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INSIGHT INTO TECHNOLOGICAL ASPECTS OF THE EVOLUTION OF THE HENNEBIQUE REINFORCED CONCRETE SYSTEM

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ABSTRACT

The Frenchman François Hennebique (1842-1921) is well known for his involvement in promoting reinforced concrete at the end of the 19th century and the beginning of the 20th century. However, the efficiency of his famous system regarding the carrying capacity has not been evaluated yet even though it is a crucial parameter for reusing any construction. Moreover, this system has changed during the decades of practice of the Hennebique Company considering the evolution and large diffusion of reinforced concrete around the 1910s.

We have the opportunity to analyse original computation details and technical constructive blueprints for several Belgian projects from the Hennebique office from 1900 up to 1930 to follow the evolution of the patented system. This paper describes two complementary structural aspects of the system based on these cases studies. Firstly, the evolution of the type and location of the rebars of the system is analysed. Secondly, the design methods are reviewed for the beam-floor element. Indeed, a careful attention is paid to the typical straight T-beam as it is the main structural element of the Hennebique monolithic construction. This study investigates the calculation methods and practical implementation of the Hennebique system in order to identify its possible structural weaknesses.

According to our research, the Hennebique firm changes his system along the years but kept his calculation method, at least until the 1930s. Therefore, the identification of the system Hennebique in an early r.c. construction, i.e. dating before the 1930s, is rather straightforward in practice.

Keywords: *Reinforced concrete, Structural analysis, Heritage preservation, History of construction, Building technologies*

1. INTRODUCTION

The name of the self-made man François Hennebique is strongly related to the birth of reinforced concrete (r.c.) as a new structural material. Indeed at the outbreak of the First World War, his firm is already responsible for more than 60000 civil engineering projects designed all over the world [1]. According to our thematic inventory on early r.c. constructions in Brussels, more than 80% of the r.c. constructions erected before 1914 is attributed to Hennebique. The main aim of this paper is to identify the possible transformations along the years of the Hennebique system composed by beam-floor and column. Hennebique used the same typology of beam-column frame, which forms his core business, for buildings, girder bridges, industrial sheds... This system brought indeed the celebrity to the Hennebique Company. Hennebique paid also attention to foundations [2]. However, the substructure is beyond the scope of this paper, as well as the realisation of stairs, silos, elevators and retaining walls. The main elements of the Hennebique system are analysed from the geometrical characteristics point of view and from the aspect of theoretical calculations. The evolution is determined thanks to careful examination of the Hennebique Belgian patents and twenty practical Belgian case studies from 1898 till 1933 [1, 2]. Furthermore, literature published at the beginning of

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the 20th century is also a source of information, even if numerous articles are copies from a few primary references (Christophe's book for example). Therefore misunderstandings and mistakes are repeated abundantly.

2. MAIN CHARACTERISTICS OF THE HENNEBIQUE SYSTEM

2.1. Geometrical features from patents to reality

2.1.1. Rough descriptions of the Hennebique system in the Belgian patents

Hennebique submitted 17 patents about r.c. in Belgium between 1886 and 1912 from floor to pipes and deep foundations [2]. All the elements of the future typical Hennebique system are already contained in the patent of July 1892 but at a preliminary stage. Indeed, the beam including the slab is made of concrete, the main longitudinal rebars present a round section and transversal reinforcement is provided by the insertion of stirrups, as flat U shaped section. However, it is only in the patent of September 1896 that the stirrup receives its final shape and the beam its actual T- shaped cross section. The column appears also in this last patent and is reinforced by means of longitudinal round rebars. Metallic plate is proposed as transversal reinforcements. Moreover, a haunch inclined at 69° above the horizontal provides the monolithic transition from the column to the T-beam [2]. From December 1897, the continuous T-beam is composed with straight and bent-up rebars, round and smooth with fish-tail ends, and hoop stirrups along the span. Moreover, reversed flat stirrups are added on the intermediary supports. In this last patent, Hennebique explains that the adequate combination of steel and concrete leads to structural elements resistant to bending and shear solicitations [2]. The description insists on the role of each components of the beam. The longitudinal rebars support tensile stresses and their position follows the bending moment. Concrete holds compression only. The stirrups counteract diagonal tension and connect compression and tension zones of the beam "acting as suspension rods in a metallic truss" [2]. Moreover, the inverted stirrups on the supports maintain in place the bent-up rebars and strengthens the building-in of the beam on the column. Besides this information, no explanation on the calculation methods neither recommendation on the materials, except briefly in January 1886, is provided in the descriptions belonging to the patents [2].

