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Geometry, strength, and efficiency: Tracing the standardization of North American structural steel, 1888-present

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Abstract: Across North America, the industrialization of steel fabrication in the late 19th century, specifically that of hot-rolled sections, enabled new scales, typologies, and economies of building design and construction. However, steel design during this early period was turbulent, as individual manufacturers provided their own catalogues of available geometries and assumed material strengths, leading to mill-specific schools of design and analysis. During the proceeding century, improvements in manufacturing technology, the merging of dominant fabricators, and advancements in engineering knowledge led to the creation of unified standards of geometry, material strength, and design methodology. In this paper, we follow the progression of steel design, from mill-specific, stress-based analysis to the highly standardized, probabilistic design methodology of the 21st century, and evaluate the impact on nominal strength and material efficiency. With a focus on doubly symmetric flanged sections (I-beams) used for flexural elements, we show the lineage of available geometry as it progressed from wrought iron rails to contemporary steel W-sections. Alongside this geometric evolution, we follow the developments in engineering methodology and its impact on the nominal strength beams. Through a computational analysis of over 2000 unique sections produced since 1888, as well as seven design methodologies from 1907 to 2022, we show that both the available distribution of section geometries and the development of engineering knowledge have had significant impacts on the assumed capacities and the material efficiency of bending-dominated steel elements since the late 1800s.

Introduction

Structural steel plays an important role in modern construction due to its unique combination of strength, mechanical consistency, and manufacturing ease. It is orders of magnitude stronger, stiffer, and more ductile than wood or concrete, and can be prefabricated into useful geometries for purchase, modification, and assembly. In the United States, these factors have been iterated and continuously improved upon for over a century to arrive at a highly standardized and economically efficient catalogue of readily accessible standard steel sections.

Arguably the most iconic of these sections, and the de facto visual representation of steel construction, is the I-beam. Known today by many names—H, S, W, or WF sections—the double-symmetric, I-shaped section concentrates mass at two parallel flanges, vastly improving bending strength and stiffness compared to equivalent geometries of the same area and mass. Beginning in the 1800s, American steel mills pushed the bounds of manufacturing capacity and engineering mechanics to develop competing ranges of lighter, stronger, and more efficient I-beams. Until the 1920s, the result was an era of inconsistent design and analysis, where the selection of mill dictated the available sections and their nominal capacities.

Today, the design of steel beams is highly standardized, both in the section geometries available to designers, as well as the method of characterizing strength and behavior. For engineers in the United States, a total of 283 W-sections are available to size against arbitrary structural demands. These sections are grouped by depth, nominally at integer inch increments, and ordered by linear weight; within a given depth, the range in linear weight typically varies by an order of magnitude. What is less clear is the origin of these geometries. Why these depths and weights? When were these decisions made, and has the context of structural design changed significantly since?

This paper seeks to clarify both the progression towards standardized geometry as well as the developments in engineering knowledge from the beginnings of hot-rolled sections to the contemporary context of structural steel design. First, we provide an historic overview of the parallel developments in both industrial steel fabrication and beam analysis methodology to identify key changes in both physical geometry and calculated capacity. Second, to evaluate the material impacts of these changes on the sizing and mass of steel structures, we perform a computational analysis on period-specific nominal capacities and material efficiency over a span of over 130 years.

1. Historical overview

The success of steel and the I-beam are both a testament to 19th and 20th century innovation. The emergence of steel and the industrial age in the US has been extensively studied (Hessen 1972; Misa 1999; Friedman 2010; Sutherland 2016) with particular emphasis on the development of rolled I-beam sections (Sellew 1913; Jewett 1967, 1969; Peterson 1980, 1993), as well as the history of steel design code (Galambos 1977, 1990). This overview combines these complementary narratives into a single timeline, showing how the development of manufacturing capacity and theoretical knowledge have interacted over the last two centuries to yield our current set of standard sections and design practices.

The first industrial use of metal in construction can be traced back to railway design. The earliest rails were made of timber and clad with thin plates of iron, making them highly subject to wear (Petticrew 2014). In 1794, British inventor William Jessop proposed a version made of solid cast iron (Clarke 1846). By modern standards, Jessop's fish-bellied rails were short and brittle, spanning only 3 ft between their stone supports, but were more durable than timber and were therefore rapidly adopted by the British railroad. Cast iron continued to be used until 1820, when John Birkinshaw proposed and patented a selection of wrought iron profiles (Birkinshaw 1824).

Birkinshaw's central insight was that cast and wrought iron handle tensile loads very differently. Whereas cast iron tends to crack like masonry, wrought iron is more elastic. Leveraging its malleability, Birkinshaw adapted and refined contemporary methods for rolling wrought iron bars to meet the needs of his rails by modifying the pattern cut into the rollers to include a flange. The material and section modifications halved the rail's linear weight compared to the equivalent strength cast iron rail (Roberts 1978). In 1830, Robert Stevens, chief engineer of the Camden Railroad, improved the Birkinshaw rail by adding a broad lower flange for more tensile capacity. Stevens' rail is considered a direct antecedent of the modern T-section (Merdinger 1962).

