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## ► To cite this version:

Hannah Franz, Mario Rinke, Emilie Lepretre, Lamine Dieng. The gap between theory, practice and regulations in design criteria for iron and steel structures in 19th century France: the example of train sheds. *Timber and Construction: Proceedings of the Ninth Conference of the Construction History Society*, Construction History Society, Apr 2022, Cambridge, United Kingdom. pp.287-300. hal-03767415

**HAL Id: hal-03767415**

**<https://hal.science/hal-03767415v1>**

Submitted on 1 Sep 2022

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# **The gap between theory, practice and regulations in design criteria for iron and steel structures in 19th century France: the example of train sheds**

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## **Abstract**

In the 19th century, the concept for designing metallic structures was to compare the stresses resulting from the loads with a working stress, a fraction of the ultimate strength of the material. The choice of the working stress depended mainly on the experience and theoretical knowledge of designers as early regulations left a lot of discretion to engineers. This paper traces the evolution of French design criteria used in practice for elements made of wrought iron or mild steel, working in tension or compression, based on an extensive survey of the French literature of the 19th century and the beginning of the 20th century. These design criteria are compared with original design reports of train sheds, which show that designers used working stresses higher than recommended in the literature and ensured buckling safety through constructive measures instead of calculations.

## **Introduction**

The basic principle of structural design is to compare the internal forces of a structure with the capacity of the material constituting it. In the 19<sup>th</sup> century, the increasing use of cast and wrought iron, later of mild steel, as construction materials, went along with the development of the theory of structures. Kurrer's *History of the Theory of Structures* detailed the evolution of structural analysis methods, which allowed the calculation of internal stresses resulting from external loads [1]. As for the material's capacity, Timoshenko's *History of Strength of Materials* gave a historical overview of experimental studies on mechanical properties of materials, such as the ultimate or the yield strength, and of the theories deriving allowable stresses from those experiments [2]. In practice, safety factors were introduced to consider unknowns such as calculation errors, material inhomogeneities, imperfections related to

manufacturing and construction, dimensional errors, etc. Schueremans et al. recently summarized design practices for iron and steel structures in 19<sup>th</sup> century Western Europe, mostly based on official regulations [3]. However, the choice of the allowable stress, the so-called *working stress*, depended a lot on the experience and theoretical knowledge of the designer and the ‘risk-willingness’ of the client. Early regulations gave little guidance [4]. The discrepancies between design criteria recommended in the literature or regulations and the ones effectively used in practice remain widely unexplored.

This paper aims to trace the evolution of French design criteria for wrought iron and mild steel based on an extensive survey of the French literature of the 19th century and the beginning of the 20th century. Beyond regulations, the survey draws on construction treatises, civil engineering periodicals and reports from the French Society of civil engineers. Some case studies of the literature highlighted the working stresses used for single constructions, such as the Garabit viaduct in France [5] or a dome in the Vienna Hofburg [6]. To illustrate more representatively the working stresses used in practice, this paper develops insight in the design criteria used for a large family of structures, namely train sheds (see Fig. 1).



Figure 1: Example of an existing historical train shed: Gare de Perpignan (built in 1896) before renovation of 2013. Photo: SNCF-AREP. D. Boy de la Tour.

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About 175 train sheds were constructed in France before 1950, 40% of which are still in service [7]. Train sheds are particularly relevant to study the gap between theory and practice, as regulations only came more than half a century after the first metallic train shed was built for the Gare St-Lazare in Paris in 1843 [8]. The first French regulation on metallic structures, published in 1869, concerned only bridges while train sheds were first covered in 1902 [9]. Archive materials on train sheds are centrally located at the National Center for Historical Archives of the French railway company SNCF in Le Mans, France [10]. Original design reports of extant train sheds have been gathered and design criteria used for the calculations compared with commonly recommended working stresses at the time of their construction.