2.1.2. Hennebique constructions through Belgian real case studies

The classic Hennebique system is well represented by the logo of the firm (Fig. 1). However, Hennebique modified the arrangement and type of rebars with time, as already observed in the analysis of patents. The study of its evolution is based on twenty actual Belgian applications of the Hennebique beam-floor and column system, from public, industrial and religious buildings, social housing and bridges.

Hennebique designs rectangular columns, even almost always square section in proportion with the loads to be carried. Therefore the section of the column and the diameter of the reinforcement usually diminish with the upper floors. The longitudinal reinforcement has always a round and smooth section, usually consisting of 4 bars without fish-tail on the contrary of the end finishing of the main reinforcements present in beam.

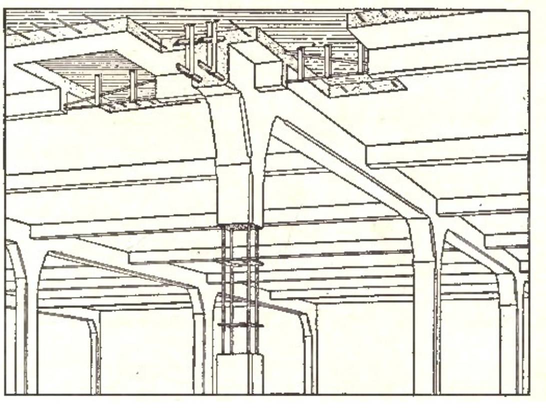

Fig. 1 Emblem of the Hennebique system [3]

The cross binding reinforcement is repeated every 10 to 50 cm, usually between 25 and 40 cm. At the beginning of his system, the transverse reinforcements are perforated metal sheets. These flat plates have a thickness of 2-2.5 mm and a width of 40-45 mm. Four holes allow to receive the longitudinal rebars. These flat plates were used up to around 1910. Their main disadvantage is the difficulty to place correctly the plates during the execution, because they have to be thread on the plain round rebars as concrete is poured. Probably due to the needs of execution, transversal reinforcements becomes wire ties, meaning round smooth cross binding, as a frame for two by two longitudinal rebars (for instance electric substation of Ixelles, 1911). Moreover, the head of the column is often a haunch support. An angle, higher than 45° above the horizontal line, is found in practical examples (as for the court house of Verviers, 1898) and in the logo of Hennebique (Fig. 1). However, all the other case studies show haunch with an angle between 24° and 45° (for instance Colo-Hugues viaduct in Braine-l'Alleud, 1904) (Fig. 2).

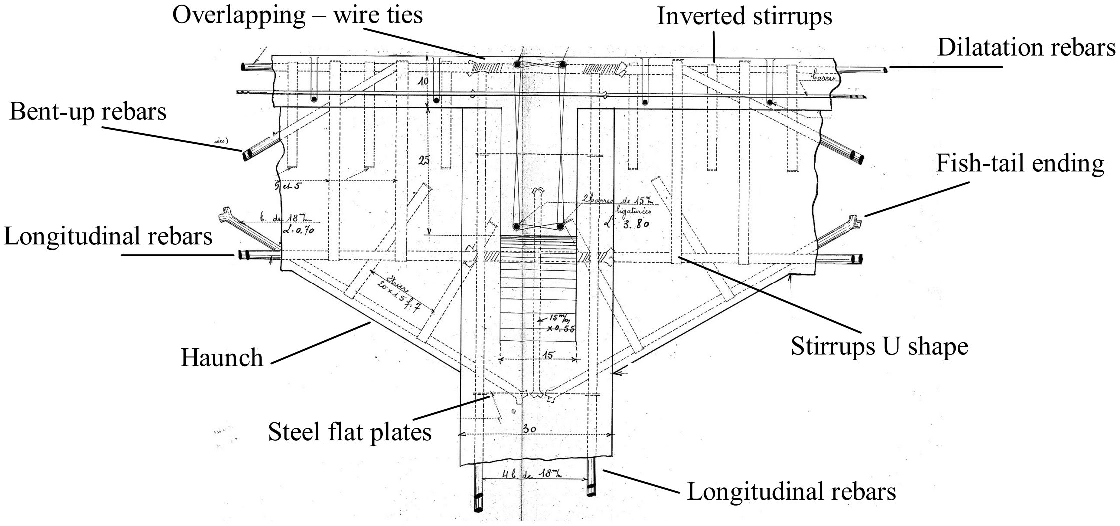

Fig. 2 Details of T-beam section at an intermediate support (Colo-Hugues viaduct in Braine-l'Alleud, 1904) [1]

