THE NEW YORK CRYSTAL PALACE: 
IRON STRUCTURE WITHOUT ENGINEERING

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ABSTRACT

The New York Crystal Palace was constructed in 1853-4 in imitation of the London Crystal Palace of 1851, and held an exhibition imitating that housed in the London building. The New York building was an octagon in plan, with two usable levels and a high roof surmounted by a large dome. It was the first large building in the United States to have all of its major structure constructed of iron: the columns were cast iron, the floor structure was a combination of cast-iron wrought-iron built-up girders supporting wood joists and planking, and the building skin was plate glass and sheet iron.

There was little formal engineering design of the iron structure. Rather, the architects created a framing layout and the various ironworks involved provided members to fit. During the building’s brief life – it was destroyed by fire in 1858 – there were reported problems with the structure that led to ad hoc repairs and reinforcing. This paper will analyze the major elements of the structure using (a) the techniques available in the United States in 1853, (b) more advanced iron-analysis as was available in the late 1800s, and (c) current analysis methods. The analysis shows that much of the structure was not properly designed for full loading and that there is little correlation between the results of analysis and the more empirical performance of various building elements.

Keywords: Cast iron, Wrought iron, Column capacity, Beam capacity

1. HISTORY OF THE BUILDING

1.1. Introduction

The New York Crystal Palace project was a literal imitation of the London building and exhibition. The Board of Directors of New York’s “Association for the Exhibition of the Industry of All Nations” stated immediately after its formation in 1852:

It is scarcely necessary to say, that the idea was suggested by the brilliant success that the attended the London Exhibition of last year... no nation gave more striking proofs of intellectual capacity and vigor, applied to the useful arts, than were manifested by our own people.

It was, therefore, a natural suggestion... that we should not only wish to see a like Exhibition in our own country, but that we should desire to re-produce in it the beneficial effects that had resulted from its great prototype [1].

The New York exhibition was planned as a copy of the London exhibition, and was to be housed in a building that would be a partial copy of the London building. The city government provided land in “Reservoir Square,” waste land next to a large above-ground distribution reservoir, contingent on the exhibition building being constructed of iron and glass. The exhibition itself was organized by a for-profit private corporation [2].

New York became an innovative center in iron and steel construction during the era of the “fireproof building” starting in the 1870s. In 1853, the use of metal as building structure was still new and still partially experimental. Cast iron was used first in fire-shutters and window frames, then storefronts, and finally entire facades by the late 1840s. Cast iron facade use grew steadily in the United States until the Chicago fire of 1871, with New York’s James Bogardus and Daniel Badger as leading promoters of the idea. Wrought iron developed later, with the first use of I-beams rolled specifically

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for use in buildings (as opposed to railroad rails) in 1854 [3]. Before then, industrial wrought iron production was limited to plates, angles, rails, and small channels. The structure of the New York Crystal Palace therefore consisted of cast-iron columns and beams, and wrought-iron girders built up of angles and plates, all supporting wood-joist floors and roofs.

1.2. Construction

The building design, by architects George Carstensen and Charles Gildemeister, respectively Danish and German immigrants, had a two-story greek-cross core with one-story triangular infill spaces between the cross arms to create an octagonal plan. The width of the building each way was 111 m and the central dome was 45 m high at a time when the tallest structure in the city was the 86 m tall masonry spire of Trinity Church [4].

![New York Crystal Palace](image)

The frame totaled 1360 tonnes of iron, a small percentage of which was “moorish” ornament but most of which was base structure. The columns, typically placed on a 8.2 m square grid, were octagonal in section and hollow. The exteriors were identical, with the octagonal exteriors nominally 203 mm in diameter; the wall thicknesses varied with load.

The columns of a typical bay supported cast-iron girders (at the upper floor and roof) in the form of double-diagonal trusses, which supported wood joists and wood-plank decks. The main north-south and east-west aisles were 12.6 m wide and mostly open to arched roofs three levels above, but the upper floor extended across the aisles at the end bays. The upper-floor long span across the aisles was supported on wrought-iron girders in the form of double-diagonal trusses composed of angles and plates. The curved roofs over the aisles, the central crossing dome, the vertical side walls, and some portions of the flat roof were glazed in small panes coated with enamel to reduce transparency [5].

