NEW YORK’S TOWER BUILDING:
STRUCTURAL ANALYSIS OF A PROTO-SKYSCRAPER

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Abstract. The Tower Building, completed in 1889 in New York, was an 11-story (39m) early skyscraper with a hybrid frame. It was among the last tall buildings completed in the United States before the introduction of skeleton framing in 1890, and attracted a great deal of attention for its extreme slenderness and its unique structural system, which is best described as a five-story bearing-wall building sitting on top of a six-story frame building.

The building occupied a narrow mid-block site, that made the use of masonry bearing walls impractical, as the wall thicknesses required by code would occupy nearly half of the lot width. The constructed solution was to use a cast- and wrought-iron braced frame for the bottom seven stories of the building using the legal fiction that it was an extended basement. Traditional masonry bearing walls were carried on the top of the frame at the seventh floor and extended up to the roof over the 11th story. It was the first commercial building of such height and such an extreme height-to-width ratio, which helps explain the difficulties that Bradford Lee Gilbert, its architect, had with both the Board of Examiners of the New York City Building Department and with public perception of safety. In the 1890s, there was an extended discussion in the American engineering community on the appropriate wind loads and methods of bracing to be used in tall buildings. At the same time there was discussion in the public press of the effect of the new type of the building, the “skyscraper,” on public safety.

This paper examines the practicality of the hybrid structure: was it adequate using the codes and state of knowledge at the time of construction? Would it be considered adequate today? Using old methods for design and current formulas, the paper compares how close the structural design was to current standards and how methods of design have evolved for the last century.
1 INTRODUCTION

The history of skyscraper structure tends to focus on buildings with skeleton frames and non-load-bearing curtain walls, because that form is used in all modern tall buildings. However, tall buildings were constructed before that form was used, and some of the hybrid frames were important steps toward the development of the skeleton frame.

The first fire-resistant skeleton-frame skyscrapers were constructed in 1890: the Manhattan Building in Chicago and the London & Lancashire Building in New York. Just before these buildings were constructed, the Tower Building was completed in New York, and attracted national attention due to its extreme slenderness for that era – its height was greater than 6 times its width – and its unique hybrid frame.[1,2]. Because the Tower Building did not have a modern-style skeleton frame, an analysis is necessary to see if its structural system is rational by modern standards.

1.1 Historical Context

The Tower Building was, for its time, quite slender, but it was not exceptionally tall. The tallest American building of “modern fireproof” construction in 1889 was the (small footprint) tower of the (large-footprint) Auditorium Building in Chicago, which was 275 feet (84m) high, and the (small) tower of the (large) Tribune Building in New York, at 235 feet (72m) high. There were some 20 extant buildings that had higher top elevations than the Tower Building when it was completed, either with small towers or entire floors. All had bearing-wall or hybrid frame structure. The first cage-frame building, a form of hybrid where the interior floors were carried entirely on a metal frame, was the 1885 Home Insurance Building in Chicago.

The unique hybrid structural system used at Tower was not a modern skeleton frame, although it approaches that form at the lower floors. Its fame stems first from the impression it made on the local real-estate community and, via the national press located in New York, the American design community (by showing that a small lot could be developed with a tall building; and later from the efforts made by its architect, Bradford Lee Gilbert, to claim for it the title of “first steel-frame skyscraper.”[3,4]
Not until 1892, two years after the first skeleton-frame building was completed and three years after the Tower Building, did the New York City Building Code recognize the existence of skeleton framing. The rapid proliferation of skeleton-frame skyscrapers in the downtown business district of New York meant that the Tower Building was completely surrounded by tall buildings by 1905, and was eventually torn down as economically obsolete in 1914. (See figure 1.)

