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ORIGINAL ARTICLE

SDSS 2022
The International Colloquium on Stability and Ductility of Steel Structures
14-16 September, University of Aveiro, Portugal
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# Technical Study of Kongo-Central Colonial Steel Houses Belonging to the DRC Patrimony

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## Abstract

The built heritage is a collection of properties of heritage value that should be protected and preserved. Designed in Belgium and built in the Democratic Republic of Congo (DRC) at the beginning of colonisation, the steel-framed houses in the province of Kongo-Central are in an advanced state of damages due not only to the climatic conditions of the area, but also to those of exploitation. An architectural and technical study of these steel houses is nowadays required to develop solutions for their conservation as part of the DRC's built heritage. In this context, the National Fund for Scientific Research of Belgium (FNRS) is financing the research project entitled "Tropicalisation of engineering and architecture". This project is carried out jointly by the Université libre de Bruxelles (in charge of the architectural part) and the Université de Liège (in charge of the engineering part). For this project, the courthouse of Mbanza-Ngungu (former ABC Hotel in Thysville) is studied in detail as a case study to identify the techniques used during construction, the pathologies affecting the structures and the solution which could be contemplated for their refurbishment. The present paper will introduce first investigation conducted in the framework of this project.

## Keywords

Lightweight structures, Historical steel houses, Patrimony, Corrosion.

## 1 Introduction

England, considered to be the pioneer of the industrial revolution, is credited with the first references to the export of metal houses for residential use outside Europe. According to the literature, one of the first main and emblematic exported metal houses is the palace for King Eyambo of Nigeria built by the Laycock Company of Liverpool in 1843 [1]. Some companies in Belgium, the second largest industrial power in the world in the 19th century [2], were also involved in the development of metal houses with examples shown at the Brussels exhibition in 1841 [1] but the first main exported metal house a hospital ward, with a metal frame and filling, at Camp Jacob, Basse-Terre in Guadeloupe, in May 1846, at the request of the French government; this building was developed by the "Société Anonyme de Hauts-Fournaux de Couillet", together with the French engineer-architect Antonin Romand and the "Etablissements Lachaussée de Liège" ([1] & [3]). Later, around 1849 at the time of the gold rush, this Couillet company as well as other Belgian steel companies supplied prefabricated metal buildings in California ([1] & [4]).

The iron and steel industry continued to develop thanks to technological innovations to meet the growing and diversified needs for metal elements. In 1855, the Englishman Henry Bessemer was the first to develop a new process for the industrial refining of cast iron to make steel [2]. Other industrial refining processes (Siemens-Martin furnace in 1864, and Thomas-Gilchrist furnace in 1877) from cast iron to steel succeeded to the Bessemer process and improved the qualities of steel towards the end of the 19th century. During the second half of the 19th century, steel and iron coexisted as materials for steel construction, but the improvement of steel production techniques allowed it to supplant iron at the beginning of the 20th century, leading to the bankruptcy of the iron industry [5].

During the construction of the Matadi-Léopoldville (now Kinshasa) railway line in the Free State of Congo (FSC) (now DRC), between 1890 and 1898, the development of housing facilities to protect workers from the sun and bad weather was required. At the beginning, the workers were accommodated in tents or adobe huts (planned to last only few months), but these accommodations were rudimentary, inflammable and poorly protected against bad weather [6]. This led to the idea of importing houses made of materials from Europe.

In 1888, the "Société Anonymes des Forges d'Aiseau" was the first to supply more than twenty houses and buildings of all kinds (stations, sheds, administration buildings, hotels, prisons, churches, etc.) [4]. However, these buildings, whose framework and filling were made of metal elements, had the disadvantage of having a low thermal inertia inducing high temperature inside the building during the day and during the hot season [7].

 

https://doi.org/10.1002/cepa.1852 

So, the idea was born to import steel framed structures associated to concrete wall constructions, which gave satisfaction [7]. Among the Belgian companies that supplied these types of construction, the “Grandes Chaudronneries de l'Escaut (GCE)” can be highlighted; this company was specialised in shipbuilding, but, in the early 20th century, also supplied residential houses, larger buildings, and hotels to Congo (see Figure 1) [1].

