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Beedle Lecture 2032: Composite Construction - Past and Future

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Abstract
This brief written contribution is meant to honor the memory of Dr. Lynn Beedle, an uncompromising, kind and generous mentor to several generations of structural engineers and researchers. I first met him at an SSRC meeting in New York City in 1989, and had numerous interactions with him in the next decade as he was a friend and collaborator of my colleague Ted Galambos. My interactions with Dr. Beedle led me to value three technical topics that he championed, and which constitute the broad themes of this paper. First, that we must recognize those who made initial significant contributions to a technical topic; too often we take their work for granted and do not make an effort to acknowledge and understand their contributions. Second, that we must have a thorough understanding of the development of current design provision in order to improve them; that means that sometimes we must break with the past and make a fresh start. Finally, that we must always be looking for new opportunities to improve and make steel and composite design more economical. Above all, however, I value the opportunity of having known Dr. Beedle and having been a part of the worldwide community that he developed and nurtured for over 50 years around issues of structural stability and tall buildings.

1. Introduction
The Spanish philosopher George Santayana's famous quote that "Those who cannot remember the past are condemned to repeat it" is not applied often to engineering endeavors. However, in the context of looking at the future of composite construction, it is useful to look back at how far this construction methodology has come in less than 150 years. The first section of this paper looks at some early examples of composite members and systems that designers may consider "modern" but which actually were invented quite early in the history of composite construction (1890-1940s). The second section looks at how these early experiences led to the development of the first design specifications and how those rules remain relevant today. Finally, the paper concludes with some remarks about future applications of composite construction.

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2. Highlights of Early Composite Construction

In his introduction to a book on composite construction (Viest et al., 1997), Viest notes that "the combined structural use of steel and concrete was first encountered almost as soon as the two materials became available to structural engineers." In fact, the earliest patent for a composite section was granted in 1808 to Ralph Dodd (1756–1822) for suspended floors: "malleable iron" tubes with "ears or flanges" were filled with "artificial stone" to form a composite beam (Pelker and Kurrer, 2015) . It is probably impossible to determine when the first use of reinforced concrete and composite construction occurred, but in the USA its first application to residential structures is often credited to the Ward House in Port Chester, N.Y., built in 1877 (ACI, 1975). The construction included the use of small I beams as floor reinforcement as well as the use of 3/8 iron rods. Applications to commercial and industrial buildings followed quickly, initially in the form of concrete overlays of structural steel members for fire resistance. Great attention was being paid to that issue in the 1880s due to a number of disastrous fires, including the 1871 fire in Chicago and the 1872 fire in Boston (Wermiel, 2000). As a consequence, major cities routinely required some form of fire testing for innovative construction methods. Encasement of structural elements by terracotta, rubble masonry and concrete was the obvious solution at that time.

American engineers soon also realized that using concrete had substantial strengthening and stiffening effects on the structure, although they had no technical tools to clarify that contribution. Their European counterparts, however, already had available design equations for reinforced slabs with bars or steel sections as the result of the work of Mathias Koenen, who published his work in the *Centralblatt der Bauverwaltung* magazine in 1886 (Pelker and Kurrer, 2015). It is interesting to note that this work was focused on the problems of bond and slip and correctly identified the dangers of shear in reinforced concrete slabs and beams.

A major impulse took place with the first patent on composite construction in the USA granted to Josef Melan for a vault system for ceilings and bridges in 1894 (Figure 1). This construction technique, which Melan had originally patented in the Austro-Hungarian Empire in 1892, was quickly adopted and numerous bridges were constructed throughout the USA with the Melan process in the next 10 to 15 years (Figure 2).

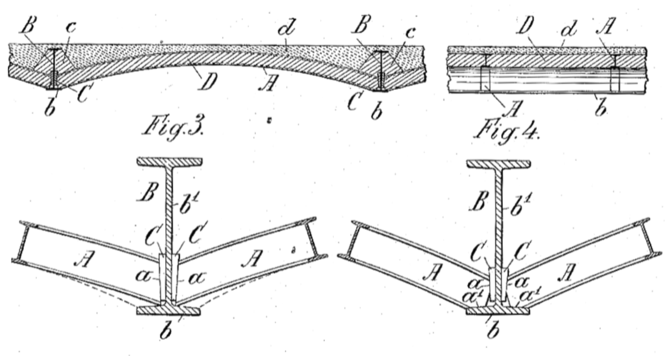
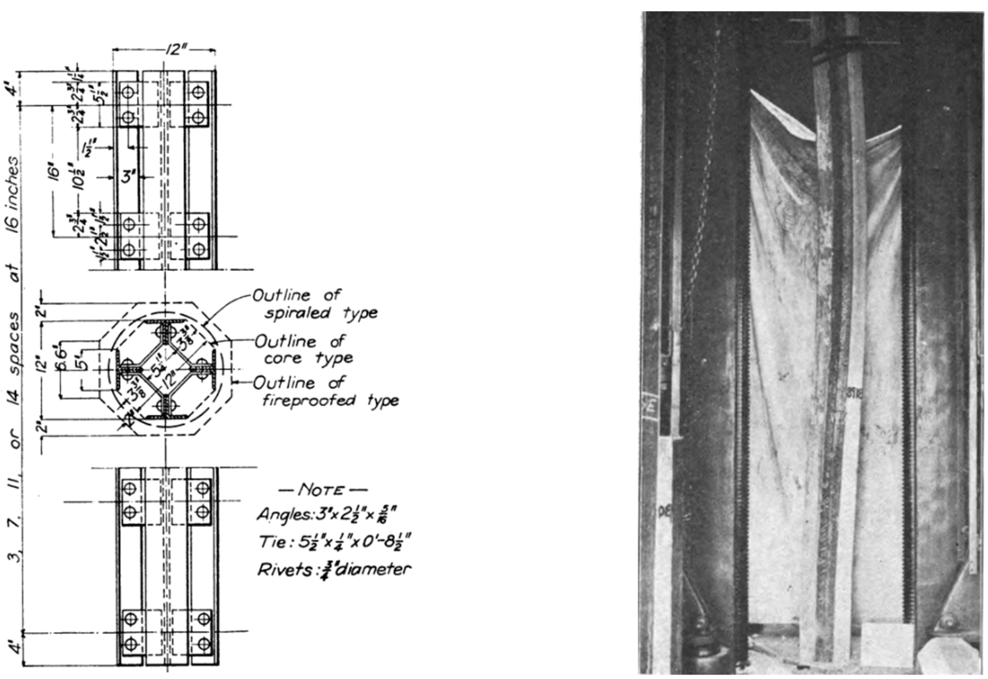


