Built Like Bridges: Iron, Steel, and Rivets in the Nineteenth-century Skyscraper
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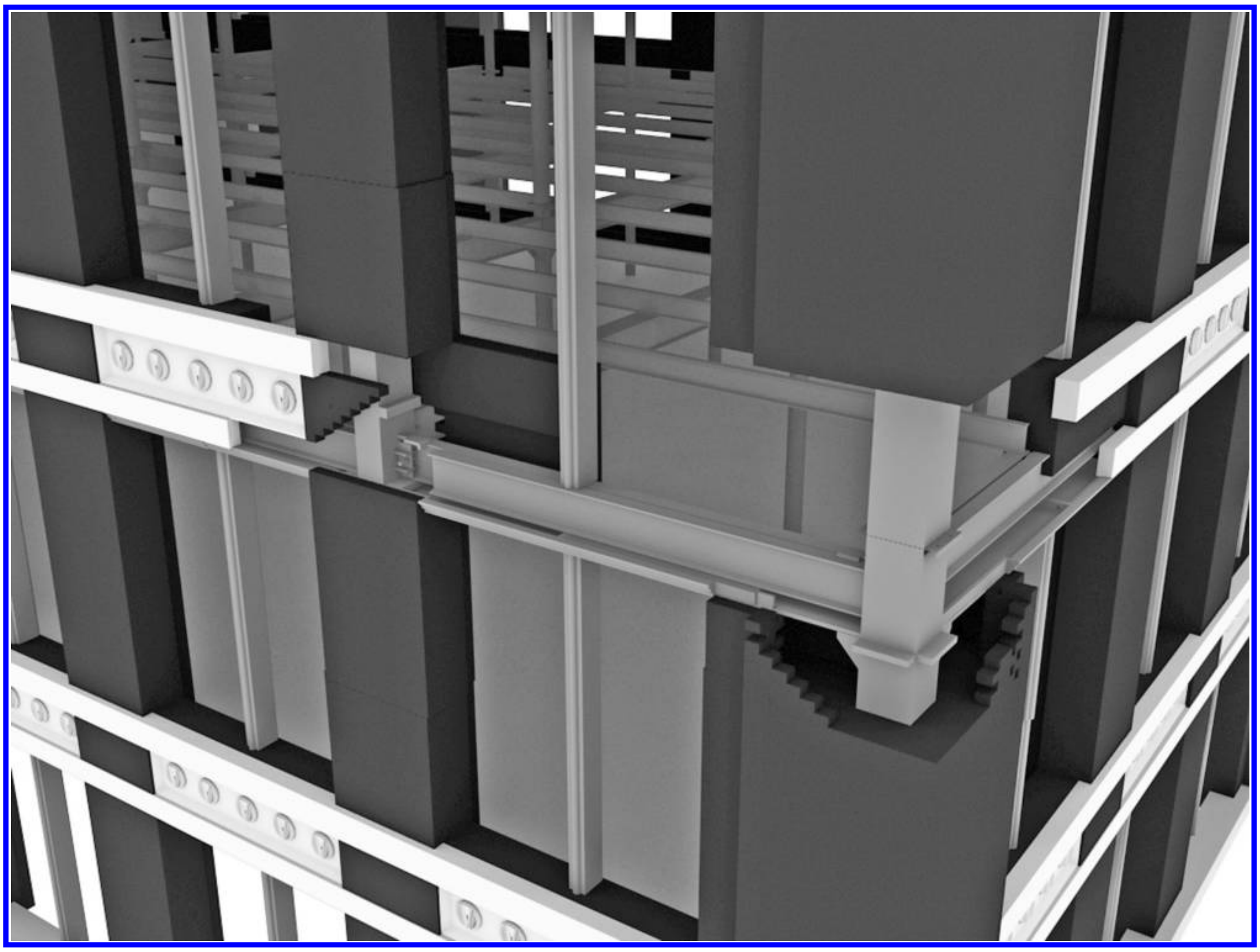

Figure 1 William LeBaron Jenney, Home Insurance Building, Chicago, 1885 (demolished 1931). View of digital reconstruction showing reinforced masonry walls (model and view by Ryan Risse). See *JSAH* online for zoomable model

Built Like Bridges

Iron, Steel, and Rivets in the Nineteenth-century Skyscraper

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> The wind pressure must be looked after. The floors are by the fire-proof arches made sufficiently rigid, but the columns may require bracing, either by knees as in naval architecture, or by X rods, as in bridge construction.
>
> —W. L. B. Jenney, 1891[1]

The decade between 1885 and 1895 saw two important structural developments that fostered greater efficiency, height, and stability in tall commercial structures: steel quickly replaced cast and wrought iron, which had been the materials of choice for columns and girders, respectively, and new systems of lateral bracing were developed that enabled tall metal frames to withstand wind loads (Figure 1). Skyscraper engineering entered this decade relying on heavy walls of masonry and on rules of thumb regarding wall thickness and building proportion to resist wind loads, but it emerged with the ability to design comparatively lightweight metal frames that absorbed and directed these loads on their own. The most visible result of these advances was greater height: the tallest buildings of the mid-1880s were ten to twelve stories, while those of the mid-1890s were over twenty. Perhaps as important to their investors' calculations, however, the plans and sections of these new structures were also clear of the thick masonry walls that had previously defined and constricted spaces.

Steel framing and wind bracing have been cited by most historians of the era as critical contributions to the development of the skyscraper. However, the circumstances in which steel replaced iron—at the very moment when wind bracing reached its most refined development—have been underexplored in the older, standard histories of Chicago architecture, and the role played by a particularly important fabricational advance—riveting—in linking steel with wind bracing has remained undeservedly obscure.[2]

More recent scholarship, notably Bill Addis's *Building: 3000 Years of Design, Engineering, and Construction* (2007) and Donald Friedman's *Historical Building Construction* (1995), makes tentative connections between the material qualities of steel and the structural performance these permitted.[3] Addis notes, for example, that cast iron presented difficulties in achieving stiff connections, that "portal framing" used rivets to join deep girders to wrought iron, as opposed to cast-iron, columns, and that the steel frame, by 1907, led to improvements to rigid connections and a method of modeling that enabled more accurate calculations.[4] But Addis does not draw the important conclusion that the material properties of these three metals determined what could be done in the shop, or on the job site, and thus how such connections could be made. That observation had appeared earlier in Donald Friedman's vital *Historical Building Construction* (1995), where it was noted that structural riveting offered

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unique opportunities to create stiff, wind-resistant connections, and that ductility—which wrought iron and steel possessed but which cast iron did not—was key to successful riveting.[5]

Friedman's analysis of the development of the riveted frame needs to be placed in the context of designers' concerns regarding wind loads. Several techniques were used to resist wind in early iron/masonry and in early steel structures, and the fabricational advances associated with riveted steel were key preconditions for the new column shapes, details, and rigid joints of self-bracing frames. Each of these developments contributed to the rapid replacement of cast and wrought iron by steel in skyscraper framing. An atmosphere of active research, application, and collaboration characterized this period of intense technological experimentation in tall building construction.

Cast Iron, Wrought Iron, Steel

As late as 1890, iron enjoyed significant advantages over steel in both reputation and cost. It had been used extensively in two forms—wrought and cast—since the mid-eighteenth century, when, as a product of the industrial revolution, it proved its merits in machinery and bridges and then found widespread use as a (more or less) fireproof material in mill construction of the 1790s and early 1800s. "Wrought" and "cast" referred to the methods of iron production, but also to chemical content. Cast iron was closer to raw pig iron in its high carbon content. It was a strong but brittle material that could not be easily worked 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—critically—an increase in ductility at relatively cool temperatures that meant it could be hammered or rolled into useful shapes.

Together, these two forms of iron predominated in most early tall building construction, from the 1851 Crystal Palace to early skyscrapers in New York and Chicago. Compared to masonry, wrought iron was lighter and capable of resisting tensile stress. These qualities made it valuable for both mill and office construction, as it vastly decreased the amount of floor and wall area consumed by structure. Its resistance to fire was not perfect, and architects developed methods of cloaking iron structures with terracotta or brick to protect it from the effects of high temperature, but this added expense was minor compared to the efficiencies it offered over more traditional construction. However, the air bubbles and internal stresses that resulted from the violent casting process limited the use of cast iron to compressive situations, where a flawed column would fail only to the extent of the imperfection, catching itself before a catastrophe occurred. Tensile or bending members were made of wrought iron, whose composition could be more definitively established and which was therefore more reliable in tension.

There was a nascent steel industry in the United States by 1860, as mechanical processes began to replace manual stirring to remove carbon from pig iron.[6] This was at best a minor industry, however, and it would have seemed unlikely at that time that steel could overtake iron as a commercially viable structural material. Steel was a specialty product; its carbon content was similar to wrought iron but more carefully balanced to achieve a combination of ductility and strength. Whereas wrought iron was made by pounding or floating out excess carbon, steel was produced by blasting carbon out of molten ore with air and often a limestone flux, and then adding the proper percentage of carbon back in carefully measured amounts. Steel required exceedingly precise refinement and careful control of ingredients to create its narrowly defined chemical composition. The result was a material that could be worked like wrought iron, but that was tough and strong like cast iron. These qualities were desirable for tools, weapons, and cutting implements (all traditional uses for steel), but had little apparent utility in building construction. Structural elements did not require sharp edges, and the ductility necessary for rolling structural shapes was a property of wrought iron, which was already widely used. The structural performance of steel was better than wrought iron, to be sure—between 10 and 30 percent, depending on the application. But this was not enough to inspire a wholesale move away from wrought iron, with its proven record and much lower price. And steel posed no immediate challenge to the use of cast iron for columns, whose statically ideal cylindrical shapes could, at the time, only be manufactured by casting, not by rolling.

Despite the apparent lack of incentives for the conversion, the extraordinary replacement of cast and wrought iron by steel took less than a decade, from the first publicized use of steel in building construction in the Home Insurance Building in Chicago in 1885 to *Engineering Record*'s definitive pronouncement in 1895 that cast iron "could not be recommended" for structural purposes.[7] What occurred in the intervening decade paired a gradual growth in the scientific understanding and testing of steel—leading to its acceptance as a reliable and calculable product—with the realization that its unique combination of strength and ductility allowed it to satisfy one of the great requirements of skyscraper construction—wind bracing—in ways that cast and wrought iron could not. Steel's unique combination of high strength and superior workability enabled engineers to design self-supporting metal frames that resisted gravity and lateral loading and that needed no assistance from masonry shear walls. Just as iron had eliminated the need to use structures made of brick to resist gravity, so steel eliminated the remaining need to employ masonry to brace buildings against the wind. Freed from massive masonry walls, steel frames could then fulfill the promise of metal to create light and open structures that occupied negligible floor and sectional space.

The Need for Bracing

Before the late nineteenth century, wind bracing had rarely been more than a minor consideration in structural calculations, because in heavy masonry buildings the dead weight of brick or stone construction absorbed all but the most severe lateral and overturning forces imposed by wind.[8] However, the lighter weight of skeletal buildings, their increased height, and the nature of steel and iron connections necessarily brought this issue to the fore. The designers of the tall buildings of the 1880s in Chicago were among the first to recognize this problem and to solve it with dedicated lateral or shear systems. The Home Insurance Building (William Le Baron Jenney, 1885), and the Rookery (Burnham and Root, 1888) both relied on masonry walls set at right angles to one another to stay themselves against wind. In these two examples, the shear walls doubled as the buildings' exterior skins, staging a competition between wall and window for space, but Holabird and Roche's Tacoma Building (1885) turned its shear walls inward, opening up its elevations at the expense of its internal flexibility (Figures 2, 3, see Figure 1).

The problems presented by wind in tall building construction were threefold. First, as buildings were built ever higher in proportion to their base, the overturning moment created by a gust of wind striking their sides increased dramatically. Buildings functioned as giant, vertical cantilevers, firmly anchored at the base, with a distributed load of wind over their entire surface. Taller buildings presented exponentially more difficult problems, as their increased area of exposed wall gathered wind load and increased the length of the lever arm by which wind could pry the building out of its foundations. Heavy masonry and hybrid masonry and iron buildings offered natural resistance to this prying action, as their windward exterior walls were far too heavy to be lifted by the wind's leverage. However, the lighter skins of the skeleton era no longer offered large-scale wind resistance through simple weight, and after Holabird and Roche's Tacoma Building, architects moved wind-bracing masonry walls inside, leaving the skins free from thick, light-blocking walls, but taking up valuable floor space.

While buildings without steel could resist the overturning effects of wind, the internal stresses induced by such resistance could be formidable, as these structures had to accept both wind-induced shear and bending throughout

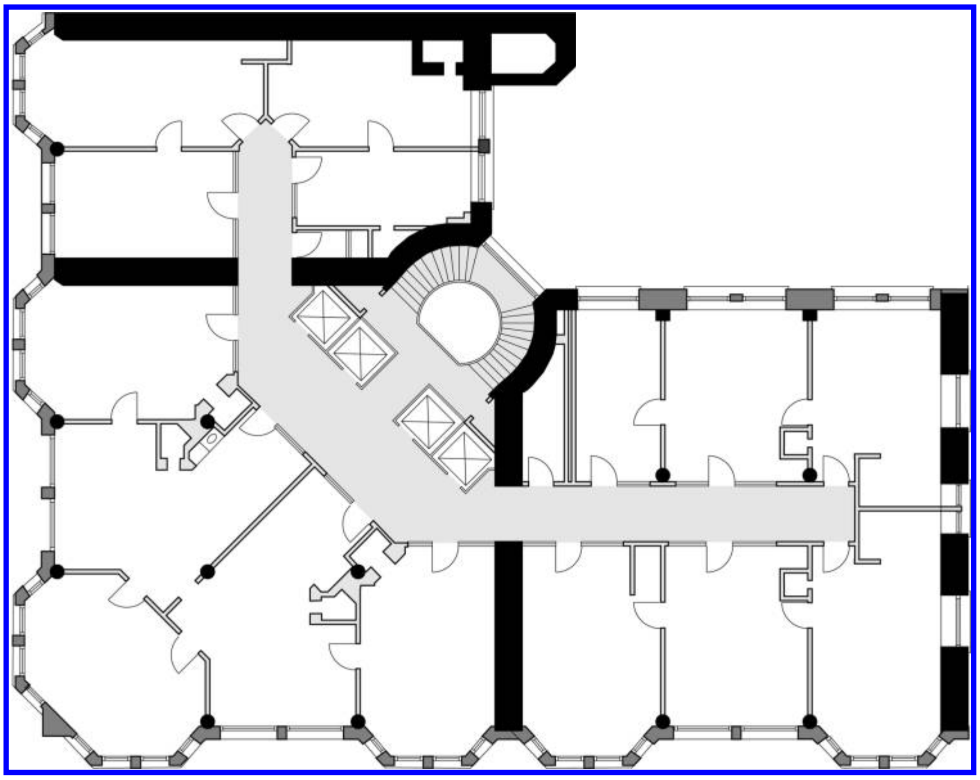

Figure 2 Holabird & Roche, Tacoma Building, Chicago, 1889 (demolished 1929). Plan of typical office floor. Drawing by the author, based on *Prominent Buildings Erected by the George A. Fuller Company* (New York: George A. Fuller Co., 1893), 26

