The Importance of Steel to Wind-Resistant Building Frames: Riveting and the Quest for Structural Rigidity

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ABSTRACT: The gradual development of the tall commercial building at the end of the 19th century is usually described as an outgrowth of steel’s development as a building material. Rarely, however, are the mechanisms of this enabling described. One might well ask why steel per se had very much to do with skyscraper construction, as it was seen by many as an expensive, unreliable version of structural iron. Iron had been the primary structural material in most tall buildings during the mid-1880s, during which record heights of eight, ten, and eventually fourteen stories had been readily achieved. Why would architects, engineers, and clients change their material preference so quickly – from about 1888 until about 1895, when Engineering Record suggested that any use of iron in building “could not be recommended” – and what about steel made it so overwhelmingly better than iron? The need for stiff connections and for lateral resistance provides one possible answer.

STEEL AND IRON

Iron enjoyed an early advantage over steel in building construction, in both reputation and cost. It had been used extensively in two forms since the mid-18th century in machinery and bridge structures before finding widespread use in mill construction of the 1790s and early 1800s. These two forms – wrought and cast – referred primarily to its method of fabrication, but also, thereby, to its range of chemical content. Cast iron was closer to raw, pig iron in its high carbon content, which created a strong but brittle material; it could not be worked easily except at temperatures near melting. Wrought iron, on the other hand, relied on time and labor-intensive puddling to remove carbon; this resulted in a loss of strength, but also in ductility at relatively cool temperatures that meant it could be hammered or rolled into useful shapes.

Together, these two forms of iron structured early tall building construction, from the 1851 Crystal Palace to early skyscrapers in New York and Chicago. Iron’s lighter weight and (in its wrought form) its capacity for tensile stress decreased the amount of floor and wall area consumed by a building’s structure over wood or masonry. Its fire-resistive properties were controversial, and eventually architects developed methods of cloaking iron structures with terra cotta or brick to protect it from the effects of high temperature, but this added expense was minor compared to the tangible efficiencies it offered. Over time, concerns were raised about cast iron stemming from its fabrication processes, and the inability of mills to guarantee the material’s freedom from internal flaws or stresses resulting from the violence of the casting process; conventional wisdom eventually suggested that cast iron be used only in compressive situations, where a flawed column would fail only to the extent of the imperfection while tensile or bending members were reserved for wrought iron, whose composition could be more definitively established and which were, therefore, more reliable in tension.

While there was a nascent steel industry in the United States by 1860, it would have seemed unlikely at that time that steel could overtake iron as structural product. Steel required careful control of raw materials and admixtures to create its narrow band of chemical composition; the result was a material that could be worked like wrought iron, but that was tough and strong like cast. This was desirable for tools and for cutting implements but it would have been difficult to imagine how this property could ever apply in building construction. Structural elements didn’t require sharp edges, and the ductility necessary for rolling structural shapes was already a natural property of wrought iron. The structural performance of steel was better than wrought iron, to be sure, but not so much better as to suggest a wholesale move away from this known, relatively cheap mate-
And, certainly, this ductility would not have seemed – at all – to have threatened cast iron’s use for columns, where statically ideal cylindrical shapes could only be manufactured by casting, not by rolling. The extraordinary and rapid replacement of cast and wrought iron by steel took less than a decade, from the publicized use of steel in building construction in the Home Insurance Building in Chicago in 1885 to Engineering Record’s definitive pronouncement in 1895. What occurred in the intervening decade paired scientific understanding and testing of steel leading to its acceptance as a reliable, calculable product with a realization that its pairing of strength with ductility allowed it to solve one of the great problems of skyscraper construction – wind bracing – in ways that cast and wrought iron could not. Steel’s combination of high strength and superior workability enabled self-supporting metal frames that themselves resisted gravity and lateral loading and that needed no assistance from masonry shear walls. By bracing themselves against wind, the generation of steel frames that developed after 1885 realized efficiencies that stemmed from the substitution of light steelwork for heavy masonry. Freed from the need to resist wind by massive brick walls, steel frames could fulfill metal’s promise of lightness and openness, and could realize the promise of the metal frame as a structure that was virtually negligible in its floor and sectional space. Wind was the major structural problem that could not be solved with iron alone; because of its precisely tuned physical properties, steel was wind’s answer.