Figure 1 – Melan system for a vaulted ceiling (US505054A, 1894)

Figure 2 - First USA Melan bridge – West Broadway over Passiac River, Paterson City, NJ.

The contribution of any cover concrete or similar material to the strength and stiffness of a composite section was ignored in design until the seminal work of Talbot and Lord (Talbot and Lord, 1912) on composite columns (Figure 3). In the introduction to that work, they state their aim as resolving the issue that:

> *Two points of view seem to exist with reference to columns having a. large percentage of structural steel: (a). that the concrete surrounding the steel simply affords protection from fire and corrosion and that the additional strength afforded by the concrete is not considerable in amount and is not available for design; and (b) that if the concrete be present it must act in uni-son with the steel and that its strengthening effect and its effect upon the permissible deformation of the column should be taken into account. The present building codes either directly or through the relation of stresses allowed virtually occupy the first position when the steel column forms more than 8% of the column section.*



(a) Specimen geometries (b) Buckled specimen

Figure 3 – Tests by Talbot and Lord (Talbot and Lord, 1912)

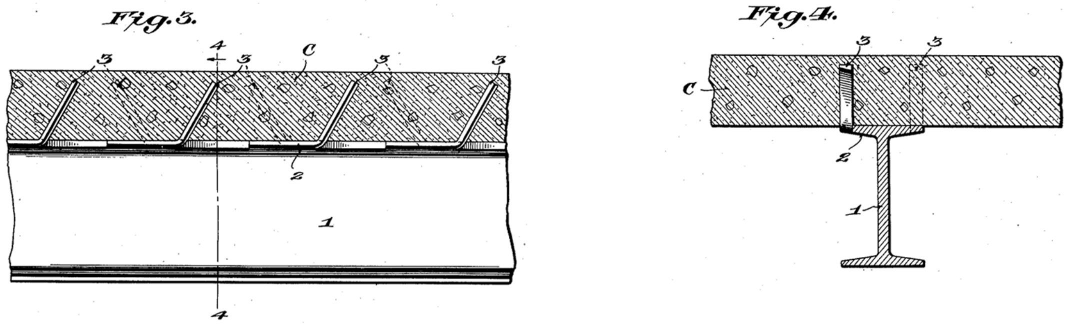


Figure 4 - Figures from Patent US1597278A granted on Aug. 24, 1926 to Julius Khan (TrusCon Corp.)

Truscon became one of the first and most important industrial building prefabricating companies in the USA and the world by inventing revolutionary construction products, such as the Hy-Rib floor system, perhaps the earliest forerunner to our current deck systems (Figure 5), as well as developing a formidable marketing team that had offices worldwide (Cody 2005; TrusCon 1957).

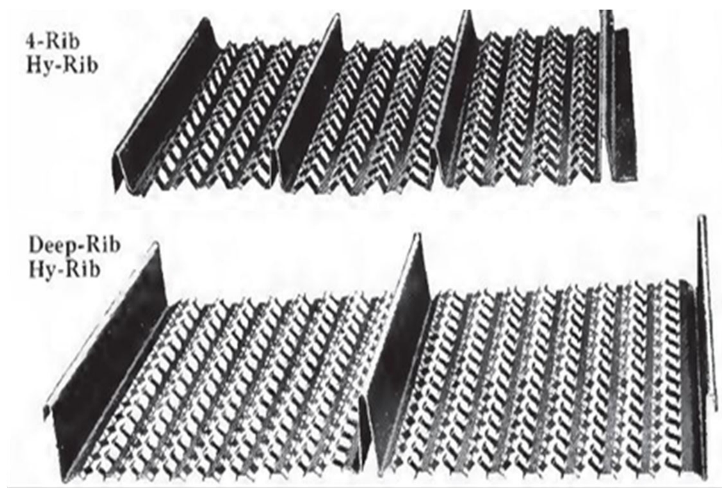


Figure 5 – Hy-Rib panels (TrusCon, 1957)

Steel buildings survived the 1906 San Francisco Earthquake and Fire almost undamaged, and the key role of any cementitious or similar fireproofing was duly noted by the experts (Duryea et al., 1908):

> *The steel frames were the least injured of any part of the various structures. Where properly protected, there was no injury. Where the protection was faulty, or where there was none, the destruction was complete.*

Following the earthquake and the recognition of the role of stiffness and mass in seismic performance, many codes were updated to require a 30 psf lateral load to take care of wind and seismic loads. That requirement lasted for many decades in USA codes.