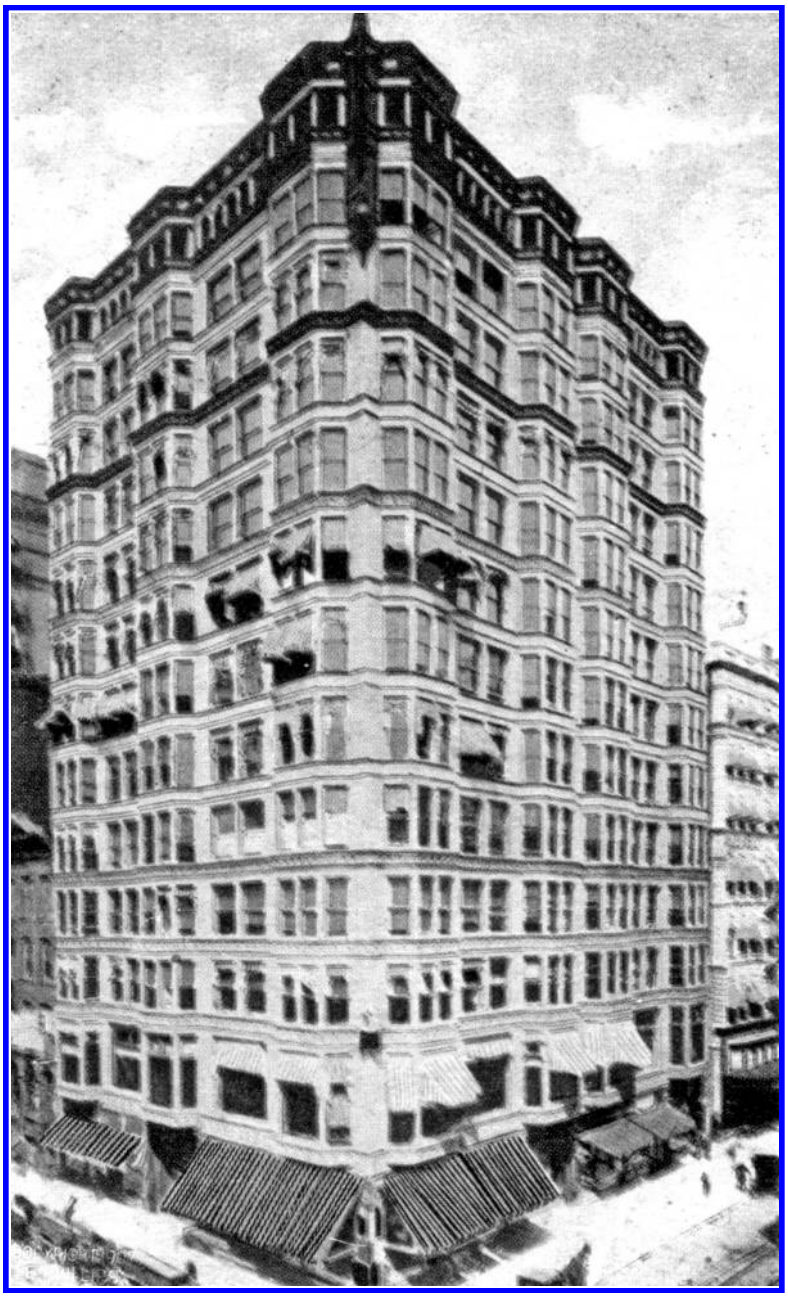

Figure 3 Tacoma Building, view from southwest. Commercial postcard, ca. 1892 (publisher unknown)

their frames. Shear occurred throughout columns, as the structure absorbed the horizontal load of the wind. If columns were too small, or if column splices were too weak, the wind could theoretically slide the mass of the building off its foundation, or literally shear off the building at a weak story.[9] This problem was compounded by the clients' understandable desire to open up ground level floors with large windows and doors and by the tendency to rent these stories to banks and shops that required large, open spaces uninterrupted by walls. Bending presented additional problems. In absorbing the internal 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 to the calculations required to engineer the frame to carry safely the weight of the building. Columns that bore the compressive effects of a sudden gust might well be pushed beyond their safe capacity by the considerable load they already carried from the floors above.

Most importantly, wind forces added unpredictable loads from unknowable directions to the connections between structural members. While large wind load could be absorbed by mass or by proper sizing of structural members, the design of safe column and beam connections involved

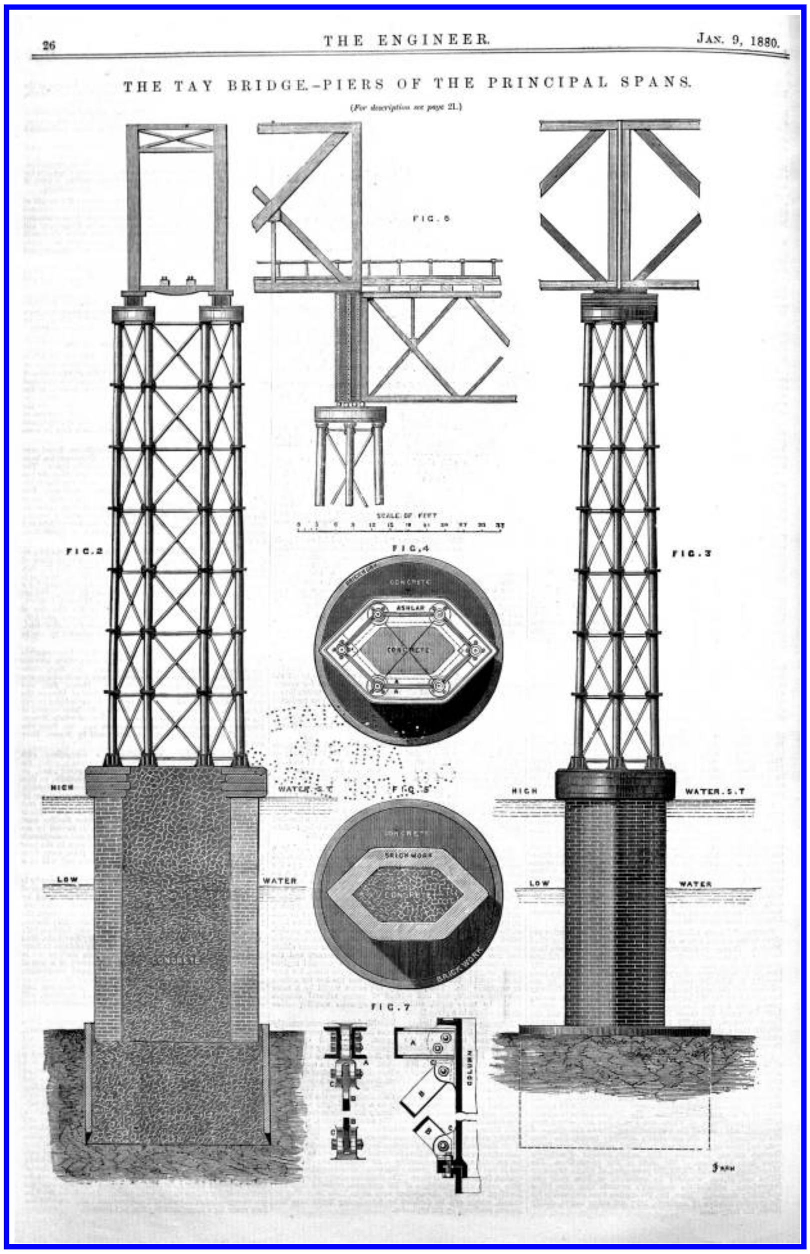

Figure 4 Thomas Bouch, engineer. Tay Bridge, Scotland, 1878 (collapsed 1879). Elevations, plan, and detail of typical pier (*The Engineer* 49, no. 2 [9 Jan. 1880], 26)

much more detailed analysis, complex math, and an understanding of load distribution and material behavior that did not exist in the 1880s. Concerns about the performance of connections had real implications. In December 1879, the Firth of Tay Bridge in Scotland collapsed in winds that were well within its claimed structural limits. A subsequent investigation proved that the bridge failed through a combination of poorly designed and manufactured connections. The geometry of the bridge's supports created huge tensile loads on its diagonal bracing members (Figure 4). These members were connected by bolts whose holes were found to be imperfectly cast and aligned. Over time, repeated loads stretched the brittle cast iron of the bridge so much that emergency shims had to be inserted to remove slackness from its frame. Excess motion caused by the stretched elements and loose connections created additional dynamic loads on fasteners, which ultimately failed and led to the structure's demise.[10] This disaster showed that connections needed to be made strong, and that they also needed to be made tight, since slackness could lead to unexpected and dangerous dynamic loads as the loose frame suddenly checked itself after being moved by wind. Over time, these dynamic loads progressively stretched the structural elements, having inevitably grave consequences.

These dangers led designers to provide stiffness by four basic methods: building mass, cross bracing, knee braces (or portal frames), and stiff connections. At the largest scale, the overall shape and section of buildings remained important, and rules of thumb told engineers and architects when the proportions of designs began to approach dangerous limits. Edward C. Shankland, who engineered Burnham's tallest buildings of the 1890s, suggested that a building's height could exceed its base by a factor of between four and six to one without requiring special frame design, but other experts suggested that the safe proportions were only three to one.[11]

Lest they be constrained by these primitive rules of thumb, engineers and architects sought ways to channel and resist lateral forces in the new lightweight frames. Masonry walls, which had been used in buildings such as the Tacoma to absorb loads from wind, reached their peak efficiency in relatively short buildings. In the late 1880s buildings had grown "so high that it does not appear possible that the masonry walls, after considering their own crushing weight, can have much efficiency left for the purposes of bracing the frame."[12] The thick masonry walls of the Monadnock (Burnham and Root, 1891) and the Woman's Temple (Burnham and Root, 1892) touched the limits of masonry bracing, with walls of cyclopean thickness at their bases that discouraged shop owners and that settled with alarming unevenness (Figure 5).

Figure 5 Burnham & Root, Woman's Christian Temperance Union ("Women's Temple"), Chicago, 1892 (demolished 1926). View from northwest (*One Hundred and Twenty-Five Photographic Views of Chicago* [Chicago: Rand-McNally, 1902], plate 9)

As masonry walls were reaching their practical limits, the metal frame, which was an efficient system for resisting gravity loads, was also being recognized as an efficient system for withstanding wind forces. Here the world of bridge engineering, where large iron and steel cantilevers were common, showed the way forward.[13] Railroad bridges employed trusses to absorb gravity loads, using triangular geometry to achieve cantilevers and single spans with far less weight than traditional masonry arch bridges. By taking bridge trusses and standing them on end, engineers had a valid model for designing against wind loads. Engineered trusses could be used in place of masonry walls to absorb the bending and shear of lateral loads, eliminating substantial weight.

While the principle of designing building frames as vertical bridges, braced against wind, made sense, the mechanics of wind loads were poorly understood, and engineers could not agree on the loads, precisely, for which they were to design. Wind effects had become noticeable in tall buildings, and a popular (though never proven) urban myth of the mid-1890s suggested that pendulum clocks on top stories occasionally ground to a halt because of the constant wind-induced sway.[14] Collapses of small buildings due to wind were not uncommon, although architecture fortunately suffered no wind-based disaster on the scale of the Tay Bridge to humble its engineers. Skepticism and nervousness about the performance of tall buildings nevertheless became an issue within the design professions. Engineers were uneasy when faced with the problem of wind, since there were few ways to measure it accurately or to understand its complex effects on buildings. Measurements of wind effects on buildings produced wildly divergent and surprising results—showing, for example, that wind deflected by skyscraper walls often had as much vertical as horizontal force, and that the eddies created by wind on the leeward sides of buildings created negative pressure that was capable of pulling windows off.[15] As early theories of wind loading and bracing took shape, the most puzzling aspect seemed to be the continued stability of buildings that were apparently constructed without regard to lateral stability. This robustness was eventually attributed to the minimal but widely spread stiffness of hollow tile partitions, but even after this contributing effect was identified, engineers were frustrated by their inability to calculate it accurately. The *Engineering Record* noted, "the construction of skeleton buildings has but recently been commenced hence but little opportunity has been afforded to test them in the face of ordinarily destructive winds. As a general rule, however, it may be considered very indifferent engineering that is fortified only by a lack of failure. If a construction is sound in principle it can be shown to be so, but if its character is not capable of a clear defense, it can only be regarded with well-grounded suspicion."[16]

Wind pressure did not lend itself to the sort of laboratory analysis to which steel and iron could be subjected. Instead, the profession had to rely on direct observation and a theoretical mechanism for turning this observation into the reliable calculation of design loads. Such an empirical approach necessarily entailed grave unknowns. The highest sustained wind speed in Chicago in the 1890s was recorded at Burnham and Shankland's private observatory, which noted a five-minute wind of eighty-five miles per hour.[17] But was this the maximum wind that Chicago might ever experience? To what extent were engineers and designers to assume the worst imaginable storm, and for what wind velocity should they design? As early as 1885 engineers possessed the mathematical tools to translate wind speed roughly into average pressure, and debates largely revolved around the pressure that conservative design should be able to withstand, given that only a few years of weather statistics were available. Velocities of 60 miles per hour translated to pressures of 18 pounds per square foot against the side of a building, while a wind of 84 miles per hour created pressures around 35 pounds per square foot. Studies in Scotland, on the Firth of Forth, noted that winds that produced 16.5 pounds per square foot also halted all marine traffic, but that local peak loads of 56 pounds per square foot—even though they were never sustained—were highly possible.[18] H. H. Quimby, who assembled the most comprehensive study of wind measurements and techniques for resistance in 1891–92, was so disturbed by the potential for disaster that he publicly recommended a horizontal design load of 40 pounds per square foot for all buildings. Other engineers (including Shankland), less conservative and concerned with the implications of this figure for the weight of their structures, argued for design loads of 30 or 35 pounds per square foot.[19] Both Chicago and New York incorporated the lower figure into their building codes in the 1890s, while the Boston and Philadelphia codes failed to include any reference to wind loads at that time. The continued survival of tall structures in all four cities suggests that the lower figure was not unsafe, and that the inherent stiffness of internal construction must have provided more wind resistance than was thought.[20]

Quimby, undeterred, noted that proper wind bracing was insurance against storms of unforeseen ferocity, and his view—if not his suggested loading value—was widely shared among the professions through the 1890s. Wind bracing became an important part of structural frames as a matter of course in the boom of 1890–91, and it took three different forms. Each system relied on metal rather than masonry, eliminating weight. Each allowed plans and façades that were more open than the masonry systems of the previous decade. Each also depended upon increasingly precise standards in manufacture, since the Tay Bridge disaster had pointed out that slackness in structural connections due to imperfect geometries or alignments could lead to failure through repeated dynamic loading. These three frame-based wind-bracing schemes added members or connections to make building frames act as cantilevered, vertical trusses. In order of increasing complexity, the systems used rod- or sway-bracing, knee braces, and portal frames (Figure 6). More spatially efficient systems, in particular lattice or plate girders, improved on these initially popular techniques by the mid-1890s.[21]