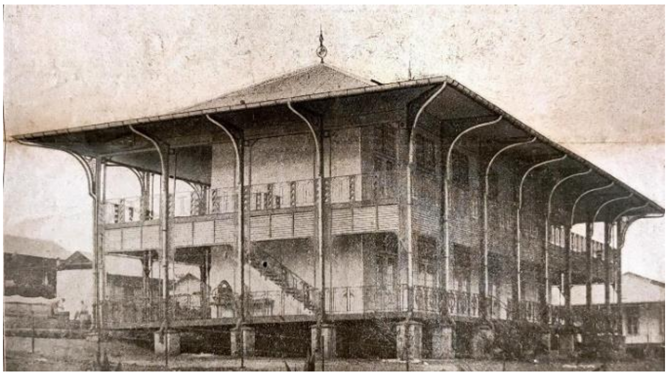

Figure 1 : former ABC Hotel of Thysville - current courthouse of Mbanza-Ngungu (source: NEPTUNE newspaper of January 20, 1914)

The present article is focusing on these houses, more precisely on the buildings in the town of Mbanza-Ngungu (formerly Thysville) in Kongo-Central (formerly the province of Bas-Congo). The courthouse of Mbanza-Ngungu (former ABC Hotel) (Figure 2), which was built in 1906 for the “Compagnie du Chemin de Fer du Congo (C.C.F.C.)”, is selected as a case study because of its current importance and its scale compared to other buildings in the town.

Although these metal houses were built in a period where (inter)national codes and rules for structural design were limited, they have stood the test of time and some of them have been able to guarantee the safety and comfort of their users despite the advanced state of deterioration of some of the load-bearing steel elements. In Section 2, the structural system of the selected case study will be detailed and analysed. Then, based on the ISO 9223[8], ISO 9224[9] and EN12500[10] standards, the effect of corrosions on the properties of key structural elements will be analysed in Section 3.

Figure 2 : courthouse of Mbanza-Ngungu

## 2 Description of the structure of the courthouse of Mbanza-Ngungu

A description of the structure of the studied building is presented below based on surveys of the courthouse building carried out during our missions in the city of Mbanza-Ngungu (in Congo), and on old plans found in the book "Le Chemin de fer du Congo (Matadi - Stanley-Pool)" by Louis Goffin (Figure 3) [7].

Before describing in details the selected structural system, it is important to put into sight different key parameters which strongly influenced the choice of the structural solution and load-bearing elements to be used, and which are linked to the context and the constraints at the time of the construction of the building:

- the need for standardisation of this type of building: at the beginning of the 20th century, there was a need to build accommodations in Congo going from single-occupancy houses to multi-storeys building as the one under consideration in this study. To optimise the building process and to avoid building mistakes, there was a need for a certain standardisation of general structural layout and of the used structural elements for these different buildings. From the missions realised in Mbanza-Ngungu, it was possible to identified similarities both from an architectural and a structural point of view for the different buildings which are still existing in this city. From an architectural point of view, the presence of verandas as a buffer zone between the outside environment and the central living area (partitioned by masonry walls) was noted for all the buildings as well as the use of hip roofs. From a structural point of view, the same steel profiles were used for the structural elements for both single-storey and multi-storey buildings. The multi-storey buildings could be seen as enlarged and/or lengthened models of the single-storey houses.
- the search for lightness: as demonstrated below, the choice was made for construction systems exploiting the full load-bearing capacity of the structural elements to limit the weight of these elements which had to be (i) transported by sea and rail between Belgium and the Congo to the building site, and (ii) handled on site with limited equipment.
- Clever connecting solutions between steel members: the assembly of the structural members was done in two phases. The first phase consisted of riveting, in the workshop in Belgium, one side of the assembly pieces to the extremities of the beam profiles and, in a second phase, to bolt the assembly pieces to the columns on site in Congo. Such a “two phase” solution allows to avoid losing small assembly pieces during transport and to use simple assembly procedures on site which are manageable by local workers.