While there were various municipal laws controlling construction, there was no building code in force at the time of construction, there were no engineering or construction non-governmental regulations or regulatory agencies, and there was no consensus in the country as to engineers’ role in building design. The architects specifically acknowledged the assistance of Julius Kroehl, a German-born New Yorker who worked as an inventor and engineer, in the iron design, but it should be noted that American engineering in 1853 was quite primitive.
Fig 2. One-quarter framing plan of upper level at 1:500. Note that (a) the building is symmetrical about both centerlines and (b) the hatched area is low roof, not floor. Beams marked A, B, and D are cast-iron trussed girders, beams marked C are wrought-iron trussed girders, columns are indicated by circles.

Fig 3. One-quarter framing plan of main roof at 1:500. Note that the building is symmetrical about both centerlines. Beams marked B and D are cast-iron trussed-girders, beams marked C and F are wrought-iron trussed girders, beams marked E are cast-iron arches, beams marked G are wrought-iron dome framing columns are indicated by circles.

1.3. Demolition
The Crystal Palace was destroyed by fire on October 5, 1858. The heat damaged the building and its contents beyond repair or reuse. Despite the adjacent reservoir, firefighters were unable to stop the fire, in part because of a lack of water pressure in the nearby hydrants [6]. The fire burned only the flammable portion of the building contents and wood flooring, but the associated heat destroyed the
building through thermal stress in the glass and cast iron, and warping of the wrought iron. By the end of the fire, the building frame was collapsed on the ground.

Even before the fire, the exhibition was considered to have failed in two important regards. First, the exhibition was a business failure. Second, the building proven to be flawed both architecturally and structurally. The complex roof leaked, a serviceability flaw that the architects blamed on changes made to their design for the roof drainage by the supervising engineer. Canvas was strapped over the outside in an attempt to stop the leaks but was both ineffectual and damaging to the building’s appearance.

The structural flaw found at the time was more serious and was addressed by the supervising engineer: the architects state that the wrought-iron truss girders showed “defects in their lower flanges and the diagonals” under a test live load of 4.0 kPa. It is unclear what the word “defects” means in these circumstances, although out-of-plane buckling of the compression diagonals was likely one piece of it. Visible excessive deflection or warping is a sign of potentially catastrophic overstress, even if no numerical analysis was performed. The general solution, installed before the building opened to the public was to tie rods from the top of the aisle columns (at the roof main spring-point of the aisle arches) diagonally down to the first panel points of the trussed girder roughly 2.5 m from the girder ends. This reduced the clear span by 40% and solved the problem to the satisfaction of the supervising engineer. Four of these girders were reinforced through the introduction of a tension rod installed below the lower chord of the truss in the manner of a tension-rod girder.

2. ANALYSIS

2.1. Analysis Context

Since there was no governing code at the time of construction and there are no records of any engineering analysis performed, it is not possible to accurately reverse engineer the original design. Assuming there was an analysis performed, which is not entirely certain, we do not know what loads, ultimate stresses, or allowable stresses may have been used. Since neither cast nor wrought iron is recognized as a structural material by current codes, it is also not possible to perform an entirely modern analysis.

Given these constraints, we have attempted an analysis of three critical elements of the structure using the known dead loads, a combination of current-code live loads and reduced live loads based on available information about use, values for ultimate and allowable stresses based on the literature of the era and the New York City Building Code for following decades that included iron use. Current code has a ground snow load of roughly 1.1 kPa at New York, as opposed to snow loads of 1.9 to 2.4 kPa used in earlier building codes. Given that the building had no insulation, was heated by iron stoves, and was at least partially occupied in the winter, the roof was likely warm enough to never collect much snow. Current snow load was used in analysis, as the absence of snow cannot be proven.

An interior floor load of 4.0 kPa was used in the analysis. This is somewhat less than the current-code value for a place of assembly, but exactly equals the test load used during construction and therefore is a load known to have been applied to the structure. The layout of the upper floor of the building, with relatively small aisle areas connecting exhibits, suggests that the lower number may be a more accurate approximation of the loading than an assembly load based on uninterrupted crowds. The three elements analyzed are a typical-bay column, a typical-bay cast-iron truss girder, and an aisle-bay wrought-iron truss girder. Lateral load analysis was not performed, but rather only gravity performance was checked. Despite the height of the central dome, the building was broad relative to its height and had hundreds of potential moment frame connections (the trussed-girder to column connections had top- and bottom-chord connection points, but see the discussion below in section 2.2), meaning that the frame moments and shears from wind would be small in any individual member.