2 BUILDING DESCRIPTION

The site for the Tower Building was assembled from two lots, one with a 21'-3" (6.5m) frontage on Broadway measuring 110 feet (33.5m) deep, and one with a 38'-3" (11.7m) frontage on New Street measuring 57'-2" (17.4m) deep. The north lot lines are the same line, making the entire site a blunt “L” in plan. It should be noted that, because of lower Manhattan’s irregular street grid, the lots were not rectangular and all dimensions vary slightly from side to side. (See figure 2). The building was 11 stories with an attic on Broadway; because of the sloped site, that made it 12 stories with an attic on New Street. There were two sub-grade levels, officially known as the cellar and sub-cellar. The typical floor to floor height was 11'-9" (3.6m), putting the attic at an elevation of 129 feet (39.3m) above grade, with the peaked roof extending up to 147'-5" (44.9m) above grade.

The building consisted of two wings that were, in structural terms, essentially two different building. Both had wrought-iron beams supporting terra-cotta tile arch floors, but the basic frame layouts were different. The east wing, on the New Street lot, had bearing walls at the north and south sides, a row of cast-iron columns and girders running east-west, and two spans of ordinary filler beams running from the north bearing wall to the column line to the south bearing wall. This system, mixing bearing walls with iron “frames” that were designed for gravity load only, had become common in American tall building construction in the late 1870s. The east wing is therefore of little structural interest.

The west wing, on the Broadway lot, is the portion of the building where constraints of geometry and code requirements forced the designers to look for a new solution to framing a tall building. The first constraint was simply the width of the west wing, 21'-3" (6.5m) north to south. In such a narrow building, developed for speculative commercial use, every bit of width matters. The minimum thickness of brick wall in use for commercial buildings in New York at that time was 12 inches (0.3m), or three wythes of bricks thick. If it were possible to use these minimum walls, they would have occupied more than 9 percent of the floor area of the wing, which is a non-negligible loss. However, the second constraint was the minimum wall thickness specified by New York City Building Code, which varied from 20 inches (0.5m) at the top two floors to 36 inches (0.9m) at the base. [5] At the 11th floor, the side
walls would occupy over 15 percent of the lot; at the first floor, more than 28 percent. This loss of rentable space was enough to mean the difference between a building that was an economic success and one that was not.

The solution worked out by Gilbert and William Birkmire, an engineer working for the Jackson Iron Works, the company that erected the frame, was to take advantage of ambiguous wording in the building code. First, the thickness of walls was typically calculated by their height above the curb, but walls that were supported on metal beams had their thickness counted from the elevation of that support. Second, the code allowed for "light partition walls" as thin as 8 inches (0.2m) of brick, with a maximum height of 50 feet (15.2m) from their support. [5] Third, there was no specific limitation put on supporting walls on iron girders and columns. The structural scheme for the west wing therefore consisted of (1) a frame consisting of cast-iron columns and wrought-iron beams and diagonal braces that ran from the lowest level of the foundations (at the sub-cellar) to the top of the sixth floor, with thin masonry curtain walls supported at every floor, (2) a five-story bearing wall building sitting on top of the frame, with its walls entirely supported by girders at the seventh-floor framing level, and (3) a continuation of the diagonal bracing into the upper bearing-wall portion of the building. [6] (See figure 3.) There is some dispute as to whether the original design included the bracing diagonals; modern analysis shows that the building would not have performed well without them.

It should be noted that Gilbert’s public statement of his logic for this building is incorrect. In 1899, Gilbert claimed that he “hit upon this thought: ‘The City authorities figure the height of a building from its foundation only; why can’t I run my foundation far up into the air and then begin my building? In other words, why can’t I build a structure to the eighth floor, which will be a composite [frame-supported] affair, and then build on top of that the rest of the edifice of three or four stories, thus using the foundation as part of the building, and fulfilling the city’s requirements?'” [3] The problem with this statement is that the building code clearly states that the height is measured from the curb line, not the foundation top, as shown in the wall-thickness diagrams accompanying the code text. [7] Since he made the quoted statement more than ten years after the design was completed, he may have misremembered, or he may have had some motive for claiming this story.

Two features mark this building as a hybrid: the bearing walls at the upper floors, and the use of cast-iron columns as part of a braced frame. The first is obviously an artifact of the transition from bearing-wall to skeleton framing. The second is evidence that the relationship between the structural engineering profession and building design was still in its early stages in the United States. Structural engineers at that time were typically concerned with civil structures (bridges, roads, railroads, dams, and similar works that were not buildable without engineering analysis and design) and would not become standard consultants during building design until the twentieth century. Most hybrid- and skeleton-frame American buildings of the nineteenth century had their iron and steel elements designed by the iron contractors, as at the Tower Building.