Interest in wrought iron as a building material was driven by the hope of developing a fireproof alternative to timber (Fairbairn 1870). However, other than full-scale load testing, there was no good way to predict how shape and material would affect a beam's performance. To arrive at modern analytical methods, significant improvements first had to be made in beam deflection theory.

The earliest milestone for elastic bending theory, which governs the behavior of beams, is attributed to Galileo. In 1632, he showed that the proportion of the maximum moment to the section modulus remains constant for a cantilever of a constant cross section (Galilei and Crew 2015). His final equation was unfortunately incorrect by a factor of three, as he had not accounted for compressive stress. Nevertheless, the equation remained popular for hundreds of years. It was not until 1826, with Navier's expansion on Euler and Bernoulli's 1744 differential beam theory, that the modern theory of in-plane beam bending theory ($M = fS$ where $S = bd^2/6$) was adopted into practice (Galambos 1977; Timoshenko 1983).

While theoreticians debated exact beam deflection equations, empirical formulae based on the results of detailed load testing were rising in popularity. The following equation, published in 1831 by Eaton Hodgkinson, was widely in use in the mid-1800s:

$$W = 26(A_f d)/L_b$$

It specified the geometric requirements for a safe beam under a centrally applied load, with respect to its tensile yield limit. These requirements assume a Stevens-style rail beam, which has bottom flange area (A_f) and unbraced length (L_b). It is the first widespread example of a limit-based design prescription in metal beams (Petroski 1994).

Sir William Fairbairn, an engineer at Leeds, built on Hodgkinson's work with a series of experiments that specifically described the material properties and failure modes of iron. In response to contemporary construction trends, his 1854 first edition, which only reported cast iron

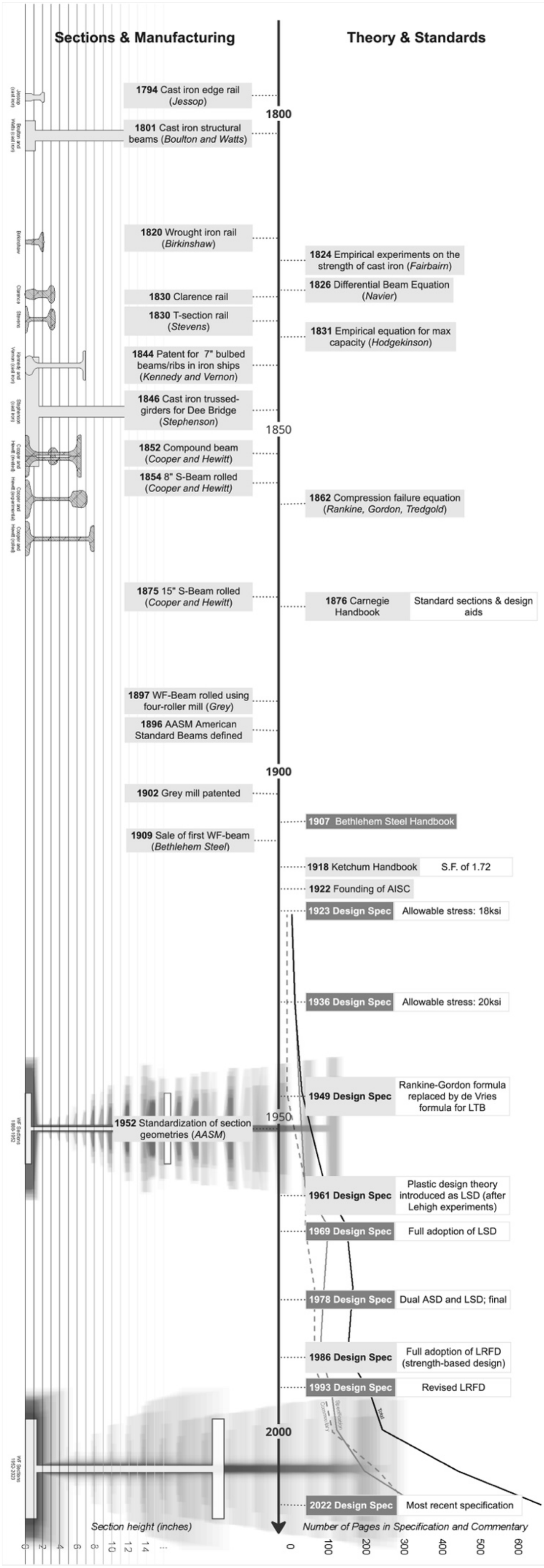


Figure 1. Progression of geometry and design methodology since the late 1700s.

tests, was quickly appended in 1857 with notes on wrought iron and its application in bridges. By the fourth published edition in 1870, Fairbairn's manual also included an extensive section on steel (Fairbairn 1870).