## **Working stresses for elements in tension**

### *Literature review*

From the first half of the 19th century, the limit of 6 kg/mm<sup>2</sup> became the default working stress for iron, for elements working in tension. This value was first recommended by Navier in his *Résumé des Leçons* published in 1826 [11]. Poncelet, in his *Mécanique industrielle* published in 1829, set the same limit of 6 kg/mm<sup>2</sup>, followed by Morin in his *Leçons de mécanique pratique* published in 1853 [12]. According to Vierendeel in his monograph on iron and steel construction published later in 1902, it was thanks to Poncelet and Morin that the limit of 6 kg/mm<sup>2</sup> was adopted by the French administration and became widely used in practice [13].

Authors often defined the working stress as a fraction of the ultimate strength or yield strength of the material. The ultimate strength remained for a long time the reference to define the working stress, mostly because the ultimate strength was easier to measure than the yield strength [14]. In 1826, Navier defined the working stress based on an average ultimate strength of 40 kg/mm<sup>2</sup> obtained from a comprehensive review of tensile tests conducted by other scientists. His safety factor of about 6 leading to a working stress of 6-7 kg/mm<sup>2</sup> became standard thereafter [15].

Recommended working stresses depended on the load combination used to calculate the stresses in the structure. Dead loads result from the weight of the construction, live loads correspond to the weight of trains, vehicles or pedestrians and climatic loads include snow and wind. In today's Eurocodes [16], dead loads are called "permanent actions" while live loads

and climatic loads are grouped under the name of “variable actions”. “Accidental actions” refer to explosions or impacts. In the literature of the 19<sup>th</sup> century, the words “permanent”, “variable” and “accidental” were used in a much less straightforward manner. Holzer also showed that the refinement of load assumptions varied depending on their nature [17]. Several authors defined the working stress as a limit for “permanent” stresses, such as Bresse in 1859, but then some applied this working stress to examples including climatic loads, such as Ardant in 1840, or accidental loads, such as Collignon in 1869 and Brune in 1888 [18]. Navier indicated a limit of 6 to 7 kg/mm<sup>2</sup> for dead loads and 8 to 10 kg/mm<sup>2</sup> for stresses resulting from dead and “accidental” loads, without detailing their nature. The limit was set higher for combinations including several types of loads because they generated a lower level of uncertainty for the maximum calculated stress.

Other references were more explicit regarding load assumptions. The working stress of 6 kg/mm<sup>2</sup>, recommended by the first French regulation on metallic road bridges published in 1869 and its revision published in 1877, applied to stresses resulting from dead loads and live loads [19]. In the first edition of his works *Stability of Constructions*, published in 1886, Flamand mentioned a formula for the working stress proposed by Séjourné:

$$\sigma_{\text{lim}} = \frac{6}{1 - 0,4 \frac{\sigma_{\text{min}}}{\sigma_{\text{max}}}} \text{ kg/mm}^2$$

$\sigma_{\text{min}}$  was the minimum stress and  $\sigma_{\text{max}}$  the maximum stress in the element considered, depending on load combinations [20]. Vierendeel calculated that with this formula the working stress varied between 6 kg/mm<sup>2</sup> if the dead load was close to zero and 10 kg/mm<sup>2</sup> for permanent loads only. In the case of “light-weight roof structures, for which the dead load is low compared to accidental loads related to wind, snow or people”, the working stress would lie between 6 and 7 kg/mm<sup>2</sup> [21]. The regulation for metallic bridges released in 1891 also gave formulae calculating the working stress depending on the ratio between live and dead loads, leading to values between 6 and 9 kg/mm<sup>2</sup> [22]. The first regulation for train sheds, released in 1902, recommended a working stress for iron of 8 kg/mm<sup>2</sup>, corresponding to load combinations including dead load, snow and wind [23]. The next revision of the regulation on metallic bridges, published in 1915, introduced a wider range of load combinations including live loads, snow, wind and temperature [24].

