ANALYSIS AND DESIGN OF WELDED PRECAST DOUBLE-TEE CONNECTIONS FOR CYCLICAL FATIGUE FROM VEHICULAR LOADING

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ABSTRACT

Welded tee-to-tee flange connections of precast concrete parking garages are poorly configured to withstand high-cycle fatigue loading from vehicular traffic. These connections violate Building Code requirements and are prone to fatigue failure. This study examines Code requirements and industry standards as they apply to high-cycle fatigue performance of commonly used details. Parameters discussed include connection configuration, cycle analysis, stress intensification effect, Allowable Stress Range, Stress Category, potential crack initiation point, and allowable design capacity. An example of the most commonly used connection detail, consisting of fillet welds on a flat erection bar, is analyzed to demonstrate explicit and implicit violations of American Welding Society (AWS) and American Institute of Steel Construction (AISC) Code provisions. Scanning electron microscope images of fractured welds are analyzed to demonstrate the fatigue failure mechanism. Consequential effects of fatigue failure and considerations for optimized fatigue-resistant connection design are also discussed.

Keywords: Connections, Construction, Repair and Rehabilitation, Research

INTRODUCTION

Precast double–tee construction has become a dominant method for constructing parking garages in North America. The double-tee beam deck system of this construction type simultaneously provides a finished wearing surface, supports vehicle loading, and forms an integral part of the lateral force-resisting system. Connections between double-tee beams are subject to a varied array of loads, including tension, compression, and shear from seismic events; tension and compression from temperature-induced expansion and contraction of the deck; and gravity loading from vehicles. Vehicular loading is comprised of static loading from parked vehicles and dynamic cyclic loading from vehicle movement, which causes fatigue stress in the tee-to-tee connections. Unfortunately, the typical pre-topped double-tee connection is poorly configured to withstand dynamic cyclical loading and is nonconforming to Code provisions in this regard, which has led to fatigue fracture of the welds. Consideration must therefore be made to improvement of the overall connection design.

This paper explores requirements of the International Building Code (IBC), the American Welding Society (AWS), the American Institute of Steel Construction (AISC), and industry standards for the design of connections resilient to fatigue effects from cyclic loading.

LOADING

The rhythmic thumping of tires while driving through a precast parking garage is a familiar experience of modern life. As the wheel traverses each beam, force is applied, the magnitude of which is dependent on the difference in elevation of the beam surfaces, the weight of the vehicle, its velocity, suspension type, and even the pressure of air in the tires.

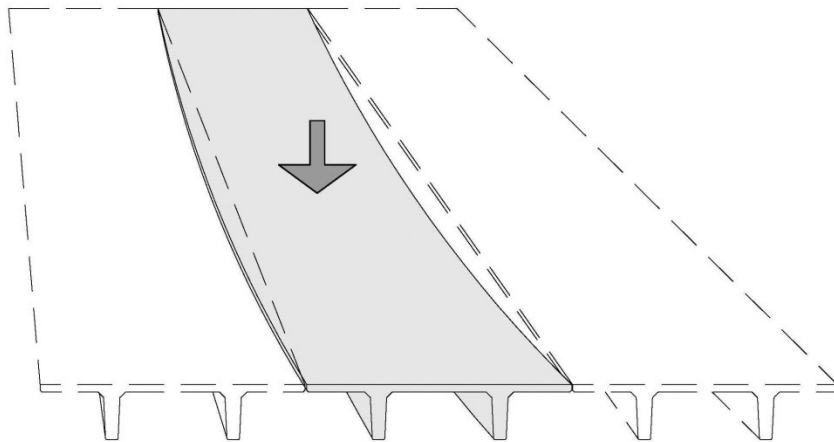As load is applied to a double-tee beam it deflects downward (Figure 1).

Figure *1* Deflection of double tee beam under loading.