The evolution of Fairbairn's manual reflected a pivotal moment in structural engineering. In 1856, Henry Bessemer patented a new steel manufacturing process that allowed him to produce malleable iron quickly and cheaply. Subsequently adding precise quantities of manganese and carbon yielded high-quality steel. It was a revolutionary material: a homogenous, isotropic product rolled from a single ingot or bloom; steel did not delaminate or chip like wrought iron. Over the next decades, steel would overtake wrought iron as the material of choice for structural engineers (Flavell-While 2010).

The 1850s also marked the first serious attempts at rolling a wrought iron heavy rail that could double as a structural beam (Misa 1999). Until then, large loads had been handled by riveting two rolled channels together, forming a compound girder. In 1849, Cooper and Hewitt's Trenton Iron Works embarked on a five-year venture to manufacture a monolithic, 7-1/8 in deep section. Their main product, a 4-1/2 in deep section, was manufactured in much the same way as Birkinshaw's 1820 rail. The pressure required to drag iron through the mill's two rollers increased for larger beams, a force which often broke the rollers (Jewett 1969). Nevertheless, in 1854, Trenton Iron Works successfully delivered the first 8-1/16 in rail beams to Cooper Union in New York. Trenton beams and girders quickly became the gold standard in fireproof construction (Peterson 1980).

The Trenton sections closely resembled the modern S-beam. Despite increased web depth, the two-roller mill was still capped at 6 in-deep flanges, since deeper flanges were prone to ragged edges. The rolled metal was not guaranteed to fill every corner of the two-roller pattern. As a result of using lighter and more flexible sections, engineers had to seriously contend with compression and buckling failure modes that had not occurred in larger cast-iron sections.

In 1862, engineers Rankine, Gordon, and Tredgold responded by publishing a formula relating the moment of inertia of a beam to its resistance to compressive forces (Rankine 1862). The Rankine-Gordon-Tredgold formula was empirically derived and claimed to universally account for lateral-torsional buckling, web and flange crippling, and column buckling. Despite its conservative nature, it was straightforward, and remained a staple design tool well into the 1950s (Galambos 1977).

The challenge of narrow flanges is illustrated in the American Association of Steel Manufacturers' (AASM) 1896 list of American Standard Beams (ASB). Their goal was to consolidate steel products, which at the time were diverse and highly mill dependent. Providing a specification complete with the linear weight, width, web thickness, and flange slopes allowed several mills to offer a common product alongside their specialty sections (Friedman 2010).

The final step towards the modern W-section was taken by Henry Grey. His proposal, patented in 1902, was to add two new perpendicular rollers specifically for the flanges. This allowed for greater control of flange geometry, ensuring consistent section properties (Grey 1912). The Bethlehem Grey Mill began producing Wide Flange (WF) sections in 1908 (Hoover 2021) (Hessen 1972). With this leap in manufacturing technology, steel became fully actualized as a construction material. Future advancements in the 20th century were largely dedicated to understanding the behavior of WF beams and finding common ground between empirical and analytical methods of beam design.

Steel engineering in 1922 was incredibly heterogeneous. Design manuals were decentralized, often peddled by steel mills as product marketing materials. For instance, 28 different column formulae were in use, with maximum stress limits varying by as much as 10 ksi. The American Institute of Steel Construction (AISC) was established that year as "a single code authority... to eliminate the confusion that then existed in the construction industry" (Galambos 2016).

By 2023 standards, the AISC's first specification was quite concise. The 1923 publication condensed guidelines for typical structural members under tension, compression, shearing, and bearing into just two pages (AISC 1923). From there, code and design standards evolved significantly to reflect advancements in engineering knowledge. We highlight seven editions of the AISC standard to showcase key milestones in design methodology, such as the shift from Allowable Stress Design (ASD) to Load Resistance Factor Design (LRFD) (Galambos 1977).

1923: In the first AISC specification, the allowable stresses in tension and flexure were defined as 18 ksi, and in shear 12 ksi, equating to an approximate factor of safety of 1.83. All compressive capacities were handled by variations of the Rankine-Gordon-Tredgold formula (AISC 1923).

1936: The allowable stress in tension and flexure was increased to 20 ksi, and 13 ksi in shear. The approximate safety factor dropped to 1.65 (AISC 1936).

1952: The extensive catalogues of mill-specific section geometries, reaching at least 1849 unique sections by 1950, were consolidated into a single AISC standard catalogue, completing the separation between designer and fabricator.