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The working stress of 6 kg/mm<sup>2</sup> for iron was a default value. This limit could be increased according to the type of structure, the quality of the material or the experience of the contractor. Poncelet stated that “contractors specialised in the construction of iron suspension bridges, guided by a long experience and sure of a consistent manufacturing” may use a higher stress limit. Regulations also explicitly allowed engineers to choose higher working stresses according to their individual judgement. Brune stated that “the safety load commonly accepted for iron is of 6 kg/mm<sup>2</sup> for public constructions, 8 kg/mm<sup>2</sup> for private constructions, and 10 kg/mm<sup>2</sup> for industrial constructions”. Ardant indicated working stresses between 6 and 12 kg/mm<sup>2</sup> “depending on the quality” of the material. Morin allowed a working stress of 8 kg/mm<sup>2</sup> for roof trusses, arguing that the rigidity of connections and the friction of girders on their supports, which were not considered for determining the stresses, played in favour of stability. Moreover, Morin stated that roof trusses were made of “selected materials”. The same idea was presented in the “General study on iron roof structures” of the periodical *Nouvelles Annales de la Construction* in 1863, thus recommending working stresses between 8 and 10 kg/mm<sup>2</sup> [25].

In the 1880s, authors began to propose working stresses for steel. The recommended values in the theoretical literature varied more than those for iron, which may be due to fluctuations in the carburization level of the steel. Flamand suggested applying Séjourné’s formula to steel, with a 50% increase compared to iron. The International Congress for construction processes, gathered in Paris during the World Exhibition of 1889, favoured an increase of 40%. Brune recommended working stresses between 6 and 10 kg/mm<sup>2</sup> for iron and 15 and 20 kg/mm<sup>2</sup> for steel. Résal, in his construction treaty published in 1892, gave working stresses between 5 and 8 kg/mm<sup>2</sup> for iron and between 7 and 9 kg/mm<sup>2</sup> for steel. The formulae in the bridge regulation of 1891 led to working stresses between 8 and 12 kg/mm<sup>2</sup> for steel. The train shed regulation of 1902 prescribed 10 kg/mm<sup>2</sup> [26]. In his lecture on material strength published in 1900, Novat stayed more conservative with recommended working stresses between 7 and 10 kg/mm<sup>2</sup> [27]. Flamand, in the third edition of his works *Stability of constructions* in 1909, cited both Séjourné’s formula and the regulation of 1891, thus clearly showing that engineers remained free in their choices [28]. In his lecture on metallic bridges published in 1917, Résal stated that in practice a working stress of 12 kg/mm<sup>2</sup> for steel was commonly used. The bridge regulation of 1915 prescribed that stresses resulting from dead and live loads should not exceed 12 kg/mm<sup>2</sup> while stresses including also the effect of wind should stay below 12,5 kg/mm<sup>2</sup> [29].

Table 1 summarizes the sources discussed above and indicates in which engineering school the authors were teaching, to enhance the potential impact of their knowledge in practice.

Table 1: List of sources used to establish the evolution of working stresses of iron and steel between 1820 and 1930.

Publication year	Author	Engineering school	$\sigma_{im}$ (iron) [kg/mm <sup>2</sup> ]	$\sigma_{im}$ (steel) [kg/mm <sup>2</sup> ]
1826	NAVIER C-L	Ecole des Ponts et Chaussées	6-10	-
1840	ARDANT P.-J.	Ecole d'Application de l'Artillerie et du Génie (Metz)	6-12	-
1841	PONCELET V.-J.	Ecole d'Application de l'Artillerie et du Génie (Metz)	6	-
1853	MORIN A.	Conservatoire des Arts et Métiers	6-8	-
1857	BRESSE J.	Ecole des Ponts et Chaussées	6	-
1863	MATHIEU E.	-	8-10	-
1869	COLLIGNON E.	Ecole des Ponts et Chaussées	5-6	-
1869	Cirulaire du 15 juin 1869 relative aux épreuves à faire subir aux ponts métalliques destinés aux voies de terre		6	-
1877	Cirulaire du 9 juillet 1877 relative aux épreuves des ponts métalliques		6	-
1886	FLAMANT A.	Ecole Centrale des Arts et Manufactures, Ecole des Ponts et Chaussées	6-7	9-10.5
1888	BRUNE E.	Ecole des Beaux-Arts	6-10	15-20
1891	Cirulaire du 29 août 1891 relative aux épreuves des ponts métalliques		6-9	8-12
1892	RESAL J.	Ecole des Ponts et Chaussées	5-8	7-9
1900	NOVAT J.	Société d'enseignement professionnel du Rhone	6-8	7-10
1902	Règlement du 25 janvier 1902 sur les halles à voyageurs et à marchandises des chemins de fer		8	10
1909	FLAMANT A.	Ecole Centrale des Arts et Manufactures, Ecole des Ponts et Chaussées	6-9	8-12
1915	Cirulaire du 8 janvier 1915 relative aux épreuves des ponts métalliques		-	12-13
1928	ARAGON E.	-	6-8	8-12

