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Analysis of archaic fireproof floor systems

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ABSTRACT: As engineers and builders developed modern steel framing in the late nineteenth century, the existing options for floors to span between steel beams were forms of masonry vaults. Many possible alternate floors were developed in the United States, but few had rational bases for design. Testing programs put in place by building officials in New York City promoted the use of certain systems in New York and, by providing a rationale for those systems, nationwide. Ten systems are described and analyzed.

Conservation and analysis of early modern structure often begins with the main structural material: we have one set of techniques for steel-framed buildings, another for reinforced concrete, and another for those buildings which are architecturally modern but are constructed using traditional wood and masonry. This focus is inevitable but tends to obscure secondary structural materials and systems, which include the framing for facade ornament, structural adaptations for mechanical systems, and floor systems. This paper will address the original design and current-day analysis of floor systems used in the United States between 1890 and 1930 to fill in between iron and steel beams.

Since iron-framed floors were first used in the late eighteenth century, floor systems have developed in parallel with the iron-framing development. A wood-joist floor is most often associated with wood plank flooring, and masonry vault structures provide masonry floors, but iron or steel plates were not used spanning between iron and steel beams. Instead, combinations of other materials have been employed between the beams. Because the designers and builders who used metal-framed floors were usually interested in increasing their buildings' resistance to fire, floor systems have typically been made of masonry and concrete.

Floor systems can be divided into three broad groups: arches, catenaries, and beams. These categories are, of course, the three basic methods of supporting vertical load across a horizontal span, but unlike complex structures such as bridges where two (or, rarely, all three) basic methods may be combined, floor systems tend to show only one method. That the earliest floor systems were arches is unsurprising since masonry vaults predate structural iron beams by thousands of years; it is far more interesting from the modern perspective that catenary systems were popular before beam systems.

There was no mechanism for the creation of national standards in the United States during 1880s and 90s because there were no national building codes or authorities. Various floor systems were promoted by manufacturers and contractors, and reviewed by local building officials who often had no technical training. Empirical load and fire-resistance analysis was used by some insurance companies, local building departments, and engineering schools. After 1896, the New York City Department of Buildings and Columbia University's School of Mines together conducted hundreds of tests on samples representing dozens of floor systems, beginning with systems already in use and inviting floor-system manufacturers to submit their products. Given the lack of national standards and the rigorous testing regime, the results of these tests were influential far beyond New York and long after the tests ended. For example, standard E119 of the American Society for Testing and Materials, currently used in the United States to determine fire-rating of floor assemblies, is closely related to the New York testing standard and can be used to corroborate some of the older results.

This paper will review the floor systems that dominated "fireproof" construction in the Untied States between 1890 and 1930 as well as selected less-common systems. For each system, analysis using current methods will be compared with available analyses from the era of construction and historic test data. Discussion will include implications for continued use and alteration in terms of design load capacity, detailing, and fire resistance.

1 HISTORICAL CONTEXT

Structural steel technology and skeleton framing of buildings matured in the United States between 1890 and 1910. At the beginning of this period, reinforced concrete was just beginning to be used and was still, to American engineers and builders, partly experimental. There was not a general consensus on reinforced concrete until the late 1910s. Given the late development of concrete relative to steel, designers and builders had already addressed the need for fireproof floors before concrete became common. The use of bar-reinforced concrete slabs never became the dominant solution for floors; after the invention of composite metal deck in the 1950s, designers and builders rarely used any other system. As a result, from 1890 to 1960, floor systems were used which are now unfamiliar to most engineers and builders.

As large steel-framed buildings were constructed in the 1890s, designers and builders became aware of an acute need for inexpensive, lightweight and "fireproof" structural floors. This need was not new: the Chicago fire of 1871 and the Boston fire of 1872 – large-scale conflagrations that had between them destroyed 18,000 buildings including much of both commercial downtown areas – had emphasized the need for buildings better able to resist fire. By the 1880s, various proprietary forms of terra-cotta tile arch floors had been patented and were in use, although all were functionally similar.