1969: Several advancements in beam theory over the previous two decades were incorporated in the 1969 design specification. These include the 1949 introduction of the de Vries formula as a replacement for the Rankine-Gordon-Tredgold Formula in LTB, as well as the theoretical findings from a three-paper series published at Lehigh University in 1961. These papers provided a full description of LTB and its failure modes, laying the foundations for LSD based on plastic behavior (Basler and Thurlimann 1961). Although the 1961 and 1963 specifications were appended with LSD provisions, it was not fully embraced until 1969.

LSD breaks with tradition by allowing for the "proportion[ing] on the basis of plastic design, i.e., on the basis of their maximum strength" rather than on maximum stress. Furthermore, the concept of separate load factors for dead (1.3) and live loads (2.17) was introduced. To reassure engineers of the validity of this method, the new load factors were calibrated such that a bridge of 12 m span would weigh the same under both ASD and LSD (AISC 1969).

1978: One of the last specifications published before LRFD was adopted by the AISC. To differentiate between the two paradigms in use, elastic ASD and plastic LSD methods are treated separately in our analysis (AISC 1978).

1993: LRFD was introduced in 1986, introducing probability in design loading as well as nominal section capacity This required a complete rethinking of the way steel structures are conceived and designed (AISC 1993).

2022: The most recent publication of the AISC specification remained quite like the 1993 specification, though there were significant improvements to the readability and concision of the code (AISC 2022).

The primary difference between ASD, LSD, and LRFD is the interpretation of maximum strength. ASD is based on the elastic strength of materials and finds itself in the same lineage as Galileo's 1632 equation. The stresses in the material are not allowed to exceed a material's yield stress, divided by a safety factor. LSD, on the other hand, is a prototypical plasticity-based limit state design method. It is based on strength with respect to the limit states beyond which failure occurs. These may be based on strength, stability, or deflection. The design strength of a structure, multiplied by a reduction factor, must be greater than the factored loads. LRFD improves on LSD by probabilistically considering the uncertainty around loading and material properties of a structural system. The former is more common in Europe, whereas the latter, which is fine-tuned to American building code, is most common in the US (Galambos 1995).

2. Large-scale structural analysis of steel cross sections

2.1. Methodology overview

The previous section has shown that since the early 19th century, significant developments occurred in materials science, engineering knowledge, and manufacturing capacity. These developments have continuously altered the process of selecting a suitable beam cross section for a given design load, and the resulting mass of steel consumed. A previous study demonstrated this variance through a flexural design example in which the optimal cross section area was reduced by 46% from the first AISC Specification of 1923 to the contemporary plastic design methodology of 1970 (Galambos 1976). The generalizability of this finding is explored in this section through an extended computational analysis of time-dependent section geometries and period-specific nominal capacities.

Both historic and contemporary I-beam geometries were numerically tabulated for a total of 2129 unique sections since 1888. The nominal shear, V_n, and moment, M_n, capacities were then calculated for each section across seven design methodologies. Next, 13,000 unique beam-load combinations were generated to provide a representative set of design loads to be resisted by these geometry-capacity pairs.

The sets of available geometry, nominal capacity, and beam demands then formed the basis for three analyses: (1) the progression of period-appropriate nominal capacities since 1888, (2) the progression of material efficiency in beam sizing using the results of (1), and (3) an investigation of the effect of each new design methodology on the nominal capacity of all tabulated sections.

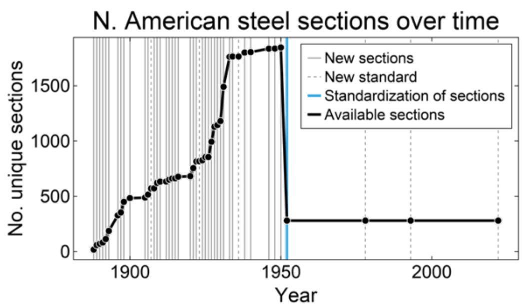


Figure 2. Availability of sections and key changes in methodology since 1888.

2.2. Section geometry

Two sources provided the data for both historic and contemporary I-beam sections: a historic sections database, published by the AISC immediately after the initial standardization of steel geometries in 1952 (AISC 1953), and the AISC's contemporary shapes database (AISC 2023).

The historic shapes database includes the geometric parameters, years of production, and whether a section was produced for beam or column loading. Two assumptions were made for historic steel sections in this study. First, for a given year before consolidation in 1952, we assumed that all sections which had been produced up to that point were available to a structural designer. Second, we focused solely on the sections designated for beam members. In total, 1849 unique sections from 1888 to 1950 were extracted, representing the production of 14 different steel mills.

