REVIEW OF FATIGUE FAILURE STUDIES FOR WELDED PRECAST DOUBLE-TEE CONNECTIONS: NEW DATA VALIDATES SAFETY CONCERNS

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ABSTRACT

Flange-to-flange connections of precast concrete double-tee parking structures are poorly configured to withstand high-cycle vehicular loading and fail to meet code requirements for fatigue-resistant design. Connection failures typically spread along the joint until the joint is completely severed. While broken connections are commonly considered a superficial maintenance item, these connections are an essential part of the lateral load resisting system; loss of connections along a joint severs the deck diaphragm and threatens the general stability of the structure. A series of studies recently performed by the precast/prestressed concrete industry draw conclusions that purport to allay concerns regarding these connections; however, review of the data and methodologies in these studies reveals that the modern, flexible flange connection exacerbates the fatigue failure mechanism by concentrating stress at the ends of the welds. This paper analyzes the findings of these recent studies and their implications for the durability and safety of tee-to-tee connections within a precast concrete parking garage assembly.

Keywords: Double-tee, Connections, Fatigue, Construction, Repair and Rehabilitation, Research

"This paper summarizes the fatigue test program of a PCI-funded research effort to assess the fatigue resistance of welded flange-to-flange connections used in double-tee precast concrete construction. This paper was reviewed by both PCI's Technical Activities and Research and Development Councils in accordance with their review processes.[1]"

"The results show that a heavily used parking structure, supporting 500 vehicles entering and leaving per day for 365 days a year, would theoretically reach 52 to 62 years before a fatigue-induced fracture would be expected to occur.[2]"

The statements quoted in the text box are from a paper published by the Precast/Prestressed Concrete Institute (PCI) in a 2019[3] PCI Journal. This paper was the culmination of a series of four studies by a "Task Group" commissioned by PCI[4,5,6,7] (herein referenced as the "PCI Task Group Studies"). As the industry claims that common precast, prestressed parking structures can be expected to remain in service for half a century or more[8], 50 years is an important benchmark. More importantly, these connections are an integral part of the lateral force resisting system; premature failure can lead to loss of general stability of the structure. As precast double-tee garages are commonplace throughout the country, an inherent defect that routinely causes loss of these diaphragm connections would create unsafe conditions on an unprecedented scale. In light of the extent and gravity of potential widespread failures that could arise from deficient connections, third-party review of the recent investigative work performed by the PCI Task Group is crucial to determine if the theoretical life expectancy of double-tee garages with respect to fatigue induced failure has been properly considered.

Prologue

PCI commissioned a series of studies in response to claims presented in a paper written by this author, herein referenced as the "Keenan Paper" (so named by the PCI Task Group)[9]. This original paper sought to identify the primary cause of failures commonly observed at double-tee connections and uncovered technical deficiencies in the standard design of welded flange connection details. The major findings of the Keenan Paper were that welded tee-to-tee flange connections of precast concrete parking garages: (1) are poorly configured to withstand high-cycle fatigue loading from vehicular traffic, (2) violate established industry standards and are prohibited by American Institute of Steel Construction (AISC) and American Welding Society (AWS) requirements, and (3) are prone to fatigue failure. It was an observation of this author that failure of one connection causes load to transfer to adjacent connections, subsequently causing other failures along the joint to occur. In effect, the joint would "unzip", severing the diaphragm and causing unsafe conditions.

It has been a major tenet of the PCI Task Group Studies that the intentional flexibility of these connections inherently aided in the fatigue resistance of the connection and that this benefit was not adequately acknowledged in the Keenan Paper. Specifically, a 2018 PCI paper asserted that the stress in the welds could not be determined by simplified engineering assumptions and required finite element analysis methods to accurately determine magnitude and distribution[10]. It was with the intention of investigating this claim that PCI commissioned the four Task Group Studies.

It is true that the flexibility of the now-common flexible precast tee connection was not explored in the Keenan Paper. It has been the position of this author that such flexibility only serves to concentrate and increase stress within the ends of the welds near the embedment anchorage; thus, it need only be proven that the connections do not meet fatigue design requirements to demonstrate their inherent fallibility. Compounding the analysis with further stress risers would only underscore this conclusion. Further, it is beyond the resources of any one individual to perform the modeling and mockup testing necessary to draw substantive conclusions about the impact of connection flexibility on resistance to fatigue failure. With the more recent, extensive data provided by the PCI Task Group studies, this paper revisits the original flange connection analysis to respond to the findings of the PCI studies.

The Cause of Fatigue Failures

The following is a summary of research and findings outlined in more detail within the "Keenan Paper"[11]. Connections between double-tee beams are subject to dynamic cyclic loading from vehicle movement. As the wheel traverses each double-tee beam, gravity loading causes the beam to deflect downwards. The adjacent beams resist this deflection through individual connections located along the beam flanges. The magnitude of force transmitted through each connection is proportional to the number and spacing of the connections in relation to the location of the load and this deflected shape, with connections close to the center of the drive isle resisting the greatest deflection and acquiring the greatest load. As the load traverses the joint, the loading of the connections is abruptly reversed.

The force transmitted across each connection is resisted by the field-installed erection bar which is welded to connection anchors embedded within each flange. As these anchors are separated by the width of the erection bar, moment is created. This moment is applied to each weld about an axis parallel to the joint, creating a concentration of tensile stress at the root of the weld, followed immediately by compression stress as the wheel traverses the joint (Figure1). With enough cycles, this stress reversal can lead to *high-cycle fatigue failure* of the weld.

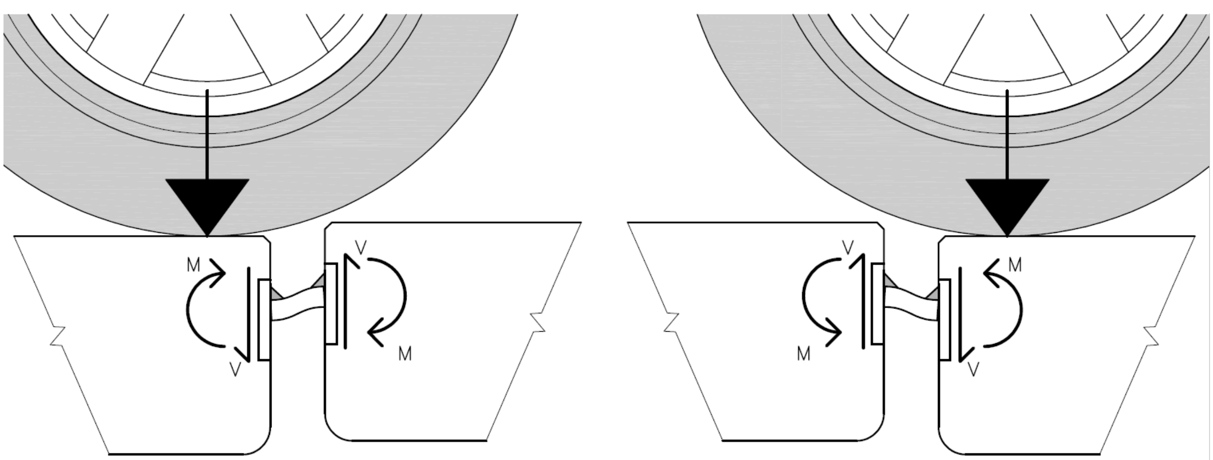

Figure 1 Exaggerated depiction of beam deflection at flange joint connection.

Fatigue fracture occurs in three steps: crack initiation at the root of the weld, crack propagation through the throat, and, finally, abrupt failure at the face as stress in the remaining weld metal increases proportionally to the section lost until the remaining strength is exceeded (Figure 2). This final failure can be delayed as stress is redistributed to adjacent connections, causing the abrupt failure zone to be very thin.