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**Data from design reports of train sheds**

To complement the values from the theoretical debate with those used in practice, the working stresses used for the design of nine train sheds built between 1852 and 1931 were collected, either from original design reports preserved in the archives of SNCF or from articles of the periodical *Annales des Ponts et Chaussées* (APC) [30]. This journal regularly published short technical and economic data as well as technical drawings regarding major structures such as train sheds. Table 2 presents the collected values of the working stress, or of the maximum calculated stress (\*), when the working stress was not explicitly indicated.

Table 2: List of train sheds for which working stresses were collected.

Date of construction	Train shed	Code	Contractor	Source	$\sigma_{lim}$ (iron) [kg/mm <sup>2</sup> ]	$\sigma_{lim}$ (steel) [kg/mm <sup>2</sup> ]
1852	Paris St-Lazare (halle Flachat)	PSL	Joly	APC, 1855	7.6*	-
1868	Bayonne	BYN	Rigolet	Arch. SNCF, 1868	9.53*	-
1869	Paris Austerlitz	PAZ	Schneider	APC, 1871	6	-
1879	Hendaye	HND	A. Moisant	Arch. SNCF, 1880	6	-
1892	Marseille St Charles	MSC	Sénès et Arnal	Arch. SNCF, 1892	-	14.38*
1898	Bordeaux St Jean	BSJ	Daydé et Pillé	Arch. SNCF, 1896	-	13
1902	Bédarieux	BED	Daydé et Pillé	Arch. SNCF, 1902	-	11
1908	St Germain des Fossés	SGF	<i>unknown</i>	Arch. SNCF, 1908	-	12.62*
1931	Paris Gare de l'Est	PGE	Schmid, Bruneton, Morin	Arch. SNCF, 1929	-	13

**Comparison between literature and design reports of train sheds**

Figure 2 graphically brings together the working stresses obtained from the literature and the values used for train shed designs. The bars show the lower and upper limit for the working stress proposed by various authors, while the dots indicate the working stresses used for specific train sheds. The 3-letter codes are referring to Table 2. These charts highlight that the default working stress for iron stayed quite constant throughout its period of use, while the upper limit fluctuated, and that the working stress recommended for steel varied much more than for iron.



They also reveal that the working stress values used for train sheds were mostly above the upper limit recommended in the literature.