The word "fireproof," as used at that time, meant what is now called "fire resisting:" a building or a structural element that could withstand some degree of exposure to fire without failure. In this paper, "fireproof" will be used in that sense, not in its literal meaning of withstanding any exposure to fire.

As new ideas for fireproof floors were developed, building officials and insurance companies were forced to consider how floors built with new, often proprietary and patented technology, could be reviewed for approval. In the end, the various parties, including engineers, companies selling proprietary systems, and insurance companies, cooperated with officials to create standardized tests. (Hill 1895) Many of these tests were performed in the late 1890s and early 1900s by various agents under the supervision of the New York City Bureau of Buildings, in accordance with the requirements of the New York City Building Code at that time. (no author 1905)

The portion of the code that defined floor systems read: "All brick or stone arches, placed between iron floor beams, shall be at least four inches thick and have a rise of at least one and a quarter inches to each foot of span between the beams. Arches over five feet span shall be properly increased in thickness . . . or the space may be filled in with sectional hollow brick of burnt clay or some equally good fire-proof material, having a depth of not less than one and one-quarter inches to each foot of span, a variable distance being allowed of not over six inches in the span between the beams." Some version of this was in the New York code from 1891 to 1916. (Birkmire 1898, no author 1899, no author 1901)

The first clause allows for traditional brick and stone vaulting, which was rarely used because of the weight of the materials and the intensive labor required for construction. The second clause allows some segmental terra-cotta tile arches. The third clause, centered on the vague phrase "some equally good fire-proof material," and led to a proliferation of proprietary systems from manufacturers looking for a portion of the lucrative New York construction market. After several years of politicized review of various systems, the city Superintendent of Buildings, Stevenson Constable, arranged for a series of tests, beginning in 1896 and performed by the Bureau of Buildings in concert with Columbia University. (Freitag 1899)

1.1 *Testing*

The building code enacted in 1898 in New York allowed for brick and segmental tile vaulted floors. Other floors were addressed: ". . . or between the said beams may be placed solid or hollow burnt-clay, stone, brick, or concrete slabs in flat and curved shapes, concrete or other fireproof composition, and any of said materials may be used in combination with wire cloth, expanded metal, wire strands, or wrought-iron or steel bars; but in any such construction and as a precedent condition to the same being used, test shall be made . . ." (no author 1899) The test defined in the code consisted of three stages of loading on a completed panel of the floor in question: first, with the 7.2 kPa load in place, the floor was heated so that an average temperature of 927 C was kept for four hours; second, with the load in place, the floor was cooled by pressurized streams of water meant to imitate fire-fighters' hoses; and third, the floor was reloaded to 29 kPa. The physical integrity of the floor was observed at all times and deflection measurements taken during the third loading stage. (no author, 1897)

Because the New York code was interpreted literally, each variation on a system had to be tested separately. A floor system had to be tested for each possible load and span combination for which it was sold, leading to over 190 tests between 1895 and 1915. (Perrine & Strehan 1915)

Similar tests were used in other cities, by insurance companies in setting fire rates for buildings using new floor systems, and by builders and inventors attempting to prove the reliability of their systems to owners and local building officials. The fireproofing aspect of the tests is obvious, but in many cases there was no structural design for the floors beyond the empirical

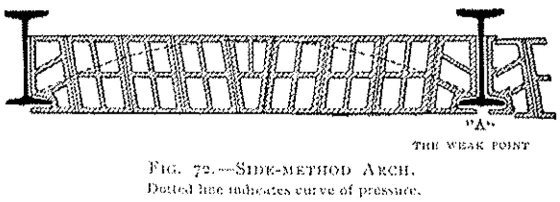

Figure 1. Side-construction Flat Tile Arch. The "weak point" refers more to fireproofing than structural load. (Birkmire 1898).

proof of supporting a high level of live load while exposed to fire. (Freitag 1899)

2 EXAMPLE FLOOR SYSTEMS

The floor systems described here are only a few of the many tested between 1890 and 1910. They include the most common floors and a few oddities that often confuse modern engineers.