The floors were the standard used for large buildings in the 1880s, and often referred to as “fireproof”: flat terra-cotta vaults supported by and providing fire-protection for the wrought-iron floor beams. (See figure 4.)
Figure 3: Typical building section looking east.

Figure 4: Typical west wing floor plans.
3  FRAME ANALYSIS

The availability of only partial information on the actual built structure of the Tower Building prevents a completely accurate analysis. Instead, we have examined the parameters of analysis, using what was known of building frame analysis at the time of construction and what is known for certain about the frame as built.

The layout of the frame is a rational one given the architectural and construction constraints. The use of cast-iron columns was still standard in New York in 1889 and would gradually diminish over the next fifteen years. Since it was well known that cast iron could fail without warning if loaded in tension, the use of moment connections for lateral bracing was not realistic. In such a narrow building, the presence of the public and private hallways along the north wall limited the width of the bracing “trusses” to approximately 60 percent of the building width. The south chords of the trusses were the cast iron columns embedded in the south wall; the north chord was an intermediate vertical member built-up of wrought-iron angles; and the north-wall cast-iron columns were not part of the bracing frames.

Since there were no rock anchors or similar methods of tying down columns against uplift forces, both the south-wall columns and the intermediate vertical had to be tied into the masonry walls to allow the dead weight of masonry walls and floors to counter wind uplift. Since the south columns supported the 7th floor girders that carried the upper bearing walls and were tied into the thinner curtain walls below, they were weighted against uplift without any special details. The intermediate vertica lefts had no such direct tie to gravity load, which appears to be the reason that the bracing pattern changed below grade. In the subcellar, the intermediate verticals’ loads were distributed to the side foundation walls through a change in the bracing pattern, so that there was no need for foundations in the center, where they would be isolated from the masonry weight.

3.1 1887 NYC code

The governing regulation for the design of the Tower Building was the 1887 building code of the city of New York. This code was largely concerned with the construction of low-rise residential buildings, which were the vast majority of buildings constructed at that time. The discussion of structural analysis and design is extremely limited with, for example, floor loads specified only for three broad classes of buildings and wind loads not specified at all. [8] No engineering design data is provided, but rather the reader is directed to use “Trautwine’s Treatise for Engineers,” a common handbook of the era, or whichever textbook is used in the engineering classes at the U.S. Military Academy at West Point.

The dead load of the building was estimated using the known geometry and materials (for example, assuming common brick for the masonry walls) and estimates where all data is not known (such as estimating the flat terra-cotta tile arch floors at 80 psf (3.8kPa)). There was no stated live load for office use in the 1887 New York Code, but rather a 75 psf (3.6kPa) minimum load for all buildings, 120 psf (5.7kPa) for places of assembly, and 150 psf (7.2kPa) for “manufacturing or commercial purpose[s]”. The rental office space in the Tower would not be classified as public assembly, but was probably designed for the commercial loading. [8] The 1892 revision of the New York code included office occupancy at 100 psf (4.8kPa) and roof snow loading at 50 psf (2.4kPa). [9] There is no wind loading mentioned in the New York code, but Trautwine provides a design wind load of 40 psf (1.9kPa) on portions of the walls exposed above neighboring buildings. [10]
New York's Tower Building

Trautwine provides several different formulas for computing the allowable stress in both cast and wrought iron. For tension in wrought iron, the ultimate stress given vary from 45 ksi (310 MPa) to 76 ksi (524MPa), but the most clear statement is for allowable load: “...good iron bar should not be trusted permanently with more than about 5 tons per square inch... [69MPa]” For compression in wrought iron, the values even more widely because of uncertainty about safety factors: the statement “for safety take [the ratio of allowable to ultimate stress] from 1/3 to 1/8, according to circumstances” makes pinning down a single allowable stress impossible. Ultimate compressive stresses range from 30.9 ksi (213MPa) (L/r = 90, “flat ended”) to 7.2 ksi (50MPa) (L/r = 300). Finally, the ultimate compressive stress in square cast-iron columns with flat ends with a length of 12 feet (3.7m) and a 20 inch (0.5m) side is 66 ksi (455MPa), leading to an allowable compressive stress of 13 ksi (90MPa). [11]