Detailed data for sections available after 1952 were difficult to obtain, and granular information on the year of introduction of modern sections could not be found. As such, we have assumed that all tabulated sections in the latest shapes database were available beginning in 1952. Further, the modern shapes database does not delineate between regular sections primarily used for beam applications and heavy sections used for columns; we have thus included all 283 tabulated sections for years after 1952. Lastly, we assumed that post-consolidation, only these sections were available to structural designers. An overview of both section availability and methodology changes is shown in Figure 2.

2.3. Capacity analysis

Seven unique design methodologies used by engineers between 1900-2023 were selected and automated to extract period-appropriate nominal capacities in both shear and moment for all sections. The nominal strength of sections for periods before the first AISC Specification in 1923 was calculated using the Bethlehem Steel design handbook of 1907 (Bethlehem Steel 1907); the remaining methodologies were based on the six representative AISC Design Specifications identified in Section 1.

The calculation of material strength has changed notably since the beginning of standardized structural steel design. Early design specifications provided explicit values for the allowable bending, F_b, and shear, F_v, stresses regardless of the actual steel alloy used to make a given section. This eventually transitioned into allowable stress limits calculated as a proportion of the material's tensile yield strength. To center the focus on the changes in engineering mechanics when comparing different design methodologies, we set the material yield strength to 36 ksi for all analyses. For early design specifications that provide explicit limits for F_b and F_v, these values were used instead.

A key development in the progression of flexural steel design is that of geometric stability in both local (slenderness of compression flanges) and global (lateral-torsional buckling) domains. These have ranged from explicit upper bounds for unbraced lengths to complex calculations of strength reduction factors in both shear and moment resistance that are functions of local geometry and internal force distribution. For simplicity, the unbraced length, L_b, was taken as zero for all analyses; however, all strength reduction factors with respect to local geometric slenderness limits were applied when calculating nominal resistances.

Throughout the seven design methodologies studied in this paper, there are three distinct approaches to addressing uncertainty in structural demand and capacity: ASD, LSD, and LRFD. To provide a fair comparison between the nominal resistances of different reliability philosophies, the capacities of LSD- and LRFD-based methods were further normalized for direct comparison with ASD-based results. The LSD method, represented by the AISC 1978 specification Part 2, provides an explicit requirement of resisting 1.7 times the nominal design loads; all calculated capacities using this specification were reduced by this factor.

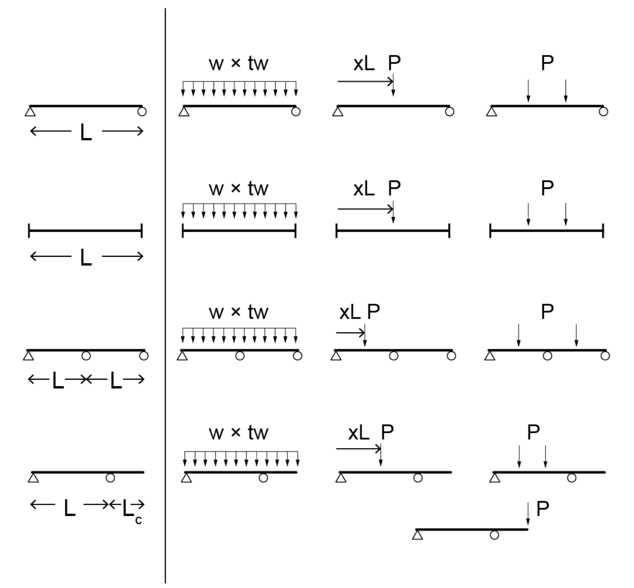
For LRFD-based capacities, used in the 1993 and 2022 methodologies, the resistance factors, Φ, were included in all calculations. The equivalent normalization with respect to the load factors, γ, was based on the principle of equivalent reliability to ASD methods when LRFD factors were first calibrated (Galambos 1999). This equivalent reliability was based on an assumed Live-to-Dead load ratio of three; thus, assuming the factored load to resist using LRFD was based on a load combination of $1.2D + 1.6L$, the normalization factor is then:

$$\mathrm{w_f} = 1.2(0.25\mathrm{w}) + 1.6(0.75\mathrm{w}) = 1.5w$$

The nominal resistances of all LRFD-based capacity measures were further reduced by a factor of 1.5.

2.4. Demand sampling

To generate the representative beam loads for optimal sizing, 13,000 unique beam-load pairs were generated across four different boundary conditions and four different load types. All geometric and load variables were sampled from assumed normal distributions; an overview of the sampled boundary conditions, loads, and statistical parameters is shown in Figure 3. For each beam-load combination, the peak absolute shear force, V_f, and moment, M_f, were taken as the design load for beam sizing. The distribution of these demands is shown in Figure 4. Demand requirements for serviceability criteria such as deflection or vibration were omitted in this study.



	L [ft]	L_c/L	w [psf]	tw [ft]	P [kip]	x
mean	25	0.25	150	8	20	0.5
std. dev	5	0.05	10	2	5	0.1

Figure 3. Parameters for demand sampling.