The floors are grouped by type (arches, catenaries, and beams) and in the order that the types became popular. The earliest floor systems used with iron beams in the Untied States were arches. During the 1880s and 90s, various tile arch forms were the most popular floors in use, and were still used into the 1920s. The draped-mesh (catenary) floors developed in the 1890s became the most popular form in the late 1910s and 1920s, and were only permanently replaced by a beam-type floor when composite metal deck was developed in the 1950s.

2.1 *Arch floors*

Neither brick vault floors nor plain segmental terra-cotta tile arches were tested in New York, as both were allowed without limitation. They were typically not tested elsewhere, as they were considered ordinary and familiar floors.

Both generic and proprietary flat terra-cotta tile arches were tested in New York because neither met the geometric requirements for exemption. These systems consisted of precast terra-cotta blocks with thin webs, arranged as flat-arch voissoirs. There were two major types: side construction, with terra-cotta voids perpendicular to the vault span, and the newer end construction, with voids parallel to the span. (Figs 1, 2) The tiles were typically set low, to cover the bottom of the steel beams, and fill was placed over the top to protect the tops of the beams and provide a base for wood flooring. (no author 1897)

The Guastavino Timbrel Vault was an interesting variant on the standard terra-cotta tile-arch floor, using Catalan hard-burned tiles and thin-shell construction to meet the American testing standards. (Fig. 3) It was not popular for ordinary floors, although the

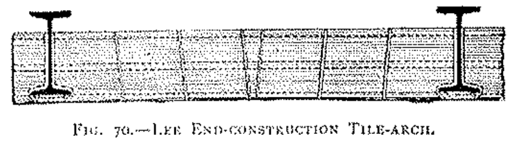

Figure 2. End-construction Flat Tile Arch (Birkmire 1898).

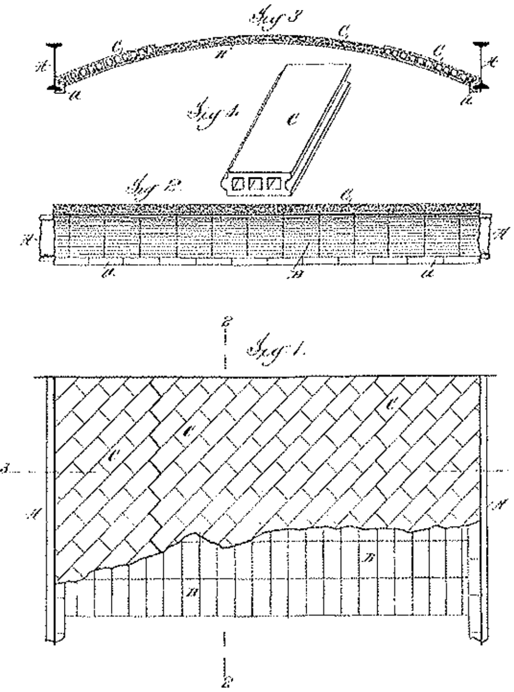

Figure 3. Uastavino section, elevation, and plan (Guastavino 1892).

Guastavino company achieved fame in the construction of domes and long-span floors, roofs, and ceilings. Two or three layers of thin tiles were arranged with staggered joints. (no author 1889, no author 1897, Freitag 1921, Collins 1982)

The Roebling Arch Floor is one of a class of early concrete floors produced by several manufacturers. (Fig. 4) Wire-mesh arches spanned between the beam bottom flanges and served as forms for stone concrete vaults. More wire mesh was hung from the beams to create a plaster ceiling for fireproofing. (no author 1897, Birkmire 1898, Hool 1913).