THE NEED FOR BRACING

Wind bracing had rarely entered into the structural calculations for heavy masonry buildings as the dead weight of its construction absorbed sideways and overturning forces presented by wind. However, the light weight of skeletal buildings, their increased height, and the nature of their steel and iron connections brought this issue to the fore. The tall buildings of the 1880s were among the first in Chicago to recognize this problem and to provide for it with dedicated lateral or ‘shear’ systems; the Home Insurance (1885), the Tacoma (1889), and the Rookery (1888) all relied on vertical masonry walls set at right angles to ‘stay’ themselves against wind.

The problems presented by wind in tall building construction were threefold. First, as buildings were built higher in proportion to their base, the overturning moment created by a gust of wind against their sides increased. The building functioned as a giant, vertical cantilever, anchored at its base, with a distributed load of wind over its entire surface. Higher buildings presented exponentially more difficult problems, as these increased the area of exposed wall that could gather wind load and increased the length of the lever arm by which wind could pry the building out of its foundation. Heavy buildings offered natural resistance to this, as their windward exterior wall would present a weight far too great to be lifted by the wind. However as building skins ceased playing a role in these buildings’ structures, they no longer needed large volumes of masonry, and the lighter skins of the skeleton era no longer offered the guarantee of wind resistance through weight. Second, even if buildings were built to resist the overturning effects of wind, the internal stresses induced by such resistance could be formidable. Again, since these structures were being asked to behave as vertical cantilevers, they had to accept both shear and bending induced by wind throughout their frames. Shear oc-

Figure 1: Plan of the Tacoma Building (Holabird & Roche, 1889) showing internal shear walls; (Author’s drawing)
curred throughout columns, as the structure absorbed the horizontal load of the wind; if columns were too small wind could slide the mass of the building off its foundation, or ‘shear’ off the building at a weak story (Quimby 1892, p. 394). This problem was compounded by the desire on the part of clients to open up ground level floors with large window and door openings, and by the tendency to rent these out to banks and shops that required large, open spaces. Bending presented additional problems (Breithaupt 1892, p. 226). To absorb the leverage of the wind acting on the building face, columns on the leeward side of the frame would be compressed, while those on the windward side would be stretched. These loadings added complexity; columns that bore the compressive effects of a sudden gust might well be pushed beyond their safe compressive load.

Third, and most importantly, wind forces added unpredictable loads to frame connections. While large-scale wind engineering could be solved by mass or by proper sizing of structural members, the failure of column and beam connections involved much more detailed analysis, complex math, and an understanding of load distribution and material behavior that simply did not exist in the 1880s. Concerns over the performance of connections were not simply theoretical; in December, 1879, the Firth of Tay Bridge in Scotland collapsed in winds that were within its claimed limits. A subsequent investigation proved that the bridge failed largely because its connections were poorly designed. Over time, repeated loads stretched the iron of the bridge so much that emergency shims had to be inserted to prevent ‘slackness’ in its iron frame; the excess motion caused by this condition created additional dynamic loads on fasteners, ultimately leading to the structure’s demise (“Tay Bridge Disaster” 1890, pp. 70-71). Connections therefore needed to be made strong, but they also needed to be made tight; any slackness could lead to dangerous dynamic loads.

These dangers led designers to three solutions: building massing, systematic structural bracing, and stiff connections. The overall shape and section of buildings remained important, and rules of thumb let engineers and architects know when the proportions of designs began to approach dangerous limits. Edward C. Shankland, who engineered Burnham’s buildings of the 1890s, suggested that a building’s height could exceed its base by a factor of four to six without requiring special frame design, while other experts suggested safe proportions of only three to one (Shankland 1896-1897, pp. 6-8).