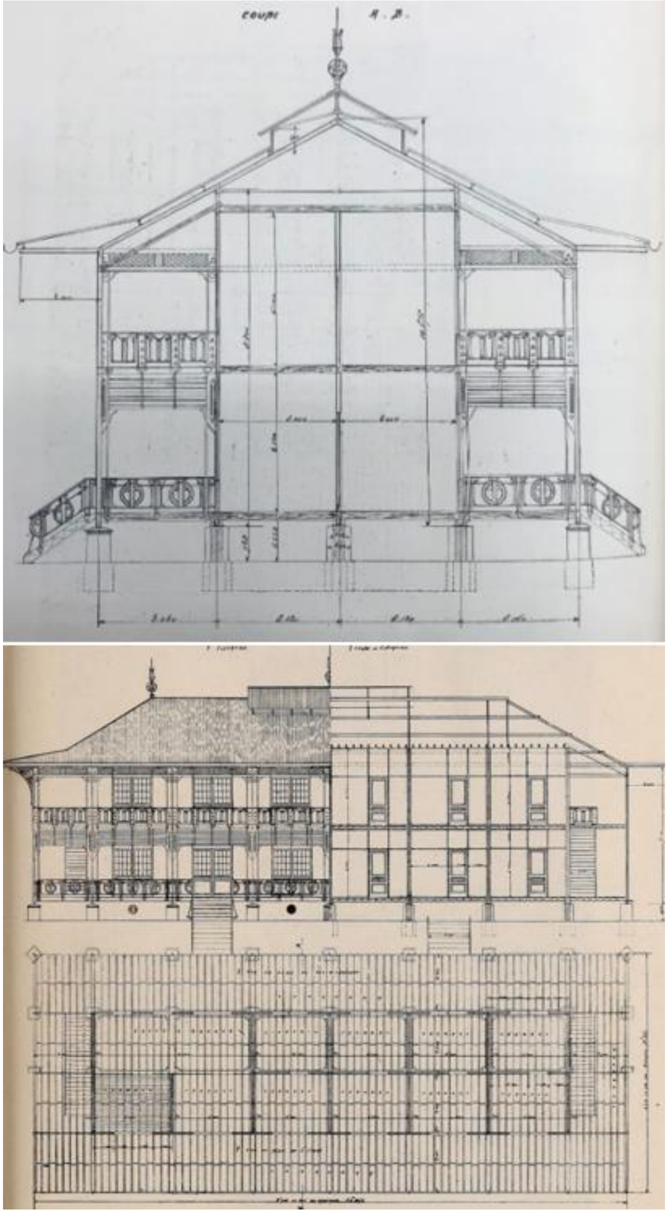

Figure 3 : elevations (transverse and longitudinal) and plan view (ground floor and upper floor) of the former ABC Hotel in Thysville (now the Palais de Justice in Mbanza-Ngungu) (source: Goffin, 1907).

Concerning now the details of the study case, the Mbanza-Ngungu courthouse is a two-storey building with a total length of 31.00m long and a total width of 12.50m. Its structure consists of five longitudinal steel frames (as numbered in Figure 10). The beam-to-column joints between the double-T structural members are bolted joints with cleats riveted to the beam web and bolted to the column flange as described above and can be assumed as pinned joints. Partition walls between the columns are placed in the central part as illustrated in Figure 3 and play the role of central core in the bracing system. This inner core is surrounded by verandas on all the perimeter of the building. The base of columns (in IPN 120 S235) is bolted to masonry piles through stiffened base plate and can be assumed as fully rigid and fully resistant; the columns are continuous throughout their height and support the roof truss members at their upper ends. The floors are made of UPN 180 S235 for the perimeter beams all around the building and of IPN 180 S235 for the internal beams only place in the longitudinal direction. In the original configuration of the building (see Figure 1), Floor are made of timber elements: 18cmx7cm rectangular timber beams placed parallel to the width of the building, bolted to the web of the longitudinal beam, and spaced about 35-40 cm apart supporting a 1-inch-thick timber plate decking. In the current configuration of the building (see Figure 2), the timber ground floor has been replaced by a reinforced concrete slab. As a result of the functional change of the building (from a hotel to an administrative building), walls were added at the level of the verandas to create new offices. However, in this article, only the initial configuration of the building will be investigated.

In the next sections, different specific constructive aspects of the building of the courthouse of Mbanza-Ngungu will be first detailed. Then, secondly, the critical analysis of the structure in its initial configuration will be presented. In particular, all the load-bearing members have been analysed and verified based on the current standards. As an example, some details of the performed analysis and verification of the most loaded purlin in IPN 120 will be present in § 2.2; this member will be then studied to account for the effect of the corrosion in Section 3.

### 2.1 Some specific constructive details

As shown here below, some structural elements of the Mbanza-Ngungu courthouse building, which at first glance could be seen as decorative or infill elements, contribute significantly to the structural stability of the investigated building. Examples include the decorative haunch used to support the rain gutter and connected to the cantilever part of the roof beams, the expanded metal frames at the head of the external frames, and the timber floor. The contributions of these elements to the stability of the structure are described below.

#### *2.1.1 Structural contribution of the decorative haunches*

Through the performed analyses, it is demonstrated that, when the presence of the decorative haunches (see Figure 4) is not accounted for, the cantilever parts of the truss members, with large spans compared to the geometry of the members (see the static scheme in Figure 5), generate excessive internal forces at the level of the end supports of these members, as illustrated through the moment diagram in Figure 5 below obtained through a first order linear analysis.


Figure 4 : decorative element used as a haunches

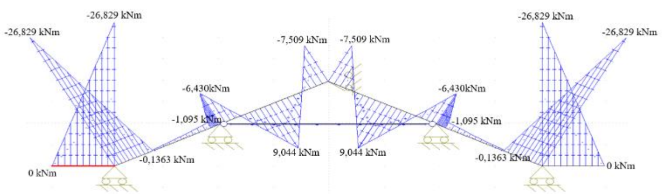


Figure 5 : bending moment ($M_{Ed}$) diagram, at ULS, in one of the beams of the courthouse roof when the presence of the decorative haunches is not considered

Considering the presence of these decorative haunches, it is found that reasonable internal forces (see Figure 6) satisfying the strength and stability criteria of EN1993-1-1 [11] are obtained which demonstrates that the structural role of these decorative haunches is crucial for the resistance and stability of these roof beams.