2.2 *Catenary floors*

The Metropolitan System was an early draped-wire floor, approved in 1899. (Fig. 5) The reinforcing consisted of twisted pairs of wires individually strung across the building, anchored at slab edges, and draping over the floor beams and under a hold-down bar at mid-span. Since the wires carried all loads, the slabs

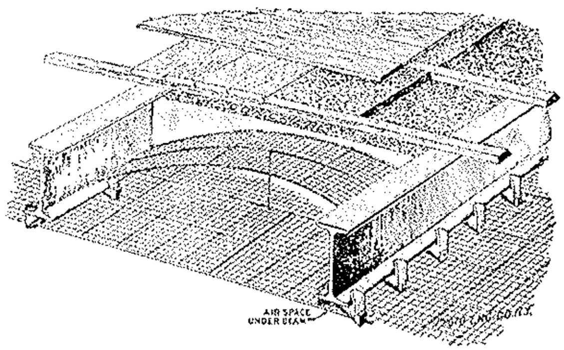

Fig. 66.—The Roebling Floor-Arch and Ceiling.

Figure 4. Roebling Arch: wood sleepers for finish flooring, concrete fill, wire-mesh arch, hung ceiling. (Birkmire 1898).

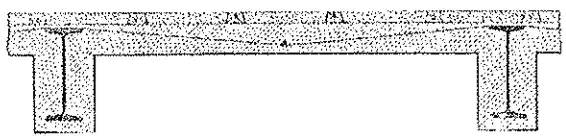

Fig. 71.—Metropolitan Floor—Panelled Construction.

Figure 5. Metropolitan Floor (Freitag 1899).

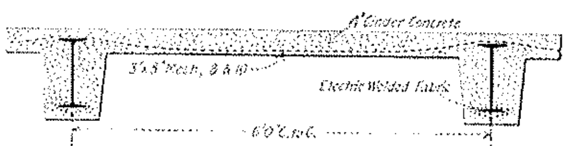

Figure 6. Cinder-concrete, draped-mesh slab, (Buel & Hill 1906).

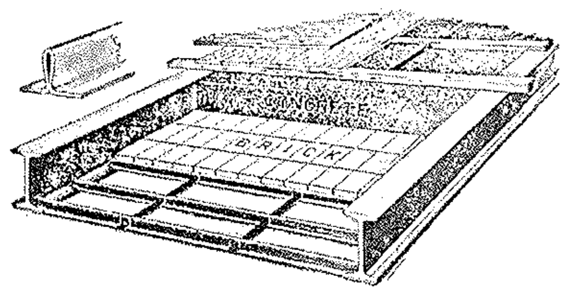

Fig. 76.—Rapp Fire-proof Floor-construction.

Figure 7. Rapp Floor: wood sleepers to attach finish flooring, fill ("concrete"), brick, and inverted Ts. (Birkmire 1898).

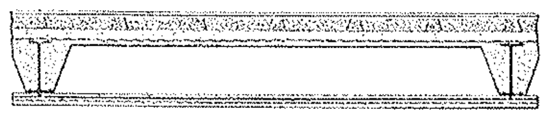

Figure 8. Expanded Metal Floor (Freitag 1899).

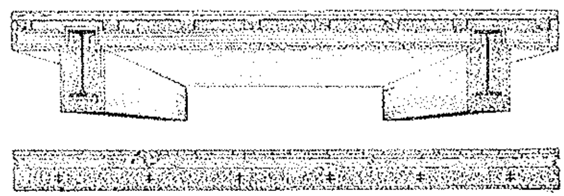

Figure 9. Columbian Floor, with wood flooring on sleepers, and rebar supported by light-gage straps over the beam tops. The bottom transverse section shows the double-cross bars. (Freitag 1899).

were structurally unstressed and were composed of gypsum plaster mixed with wood chips. (no author 1897, Freitag 1898, Birkmire 1899, no author 1895)

Draped-mesh floors were first developed under proprietary names, but the presence of the existing catenary patents meant that the basically similar mesh floors quickly became generic commodities. (Fig. 6) Various proprietary forms of mesh were used early on, but since the floor only required a specified standard size of wire at a specified spacing, the generic floors were ultimately more common than the named ones. As with the Metropolitan floor, the mesh passed over the top of the floor beams, near the top of the slab, and at midspan draped down to the bottom of the slab. Because the mesh wires take all of load in tension, the slab serves only to provide a walking surface and fireproofing. Because the concrete is not stressed, material of poor quality and low strength, such as cinder concrete, was frequently used. (Perrine & Strehan 1915, Waite 1914)