Concerning the beam-floor, Hennebique proposed that the beam and the slab collaborate to form a T section. In most cases, the Hennebique system includes secondary beams supported by the main beams (Fig. 1). As already explained plain round bars are located in tension and follow the bending moment. Half of the rebars are straight in the bottom of the web and half are cranked-up rods that lie on top of the flange on the intermittent supports (Fig. 2). Usually the bars are inclined at one third of the span of the beam. In late projects around 1930, the inclination of the rebars starts closer to the support than at one third of the span. Sometimes, inclined bars are also placed in the slab. The bars are flattened and opened at the end to ensure anchorage in concrete (Fig. 2). The actual role of the fish-tail ending is not demonstrated nowadays. Even if other methods of anchorage existed at that time (for instance the Considère hook), many examples suggested that Hennebique kept the fish-tail ending till the 1930s. The overlapping of the rebars occurs at the supports. Bent-up rebars are joined by means of wire ties to resist tension (Fig. 2). The diameters of plain round bars from 1 to 50 mm were easily available in the iron and steel industry [4]. According to our studies, Hennebique favoured diameters for the longitudinal rebars between 8 and 40 mm. The vertical hoop iron stirrups have a U shape, open and curved at 45° or a quarter of circle on top (Fig. 2). The section is standardised, with a width of 30 mm and a thickness of 2 mm or a width of 20 mm and a thickness of 1.5 mm. The section of the stirrups remains constant and the spacing follows the increment of shear force [5]. However this well-known geometry changes in the 1920s, when stirrups became closed and even round wire section (for example for the office for the newspaper Le Soir in Brussels, 1928-1933). Some inverted stirrups are also placed downwards to maintain the bent-up rebars on the supports and along the length of the beam when there is compression reinforcement or dilatation rebars (usually Hennebique did not provide expansion joints in bridges) (Fig. 2).

In conclusion, a simplification of the Hennebique system is already observable in 1910. For instance, the spacing of stirrups along the span is almost always the same in the different projects. Hennebique made uniform the width of the slab according to the thickness of the compression flange, 3 m for 8 cm thickness, 3.50 m for 10 cm and 4 m for 12 cm thickness. The depth of the rib is usually between 1/18 and 1/14 of the span of the beam [6]. The longitudinal rebars in columns are placed at 2.5 to 4 cm from the exterior surface. For beams and slabs, the cover of bottom longitudinal reinforcement is theoretically between 1.5 and 3 cm, due to technological reason of pouring of concrete specific to the Hennebique system (section 2.2.).

2.2. Components of the materials and execution onsite

The first Belgian patent of Hennebique (1886) mentions that reinforcements could be in iron or steel, with round, square or plane sections connected with cross-bars and that concrete is composed of hydraulic lime or cement, added with sand, gravels, clinker or fly ashes. Except in this patent, the recommendations on the materials are often missing in the Hennebique folders.