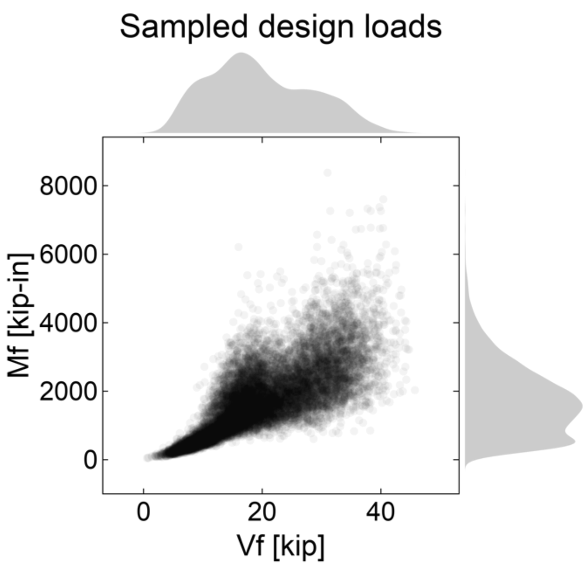


Figure 4. Distribution of shear/moment demands.

3. Findings

The combined sets of available geometry, engineering methodology, and structural demand were then evaluated over time to study the changes in nominal capacity and the resulting material efficiency of steel beams.

3.1. Period-specific capacity

The distributions of nominal shear and moment capacity available to structural designers between 1888 and 2023 are shown in Figure 5. Each point represents the nominal capacity for a unique section at a given year, using the most up-to-date design methodology.

Up to section consolidation in 1952, the nominal structural capacity steadily increased due to both the larger distribution of available cross sections and improvements in design methodology. Post-consolidation, the mean capacity of both shear and moment increased significantly, primarily from the inclusion of heavy W-sections intended for columns in the pool of available geometries. However, as observed by the increase and upward shift of the quartile bands, the standardization of section geometries generally enabled a larger mean and range of structural capacity.

When normalized by gross area to measure material efficiency, as shown in Figure 6, the changes in capacity over time are less prominent. Further, for area-normalized shear capacity, there is no clear trend of increased efficiency over time. The mean value has decreased since the start of the analysis. For area-normalized moment capacity, a consistent

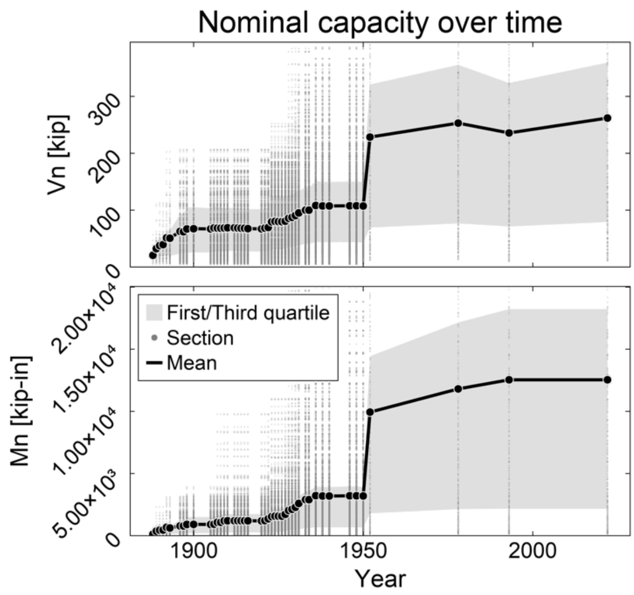


Figure 5. Progression of nominal capacity over time.

increase in the mean value and bounds is maintained, albeit with a less prominent jump in efficiency in 1952.

3.2. Period-specific optimal sizing

For each of the 13,000 sampled beam demands, the minimum-area section with sufficient nominal capacity was identified for each year. The results of this period-specific optimal assignment are shown in Figure 7. In the early years of the analysis, a lack of variation in available cross sections, specifically that of deep members, resulted in insufficient capacity for a portion of the sampled demands. This period is represented by the pink dashed line from 1888 to 1907.