Composite members were ideal to provide the additional required strength and stiffness, and by the early 1920s, it was common in San Francisco to use encased columns and beams in tall structures with brittle facades. Figure 5 shows one such building, at 450 Sutter Street building, having 26 stories, designed, and built in 1923. The beams and columns are all encased with 2 in. concrete layers and a minimal amount of transverse wire reinforcement. Construction of similar buildings became common throughout the USA in the late 1920s, including the Empire State Building in New York.

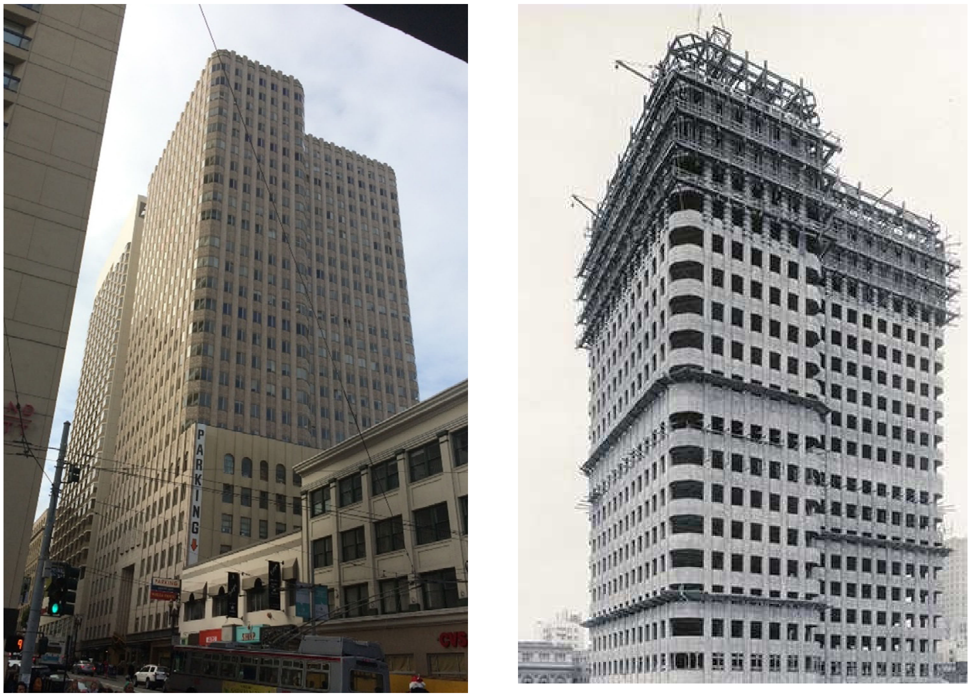


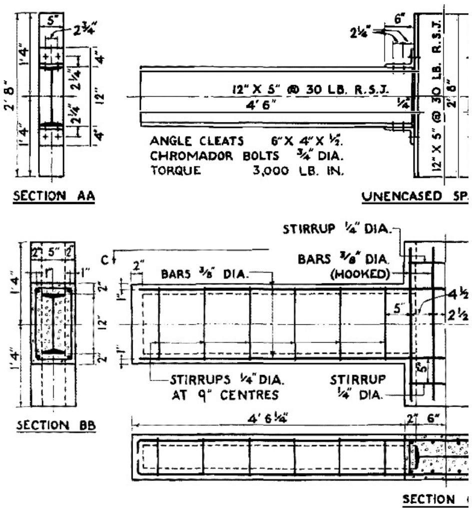
Figure 6 – 450 Sutter Street (today and under construction in 1926)

While private companies led the way in implementing composite construction, the need for code provisions grounded on a sound understanding of structural behavior led to a large number of important test series in the 1930s. Most important among these is the work of the British Steel Structures Research Committee, which carried out numerous full-scale experiments on unencased and encased beams columns and their connections. Figure 7 shows one set of results; the differences in strength are large, but even larger are those in stiffness, including in the unloading and reloading regime.

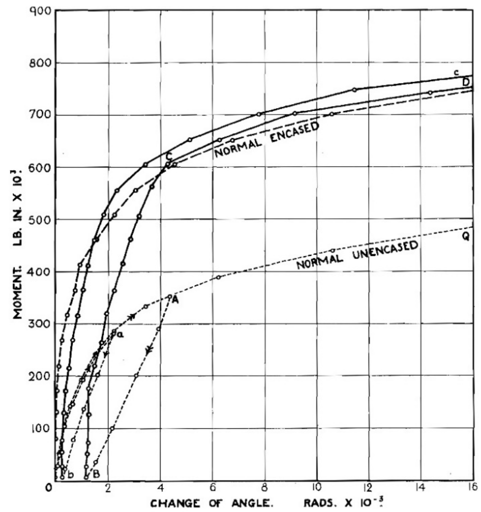
The care in planning and executing these experiments set a new standard in research. The magazine *Nature*, which even then was not given to hyperbole, noted that for this study (Nature, 1937):

> *In the structural trades, the rules (code provisions, n.b.) will be the ready measure of the Committee's achievement; but to those who understand the technique of data accumulation, analysis and reduction in large-scale work, the labours that lie behind them, and lead to them, will be significant and highly impressive.*

The results of those studies were reaffirmed 50 years later, when full-scale connections taken from the drawings for 450 Sutter were tested and analyzed (Roeder et al., 1996). Figure 8 shows the envelopes of cyclic behavior, indicating large strength and stiffness gains, with the minimal encasement sustaining drifts of 2.5% to 3% before the cover concrete began to spall. In the case of seismic design, ignoring this additional strength and stiffness is probably uncoservative, as it lowers the design forces (Forcier et al., 2002).