Of these structural systems, the most similar to actual bridge construction 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 and girder. The resulting cross bracing triangulated each panel, providing a shape that could resist loading through its geometry. Any lateral load on the vertical truss would be unable to change the shape of these triangular panels without stretching the metal tension rods, and the extraordinary tensile strength of steel could thus be directly deployed against lateral loads. 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. Buildings braced by sway-rods typically had two or more dedicated vertical planes on which, at every level, these rods would connect to columns and girders. These planes of metal bracing took the place of large masonry shear walls, but sway-bracing occupied a plan width of a few inches at most, while masonry shear walls required a foot or more of material to be effective. With proper planning, these diagonal braces could be built into walls; however their geometry restricted or prevented the placement of doors, windows, or other openings in these panels, unless the rods were extended over two stories instead of one or offset using a secondary set of structural elements. Sway-rods were typically the most economical solution to wind bracing, as well as being

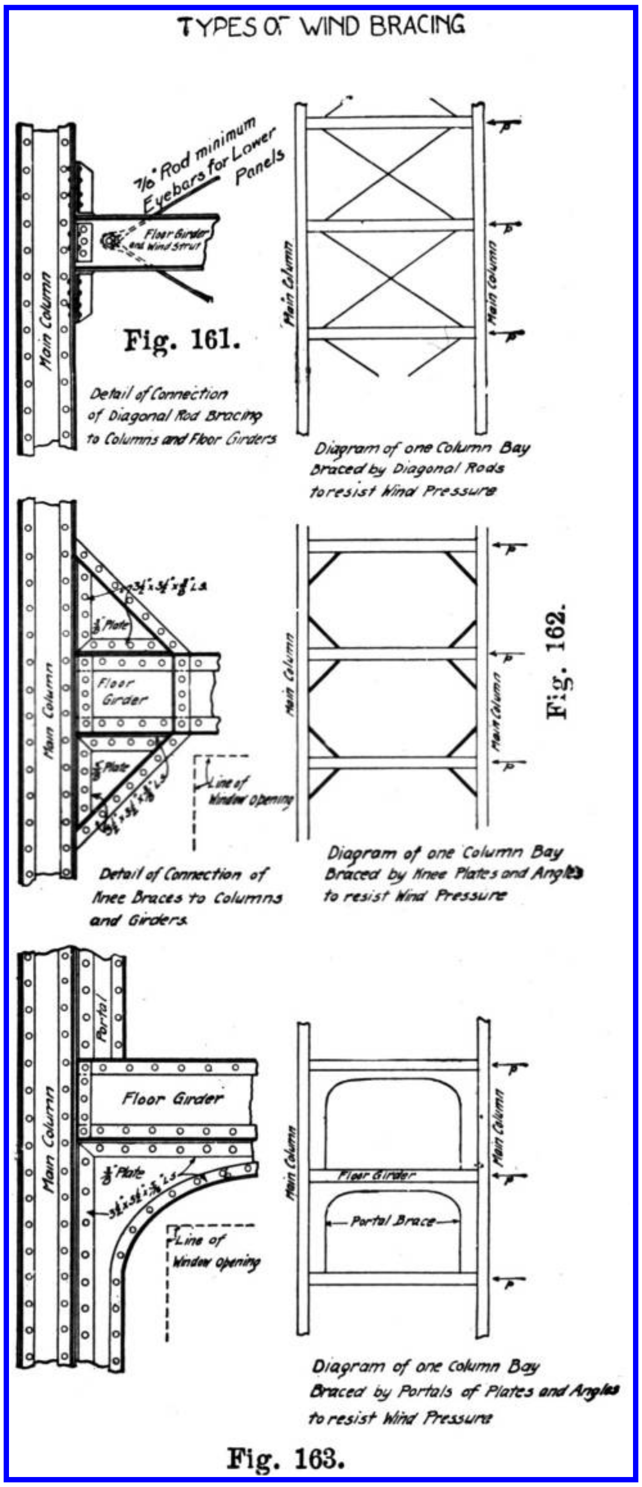

Figure 6 Three major types of wind-bracing details: cross or sway-bracing (top), knee braces (middle), and portal frames (bottom) (delineator unknown, from James C. Plant et al., *Cyclopedia of Architecture, Carpentry and Building* [Chicago: American Technical Society, 1907], 173)

Figure 7 Burnham & Root, Masonic Temple (later Capitol Building), Chicago, 1892 (demolished 1939). View from southwest. (from *One Hundred and Twenty-Five Photographic Views of Chicago* [Chicago: Rand-McNally, 1913], plate 79)

the lightest, but the problem of planning doorways and openings—particularly at ground level where open space was at a premium—often obviated their use.[22]

Sway-rod wind bracing was adopted for two well-known Chicago buildings of 1891–92: the thirteen-story Venetian, designed by Holabird & Roche, and the more widely recognized twenty-one-story Masonic Temple by Burnham and Root (Figures 7, 8). Both buildings had narrow, rectangular floor plans whose proportions offered enough footprint to resist wind loading in one direction, but they required additional bracing parallel to their shorter axes. The Venetian employed a compromise system that relied primarily on diagonal bracing concealed within four continuous partition walls. These were staggered through the floor plan to allow greater flexibility in planning, and the connections

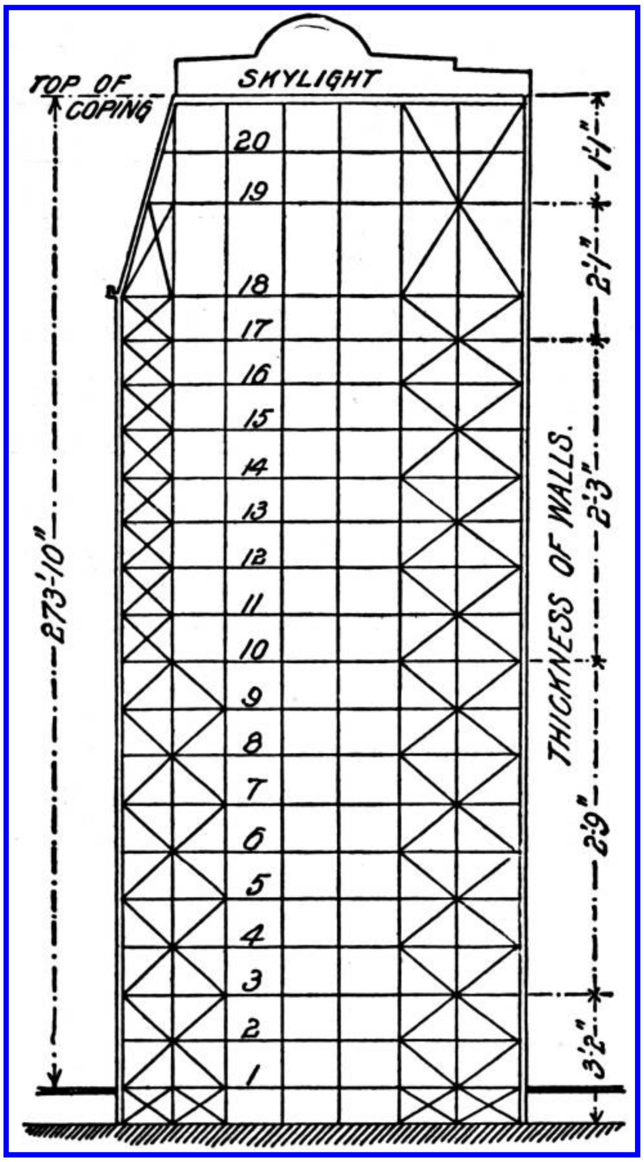

Figure 8 Masonic Temple, section showing configuration of wind-bracing rods (delineator unknown, from J. K. Freitag, *Architectural Engineering*, rev. ed. [New York: John Wiley & Sons, 1904], 264)

between sway-rods were shifted to a separate system of struts beneath each floor, enabling more convenient door placement. A further compromise was made at the second floor, where the bracing was interrupted in favor of a portal frame system (discussed below) that spanned the entire width of a banking hall. Such structural gymnastics proved the possible flexibility of such bracing, but at considerable material cost.[23]

The Masonic Temple was the ultimate example of the potential of diagonal sway-rods, using single-, double-, and triple-story panels to brace its record-breaking 273-foot height along two partition lines that were skillfully integrated into its floor plans. Like the Venetian, the Masonic Temple's sway-rods, "similar to those in an iron pier of a railway viaduct" according to Edward Shankland, were connected to a secondary system of horizontal struts located beneath the floors that allowed doors to be placed nearer to column lines.[24]

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 triangulating a whole panel with tie rods. This strategy was 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 allowed immensely greater flexibility in floor plans, since there was no need to sacrifice whole panels to structure. However, in section these braces took up headroom near the columns. Architectural solutions to this difficult structural imposition included coved ceilings, corridors placed away from these restrictions, and large column heads that concealed the short diagonal braces. In narrow buildings, such braces could take up significant sectional space.

Portal frames created arched and triangulated structural shapes in which the distinction between girder and column was practically lost. Buildings with portal frames 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 knee-braced and portal-framed buildings were constructed, the consensus reached by the mid-1890s was that the weight of metal they required, coupled with the need for exacting fabrication and erection, made them uneconomical, and their use faded for towers of typical height.[25]

The best-known example of portal framing was Holabird & Roche's Old Colony Building (1893), constructed on a narrow block between Dearborn and Plymouth Place in Chicago (Figures 9, 10). Like the Venetian and the Masonic Temple, its floor plan was a narrow rectangle, offering a deep enough footprint in the north-south direction to overcome wind loading but requiring added strength in its shorter, east-west direction because of its height, 212 feet.[26] While the building's engineer, Corydon Purdy, originally specified a system of offset tie rods, this was changed late in the design process due to a dispute with the steel supplier. Wrought-iron columns were substituted for steel, and Purdy was forced to change the Old Colony's structural design.[27] The result was a system of paired iron arches, elliptical in shape, that engaged beams and columns along their full lengths and that provided deep iron webs that reinforced the structural joints between framing members (Figure 11).[28] These arches

Figure 9 Holabird and Roche, Old Colony Building, Chicago, 1894. Commercial postcard, ca. 1895, view from northwest (A. C. Bosselman & Co., New York)

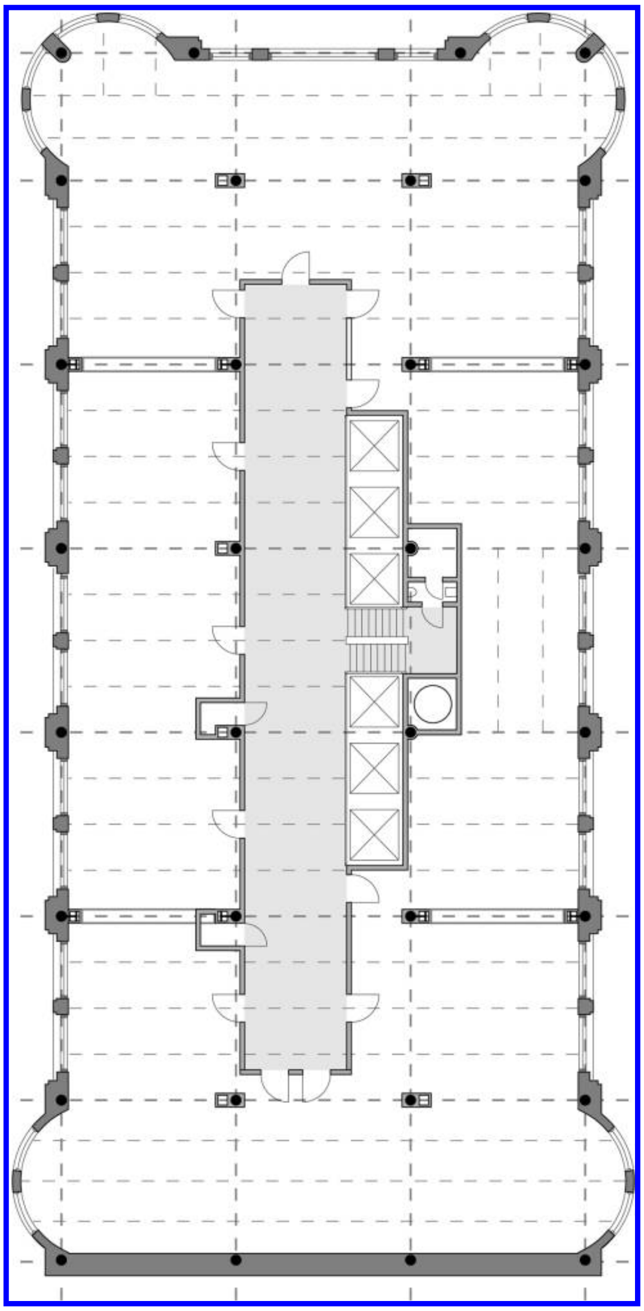

Figure 10 Old Colony Building, typical floor plan showing location of portal frames (double lines). (Drawing by the author based on *Prominent Buildings Erected by the George A. Fuller Company* [New York: George A. Fuller Co., 1893], 23; and William H. Birkmire, *Skeleton Construction in Buildings* [rpt. New York: Arno Press, 1972], 189)

provided needed stiffness in the building's short direction, but they were heavy, and they reduced ceiling height considerably, especially in the corners. Holabird and Roche designed coved plaster ceilings in the affected offices, but the wasted volume, extra weight, and expense of the Old Colony's portal system reflected the drawbacks of this approach.[29] Jenny used shorter braces, which he described as being similar to those used in ship construction, in the Isabella Building in Chicago (1892), but even this revised system interfered with ceiling heights where the knee brace attached to the column.[30]

Such spatial conflicts were eliminated by lattice girders, 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 terracotta fireproofing and floor arches provided reasonably rigid diaphragms in all three directions.[31] While no engineer was willing to rely entirely on these partitions for lateral stiffness, Edward Shankland (among others) realized that if the building frame itself could be made stiffer at each major junction, it could on its own develop reliable resistance to wind load. Essentially, Shankland proposed that the stiffening function of a

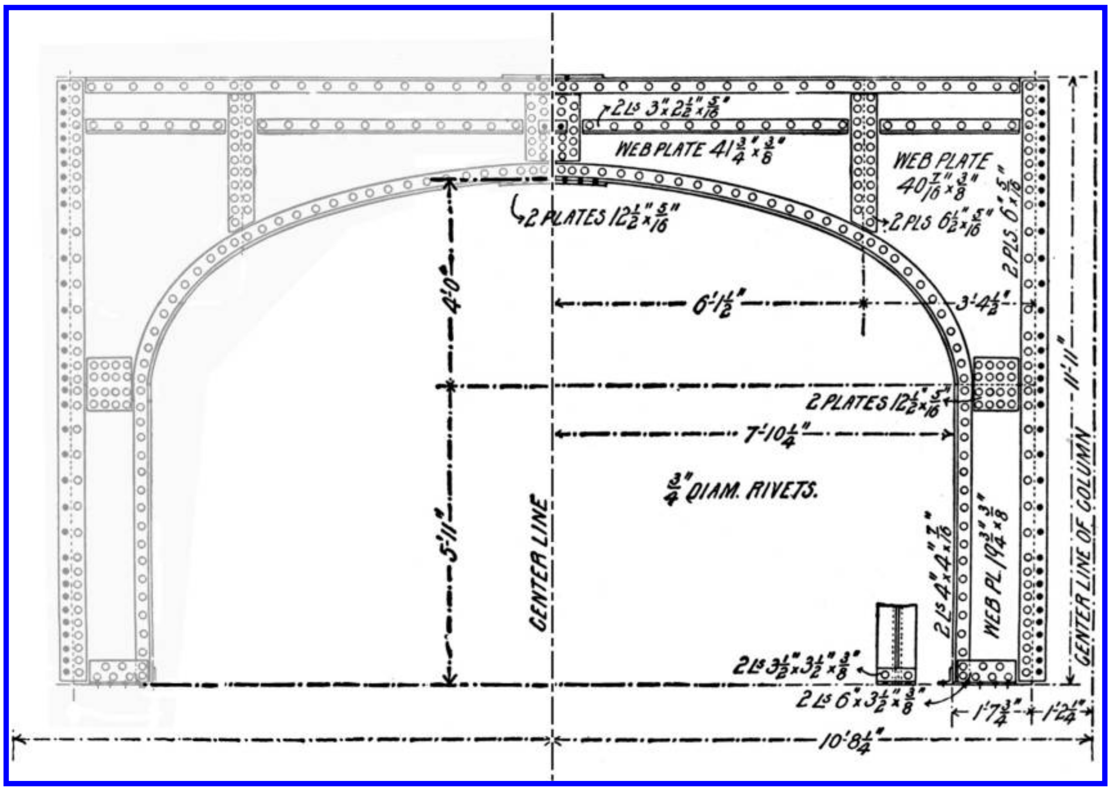

Figure 11 Old Colony Building, detail of typical portal frame (digital mirroring by the author) (delineator unknown, from J. K. Freitag, *Architectural Engineering* [rev. ed., New York: John Wiley & Sons, 1904], 272)

knee brace might, with the right materials, be accomplished within the joints between columns and beams themselves. Such an approach walked an engineering tightrope, since 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 substantial additional members. But 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.