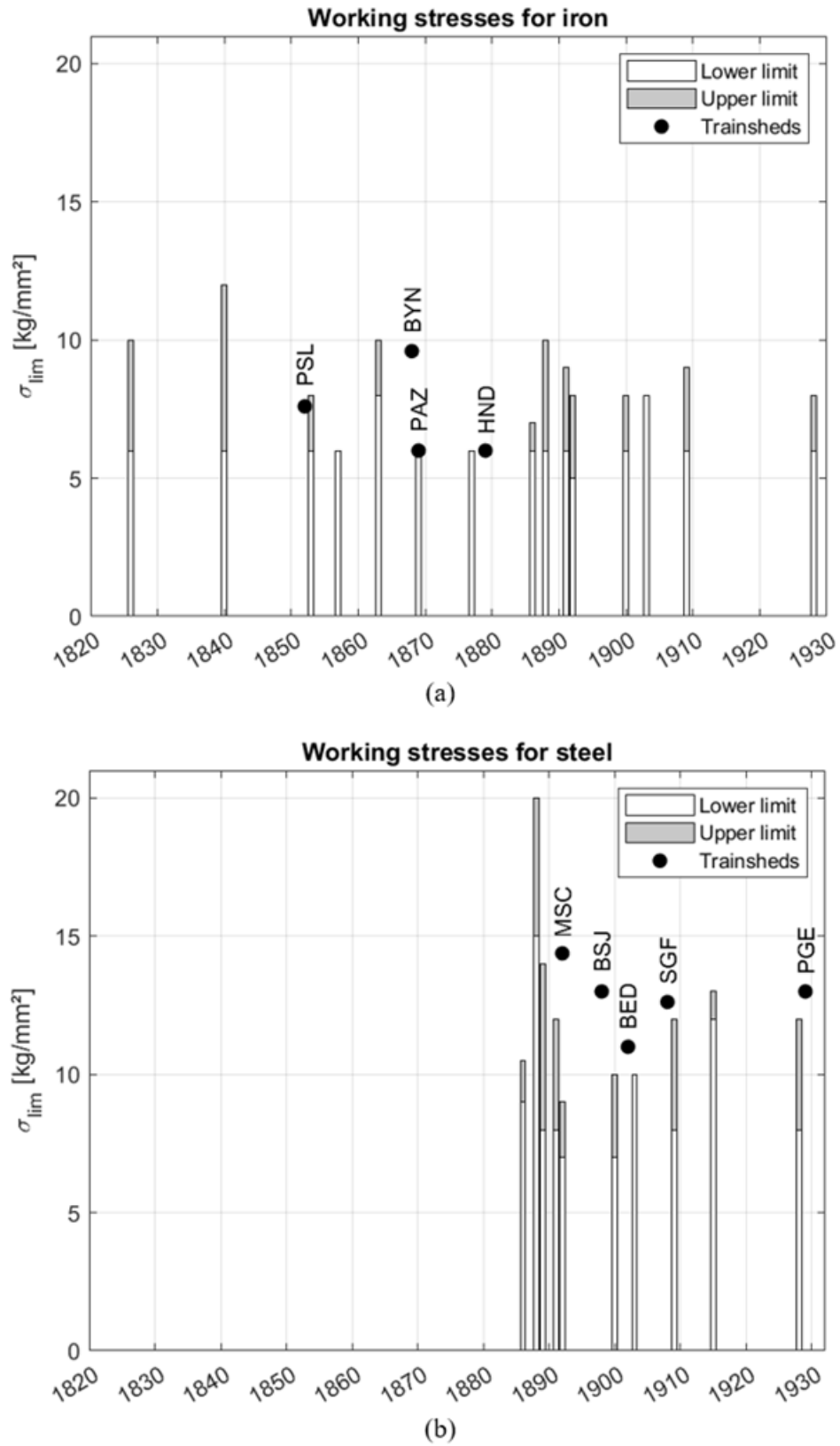


Figure 2: Evolution of the working stresses recommended in the literature for (a) iron and (b) steel between 1820 and 1930, compared with working stresses used for train sheds.

## **Working stresses for elements in compression**

Unlike cast iron, the yield strength for wrought iron and mild steel was the same in compression and tension. However, elements working in compression were subjected to buckling and therefore necessitated a different approach to determine working stresses.

### *Evolution of the state of knowledge on buckling*

The first studies on buckling of structural elements in compression date back to the 18th century. Around 1730, Van Musschenbroeck carried out experiments from which he concluded that the resistance of a column did not only depend on the material, but also on its geometry. In 1744, Euler developed a formulation of the critical load that a column could admit without bending [31]:

$$\sigma_{cr} = \frac{\pi^2 E}{\left(\frac{L}{r}\right)^2}$$

With  $\sigma_{cr}$  the critical stress,  $E$  the Young modulus,  $L$  the buckling length and  $r$  the radius of gyration, depending on the moment of inertia and the section of the bar. The ratio  $L/r$  defines the slenderness  $\lambda$  of a bar.

