3.2.1.The object of the test

The testing purpose of the 24m bridge beam was to check the requirements of the beam at it's ultimate limits in service, respectively cracks, deformation and testing the behavior under loading to the up to failure, for legal quality certification of the production of these elements by testing the first series and obtaining the necessarry data for the legal certification of conformity of the prefabricated element [3.10].

The testing was effectuated by the application of the forces in cycle (with unloading), in 2 stages.

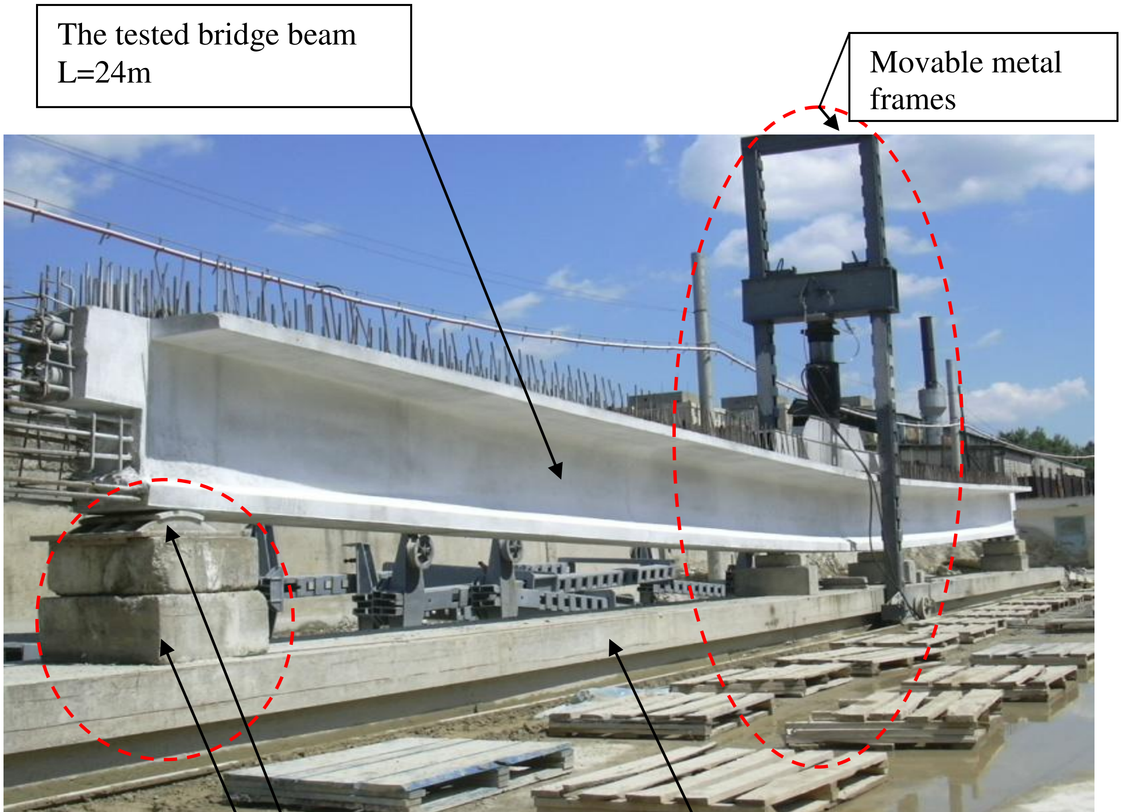


Fig.3.1. Bridge beam L=24m Passage DN1

3.2.2. General data

The beneficiary is KOTA KONSTRUCT SA Bucureşti, the special designer is EUROPROIECT SRL and the roof beam tests were carried out by SC POPAESCU & CO SRL Bucureşti on stand PREFA SA Braşov.

3.2.3. Data about the tested beam [3.8]

The experimental theme was prepared on the basis of the Experimental Project elaborated by EUROPROIECT SRL and the technical requirements and conditions stated by STAS 12313-85 *Railway bridges and main roads. Stand testing of the prefabricated and prestressed elements;* STAS 6657/1-89 *Reinforced, prefabricated and prestressed elements. General technical condition of quality and* C181-88 *Guid for first series testing methodology of prefabricated elements in statical loading*.

3.2.4. The test

The tests were carried out on the PREFA SA Braşov stand, consisting of a 30m beam stand and movable metal frames. The study was carried out with 2 concentrated forces, within a distance of 3.0 m and a 2400 kN hydraulic press, the forces acted in two points as it is shown below.