Beyond these proportions, however, engineers and architects had to find ways to channel and resist lateral forces in lightweight frames. Masonry walls, which had been used on buildings such as the Tacoma to absorb loads from wind, also reached their peak efficiency long before the heights being achieved in the late 1880s (Quimby 1892, p. 298). Instead, the metal frame, which had been seen as an efficient way to build up against gravity, also came to be seen as an efficient way to channel wind forces down to the foundation. Here there were few architectural precedents but the world of bridge engineering showed the way forward (Waid 1894, p. 158). Railroad bridges employed trusses to absorb gravity loads, using the triangular geometry of truss configurations to achieve cantilevers and single spans with far less weight than traditional masonry arch bridges. By taking these principles and standing them on end, engineers had a valid analogy for designing against wind loads; trusses could be used in place of masonry walls to absorb the bending and shear of lateral loads.

Wind-bracing systems became important parts of structural frames as a matter of course in the boom of 1890-91, and took three different basic forms. Each of these relied on metal rather than masonry, eliminating weight; each thus allowed more open plans and facades than the masonry systems of the previous decade. Each also depended upon increasingly precise standards in manufacture, since as the Tay Bridge disaster had pointed out, slackness in structural frames due to imperfect geometries or alignments could lead to failure through repeated, dynamic loading. The three basic wind-bracing schemes added members or connections to induce building frames to act as cantilevered, vertical trusses. In order of increasing complexity, they were rod- or sway-bracing, knee braces and portal frames, and lattice or plate girders.

![Figure 2: Four types of wind-bracing. Sway-rods (over one or two stories), portal frames, knee braces, and ‘lattice girders’. (Freitag 1904, p. 258)](image-url)
Of these, the most bridge-like was rod- or sway-bracing. This technique employed diagonal tension members set within rectangular panels of the building frame, and connected, typically, to intersections of column to girder. The resulting cross-bracing thus triangulated each panel, providing a shape that could resist loading through its geometry. Any lateral load trying to deflect the vertical truss would be unable to change the shape of these triangular panels without stretching the metal tension rods; thus, the extraordinary tensile strength of steel could be directly deployed against such sidesway. Over multiple stories, these rods had to connect to one another – the tensile loads they absorbed had to be transferred to similar triangulated panels in stories below, and indeed all the way to the foundation in order to avoid ‘weak stories’ in which the combined shear force of the wind above could not be absorbed. Thus, buildings braced by sway-rodss typically had two or more dedicated lines in plan on which, at every level, these rods would connect to columns and girders. Plans of sway-braced buildings thus essentially mimicked those of masonry shear wall buildings, locating the lateral resistance along carefully thought out lines of structure – but with the key difference that sway-bracing occupied a plan width of an inch or two at most, while masonry shear walls required a foot of material or more to be effective. With proper planning, these diagonals could be absorbed into walls, however their geometry restricted or prevented doors, windows, or other openings in these panels, unless the rods were extended over two stories instead of one. Sway-rods were typically the most economical solution to wind bracing as well as being the lightest, but these problems of planning – particularly at ground level where open space was at a premium often obviated their use (Shankland 1896–7, pp. 7–8).

A common alternative to full diagonal sway-bracing was the knee brace, in which shorter diagonal members were placed between columns and girders to triangulate their junction, instead of a whole panel. Such elements were borrowed from ship construction, where stiff connections between deck and hull were required. By fixing the angle between girder and column, designers assured that bending loads in one would be transferred to the other, effectively ‘recruiting’ the cross section of one member to assist in resisting the load on another. This added immense flexibility to floor plans, since there was no need to sacrifice whole panels to structure. However in section these braces provided some problems, as they took up headroom near the columns; architectural solutions included coved ceilings, the alignment of corridors away from these restrictions, and large column heads that concealed the short diagonals. In narrow buildings, such braces could grow to unwieldy size; so-called ‘portal frames’ provided arched or triangulated shapes in which the distinction between girder and column was practically lost. These buildings (particularly the Old Colony) show sections that can almost be read as steel walls with holes cut through them, rather than spidery skeletons with wind-bracing attached. While numerous examples of knee-braced and portal-framed buildings were constructed, the consensus reached by the mid-1890s was that the weight of metal in these solutions, coupled with the need, again, for exacting fabrication and erection, made them uneconomical, and their use faded quickly (Freitag 1904, p. 259).