2.3 *Beam floors*

The Rapp Floor consisted of common brick, supporting a layer of fill and supported by light-gage steel inverted Ts, which span between the bottom flanges of the floor beams. (Fig .7)(Birkmire 1899)

The Expanded Metal Company Floor is a thin concrete slab reinforced with a sheet of "expanded metal," created by slitting a light-gage steel sheet and pulling the sheet to open the slits. (Fig. 8) The reinforcing sheet sat directly on the tops of the floor beams. The system had an overall slab thickness of only 5 inches, in part because the beams were at a close spacing. (no author 1897, Freitag 1899)

The Columbian Floor system was an early bar-reinforced concrete slab. Bars with a cross or double-cross section were hung from the beam tops using light-gage steel straps to serve as flexural reinforcing in a cinder-concrete slab. (Fig. 9) The typical beam encasement provided the only shear transfer. (no author 1897, Birkmire 1898, Hool 1913)

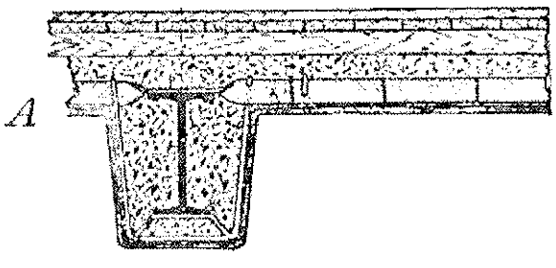

Figure 10. Roebling Flat Slab Floor, showing strap reinforcing twisted at the beam top. (Freitag 1899).

The Roebling Flat Slab Floor was basically similar to the Columbian floor, except that the reinforcing was rectangular bars set vertically and twisted horizontally to rest on the floor beams. (Fig. 10) (Freitag 1899, Hool 1913)

3 RE-USE AND ANALYSIS

Since the original use of these floors depended only on empirical testing, analysis came after the fact. For the arch floors, there was rarely analysis at all until the floors fell out of use, while justifications for the beam and catenary floors have changed over time.

3.1 *Current codes*

During the course of the twentieth century, numerical analysis replaced the empirical tests remaining in the structural design. Steel beam design had been rationalized in the nineteenth century, and concrete beam design after 1900. At the time of the floor testing, all American building codes were local, created by either municipalities or states. Regional and national codes were only established later, and have only allowed analysis-based structural design. In the case of New York, the 1916 building code removed the testing requirement in favor of analytic design. This code was revised but kept in use until 1968, when an entirely rewritten code was put into effect. All references to tile arches or other masonry floors were removed, but draped-mesh slabs remained as "Short-span concrete floor and roof construction supported on steel beams." The formulas for minimum allowable slab thickness and wire stress were the same as the old code, and values were given for stone concrete and unspecified-material "lightweight aggregate concrete." This code remained in effect, with revisions, until July of 2007, when it was superseded by a local version of the 2003 International Building Code, with a one-year overlap until the summer of 2008. The new code retains one last vestige of the old floors: a statement that wire mesh reinforcing "is permitted to be curved from a point near the top of the slab over the support to a point near the bottom of the slab at midspan, provided such reinforcement is either continuous over, or securely anchored at support." (no author 2004, no author 2007a)

The load analysis for the different floor types that follows is based on current codes except as noted.

Modern code analysis is somewhat simpler with regard to the fireproofing capacity of the old floors. Under current American practice, fire ratings for structural assemblies are determined using tests defined in the standard specification ASTM E119. The standard has been published and periodically revised by the ASTM since 1917 and carries no authority of its own, but is regularly adopted into building codes and other statutes. (no author 2007b)