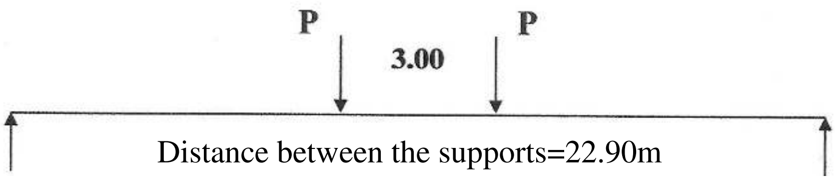


Fig. 3.14.-Loading scheme

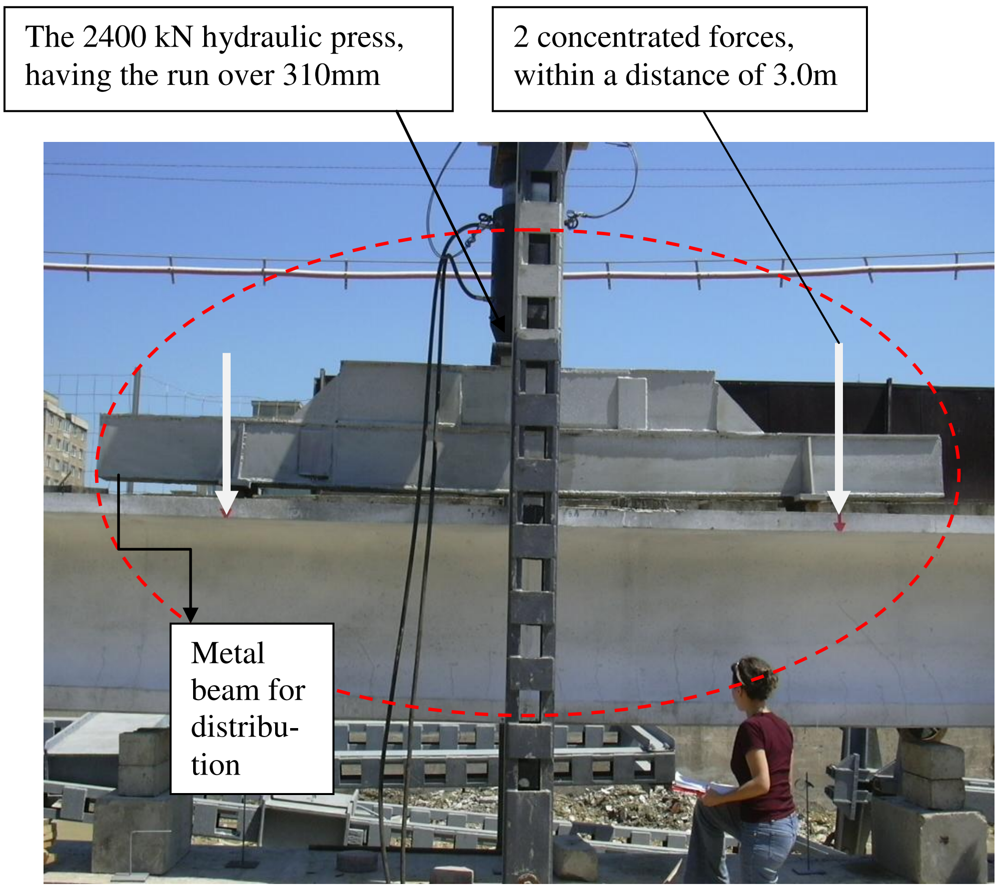


Fig.3.12. Study time

3.2.5. Supports

3.2.6. Measurements

3.2.7. Results and interpretation

3.3.REINFORCED CONCRETE ROOF BEAM L=11.45M

The testing of the 11.45m roof beam had as it's purpose the checking, by tests on the stand, of the behavior up to failure as well as the resistance of the beam for the ultimate limits in service, with special attention to the support points as well as that of the preponderant action of the shear force [3.9].

3.3.1. General data

The test was effectuated on a stand of the SOMACO SA Branch ROMAN, built of two skillet 12m ECP, connected in transversal direction by 2U profile with soldering and a prefabricated bridge beam, placed on concrete supports.

Fig. 3.20.-Testing the reinforced concrete roof beam

3.3.2. Data about the tested beam

The beam leaned on two device support, one fixed and the other mobile.

It was loaded static with two 1200 kN hydraulic press, having the run over 250 mm, in points of load[3.5]. There were foreseen two concentrated forces at 2.8m distance of the support's axle.

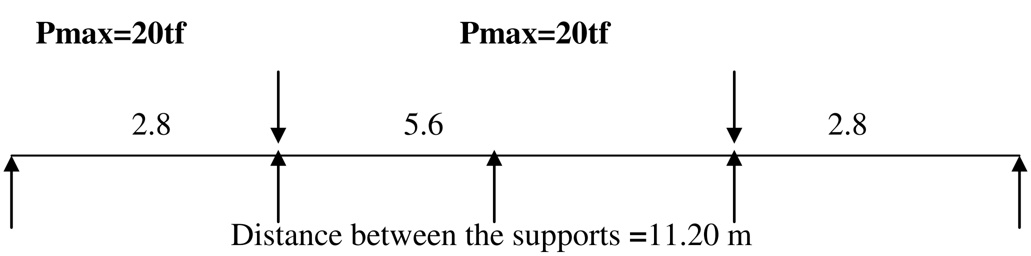


Fig.3.23. Loading scheme

3.3.3. Measurements

CHAPTER 4
RESULTS AND INTERPRETATION OF THE TESTING OF THE PRESTRESSED AND REINFORCED CONCRETE BEAMS

4.1. RESULTS INTERPRETATION. GENERAL

4.1.1. Cracking conditions

4.1.2. Deformations

4.1.3.Bearing capacity

At the results interpretation of the testing of the prestressed and reinforced concrete beams, first is persuited the cracking condition and distance between the cracks-experimental confronted with the calculated one. Admissibility conditions at verification of the reinforced concrete elements are presented in this subchapter.

4.2. INTERPRETATION OF THE RESULTS FROM TESTING OF THE FIRST SERIES OF THE PREFABRICATED REINFORCED CONCRETE BEAMS

4.2.1. Appearance of the cracks*[4.1]*

4.2.2.Closing of the cracks *[4.12]*

4.2.3.Deformations*[4.1]*

4.2.4.Bearing capacity *[4.1]*

The tests and interpretation of the results obtained on the first series is based on the simultaneously respect of some conditions imposed by the experimental failure stress.

4.3.THE REINFORCED CONCRETE ROOF BEAM

4.3.1. Calculation of the reinforced concrete roof beam

After the presentation of the material characteristics and estimation of the acting loads, was calculated the maximum bendig moment and shear force.

In the design at resistance limit state, the verification is important through comparing the maximum section stresses with the capable section stresses.

The calculation of the bendig moment in inclined section is made by verification with the equilibrium equation of the moments reported to the center of gravity of the compressive zone.

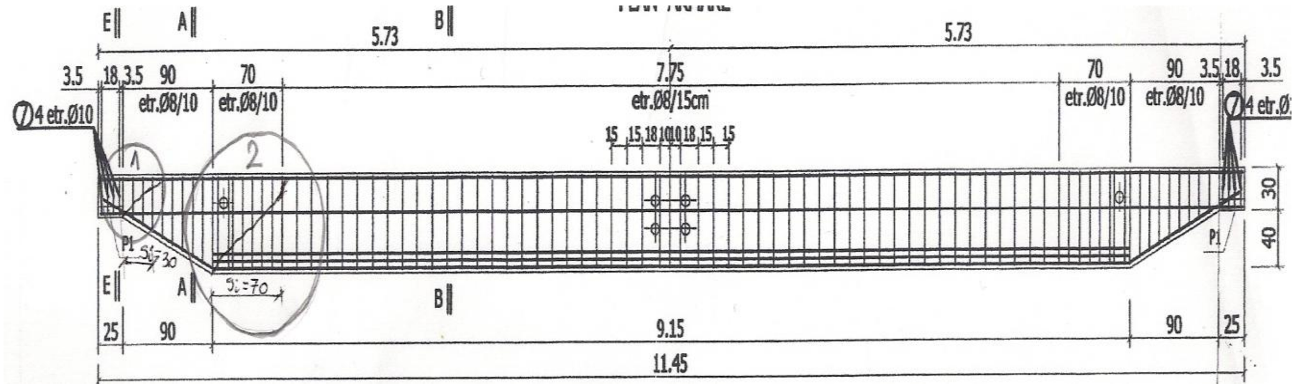


Fig.4.5.

The shear strength verification to the resistance limit state, by STAS 10107/0-90, is realized at the change of the transversal section of the roof beam, that is at the superior and inferior zone of the widening.

Q=117,9kN - (design shear force – see table 1 in the Doctor's Thesis).