Once sufficient variation in available geometry was available, the average area of the optimally assigned section steadily decreased from 16.8 in^2 in 1907 to 12.1 in^2 in 2023, a 28% reduction. This change in material efficiency is primarily due to advancements in design methodology: the

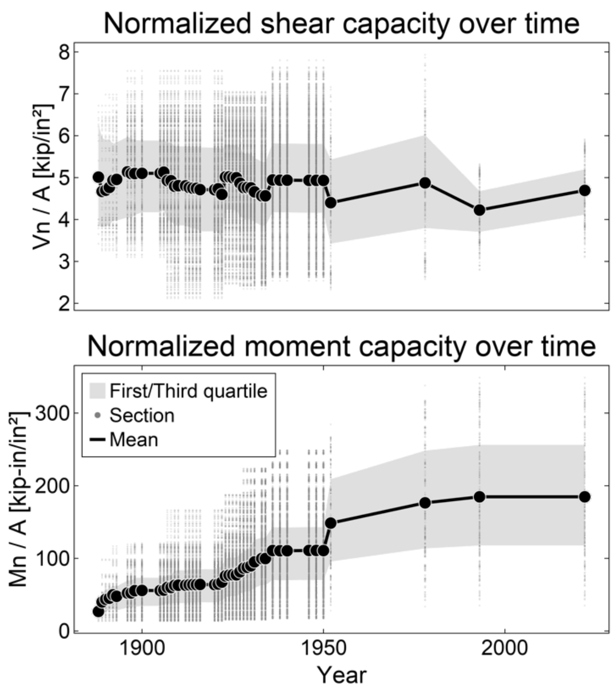


Figure 6. Progression of normalized capacity over time.

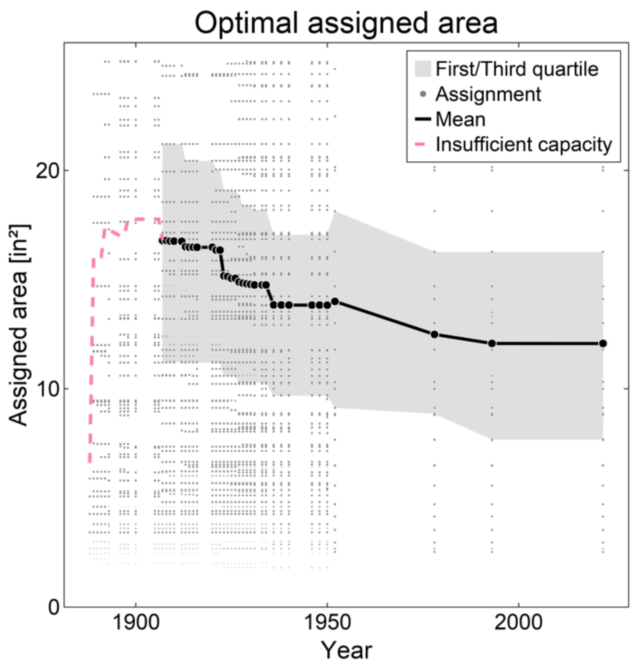


Figure 7. Area of optimally assigned section.

observable large drops in mean area coincide with the introduction of new design standards, as shown in Figure 2.

An alternative interpretation of efficiency, that of capacity utilization, is shown in Figure 8. The utilization of shear capacity has varied significantly over time, with a large spike in the early 1900s before a stable reduction and subsequent increase post-consolidation. At no point does the mean shear capacity utilization greatly exceed 50%. In contrast, the utilization of moment capacity has steadily remained near 100%, with peak mean utilization occurring before the consolidation of steel sections in 1952. This consistently high utilization affirms that the design of flexural members is primarily dominated by moment capacity.

3.3. Progression of nominal capacity

To further investigate the effect of design methodology on section capacity, our final analysis compared the change in nominal strength for all 2129 unique sections, regardless of

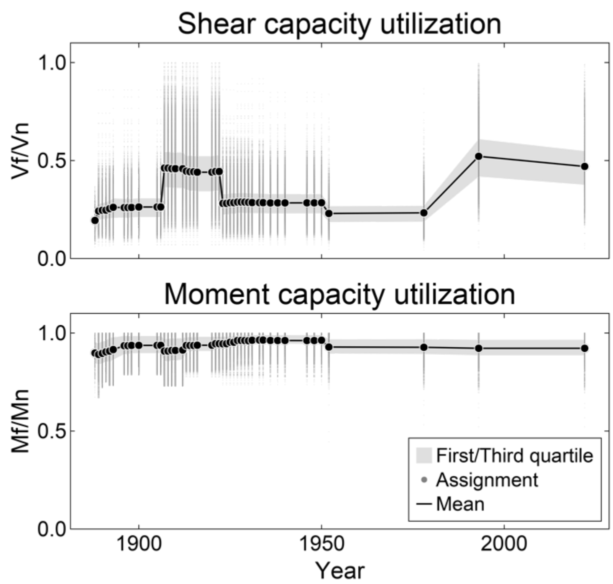


Figure 8. Nominal capacity utilization over time.