From around 1902, the most common proportion for Hennebique concrete mixture is 300 kg of Portland cement, 800 l for gravels and 400 l for sand. It has been confirmed in a few documentary calculation evidences. Christophe reported also other compositions for instance, with 250 kg of cement, 850 l gravels and 400 l sand if concrete is mixed mechanically or even 300 kg cement, 1000 l gravels and 150 to 250 l sand [5]. This last composition needs supplementary water to compensate the voids left by the aggregates. Furthermore, Hennebique modified his proportion for piles, vaults and hydraulic works [4]. Changes in proportion according to the elements (column, beam, floor) in one construction was also an accepted practice at the turn of the 20th century [8]. However, this practice did not occur anymore from around 1905 onwards [6]. The amount of water is a missing data in the literature, even if building contractors were aware of the role of this component on the final resistance of concrete. The quantity of clean water had firstly to be sufficient enough for the setting of cement and secondly to be limited for avoiding segregation of the aggregates. But due to the hand ramming, water was probably in excess according to present prescriptions. Furthermore, no additive is explicitly mentioned in the composition of concrete used by Hennebique or by any other competitors. However, in the journal "Le Béton Armé", calcium chloride in concrete mixture is proposed to speed up the setting, to lower the freezing point and to increase the water imperviousness [7]. This addition is actually detrimental for durability issues. Nevertheless, so far, no evidence of the use of this product had been found in the Hennebique concrete mixture. It suggests that the mixture was basically cement, sand and aggregates, even if the possibility of admixture is plausible. The nature of the aggregates is sometimes stipulated, for instance in the Caisse Générale de Report et de Dépôts, Brussels, 1910 [1]. In Belgium, Hennebique seems to have used either pit gravels from quarries of Quesnat (porphyry stones), or river gravels from Meuse or Rhine or an equivalent quality. The dimensions of the coarse aggregates varied between 5 to 20 mm in one single composition, to offer a high compactness and optimize the voids and fillings. Quartz sand was usually recommended. In Hennebique constructions, Portland cement seems exclusively to be used as sole binder since the 1900s, excluding any slag cement. Furthermore, the origin of cement was sometimes specified as coming solely from the Belgian cement industry [1]. Around 1900, mixing the components of concrete was often handled by hand but from 1904, the use of mechanical mixers increased and at the outbreak of the First World War, the majority of r.c. constructions employed them [6, 8].

The Hennebique firm did not explain its usual procedure of pouring onsite but several sources in the literature comment it similarly [3, 5]. Firstly, a concrete layer of around 2.5 or 5 cm is placed in the bottom of the beam mould and rammed up to half of the initial thickness. Punning was made by hand with a wooden or cast-iron tamp. Indeed the "concretors' tools" were simple and closed to the mason utensils [9]. The falsework was usually in timber. Secondly, the first tension bars and the stirrups slipped under them are introduced and maintained in the right position thanks to a small mound of concrete. Thirdly, a second layer of concrete is poured and rammed and thus the second layer of rebars is installed. Fourthly, concrete is added by layers and reinforcing rods are inserted gradually up to the top of the flange. The main drawback of this way of erection is the necessary care to avoid any movement of the reinforcements during the pouring and ramming of concrete. The benefit of such method is the possible inspection to control regularly the correct location of each element [3, 5].

For metallic reinforcement, Hennebique used first iron but recommended as early as 1900 the use of mild steel for all the reinforcements. According to tensile tests and metallographic analyses performed in our laboratory on 17 samples from the Colo-Hugues viaduct of 1904 (straight and bent-up rebars of diameter 31 mm and stirrups of 20 mm width and 2.5 mm thick), the mechanical properties and carbon percentage correspond to this classification with an average yield strength of steel of 298 MPa, a mean ultimate tensile strength of 380 MPa, an ultimate strain around 33% and a carbon percentage less than 0.1%. It is worth mentioning that the ductility of those plain smooth rebars is larger than in today r.c. constructions. Moreover, flat stirrups and round rebars are from the category of ingot iron/steel, even if weld iron/steel were also produced for rods for r.c. [10]. Indeed, the presence of local necking and the values of ultimate strain prove the affiliation to the category of ingot procedure. It is also confirmed by the name conferred to the ingot process ("acier" in French) in Belgium at that time [11, 12]. From the ingot procedure, Mouchel, the licensee of Hennebique in the United Kingdom, indicates "steel shall be mild steel, produced by an open-hearth, basic or acid process. Neither Bessemer steel nor high carbon-steel shall be employed" [13]. However, Bessemer steel seems to have been used in the case of the Luiz Bandeira Bridge, Portugal, 1907 [14]. Several works had to be done onsite to adapt the iron work to their final destination, such as cutting, bending, ends splitting. In the Hennebique system, steel rebars were bent whilst cold with wrenches or equivalent machineries [15]. However, if the diameter of the rebar was higher than 25 mm, the splitting in fish-tail shape was made when iron was still hot [5]. The stirrups were prepared *in situ* and bent by hand around the rebar of the same diameter as in the final place. The extremities are bent with a hammer. The steel plates for the transversal reinforcement in a column were usually perforated before the delivery onsite.