The calculation of the shear strength in inclined section with the crack started at the superior zone of the widening is made using the formula: $Q \leq Q_{cap} = Q_b + \sum n_e A_e R_{at}$

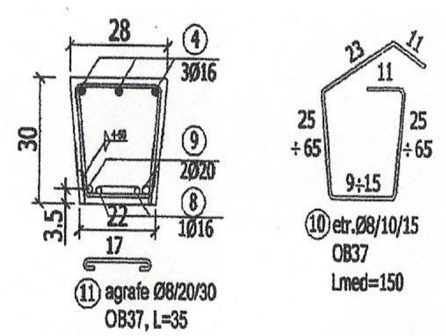


Fig.4.6.a)-Cross section of the roof beam –superior zone (1)-

The shear force load is given by the relation:

$$\bar{Q} = \frac{Q}{A_{bo} R_t} = \frac{11{,}79 \times 10^4}{25 \times 26{,}5 \times 110} = 1{,}617$$

-where $A_{b0}=bh_0$.

R_t is the design tensile strength for the Bc25 concrete (=1,10N/mm^2).

m_t is the coefficient of the working conditions and equals $m_t = \frac{3-\bar{Q}}{2} \leq 1 \rightarrow m_t = 0{,}69$

- Q_b is the shear force taken over by the concrete from the prestressed section and determined by the relation: $Q_b = \frac{b h_o^2 \sqrt{p}}{s_i} m_t R_t = 34{,}41 kN$

- where p is the percentage of longitudinal reinforcement in the tensile area in the vicinity of the inclined crack.

$$p\% = 100\frac{A_a}{bh_0} = 0{,}60 \rightarrow A_a = 4{,}02cm^2 (2\phi16)$$

-s_i : horizontal projection of the inclined crack length which, upon the first change in the cross section, equals 30cm.

-n_e : number of arms in a stirrup.

-A_e: cross section of the transverse segment in a stirrup arm, crossed by the inclined crack, equal to 0,503 cm2.

$$\sum n_e A_e R_{at} = 3\times 2\times 0{,}503\times 30000 = 90{,}54kN \rightarrow Q_{cap} = 34{,}41 + 90{,}54 = 124{,}95kN$$

$$Q(=117{,}9kN) \leq Q_{cap}(=124{,}95kN)$$

In consequence the roof beam resists in given loading condition.

The calculation of the shear strength in inclined section with the crack started at the inferior zone of the widening is made using the same formulas:

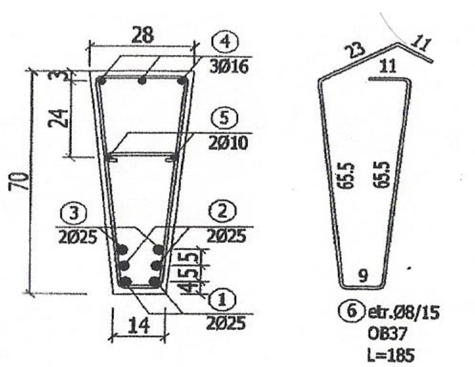


Fig.4.6.b.- Cross section of the roof beam – inferior zone (2)-

-we considere 2 bars in calculus because the rest 4 bars were interrupted at the inferior zone of the widening.

$$Q(=117{,}9kN) \leq Q_{cap}(=124{,}95kN)$$

-superior zone of the widening.

$$Q(=117{,}9kN) \leq Q_{cap}(=243.42kN)$$

-variable section zone.

$p = 0{,}69\%$ $\qquad \bar{Q} = \frac{Q}{A_{bo}R_t} = 0{,}77$

$s_i = 70cm$

-where $m_t = \frac{3-\bar{Q}}{2} > 1 \rightarrow m_t = 1$

$$Q_b = \frac{bh_o^2\sqrt{p}}{s_i} m_t R_t = 119{,}4kN$$

$$\sum n_e A_e R_{at} = 7\times 2\times 0{,}503\times 30000 = 211{,}26kN$$

$$R_{at} = m_{at}\cdot R_a = 1\cdot 300 = 300N/mm^2$$

-inferior zone of the widening;

$$Q(=117{,}9kN) \leq Q_{cap}(=330{,}66kN)$$

In consequence the roof beam resists in given loading condition.

According to EC2, the design value for the shear resistance of the roof beam not requiring design shear reinforcement at the superior zone of the widening (1) is[4.4]:

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}\right] \cdot b_w \cdot d =$$

$$= 0{,}12 \cdot 1{,}87 \cdot (100 \cdot 0{,}02 \cdot 20{,}5)^{1/3} \cdot 227 \cdot 265 = 73{,}88kN$$

$$V_{Rd,c\min} = (v_{\min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d = 0{,}406 \cdot 227 \cdot 265 = 24{,}42kN$$

$$V_{Ed} = 117{,}9kN > V_{Rd,c} = 73{,}88kN$$

The value of the angle θ must be within: $1 \le ctg\Theta \le 2{,}5$ [4.14].

There was calculated the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts, $V_{Rd,\max}$ depending of the type of the used reinforcement and maximum value of ctgΘ=2,5 [4.9]:

$$V_{Rd,\max} = \alpha_{cw} b_w z v_1 f_{cd} / (ctg\theta + tg\theta)$$

$$V_{Rd,\max} = 1 \cdot 227 \cdot 238{,}5 \cdot 0{,}54 \cdot 15 / 2{,}9 = 151{,}22kN$$

The verification whether $V_{Ed} \le V_{Rd,\max}$ it holds and the dimensions of the concrete segment are proper.

The design value of the shear force which can be sustained by the yielding shear reinforcement is:

$$V_{Rd,s} = \frac{A_{SW}}{s} \cdot z \cdot f_{ywd} \cdot ctg\theta$$

$$V_{Rd,s} = \frac{2 \cdot 0{,}503 \cdot 100}{100} \cdot 238{,}5 \cdot 210 \cdot 2{,}5 = 125{,}96kN$$

According to the truss model the failure can be produced either by the yielding of transversal reinforcement ($V_{Rd} = V_{Rd,s}$) or by the crush of the concrete in the compression struts ($V_{Rd} = V_{Rd,\max}$)

For a ductile failure the following inequality must be fulfilled: $V_{Rd,s} \le V_{Rd,\max}$.

The shear resistance of the roof beam with vertical Φ8 stirrups is the minimum value of:

($V_{Rd,s}$; $V_{Rd,\max}$)=125,96kN and $V_{Ed} = 117{,}9kN < 125{,}96kN$.

According to EC2, the design value for the shear resistance of the roof beam not requiring design shear reinforcement at the inferior zone of the widening (1) is[4.4]:

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp}\right] \cdot b_w \cdot d = 0{,}12 \cdot 1{,}55 \cdot (100 \cdot 0{,}01 \cdot 20{,}5)^{1/3} \cdot 147 \cdot 660 = 49{,}38kN$$

$$V_{Ed} = 117{,}9kN > V_{Rd,c} = 49{,}38kN.$$

There was calculated the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts, $V_{Rd,\max}$ depending of the type of the used reinforcement and maximum value of ctgΘ=2,5 [4.9]: $V_{Rd,\max} = \alpha_{cw} b_w z v_1 f_{cd} / (ctg\theta + tg\theta)$

$V_{Rd,\max} = 1 \cdot 147 \cdot 594 \cdot 0{,}54 \cdot 15 / 2{,}9 = 243{,}88kN$

The verification whether $V_{Ed} \le V_{Rd,\max}$ it holds and the dimensions of the concrete segment are proper.