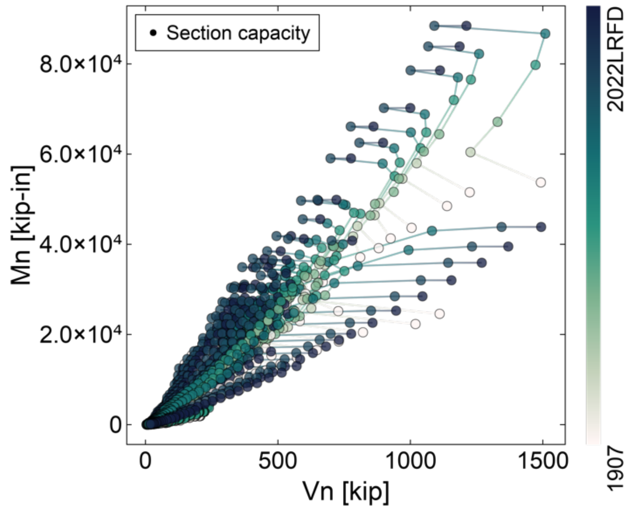


Figure 9. Progression of nominal capacity.

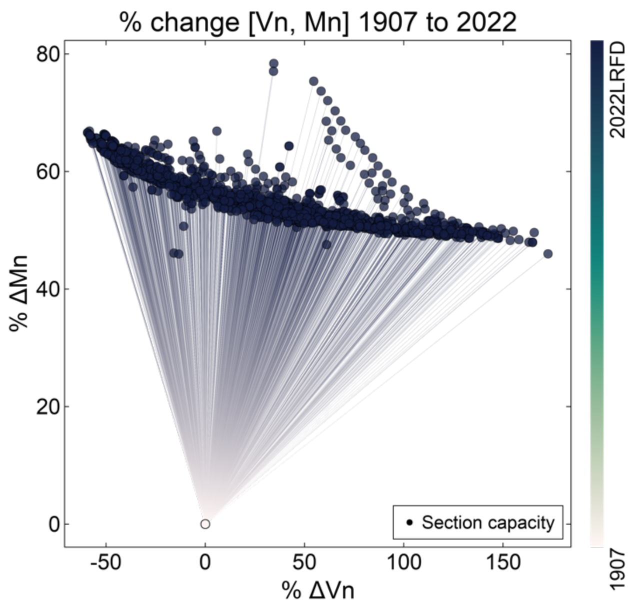


Figure 10. Relative change in nominal capacity since 1907.

the year of production. The progression of nominal capacity for all sections is shown in Figure 9. The change in capacity of a single section is represented by a polyline whose vertices represent one of seven design methodologies.

In general, nominal moment resistance has increased with each new development in design methodology. Changes in shear resistance were more sporadic: on average, the nominal shear capacity has slightly increased over time, but with a large variance among all sections. This variation in capacity is a result of the turbulent history of shear design methodologies. Whereas moment resistance was based on well-established beam theory, the calculation of shear strength ranged from allowable stresses measured by scalar factors of tensile yield strength to a solid mechanics based equivalent shear stress in modern approaches. Lastly, a more thorough understanding of web buckling and slenderness effects led to the use of empirical reduction factors based on the slenderness of unstiffened webs by 1961. By contrast, changes in nominal moment capacity when lateral-torsional buckling is ignored were due to increases in material strength, and the shift from section to plastic modulus.

The direct change of nominal capacity from a pre-AISC, mill-specific design methodology to the latest LRFD-based design specification is shown in Figure 10. As observed, the nominal moment capacity has increased by over 50% on average, whereas a significant portion of section geometries have had a net decrease in overall shear capacity.

Conclusion

Through an extensive computational investigation into the intersections of available geometry and engineering knowledge, we have shed light on the material impacts of the progression of both geometry and engineering knowledge of steel beams.

There is an overall trend towards increased material efficiency, driven by the dual factors of available geometry and engineering knowledge. As more unique sections were offered in the late 19th century to 1952, a finer gradation of section capacities was provided, allowing engineers to choose sections with capacities that more closely matched structural demand. Simultaneously, improvements in the understanding of structural mechanics resulted in an increase in the as-calculated capacity of steel sections. Between these two developments, it was observed that changes in design methodology had a much larger contribution in the improvement of structural efficiency, where the large drops in Figure 7 consistently coincided with improvements in the AISC specification.

Although both the nominal shear and moment capacities of steel sections have increased up to 80% from 1888 to the present day, the progression of as-calculated shear strength was often sporadic and dependent on the cross-sectional geometry. Unlike the modest changes to moment resistance calculations, the interpretation of the mechanics of web shear changed significantly well into the 1970s, resulting in nominal capacities that have decreased over the past century.

Overall, we have shown that the changes in available nominal capacity, primarily driven by advancements in engineering knowledge, have had a significant material impact on the mass and material consumption of steel structures. Since the late 1800s, a 25% decrease in mean structural mass was observed. Future work will focus on introducing additional design variables to the analysis, including the increase of material strength over time, serviceability limit states, and the effect of unbraced length on nominal moment capacity.

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