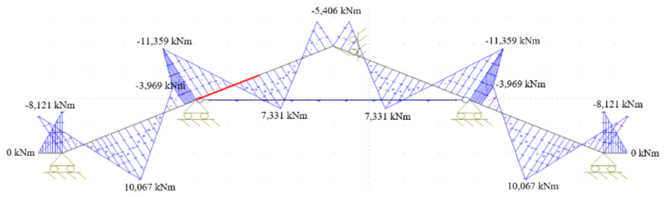


Figure 6: bending moment (MEd) diagram, at ULS, in the same beam and under the same load combination as in Figure 5 when considering the presence of decorative haunches

Through the conducted study, it has also been demonstrated that these haunches also play a key role during the construction phase of the building, i.e. when the partition walls are not yet built, as these haunches are stiffening the joints at the top of the columns (which can be then assumed as rigid joints), these joints contributing to the global lateral stability of the internal steel frames, i.e. in the frames were these haunches are met.

#### 2.1.2 *Effect provided by expanded metal frames on the external frames*

As described above, the beam-to-column joints of the frames can be assumed as hinges. According to the collected information, it also appears that, during the construction phase, these frames were not braced. Accordingly, the global stability of these unbraced frames is only ensured by the fact that the columns are encased at their bases and by the presence of the haunches which rigidify the joints at the top of the internal frames (see previous section). Performing critical analyses, it has been demonstrated that the building frames can be classified as sway frames and so, their analyses, based on the current standards, should integrate global second order effects. Considering the above-mentioned assumptions for the joints and including the second-order effects in the analyses, it was demonstrated that the external longitudinal frames were not sufficiently resistant to satisfy the ULS.

However, by analysing more deeply the beams at the top of the frames, it can be observed that expanded decorative stiff steel frames are present in these external frames (see Figure 7) with the effect of stiffening the joints at the extremities of the upper IPN 120 beams of these frames (see Figure 8). Considering the effect of these decorative frames, the analyses demonstrates that the external frames of the structure were stable during the construction phase.

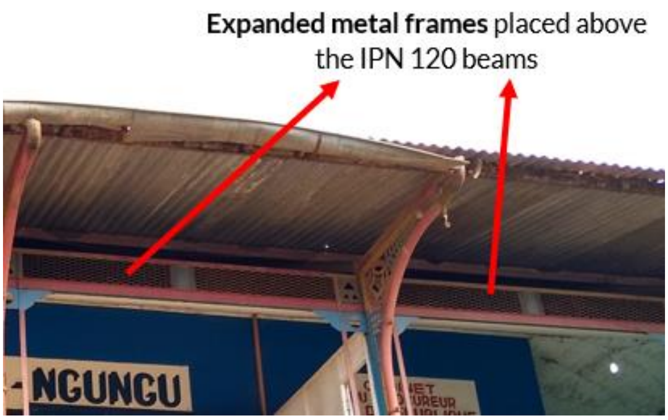


Figure 7: view of the expanded decorative metal frames connected to the IPN 120 beams and the columns of the external longitudinal frames

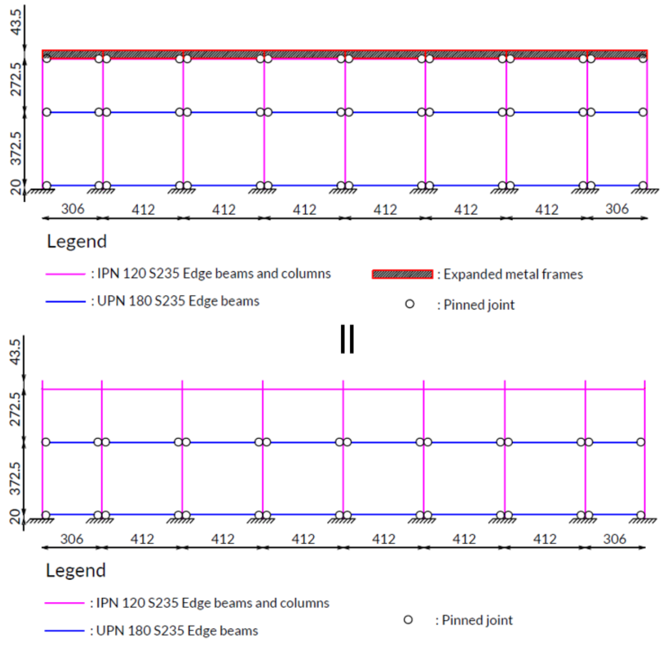


Figure 8: consideration of the presence of expanded metal frames on the static scheme of the longitudinal frames (dimensions in cm)