The design shear force is within: $V_{Ed,c} \le V_{Ed} \le V_{Rd,\max} \rightarrow 49{,}26kN < 117{,}9kN < 243{,}88kN$

We must calculate the value of ctgΘ[4.9]:

$$ctg\,\theta = \frac{1{,}2}{1-\dfrac{V_{Rd,c}}{V_{Ed}}} = \frac{1{,}2}{1-\dfrac{49{,}38}{117{,}9}} = 2{,}07$$

-must be within 1,0 and 1,75.

The design value of the shear force which can be sustained by the yielding shear reinforcement is:

$$V_{Rd,s} = \frac{A_{SW}}{s} \cdot z \cdot f_{ywd} \cdot ctg\,\theta \qquad V_{Rd,s} = \frac{2 \cdot 0{,}503 \cdot 100}{100} \cdot 594 \cdot 210 \cdot 1{,}75 = 219{,}60kN$$

The shear resistance of the roof beam with vertical Φ8 stirrups is the minimum value of:

($V_{Rd,s}$; $V_{Rd,\max}$) =219,60kN, $V_{Ed} = 117{,}9kN < 219{,}60kN$

In consequence the roof beam resists in given loading condition formulated in the new and old calculation standards.

4.3.2. Test results of the roof beam

Experimental valeus were compared with the control values:

$M_r^{\exp}$ =64.4tfm;

M_{cap}^{ULS} =45.06tfm,

-relation $M_r^{\exp} / M_{cap}^{ULS}$ =1.429 calculated for 6Φ25 –PC 52 and Bc 30, foreseen at the execution, resulted **1.429>1.4- accepting condition** $\dfrac{M_r^{\exp}}{M_{cap}^{SLR}} \ge c$ [4.1].

$T_r^{\exp}$ =23.0tf; T_{cap}^{ULS} =14.70 tf relation $T_r^{\exp} / T_{cap}^{ULS}$, calculated for 3 stirrup Φ8 and Bc 30, foreseen at the execution, resulted 1.56- **accepting condition** [4.1].

The first cracks appear at the $0.9\,P^E$ (1) step, there are marked in continuation at steps :9 tf(2), 12.7tf(3), 17.4tf(4), 20.2tf(5).

- at the (4) step was measured 0.35mm;
- at the (5) step the inclined cracks which starts from the inferior part of the widening , at the point of interrupt of the 4 longitudinal steel, presented an opening over 2.0mm.

The beam was loaded in continuation till P=21 tf, where the deflection at the middle of the span measured 132.6mm, representing 1/84.4 of the opening, and the maxim moment M_r^{exp} represented 1.429 M_{cap}^{SLR} calculated for 6Φ25 –PC 52 and Bc 30.

There were no damages without the opening of the inclined cracks over 0.2mm, which starts at the inferior part of the widening, in the point of interruption of the 4 longitudinal steel.

The behavior of the reinforced concrete roof beam made by the SOMACO SA Branch ROMAN, conformable to Proiect PROCEMA Engineering SRL was according to the technical requirements and conditions specified by the standards şi technical regulations.

There was recomanded that at the design of similar beams to take carre of a better anchorage of the bars which are interrupted at the changing of direction at the widening and of the inclined bars on support.

4.4. BRIDGE BEAM L=24M, PASSAGE DN1

4.4.1. Calculus of the bridge beam

The calculation of the inclined cracks sections under the shear forces is made with the formula (130) in the standard STAS 10107/0-90 and Q_{cap}=2043,37kN>> Q=300,37kN.

The behavior of the beam under the ultimate limit conditions shows that the verification relations are being met in excess due to a design made for an operational beam arrow.

4.4.2.Test result interpretation of the prestressed bridge beam L=24m, passage DN1

The testing was effectuated by the application of the forces in cycle (with unloading), in 2 stages. The results are :

I stage

The **crack apparition** produced at P = 158kNm, respectively ΔM = 1594 kNm > 1500 kNm (supreme value).

The **reopening of the cracks** produced at P = 132 kN, ΔM =1313 kNm, M = 1838 kNm. ΔM =1313 kNm represented 0.96 of $M_{dec} – M_{dl}$ = 1370 kNm.

A the loading corresponding to 0.98 M_{capULR} the arrow was 38.5mm **(1/600) < <44.7 cm.**

The permanent arrow was 3.8% from the arrow at this cycle < 10% admissible.

II stage

At P=307 kN, ΔM =3055 kNm, M = 3579 kNm, the arrow was **208 mm** (1/ 110), and the cracks reached 1,5mm- ***the first failure criterion.***

At the unloading, the remanent arrow was 20mm.

P_r = 340 kN, ΔM_r =3383 kNm, M_r = 3907 kNm.

At this upload the deflection, at the middle of the span, measured 29.05 mm representing 1/78 of the opening and the maxim bending moment was M = 3907 kNm representing 1.61 M_{capULR} .

The second failure criterion was f_r=1/78 L and 1.61 M_{capULR} .

The test results showed a satisfactory behavior of the beam, without any weaknesses and no adverse effects of the shear force, anchorage of the reinforcement, touching of the limits of the deformation in the reinforcement and concrete, attesting the manufacture manner of the beam in the technical manufacturers condition [4.18].

CHAPTER 5
SHEAR RESISTANCE INCREASE OF THE REINFORCED CONCRETE BEAMS

5.1. INTRODUCTION

Shear deficiencies are the result beside the injurious influence of the aggressive medium and the increased load requirements and/or more stringent design codes. Irrespective of the degradation cause, the necessity of shear strengthening of reinforced concrete beams is huge in international area. Considering both the cost, and the number of structures involved, there is a need to find an effective shear retrofitting system for this structures.

5.2. GENERAL METHODS USED TO INCREASE THE SHEAR RESISTANCE OF THE REINFORCED CONCRETE BEAMS

Shear deficiency can be improved with structural material addition on the lateral surface of the elements, action called externally bonded technique, which can be realized with reinforced concrete, steel or fibre reinforced polymer[5.1].

5.3. TESTS ACHIEVED IN INTERNATIONAL AREA IN ORDER TO REDUCE THE STRENGTHENING COSTS OF THE REINFORCED CONCRETE BEAMS WITH LOW RESISTANCE

Advanced composite materials as fibre reinforced polymer (FRP) have received a great interest in rehabilitation of existing reinforced concrete elements (columns,beam,etc).

The application of Fibre Reinforced Polymer as an external reinforcement is an evergreen technique of improving the structural performance of reinforced concrete structures. Over the past decade, much of the research in the area of shear retrofitting has looked at the use of fibre reinforced polymers (FRPs) epoxy bonded to the sides of specimens.