2.2. Columns

Analysis of cast-iron columns was addressed by the New York City Building Code from 1882 (the first comprehensive code in the city) until the 1916 revision [7, 8]. During this period, the code-allowed stresses assumed concentric loading and used a straight-line formula to account for slenderness. It must be noted that the 1916 code significantly reduced the allowable stress in the formula, in reaction to several catastrophic failures and none of the known failures was caused by over-stress from primary loading, but rather were the result of foundation failures or eccentric overload.[9] Structural cast iron use is no longer allowed by American building codes, so there are no current design guides.
A typical-bay column had an unbraced length of 6.4m from the ground floor to the upper floor framing, and carried a total load of 424 kN. These columns were 203 mm octagons with 24 mm walls, and the resulting section gives a compressive stress of 31 MPa.

Fig 4. Original drawing (as reproduced in reference 4) showing column splice and lugs for a girder connection. Note that distortion is the result of reproducing the fragile original and the two-piece bottom lugs.

The only cast-iron column formula generally available in 1853 New York was that published by Eaton Hodgkinson in London in 1846, summarizing extensive research into the properties of iron [10]. It is, of course, not known if this reference was used in the design of the columns. Hodgkinson’s formula gives an allowable load of 370 kN. The fact that this value is inadequate is moot since the formula is restricted to columns with a maximum length-to-diameter ratio of 15, while the actual ratio is 31.5.

If we anachronistically use the 1882 New York Building Code, which was written by a generation of New Yorkers experience in cast iron use, we are referred to Trautwine’s Civil Engineer’s Pocketbook [11]. Trautwine has a maximum stress of 91 MPa for a hollow round cast-iron column with a length 32 times its diameter; applying the safety factor of six required by the city code results in an allowable stress of 15 MPa. The 1901 New York Building Code[12] contains its own formula, which leads to a resulting allowable stress of 57 MPa, which is adequate, but again the formula is restricted to a length-to-radius-of-gyration ratio of 70, while the actual ratio is 100.

In other words, the column stress is adequate by only one of the three formulas checked – that in the 1901 building code – but the columns used would be disallowed as too slender by that formula. It should be noted that the columns empirical performance was satisfactory, but it is unlikely that they were loaded to their design loads.

2.3. Cast-Iron Truss Girders
Analysis of the cast-iron trussed girder depends on whether or not the girders are considered to be connected at the top and bottom to the columns, providing end moment restraint. Our analysis was based on simple supports for two reasons: slop and breakage. First, the distance between the top and bottom end connections was fixed at 6 mm more than the girder depth. Since the girders would sit on the bottom connections, the bottom connections had friction to help resist movement in addition to the single bolts that connected the column connection lugs to the girders. The top connections were
fastened solely by those single bolts. Any misalignment or minor discrepancies in size (i.e., ordinary connection slop) would affect the top connection far more than the bottom connection. Second, the top connections were a single integral lug, which was more vulnerable to fracture than the larger, two-piece bottom connections. Fracture of integrally-cast lugs has been commonly observed in a cast-iron facade renovations, likely from incidental moments created by connection eccentricity. The analysis was performed using the same 4.0 kPa live load used for the test load during construction and for our analysis of the columns. The top chord of the girder directly supported the upper floor wood joists and was therefore subject to bending between the truss verticals. The maximum local bending stresses are 240 MPa in compression and 135 MPa psi in tension. However, the overall truss form of the girder creates an axial compression of 76 MPa in the top chord, leading to maximum combined stresses of 320 MPa in compression and 59 MPa in tension. The bottom chord has a maximum compressive stress of 22 MPa, the verticals have a maximum compressive stress of 27 MPa, and the diagonals have maximum stresses of 44 MPa in tension and 87 MPa in compression.

Hodgkinson gave ultimate stresses of 93 to 150 MPa in tension and 600 to 1000 MPa in compression. Trautwine provides similar values, and the 1882 building code prescribes a safety factor of 6 for compressive posts and tensile members, or in other words, all of the components of a truss. The code also provides a safety factor of 3 for “beams, girders, and all other pieces subject to transverse strain” so it is not clear that the safety factor of 6 should be used, but this safety factor was used in later codes for all cast iron analysis. The allowable stresses would therefore be approximately 21 MPa in tension and 100 MPa in compression; the 1901 building code states explicitly that the maximum tension stress was 21 MPa and the maximum compressive stress 110 MPa. The stresses calculated for the cast-iron trussed-girder are less than the generally accepted ultimate stresses, but considerably more than the allowable values using any of the sources. This analysis does not take into account any possible live load reduction (such as would be caused by fixed exhibition displays that weighed less than 85 psf and blocked human access to portions of the floor).