Such a system was typically called a plate or lattice girder, but this was something of a misnomer in that it relied on columns as well (Figure 12). The girder in question was made intentionally deep—usually 24 inches or more, greater than the depth needed to span typical bays. Likewise, columns in this system were oversized to help absorb bending, and special column shapes were produced with wide flanges on all four sides. These flanges could be connected directly to the webs of the deeper girders to form a very rigid joint. As these multiplied throughout the frame, they worked collectively to resist any deformation due to wind. Instead of relying on individual truss panels, the entire frame could absorb and direct

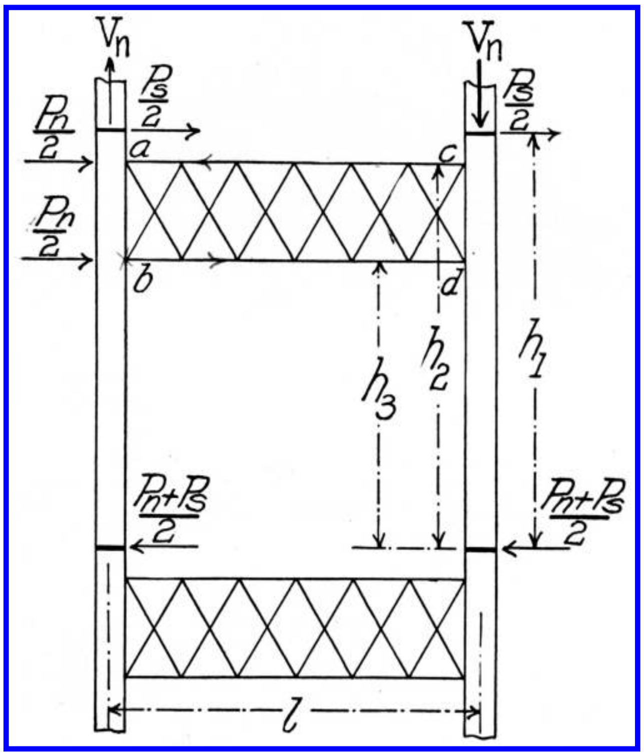

Figure 12 "Figure Showing Analysis of Lattice-girder Bracing" (from J. K. Freitag, *Architectural Engineering*, 276)

wind loads by providing multiple paths to channel lateral loads to the foundation. Shankland termed this the table leg principle, drawing an analogy between stiff, oversized carpentry joints 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 in later generations.[32] Each 'table' in plate girder buildings was rigidly connected in three dimensions to adjoining tables above and below, and while the result was a distribution of lateral stresses too complex to compute mathematically, the redundancy of load paths and the sharing of loads across and through the building frame left floor plans and sections entirely unencumbered by lateral bracing. Girders, columns, floors, and partitions were 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 would only come into use in tall building construction by the mid-1890s, coincidentally at the moment when the curtain wall reached its apogee in Chicago construction. While the full potential of this system was denied to the skyscrapers of the boom years of 1890–94, the earlier types—swayrods, knee braces, and portal frames—were common during this era.[33] Quimby and others, however, recognized the advantages of the stiff frame, and there is evidence that stiffer connections between beams and columns played significant roles in skyscraper engineering—intentionally or not—well before pure lattice girders became common. Quimby noted in 1892 the desirability of designing columns that could absorb shear and bending independent of a building's partitions:

> The stability of the individual columns in a framed structure is an element of resistance of considerable value if the connections are rigid . . . much ultimate resistance can be counted on from the bracketed fastenings and dead load, the ratio of base to height of each column being commonly about 12, but because of imperfect workmanship referred to above, they may at first act with, instead of against, the destroying force, and their resistance be developed only after that of the partitions is overcome or impaired. Wherever adequate rod bracing is not employed, the columns should be joined together by complete splices, making each column a unit throughout the whole height of the building and then failure could only follow the bending or breaking of the body of it at two points.[34]

In response to Quimby's paper, engineer J. P. Snow suggested the plate girder approach, which would later be perfected by Shankland. "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."[35] Stiffness in the building frame itself was therefore seen as an important part of wind resistance, even though most buildings of the early 1890s employed the additional insurance of swayrods or braces. More spatially efficient solutions required material performance that could not be achieved by iron alone.

Cast Iron, Steel, Bolts, and Rivets: The Quest for Stiffness

As Quimby noted, achieving stiffness within the skeletal metal frame was difficult, requiring careful attention to design, fabrication, and assembly. To ensure that girders and columns could act together as a stiff frame, the 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 rolled 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 that could be connected to one another. The development of structural steel allowed solutions that were not available in cast iron and enabled the construction of far taller skyscrapers than had been achievable with earlier materials.

Cast-iron construction was inherently limited by its brittle nature. In addition to its deadly lack of robustness in fire, cast-iron possessed neither the ductility that would allow drilling, nor could iron members be fabricated with sufficient accuracy to allow precision bolting or riveting on site. Once out of the mold, columns could not be altered, and were often slightly out of plumb, dimensionally inaccurate, or slightly twisted by 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 often made with loose-fitting pintles (pins) and gudgeons (cast holes) that required bracing systems to stand upright against lateral forces.[36] These loosely pinned connections created problems that were exacerbated by imperfections in the columns. Even in the best cast-iron connections, columns sat atop one another directly, requiring that top and bottom surfaces be planed accurately. If connecting faces were even slightly out of plumb, the bearing surfaces between them could be reduced to a very slender

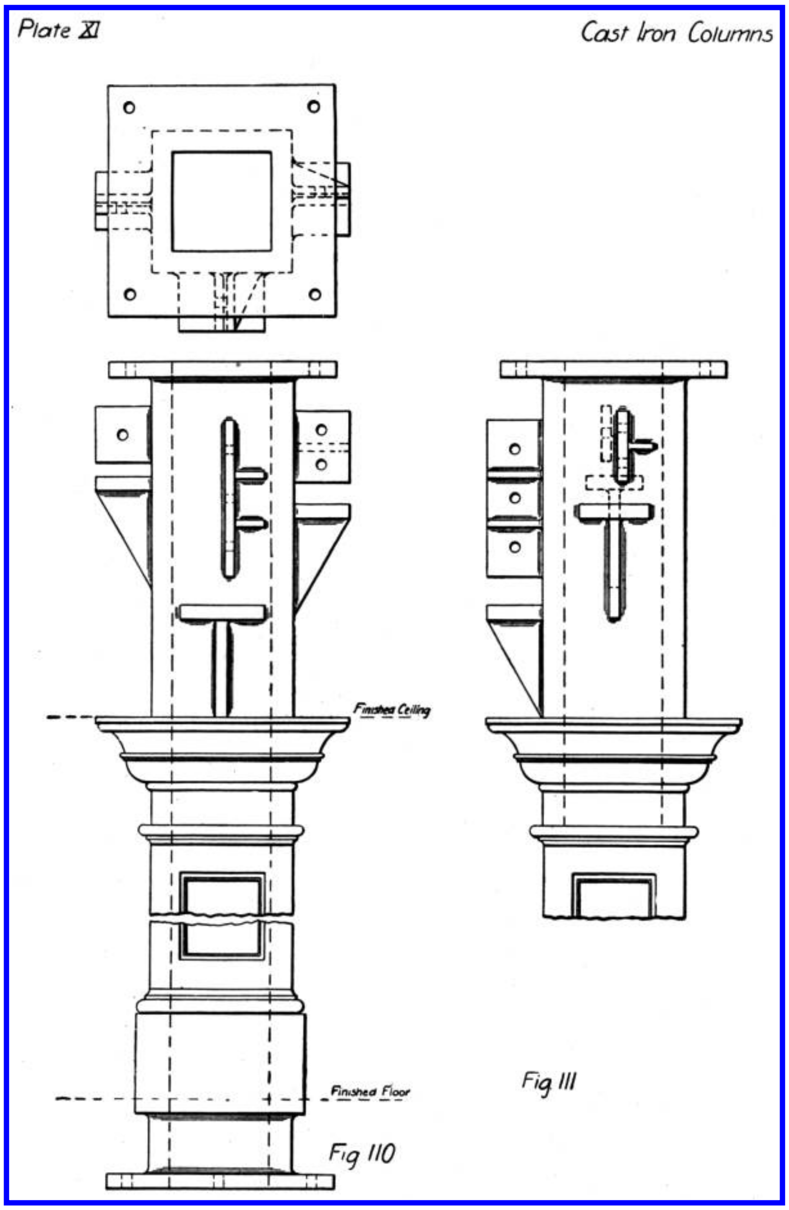

Figure 13 Typical cast-iron column connections, including lugs, shelves, and bolt-holes (delineator unknown, from James C. Plant et al., *Cyclopedia of Architecture, Carpentry and Building* [Chicago: American School of Correspondence, 1907], 123)

edge, stressing the iron beyond its capacity.[37] For connections between columns and girders, the situation was just as dire. Lugs or shelves cast into the column were the most effective method of transferring girder loads into the column, but there was no good way to make a perfect connection. Boltholes had to be cast significantly larger than the bolts to allow placement, which allowed significant movement between structural elements (Figure 13). Oversized boltholes, in fact, were listed as a major contributing factor to the Tay Bridge's collapse (see Figure 4, lower center: detail of column to strut/tie connection).[38]

Boltholes could be molded or carefully bored into cast iron elements to provide more reliable connections, but this technique offered its own problems. The brittle nature of cast iron 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 connected members to slip. Even a very small amount of motion, as was seen in the Tay Bridge disaster, could be multiplied by repeated dynamic loading.[39] Or, just as critically, the bolt might bear against only a portion of the metal at the edge of its hole, transferring a full load to only a fraction of the cross-sectional area designed for it.[40] As a result, "drilled holes and turned bolts" were according to the *Engineering News*, "scarcely feasible" in cast-iron construction.[41]

Another means of connection in the field, riveting, was a potentially tantalizing solution, but one that was obviated by cast iron's stubborn brittleness. Riveting entails heating metal plugs to the point of soft pliability, inserting them into pre-drilled holes in two metal plates, and then hammering both ends of the plug flat (or with a slight dome). This fills the hole completely with hot metal and, once cool, the two pieces are held 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. He noted that the shape of the rivet hole was crucial to the rivet's performance, and suggested punching the holes, rather than drilling, to eliminate sharp edges that could shear rivets after repeated loading.[42] He also noted the complex behavior of joints with multiple rivets, which had to be designed against three modes of failure—some of them complex in their mechanics:

> a riveted joint may give way either: (1) by the tearing of the plates from the rivet-hole to the edge of the plates; (2) by the tearing of the plates from rivet-hole to rivet-hole; (3) by the shearing of the rivet. When the plate gives way by tearing from the rivet-hole to the edge of the plate, a bending stress is induced in the part of the plate in front of the rivet. . . . When the plate gives way by tearing from rivet-hole to rivet-hole, it is commonly assumed that the stress on the part of the plate between the rivets is a uniformly distributed stress. This is shown to be not strictly correct, and the want of uniformity of stress will cause the plate to give way with a lower average intensity of stress than that which corresponds to the ultimate resistance of the plate to tension.[43]

Such failures were common to bolted joints as well. The major advantage of riveting over bolting 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; as the rivet cooled, it also shrank, tightening elements to one another.[44] Properly done, a riveted connection offered remarkable stiffness and reliability. It also offered significant speed, as riveting gangs using machine-powered tools could drive a single rivet in less than four seconds.[45] But given the brittle nature of cast iron, the repeated hammering of rivets would have catastrophic effects. 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 manufacturing produced relatively thin planes that could be punched easily and with much greater accuracy than iron. Bolts or rivets could be used to secure steel members with some confidence. Even greater accuracy could be achieved by reaming, or precisely widening punched holes by redrilling them with a slightly larger bit in the shop or field. Without redrilling, punching left slight funnel-shaped holes in steel, with a difference in the diameter of the hole of from 2/3 to 3/4 the material's thickness. Such tapered holes posed the same problems as inaccurately molded cast-iron holes, in that their sharp edges could slice through rivets. Engineers therefore typically specified slightly undersized holes in steel members. Once in place, temporary bolts would hold members together, the undersized rivet holes would be re-drilled to a consistent, straight profile, and the riveting gang would then begin work.[46] This produced a reliable, robust connection. The perfect alignment of the holes guaranteed that each rivet would absorb a predictable percentage of the total load, and that the entire cross section of each rivet would be recruited into resisting the load. Steel offered an additional advantage, in that relatively thin, reliably dimensioned steel plates and angles offered readily available locations for making simple riveted connections in the field. The lugs and shelves used in cast-iron construction were replaced by separately fabricated steel connectors, which could be pre-punched and reamed in the factory or in the field to ensure a tight fit (Figure 14).[47]

As early as 1891, riveted connections, using drilled and reamed holes, had become standard in steel building structures: Contractor George Fuller noted that this technique made structures "more solid," while Jenney praised the technique's scientific basis:

> The columns [in Chicago construction] were at first of cast iron with ingenious devices to tie the beams rigidly to the columns. As soon as riveted steel columns of a proper quality could be manufactured, their superior advantages at once brought them into use, which has now become general. All column connections are now made with hot rivets. The metal for the work is all tested, and the workmanship inspected at the mills by professional inspectors. The same science, and the same superintendence is required in calculating and erecting one of these high buildings as in a steel railroad bridge of the first order.[48]

The considerable superiority of riveting was sufficient for the *Engineering News* to declare in 1897 that cast iron, which could not be riveted, was no longer a suitable material, that in fact it had not been one for some time, and that Chicago had led the way in this assessment:

> It is, moreover, strictly true that the best class of structural practice is not feasible with cast-iron members. Riveted joints between girders and columns or brackets and columns are in the highest degree essential for good structural work, since drilled holes and turned bolts are scarcely feasible; but such riveting cannot be done, as the cast-iron taking the rivets would

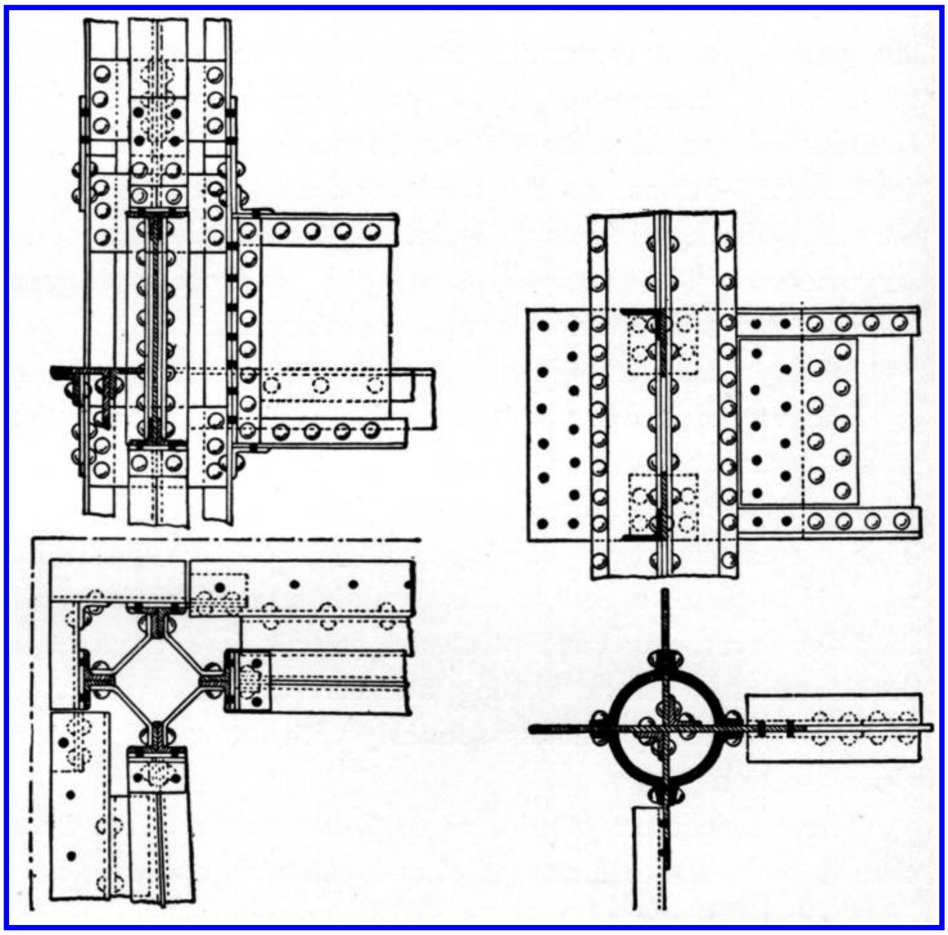

Figure 14 "Detail of Gray Column and Connecting Girders" and "Detail of Phoenix Column" showing riveted steel connections (delineator unknown, from J. K. Freitag, *Architectural Engineering* [rev. ed., New York: John Wiley & Sons, 1904], 215)

frequently or usually be broken in the operation. Hence it may be broadly stated that in consequence of the brittle, uncertain and treacherous character of the metal as well as the kind of stress which must be resisted by a column, it is usually not feasible to make a satisfactory structural design in which cast-iron columns are employed. The architects of Chicago seem to have reached practically that conclusion, and those of New York City now have had sufficient experience, one would suppose, to satisfy them of the wisdom of it.[49]

In addition to its reliability, riveting was affordable and rapid. By 1904, the average riveting gang of five (one tending a small furnace, two to toss and catch the hot rivets, and two manning the riveting hammer) could fix over 200 rivets in a nine-hour day, with an average cost per rivet of under ten cents.[50]

Problems with Cast-Iron Columns

As reliable as riveted connections became, they were only as secure as the members they joined, and intense experimentation and innovation was needed to create efficient, reliable columns in skyscrapers during the 1890s. This was matched by applied research into riveted connections. Columns were essential to providing stiffness against wind in all types of wind-resistant frames, but the development of reliable vertical members played a particularly important role in the development of the steel moment frame in mid-decade.

By 1890 considerable theoretical effort had gone in to understanding column behavior, particularly their hybrid performance when stressed both axially, by gravity, and laterally, by wind. William Burr, in his 1888 book *Elasticity and Resistance of Materials*, codified basic column theory by analyzing the column's bending behavior as though it were a beam that might be loaded in any direction at any time. While the material in a metal beam had to be concentrated at its top and bottom edges to provide a resisting lever against bending loads, a metal column had to be shaped to resist bending in all directions—or at least in as many directions as possible. This theory favored hollow round shapes, which placed all of their material at their perimeter. However, closed, hollow columns presented unique fabricational and constructional problems. And most importantly, they offered no good opportunities for bolted or riveted connections,

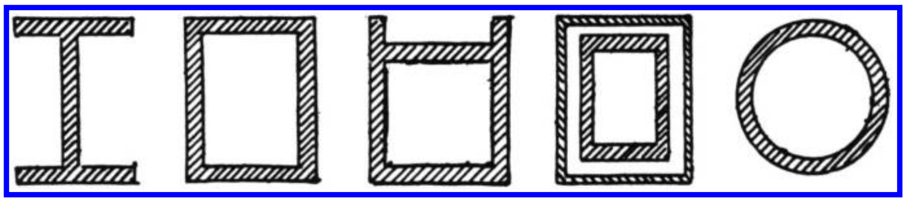

Figure 15 Typical cast-iron column sections (delineator unknown, from William H. Birkmire, *Skeleton Construction in Buildings* [reprint, New York: Arno Press, 1972], 21)

since it was impossible to access the interior to fix bolts or to hammer the backsides of rivets. The interiors of round, hollow sections were also inaccessible for inspection. Nor was every foundry equipped to cast or roll hollow shapes with precision, and fireproofing these shapes was difficult.

Most directly, however, the use of hollow columns could cause significant eccentric loading. Column theory assumed a consistent application of loading across the column's section. Under ordinary circumstances—for example, when a beam rested atop an evenly planed column top—this was a reasonable assumption. However, beams that were attached to the side of a hollow shape would transfer their load to the column section asymmetrically. This was particularly a problem for columns at building perimeters, where girders would engage columns on only the inside face. The resulting load on a column resulted in both compression, as the material of the column sought to bear the actual gravity load of the connected girders and floors, and bending, since these loads would occur at the edge of hollow sections, applying an eccentric load and a twisting force to the entire element. Even if the connection did not transmit bending moment to the column from the girder, a load on a column's edge instead of its center would impart a bending load to the column itself, which would interact with its compressive, bearing capacity in complicated ways.[51] There were few suitable mathematical models for such stresses, and the unpredictable synergetic effects of these bending loads with the gigantic compressive forces being placed on columns created incalculable static conditions. Engineers were advised, simply, to find ways to bring girder loads to the centerlines of column sections as efficiently as possible.[52] This advice erased the theoretical advantages of hollow shapes, and the struggle to reconcile ideal performance with the need to minimize eccentric loading constituted the primary narrative of steel column design for a generation.

Cast-iron columns were available through the late 1890s in four configurations: hollow cylinders, hollow rectangles, cruciforms, and H shapes (Figure 15). While hollow castings made more efficient use of material, the crosses and H sections offered better opportunities to transmit loads directly to the column's center. H shapes also had flanges and webs that were easily accessible on both sides, and boltholes, lugs, and shelves could be cast in to them. Hollow section columns could not be inspected, and unseen variations in the thickness of the shapes' walls could dramatically reduce their capacity.[53]

The lack of consistency in cast iron occurred in all column shapes. There was no way of knowing whether even a thin-edged member concealed trapped air or impurities that could create a fatal flaw.[54] A worrying record of failure, in fire or simply in daily service, dogged the use of cast iron in the 1890s, and as steel became affordable engineers began to condemn the use of cast iron in structural applications, although building codes allowed it well past 1900. The collapse of a railway bridge in Eibenshitz, Austria, in 1894 struck a major blow against cast iron because it was determined that differential expansion due to simple temperature changes had caused fatal cracks in its piers.[55] Already on record as opposing cast iron, the *Engineering News* employed increasingly agitated language in its campaign against the continued use of the unreliable material after the 1897 failure of the Ireland Building in New York:

> It is not a question with any new features; even this latest collapse reveals absolutely nothing new. Essentially everything that has happened structurally is completely consistent with what experienced and competent civil engineers would have predicted as extremely likely to happen, for the simple reason that both engineering theory and engineering practice show that it should have happened. Nevertheless, so many well-intentioned people, particularly in New York City, apparently place their faith in these treacherous cast-iron members and their use affects such large interests that it is advisable to restate and consider again various things which are tritely familiar to some and should be so to all.[56]

Testing in the mid-1890s likewise revealed that cast-iron's supposed fire-resistant qualities were also less than conventional wisdom had assumed.

Steel Columns

Writing in 1896, William Le Baron Jenney argued that the switch from cast-iron to steel columns had been the most crucial development in the realization of the tall metal frame:

> Since the Home Insurance Building, the most important improvement that has been made in this class of construction,

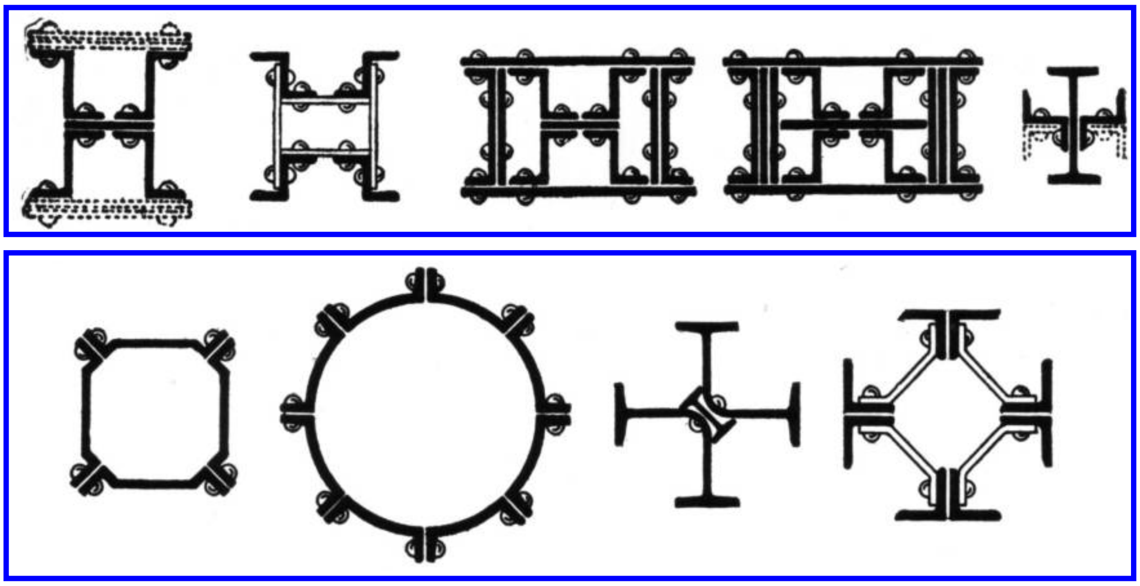

Figure 16 "Typical Forms of Z-bar Columns" and "Special Forms of Steel Columns," including the Larimer (bottom, 3) and the Gray (bottom, 4) (delineator unknown, from J. K. Freitag, *Architectural Engineering* [rev. ed., New York: John Wiley & Sons, 1904], 198–99)

> now generally known as the Chicago construction or the steel-skeleton construction, was the introduction of steel-riveted columns, which are now made cheaply and in all respects thoroughly satisfactory. All the assembling at the building is done with hot steel rivets; increased rigidity is secured, as well as a material reduction of the weight of the columns. Steel-riveted columns as now manufactured are considered perfectly safe with a coefficient of safety of 4, while for cast-iron columns a coefficient of safety of 8 is not considered other than reasonably safe.[57]

The advent of steel columns allowed not only rigid, moment-transferring connections between girders and columns, it allowed columns that were substantially more rigid and reliable than their cast-iron forebears.[58] Inaccuracies and brittleness had prevented tight connections in cast-iron columns. Steel's ductility, workability, and reliable strength permitted columns whose shapes were better able to balance ideal static geometry with ease of fabrication and assembly. First, the tighter quality control to which the material was susceptible permitted much greater confidence in its performance and allowed smaller factors of safety. Second, steel rolling processes produced more consistent products than casting, as air bubbles were pressed out of the soft material, and impurities tended to be widely distributed, rather than concentrated, by the constant kneading of the hot steel. Third, steel could be rolled to precise, thin dimensions, which allowed easier bolting and riveting.

Most important was the fact that steel could be riveted, since as in girder connections, stiffness in column splices was vital to the performance of a building frame in wind loading conditions. Rolled sections also permitted longer column lengths than could be procured by casting, and so a critical area of looseness in building frames could be eliminated by making columns continuous over multiple stories. However, columns could not readily be fabricated, or for that matter transported, in lengths much greater than two—or occasionally three—stories. Therefore, columns in tall buildings, no matter how rigid, had to be spliced, and each column could only be as stiff and as strong as the splices.

Riveting was essential to achieve stiff splices, where the play or loosening of bolts could quickly lead to disaster.[59] Columns were generally fabricated in the longest lengths possible, and by the mid-1890s they were typically staggered in construction, so that splices in adjacent columns occurred on alternate floors, which avoided concentrations of potentially weak connections on one story. Erectors convinced engineers to locate splices just above finished floor levels, usually twelve to twenty-four inches clear, which enabled them to use floor beams to position firmly the long ends of multiple-story columns.[60]

Steel manufacturers began to produce specialized, rolled sections for columns by 1890 (Figure 16). These all balanced the desire for ideally strong cross sections with the need to provide reliable surfaces for connections. The purest shape was the Phoenix column (Figure 16, bottom, no. 2), manufactured by the Phoenix Iron Works near Philadelphia. Its circular cross section was assembled from curved, riveted plates that each had straight flanges at its edges, providing adequate surfaces for multiple rivets that were driven on site. While steel Phoenix columns could be drilled and planed to exacting dimensions in the shop, their curved surfaces presented problems for making connections, particularly stiff moment connections. Various solutions developed, notably the use of cross plates that were sandwiched between the column's segments. These provided steel tabs for attachments and established continuity between girders attached to opposite sides of the column.[61] Such cross plates, however, added weight nearer the less efficient center of the column section, which reduced the Phoenix columns' effectiveness. Problems with splices meant that the Phoenix column's shape incurred a penalty in terms of stability, and around the turn of the century it gradually fell from use.