3. SEMI-EMPERICAL METHOD OF DESIGN: HENNEBIQUE PECULIARITIES

3.1. Introduction of the working stresses theory, the establishment of the classic theory

The theoretical design of r.c. is still under construction at the beginning of the 20th century with numerous methods proposed. "The theory of r.c. is, unfortunately, in a very unsatisfactory state at the present time, and (...) no kind of universal agreement has been arrived at." [9]. Nevertheless, the so-called working stresses theory was encouraged by many national standards, among which the French regulation of 1906 used also in Belgium [16]. The elastic theory determines the stresses in a homogenised section, based on the modular ratio between the modulus of elasticity of the two materials, concrete and steel, often named n or m . Depending on national regulations, its value varies from 8 to 20 [16]. The theory leans on the equations of equilibrium, deducing the position of the neutral axis and finally determining the maximum stresses in concrete and steel sections. This classic theory, formalized at that period, will become the common theory for the next decades, until the introduction of the limit state analysis in the 1970s. Five assumptions underlying the working stresses theory were widely accepted. Firstly, the conservation of the plane section before and after deformation, called the principle of Navier-Bernoulli. Secondly, the stresses in steel or concrete are proportional to strain, thus it implies that the Young moduli of steel and of concrete are constant.

$$n = \frac{E_s}{E_c} \tag{1}$$

with E_s = the Young modulus of steel, E_c = the Young modulus of concrete.

Thirdly, the concrete tensile strength is equal to zero. Fourthly, the full composite action is developed between steel and concrete. Finally, initial stresses due to changes of temperature or shrinkage are neglected.

According to this theory, the values of stresses inside sections must remain below a fraction of the value of the maximal strength (2).

$$\sigma_{\max} \leq \sigma_{allow} = \frac{\sigma_{failure}}{K} \tag{2}$$

with $\sigma_{\max}$ = maximum stress in the critical section, σ_{allow} = permissible stress, $\sigma_{failure}$ = failure stress of the material, K = safety coefficient on the material (around 2 for steel and at least 3 for concrete).

For steel, the limit is a fraction of the yield strength, imposed at around 100 MPa at that time. For concrete, the limit is a fraction of the compressive strength on cube, around 4 MPa. The modelling based on this theoretical design allows no redistribution of stresses inside sections, thus the real bearing capacity is not approached by this method.

3.2. Theoretical conception of the Hennebique office: focus on T-beam

The peculiar method of calculation of Hennebique, in contrast with its contemporary, was divulged in several publications as for instance by Christophe (1899), De Tedesco and Maurel (1904), *Ferro-Concrete* (1910) [5, 6, 10]. Nonetheless, no detailed explanation accompanied the assumptions, the formulae, etc. For decades (at least up to the 1930s), the Hennebique office did not change its calculation method, whatever the regulations in force. Indeed, Hennebique did not have any confidence in the hypotheses behind the modular ratio method [1].

3.2.1. Assumptions of the Hennebique calculations

The values of the applied loads in Hennebique projects depend on the destination of the constructions (Table 1). These values are comparable to the estimations published in literature of that time and are also similar to current standards [9, 17]. However, even if the actions are differentiated between dead loads and live loads, they are not weighted.

Table 1 Common values of loads applied by Hennebique firm

	Type of loads	**Values**
Self weight (in kN/m³)	Reinforced concrete	25
	Masonry	20
	Pavement	18
Superimposed loads (in kN/m²)	Ordinary dwelling house, flat, etc.	2.5
	Staircases	3
	Ordinary loads	4
	Public building (bank, office, theatre, etc.)	4 - 6
	Factory, warehouse	10 - 30
	Library, archive	20 - 30
	Interior pavement	1 – 1.5
	Roof	2.5

The allowable values are frequently specified by Hennebique: allowable compressive strength between 2.5 and 5 MPa for concrete (in average 4 MPa), allowable tensile strength from 90 to 150 MPa for steel in tension (in average equal to 100 MPa or 120 MPa), and 70 to 80 MPa for steel in shear.
The hypotheses of the Hennebique calculations are the following.