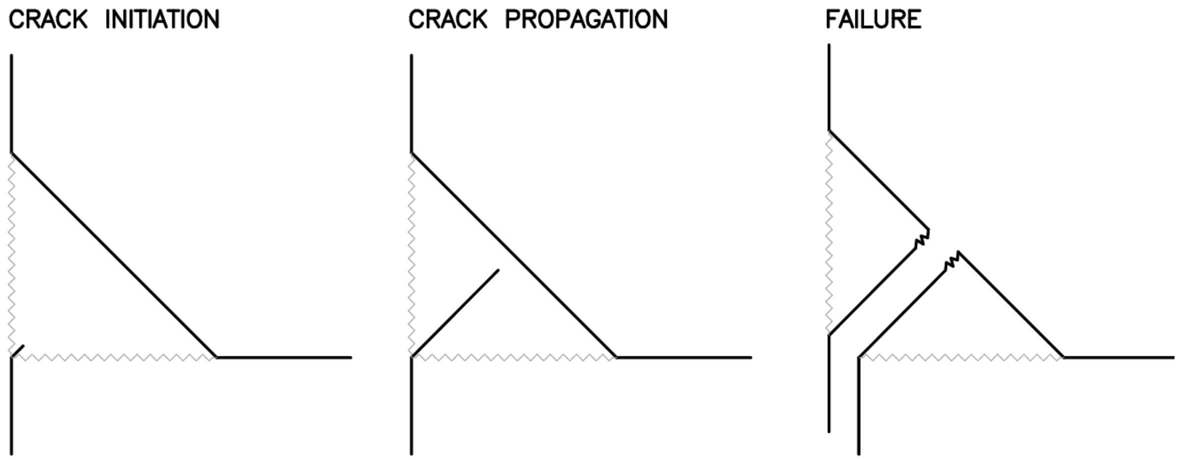

Figure 2 Three stages of fatigue failure of a fillet weld in a double-tee connection.

Fatigue fractures are easily distinguished from other types of connections failures by appearance. The fatigue fracture, occurring due to moment about and axis parallel to the weld, is necessarily straight (Photo 1). Further, the failure surface typically has two visibly distinct regions created during the crack propagation and subsequent abrupt failure stages. The crack propagation stage is distinguished by a smooth, flat, burnished appearance, a result of microscopic striations arranged in parallel (Photo 2). These *beach marks* are a visual record of the numerous steps in crack formation caused by individual cycles of loading and are easily discernable on scanning electron micrographs (Photo 3).


Photo 1 Fatigue failure of weld. Straight, flat weld fracture through throat of weld.

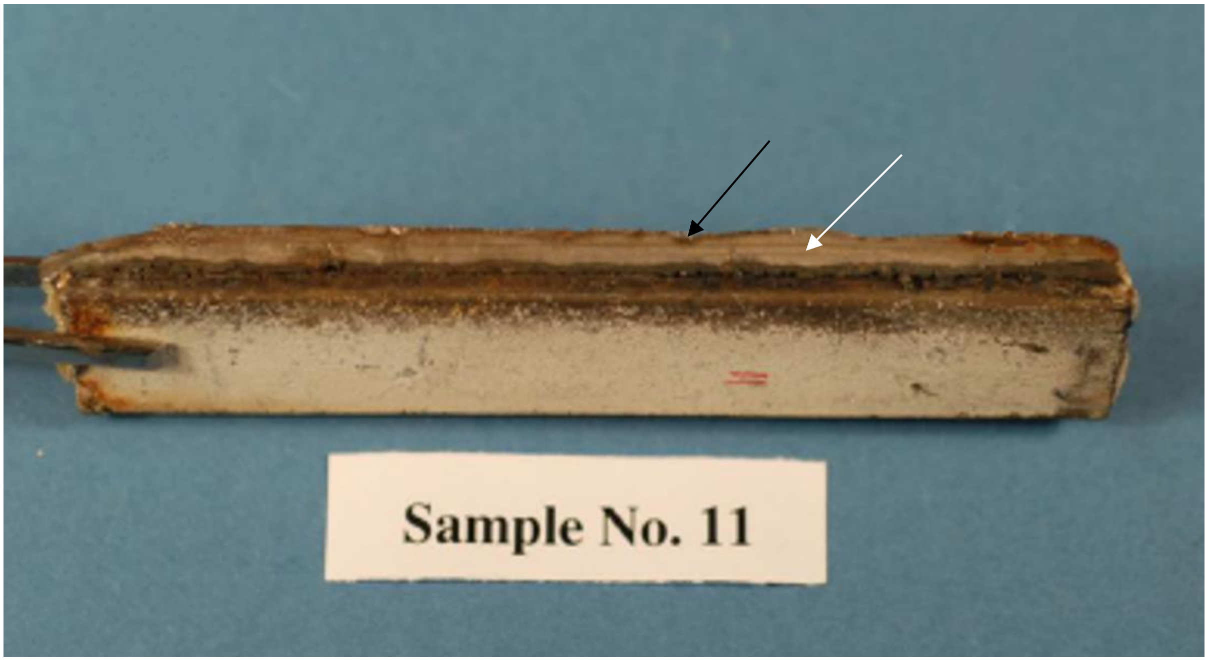

Photo 2 Fatigue fracture through throat of weld. Note straight, flat burnished appearance at surface of fatigue fracture (white arrow) and jagged fracture at abrupt failure (black arrow).

Photo 3 Scanning electron micrograph showing "beach marks" indicative of fatigue cycling.

Design Requirements

The following is a brief overview of code design requirements, which are outlined in more detail within the "Keenan Paper"[12]. For steel construction, the International Building Code (IBC)[13] largely defers to the American Institute of Steel Construction's AISC 360, Specifications *for Structural Steel Buildings*[14]. However, AISC 360 applies to a very limited and specific range of structural steels, for which all stainless-steel varieties are excluded. As stainless steel is currently the dominant material utilized for such connections, this reference does not directly apply.

For instances where the IBC does not offer specific direction, Section 1706, "Design Strength of Materials," applies. Per this section, in the absence of applicable standards, the design strength and permissible stress of a material must conform to the specifications and methods of accepted engineering practice or approved rules.[15]

Manufacture of stainless-steel is a well-developed industry and model codes exist to provide accepted engineering practice and approved rules[16]. Such codes and standards are provided by both the American Institute of Steel Construction (AISC) and the American Welding Society (AWS). It is notable that neither institution recommends that the welds be loaded in a manner that stresses the root of the weld in tension, and that AWS expressly prohibits such a configuration.[17,18]

Per the AISC "Steel Design Guide 27: Structural Stainless Steel," strength may be determined in accordance with AISC specifications; however, a reduced resistance factor (Φ=.55, LRFD) or an increased factor of safety (Ω=2.7, ASD) must be applied[19]. *AISC Steel Construction Manual* Part 8, "Design Consideration for Welds," recommends that one-side fillet welds not be loaded, even statically, in a manner that stresses the root of the weld in tension, warning that a notch may form from the unwelded side, causing a crack to occur[20] (see Figure 3). This statement of caution is echoed by AWS D1.6, *Structural Welding Code – Stainless Steel*, which explicitly prohibits welds to be arranged in a manner that allows bending of the weld about an axis parallel to the joint that causes tensile stress at the root of any weld.[21]

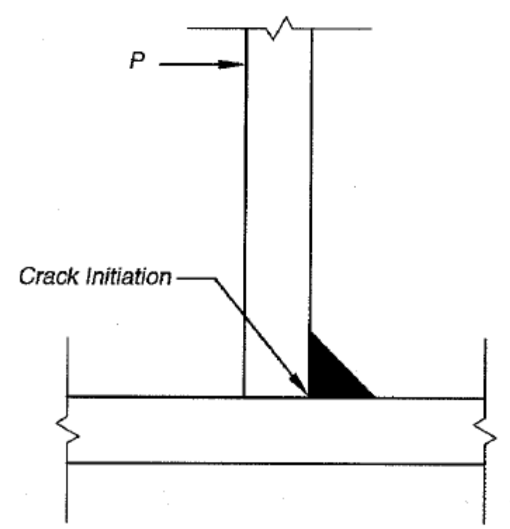

Figure 8–8. Notch effect at one-sided weld.

Figure 3 *AISC Steel Construction Manual,* Fourteenth Edition, Figure 8-8.

Per AISC, fatigue shall be considered in accordance with Appendix 3, "Design for Fatigue,"[22] which stipulates that evaluation of fatigue resistance is required if the number of cycles exceeds 20,000[23]. A double-tee connection is typically subject to 20,000 cycles within a matter of weeks, and total cycles over the life of a garage are typically measured in the tens of millions.