By 1892, the importance of strong connections had eclipsed the concern for ideal structural shapes. Engineer W. H. Breithaupt, for example, noted in the *Engineering Record* "for proportion of length to diameter, as occurring in the great bulk of columns used in buildings, there is practically little or no difference in unit strength among the various sections in use. The one therefore which best admits of connections is the one in general to be preferred."[62] Given standard dimensions for plates and angles, it was relatively simple to design columns by building up thickness at the column perimeter, and connecting this with thinner webs, similar to beam and girder design. This distributed the material in a column toward the edges, approaching if not equaling the excellent theoretical performance of hollow, round columns. Many columns of the era used such fully engineered, custom-designed sections. Among these were box sections that approached the theoretical efficiency of the Phoenix shape, but which also provided flat surfaces on all four sides. Such shapes presented difficulties in assembly, as riveting was impossible within the small dimension of the interior voids. But these problems were eliminated either through methods like those adopted for the Phoenix, with added flanges to provide easily accessible riveting surfaces, or through the use of latticed planes, which replaced solid column walls of steel with lighter trusswork. This development removed more dead weight from columns, and it also permitted inspection of the interior surfaces for workmanship and corrosion, neither of which was possible with completely closed columns.

Most popular in Chicago were H-shaped or box sections assembled from rolled Z-bars. Z-shapes were relatively easy to roll, and building a column from them eliminated two angle connections and the cost of riveting one whole row of connectors. Z-shapes could form a variety of complex sections, and in conjunction with carefully tuned cover plates could form a consistent chassis for the full height of a building, which could easily be supplemented on lower floors by cover plates of greater thickness, tuning the columns to match the load at each level.[63]

This idea of a central chassis, to which thicker, harder working sections of steel could be attached, also formed the basis for two patented column shapes that saw wide use in the more technically advanced buildings of the mid-1890s. The Larimer column deployed two I-beams, each bent at 90 degrees at the center of their web, to form a cruciform section that behaved like two regular I-beams perpendicular to one another (see Figure 16, bottom, no. 3). However the Gray column bettered the Larimer with its efficient deployment of angles and a chassis of flat steel bars holding them in place (see Figure 16, bottom, no. 4). The angles were placed back-to-back in four pairs, each pair forming the vertex of a square cruciform shape. Between the angles, at regular vertical intervals, four bent steel bars were riveted to adjacent pairs of angles, forming a central diamond that maintained the angles' positions relative to one another. Nearly all of the column's material was thus located at the edge of its inscribed square—there was no material whatsoever on or near the column's neutral axis, and even the diagonal bars, the components nearest the center, occurred only intermittently. The flat surfaces of the steel angles offered convenient locations for bolting or riveting, and the hollow form that resulted from this statically efficient assemblage also offered a convenient conduit for pipes and cables.[64] Perhaps most importantly, the Gray column's angle thickness and depth could be adjusted within the overall dimensions of its square plan. Its performance could be attuned to its location in the building, with lighter angles used higher in buildings where loading was less, and heavier, deeper angles used lower, where loads were greater. These variations could all occur within the same footprint, which made splices simpler. The Gray was not perfect, however. It required extensive riveting to fabricate, and the intermittent nature of its internal bracing meant that it was susceptible to localized eccentric loading where the angles formed relatively weak, short columns between the diagonal connections.[65] But the Gray shape provided, despite its relatively high cost, the neatest balance between ideal static behavior and expedient assembly and connection on the job site. It would be the column of choice in the late 1890s and was used through the 1930s, when it was gradually superceded by specially rolled steel sections that approached its mathematical performance while eliminating its complex shop and field riveting.[66]

The first syntheses of riveted steel construction, stiff moment connections, and columns shaped to perform in concert with girders in standing against wind forces were the Reliance and Fisher Buildings in Chicago (1895 and 1896). Both were designed by D. H. Burnham and Co., with Charles Atwood as lead designer and Edward Shankland the engineer. In the Reliance, Shankland was faced with a

Figure 17 D. H. Burnham and Co. (Charles Atwood, designer), Reliance Building, Chicago, 1895. Contemporary postcard (W. G. MacFarlane, Toronto)

building site of alarmingly narrow proportions—less than 60 feet of frontage on State Street and only 80 feet on Washington (Figure 17). Given the projected height of around 200 feet, such dimensions offered little inherent resistance to wind. While the building's corner site required two perpendicular masonry firewalls, Shankland chose not to rely on these entirely, instead designing the Reliance's frame using oversized girders and Gray columns (Figure 18). Each connection had no fewer than seven pairs of rivets, taking full advantage of the Gray columns' generous flat surfaces.[67] Pre-drilled rivet holes were reamed on site, guaranteeing a secure fit.[68] The result, according to *Scientific American*, was the first large structural frame to employ the "table-leg" principle of wind bracing: "For wind bracing, instead of tension rods, which had been used heretofore, it was determined to put plate girders, 24 in. deep at each floor between the outside columns, thus binding the columns together and transferring the wind strain from story to story on the table leg principle."[69]

The Fisher followed the same principle. It stood on a lot just north of Van Buren Street from the Old Colony, with the same short east-west dimensions (Figure 19). Unlike the Reliance, it required no significant party walls, as it had no neighbors on three sides and only a short, three-story

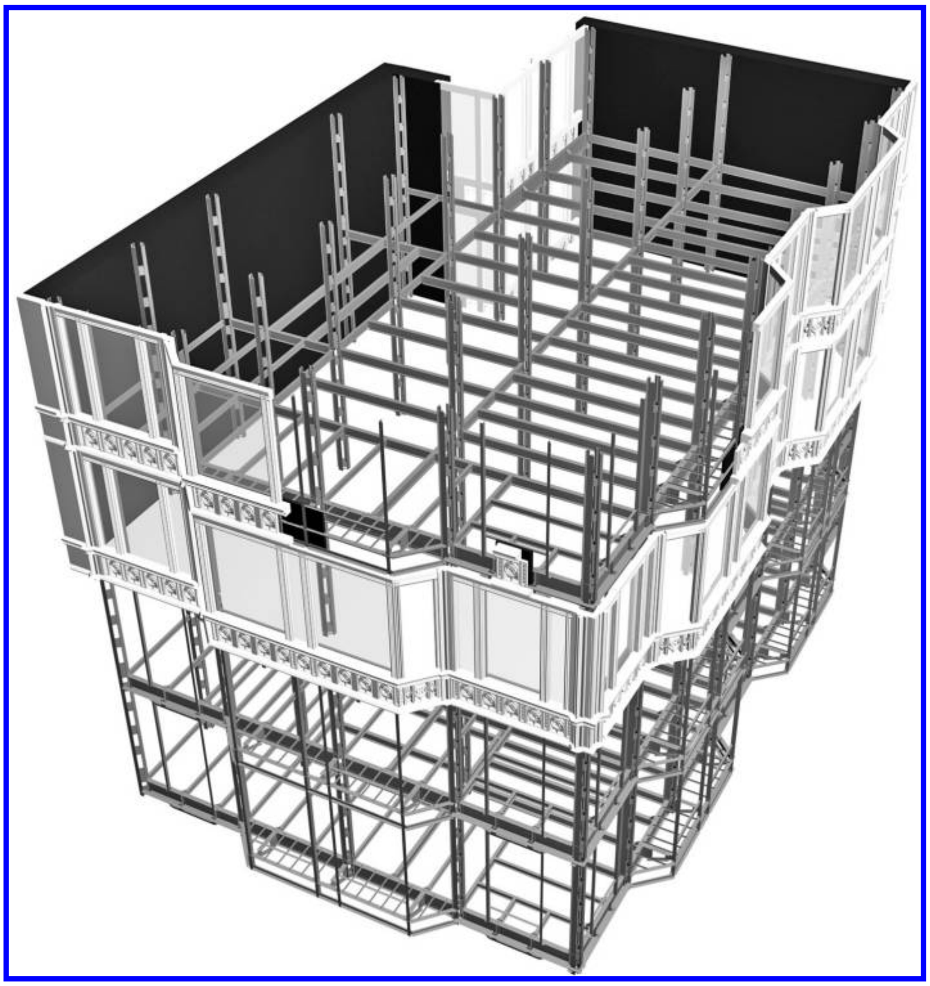

Figure 18 Reliance Building, digital reconstruction showing relationship between self-braced steel structure and skin. See *JSAH* online for interactive model (model and view by Ryan Risse)

structure to the north. Shankland broke its footprint with nine lines of Gray columns that ran across the short, east-west dimensions of the site, positioned at the building edges and in two rows either side of the main, double-loaded corridors (Figure 20). The moment frame and a particularly wide span between girders contributed to an extraordinarily light building—Shankland estimated the dead load of the system at only 75 pounds per square foot, much lighter than buildings such as Old Colony, which had weighed over 90.[70] This was matched by a remarkably free floor plan, punctuated only by columns at the central corridor and devoid of the sectional intrusions that were inherent in cross-braced and portal-framed structures. Combined with a light, glassy terracotta and glass exterior, the Fisher's minimal structure prompted the *Inland Architect* to marvel that it seemed to be "a building without walls."[71] (Figure 21)

These seemingly minor refinements to the basic steel skeleton proved decisive. The inherent qualities of steel and its ability to be assembled into calculable, efficient column shapes made reliable connections possible; with this the metal frame lost its experimental status and became a ubiquitous feature in all major North American cities. The differences between the masonry-bearing structures of 1885–90 and the more refined examples of the 1890–91, such as the Venetian and the Old Colony, 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 often-unreliable materials, inefficient shapes, and troublesome slack joints. 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 columns were increasingly sophisticated, of more scientifically studied material and of more mathematically calculated shapes. Most importantly, their lightweight metal frames used portal bracing or sway-rods to stand against the wind on their own, with immediate positive consequences for overall weight and internal planning. The use of steel allowed riveting techniques that in turn enabled a subsequent generation self-braced, plate or lattice girder frames such as the Reliance and Fisher in the mid-1890s. These eliminated the remaining planning

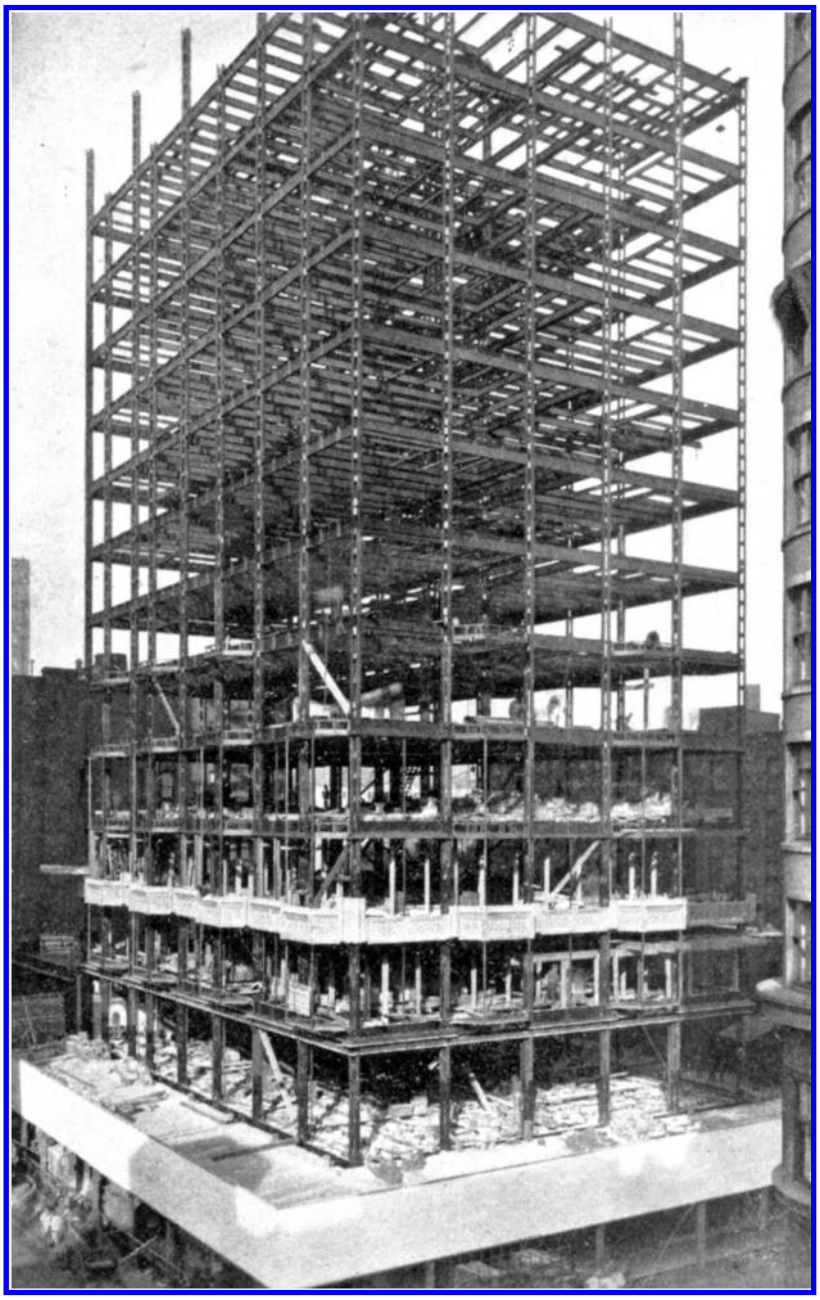

Figure 19 D. H. Burnham and Co. (Charles Atwood, designer), Fisher Building, Chicago, 1896. Construction photograph, Nov. 1895 (photographer unknown, from William H. Birkmire, *The Planning and Construction of High Office Buildings* [New York: John Wiley and Sons, 1900], 59)

problems inherent in planar wind-bracing systems, and established the moment-resisting connection as an important element for subsequent skyscrapers.