2.4. Wrought-Iron Truss Girders
Analysis of the wrought-iron trussed girders is similar in most respects to that of the cast-iron girders. However, the individual chords, verticals, and diagonals of the cast-iron girders all have tee or cruciform cross-sections, while the wrought-iron girders were built of angles and plates. The top chords were pairs of angles reinforced with pairs of plates, while the bottom chords of the wrought-iron girders were pairs of plates and the diagonals were single plates. This section implies that the girders were originally analyzed as simply-supported beams, since the bottom chord plates would perform badly under compression. The verticals were cast-iron tees, obviously intended for compression. In our analysis, those diagonals subject to compression were removed from the model, since they would buckle in service, shedding load. We reviewed the wrought-iron girders as originally built, and as modified after load testing by the introduction of diagonal rods to the columns tops. The top-chord bending stress from the local joist-support moments was 130 MPa. Axial loads provide a maximum compressive stress in the top chord of 400 MPa and tension in the bottom chord of 200 MPa. These axial-force stress values are not realistic, as they exceed the ultimate stress in wrought iron using any of the sources. When the reinforcing rods are added, changing a single-span girder to a three-span girder, the local moments in the top chord are unchanged. The axial-force stresses are reduced to approximately
NNOR compression or tension if we assume that wrought-iron yielding allowed moment redistribution. The earliest general reference is Trautwine, who provides ultimate stresses of 200 MPa in compression and at least 310 MPa in tension. Per the safety factor discussion in section 2.2, it is not clear whether the 1882 New York code required the use of a safety factor of 3 or 6, although 3 is closer to the values used later for wrought iron. Using either safety factor, the stresses are too high even with the reinforcing rods in place. The location of these girders provides one possible explanation for their adequate performance in service: they were used exclusively in two bays in each quadrant of the upper floor, connecting the stairs to the main portions of the upper floor where exhibits were located. These girder were loaded exclusively by people, not exhibits, and would only have reached full design loads if the floor were packed solid. It is possible that in the limited life of this building, such loading never occurred.

Fig 6. Original drawing (as reproduced in reference 4) showing wrought-iron girder from end to center panel in elevation and plan. Note that distortion is the result of reproducing the fragile original

### 3. CONCLUSIONS

New York grew rapidly in the second half of the nineteenth century, not just in population, but in the complexity of the built environment. In the decades after the Crystal Palace was built and burned, the use of first cast iron, then wrought iron, and finally steel would make possible buildings of exceptional height and with exceptionally long spans. The experience and expertise that led to the the skyscraper boom that began in the 1870s did not yet exist at the time the Crystal Palace was constructed. The design of this building was instead based on experience with cast- and wrought-iron bridge design, the only significant iron structures in the country at that time.

The Crystal Palace did not serve as a model for other buildings. The exhibition was a commercial failure and the known problems with the building created an impression that it was a constructive failure as well. Animosity between the architects and the board, resulting in part from the poorly-coordinated building process, showed that the design and construction community in New York was not yet ready for a building of this scale and innovative type.

It is difficult to say how well the building would have performed if it had not burned. The relative extremes of temperature in New York – average low temperatures in the winter of -3°C and average high temperatures in the summer of 29°C – would have caused the exterior portions of the building to “work” in repeated expansion and contraction cycles. This movement has damaged the connections in cast-iron facades in the city and would likely have caused more extreme damage in a building without other structure. More importantly, continued use over an extended period of time would almost certainly have included changes in load patterns that could have exacerbated the problems with the trussed girders. Analysis shows that these members were, under full load, overstressed by both the standards of the mid-to late-nineteenth century and the standards in use today. These members appear to have performed adequately during the building’s limited life only because they were not loaded to their full capacity.

The introduction of reinforcing for the long-span wrought-iron trussed girders during the course of construction shows the true nature of American engineering at that time: it was a mostly empirical process. Even though the trusses were designed by an engineer, the lack of standards for allowable material stresses, the lack of standards for loading, and the lack of generally-accepted analysis methods meant that these girders could be completely inadequate as designed. The experience that later led to the construction of impressive iron- and steel-framed buildings was, specifically, experience of failure of inadequate structures. In this context, the Crystal Palace is one of the early steps towards modern iron design in the United States.

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