#### 2.1.3 *Diaphragm effect from the timber floors*

During the exploitation phase, the central part of the building made of steel frames and partition walls between the columns plays the role of bracing central core. However, to ensure the stability of the external longitudinal frames, it is required to activate a diaphragm effect at the level of the timber floor. Looking to the constructive details of these timber floors, it can be observed that the activation of this diaphragm effect was made possible by placing a dense beam grid (beams spaced between 35 and 40 cm apart) with the timber beam extremities rigidly bolted to the steel beams as described above. Without this diaphragm effect, the external longitudinal frames are not stable considering the increase of loads during the exploitation phase in comparison to the construction phase.

### 2.2 Detailed study of an IPN 120 S235 purlin

The analyses and verifications of the load-bearing elements of the structure of the courthouse of Mbanza-Ngungu were made based on recent standards (Eurocodes, NV65 rules [14]...) by considering both the construction and exploitation phases of the initial structure, i.e. when the building was used as a hotel. As stated in the literature: "the assessment of the bearing capacity of old buildings is not easy because it is based on empirical data and not on current calculation methods and models. Often it requires the development of very complex calculation models that would come as close as possible to the real situation. In most cases, it is almost impossible to meet current safety standards for such an assessment" [15]. This statement is confirmed through the investigations conducted through the present study which show that the load-bearing steel elements of the building structure satisfy the different ULS and SLS check criteria under current load combinations in a very optimised way. As an example, some results of analysis and verification performed on the most stressed purlin on 3 supports (put into sight by a circled in Figure 9) are detailed.

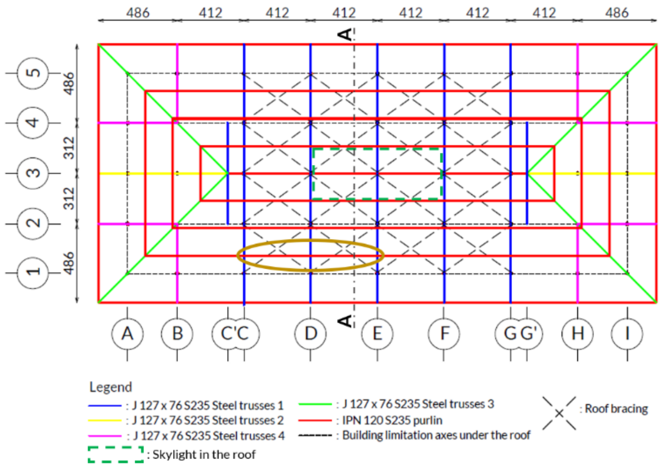


Figure 9: plan view of the roof elements of the courthouse in Mbanza-Ngungu (dimensions in cm)

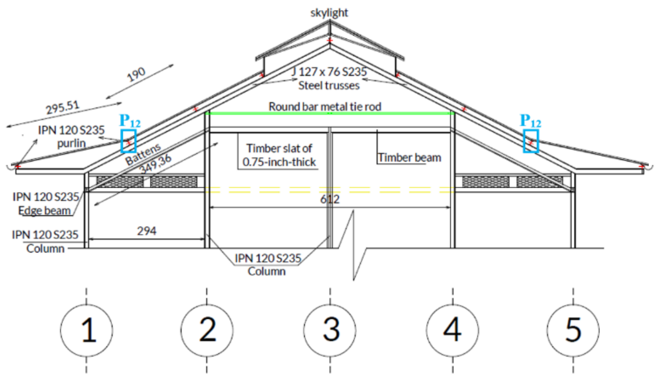


Figure 10: A-A cut (see Figure 9) of the roof of the courthouse in Mbanza-Ngungu (dimensions in cm)

### 2.2.1 Analysis of the purlin under study

As a result of the purlin loading scheme (see Figure 11 below), the worst-case combination used for the ULS verification of the purlin is the one considering (i) the sum of the dead loads ($\sum g_{i,\,Ek}$ = 0.257 kN/m) inducing forces in the two main axes of inertia of the purlin (with α = 27°- Figure 11), and (ii) the wind load ($w_{Ek1}$ = 2.715 kN/m):

$$p_{z,\,Ed} = 1.35 \sum g_{z,\,Ek} + 1.5\, w_{Ek1} = 4.38 \text{ kN/m (z-z axis)} \quad (1)$$

$$\text{and } p_{y,\,Ed} = 1.35 \sum g_{y,\,Ek} = 0.16 \text{ kN/m (y-y axis)} \quad (2)$$

with $p_{z,Ed}$ and $p_{y,Ed}$ the resultant loads according to z-z axis and y-y axis respectively considering the above-mentioned load combination.