In addition to the number of cycles (N), design for fatigue resistance requires consideration of the Stress Range (SR) and the constant (Cf) relating to the Stress Category. For reverse loading, the Stress Range is defined as the numerical sum of the maximum repeated tensile and compressive stress. The Stress Category is obtained by application of pictorial representations and descriptive narrative of connection configurations and geometries shown in AISC Table A-3.1, "Fatigue Design Parameters."[24] AWS is in agreement with these provisions, providing an identical table of design parameters and pictorials.[25] The AWS code also provides a figure that graphically shows the relation of the allowable stress range F_{SR} to both the number of cycles (N), and the Stress Category.[26]

Notably, neither AISC Table A-3.1 nor AWS D1.1 provide a pictorial representation suitably analogous to the welds used at typical double-tee connections. These model codes and standards do not anticipate a connection of this geometry and so cannot be applied.

To demonstrate the fatigue design process, the Keenan Paper analyzed a typical garage connection under anticipated loading. The connection model assumed a 1" wide field erection bar welded to embedments with a ¼" fillet weld, 3" long. The embedments were assumed fixed such that the shear and moment remained constant along the length of the weld (Figure 4).

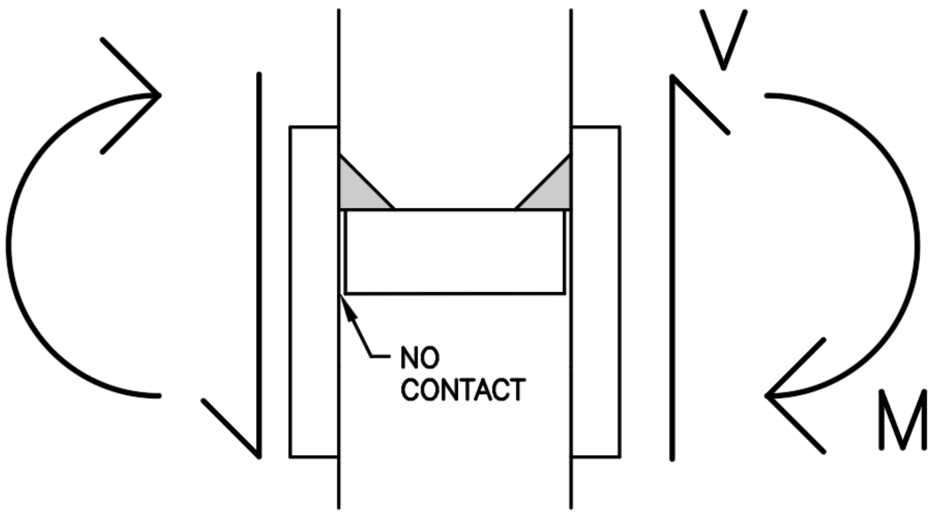

Figure 4 Connection model assumes fixed embedments within the concrete flanges such that stress along the length of the welds is constant.

This design exercise revealed the stress range in the weld due to bending to be approximately 11 ksi. For comparison purposes, this is far greater than the *Fatigue Threshold* (F_{TH}) of 8 ksi that would apply to a Stress Category F fillet weld loaded in an acceptable manner.[27] As the fatigue threshold for the connection model would be less than that of a Category F detail, the connection was deemed unsuitable.

PCI Task Group Studies

The above findings were presented by Keenan at the 2017 PCI Convention and National Bridge Conference, in Cleveland, OH. In response, PCI commissioned the four PCI Task Group Studies[28,29,30,31]. The stated purpose of these studies was to determine if the concerns raised in the "Keenan Paper" were valid. Ultimately, the PCI Task Groups concluded that a garage utilizing the common flexible double-tee connection would have a design life in excess of 50 years. The following analysis is respectfully offered to evaluate the design assumptions and criteria utilized by the Task Group in support of this conclusion.

General

Much of the information provided in the Keenan Paper remains unrefuted or was, in fact, verified by the PCI Task Group Studies. A 2017 PCI Conference paper on experimental analysis of double-tee flange connections acknowledged that the loading applied to individual connections had not been adequately considered within the industry[32], nor had fatigue resistance in particular; rather, these conditions were being investigated in response to concerns raised by the Keenan Paper[33]. Through experimental evaluation, PCI researchers have since conceded that wheel loading causes bending stresses in the welds[34], not just simple shear as previously postulated by PCI in the connection design manual[35]. In a 2019 study, the PCI Task Group further concluded that the "strap steel plate," from which the flexible connections are typically fabricated, allows deformations that induce high stress on the root of the fillet weld,[36] and that the flexibility of the faceplate creates elevated stress at the ends of the connection[37].

While the PCI Task Group Studies do not offer specific responses to concerns raised about AISC and AWS requirements, they do concede that the fillet weld detail used in flange-to-flange connections is not classified by AISC. Rather, the Task Group Studies propose that the closest analogue is the AISC Fatigue Category F connection, from which they conclude: "*As a category F detail, the fatigue life expected by AISC would be negligeable"*[38]. Notably, the AISC Fatigue Category F connection is a fillet weld loaded in longitudinal and transverse shear (Figure 5), which are configurations allowed by code and the manner in which these welds are intended to be used. Further, the finite element analysis (FEA) conducted by the PCI Task Group revealed that the average stress range experienced by these welds was higher than both that allowed by the AISC Category F detail and that comparatively calculated by Keenan.[39]

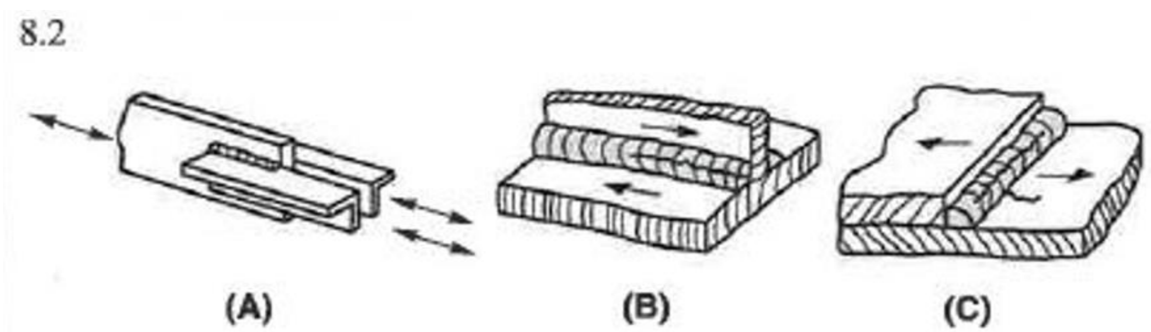

Figure 5 AISC 360*, Specifications for Structural Steel Buildings,* Table A3.1[40]

While these findings would appear to validate all major assertions and concerns presented in the Keenan Paper, the PCI Task Group Study contends that the inherent flexibility of the standard tee-to-tee connection resulted in a more "compliant" detail, a claim that eschews both the findings of the Keenan Paper and the S-N approach required by AISC and AWS.[41] For clarity, the following is stated in context:

"The fillet-weld detail used in flange-to-flange connections is not classified by the American Institute of Steel Construction (AISC). The closest comparison is the AISC fatigue category F connection (AISC detail 8.2), because a category F weld is subject to shear, tension [sic], and

bending [sic]. As a category F detail, the fatigue life expected by AISC would be negligible. However, the observed longevity of this connection detail in many parking structures indicates that the AISC category S-N curve is not applicable for this connection detail. The flexibility of the faceplate in the flange-to-flange connection results in a more compliant detail and a complex stress field that cannot be analyzed using nominal stress S-N approach."[42]

In summary, the Task Group observed this connection to perform well in "many" garages and resolved that AISC design criteria therefore must not apply. Not only is this conclusion based solely on anecdotal evidence, but it also relies on inconsistent logic. If it were true that the connection design defies AISC standards, then not "many," but *all* garages would benefit from the same longevity. That failures have occurred in *any* garages points to inherent deficiencies and raises grave safety concerns, as these are seismic connections integral to the stability of the structure. Further, in the context of a 50-year design threshold, it must be noted that this flexible connection has only been in common use for about 20 years. Thus, there is no basis to assume that the "*AISC category S-N curve is not applicable for this connection detail"* through empirical evidence alone.