The adjacent beams resist this deflection through individual connections located along the beam flanges. Under symmetric loading as shown in Figure 1, the beam deflects uniformly and connections on either side of the beam share the load equally. As the load approaches the edge of the beam, torsional rotation of the beam occurs, transferring a portion of the load to the connections along that side. Localized warping of the flange under this asymmetric loading also occurs, concentrated at the point at which the load is applied; however, the effects associated with this localized warping are secondary in comparison to the vertical deflection and torsional rotation that occurs over the typical 60' beam length.

As the load traverses the joint, the loading of the connections is abruptly reversed (Figure 2). As is apparent when considering the deflected shape of Figure 1, the connections close to the center of the beam resist a greater amount of deflection. Consequently, the magnitude of the resistive force applied to each connection is proportional to the number and spacing of the connections in relation to this deflected shape.
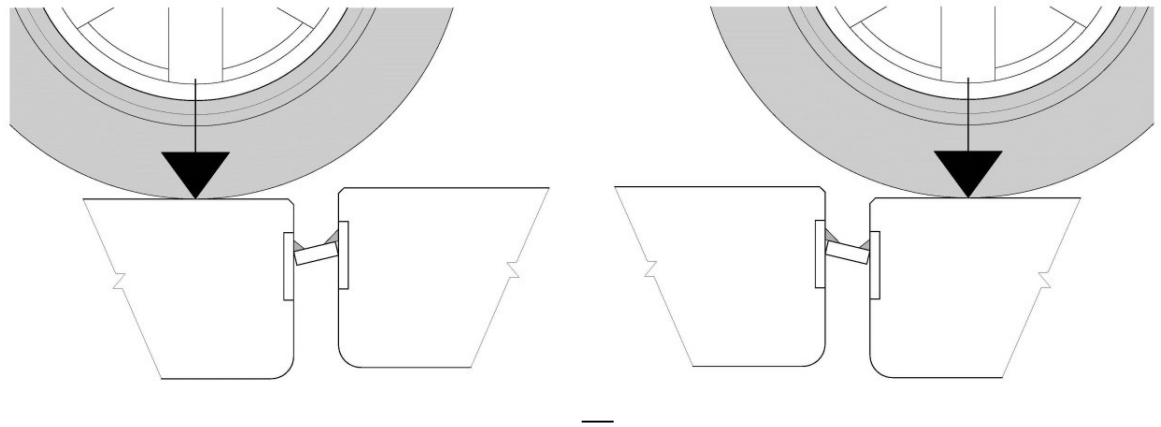

Figure 2 Exaggerated depiction of beam deflection at flange joint connection.

As load is applied to the connection, relative deflection of the adjacent concrete flanges is resisted by the field-installed erection bar. This erection bar, being fixed on either side by welds, is forced into reverse curvature (Figure 3). Bending moment is applied to each weld about an axis parallel to the joint, much like a hinge. This creates concentration of tensile stress at the root of the weld, followed immediately by compression stress as the wheel traverses the joint. With enough cycles, this stress reversal can lead to fatigue failure of the weld (Figures 4 through 7).
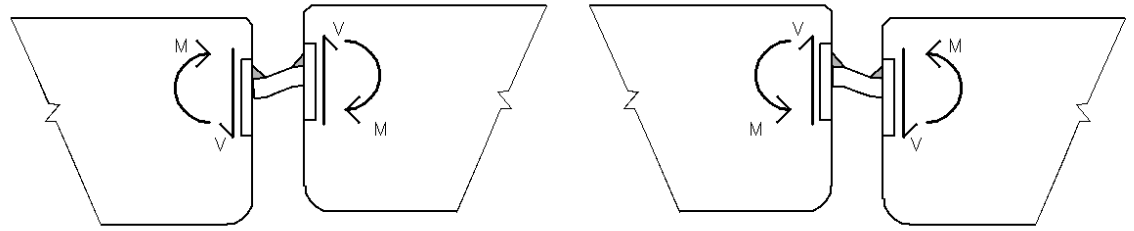
Figure *3* Internal forces and deflected shape (exaggerated) of typical flange connection under vertical loading.

Figure *4* Fatigue failure of flat bar-type connection.