The provisions of E119 are similar in many respects to that of the New York City tests. The floors are to be loaded "to simulate a maximum load condition . . . under nationally recognized structural design criteria," the heating is to follow an upward curve from 538 C at five minutes to 1093 C at 4 hours (and up to 1260 C at eight hours if a rating past four hours is sought), the floor is to be cooled by a pressurized hose stream, and the test report is to include information on damage and deflection. Since (a) the maximum design load on most floors is less than the 7.2 kPa used in the New York tests and (b) there are no standard uniform loads of 29 kPa as was used in the New York reloading test, the current loading criterion is less stringent than the New York tests. The total amount of heat to be absorbed in the early stages (as measured by the area of the time/temperature curve) is less in the ASTM tests than in the New York tests, the New York tests are more conservative for systems with low fire-ratings. At four hours, the total energy is nearly identical between the two tests, so that a system that passed the New York test would be assigned a four-hour rating using the ASTM test. Only when the tests are continued past four hours does the total energy in the ASTM test exceed that of the New York test. As few floor systems are now required to have ratings greater than three hours, it can be generally stated that any floor that passed the New York test has effectively passed the ASTM test and can be considered to have a four-hour fire rating if it has been maintained in its original state.

The most significant difference between the New York tests and the ASTM test is one of perception: the older tests were used as proof of both fire-resistance and structural load capacity, while the current tests are used only to determine fire ratings for assemblies assumes to have been already numerically designed for structural loads. This gap is less of a problem for existing buildings than it might appear, since the current building codes allow existing structure to be load tested to prove capacity, in a manner reminiscent of the historic tests.

The use of historic test data, common among engineers, has been given a government imprimatur in the form of a guide to fire ratings on historic structures compiled by the US National Institute of Building Sciences and published by the federal Department of Housing and Urban Development. (no author 2000) This guide is a compilation of historic and modern tests on various types of building elements, including floor systems. Gypsum slabs, terra-cotta tile arches, and tile and concrete rib systems are all included and assigned fire ratings of up to four hours. While the guide does not comprehensively address all of the floor systems of interest, it does provide an official opinion on some and it does show that the reuse of historic test data can be an officially-accepted method of research.

3.2 *Arch analysis*

Arch analysis is one of the oldest problems in structural design. In ordinary masonry practice, symmetrical and uniformly loaded arches are rarely dependent on the compressive strength of the material, but the terra cotta tile used in the floors was often weaker than stone or ordinary brick, and was used in thin webs vulnerable to local concentrations in stress.

Among the various formulas developed for masonry arches exists a class of simplified formulas for minor arches – those with short spans and low rises. In such cases, the formulas also include the assumptions for a flat arch of uniform loading and a specific location for the thrust line.

A simplified formula suggested while terra-cotta floors were still in use is $H = WS / 8D$, where H is the thrust, W is the total uniform load, S is the span, and D is the effective rise of the arch. This formula assumes that the thrust line curve was 61 mm less than the height of the terra cotta blocks, as D is the total arch depth minus 61 mm. (no author 1919) Note that the load and analysis are for a unit width of arch. In this case, the main simplifying assumption, represented by the use of D rather than the arch depth is that the thrust line curve is proportional to the block depth excluding the lower portion of the blocks which extend below the steel beams. For example, a typical application spanning 1830 mm between beams and consisting of wood flooring, 125 mm inches of cinder fill, 254 mm deep terra cotta blocks making up the flat arch, 12 mm of plaster on the block soffits as a ceiling, and an office live load of 2.9 kPa, had a total weight, including steel framing, of 6.8 kPa. This translates to an arch thrust of 15 N/mm to be carried by tie rods from beam to beam.

A typical terra cotta block had 19 mm thick webs at 100 mm spacing. Since the horizontal webs are oriented incorrectly to carry the arch thrust, the thrust must lie entirely within the vertical webs. With an effective web area of 254–61 or 193 mm by 19 mm, the compression in the terra cotta is .41 MPa, well within the allowable range for clay masonry.

A modern simplified formula for arch thrust used by the Brick Industry Association is $H = 3\,WS/8d$, where H is the thrust, W is the total uniform load, S is the span, and d is the arch depth. (no author 1986) Again, the analysis is for a unit width of arch. In this case, the main simplifying assumption, represented by the "3" in the formula, is that the thrust line is within the middle third of the arch height. This is more conservative than the older formula in that it increases the thrust force by confining its path. Using the previous example, the thrust is 34 N/mm and the net compression on the webs is 0.93 MPa, which is still within the capacity of clay masonry.