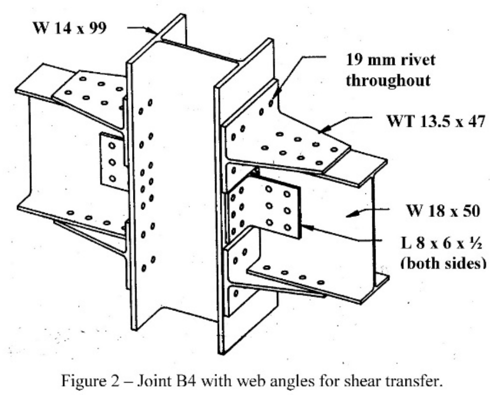


(a) Details of encased specimens

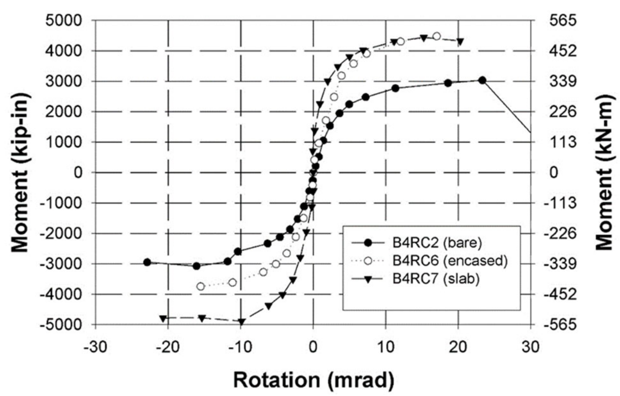


(b) Comparison of moment-rotation curves for unencased and encased specimens

Figure 7 – Effect of encasement on behavior of riveted connections (Batho, 1934)



(a) Riveted connection specimen



(b) Envelopes of cyclic response

Figure 8 – Comparison of behavior for unencased, encased and encased with slab specimens (Forcier et al, 2002)

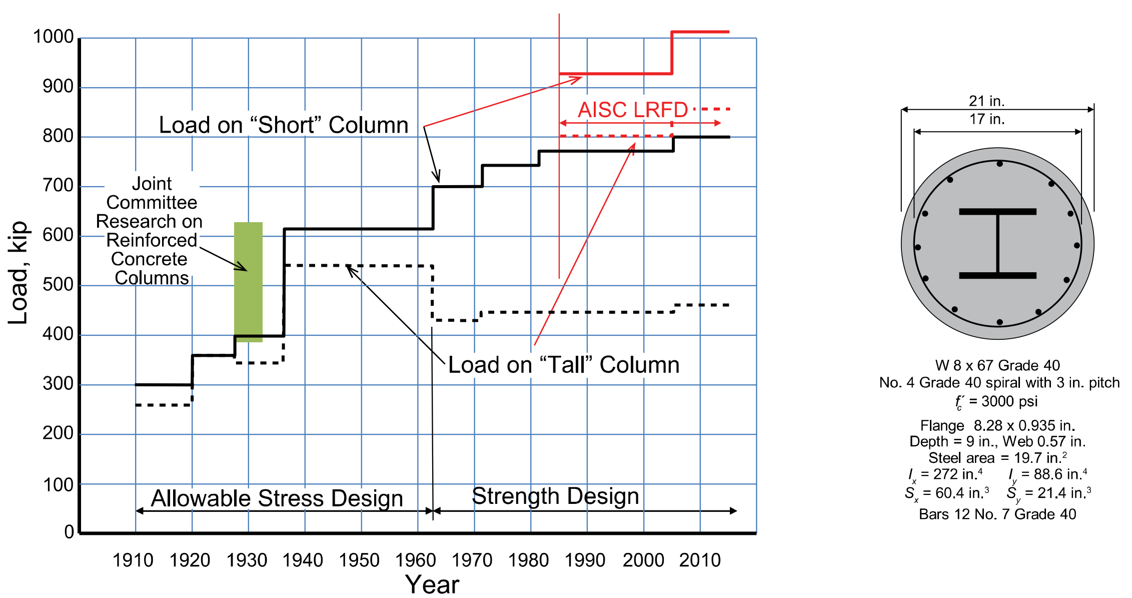
3. Design Specifications

The research on columns by Talbot and Lord, as well as tests on composite beams by TrusCon in the early 1920s (TrusCon, 1957), led to the development of design rules for composite members and their acceptance by major cities building departments. It is interesting to note that design by qualification testing was the primary vehicle to propel new developments in construction in the 1910-1920s, and that building departments in majors cities routinely required such proof of performance. Such type of acceptance, in the absence of mechanics-based design provisions, led to widespread application of composite construction.