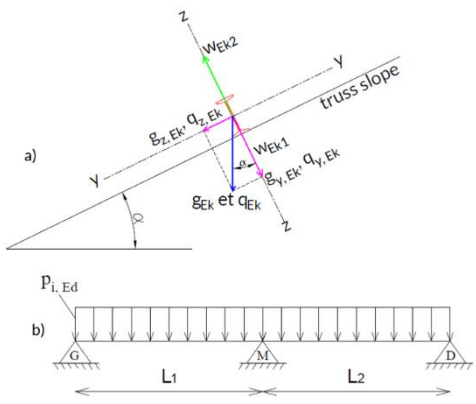


Figure 11: a) purlin loading scheme, b) purlin static scheme

Considering the static scheme of the purlin given in Figure 11 with $L_1 = L_2$ = 4.12 m, the maximum internal forces developed at the middle support (support M), as shown in Figure 12, and their values, at the ULS, are :

Along the z-z axis: $M_{y,\,Ed}$ = 9.30 kNm; $V_{z,\,Ed}$ = 22.56 kN and

Along the y-y axis: $M_{z,\,Ed}$ = 0.33 kNm; $V_{y,\,Ed}$ = 0.82 kN

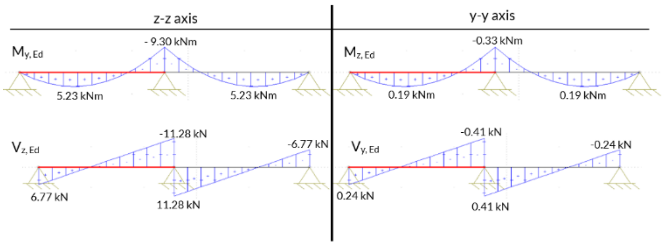


Figure 12: diagrams of bending moments and shear strength on the IPN 120 S235 purlin

### 2.2.2 *Checks on the purlin under study*

A summary of the different required checks is given in Table 1 below. These checks are made in accordance with EN1993-1-1. As can be seen, the design criterion for this purlin is linked to the lateral-torsional buckling resistant corresponding to a loading rate of 93% demonstrating the optimised character of the initial design. Such high loading rates have also been observed for all the other bearing elements of the studied structure.

Table 1: checking of the studied steel purlin

| Checks | z-z axis | y-y axis |
|---|---|---|
| **Shear strength** | $V_{Ed}$ = 22.57 kN<br>$V_{pl,\,y,\,Rd}$ = 89.95 kN<br>No M-V interaction | $V_{Ed}$ = 0.82 kN<br>$V_{pl,\,z,\,Rd}$ = 120.28 kN<br>No M-V interaction |
| **Cross-section resistance** | Symmetrical I-section profile<br>$\left[\frac{M_{y,\,Ed}}{M_{pl,\,y,\,Rd}}\right]^{\alpha} + \left[\frac{M_{z,\,Ed}}{M_{pl,\,z,\,Rd}}\right]^{\beta} < 1$<br>$\left[\frac{9{,}30}{14{,}95}\right]^{2} + \left[\frac{0{,}33}{2{,}91}\right]^{1} = 0.39 + 0.12 = 0.51 < 1$ | |
| **Lateral-torsional buckling** | The lateral-torsional buckling is satisfied with a loading rate of 93% (i.e., 0.93 < 1) | |
| **Check of the deflection** | $w_{max}$ = 6.9 mm<br>$w_{lim,\,max}$ = L/200 = 20.6 mm<br>$\Rightarrow w_{max} < w_{lim,\,max}$ | $w_{max}$ = 6.1 mm<br>$w_{lim,\,max}$ = L/200 = 20.6 mm<br>$\Rightarrow w_{max} < w_{lim,\,max}$ |

## 3 Influence of corrosion on the resistance of the IPN 120 profile

In the previous section, the resistance of the IPN 120 purlin was checked in its initial state. However, as observed during the onsite missions, the different load-bearing members are suffering from the development of significant corrosion.

Accordingly, investigations have been initiated to study the influence of this corrosion on the load-bearing capacity of the steel members of the courthouse structure. This involves the prediction of the corroded thickness, which is a function of the exposure time of the structure and the corrosion rate, which in turn is linked to various parameters characterising the environment/atmosphere in which the analysed elements are located. Among these parameters, the climatological factors (temperature, sunshine, wind speed and direction, precipitation, but above all the degree of humidity of the analysed element) or the nature and the content of aggressive agents (SO2, CO2, chlorides...) of a polluted atmosphere can be mentioned [16].