The formulas for segmental arches do not require the complicated assumptions regarding the location of the thrust curve: it is taken within the curve of the vault. Since the floor vaults based on segmental curves typically support fill above, a thrust line that extends past the vault material on the top side and passes through the fill may still be viable.

Using the Guastavino vault as an example of a segmental-arch floor, with a span of 1830 mm and a rise of 150 mm, the average total load is 7.9 kPa, with the extra dead load resulting the deeper fill over the arch ends. The basic thrust formula, $H = WS/8d$, gives a thrust of 22 N/mm. Unlike terra cotta vaults, the stress in a Guastavino vault is evenly distributed along the solid masonry section, so with a vault thickness of 75 mm, the compressive stress is 0.29 MPa.

Finally, the Roebling arch is the simplest of the type. The minimum arch depth is the distance from the floor to the top of the arch center, and the material is stone concrete, with a compressive strength in the range of 14 MPa, so an analysis similar to the Guastavino floor shows a gross over-capacity.

3.3 *Catenary analysis*

Analyses for catenaries with fixed supports were available in the late nineteenth century, both as general formulas based on statics and simplified formulas that became part of building codes and manufacturers' recommendations during the first half of the twentieth century.

One of the simplified catenary formulas that was publicized by the wire-products division of United States Steel was also incorporated in the New York City Building Code in 1916 (no author 1944) This formula is $W = 3CA_s / L^2$ where W is the total allowable floor load, L is the beam-to-beam centerline spacing , A_s is the unit wire cross-sectional area, and C is a constant representing the maximum allowable wire stress and equal to 138 MPa. The referenced versions of this formula refer to stone or cinder-concrete slabs; however, the results also agree closely with the stated capacities of the Metropolitan Floor's gypsum slab. It should be noted that the maximum allowable tension in the wire of 138 MPa is very low for steel wire produced after 1900. The minimum yield stress expected in historic wire systems has been conservatively established as 345 MPa. (no author 1981) Given that current reinforced-concrete codes have combined safety factors in the range of 1.5 to 1.8, it is evident that there is excess capacity in the wire in the original designs.

For example, in a 100 mm-thick draped-mesh slab with a 1830 mm span, the total load based on wood flooring, the cinder-concrete slab, 12 mm of plaster on the soffit as a finish ceiling, and an office live load of 2.9 kPa was 5.1 kPa. With mesh reinforcing of 8-gage (4.2 mm diameter) wire at 75 mm spacing, the maximum allowable load is 41 KPa.

A more basic formula derived from statics by approximating the catenary curve as a parabola (a reasonable assumption since the actual wire curve is not particularly accurate) is $T = (wsL^2 / 8\,h) + wsh$ where T is the tension in a wire, w is the combined dead and live load, s is the wire spacing, L is the span, and h is the wire sag. By setting T equal to the allowable tensile stress (T_a) multiplied by the wire cross-sectional area (A_s), the maximum floor load can be determined as $W = 8\,hT_aA_s / (sL^2 + 8\,sh^2)$. In the previous example, using the 138 MPa maximum stress in the wire and assuming a wire sag of 75 mm for the 100 mm slab thickness, the allowable load per square foot is 25 kPa.

In a Metropolitan floor example using the simplified formula, with pairs of 12-gage (2.8 mm diameter) wire at 25 mm spacing and a 1830 mm span, the total load capacity is 36 kPa. Using the basic formula, the total load capacity is 22 kPa.

In short, the catenary floors show the excess capacity required to pass the load test.