However, in further experiments, such as Hodgkinson's in 1840, instabilities occurred for compressive loads much lower than the critical load calculated by Euler. The idealized assumptions behind Euler's theory were unravelled progressively. Firstly, Euler assumed that bars were perfectly straight, with a load applied in a perfectly centred manner. The influence of geometrical imperfections (eccentricity of the load or initial curvature) was described at the beginning of the 19th century, for example by Young in 1807 and Navier in 1826. Secondly, Euler's formula necessitated bars to remain elastic. Navier in 1826 defined ranges of slenderness for which Euler's formula was valid but Lamarle was the first in 1845 to link this validity domain to the yield strength. The first theories of inelastic buckling were proposed by Engesser and Considère in 1889. Finally, Euler's theory was valid only for a homogeneous material. The fluctuation of the strength within the section as well as the residual stresses were accounted for from the 1950s onwards [32].

As an alternative to Euler's formula, many authors strived to propose an estimate of the critical stress that would be more in line with experimental data. Instead of defining the equilibrium like Euler, they conceptualised the maximum axial stress of a compressed bar as a combination of compression and bending. The bending stress was derived by assuming geometrical imperfections or by assimilating it to a stress due to the critical load. The resulting formula yielded:

$$\sigma_{\text{lim}} = \frac{a}{1 + b\lambda^2}$$

Where  $a$  and  $b$  were constants, whose values could vary depending on ranges of slenderness. The first author to give such a formula was Tredgold in 1822. In the 1850s, several authors such as Schwarz, Gordon and Rankine proposed different approaches leading to a similar result. This formula was convenient to use and became very popular in the following decades, known mostly as Rankine's formula [33].

Euler's formula was rehabilitated in the 1880s, thanks to the experiments of Bauschinger and Tetmajer in Germany and Considère in France. They proved Euler's theory right for very slender elements when the experimental setup reflected theoretical boundary conditions. Tetmajer proposed to give up the unification of all results in a single formula as Rankine's formula tried to do. For intermediate slenderness values, Tetmajer described the maximum stress as a function of slenderness using a straight line, which became widely used in German-speaking countries [34].

### ***Design criteria for buckling in France***

As theoretical and empirical approaches regarding buckling were very diverse, the buckling design criteria recommended in construction treatises varied accordingly. In France, the first reference on which we can assume designers relied on for their design of iron columns is Navier's *Résumé des Leçons* from 1826. For large slenderness values, Navier recommended using Euler's formula with a safety factor of 4 to 5. For smaller slenderness values, he gave isolated experimental stresses [35]. In 1851, Love submitted a memorandum to the French Society of Civil engineers in which he presented the experiments of Tredgold and Hodgkinson. Based on those experiments, he proposed a Rankine-type formula, which, according to Buchetti in 1888, became prevalent in France until the end of the 19th century. Love did not recommend any safety factor but Buchetti indicated that Love's formula was to be reduced by a safety factor

of 6 [36]. The first bridge regulations from 1869 and 1877 did not mention buckling. They recommended a working stress of 6 kg/mm<sup>2</sup> for iron, both in tension and in compression. At the end of the 1880s, several authors such as Flamand and Résal proposed Rankine-type formulas with different constants. Brune preferred the use of Euler’s formula with a reduced modulus of elasticity  $E = 80$  GPa instead of 210 GPa [37]. The 1891 bridge regulation was the first to require that “elements in compression be not exposed to buckling” but it did not indicate any method on how to ensure it. The same line was adopted by the 1902 train shed regulation. In 1915, the new regulation for bridges finally favoured a Rankine-type formula but it stayed quite vague, as it left engineers free to decide which constants to use [38]. Fig. 3 represents the buckling criteria proposed by the authors cited above.