In the present section, the effects of corrosion on the evolution of the cross-section resistances and the lateral-torsional buckling vs. time are illustrated for the IPN 120 purlin treated in § 2.2. The approach consists, firstly, in determining the geometrical characteristics and the class of the steel section affected by corrosion at time t (expressed in year) and, secondly, to calculate, with these new geometrical characteristics of the cross-section, the section resistances in accordance with EN1993-1-1. The corroded thickness at time t is predicted based on formulas given in ISO 9224, for corrosivity classes defined in ISO 9223 and for the type of environment defined in EN 12500 as described here below. A life span of 150 years will be considered.

### 3.1 Determination of the total attack D of a steel part

According to ISO 9224, the total corrosion attack (D) of a metal element is determined by the following formulas:

$$D = r_{corr} \,.\, t^{b} \text{ for } t \leq 20 \text{ years} \tag{3}$$

$$D = r_{corr} \,.\, [20^{b} + b\,(20^{b-1})\,(t-20)] \text{ for } t > 20 \text{ years} \tag{4}$$

where:

- t is the exposure time expressed in years
- $r_{corr}$ is the corrosion rate during the first year expressed in micrometres per year (µm/y) according to ISO 9223.
  The studied building being in the town of Mbanza-Ngungu (south-west of the DRC), i.e. in a tropical area with a moderately polluted atmosphere (high corrosivity zone according to EN12500), the maximum corrosion rate is 80 µm/y according to ISO 9223; this value is used in the present study.
- b is the specific time exponent of the metal-environment relationship which is usually less than 1.
  As the detailed composition of the steel used for the studied building is unknown, the generalised b value used in this paper is chosen from Table 2 of ISO 9224 for carbon steel and is safely taken as equal to b = B2 = 0.575. This value is increased by 0.026 to account for uncertainties in the data to estimate a conservative upper limit of corrosion attack after prolonged exposure as required by the standard. Thus, b = 0.601.

### 3.2 Determination of the new geometrical characteristics of the corroded profile at time t and classification of the cross-section

#### *3.2.1 Dimensions and characteristics of the corroded cross-section*

Considering Figure 13, the dimensions of the corroded profile at time t will be determined by the following formulas:

$$(h', b', t'_{w}, t'_{f}) = [(h, b, t_{w}, t_{f}) - D] \text{ (mm)} \tag{5}$$

$$(r'_{1}, r'_{2}) = [(r_{1}, r_{2}) + D/2] \text{ (mm)} \tag{6}$$

The geometrical characteristics (A', $I'_{y}$, $I'_{z}$, $W'_{pl,y}$, $W'_{pl,z}$...) of the corroded profile can be then obtained considering these new properties.

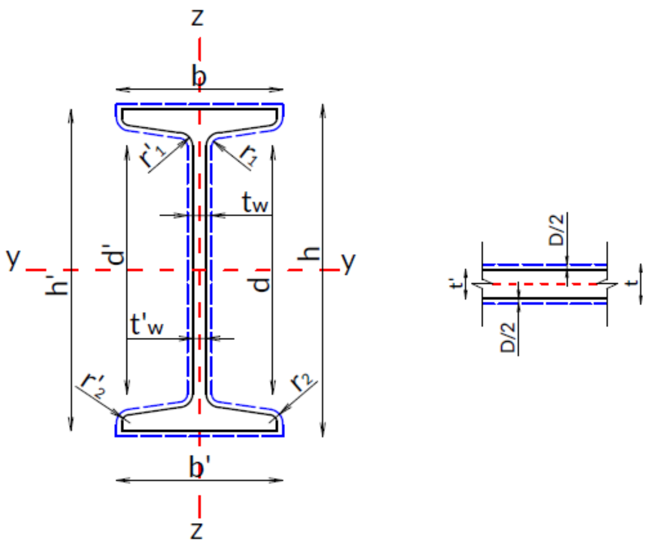


Figure 13: illustration of the evolution of the I-profile cross-section under corrosion (in blue: initial cross-section; in black: corroded cross-section)

#### *3.2.2 Classification of the cross-section of the corroded cross-section*

As for non-corroded cross-sections, the formulas for classification of cross-section as given in §5.5 of EN1993-1-1 are used considering the geometrical properties of the corroded cross-section.

### 3.3 Strength assessment of the corroded cross-section

#### *3.3.1 Cross-section resistance in bending*

Knowing the new section class of the profile, the resistant moments, along the two main axes of inertia can be determined considering the properties of the corroded cross-section, in accordance with EN1993-1-1:

$$M_{pl,\,y,\,Rd} = W'_{y} f_{y} / \gamma_{M0} \tag{7}$$

$$M_{pl,\,z,\,Rd} = W'_{z} f_{y} / \gamma_{M0} \tag{8}$$

where $W'_{y}$ and $W'_{z}$ are the bending modulus of the corroded section according the major and minor axes respectively:

- $W'_{y} = W'_{pl,y}$ and $W'_{z} = W'_{pl,z}$ for Class 1 and 2 sections ;
- $W'_{y} = W'_{el,y}$ and $W'_{z} = W'_{el,z}$ for Class 3 sections ;
- $W'_{y} = W'_{eff,y}$ and $W'_{z} = W'_{eff,z}$ for Class 4 sections.