3.4 *Beam analysis*

Beam theory is generally more complex than arch or catenary theory as long as fixed supports are assumed. In addition, the design of reinforced concrete in the United States at the time of the testing was hampered by over-conservative allowable stresses. When the New York code was amended in 1911 to allow use of reinforced-concrete floors proved by analysis rather than by testing, the maximum compression in the concrete was limited to 4,5 MPa, the maximum tension in the reinforcing was limited to 110 MPa, and n, the ratio of the elastic moduli, was fixed at 15. (Waite 1914) Only elastic concrete theory was in common use at the time, and using the notation common for that theory, the location of the neutral axis from the compression face of the slab is kd, where d is the depth from the compression face of the slab to the centroid of the reinforcing and the ratio $k = (2\,pn + p^2n^2)^{0.5} - pn$, where p is the reinforcing ratio. The arm of the resisting moment (the distance from the centroid of the triangular elastic compression block to the centroid of the reinforcing) is jd, where $j = 1 - k/3$. The resisting moment is the lesser of the two calculated moments $M_s = A_sf_sjd$ and $M_c = f_ckjbd^2/2$, where M_s is the maximum resisting moment if the slab is under-reinforced and steel yield controls, A_s is the reinforcing area in square inches per foot, f_s is the allowable tensile stress in the steel, M_c is the maximum resisting moment if the slab is over-reinforced and concrete crushing controls, and f_c is the allowable compressive stress in the concrete.

For example, the code-enforced maximum stress values would show a 100 mm stone-concrete slab with 6 mm diameter bars at 150 mm on center, and spanning 2440 mm, to have a maximum allowable moment of 4.9 kN-m/m, or a maximum total load of 6.5 kPa, compared to the 7.2 kPa allowed from testing the same floor. This is not a hypothetical comparison: the Monier Floor System, which was essentially a numerically-designed, modern reinforced concrete slab as used in the in Europe, was tested for use in New York by 1900. If the current American Concrete Institute code and ultimate-stress theory is used on the same slab, assuming the 14 Mpa concrete often used in that era and a steel yield stress of 138 MPa based on the weakest available reinforcing steel bars, the maximum allowable moment is 8.5 kN-m/m, and, assuming that the dead load to live load ratio is approximately 1, a maximum unfactored total load of 8.0 kPa.

The Columbia and Roebling Flat Slab floors can be analyzed like any other reinforced slab as long as certain restrictions are observed: the bars must be converted to a cross-sectional area per unit width of slab, the slab strength must be adjusted to match the material, and the lack of deformations must be accounted for by a check on the development length of the bars.

The Expanded Metal Floor can be analyzed using ordinary reinforced-concrete formulas and converting the cross-sectional area of the expanded-metal sheet into the bar equivalent.

The analysis of the Rapp Floor, a beam-type floor that did not rely on reinforced concrete, is simpler. The bricks spanning 200 mm and the light-gage Ts spanning the beam-to-beam spacing can be checked for maximum stress using a simple beam formula.

4 CONCLUSIONS

The simplest result of the analysis is the conclusion that these floors can be demonstrated to pass modern requirements for fire resistance and load capacity. In the course of renovation projects, many engineers and contractors prefer the removal of unfamiliar and archaic structural elements rather than their reuse, even if no damage is visible. The explanation for this behavior is often "better to be safe than sorry," as if the reuse of existing structure that has functioned properly for decades is somehow unsafe. It must be emphasized that unknown structural capacity and fireproofing are not excuses for the wholesale demolition of historic fabric that often takes place, leaving a historic facade covering an essentially new building. It is incumbent upon the engineers involved to investigate unfamiliar structures and find methods by which they can be analyzed.

This is not to say that there are no difficulties in reuse once an archaic floor is analyzed. There may be damage to the floors, particularly with the more fragile systems such as tile arches, that reduce their load capacity or fire protective abilities. Modern seismic analysis of a building frame may depend on diaphragm action of the floor, which may or may not be available depending on the details of original construction. The need for a diaphragm implies a second investigation into details and a second analysis, not automatic disqualification. Finally, alterations accompanying reuse may require special details, such as providing new anchorage for catenary floors when new openings are cut for circulation or HVAC shafts. Some of the floors are particularly vulnerable to damage during alteration: the Ts in a Rapp floor can shift when an adjoining portion of the floor is removed.

In a broader sense, the continued viability of many of these floors emphasizes that the products of empirical design can still be used today. The rise of "scientific" numerical analysis in the twentieth century can obscure the value of older forms of engineering design, even as we still rely on some of the methods and data produced by those older forms.

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