The third method of wind-bracing was one of the key innovations in tall building construction to emerge from the laboratory conditions of Chicago. It had long been noted that tall buildings had inherent stiffness due to the dead weight and the geometry of their internal partitions and floors – properly mortared terra cotta fireproofing and floor arches provided reasonably rigid diaphragms in all three directions (Freitag 1904, p. 260). While no engineer was willing to rely entirely on these partitions for lateral stiffness, Shankland (among others) realized that if the building frame itself could be made stiffer at each major junction, then the overall building frame could on its own develop a reliable resistance to wind load. Essentially, Shankland proposed that the stiffening function of a knee brace might, with the right materials, be compressed into a single joint. Such an approach walked an engineering tightrope; it required far greater precision in fabrication and assembly than either of the other, coarser approaches, and it relied on hundreds of small joints rather than dozens of additional members. But creating a stiffer frame offered extraordinary flexibility in plan and section and, crucially, it opened up the sections and elevations of these buildings entirely, as the frame could be designed to stand on its own, without additional members or stiffening elements interrupting the open spaces of the gridded cage. Shankland’s innovation was typically called a plate or lattice girder, but this was something of a misnomer in that it relied on columns as well. The girder in question was made intentionally deep – usually 24” or more, greater than the depth needed to simply span typical bay sizes. Likewise, columns in this system were oversized in width to help absorb bending, and special shapes were produced that offered wide flanges on all four sides. These flanges could then be connected directly to the webs of the deeper girders (using newly available techniques described below) to form a very rigid connection. As these connections multiplied throughout the frame, they enabled lateral loads to be transferred directly from girder to column. Any deformation due to wind could thus be resisted, at once, by all of these stiff connections working in concert; under such loading, the entire frame could absorb and direct wind loads, rather than single truss panels. Shankland termed this the ‘table leg’ principle, drawing an analogy between the stiff, oversized carpentry joints between tabletops and legs and his own development; but the genius of this principle was that it also worked with multiple stories – a more accurate but less picturesque term, “moment frame” became the standard designation for this principle in later generations (Shankland 1896–1897, pp. 8-9). Each ‘table’ in plate girder buildings was rigidly connected in three dimensions to each adjoining tables above and below, and while the result was a distribution of lateral stresses too complex to mathematically compute, the redundancy of load paths and the sharing of loads across and through the building frame provided extraordinary efficiency, and left floor plans and sections free of any lateral bracing whatsoever. Girders, columns, floors, and partitions were thus all recruited to the task of standing firm against wind, with the result that, while each element might need to gain in depth or thickness,
the overall building maintained the openness that was becoming the functional – and architectural – hallmark of commercial construction in Chicago.

Fully operational lattice or plate girders, though, would only come into use by the mid-1890s, coincidentally at the moment at which the curtain wall likewise reached its apogee in Chicago construction. Thus, while the full potential of this system was denied to the skyscrapers of the boom years of 1890-1894, wind bracing of the former two types – sway rods and knee braces or portal frames – became common in the structures of this era (Jenney 1891, p. 390). Quimby and others, however, recognized the advantages of the stiff frame, and there is considerable evidence that monolithic connections between beams and columns played significant roles in the engineering of Chicago skyscrapers – intentionally or not – well before pure moment frames became common.

In response to Quimby’s paper, engineer J. P. Snow suggested precisely the plate girder approach that would be perfected by Shankland in later years. “If architects and architectural engineers would use built sections of plates and angle irons for their large girders, instead of the conventional rolled beams,” Snow noted, “they could make much more efficient connections with their columns than is usual in ordinary building construction.” (Quimby 1892, p. 99). Stiffness in building frames was, therefore, seen as an important part of wind resistance, even as most buildings of the early 1890s employed the additional insurance of sway rods or braces.