1. Concrete resists only in compression, not in tension.
2. No slip occurs between concrete and steel.
3. Initial stresses due to changes of temperature or shrinkage are neglected.
4. The bending moments of internal concrete compressive force and of internal steel tensile force are each equal to half of the external bending moment.
5. The distribution of the concrete compressive strain is uniform over the compressive zone.
6. The resultant of compressive force is applied at the middle of the depth of the flange and the resultant of tensile force at the centre or the reinforcement bars located in tension.
7. The position of the reinforcement working in tension is chosen (usually equal to 2.5 or 5 cm from the bottom fiber).
8. Straight longitudinal rebars and bent-up rebars present the same section (diameter and number).
9. The shear force is taken half by the stirrups and half by the bent-up rebars.
10. The section of the stirrups is determined beforehand (20 mm x 1.5 mm or 30 mm x 2 mm).
11. The dimension of the column is imposed beforehand.
12. The geometry of the flange, width and depth, is established beforehand.
13. For resolving the moment equilibrium for a T-shaped cross section, two different hypotheses are made; either the neutral axis is fixed beforehand, and in such a way always located in the web, or the section of reinforcements is defined.
14. If the neutral axis lies in the web, only the section of the slab is taken as the section of concrete participating in the compressive resistance (the concrete compressive zone in the web is not considered). If the neutral axis lies inside the slab, the whole thickness of the slab is still resisting the compressive stresses.

3.2.2. Longitudinal stresses

The beam is designed as a continuous structure, therefore the maximum positive bending moment at mid-span should be determined as well as the maximum negative moment at the supports.

Firstly, Hennebique supposed a simply supported beam under uniformed loads and found the bending moment at midspan (4). If the bending moment concerns secondary beam, it is equal to (5), as partially built-in beam.

$$M_{iso}^{+} = \frac{p \cdot L^2}{8} \tag{4}$$

$$M_{iso}^{+} = \frac{p \cdot L^2}{10} \tag{5}$$

with M_{iso}^{+} = bending moment at midspan, p = uniformed external loads, L = span.

Secondly, he stated the conditions of equilibrium between moments of a simply supported beam and of a statically undetermined beam (6).

$$M_{iso}^{+} = M_{hyp}^{+} + M_{hyp}^{-} \tag{6}$$

with M_{iso}^{+} = bending moment at midspan for a simply supported beam, M_{hyp}^{+} = statically indeterminate positive moment at midspan, M_{hyp}^{-} = statically indeterminate negative moment at intermediate supports.

Thirdly, the calculation of the statically indeterminate positive moment is executed, based on the equilibrium conditions freely imposed (7) and (8) (Fig. 3).

$$\frac{M_{hyp}^{+}}{2} = \sigma_{c,allow} \cdot e \cdot b \cdot (x - \frac{e}{2}) \tag{7}$$

$$\frac{M_{hyp}^{+}}{2} = \sigma_{s,allow} \cdot A_s \cdot z_{s1} \tag{8}$$

with M_{hyp}^{+} = bending moment at the mid-span due to the applied loads, $\sigma_{c,allow}$ = concrete allowable stress, x = position of the neutral axis, b = width of the slab (distance between centre to centre beam), e = depth of the slab, A_s = steel section in the mid-span, $\sigma_{s,allow}$ = steel allowable stress, z_{s1} = distance of the tensile steel section to the neutral axis.

Moreover, Hennebique imposed also the depth of the beam (9).

$$H = x + z_{s1} + c \tag{9}$$

with H = depth of the beam, x = position of the neutral axis, c = the cover of the reinforcement, z_{s1} = distance of the steel tensile section to the neutral axis.

As mentioned in section 3.2.1 (hypothesis 13), two options are possible. Either A_s is chosen and the values of M_{hyp}^{+}, z_{s1}, x are determined with the equations (7), (8) and (9). Or Hennebique fixed the position of the neutral axis, so the position of the neutral axis x and M_{hyp}^{+} can be deduced from the equation (7) and the steel section A_s is calculated with equation (8). Then, for both choices, the value of M_{hyp}^{-} is deduced from the static condition of equilibrium between M_{hyp}^{+} and M_{hyp}^{-} with the equation (6). Now, the section at midspan is completely determined or verified. However, it should be noted that the fundamental equation of equilibrium of the internal forces is not verified within this model.