The verification of the cross-section strength is done in the same way as in Table 1. The evolution of the loading rate vs. time is provided in Figure 14. It can be observed that, as expected, the loading rate is increasing with time due to the decrease of the bending resistance of the corroded cross-section. The failure of the corroded cross-section under bending is reached for a lifetime of around 135 years considering the adopted assumptions.

The resistance of the corroded cross-section under shear forces has also been checked. The failure of the corroded cross-section under shear is not reached for the considered life span of 150 years; also, no M-V interaction needs to be accounted for.

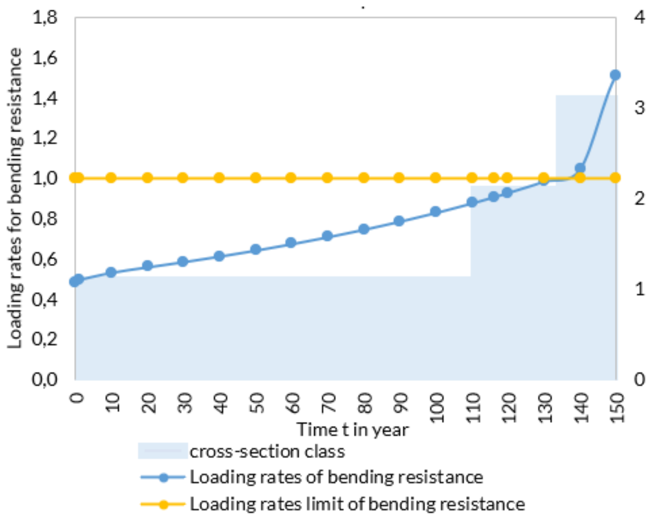


Figure 14: Evolution of the cross-section bending resistance of the corroded IPN 120 profile vs. time

#### *3.3.2 Member resistance - lateral-torsional buckling*

The check of the lateral-torsional buckling of the corroded IPN 120 purlin was carried out in accordance with formulas of §6.3 of EN 1993-1-1 [11] and the results are presented in Figure 15.

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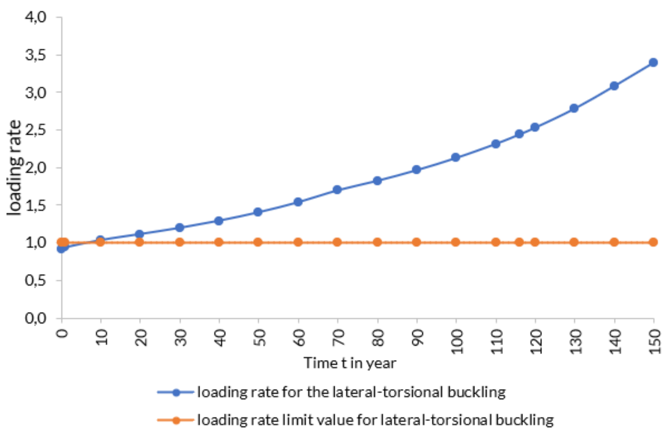


Figure 15 : Evolution of the lateral-torsional buckling resistance of the corroded IPN 120 purlin vs. time

By analysing the results presented in Figure 15, it can be observed that the resistance to lateral-torsional buckling is no longer satisfied after the sixth year since the loading rate exceeds 100% for the considered exposure conditions as descripted in §3.1. However, as mentioned in §3.1, some safe assumptions have been considered to evaluate the total corrosion attack coefficient and, so, the latter can be seen as overestimated. Accordingly, the prediction presented here can be seen as a safe estimation of the actual behaviour of the corroded members. However, as illustrated in Figure 16, lateral-torsional buckling is observed for some of the purlins of the studied structures which confirms the sensitivity of these members to corrosion effects.

Figure 16 : view of some deteriorated roof edge purlins of the courthouse building of Mbanza-Ngungu.

## 4 Conclusions

Within this paper, the preliminary studies conducted on the structure of the building of the courthouse of Mbanza-Ngungu have been summarised. The initial design of this structure has been first evaluated considering the present design norms. It has been demonstrated that this structure in its initial state satisfies all the limit states in terms of deformation, resistance, and stability. Then, the influence of corrosion has been accounted for to estimate the effect of the latter on the resistance of key structural members. Through the results presented herein, it has been demonstrated that the purlin is sensitive to the corrosion effects and, in particular, should exhibit a limited lifespan as confirmed by the observations made during the mission conducted in Mbanza-Ngungu.

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