To support the stated conclusion, the PCI Task Group performed a series of four studies, including mockup testing, finite element analysis, numerical analysis, and fatigue life testing and assessment. This has since been accepted within the industry as the definitive work on this subject. However, a close review of the studies reveals methodologies and assumptions that are often at odds with, or divergent from, good and common engineering practice. Rather than verify the safety and longevity of these connections, independent review of the Task Force investigations instead confirms the common industry connection to be fatally flawed.

In support of this analysis, the following are five critical design decisions made by the PCI Task Group to support the assertion that the flexible connection results in a more "compliant detail" that would theoretically reach 50+ years before fatigue-induced fracture would be expected to occur. The reader is asked to consider these decisions and to determine if they are mathematically sound, suitably conservative, and in keeping with good engineering practice.

Vehicle Weight

The vehicle weight distributions utilized in the fatigue life assessment were based upon a 2001 study of nine parking garages located in Massachusetts and Illinois. The average vehicle weight in this study was 3,411 lbs. For the purpose of the PCI Task Group Study, this average weight was increased by four percent to 3,617 lbs. in proportion with an EPA study[43] that indicated the vehicle weight increase from 2001 to 2016.[44]

The 2001 weight used as the basis for these calculations was based upon cars in existence and common use at that time, most of which had been manufactured in preceding years. It is noteworthy then that the weight of vehicles had steadily increased from a low of 3,228 lbs. in 1980[45]. Logically, the weight of vehicles in production would have been a better metric as to the weight of cars that would be on the road in the future and more appropriate for use in predicting fatigue life. In 2015, the average production vehicle (trucks and cars) weighed 3,735 lbs.[46] While this three percent increase over that used by the Task Group is not insignificant, it is indicative of a trend and the weight increase since this time has been dramatic. Per a 2020 EPA study, average vehicle weight was highest on record at 4,156 lbs[47], which is 539 lbs, or 15 percent, heavier than that used by the Task Group; a more recent 2022 EPA study shows vehicle weight continuing to increase at an accelerated rate[48]. The vehicle weights used by the Task Group, which included vehicles manufactured in prior years, were not suitably conservative for design purposes. Rather, the weights should be based upon current production information and trends with a suitable allowance for future variation.

Vehicle Placement

More important than vehicle weight is its placement. Load testing performed by the Task Group revealed connections located near midspan are subjected to the highest strains from, and thus are most sensitive to, applied load. [49] Further, influence lines calculated and graphed by the Task Group (Figure 6) reveal the magnitude of shear load transferred across a single connection is highly sensitive to placement of the load, with applied shear across a connection diminishing greatly within relatively little distance from the applied load.

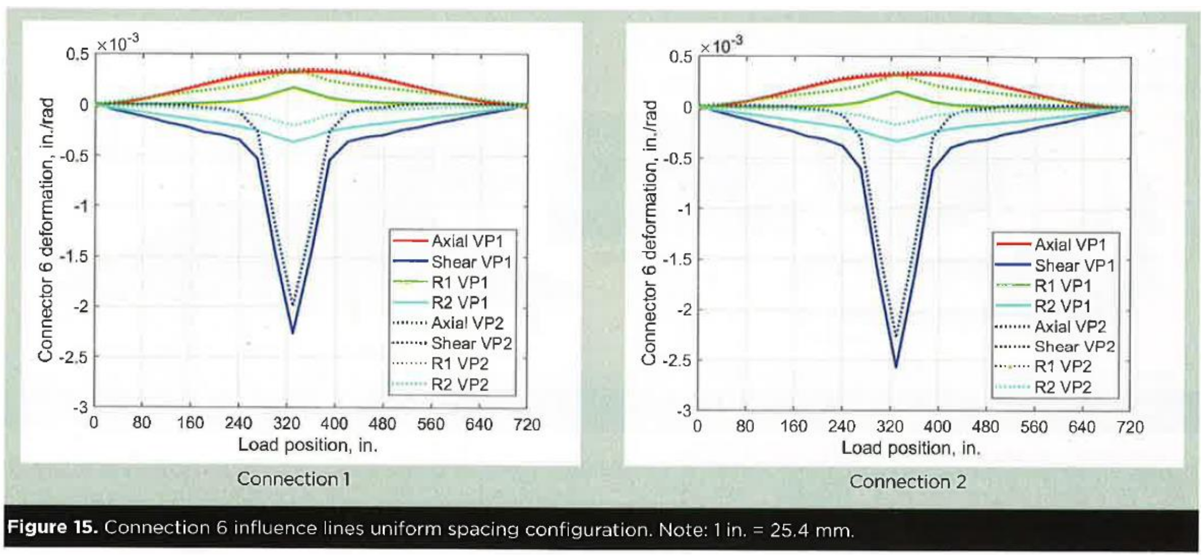

Figure 6 "Flange-to-flange double-tee connections subjected to vehicular loading part 2: Fatigue life assessment" Figure 15[50]; influence lines for connection near mid span on double-tee beam.

To calculate the fatigue loading, the Task Group assumed vehicles would traverse the deck in accordance with "two common scenarios": a normal distribution or a uniform distribution (Figure 7). For the normal distribution, a standard deviation of 86 in. was used, encompassing 90 percent of the vehicles within the 24 ft. drive aisle. For the uniform distribution, the center of vehicles was assumed to occur evenly at all locations within the 24 ft. drive aisle.[51]

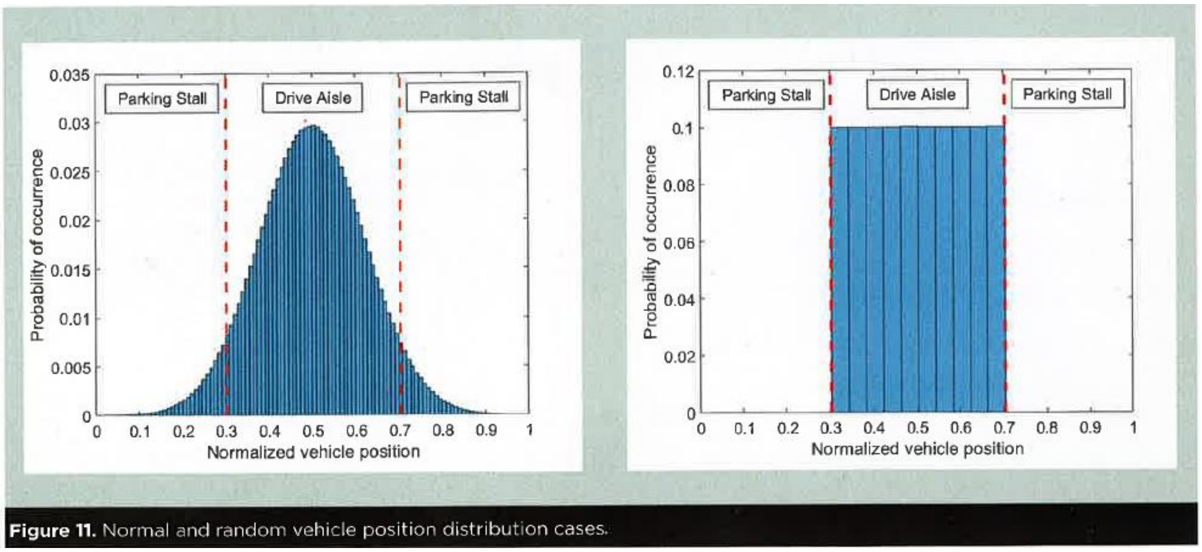

Figure 7 "Flange-to-flange double-tee connections subjected to vehicular loading part 2: Fatigue life assessment" Figure 11[52].