The author analysis several experimental programs achieved in international area, in which the use of the advanced composite materials is realized by bonding external strips at certain spaces, for example:

1. study at the Structural Engineering Laboratory of the University Putra Malaysia, which exemplifies the shear strengthening of 8 reinforced concrete T-beams by bonding external bidirectional Carbon Fibre Fabric (CFF) strips (spaced at 150mm or at 200mm centre to centre)[5.2].

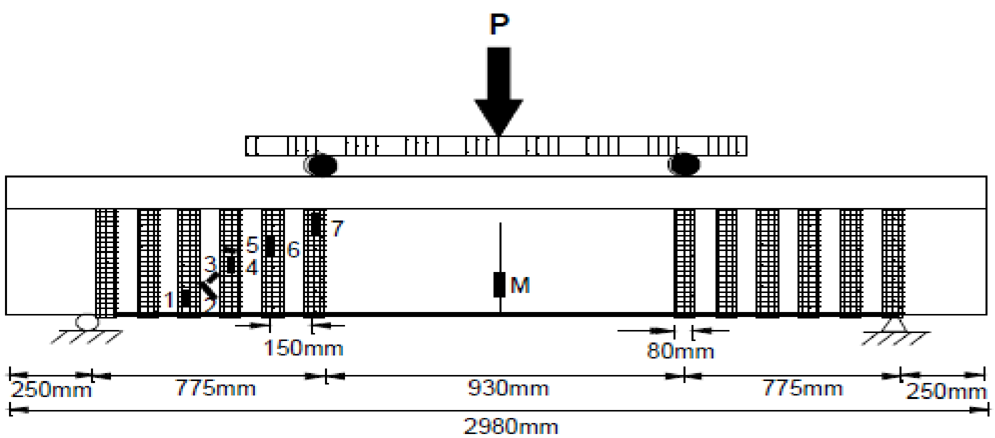


Fig.5.16.A Speciments TT1-1, TS1-1, TS1-11

2. study at the University of Cambridge, Trumpington St., Cambridge, which attests the potential application of thin CFRP (carbon fibre reinforced polymer) straps as external shear reinforcing elements, using an under-slab installation technique, which involved drilling holes into the flange of a beam so that the strap could form a closed loop.

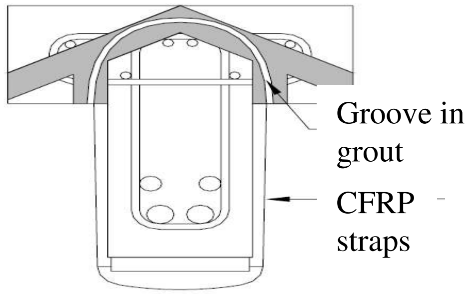


Fig.5.18. Strap configurations using grout

3. study at the Laboratory of Reinforced Concrete, National Technical University of Athens, Greece, on T-beams strengthened by means of FRPs and subjected to cyclic displacements [5.6].

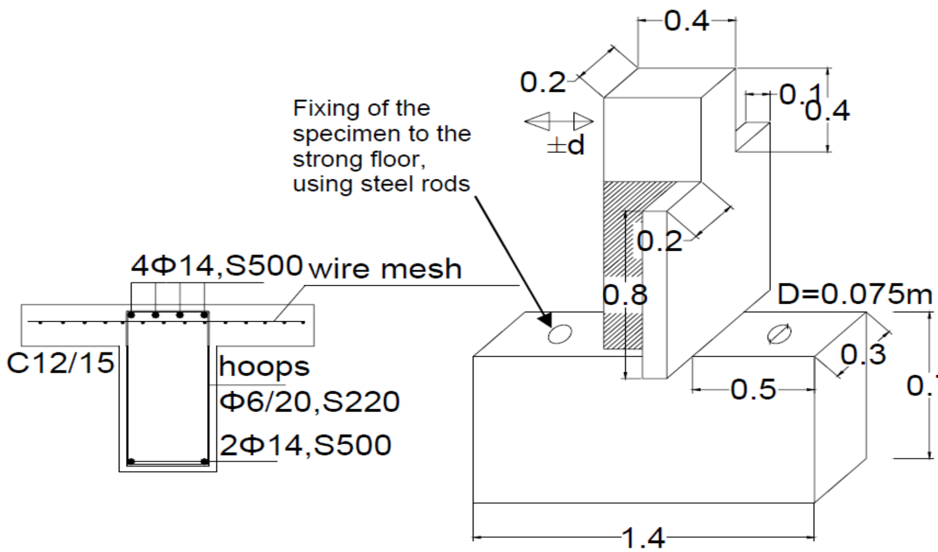


Fig.5.19. Geometry, reinforcement and (schematic) testing procedure of beams

5.4. THE DESIGN SHEAR STRENGTH OF A CONCRETE MEMBER STRENGTHENED WITH COMPOSITS

There is presented the verification of the shear strength given in representative calculation standards, guides:

1. ACI 318-05,*Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structure, American Concrete Institute[5.8]:*

The design shear strength should be calculated by multiplying the nominal shear strength by the strength reduction factor φ: $\phi \cdot V_n \geq V_u$

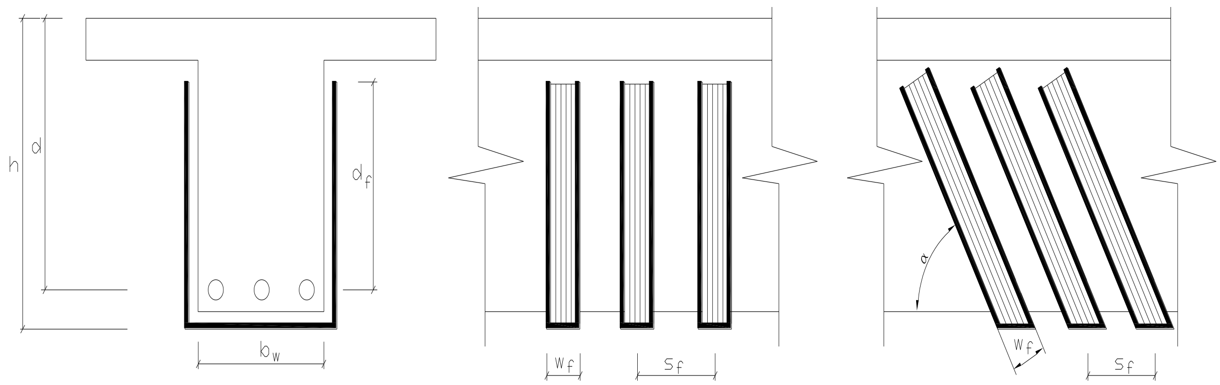


Fig. 5.20. Illustration of the dimensional variables used in shear-strengthening calculations [5.7]

Where: φ : reduction factor

V_u-shear force given by the loads;

V_n -nominal shear strength.