The beneficial effect of steel sections embedded in concrete with confining hoops or spirals was recognized in the earliest NACU specification (NACU, 1910). Allowable working stresses for compression on columns reinforced with structural steel units which thoroughly encased the concrete core were set at 540 psi (3.7 MPa) for the concrete and 8100 psi (56 MPa) on the structural steel. NACU morphed into the American Concrete Institute in 1913, and the provisions for composite columns were updated in the second edition of its specification (ACI, 1924), with the following requirements:

> 170. The safe carrying capacity of composite columns in which a structural steel or cast-iron column is thoroughly encased in a spirally reinforced concrete core shall be based on a certain unit stress for the steel or cast-iron core plus ·a unit stress of 0.25 f'c, on the area within the spiral core. The unit compressive stress on the steel section shall be determined by the formula:
>
> $$f_r = 18{,}000 - 70\left(\frac{h}{r}\right) \leq 16{,}000\,psi$$
>
> The unit compressive stress on the steel section shall be determined by the formula:
>
> $$f_r = 12{,}000 - 60\left(\frac{h}{r}\right) \leq 10{,}000\,psi$$

where f_r is the stress in the iron or steel section, h is the characteristic dimension and r is the radius of gyration. The rules appear for application to encased rolled sections, but could have been applied to a filled section. Note that the 1924 provisions almost doubled the member compression capacity, but were still well below the usual material limits. These limits kept on increasing with each new set of specifications (Furlong, 2012a,b) as shown in Figure 8.



(a) Column strength changes with new code provisions (b) Column cross-section

Figure 8 – Evolution of circular encased composite column strength (Furlong, 2012b)

Provisions for composite beams were developed in the 1920, mostly as a result of the tests by MacKay (Mackay, 1923) , which looked carefully at bond and horizontal shear stresses. These tests included sections fully encased as well as with only top flange embedment. Specific design provisions for composite beams were first introduced in 1936, starting with its first formal definition of a composite beam as (AISC, 1936):

(a) Composite Beams

"The term "composite beam" shall apply to any rolled or fabricated steel floor beam entirely encased in a poured concrete haunch at least four inches wider, at its narrowest point, than the flange of the beam, supporting a concrete slab on each side without openings adjacent to the beam; provided that the top of the beam is at least 13^ inches below the top of the slab and at least 2 inches above the bottom of the slab; provided that a good grade of stone or gravel concrete with Portland cement, is used; and provided that the concrete haunch has adequate mesh, or other reinforcing steel,throughout its whole depth and across its soffit.

(b) Composite beams may be figured on the assumptions that:

1. The steel beam carries unassisted all dead loads prior to the hardening of the concrete, with due regard for any temporary support provided.

2. The steel and concrete carry by joint action all loads, dead and live, applied after the hardening of the concrete.

(c) Composite Beams.

The total tensile unit stress in the extreme fibre of the steel beam thus computed shall not exceed 20000 pounds per square inch. [Section 10 (a)].

(d) The maximum stresses in the concrete, and the ratio of Young's moduli for steel and concrete, shall be as prescribed by the specifications governing the design of reinforced concrete for the structure.

(e) The web and the end connections of the steel beam shall be adequate to carry the total dead and live load without exceeding the unit stresses prescribed in this Specification, except as this may be reduced by the provision of other proper support.

These provisions are copied here in their entirety to remind readers that design rules can be simple and generic. In particular, part (b) seems prescient, as we have not moved from those principles in almost 100 years. The author does not doubt that we could design 80% or more of current composite beams with similar concise rules. However, it should be noted that there is no mention of shear connection, as bond stresses were deemed sufficient to transfer forces between the steel and concrete portions.

These few, selective examples of early composite member applications are meant to remind younger engineers that they should spend time looking back at the history of their profession. The availability of this early material on internet sites such as the HathiTrust makes this an enjoyable task for evenings and weekends.

4. Current Research Needs

In a paper written for an SSRC conference more than 20 years ago (Leon, 2001), the author cited a number of topics on composite construction that needed to be better addressed in the Specification. Much of the discussion on that paper found its way, and remains almost verbatim, in the commentary to Chapter I of the AISC Specification. With respect to composite beams, the main topics were:

1. Lack of a clear discussion of why the LRDF Specification does not require a specific check on yield under construction loads.
2. Lack of provisions for checking shear capacity in concrete slabs, particularly for cases where the deck runs parallel to the beam.
3. Lack of provisions on minimum rotational capacity of composite beam sections.
4. Adequate attention to cambering issues.
5. Lack of reliable methods for calculating elastic properties for short-term serviceability calculations for composite beams.
6. Lack of reliable methods to account for long-term deflections for composite beams and columns.
7. Potential problems with beams with partial interaction.

In the author's view, these remain significant weakness in our specification. Of course, as the author served as chair and member of the AISC TC 5 – Composite Construction, he shoulders a substantial portion of the blame for the lack of progress. However, it remains a fact that the construction industry and federal organizations have not seen the need for sponsoring the research necessary to answer these questions. The principal reason for this lack of interest is that we have not seen any significant field failures that would spur such work. The inherent robustness of composite construction, as well as the conservative live load values assumed in design, has led to a negligible failure rate at both the strength and service limits, arguing against the need of this type of investment. However, a recent unexpected failure in the longest composite beam tested in the USA may change that dynamic.

As part of a much larger project on fire resistance of composite floor systems (NIST, 2019), five 42 ft. long, composite beams were tested (Figure 9). The beams were W18x35 with an 8 ft. wide LW concrete slab. The slab was 5.5 in thick on a 3 in. deck and the shear connection was provided by ¾ in. studs at 12 in., which provided upwards of 80% composite action.