Close review of these loading conditions reveals peculiarities. In the normal distribution, 10 percent of the vehicles are assumed to drive through the parking stalls. While it is debatable that these areas would be traversed routinely if empty, the fact that a single parked car would render such an occurrence impossible is incontrovertible. Further, in most garages, the parking stalls are also the location of immovable K-frames and shear walls. Therefore, assuming one out of 10 cars traverses these areas is not a conservative or realistic assumption; the true number would be negligible. Should a normal distribution be assumed, the outer limits should be defined by the edges of the drive lane, such that 100 percent of vehicles are within the drive aisle. This would necessarily indicate a concentration of traffic at the center of the aisle.

Review of the uniform distribution reveals further erroneous assumptions. Based on this model, the centerline of the vehicles can be placed anywhere within this distribution.[53] This would place the outside wheel of the vehicle at the edge of this drive lane within the parking stall, which is also occupied by cars and shear walls. Further, it is both poor practice and uncommon to drive immediately adjacent to a parked vehicle, as a vehicle may pull out or a pedestrian or child may exit from between the vehicles. It is unlikely that cars would drive with the outside tire closer than four to five feet of the parked cars. Therefore, the uniform distribution model used in the Task Force Study assumes a greater proportion of vehicles travel at the periphery of the drive lane than should be expected.

It is the observation of this author that vehicles typically drive at or near the center of the double-tee span. In the event of oncoming traffic, the vehicle moves over and then quickly returns to near center. This was previously also the opinion of the PCI Task Group, who stated that fatigue-induced cracking, *"…were it to occur, would manifest at the middle of the double-tee span where vehicles are most likely to pass."*[54] The real-world behavior of drivers renders the distributions assumed by the Task Group either impossible due to conflict with parked cars or improbable due to good driving practice. This is significant, as these assumptions have removed as much as 50 percent of the vehicles from near center of the double-tee where loading is both most critical and most likely. Such assumptions are not conservative for design purposes.

The Connection Model

The connection model used by the Task Group is shown in Figure 8. This model assumes common manufactured connection embedments joined with a 3/8" erection bar and ¼" fillet welds, 2½" long. Importantly, the model assumes "'hard' frictionless contact" between the bottom of the erection bar and the face of the embedded connector. The Task Group Study neglects to explain how this hard contact is to be consistently achieved or verified in the field. Further, the study does not discuss whether this condition has been witnessed or is achievable in practice. As the Keenan Paper explores the possibility of such hard contact extensively and concludes that it is not attainable, overlooking these concerns is a grave omission by the Task Group.

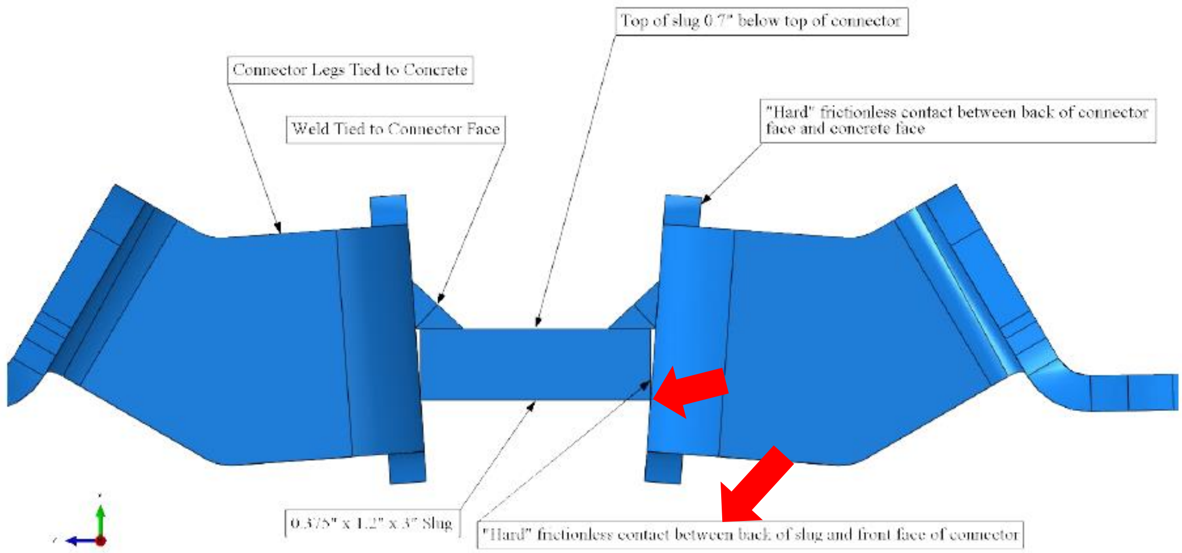

Figure 39: Details of Combined Assembly for Manufacturer 1

Figure 8 "Double Tee Flange Connections – Analytical Evaluation" Figure 39[55]; Note "'hard' frictionless contact" where bottom of erection bar meets connector face (arrows).

The existence or an absence of a root opening at this location dramatically affects the model behavior. If there is a root opening at both sides of the erection bar (no hard contact at either connection face), shear across the connection will cause equal and opposite moments in the welds about an axis parallel to the welds. At normal joint widths, the moments would be the primary forces with shear being secondary.

If hard contact is assumed, as shown in Figure 8, the stress in the weld is greatly reduced. At the side where the hard contact is in compression, a couple is created between the weld and the point of contact, greatly reducing stress in the weld and transforming it primarily from moment about the weld axis to tension between the erection bar and faceplate. The reduction in stress this creates is estimated by the Task Group to be as much as 75 percent[56]. Further, the nature of the stress is changed, as much of the moment about the weld axis is transformed into a tension force parallel to the compression strut created by the hard contact; the fillet weld transfers this force from the embedment to the erection bar through shear stress, a condition for which fillet welds are well suited. At the far side, the stress in the weld is also both reduced and transformed due to the increased fixity of the opposing side; moment at this weld is reduced and shear is increased, which, as noted, is a condition for which the fillet weld is better suited.

This condition is graphically represented in a figure provided by the task group (Figure 9). The figure and associated graphs illustrate the stresses in the welds under reverse loading conditions. The stress is provided both as actual calculated values along the weld, with stress elevated at the ends of the weld, and as an average value. If there were no hard contact of the erections bar, such that the moment was entirely resisted through bending stress in the welds, the graphs would indicate average stress levels to be roughly equal but opposite for the reverse loading conditions. However, the graph indicates a dramatic shift in stress levels during reverse loading, with an average of -7.6 ksi and 0.085 ksi for these opposing conditions.

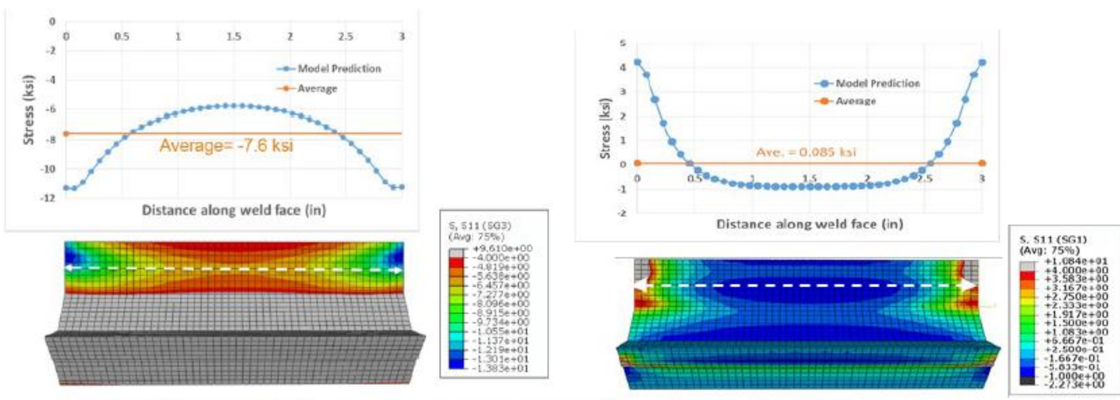

Figure 30: Variation in strains along mid-longitudinal axis of welds on both sides of the joint

Figure 9 “Double Tee Flange Connections – Analytical Evaluation” Figure 30.[57]