3.2 2008 NYC code

The current New York City Building Code, issued in 2008, is a local adaptation of the International Building Code now used in nearly all jurisdictions in the United States. [12] The provisions of this code were used as written with the exception of the seismic loading provisions. Seismic loading was excluded because (1) it was not used at all at the time of construction and (2) it may be neglected for existing buildings in New York under current code as long as there is no expansion of the building. If the Tower Building still stood, it would be exempt from seismic analysis unless it was expanded, so an analysis of its original configuration is exempt from seismic analysis. [13]

The dead load used for modern analysis is obviously the same as that used in the replicated 1887 analysis. The floor live load is the 50 psf (2.4kPa) used for offices, which was also used for roof occupancy. Current snow loading is 23 psf (1.1kPa), using the ASCE 7 method specified in the New York code and therefore over-ridden by the roof occupancy loading. Wind load, per ASCE 7 and the New York code, varies from the minimum allowable value of 20 psf (1.0kPa) at grade to 24.5 psf (1.2kPa) at the roof elevation. Under current code, wind load is applied at all floors, including this shadowed by neighboring buildings.

The most interesting aspect of the modern code loading is that it is significantly lower than Trautwine's recommendation of 40 psf (1.9kPa). The total 2008 wind load is less than 60% of the total 1887 Trautwine wind load.

Finally, the modern code does not address allowable stresses in wrought or cast iron. If wrought iron is treated as a weaker form of steel, current code would allow tension of 0.6 times the yield stress, or roughly 15 ksi (103MPa). The best current method of evaluating cast-iron columns as used in American practice provides provides a critical LRFD compressive stress of 17 ksi (117MPa) for all columns with slenderness less than 108 (the cellar and subcellar columns have slendernesses of 18), to be used with a strength reduction factor \( \varphi = 0.65 \) leading to a design stress of 11 ksi (76MPa) to be used with factored loads. [14]

3.3 Method of frame analysis

The west-wing building frame was extremely simple and lends itself to 2-dimensional analysis. Wind load in the north-south direction was resisted by seven braced frames consisting of the a cast-iron column at the north wall, a cast-iron column at the south wall, a wrought-iron intermediate vertical truss chord between (located 13 feet north of the south wall), a wrought-iron girder at each floor level, and wrought-iron diagonals (running from the south column to the intermediate truss chord) that alternated direction at each floor. (See Figure 3 for the bracing layout.) The bracing greatly resembles Gilbert’s description of it as “an iron bridge truss stood on end” with the actual bracing confined to the southern 13 feet (4.0m)
of the wing, where the diagonals were hidden within room partitions, and the diagonal-free northern 8 feet (2.4m) including the public hallway. [15] Given the small size of the truss diagonals, which were all double or quadruple angles, with no individual angle larger than 6 inches (0.15m) by 6 inches, the typical steel-truss-analysis assumption – that the truss members carry only axial loads, with any minor moments released by plastic bending near the connections – is accurate. The bracing truss was therefore analyzed as pin-connected, even though the braces were connected by multiple bolts to bent-plate connectors riveted to the floor beams. [6]

The analysis of a pin-connected, single-diagonal Warren truss, even one that is cantilevered rather than simply supported, is easily accomplished using the method of sections. This method has the advantage that it is the method that would have been used by the designers of the Tower Building and the frames of similar truss-braced buildings of that era. The gravity load in the cast-iron columns was computed separately and added to the wind-induced forces.

3.4 Results

Analysis of the truss frames gives two sets of results: the stresses in the members and the allowable stresses under the 1887 and 2008 codes. Both sets are inherently inaccurate due to missing information. For the member stresses under load, we do not know what wind load was used in analysis, whether Trautwine’s 40 psf (1.9kPa) or some other (probably lesser) amount; we do not know all of the member sizes; we do not know the exact floor dead load; and we do not know what floor live load was used. For the allowable stresses, we have contradictory information for cast and wrought iron in the 1887 sources and none in the 2008 code.