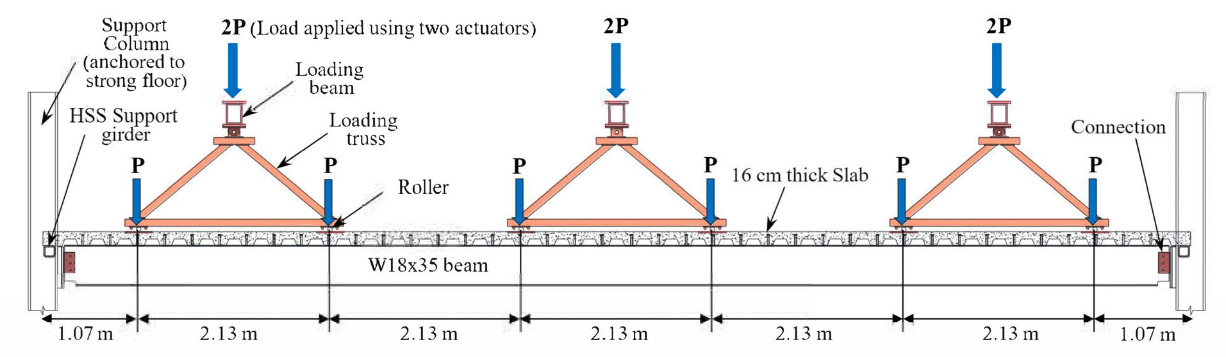
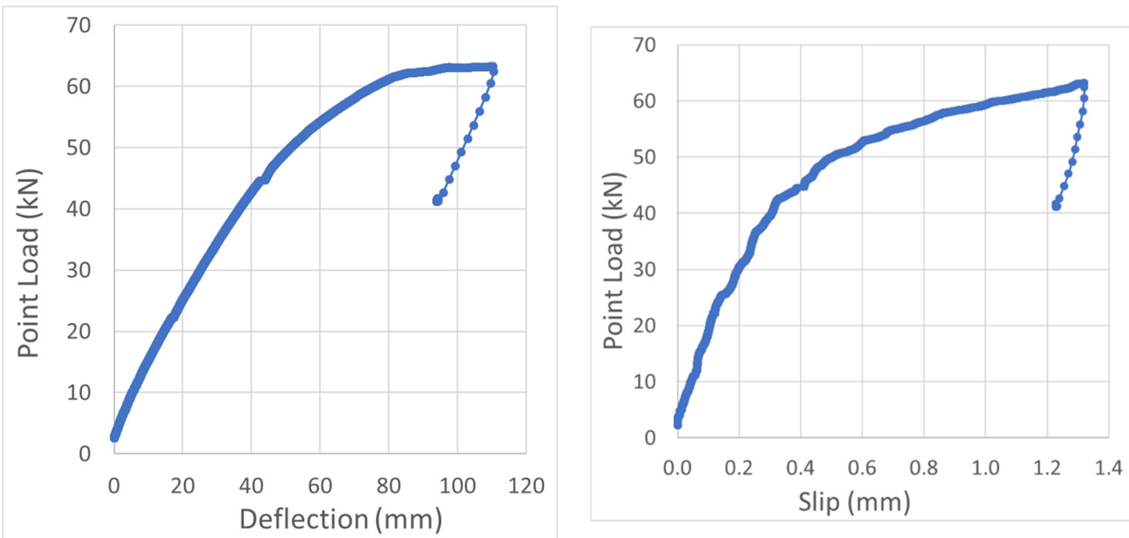


Figure 9 – NIST beam (Ramesh et al., 2018)

The beams were supported at their end by typical shear tab or double angle shear connections. The load was applied by six equally spaced point loads, which provided an extremely close match to the moment from a distributed load condition.

The first beam tested, which was the control beam for the fire project, failed unexpectedly at about 80% of its nominal plastic capacity of 83 kN (Figure 10(a)). The collapse was triggered by the failure of the last stud at one end of the beam at a relatively small slip (Figure 10(b)). Except for an explanation based on poor welding of the critical stud, which does not appear to be the case, there is no apparent reason for the failure to reach at least the nominal plastic capacity as this beam falls well within the limits of the Specification.



(a) Point load vs. vertical deflection

(b) Point load vs. end slip at East end of beam

Figure 9 – Behavior of NIST beam during initial load cycle (NIST, 2019).

Analyses by both the NIST and another team (Adhikari et al., 2022) using advanced FE models have reproduced the failures, but have not shed light on which design provision was violated. From the design standpoint, assuming materials and workmanship are not the reasons, one can hypothesize at least two explanations for the early failure:

- Lack of adequate ductility of the anchor: Conventional wisdom is that properly installed anchors should be able to deform around 5 mm before failing and that their behavior is roughly elasto-plastic. Typically, one would assume that yielding begins around 0.4mm and that the anchor reaches its maximum resistance at 1-2mm of slip. Assuming 19 mm anchors at 300 mm spacing and with a unit shear capacity of 93 kN for the anchor, a linear analysis would predict a slip of about 1.4mm at the load causing failure (a shear connection stiffness of about 0.57 kN/mm per mm). Figure 10 shows the distribution of the forces in the studs at initial yielding of the last anchor (Point load = 30 kN) and ultimate (P = 62 kN). At ultimate, the last six anchors were beyond their strength capacity with the last anchor at about 117 kN (25% above ultimate) and ΣQ_n = 1472 kN (or about 75% of the total shear connection capacity based on plastic analyses). These numbers indicate that as we go to longer beams (beyond about 10m), we need to rethink our ductility requirements. That rethinking has actually been required by the last two editions of the AISC Specification for certain situations, but not for this beam as its degree of interaction was above 50%. Issues related to anchor strength, stiffness and deformation capacity need clarification, but perhaps the best way to address these is from a performance based design approach as per AISI (AISC 923,2020)