It is this assumed behavior by which the Task Group claims the welds are subject to a combination of bending, shear, and membrane action more akin to connections utilized in the “offshore structure industry.” It is also the nexus by which the Task Group’s analysis was performed in accordance with research performed by Sorensen, et al.[58], as opposed to requirements of the AISC or AWS. Specifically, the PCI study asserts that the connection behavior is comparable to a pictorial provided in the Sorensen et al. research (Figure 10).[59]

Where:

< 5,000,000

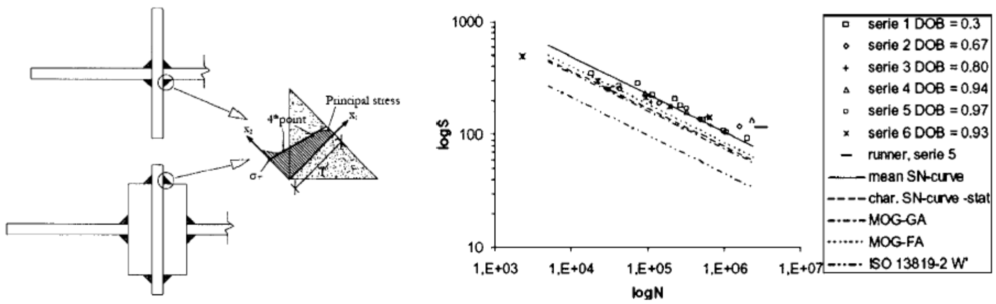

Figure 46: Weld detail [*Figure 12 - from Sorensen et al.*] and SCF4 Stress (in MPa) Parameter SN Curve [*Figure 14 - from Sorensen et al.*]

Figure 10 “Double Tee Flange Connections – Analytical Evaluation” Figure 46[60].

The PCI Task Group model relies entirely on this hard contact occurring within the connection, as stress levels would greatly increase due to the small elastic section modulus provided by the ¼” tall fillet welds alone. It is therefore necessary to examine the probability that such hard contact may be reliably assumed for design purposes.

No recommendations are offered by the Task Group that Special Inspections for hard contact be made or that such requirements be Qualified or included within Weld Procedure Specifications (WPS) to ensure such contact. There is also no indication that the Task Group has investigated this condition in the field to determine if it is feasible or commonly achieved. As such, it remains possible, if not probable, that the erection bar may not be placed in full contact with the embedments during construction. Further, contraction of the deck in cool weather would cause a root opening to form and contact to be lost for at least a portion of the year.

More importantly, the act of welding itself necessarily causes such a gap to occur. As the weld metal cools, transverse shrinkage of the weld pulls the members to the weld. This well-documented phenomenon, termed *angular distortion* (Figure 11),[61] creates a root opening on the side of the erection bar first welded and lifts the other side of the erection bar off the opposing embedment. The distortion effect is greatly exaggerated by the relatively thin ribbon of steel from which the embedments are fabricated. Such deformation is further exacerbated by the 50 percent higher coefficient of thermal expansion of stainless steel over mild steel, making it highly susceptible to warping when welded. This angular distortion commonly creates a visible separation between the erection bar and the embedment into which at least a business card can be inserted (Photo 4).

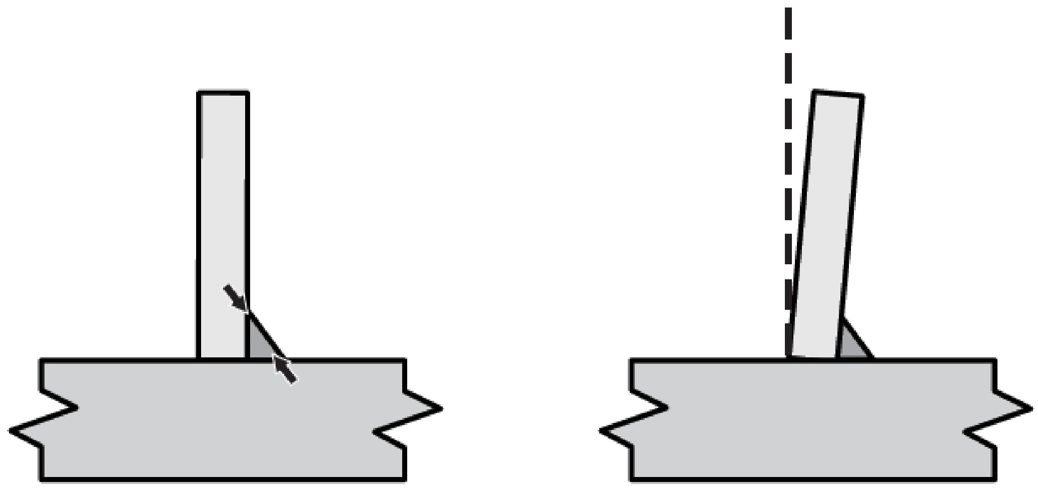

Figure 11 Angular Distortion (Figure 6-3, AISC Design Guide 21)[62]

Photo 4 Typical gap between bottom of erection bar and embedment.

One may argue that the typical root opening may be small enough that it closes under load, allowing the connection to behave much like a connection with no root opening. It is useful, then, to consider the root opening width required to remain open during fatigue loading and then to consider whether it is likely for such an opening to exist. For comparative purposes, the maximum fatigue threshold for the superior AISC Category F weld detail in mild steel is 8 ksi. Thus, prior to stress reversal, the average design stress in a symmetrical connection would be 4 ksi or less. This very low stress causes correspondingly low strain and ultimately very small deflections. With consideration of the geometry of the connection, calculations demonstrated that the open dimension between the erection bar and the embedment need only be approximately 0.0003 inches for it not to close under service load.[63]

As this is arguably the most important aspect of the design model, it is notable that the PCI Task Group Study does not consider the probability that hard contact may not be a given in the connection assembly. It is notable, then, that included within the Task Group's studies is a photograph of a cross-section cut through a connection that visibly shows a large root opening (Figure 11)[64], proving that such a gap does, and is likely to, occur.