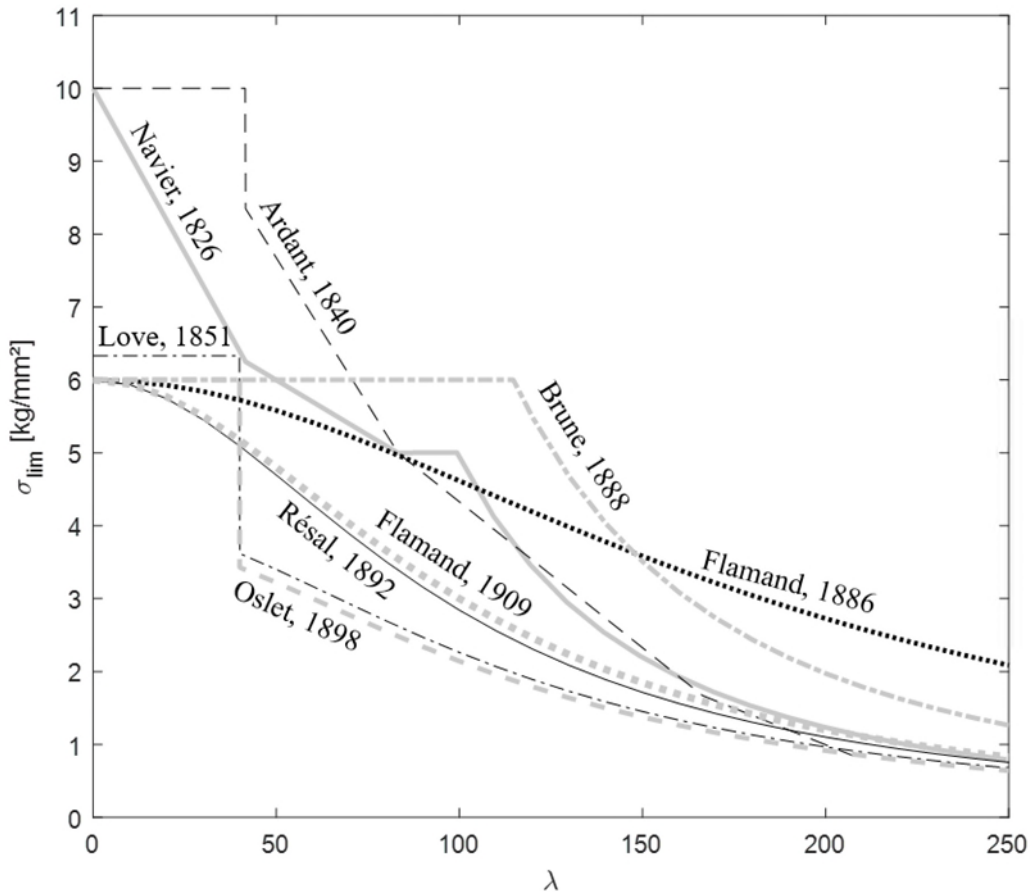


Figure 3: working stresses in compression as a function of slenderness  $\lambda$  in French literature

### ***Buckling safety in practice***

As pointed out by Gargiani in 2008, it was the search towards designing the “perfect column” that motivated engineers and physicists in the 18th and 19th centuries to progress in modelling

the buckling problem. The acquired knowledge thus mostly benefited the design of structural columns. This was also reflected in the vocabulary used by authors dealing with buckling: in construction treatises, chapters dealing with buckling were usually entitled “Pièces chargées debout”, literally “Standing members under loads”. The risk of buckling for other elements working in compression, such as truss bars in bridges or roof structures, was neglected. Oslet, for instance, in his *Traité de charpente en fer* from 1898, referred to Love’s formula for the design of columns only [39].

Several design reports of train sheds confirm this tendency. In a design report for the Gare de Marseille St-Charles from 1892, working stresses for Polonceau rafters in compression reached the extraordinarily high value of 14 kg/mm<sup>2</sup> while buckling was not discussed at all. In a design report from 1880 for the Gare d’Hendaye, also featuring Polonceau roof trusses, it was acknowledged that rafters could be subjected to buckling but their buckling length was reduced by clamping through the connected purlins. The slenderness of rafters was then low enough to disregard buckling, following Love’s recommendations of 1851. In the design report for the Gare de Bordeaux St-Jean drafted by Daydé et Pillé in 1896, buckling was again not explicitly verified. For certain compression elements, though, the contractor pointed out that the maximum stress had been kept “voluntarily low”.

For diagonal members in trusses, some authors argued that constructive measures were enough to ensure buckling safety. Collignon and Brune declared that truss diagonals were not exposed to buckling thanks to their clamped riveted connections. For elements in compression, other than columns, Novat recommended cross-sections shaped like an X, a T, a U or an L to avoid buckling [40].