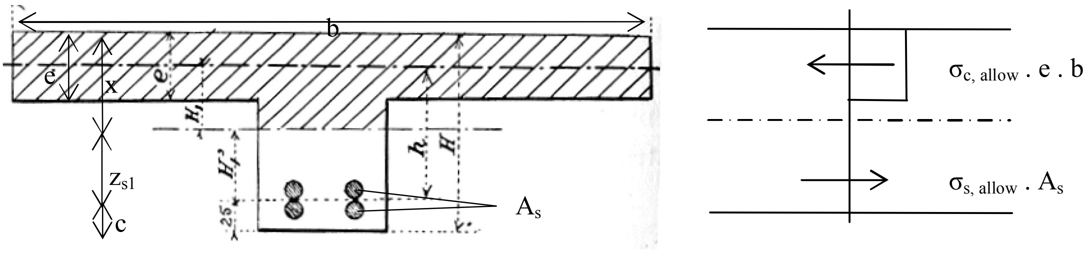

Fig. 3 Flexural analysis of Hennebique, example for a T beam section at mid-span [9]

For the section at the supports, Hennebique settled the steel sections and position in tension and in compression. Thus, the calculation of the neutral axis at the support is performed with (10).

$$\frac{M^{-}_{hyp}}{2} = \sigma_{s,allow} \cdot A_{s1} \cdot z_{s1} \tag{10}$$

with M^{-}_{hyp} = bending moment at the support due to the applied loads (known, from (6)), $\sigma_{s,allow}$ = steel allowable stress, A_{s1} = steel section working in tension at the support (chosen), z_{s1} = distance of the tensile steel section to the neutral axis (unknown).

When the neutral axis is found out with equation (10), the last step is to verify the stresses inside the section in compression at the support (11).

$$\sigma_{c,allow} \cdot z_{s2} \cdot a \cdot (z_{s2} + \frac{c_2}{2}) + \sigma_{c,allow} \cdot A_{s2} \cdot z_{s2} \geq \frac{M^{-}_{hyp}}{2} \tag{11}$$

With M^{-}_{hyp} = bending moment at the support due to the applied loads (known), $\sigma_{c,allow}$ = concrete allowable stress, a = width of the web of the beam, c_2 = concrete cover of the compressive steel section (chosen), $\sigma_{s,allow}$ = steel allowable stress, A_{s2} = steel section working in compression at the support (chosen), z_{s2} = distance of the compressive steel section to the neutral axis (known).

As explained previously, many case studies present a haunch at the head of the column. However, there is seldom a calculation detail of the geometrical dimensions of the haunch. According to our understanding, the depth of the haunch follows these two rules (12) and (13), leaving the resistance of concrete in compression aside.

$$M^{-}_{hyp} = \sigma_{s,allow} \cdot A_s \cdot H \tag{12}$$

with M^{-}_{hyp} = bending moment at the support due to the applied loads, $\sigma_{s,allow}$ = steel allowable stress, A_s = steel section working in tension at the support, H = distance of the haunch from the tensile steel section to the surface (unknown).

Therefore, the depth of the haunch is equal to (13).

$$H_{haunch} = H + c \tag{13}$$

with H_{haunch} = distance of the haunch (unknown), H = distance of the haunch from the tensile steel section to the surface (found with (12)), c = concrete cover of the tensile steel section (chosen as 2 cm usually).

3.2.3. *Shear stresses: vertical and longitudinal shear*

The combination of vertical and inclined reinforcements is the system adopted by Hennebique since 1897. Furthermore, the stirrups are the trademark of the Hennebique system, which also contributed largely to the notoriety of the firm. However, "from the experiments made by Mr. Hennebique on beams reinforced differently, it seems that bent-up bars of his system give more resistance to sliding forces than stirrups" [5].

First, Hennebique checked that a "notional" shear stress is lower than the allowable shear stress (14).

$$\tau = \frac{V}{A_s} < \tau_{s,allow} \tag{14}$$

with A_s = cross section area of the main longitudinal steel tensile section at mid-span, V = maximum value of the shear force (at the support), $\tau_{s,allow}$ = shear steel permissible stress.

Secondly, for the seizing of the stirrups, Hennebique assumed that the shear force is taken half by the stirrups and half by the bent-up rebars. However, the design of the bent rebars is always missing in the engineering computations of Hennebique office. Only the section of the stirrups is calculated from (15).

$$A_{TotalStirrup} = \frac{V/2}{R} \tag{15}$$

with $A_{TotalStirrup}$ = total section of the stirrups used in a transversal section of the beam (= $A_{Stirrup} \cdot n$ with $A_{Stirrup}$ = 30 mm² or 60 mm² for one arm and n = 4 arms of stirrups in one section if there is 2 stirrups, which is the usual reinforcement), $V/2$ = half of the shear force at the supports, R = the maximum resistance that the stirrups can withstand. This value is imposed by Hennebique, equal either to 100 MPa, 80 MPa or 70 MPa. Actually, the meaning of this value is not consistent in the literature. When following the calculation details of Hennebique, it seems that the value corresponds to the shear allowable stress of steel. But this conception is opposed to the principle of stirrups, supported by this quotation "the company emphasises that the bars and stirrups reinforce the concrete against diagonal tension" [9]. According to Talbot, R equals the value of the allowable direct tensile strength of steel [4, 18]. According to Marsh "it is usual to allow a safe tensile working stress on diagonal tension reinforcement of only three-quarter that allowed for direct tension" [16].