2. Report14 of *fib* TG 9.3 -*Externally Bonded FRP Reinforcement For RC Structures[5.9]:*

Bearing capacity to the shear strength of a reinforced concrete element to U.L.S. is[5.9]:

$$V_{Rd} = \min(V_{cd} + V_{wd} + V_{fd}; V_{Rd2})$$

3. CNR-DT 200/2004: *Guide for the design and construction of Externally Bonded FRP Systems for Strengthening Existing Structures, july 13,2004[5.10]:*

Bearing capacity to the shear strength of a reinforced concrete element is:

$$V_{Rd} = \min\{V_{Rd,ct} + V_{Rd,s} + V_{Rd,f}, V_{Rd,\max}\}$$

CHAPTER 6. CONCLUSIONS

In this thesis we have studied, researched the four stated objectives and the conclusions obtained are presented below for each objective.

6.1. TEST AND RESEARCH SYNTHESIS PERFORMED IN PhD THESIS RELATED TO SHEAR DESIGN OF THE REINFORCED AND PRESTRESSED CONCRETE BEAMS USING THE OLD AND NEW TECHNICAL REGULATIONS

The author of the PhD thesis entitled „The behavior and calculus of the reinforced and prestressed concrete elements under the shear force", realized a theoretical investigation of the design and behaviour of reinforced and prestressed concrete members under the shear action in international and national area.

After the documentation, there were concretized the following conclusions:

- EC2, through the strut and tie model, gives freedom to make decision on the size of Θ: the angle between the concrete compression strut and the beam axis perpendicular to the shear force. In Romanian standards the angle Θ has not a fixed value. The aim of this freedom, besides the load evaluation and member sizing, is to pronounce the essence of design by the engineer: to take in consideration the possible solutions and choosing the most practical and economic solution. During the design can be conceived different structural models and after the verification of this models with a softver program and analysis of the cost, can reach the optimal solution.We can choose either $\Theta=45^{\circ}$ or (expl. $ctg\Theta=2$ depending on the quantity of the longitudinal and transverse reinforcements [2.7].
- The Romanian technical standards, in the design of the reinforced concrete members, were based on the plane section assumption of flexural theory, but this theory can not be applied in many situations for example at the corbels, deep beams, openings int he web of the elements, abrupt change in the geometry of a structural element, concentrated loads with significant intensity, and the European regulations in force, offer solution of this cases through the strut-and-tie model, which generalizes the truss analogy in designing of concrete structural elements[2.7];
- The new EC2 shear procedure for members with web reinforcement is, indeed,a very simple method to calculate the shear strength for practising engineers and it verifies the lower bound theory of plasticity. As Regan has pointed out, for simpler models the problem is mostly that of the need to neglect some factors, considered secondaries. However, what is secondary in one case may be primary in another[1.8].
- The benefit of prestressing is not taken into account in EC2 due to the excessive simplicity of the model, but can be considered as Romanian standards[1.3][1.4].
- Equations from the technical regulations, eurocodes, public comments (the old EC-2 (ENV 1992-1-1:1991), the equation 11-3 of the ACI 318-02 Code (American Concrete Institute. ACI Building Code Requiments for Reinforced Concrete. ACI, Farmington Hills), the draft for public comment of the CSA (CSA Commitee A23.3. Design of Concrete Structures. Public rewiew draft, Canadian Standards Association, Rexdale, ON, September 2003,p.233)) correlates better with the empirical tests than the new EC-2 does[1.8].

- The truss model proposes a liniar response (without any concrete contribution) as long as the failure is determined by the crush of the compressed struts. This result may be slightly unconservative for highly shear-reinforced members, and they are too conservative for slightly reinforced beams [1.8].

6.2. ANALYSIS OF THE STRUT AND TIE METHOD

Within the thesis it was analysed the evolution of the shear design conception of the reinforced and prestressed concrete beams, based on truss model as far back as in 1899, when Wilhelm Ritter developed a truss mechanism to explain the role of transverse reinforcement in the shear strength of a beam. Ritter's model was later refined by Mörsch in 1902, so we come to the conclusion that the design model in EC2 is studied since the XIXth century.

Conclusions formulated after the analysis of the strut and tie method:

- The application of the strut and tie method requires engineering justification and experience gained over time in decision making during the adequately use of the model or even combination of two models in order to reach optimal results of economic and technical point of view.
- the stress analysis and the graphical representation of the elastic stress trajectories for the given loading case and the developing of the strut-and-tie model based on the graphical representation of the stress, respectively the static analysis of the strut-and-tie model requires hard work and repetition in a large number of computational operations which leads to a long time for the calculation.

6.3. STUDY SYNTHESIS ON STAND UP TO FAILURE OF PREFABRICATED BEAM ELEMENTS MADE OF PRESTRESSED AND REINFORCED CONCRETE

1.)The testing purpose of the 24m bridge beam up to failure, was to check the requirements of the beam at it's ultimate limits in service, respective cracks, deformation and testing the behavior under loading to the up to failure, for legal quality certification of the production of these elements by testing the first serie and obtaining the necessarry data for the legal certification of conformity of the prefabricated element [3.10]. The roof beam tests were carried out by SC POPAESCU & CO SRL, București, with the author presence.

The test results of the roof prestressed beam L=24m passage DN1, was according to the technical requirements and conditions stated by STAS 12313-85 *Railway bridges and main roads. Stand testing* *of the prefabricated and prestressed elements,* STAS 6657/1-89 *Reinforced, prefabricated and prestressed elements. General technical condition of quality and* C181-88 *Guid for first series testing methodology of prefabricated elements in statical loading*[4.18].

2.)The testing of the 11.45m roof beam had as it's purpose the checking, by tests on the stand, of the behaviour up to failure as well as the resistance of the beam for the ultimate limits in service, with special attention to the support points as well as that of the preponderant action of the shear force [3.9].

The author made verification through calculation to different limit states (resistance limit state, limit state of the cracks opening, deformation limit state) of the roof beam by STAS 10107-90, respectively shear strength verification by EC2. In consequence this beam resists in given loading condition. There was recomanded that at the design of similar beams to take carre of a better anchorage of the bars which are interrupted at the changing of the direction at the widening and of the inclined bars on support. The value of the medium openings of the inclined cracks, which starts from the inferior zone of the widening, at the interruption point of the 4 steels, in diameter 25mm, by the author's calculation oversteps the admissible limit, fact which must be underlined and considered in design of similar elements.

6.4. METHODS USED FOR SHEAR STRENGTHENING OF THE REINFORCED CONCRETE BEAMS

The author analysis several experimental programs achieved in international area, in which the use of the advanced composite materials is realized by bonding external strips at certain spaces. The following studies can be mentioned:

1. study at the Structural Engineering Laboratory of the University Putra Malaysia, which exemplifies the shear strengthening of 8 reinforced concrete T-beams by bonding external bidirectional Carbon Fibre Fabric (CFF) strips (spaced at 150mm or at 200mm centre to centre)[5.2].