Trussed beams used in train sheds for purlins or rafters also provide examples of constructive solutions. In some train sheds, one can observe differentiated cross-sections between truss bars in compression and in tension. For example, in the ridge purlin truss of the Gare d’Etampes, the diagonals in tension feature flat sections while the vertical bars in compression have a double L section (Fig. 4). As a common strategy to resist buckling, bars in compression were given a larger inertia than the bars in tension.

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Figure 4: Trussed ridge purlin at the Gare d'Etampes (built in 1879). Photo: SNCF-AREP.

In trussed beams with diagonals crossing each other (Fig. 5 and 6), diagonals pointing upwards from the supports towards the centre of the beam usually work in compression while the diagonals pointing downwards work in tension. Compression diagonals therefore have a bigger cross-section. In the lower purlins of the Hall St-Etienne of the Gare de Lyon-Perrache, the compression diagonals close to the supports are reinforced by additional riveted plates (Fig. 5). In the purlins of the Gare de Montauban, compression diagonals consist of a double layer of flat plates while the tension diagonals only feature single flat plates (Fig. 6). Moreover, the intersection with tension diagonals provides another local stabilization measure.



Figure 5: Trussed lower purlin at the Gare de Lyon-Perrache (halle St-Etienne, built in 1885).  
Photo: H. Franz.

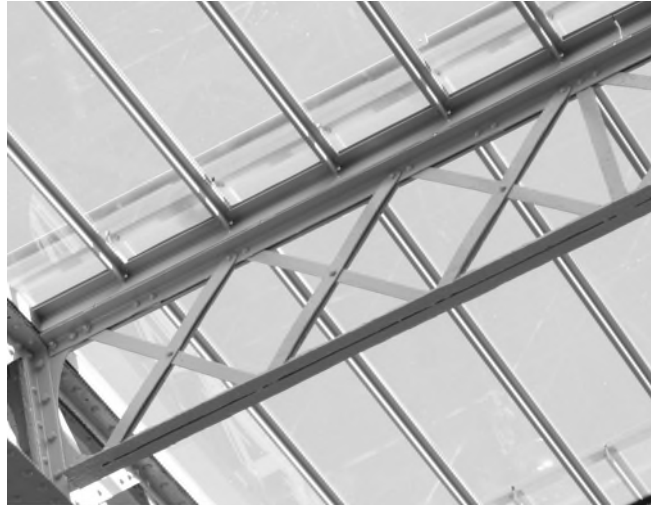


Figure 6: Trussed upper purlin at the Gare de Montauban (built in 1903). Photo: SNCF-AREP.

## **Conclusion**

This paper traces the evolution of design criteria for wrought iron and mild steel structures in 19<sup>th</sup> century France and gives insight into the differences between literature, regulations, and practice. Authors of construction treatises or technical papers, who were often also professors in renowned engineering schools and practising engineers, recommended working stresses based either on experiments or on their practical knowledge. For elements in tension, the limits varied among authors, according to the load assumptions, the type of structure, or the quality of the material, and regulations followed the global trend. These approaches were compared with the design criteria used in practice for nine train sheds. Overall, designers chose working stresses higher than recommended. For elements in compression, design criteria were even more diverse in the literature, due to the theoretical struggle with the phenomenon of buckling, and regulations withdrew from the debate. Facing this disarray, designers ensured buckling safety with qualitative arguments and constructive measures, as it has been shown through examples of train sheds.

The presented debate among engineers did not only question the limits and strategies to be used for the design of structures but also the role of the designer within this process. What is surprising is that, while most narratives of the history of structures see an increasing influence of mechanical models and design procedures throughout the 19<sup>th</sup> century, the design practice still relied on an engineer as an experienced individual making deliberate design decisions. This apparent duality of a growing science and determinism on one side and an explorative practice and individual engineering agency on the other is a fascinating aspect of that period. This paper

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provides a basis for following research steps on French iron roofs designed and constructed in the 19<sup>th</sup> century.

## **Acknowledgments**

This research project is funded by AREP, subsidiary of SNCF, and supported by the society Rails & Histoire.

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