Wind bracing was one of many technical, economic, and stylistic developments in the late nineteenth century that can be traced in the tall buildings of Chicago and elsewhere. Like these other developments, its successful implementation and gradual refinement under functional and cost pressures involved collaboration and communication among architects, engineers, builders, industrialists, and clients. Tall building design in this era—as today—exceeded the abilities of single minds, or even of single firms. Successful conception and execution required extensive integration of structural, planning, fabricational, and constructional techniques. This was only possible through widespread collaboration and the sharing knowledge by means of relatively new media such as professional meetings and journals. This altered the image of the architectural profession. With such complexity, and with structural, cladding, and other systems so tightly woven together, the tall office building required architects to adjust subtly their traditional roles as omnipotent master builders, and to cede important responsibilities in structural

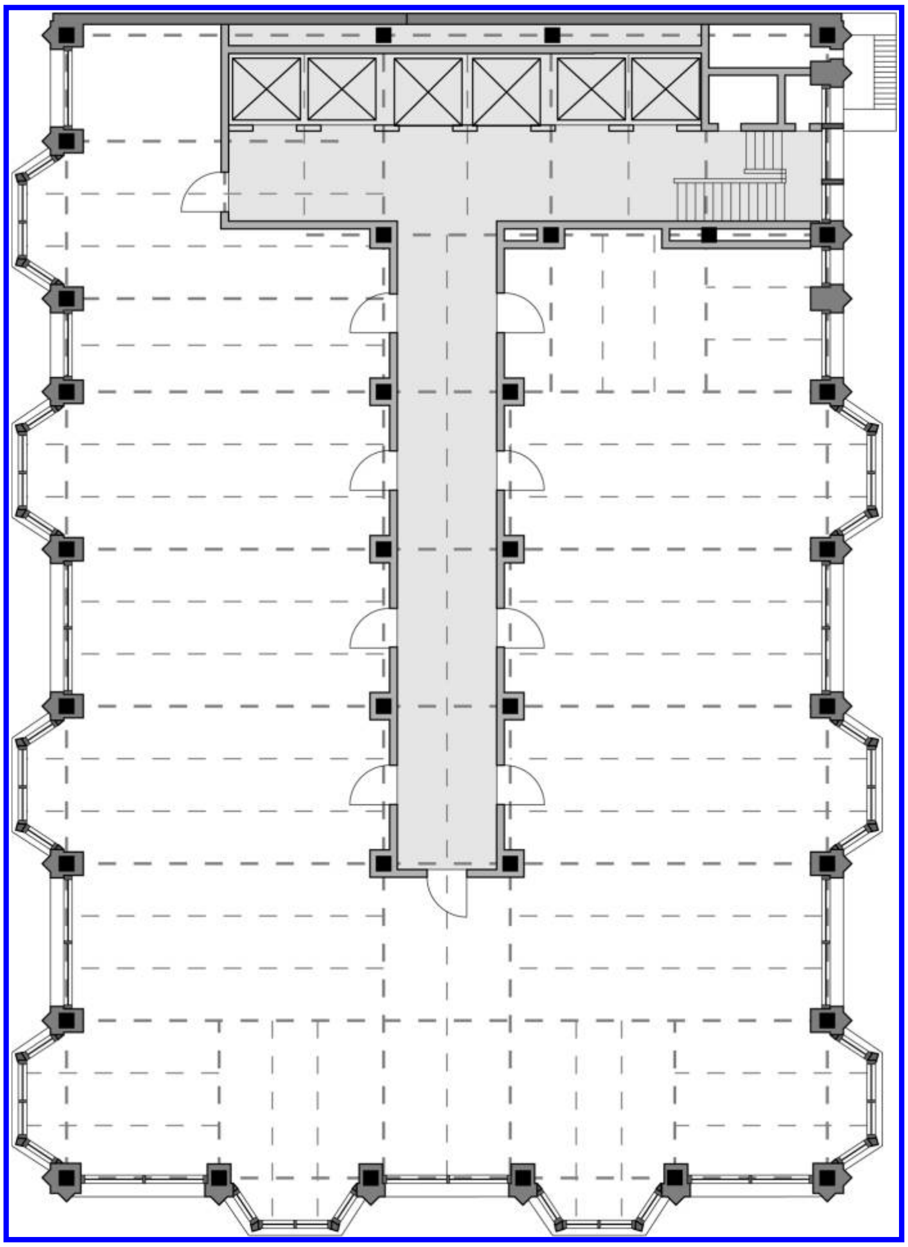

Figure 20 Fisher Building, plan of typical floor (drawing by the author based on Edward Clapp Shankland, "Steel Skeleton Construction in Chicago," *Journal of the Institution of Civil Engineers*, 128 [1896–97], part II, 3; and "Technical Review: The Fisher Building, Chicago—A Building without Walls," *The Inland Architect and News-Record, Special Supplement* 27, no. 4 [May 1896], n.p.)

engineering and construction methodology. But this sacrifice of the master's role, overseeing a loose amalgamation of crafts and expertises, brought with it tremendous new efficiencies and new, deeply collaborative ties to industry and engineering.

This change has been noted by historians Carl Condit, Tom Peters, and Sara Wermiel, among others. It can be traced in the rise of general contracting as a profession, in the development of scientifically calculated foundation systems, and in the growing importance of mechanical systems and environmental control, all of which also occurred in the decade between 1885 and 1895.[72] The advance of technological solutions and their multiplicative effects on the economics of skyscraper construction required greater orchestration, more precise coordination, and far greater job site expertise than the building culture of the 1880s had provided. Tall building construction emerged from this period of intense development a highly refined, well-organized, and technically advanced practice involving greater specialization among consultants and, simultaneously, a broader function for architects as orchestrators. Wind bracing—with its reliance on newly affordable materials, newly minted engineering methods, and newly developed construction techniques—was one of several developments that marked the transformation of American building culture into a coordinated system of professional and technical relationships.

Figure 21 Fisher Building. Commercial postcard (The Rotograph Co., New York, card no. D28034)

Notes

The author gratefully acknowledges Ryan Risse for the digital reconstructions of the Reliance and Home Insurance Buildings, and Ryan Gauquie and Shaghayegh Missaghi for their assistance in translating these for online publication.

1. W. L. B. Jenney, "Chicago Construction, or Tall Buildings on a Compressible Soil," *Engineering Record*, 14 Nov. 1891, 389–90. Other references to skyscraper framing as bridge construction can be found in John A. Howland, "Modern Skyscraper," *Chicago Daily Tribune* (10 Oct. 1904), B5; Edward Clapp Shankland, "Steel Skeleton Construction in Chicago," *Journal of the Institution of Civil Engineers* 128 (1896–97), part II, 1–27; and Corydon T. Purdy, "The Evolution of High Building Construction," *Journal of the Western Society of Engineers* 37, no. 4 (Aug. 1932), 201–11.

2. Frank A. Randall in *History of the Development of Building Construction in Chicago* (Urbana: University of Illinois Press, 1949), places the developments of steel columns and wind bracing adjacent to one another in a chronology entitled "The Development of Skeleton Construction," noting the remarkably short two years between the first use of steel columns in Chicago in Burnham and Root's Rand McNally Building of 1890, and "one of the last tall buildings to be constructed with cast-iron columns," Clinton J. Warren's Unity Building of 1892. Under "Wind Bracing," Randall notes the achievements of the William Le Baron Jenney's Manhattan Block (1890), Burnham and Root's Monadnock (1891), and Jenney's Isabella (1892), but does not explore the coincidence of these two developments. Similarly, Carl Condit, *The Chicago School of Architecture: A History of Commercial and Public Building in the Chicago Area 1875–1925* (Chicago: University of Chicago Press, 1964) only treats wind bracing as a footnote (note 64), mentioning precedents in metal construction as early as Charles Fowler's Hungerford Fish Market of 1835, and drawing links to other structural types including Joseph Paxton's Crystal Palace (1851) and the Statue of Liberty (1883). While elsewhere mentioning wind bracing systems used on the Manhattan, Isabella, and other structures built during this brief moment of transition, *The Chicago School of Architecture* does not discuss the change to steel in any depth, and thus, like *History of Chicago Building*, it does not explore the relationship between steel and wind bracing.

3. Bill Addis, *Building: 3000 Years of Design, Engineering, and Construction* (London: Phaidon Press, 2007), 403–4.

4. Ibid., 405.

5. Donald Friedman, *Historical Building Construction* (New York: Norton, 1995), 80–81.

6. Cecil Elliott, *Technics and Architecture* (Cambridge: MIT Press, 1992), 91–96. contains a cogent summary of iron and steel developments in Europe and America.

7. The Home Insurance Building's alleged first use of steel beams is countered by an unsubstantiated claim in the Chicago Tribune by contractor George Fuller, who believed that steel beams had been used in Cobb and Frost's Chicago Opera House of 1885. "Like a City of Steel," *Chicago Daily*, 25 June 1891, sec. 1, p. 8.

8. The two exceptions in Western history were Gothic cathedrals, which employed an empirical approach to wind resistance as Robert Mark has shown (spurring some debate) in *Light, Wind, and Structure*, and in seismic zones, where masonry structures often employed timber bracing to resist lateral forces that were exponentially greater than those of wind.

9. By far the most cogent contemporary assessment of problems induced by wind was H. H. Quimby's series of articles in the *Engineering Record* in 1892–93, from which much of the following discussion is taken. In his introductory to the series, Quimby summed up the problems as follows: "The action of the wind against the side of a building produces the effects of overturning and shear, both greatest at the highest point of external resistance, which is the roof of an adjoining building, if there be any, or otherwise the surface of the ground. The overturning or the lift on the windward side is likely always to be less than the resistance of dead weight, but the shear is liable to be overlooked and is probably the immediate cause of the collapse of most of the buildings destroyed by wind. In the type of structure under consideration, the shearing action tends to topple the columns and crush the partitions or rupture the bracing, all in one story. The column fastenings described are not stiff enough to prevent a slight movement of the tops of the columns, which can be firmly held by the bracing alone. If this bracing is mortared work its cohesiveness is liable to be gradually destroyed by severe vibrations or many successive impacts of pressure; and once its hold is loosened, its deterioration will be rapid." H. H. Quimby, "Wind Bracing in High Buildings," *Engineering Record*, 26, no. 25 (19 Nov. 1892), 394.

10. "The Tay Bridge Disaster," *Science* 1, no. 6 (7 Aug. 1880), 70–71.

11. Shankland, "Steel Skeleton Construction in Chicago," 6–8; and William H. Birkmire, *The Planning and Construction of High Office Buildings* (2nd ed., New York: John Wiley & Sons, 1900), 205–6.

12. Quimby, "Wind Bracing in High Buildings," 298.

13. "In completing the details of construction, the assembling of the parts, wind bracing, etc., it was found necessary to invent special arrangements, the iron railroad bridge being the only precedent." D. Everett Waid, "A History of Steel Skeleton Construction," *The Brickbuilder* 3, no. 8 (Aug. 1894), 158. Waid also noted that Jenney's frame for the Home Insurance had been described at the time as a "railroad bridge standing on end."

14. "A local corporation, recently organized, established its headquarters on the top floor of one of the tallest buildings in town . . . The crowning glory of [the] office was a big clock with an elaborately carved case . . . it had a long, shiny pendulum, which was to swing slowly and with regularity . . . On the first day the pendulum stopped. The clock was sent back to the dealer, whose experts took it apart, oiled it and set it to running again. . . . Once more it was taken up to the president's office and once more it ceased running. . . . An architect who became acquainted with the facts in the case solved the mystery. He said the oscillation of the high building counteracted and stopped the swing of the pendulum. The pendulum couldn't work with any regularity, so long as the building was nodding around in the changing winds like a cat-tail before a summer zephyr." "Chicago Building that Stopped Clocks," *The American Architect and Building News* 44, no. 959 (12 May 1894), 68. Other commentators were more skeptical; *Architecture and Building* claimed such tales were "ridiculous," and no firm reports of pendulum clocks stopping on account of building sway seem to exist.

15. "Some Phenomena of the St. Louis Cyclone," *Engineering Record* 34, no. 3 (20 June 1896), n.p. and editorial, *The American Architect and Building News* 47, no. 1002 (9 Mar. 1895), 97–98.

16. "The Stability and Security of Skeleton Buildings," *Engineering Record* 27, no. 8 (21 Jan. 1893) 149.

17. William H. Birkmire, *The Planning and Construction of High Office Buildings* (2nd ed., New York: John Wiley & Sons, 1900), 20.

18. "Maximum Wind Pressure," *Engineering Record and American Contract Journal* 14 (10 Oct. 1885), 235.

19. Quimby, "Wind Bracing in High Buildings," 260; and Shankland, "Steel Skeleton Construction in Chicago," 9. Shankland did add a further factor of safety to columns using this load, however. For comparison's sake, code-mandated loads in the United States today vary by geography, with those in hurricane-prone regions requiring between 44 and 55 psf.

20. Joseph Kendall Freitag, *Architectural Engineering with Especial Reference to High Building Construction, Including Many Examples of Prominent Office Buildings* (2nd ed., New York: Wiley & Sons, 1904), 82.

21. "Absolutely positive results can be obtained only through the use of some definite form of metallic bracing. This may be in the form of sway-rods, portals, or deep girders between the columns, a selection depending largely upon circumstances." Freitag, *Architectural Engineering*, 258.

22. "This method can never be carried out completely, owing to its interference with the doors and windows, and to its rendering permanent the partitions in which the systems are placed. To show that it is effective, however, the case of the [Great Northern] hotel may be cited. The lateral rods, which were shown in the original plans, did not arrive at the building until the frame had reached a height of five or six storeys, and the building of the outside walls had been begun. A traveling derrick was used to set the steel-work, but the vibration which it caused in the building made it impossible to fix the terra-cotta. Work was accordingly stopped until the rods were inserted, when there was no further trouble, although at one time the steel-work was six or seven storeys above the masonry work." Shankland, "Steel Skeleton Construction in Chicago," 7–8.

23. See Corydon T. Purdy, "The Steel Skeleton Type of High Buildings—II," *Engineering News*, 12 Dec. 1891, 560–61; and Freitag, *Architectural Engineering*, 264–68. The Venetian's wind bracing system was designed to absorb 70 percent of the assumed load, with the other 30 percent to be taken up by the inherent stiffness of the building's partitions, one of the earliest quantifications of this residual performance.

24. Shankland, "Steel Skeleton Construction in Chicago," 6–8; and Freitag, *Architectural Engineering*, 263–64.

25. "Knee-braces . . . may only be considered as a partial means of wind-bracing, or as supplementing the stiffness in connections secured elsewhere in the structure. These are not to be recommended as the only means of bracing in any important work, but are used rather as an act of necessity where only partial bracing is required. They can be conveniently arranged in the exterior walls either above or below the girders, or, if required, both above and below, without interfering with the architectural requirements as to windows or other openings. Any form of knee-bracing requires great exactness in manufacture, and care in erection." Freitag, *Architectural Engineering*, 259.

26. Historic American Buildings Survey No. ILL 1053, "Old Colony Building, 407 South Dearborn Street, Chicago, Illinois," Chicago Project II, 1964, sheet 3 of 3, West Elevation.

27. Robert Bruegmann, Holabird & Roche/Holabird & Root: An Illustrated Catalogue of Works, 1880–1940 (New York: Garland, 1991), 131.

28. Freitag provided perhaps the most cogent summary of the Old Colony structure's composition: "The portal system was used in the Old Colony Building, Chicago, completed in 1894. The portals are placed at two planes in the building—a cross-section of one set being shown in Fig. 158. Wind pressure was figured at 27 lbs. per sq. ft. on one side of the building at a time. Each portal was calculated independently for the sections of both top and bottom flanges, thickness of web, cross-shear on rivets connecting the curved flanges, and for all splices and connections. A detail of one portal is shown in Fig. 159. This arrangement of wind-bracing proved very satisfactory in all respects, and, according to the designer, was cheaper in the end than the sway-rods provided in the first design." Freitag, *Architectural Engineering*, 271–72.