The most heavily loaded members are the cellar and sub-cellar braces and columns. If we use the worst case 1887 loading – 40 psf (1.9kPa) wind, 120 psf (5.7kPa) live load – the stresses in the braces are between 14 ksi (97MPa) and 16 ksi (110MPa) and the maximum (gravity plus wind) stress in the columns is 14 ksi (97MPa). The best case loading is that in the current code – 25 psf (1.2kPa) maximum wind, 50 psf (2.4kPa) live load – leads to brace stresses of 9 ksi (62MPa) to 12 ksi (83MPa) and the maximum stress in the column is 12 ksi (83MPa). The best case could be improved by taking advantage of two analysis techniques not in use in 1887: live load reduction, and allowable stress increase for combined gravity and wind load. Both are based on reducing the effect of low-probability events, respectively the presence of full live load on a large area of floor or on multiple floors at once and the presence of full live load simultaneously with full wind load. The amount of benefit from these two techniques varies, but live load reduction alone could reduce the column compressive stress by as much as 15 percent; the reduction of the combined live and wind load stress by 25 percent would have a similar effect. [12]

The total gravity load in the cast iron columns was roughly three times the maximum wind force (tension or compression) in the columns and the dead load in the columns was more than twice the wind force, meaning that the cast iron was never subjected to tension from wind loading. (Using the modern code, the maximum column dead load was 785 kips (3.5MN) while the maximum wind tension was 342 kips (1.5MN).)

In short, without access to accurate information about the original design that is apparently not recorded, there is no way to be certain if the design met the standards of its day for stress in both the wrought and cast iron. However, it can be stated that the apparent gap between actual stresses and allowable is relatively consistent, indicating that any design inadequacy relative to the 1887 code was based on the loads and allowable stresses used rather than an irrational design or improper analysis of the bracing trusses.
4 CONCLUSIONS

The 1880s were still an era of empirical design in the United States. The New York City Building Code of 1887 allowed bearing-wall buildings of any height to be constructed without an engineering analysis of either gravity or wind load, relying on a linear increase in wall thickness to ensure adequate masonry to resist the stress. None of the buildings that are known to have met the provisions of this code are known to have failed, but that is simply because the rules were conservative, and kept the masonry stresses low. Most known failures of completed buildings of that era concern design deviations from the requirements or construction errors. The code made little provision for the partial skeleton frames already in construction and none for the skeleton frame buildings that followed the Tower, starting in 1890. This situation was the logical outgrowth of the mid-1800s, when no engineering design at all was needed for the low-rise, masonry-walled and wood-floored buildings that made up nearly the entire country. In the few cases where iron beams were used, the engineering design was performed by the iron contractors.

Structural design for American buildings in the 1880s was, not surprisingly in those circumstances, rather simple. The most important portion of the Tower Building’s frame, famous at the time and in early histories of skyscrapers, was a single-diagonal Warren truss, with cast-iron used for members that known to be in compression and wrought iron used for those that would be subjected to tension. Double-diagonal trusses would have been stiffer in the Tower Building, but would have been more difficult to build because of the crossing connection. Since the truss was analyzed only for stress, and not for deflection, the additional stiffness was not an observable benefit, while the additional cost of the more complicated connection was certainly visible in a speculative venture such as this.

Within these constraints, the design of the frame appears to have made sense both by the standards of its time and the standards of ours. The truss members appear to have been sized to give a consistent maximum stress, which is both logical and efficient. The potential for catastrophic failure of cast-iron stressed in tension, which was already known in 1889 and would become a major debate in the American engineering community in the following decade, was avoided through the top-heavy configuration of the exterior walls, which increased the dead-load compression in the cast-iron columns. [16] While the building had a short life because it became economically obsolete – it was replaced with a wider structure that had larger floors that could be more easily rented for a higher price – its structure performed, which is all that an engineering analysis demands of it. (See figure 5.)
REFERENCES

[1] n.a., *Architecture and Building*, v12 n9, March 1890.