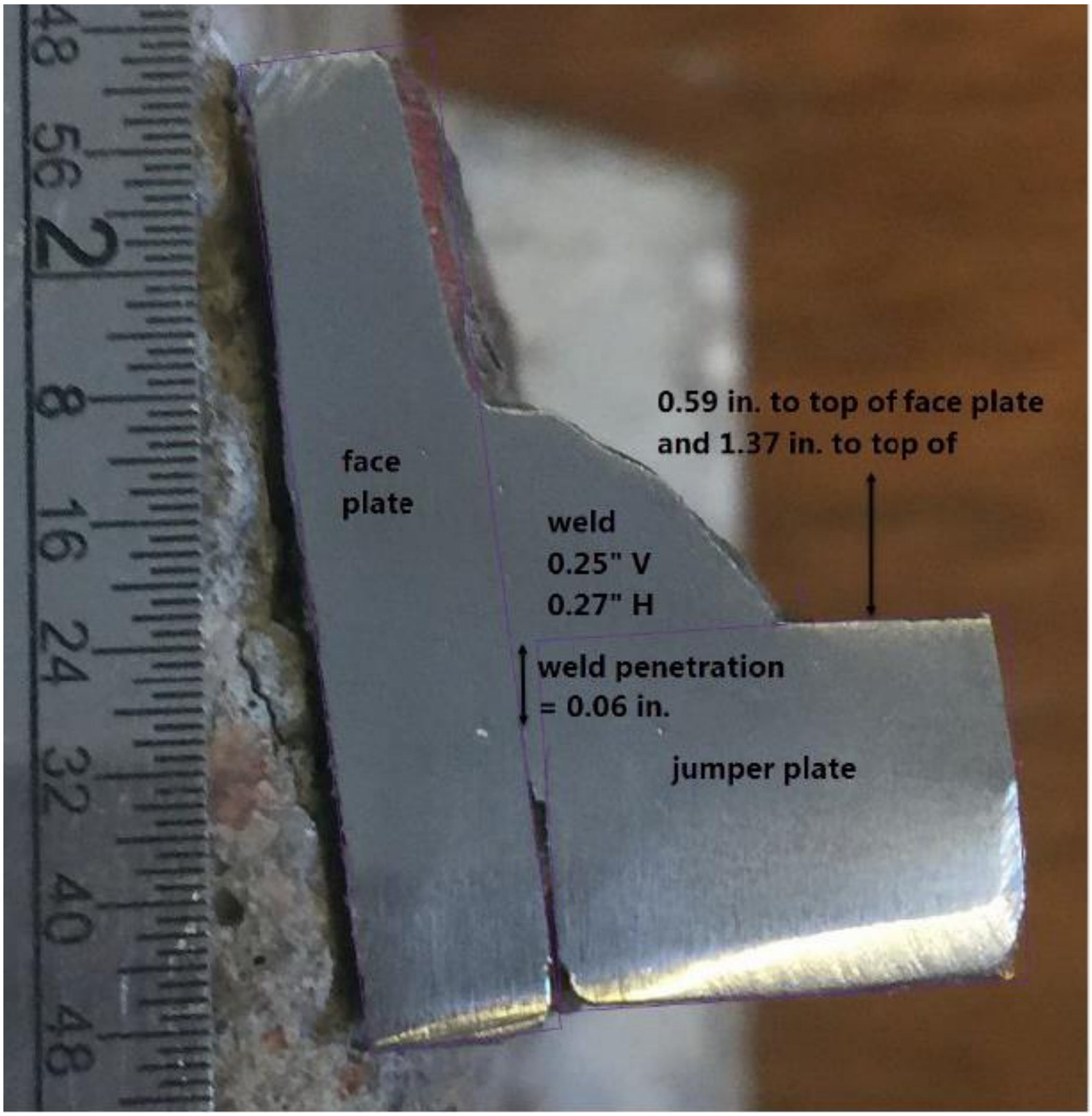

Figure 12 Excerpt from "Double Tee Flange Connections – Experimental Evaluation" Figure 15. [65] **Note gap between jumper plate and face plate.**

Average Stress vs. Maximum Stress

Throughout the Task Group's papers, the authors maintain that the flexibility of the connection creates complications that cannot be analyzed with simple calculations. Specifically, due to the flexibility of the boundary conditions and the configuration of the traditional flange-to-flange connection, the stress varies considerably over the face and root of the welds.[66] The Task Group faulted the Keenan Paper for not adequately exploring the influence such flexibility would have on the behavior of the connection and thus ignoring the presumed beneficial effects. Rather, the Task Group asserts that such flexibility "results in a more compliant detail"[67], which benefits the connection such that the fatigue resistance is improved.

The Keenan Paper assumed the embedments as fixed along the length of the weld, such that applied moment is distributed evenly across the length of the weld. As such, the stress at any point along this length is equal to the average stress. The moment at either weld is a fixed quantity, a function of shear force and joint width. Thus, any mechanism that causes stress to be unevenly distributed across the weld serves to remove stress from one location and concentrate it at another. Since it could be demonstrated that the connection was faulty without the addition of such stress risers, it was superfluous to explore the increase in stress created by the flexibility of the connection.

The typical stress along the length of the weld, as calculated by the Task Group, is depicted in Figure 13, which shows both calculated stress along the length of the weld and the average stress.[68] It is apparent from this graph that the anchorage of the common flexible connection to the concrete beyond either end of the welds causes stress to be concentrated at the weld ends. This was confirmed by the Task Group, which concluded that the "…stress varies along the length of the weld due to the flexibility of the faceplate."[69]

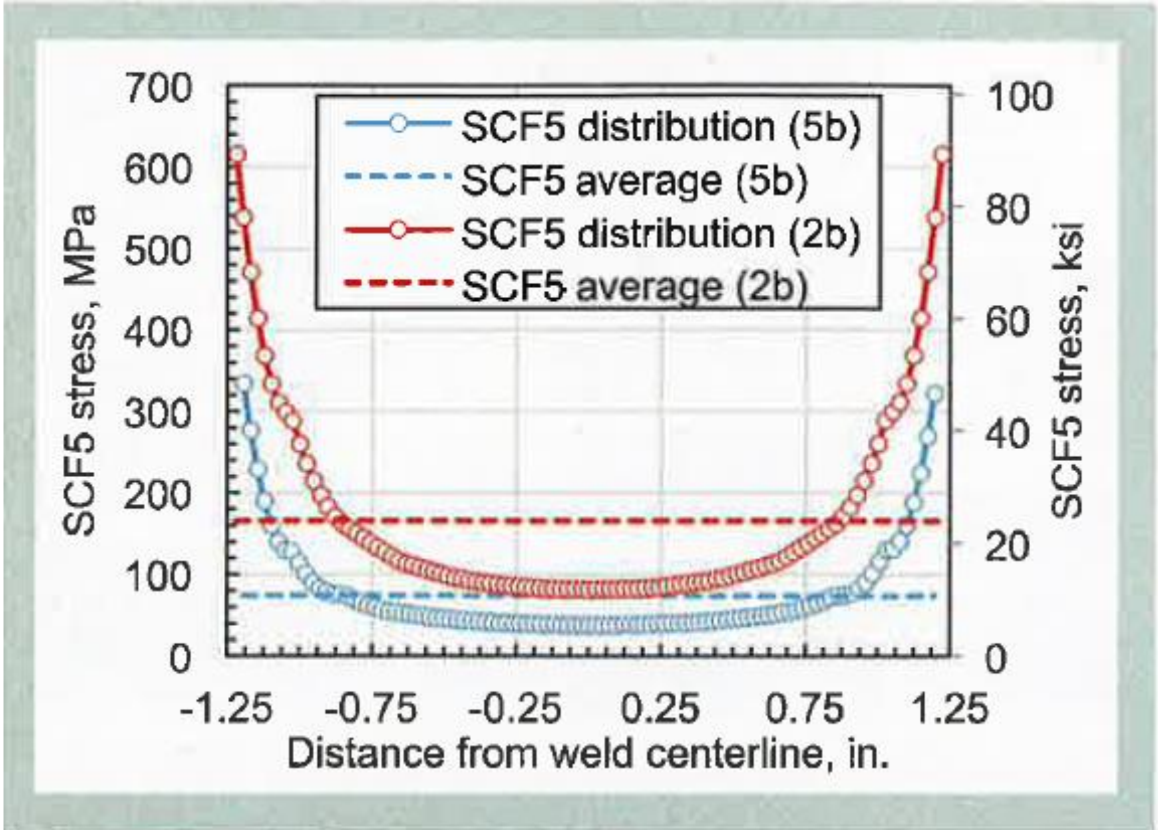

Figure 6. SCF5 stress in welds. Note: SCF5 = stress concentration factor. 1 in. = 25.4 mm.

Figure 13 "Flange-to-flange double-tee connections subjected to vehicular loading part 2: Fatigue life assessment" Figure 6[70].

Nonetheless, the increase in stress at the ends of the weld was not considered by the PCI Task Group for design purposes, despite their determination through fatigue testing that *"…the fatigue failure surfaces started at the ends of the weld and propagated toward the middle. The failure surface was also greater at the ends and smaller at the middle of the weld due to the elevated stress generated from the flexibility of the faceplate."*[71] The final calculations for fatigue, which resulted in a predicted 50+ year design life, did not use the *maximum* stress calculated but rather were performed using the much lower *average* calculated root stress along the weld.[72]

The design methodology utilized by AISC and AWS for fatigue resistance is Allowable Stress Design (ASD).[73] The basic principle of ASD is that service load stress does not exceed a prescribed allowable stress under service conditions. Although the PCI Task Group calculated that the maximum stress was located at the ends of the welds, and that it far exceeded that of the average stress, they persisted in using the lower average stress values to determine fatigue life. This approach is in direct conflict with the principles of allowable stress design and defies code and industry standards provided by AISC and AWS. In short, the allowable stress was exceeded, invalidating the Task Group's conclusions.

Factor of Safety of S-N Curve

The Task Group conducted a series of fatigue tests on a generic subassembly of the flexible connection. The test specimens consisted of cantilevered erection bars welded to sections of steel face plate. The face plates were anchored such that they mimicked the flexibility of the typical embedded connection. The welds were loaded such that the roots of the welds were directly stressed in tension. Twenty test specimens were tested, and the results were used to create an S-N Curve (Figure 14). This curve, which was a plot of mean/average fatigue life values, was then used to calculate the fatigue life of the model garage.