29. "[T]he writer would question whether portal-bracing can be provided cheaper than tension-rods, as claimed. With good details in connections and proper regard for their location in the original planning of the building, sway-rods can be used without great expense or trouble. The portal arrangement certainly makes a fine interior appearance if the arched openings are given a slight decorative treatment in plaster, as was done in the Old Colony Building. The floor plan will generally govern the use of either one or the other system, whether the rooms are to be connected by large openings or small doorways." Freitag, *Architectural Engineering*, 272.

30. W. L. B. Jenney, "Chicago Construction, or Tall Buildings on a Compressible Soil," *Engineering Record* 24, no. 24 (14 Nov. 1891), 390.

31. "These shears are undoubtedly reduced to some considerable extent through many practical considerations. The dead weight of the structure itself, the resistance to lateral strains offered in the stiff riveted connections between the floor systems and the columns, the stiffening effects of partitions (if continuously and strongly built), and linings, coverings, etc., all tend to decrease the distorting effects of the wind pressure. But, in view of the uncertainty in regard to the efficiency of these latter considerations, they may not be relied upon, and are therefore disregarded in the calculations." Freitag, *Architectural Engineering*, 260. Some engineers did, in fact, rely entirely on the stiffness of terracotta partitions and fireproofing to sustain their designs against lateral load.

32. Shankland, "Steel Skeleton Construction in Chicago," 8–9.

33. Jenney, writing in 1891, noted that "knees as in naval architecture," or "X rods, as in bridge construction" were the only two available systems of wind bracing, suggesting that the plate and lattice girders of Shankland's development were not only unrealized, but not even considered even at this relatively late date. Jenney, "Chicago Construction, or Tall Buildings on a Compressible Soil," 390.

34. Quimby, "Wind Bracing in High Buildings," 395.

35. H. H. Quinby, "Wind Bracing in High Buildings" [cont.], *The Engineering Record*, 31 Dec. 1892, 99.

36. Foster Milliken, in H. H. Quimby, "Wind Bracing in High Buildings," *Engineering Record* 27, no. 8 (21 Jan. 1893), 162.

37. H. W. Brinckerhoff, in H. H. Quimby, "Wind Bracing in High Buildings," *Engineering Record* 27, no. 9 (28 Jan. 1893), 180.

38. "[Henry Law, author of the Tay Bridge report] finds that the base of the pier was too narrow, occasioning a very great strain upon the struts and ties; that the angles at which the latter were disposed, and the mode of connecting them to the columns, were such as to render them of little or no use; and that other imperfections, which he points out, lessened the power of the columns to resist a crushing strain; and further, that the yielding of the struts and ties was the immediate cause of the disaster. Among the other imperfections alluded to above, one was the defective mode of connecting the columns at the flange joints, the bolts being one-eighth inch less in diameter than the hole, and the flanges being separated in some places as much as three-quarters of an inch. The mode of attaching the ties to the columns by means of lugs, he held to be insufficient, as in every instance it was found that the lugs were torn away." "Report on the Tay Bridge Disaster," *The Manufacturer and Builder* 12, no. 12 (Dec. 1880), 268.

39. "Field riveting has now entirely superseded the use of bolts in skeleton or cage construction, or indeed in any character of high-class building work. Bolted connections were tried, but were soon discarded on account of the cracks which developed in the plastered ceilings. These cracks were always found to radiate form the column connections with the floor system, thus demonstrating the play of bolts in the holes." Freitag, *Architectural Engineering*, 68.

40. "I would like to call your attention to the fact that too many bolts are used in building work for field connections. My objection stands only when the number of bolts is calculated to resist the load carried by the girders. Most of the time the connecting holes are punched and not reamed, hence the bolt is only tangent to the circumference of the connecting holes and cannot be calculated for bearing before a certain deformation takes place in the body of the bolt or in the shape of the hole. Now let us suppose one of the two has become out of shape so that there is a certain bearing we can rely upon. The question springs to our mind: what is that bearing? Is it the diameter of the bolt multiplied by the thickness of the plate? We don't know." A. V. Gravelle, C.E., "Riveted versus Bolted Field Connections," [letter] *Engineering News* 35, no. 4 (26 Dec. 1896), 75.

41. "Cast-Iron Columns in Buildings," *Engineering News* 36, no. 5 (3 July 1897).

42. "It had been found in certain cases that the rivets sheared across with a less force when in drilled than when in punched holes, and this had been attributed to the sharpness of the edges of the drilled holes. This point seemed to require investigation, and experiments to elucidate it had, therefore, been made, the results of which are narrated in the paper. The theory that the sharp edges of drilled holes induced a cutting action, which diminished the strength of the rivets, had suggested to Colonel Inglis, R.E., to try whether a greater resistance in the joint could be obtained by purposely rounding the edges of the rivet-holes. Some experiments had been made for Colonel Inglis by Mr. Kirkaldy, from which it appeared that the resistance of the rivet was increased 10 per cent by this rounding of the edges of the rivet holes." William Fairbairn, "On the Durability and Preservation of Iron Ships, and on Riveted Joints" [abstract], *Proceedings of the Royal Society of London* 21 (1872–73), 260.

43. Fairbairn, "On the Durability and Preservation of Iron Ships, and on Riveted Joints," 262–63.

44. Fairbairn disproved the common assumption that riveted joints added resistance to load by imposing friction between two members as they cooled and drew tight: "The friction between the plates induced by the contraction of the rivets in cooling has been supposed sometimes to add to the apparent resistance of the rivet to shearing. It is shown that a considerable displacement of the plates takes place before ultimate facture, and that the deformation of the rivets is so great that it can hardly be supposed that they exert any tension, holding the plates together at the moment of fracture. The friction should therefore be entirely neglected in estimating the ultimate resistance of riveted joints; and this, indeed, has been done." Fairbairn, "On the Durability and Preservation of Iron Ships, and on Riveted Joints [Abstract]," 263.

45. Fairbairn, "On the Durability and Preservation of Iron Ships, and on Riveted Joints [Abstract]," 260. Fairbairn noted, however, that hand riveting, which took roughly 90 seconds per rivet, offered the advantage of tempering the hot rivet by hammering after it had cooled somewhat.

46. "Beams and girders are first bolted temporarily in place, about one-third of the holes being filled. The riveting gang then follows behind the erectors, making permanent connections with iron rivets heated in portable forges." Freitag, *Architectural Engineering*, 68. See also Gravelle, "Riveted versus Bolted Field Connections."

47. Freitag, *Architectural Engineering*, 138.

48. "Chicago's Big Buildings," *Chicago Daily Tribune*, sec. 1, 13 Sept. 1891.

49. "Cast-Iron Columns in Buildings." Other contemporary reportage noted the acceptance of much lower factors of safety for steel over cast-iron: "Steel riveted columns as now manufactured are considered perfectly safe with a coefficient of safety of four, while for the cast-iron columns a coefficient of safety of eight is not considered too great." D. Everett Waid, "A History of Steel Skeleton Construction," *The Brickbuilder* 3, no. 8 (Aug. 1894), 158–59. Engineer C. L. Strobel provided the most direct link between the change in material from cast-iron to steel as a precursor to the developments of the mid-1890s: "Other improvements followed [the Tacoma]. Steel columns were substituted for cast-iron columns, and this made it possible to construct the steel frame so that it furnished the resistance to wind pressure required, and walls for staying the building could, in certain cases, be entirely dispensed with." C. L. Strobel, "The Design of Steel Skeleton Buildings," [letter] *Engineering Record* 34, no. 8 (25 July 1896).

50. Freitag, *Architectural Engineering*, 68, 77.

51. "Connections should transmit, as directly to the axis of the column as may be, the full strength of the various members which attach to the column; the closer they can be to the column axis, and thus minimize eccentricity of loading, the better. Eccentricity of loading, such as must necessarily occur in any closed columns loaded on one side more than the other, may, depending on leverage, reduce the unit strength of the column 20 or more per cent." W. H. Breithaupt, M. Am. Soc. C.E., "On Iron Skeletons for Buildings," *Engineering Record* 26 (5 Mar. 1892), 226.

52. "However carefully or slightly the calculations for eccentric loading may be treated, certain practical considerations at least must be regarded in an attempt to secure the best possible transfer of girder loads, etc., to the centre of gravity of the column section. It is very important that the brackets or girder seats which transmit the girder loads to the columns should be designed with reference to bringing such loads to the centre of the column as soon as possible, and also that the column should be capable of acting as a unit under the application of such loads." Freitag, *Architectural Engineering*, 214.

53. "Being entirely open, with both the interior and exterior surfaces exposed, any inequalities in thickness [of H-shaped columns] can be readily discovered, and the thickness itself easily measured, thus obviating any necessity for boring, and rendering the inspection of the columns much less tedious.

"2. The entire surface of the column can be protected by paint."

"3. When built in brick walls, the masonry fills all voids, so that no open space is left . . . only the edge of the column comes near the face of the wall."

"4. Lugs and brackets can be cast on such columns better than on circular columns, especially for wide and heavy girders."

"5. The end connection of the columns do not require projecting rings, or flanges, which are often objectionable in circular columns. . . . The columns may be fireproofed in the same way as the Z-bar columns, which it much resembles. The space occupied by the column slightly exceeds that of both the cylindrical and Z-bar column, but not enough to be of any serious consequence. . . . The beams running at right angles to the web should be tied together by wrought-iron straps passing through holes in the web of the column." F. E. Kidder, "Safe Loads for H-Shaped Cast-Iron Columns," *The American Architect and Building News* 45, no. 969 (21 July 1894), 27–28.

54. "For columns, cast iron would be preferable to both wrought iron and steel were it not for its lack of uniformity, the difficulty of making rigid connections to it, as it cannot be riveted to, and the impossibility of inspecting the ordinary cast-iron columns. Of such moment are these objections, however, as to entirely condemn, notwithstanding its smaller cost per pound, the continued use of cast iron for columns of high buildings. The metal in a closed cast column may be ¼-inch thick on one side, while it is an inch or more on the other; the casting may have numerous blow holes or other destructive faults, and yet to feasible inspection, which can cover its exterior only, the column may appear perfect. To cast in longitudinal sections, admitting of proper inspection, and then to machine these, as would be necessary to properly unite them into a column, would give more expensive results than their equivalents in rolled material." Breithaupt, "On Iron Skeletons for Buildings," 226.

55. "If there is one situation in which, under certain conditions, the architect and the engineer feels himself thoroughly justified in employing cast-iron, it will be universally conceded that it is in the form of the column or pillar. It matters little or nothing, so far as the principle is concerned, whether the section be that of the hollow cylinder or of the solid cruciform, or of the H, or of any other shape, which might suit the particular circumstances of the case. It is exceedingly rare that any instances have occurred in which cast-iron has been known to fail when subjected to the proper and fair amount of stress, both in character and amount, while acting as a vertical support to a vertical load. When, therefore, the failure of the material takes place under all these conditions favorable to the stability and durability of the column, especially when as in the example we intend describing, the failure is on a scale of considerable magnitude, it not only becomes endowed with a large amount of interest." "Failure of Cast-Iron Columns," *The American Architect and Building News* 45, no. 971 (4 Aug. 1894), 46–47.

56. "Cast-Iron Columns in Buildings," *Engineering News*, 36, no. 5 (3 July 1897), n.p.

57. "The Design of Steel-Skeleton Buildings," [selection of letters] *Engineering Record* 34, no. 6 (11 July 1896), 103.

58. Steel during this era began to realize significant reductions in production costs as well, making it a viable alternative to cast iron in particular. Sara E. Wermiel, "Introduction of Steel Columns in American Buildings, 1862–1920," *Engineering History and Heritage* 162, no. 1 (Feb. 2009), 19–27. I am grateful to Dr. Wermiel for sharing an early version of her paper.

59. "The columns in the modern design must be capable of affording stiff connections so as to withstand both the direct dead- and live-loads transferred from the floor system, as well as sufficient connections for the wind-bracing. These cannot be secured well by means of bolts passing through the horizontal flanges of cast columns, even if the workmanship be considered accurate. The workmanship, however, can seldom, if ever, be relied upon as perfect; the bolts never completely fill their holes, and 'shims' are constantly employed to plumb the columns. These constitute elements of weakness which may easily allow considerable distortion. The girder connections to the columns, resting on cast brackets and bolted through the flanges, are bad in the extreme, especially for cases of eccentric loading and the irregular placing of beams." Freitag, *Architectural Engineering*, 192.

60. Ibid., 40, 58, 68.

61. Ibid., 56–57.

62. Breithaupt, "On Iron Skeletons for Buildings," 226.

63. Birkmire, *Planning and Construction of High Office Buildings*, 194.

64. "Recently a column made of eight angle-bars in pairs, connected together by tie-plates, has been used. It can be kept the same size from back to back of the angle bars, from the basement to the roof, so that the joint can be made with vertical splice-plates. The column has . . . hollow spaces throughout its length, in which water-, steam-, and gas-pipes are placed. Beams and girders are connected directly with the faces of the column, a method which gives great lateral stiffness." Shankland, "Steel Skeleton Construction in Chicago," 6.

65. Freitag, *Architectural Engineering*, 209.

66. Wermiel, "Introduction of Steel Columns in American Buildings, 1862–1920."

67. Freitag, *Architectural Engineering*, 183.

68. Specification for columns: "The columns will be made in two-story lengths, alternate columns being jointed at each story. The column splice will come above the floor, as shown on the drawings. No cap plates will be used. The ends of the columns will be faced at right angles to the longitudinal axis of the column, and the greatest care must be used in making this work exact. The columns will be connected, one to the other, by vertical splice plates, sizes of which, with number of rivets, are shown on the drawings. The holes for these splice plates in the bottom of the column shall be punched ⅛ small. After the splice plates are riveted to the top of the column, the top column shall be put in place and the holes reamed, using the splice plates as templates. The connection of joists or girders to columns will be standard wherever such joists or girders are at right angles to connecting face of column. Where connection is oblique, special or typical detail will be shown on the drawings." Charles E. Jenkins, "A White Enameled Building," *Architectural Record* 4, no. 3 (Jan.–Mar. 1895), 302.

69. "The Reliance Building, Chicago," *Scientific American, Building Edition* (Jan. 1895), 17.

70. Freitag, *Architectural Engineering*, 122–23.

71. "Technical Review: The Fisher Building, Chicago—A Building without Walls," *The Inland Architect and News-Record, Special Supplement* 27, no. 4 (May 1896) n.p.

72. See, respectively, Sara E. Wermiel, "Norcross, Fuller, and the Rise of the General Contractor in the United States in the Nineteenth Century," *Proceedings of the Second International Congress on Construction History* (Exeter: Short Run Press, 2006), 3: 3297–313; Donald L. Hoffman, "Pioneer Caisson Building Foundations: 1890," *JSAH* 25, no. 1 (Mar. 1966), pp. 68–71; and Cecil D. Elliott, *Technics and Architecture: The Development of Materials and Systems for Buildings* (Cambridge: MIT Press, 1992), chaps. 11–13, 271–362.