CAST IRON, STEEL, BOLTS AND RIVETS: THE QUEST FOR STIFFNESS IN BUILDING FRAMES

As Quimby noted, finding stiffness within the skeletal proportions of steel frames was a difficult task, requiring careful attention to structural design, element fabrication, and assembly. To ensure that girders and columns could act together as a stiff frame, the individual elements had to be designed for complex flows of force and resistance, and their erection had to be carefully considered, especially for columns that were typically assembled out of linear elements spliced together to achieve continuity. Likewise, the connections between columns and beams posed particular difficulties, as typical rolled shapes offered only limited surfaces and interfaces to connect to one another. The development of structural steel allowed solutions that were not available in cast iron; this was the area in which steel really proved itself, and enabled the construction of far taller skyscrapers than had been achievable with earlier materials.

It is here worth recalling that typical cast-iron construction was inherently limited by its fabrication processes and by the brittle nature of the finished material. In addition to its deadly lack of robustness in fire, cast-iron possessed neither the ductility to allow drilling, nor the accuracy to allow precision bolting or riveting on site. Once out of the mold, erectors had to contend with columns that could not be altered, and that often were slightly out of plumb, dimensionally inaccurate, or slightly twisted from the violence of the cooling process. Cast-iron column construction was, therefore, reliant on connections that allowed great tolerance and that did not require careful alignment. Connections between cast-iron columns and floor beams (of wood or wrought-iron) were thus typically made with pintles and gudgeons, rather than with bolts or rivets, which made them inherently loose and thus required separate bracing systems to keep them upright against lateral forces (Quimby 1892, p. 162). These loosely pinned connections were exacerbated by imperfections in the columns themselves; even in the best cast-iron connections, columns sat atop one another directly, requiring their top and bottom surfaces to be planed accurately. If connecting faces were even slightly out of plumb, the bearing surface between them could be limited to a very slight edge, stressing the iron well beyond its limit (Quimby 1892, p. 180). For connections between columns and girders, the situation was just as dire. Lugs or shelves cast into the column shape were the most effective method of transferring the girder’s loads into the column, but again there was no good way to make a monolithic connection between the two; the beam would typically be set on the lug or shelf with no way of preventing the two elements from racking, or rotating with respect to one another (Birkmire 1900, 192).

Boltholes molded or carefully bored into cast iron elements could provide more reliable connections, but they offered their own problems. Cast-iron’s brittle nature meant that a considerable number of pieces would simply fracture when drilled or, worse, when bolts were tightened in the field. Likewise, the slight inaccuracies that were inherent in cast-iron fabrication were disastrous for bolted connections. A small variation in the shape of a bolthole, for example, would allow slippage between the bolt and its host member. Even a very small amount of motion, as was seen in the Tay Bridge disaster, could be multiplied by repeated dynamic loading (Freitag 1904, p. 68). Or, just as critically, the bolt might rest against only a portion of the metal at the edge of its hole, thus transferring a full load over only a fraction of the cross-sectional area designed for it (Gravelle 1896, p. 75). As a result, “drilled holes and turned bolts” were according to The Engineering News, “scarce ly feasible” in cast-iron construction (“Cast-Iron Columns in Buildings” 1897).

Another means of connection in the field, riveting, offered only a partial solution to cast-iron’s problems. Riveting consisted of heating metal plugs to the point of soft pliability, inserting them into pre-drilled holes in two metal plates, and then hammering both sides flat (or with a slight dome), thus filling the hole completely with metal and, once cool, clamping two pieces together with a durable mechanical connection. Riveting had emerged as a technique for connecting wrought iron before 1850, and its strengths and potential flaws were rigorously examined by William Fairbairn in 1872. The major advantage to riveting lay in the compression of the soft, hot rivet metal within the joint, which would completely fill even an imperfect hole, guaranteeing full bearing of the rivet on both elements; and in the tight fit between the two caps and the metal itself. Properly done, a riveted connection thus offered remarkable stiffness and reliability. It also offered significant speed, with machine-driven gangs capable of driving a single rivet in less than four seconds (“Cast-Iron Columns in Buildings” 1897).
Buildings” 1897). But given cast iron’s brittle nature, the potentially catastrophic effects of hammering rivets continually into place obviated this technique entirely. Riveting could only be done in wrought iron at the time of Fairbairn’s experiments.