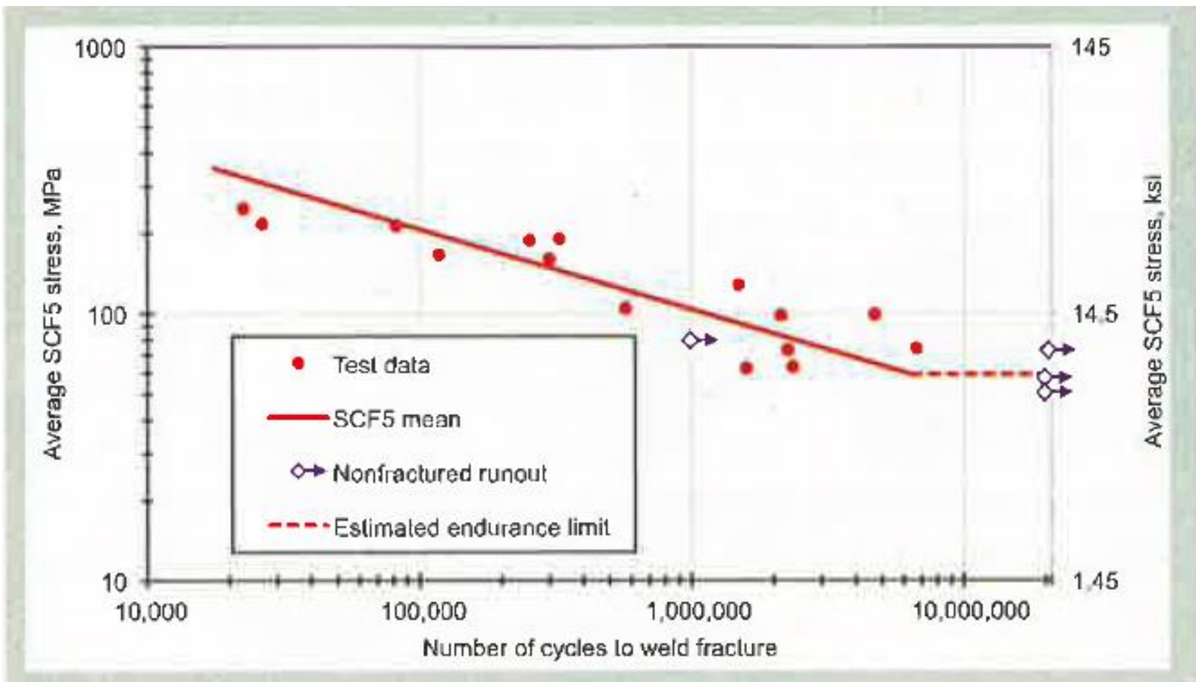

Figure 8. Fillet weld S-N curve. Note: SCF5 = stress concentration factor.

Figure 14 "Flange-to-flange double-tee connections subjected to vehicular loading part 2: Fatigue life assessment" Figure 8[74].

Upon cursory examination, it would appear that the endurance limit (fatigue threshold) for these specimens, loaded in a manner that stresses the root of the weld in tension, was determined to be 8.6 ksi,[75] which exceeds that of an AISC/AWS Category F fillet weld utilized as commonly intended (8 ksi). However, close review of these results reveals that the Task Groups S-N curve is expressed in *mean/average* stress range at failure, while the industry standards are expressed in *allowable* stress range for design purposes. An inquiry to the Chairman of the AWS Welding Code provided insight as to how a factor of safety was incorporated into the AISC/AWS S-N Curve: "The data contained in the chart started as experimental results with experimental variations. The mean was reduced two sigma, so 97.7% of the experimental data lies above the design curve."[76] No such allowance for safety has been claimed by the Task Group, rather the average stress range at failure was utilized for determination of the design life of the model garage.

It is important to recall that it is the position of the Task Group that, "*as a Category F detail, the fatigue life expected by AISC would be negligible."*[77] It seems unlikely that the modest increase in the fatigue threshold (F_{TH}) from 8.0 to 8.6 ksi would dramatically change the fatigue life expectancy from "negligible" to acceptable for a 50+ year period. In any case, following the methods used by AISC/AWS to decrease the Task Group's test data by two sigma would reduce the fatigue threshold below that of the AISC Category F detail, thus affording the connection negligible fatigue life per the Task Group's stated opinion.

In utilizing an S-N curve based upon the average stress range at failure, the Task Group did not establish a 50+ year design life, but rather predicted the life expectancy by which half of the welds would fail. With two welds per connection, failure of only one weld is required for failure of the connection as a whole. Further, failure of a single connection would cause failure of additional connections along the same line through stress redistribution. A garage designed under such parameters would completely fail well before the stated 50+ year life expectancy.

It is unclear if the Task Group's choice to follow the design methods of the offshore industry, rather than applicable building codes and industry standards, accounts for their use of *average/mean stress* in lieu of the *allowable stress range* that forms the basis for the AISC and AWS S-N Curve and Fatigue Threshold (F_{TH}).[78] Nevertheless, incorporating a sufficient factor of safety is a universal requirement of applicable building codes.

Materials Used for Models and Testing

Most double-tee garage flange connections are currently constructed of stainless steel. While regional climate variations may allow mild steel connections to be utilized in some locales, stainless steel is the recommended material for connections in garages subject to severe weather and/or deicing chemicals. However, stainless is generally weaker than mild steel when subject to loading and requires a larger factor of safety. [79] Therefore, as stainless steel is both the more commonly used metal and the inferior material for structural performance, it would have been the appropriate choice for testing and analysis, especially should it prove impractical to perform a separate study specific to this material.

It is notable, then, that the models used by the PCI Task Group for analysis was structural steel with a modulus of 29,000 ksi[80], and the specimens used for fatigue testing and creation of the S-N curve were A36 and AWS E7018 respectively[81], which are both mild carbon steel. The results of the Task Group studies are therefore not applicable to stainless steel connections. There is no mention within the Task Group Studies that applicability to stainless steel connections remains unexplored or that the investigation results do not apply to stainless steel connections in general.

Call to Action

It is the position of the PCI Task Group that fatigue failure of double-tee connections is unlikely and, should it occur, would not precipitate further fractures along the joint, jeopardizing resistance to seismic forces. The following is reproduced in context:

"The research results clearly indicate that the force transfer across a flange to flange connection as a result of vertical loads on the floor is complex. Stresses can be high in some cases but the occurrence of fatigue induced cracking is limited to a few instances. These cases, if they were to occur, would manifest at the middle of the double tee span where vehicles are most likely to pass. As shown in the study, the vehicle loads are resisted primarily by local flange flexure and the connections nearest the load point. Consequently, in the rare case that a fatigue induced failure were to occur it is unlikely that failures would propagate along the joint to other connectors outside the drive path. Furthermore, the integrity of the floor system to lateral earthquake and wind loads would be minimally impacted in the rare case of a fatigue induced failure. As shown, possible failures would be at the midspan leaving the connections at the ends of the double tee, especially the chord connection, intact to resist the required in plane shear and tension/compression forces. In other words, precast double tee parking structures are indeed safe against fatigue induced failure due to seismic events."[82]

The claim that fatigue-induced failures do not precipitate further connection failures is simply stated but unsupported and patently false. The stress redistribution mechanism by which such progressive failures occur is not explored by the Task Group; however, it is a common observation in the garage repair industry that such failures do occur, often severing multiple diaphragms within the same garage and creating unsafe conditions. Further, failure of multiple connectors along a joint along will cause the moment applied to the cantilevered double-tee flange to nearly double, a condition for which they are not typically designed.[83]

Precast double-tee garages have become omnipresent in the landscape. The diaphragm connections used in their construction are poorly configured to withstand fatigue loading from vehicular traffic, violate well-established industry standards and model codes, and are prone to fatigue failure. Such failures cause diaphragms to be severed, jeopardizing the general stability of these structures. The precast/prestressed industry has undertaken extensive studies and published four reports with conclusions in direct opposition to these findings. In short, both opinions cannot be correct on this important issue. Investigation must be performed by unbiased model code institutions having requisite expertise and resources to determine if these structures can be considered safe over the life of the structure.

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