The following conclusions are drawn based on the experimental investigation:

-The shear enhancement of the precracked/strengthened and initially strengthened specimens was increased ranging from 20% to 61% over the control beam.

-Results have shown that the increased longitudinal tensile reinforcements influenced the shear capacity of the strengthened beams.

-All CFF strengthened specimens were failed in flexure. This premature flexural failure increased the ductile behaviour of the strengthened beams[5.2].

2. study at the University of Cambridge, Trumpington St., Cambridge, which attests the potential application of thin CFRP (carbon fibre reinforced polymer) straps as external shear reinforcing elements, using an under-slab installation technique, which involved drilling holes into the flange of a beam so that the strap could form a closed loop.

Based on the experiments carried out to date, all of the beams tested with CFRP straps had a shear capacity at least 22% higher than that of an equivalent unstrengthened beam and an increase of ductility.

3. an experimental programme was carried out at the Laboratory of Reinforced Concrete, National Technical University of Athens, Greece, on T-beams strengthened by means of FRPs and subjected to cyclic displacements[5.6]. The modification of shear to flexural mode of failure leads to an increase in bearing capacity of the beams, as dictated by the flexural capacity of the unstrengthened element.

The author of the thesis underlines the difference between the analized international calculation standards, considered representative in table given below:

Tabel. 5.4.

	Raportul 14 al fib/2001	***ACI 440.2R-08***	***CNR-DT 200/2004***
Limitation of effective strain level in FRP reinforcement	$\varepsilon_{fe} = 0.006$	$\varepsilon_{fe} = 0.004$	$\varepsilon_{f,\max} = 0.005$
Contribution reduction of the shear strength of FRP	-	- reduction factor is: ϕ	-
Calculus at limite state	U.L.S.+S.L.S.	U.L.S.	U.L.S.
Limitation of the reinforcement	It is not limited	$V_s + V_f \leq 0{,}66 \cdot \sqrt{f_c^{'}} \cdot b_w \cdot d$	It is not limited

To prevent failure from debonding mechanism	The strain level in FRP reinforcement is limited	The effective strain level in FRP reinforcement is limited	The stress variation is limited in the FRP reinforcement

6.5. PERSONAL CONTRIBUTION

Personal contributions, which result from the theoretical research and experimental program, relating to the design and behaviour of liniar reinforced and prestressed concrete members under the shear action, are:

-the results obtained after the design procedure by EC2 may be slightly unconservative for highly shear-reinforced members, and they are too conservative for slightly reinforced beams since the contribution of the concrete is not taken into account;

-the shear design of the beams with special detailing requires profesional experience;

-the application of the strut and tie method, requires engineering justification and experience gained over time in decision making during the adequately use of the model or even combination of two models in order to reach optimal results of economic and technical point of view.

-for the 24m bridge beam, passage DN1, there were achived studies which checked the requirements of the beam at it’s ultimate limits in service , respective cracks, deformation and tested the behavior under loading to the up to failure, for legal quality certification of the production of this elements by testing the first series and obtaining the necessarry data for the legal certification of conformity of the prefabricated element;

-for the L=11.45m roof beam, interex Vaslui, there were achieved studies which had as it’s purpose the checking, by tests on the stand, of the behavior up to failure as well as the resistance of the beam for the ultimate limits in service, with special attention to the support points as well as that of the preponderant action of the shear force;

-the application of composite materials to shear strengthed reinforced concrete beams with insufficient shear reinforcement, is an efficient means to improve their seismic behavior.

The author has published or communicated partial results of the achieved experimental studies, as follows:

- *Bartók (Pénzes) C., Verificarea experimentală prin încercare pe stend până la rupere a*

comportării unor prefabricate din beton armat, Referat nr.2 elaborat în cadrul proiectului de cercetare cu tema " Calculul și comportarea la forță tăietoare a elementelor din beton armat și precomprimat", Universitatea Ovidius Constanța, 2008;

- *Popaescu A., Deaconu O., Bartók (Pénzes) C., Încercarea pe stend până la rupere a unor elemente prefabricate de tip grindă din beton precomprimat și beton armat, Conferința "Structuri prefabricate din beton în centrul și estul Europei", 8-9 noe 2007 Cluj-Napoca;*
- *Bartók (Pénzes) C.,Study on test results of breaking point trials of prefabricated beam elements made of precompressed and reinforced concrete, Ovidius University Annals of Constanța, Series Civil Engineering, 2008.*

Recommendation for future research:

- Experimental studies relating to shear capacity of the continous reinforced concrete beams;
- Monitoring the behavior of the shear strengthened beams with deficit in shear resistance.

Bibliography

[1.1] [4.2] Agent R., Dumitrescu D., Postelnicu T.- Îndrumător pentru calculul şi alcătuirea elementelor de beton armat. Comentarii la prevederile prescriptiilor de proiectare. Scheme logice, tabele ajutatoare si exemple de calcul. Prescriptii si detalii de alcatuire constructiva. Editura Tehnică București 1992;

[1.2] [2.8] Nicula I. , Oneţ T.-Beton Armat. Editura Didactică și Pedagogică București. Ediția 1982;

[1.3] [4.12] Dumitrescu D. şi Popăescu A.-Beton precomprimat, Editura Academiei Române,1978;

[1.4] [4.3] STAS 10107/0-90, Calculul şi alcătuirea elementelor structurale din beton, beton armat, beton precomprimat;

[1.5] Kollár L., Dulácska E.-Ivhatás figyelembevétele vasbeton gerendákban az Eurocod 2 szerint (Importanţa mecanismului de arc în grinzile de beton armat după Eurocod 2)., articol din revista de specialitate Vasbetonépités, 2009-3;

[1.6] [2.3] [4.4] SR EN 1992-1-1 Eurocod 2: Proiectarea structurilor de beton, Partea 1-1: Reguli generale şi reguli pentru clădiri, iunie 2006;

[1.7.] [2.6] [4.9] Kiss Z., Oneț T.-Proiectarea structurilor de beton după SR EN 1992-1, 2008;

[1.8] Cladera A. și Mari R.-Shear strength in the new Eurocode2. A step Forward?(Forţa tăietoare în noul EC2. Un pas înainte?), Structural Concrete.2007. 8.No2;

[2.1] Barney T. Martin, Jr., Ph.D, P.E. Modjeski and Masters: American Association of State Highway and Transportation Officials (AASHTO)-Verification and Implementation of Strut-and-Tie Model in LRFD Bridge Design Specifications, november 2007;

[2.2] Brown, Michael D. Bayrak, Oguzhan- Design of Deep Beam Using Strut and Tie Models Part I: Evaluating U.S. Provisions;

[2.4] FIP[CEB-FIP]-Seismic design of precast concrete building structures, state-of-art report, preparated by Task Group 7.3, october 2003;