The Hennebique shear calculation method has absolutely nothing in common with the (correct) concepts developed by Ritter (1899) and Mörsch (1902) about shear stresses [18].

Then, the spacing of the first stirrups located near the supports is based on (16)

$$e = \frac{A_{TotalStirrup} \cdot z \cdot R}{V/2} \tag{16}$$

with e = the spacing between stirrups, $A_{TotalStirrup}$ = total section of the stirrup, z = level arm of the internal forces, R = the maximum resistance that the stirrups can support, $V/2$ = half of the shear force.

The spacing of the following stirrups should also be determined "if, as usual, we give them (the stirrups) all the same section, it is the spacing that should vary from the supports to the middle. (...) The stirrup spacing increases so that for each stirrup the variation of bending moment remains the same" [5]. As far as the Hennebique stirrup spacing is concerned, the formula in use was (17) [19, 20].

$$\Delta e_k = \Delta e_{k-1} + k \tag{17}$$

with Δe_k = interspacing of the stirrup, k = number of the stirrup, Δe_{k-1} = interspacing of the previous stirrup. We observed in many examples this arithmetic progression, until the spacing reaches the effective depth of the beam. Then, the spacing remains constant (usually around 40 cm).

3.2.4. Discussion

According to the concepts presented above, the Hennebique r.c. design is closer to the physical behaviour of the structure than the estimation made with the elastic classic theory commonly in use at that time. The designs of slab, beam and column are not based on the compatibility of deformation, but on the hypothesis that concrete and steel work at their maximum of capacity. His estimation of the positive and negative bending moments is actually approaching the solution of the plastic design of a continuous beam, even if his method violates one fundamental condition of equilibrium which constitutes an important drawback [21]. He also assumed that concrete has a rectangular stress block in compression, which is innovative at that time (proposed in 1912 only by Suenson). However, several parameters are taken arbitrary. For instance, Hennebique considered a partially built-in supported slab but do not always provide the reinforcement at the supports. Moreover, his calculation is based on fixing *a priori* the neutral axis. In addition, several parameters were not calculated as for example the inclined rebars and many technological measures were lacking in calculation details, as anchorage length, overlap length, concrete cover, interspace between rebars, etc. Furthermore, the stirrups are open in the compression zone which is totally ruled out today and since a long time.

4. CONCLUSION

The famous commercial success of the system is due to several reasons. One of those is, firstly, the simplicity of the materials used by the Hennebique Company to erect a r.c. construction, with plain round rebars and an elementary concrete composition, all available everywhere. Secondly, his system beam-floor and column, valid for any type of construction, was almost standardized from design to onsite construction techniques. Nevertheless, an evolution of the Hennebique r.c. components and global geometries is observed from the first patents in 1886 to the system applied in the 1920s. For the column, the main changes concern the transversal reinforcements. The perforated metal sheet became wire ties. Moreover, the presence of haunch is unsystematic as well as the angle of inclination from 24° to 69°. For the beam, the inclination of the bent-up rebars change with time. The typical flat stirrups became round rebars in the 1920s. For the rest the system remains largely unchanged for decades. Thirdly, the design of the different structural elements is simplified to be efficient for the engineers calculating in offices and to result for hand workers in an economical system onsite. The organisation with building contractors and agents, exclusive licenses of Hennebique, reinforces also the simplicity and homogeneity of the system. In Belgium, the exclusive patents were no longer applied systematically and their predominant role stopped gradually thanks to the French regulation (1906). However, Hennebique did not change his method of calculation until the late 1920s. Indeed, Hennebique relied more on his own semi-empirical method for designing r.c. works than on working stress analysis based on the elastic modular ratio theory. His approach is actually close to current design based on the real bearing capacity, with both materials working at their limit. As Lossier wrote "shall we see in his original designs one of these rare intuitions that are parts of a genius mind?" [22].

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