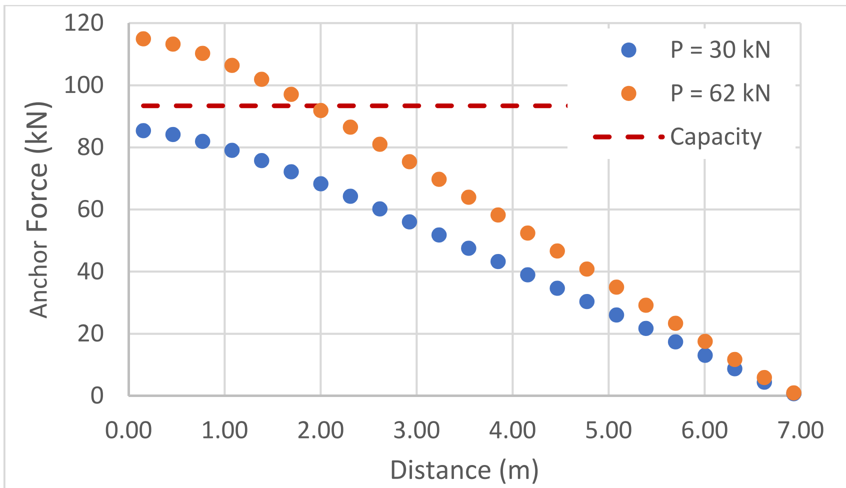
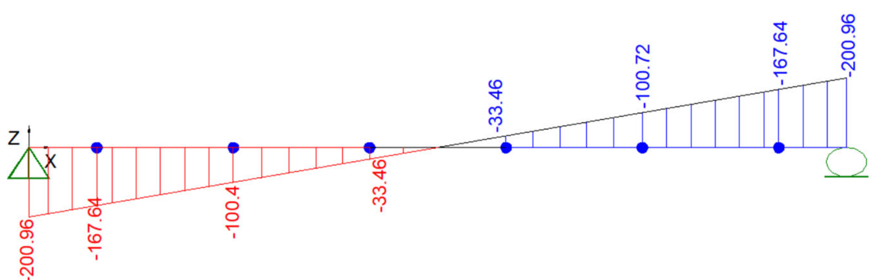
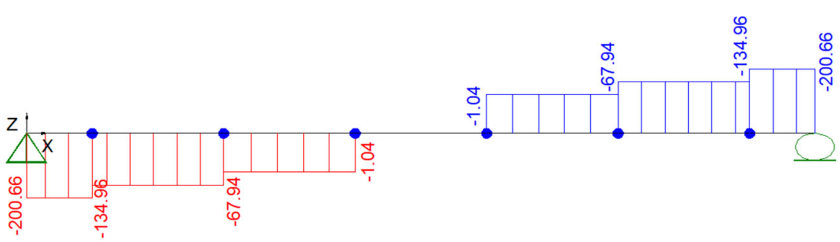


Figure 10 – Elastic distribution of anchor forces



(a) Shear diagram – distributed load



(b) Shear diagram – six point loads

Figure 11 – Comparison of shear diagrams

- Improper distribution of the shear studs: The shear studs in this beam were spaced uniformly based on the initial assumption of a distributed load. In fact, the load was applied by 6 actuators, resulting in a very different shear diagram (Figure 11). For the case of the distributed load, the horizontal shear between the last point load and the support varies from 383kN to 458kN for the case of a distributed load and has a constant value of 456kN for the case of the point loads. Given the position of the last point load at about 1m from the support (Figure 9), only three anchors could be placed within that interval. This would result in demands of 153kN per anchor, well above the nominal resistance of 93kN, for the point load case. The additional very local demands on the capacity of the shear stud due to the large vertical tensile force imposed by the concentrated force may also have played a role (Robinson and Naraine, 1988). The end result is that designers need to be very careful when concentrated loads are present near the support, even if the moment diagram does not raise any warning flags.

The previous paragraphs depict a rather disappointing progress in addressing issues related to composite beam design over the last 20 years. Fortunately, there have been significant advances with respect to composite columns and beam-columns. In addition, there has been the extensive development of composite walls, a type of member that was not addressed in the previous paper.

The main weaknesses for columns and beam-columns identified in 2011 included:

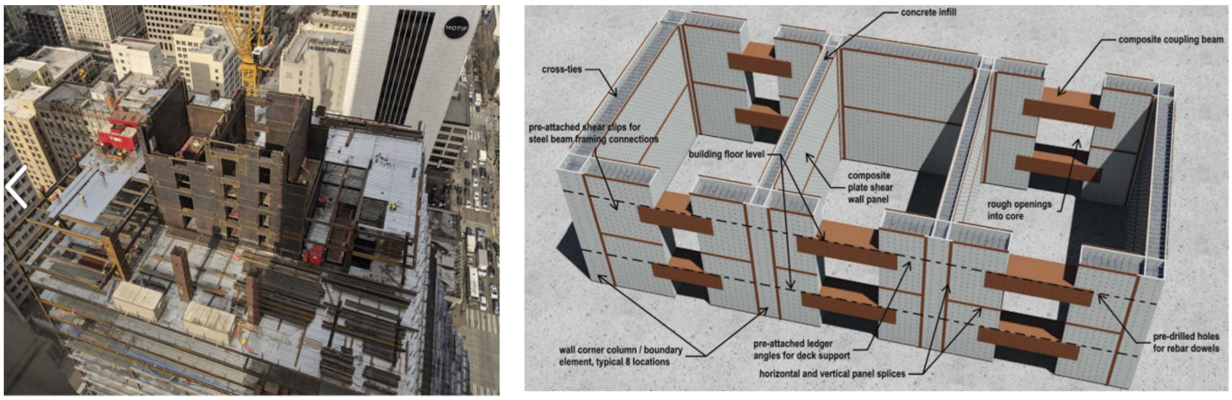
1. Treating composite columns as equivalent steel ones, without properly recognizing the significant differences in cross-sectional behavior of these members as beam-column that result from the addition of the concrete.
2. Failing to properly recognize the inherent additional stiffness that the concrete adds in addressing local and global stability.
3. Limits on material properties that are unrealistic in today's construction environment.
4. The effects of confinement are not adequately addressed, as the required amounts of transverse steel in SRC members are low and no credit is given for additional confinement to concrete-filled sections.
5. Long-term deformations, particularly creep and shrinkage deformations which can induce deformations greater than those due to elastic shortening, are not considered at all.

Thanks to a large number of analytical and experimental studies, issues (1), (2) and (4) have been well-addressed in a series of successive editions of the Specification (Leon and Hajjar, 2008). The AISC 2022 Specification now contains a set of robust and comprehensive provisions for design of composite columns and beam-columns. Relaxing the very conservative current shear design provisions for certain cross-sections remains perhaps the last major topic to be addressed. Clearly, incremental improvements, such a harmonization of the many equations available today into a single, consistent framework, must and will be made (Varma et al., 2014); however, the behavioral aspects are well understood and provisions for columns and beam-columns have reached maturity. Item (3) has been delayed until the next cycle of the Specification as the incorporation of high-strength materials impacts all parts of the Specification.

Composite walls, originally envisioned as part of protection systems for North Sea offshore platforms, have gained traction both in the nuclear and high-rise markets (Figure 11).

A final, but not less important improvement over the last 20 years, is the full incorporation of composite design into the AISC 341 – *Seismic Provisions for Structural Steel Buildings* (AISC 2022). Seismic design provisions in the USA originated with an effort by the Building Seismic

Safety Council (BSSC) in the early 1990's and culminated with the publication of the proposed provisions as Part II of the AISC Seismic Provisions (AISC, 1997). The work was heavily influenced by the interaction with Japanese researchers at the Conferences and its predecessor meeting (Roeder, 1985), as well as by the 1994 Northridge earthquake and its effects and interactions at a number of international conferences. In the most recent editions, Part II has been fully incorporated into the main provisions, highlighting the large ductility that properly detailed composite members are a superior choice in seismic design.



(a) Speedcore erection for Ranier Square (Seattle, MKA)

(b) Speedcore basic layout

Figure 12 – Composite wall system (Speedcore)

5. The Future of Composite Construction

In thinking about the future of composite construction, the author's views are that:

- Composite construction remains a common choice for tall buildings where stiffness governs the design, but its use would become dominant if its advantages with respect to fire design were fully exploited. A number of recent performance-based fire design applications clearly point to the large economies available when prescriptive methods are not used.
- Composite construction could be more economical in moderate height buildings (10 to 30 stories) if efficient connection designs could be implemented. Numerous ideas exist as to how this can be done, but the funding for the large-scale studies needed to provide AISC 358-like design provisions is lacking.
- Composite construction in low-rise structures could be economically implemented using modular construction techniques, with infills made of foam-like materials rather than concrete.
- Much work, and possibly development of new types of infill materails, is needed to implement use of high-strength materials in composite construction, Currently, high-strength steel and concretes are developed separately and not optimized for use in composite construction. That paradigm needs to change, particularly with respect to improving behavior under large inelastic deformations.

- As for the case of composite walls, the industry needs to quickly implement innovative solutions. This requires an extensive outreach educational effort to both designers and building officials so that composite construction alternatives are properly evaluated.
- Better testing and qualification techniques need to be employed by the industry to characterize anchor performance, in order to permit the use of larger and more economical varieties of anchors.
- Long-term, large-scale studies need to be undertaken to determine if creep, shrinkage, temperature and other types of volumetric changes need to be considered in design. Current methodologies are anchored in studies conducted with materials very different from the ones used today. For example, we do not have long-term studies on composite beam deflections with concretes utilizing the typical admixtures and supplementary cementitious materials that the industry uses today.
- Combinations of steel with other materials, such as cross-laminated timber and FRP, need shear connections based on friction and adherence. Provisions need to be developed to enable different shear transfer mechanisms to be used in design.
- Better techniques to define composite member stiffness for the service, ultimate strength and stability limit states need to be developed.

Many of these issues need to be addressed from a global point of view. The American specifications benefited greatly from early interactions with the writers of the Eurocodes (ASCE, 1987; ASCE 1993). With the new Eurocodes (EN 1994-1-1 (2009) coming out in the next few years and the explosive growth and research on composite construction in China, there should be new efforts at harmonizing the basis for design across the globe. There are significant differences in construction practices and economic constraints that will inevitably color local implementation of design rules. However, in the author's opinion, there should and can be agreement about the basic mechanics and limit state that must be applied.

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