[2.5] FIP[CEB-FIP] Practical design of structural concrete. (chapter 6.5. Design of discontinuity regions), september 1990;

[2.7] Stoian V., Clipii T., Pintea D, -Proiectarea elementelor din beton armat folosind metode alternative-Conferința din Cluj Napoca, Structuri Prefabricate din Beton în centrul şi estul Europei, 2007;

[2.9] fib Bulletin 3:Structural Concrete-Textbook on behaviour, design and performance, vol. 3;

[2.10] Schlaich,J., Schafer, K.:Design and Detailing of structural concreteusing strat-and tie models. The Structural Engineer, Vol 69, No.6,1991 A;

[3.1] [4.15] Bartók (Pénzes) C., Verificarea experimentală prin încercare pe stend până la rupere a comportării unor prefabricate din beton armat, Referat nr.2 elaborat în cadrul proiectului de cercetare cu tema " Calculul și comportarea la forță tăietoare a elementelor din beton armat și precomprimat", Universitatea Ovidius Constanța, 2008;

[3.2] [4.16] Popaescu A., Deaconu O., Bartók (Pénzes) C., Încercarea pe stend până la rupere a unor elemente prefabricate de tip grindă din beton precomprimat și beton armat, Conferința "Structuri prefabricate din beton în centrul și estul Europei", 8-9 noe 2007 Cluj-Napoca;

[3.3] Bartók (Pénzes) C.,Study on test results of breaking point trials of prefabricated beam elements made of precompressed and reinforced concrete, Ovidius University Annals of Constanța, Series Civil Engineering, 2008;

[3.4] [4.1] Indicativ C181-88 Îndrumător pentru metodologia de încercare a prototipurilor şi seriei zero la elementele prefabricate, din punct de vedere al comportării la solicitări statice;

[3.5] [4.7] S.C. PROCEMA Engineering S.R.L., Proiect "Pană prefabricată", beneficiar S.C. SOMACO S.A. București,2006;

[3.6] [4.5] STAS 6657/1-89 Elemente prefabricate de beton, beton armat şi beton precomprimat. Condiţii tehnice generale de calitate;

[3.7] [4.6] STAS 12313-85 Poduri de cale ferată şi şosea. Încercarea pe stend a elementelor prefabricate din beton, beton armat şi beton precomprimat;

[3.8] S.C. EUROPROIECT S.R.L, "Pasaj Km 8+900", din cadrul proiectului "Fluidizarea Traficului pe DN1, Lărgirea DN1 între Aeroportul Băneasa (Km 7+535) și Pasaj Otopeni (Km 11+938)", beneficiar KOTA KONSTRUCT SA, 2005, reactualizat în 2006;

[3.9] [4.17] POPAESCU & CO SRL , Încercarea pe stend până la rupere a panei din beton armat l=11.45 m, Interex Centru Comercial Vaslui, Referat cu concluzii, 2007;

[3.10] [4.18] POPAESCU & CO SRL, Încercarea pe stend până la rupere a grinzii precomprimate l=24 m, pasaj DN1, km 8+900, între Aeroport Baneasa și Pasaj Otopeni, 2006;

[4.8] Șerbu A., Dimitriu Șt., Rezistența Materialelor,Universitatea Transilvania Brașov, 1999;

[4.10] Oneț T., Kiss Z., Muntean G.-Ghid pentru proiectarea betonului armat, Editura Transilvania, 1999;

[4.11] CR-1-3-2005-Cod de proiectare-Evaluarea acţiunii zăpezii asupra construcţiilor;

[4.13] Dumitrescu D., Agent R.,Nicula I., Găină N.,Popăescu A., Weissenberg M., Lissai F., Săndulescu G., Stănescu A.-Îndrumător pentru Proiectarea și calculul construcțiilor din beton, beton armat și beton precomprimat, Editura Tehnică, București, 1978;

[4.14] SR EN1992-1-1/NB, Eurocod 2: Proiectarea structurilor de beton, Partea 1-1: Reguli generale şi reguli pentru clădiri, Anexă națională, ianuarie 2008;

[5.1] Cod de proiectare seismică-Partea a III-a –Prevederi pentru evaluarea seismică a clădirilor existente, indicativ P100-3/2008

[5.2] Jayaprakash, J.(Department of Civil Engineering, University Putra Malaysia, Malaysia), Abdul Aziz, A.A.(Department of Civil and Environmental Engineering, College University Technology Tun Hussein Onn, Malaysia) Abang Abdullah, A.A.(Department of Civil Engineering, Universiti Putra Malaysia, Malaysia), Ashrabov, A.A.(Tashkent Automobile and Road Construction Institute, Tashkent). Shear resistance of Reinforced Concrete Beams with Bidirectional Carbon Fibre Fabric Strips. Proceedings of the 2nd International Congress June 5-8, 2006-Naples, Italy

[5.3] Hassan Dirar, S.M.O., Hoult, N.A., Morley, C.T., Lees, J.M. Shear Strengthening of Pre-cracked Reinforced Concrete Beams Using CFRP Straps.(Hassan Dirar, S.M.O., Hoult, N.A., Morley, C.T., Lees, J.M.Department of Engineering, University of Cambridge, Trumpington St., Cambridge, CB2 1PZ, UK. Proceedings of the 2nd International congress June 5-8,2006 Naples, Italy)

[5.4] Hoult, N.A. și Lees, J.M. Shear Retrofitting of Reinforced Concrete Beams Using CFRP Straps(Hoult, N.A. and Lees, J.M. Shear Retrofitting of Reinforced Concrete Beams Using CFRP Straps. Proceedings of the 4th International Conference on Advanced Composite Materials in Bridges and Strucutres (ACMBS-4), July 2004, Calgary, Canada.)

[5.5] Hoult, N.A. și Lees, J.M. Long-Term Performance of CFRP Strap Shear Retrofitting System. (Hoult, N.A. and Lees, J.M. Long-Term Performance of CFRP Strap Shear Retrofitting System.Proceedings of the 7th International Symposium on FRP Reinforcement for Concrete Structures(FRPRCS-7), November 2005, New Orleans, USA. (in press))

[5.6] Vintzileou, E.(Department of Structural Engineering, National Technical University of Athens, 5, Iroon Polytechnion Str., GR 15773, Zografon, Greece.). Shear Strengthening of RC Beams by means of CFRPs. Proceedings of the 2nd International Congress June 5-8, 2006-Naples, Italy

[5.7] Stoian V., Nagy-György T., Dan D., Gergely J., Dăescu C. -Materiale compozite pentru construcții, Editura Politehnică Timișoara, 2004

[5.8] Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structure, American Concrete Institute(ACI 440.2R-08), july 2008

[5.9] fib Bulletin 14: Externally Bonded FRP Reinforcement For RC Structures, Technical report, 2001

[5.10] CNR-DT 200/2004: Guide for the design and construction of Externally Bonded FRP Systems for Strengthening Existing Structures, july 13,2004