Steel, on the other hand, had nearly the ductility of wrought iron. Its rolled nature meant that it presented relatively thin planes that could be punched easily and with much greater accuracy. Bolts or rivets could thus be placed with some assurance. However, even greater accuracy could be achieved by reaming adjacent bolt or rivet holes in the field. The punching or drilling process often left slight tunnel-shaped holes in steel, with a difference in the diameter of the hole of from 2/3 to ¾ the material’s depth. Such a cross sectional shape offered the same problems as inaccurately molded cast iron holes, in that the resulting “blade” profile of the hole’s edge could literally cut through rivets. Engineers typically specified slightly smaller drilled holes in steel members, which were then to be reamed once in place with their connecting member; temporary bolts would hold the two together, the rivet holes would be reamed to a consistent, flat profile, and the riveting gang would then begin work (Gravelle 1896, p. 75). The two elements would now have a reliable, robust connection; the alignment of the holes guaranteed that each rivet would absorb a predictable percentage of the total load being transferred, and that the entire cross section of each rivet would be fully recruited into resisting the load.

Steel offered an additional advantage, in that relatively thin, reliably dimensioned steel plates and angles offered readily available elements for making simple riveted connections in the field. Lugs and shelves could be replaced by separately fabricated steel connectors, which could be pre-punched and reamed either in the factory or in the field to ensure a tight fit (Freitag 1904, p. 138).

As early as 1891, riveted connections using drilled and reamed steel elements and attachments had become standard in all tall Chicago buildings; George Fuller noted that this technique made structures “more solid,” while Jenney praised the technique’s scientific basis (“Chicago’s Big Buildings” 1891, p. 25). The considerable superiority of riveting alone was sufficient for the Engineering News to declare, in 1897, that cast-iron was no longer a suitable material for structural design, and that in fact it had not been one for some time (Strobel 1896, Waid 1894, pp. 158-159). In addition to its reliability, riveting quickly proved to be affordable and rapid. By 1904, the average riveting gang of five (one tending a small furnace, two to toss and catch the hot rivet, and one manning each side of the riveting hammer) could fix over 200 rivets in a nine hour day, with an average cost per rivet of under ten cents (Freitag 1904, pp. 68, 77). Rivets became the key innovation that allowed the ‘lattice girder’ or moment frame, providing reliable, slip-free connections that permitted columns and beams to act as monolithic, wind-resisting systems, rather than as discrete, simply connected elements. Such connections allowed lateral loads to be distributed throughout a frame, providing multiple load paths and redundancy that effectively allowed many small connections to do the work of larger shear walls or panels.
CONCLUSIONS

These seemingly minor refinements to the basic idea of the steel skeleton proved decisive, and a generation of self-braced steel frames followed. With the reliable connections promised by steel manufacture the metal frame quickly lost its experimental status and became an everyday occurrence in all major North American cities. The differences between the pioneering structures of 1885-7 and the more refined examples of the 1890-91 boom were profound. The earlier buildings were necessarily hybrid structures, with a heavy reliance on masonry walls for lateral support, and with columns and connections of unreliable materials, inefficient shapes, and troublesome slackness. Those constructed in the boom years of the early 1890s, on the other hand, relied less and less on masonry for anything other than environmental enclosure. Their connections were increasingly sophisticated, of more scientifically studied materials and of more mathematically derived shapes. Most importantly, they used connections that enabled their lightweight metal frames to stand against the wind on their own, with immediate positive consequences for these buildings' overall weight, and thus their foundations, and thus their height.

REFERENCES

Quimby, H. H., 1892. Wind Bracing in High Buildings. The Engineering Record. 394.