Figure *5* Fatigue failure of flat bar-type connection.

Figure *6* Fatigue failure of flat bar-type connection.

Figure *7* Fatigue failure of dowel-type connection.

FATIGUE

Fatigue is the process by which a material becomes weakened through cyclic loading. With many thousands to millions of cycles, cracks can develop within the elastic range of the material. This is termed *high cycle fatigue*. Over time, repeated loading leads to the propagation of these cracks, which compromises the strength of the material and can ultimately lead to failure. Fatigue failure occurs in three steps:

1. *Crack Initiation (Nucleation):* Surface discontinuities precipitate crack formation by causing a localized increase in stress and a nucleation point for crack initiation. This stage may be suspended or delayed by smooth surfacing, or it may be triggered prematurely by notches, sharp corners, irregularities, and any configuration that does not allow the smooth transfer of stress. In fillet weld connections, the root of the weld is highly susceptible to crack initiation. The joint between the connected plates forms a severe notch creating a tear point directly at the root of the weld. Furthermore, the weld metal deposited along this notch can be highly irregular, with variable weld penetration, porosity, and slag inclusions forming precipitating points for crack initiation.

2. *Crack Propagation:* The crack grows at this stage in a slow, stable manner, the rate of which is dependent on the magnitude of the stress cycle. As cycles increase, microscopic cracks grow into macroscopic cracks. If the connection were acting alone, stress on the remaining cross-section would increase as the intact cross-section diminished, accelerating the rate of propagation. However, the loss of cross-sectional area causes a corresponding reduction in the elastic section modulus of the remaining weld area, which in turn allows rotational deformation of the connection to occur more freely, transferring load to adjacent connections. Consequently, crack propagation at double-tee connections can extend farther across the weld throat than would otherwise be expected.

3. *Failure:* When the strength of the remaining weld cross-section is insufficient to withstand the applied load, failure occurs. This type of failure is typically abrupt. As noted above, failure can be substantially delayed through load redistribution to adjacent connections; this allows crack propagation to approach the weld face prior to fracture of the remaining cross-section. The resultant abrupt failure therefore often appears as a thin ridge along the face of the weld (Figure 8).
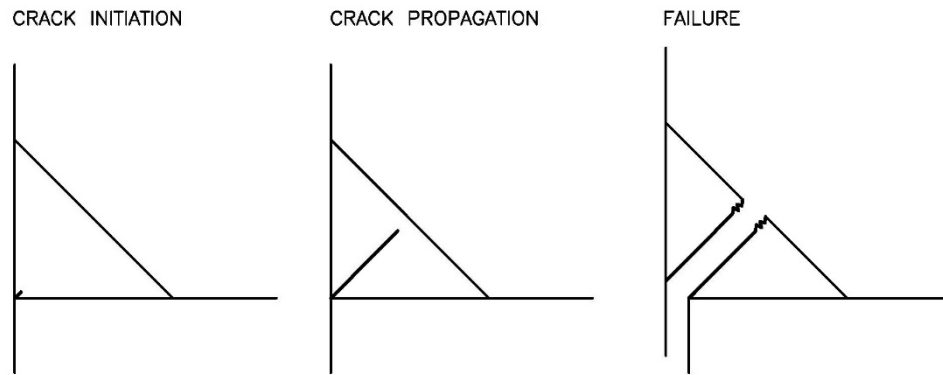
CRACK INITIATION   CRACK PROPAGATION   FAILURE

Figure *8* Three stages of fatigue failure of a fillet weld in a double-tee connection.

It is a characteristic of high cycle fatigue that crack initiation and propagation need not occur at high stress. The main factors leading to fatigue failure are the number of cycles, the severity of the stress concentration created by surface defects and the geometry of the detail, and the *stress range*. Stress range is defined as the magnitude of the fluctuation in stress that occurs in each cycle. Importantly, in the case of stress reversal, this is the numerical summation of the maximum repeated tensile *and* compressive stresses.

As the stress range decreases, the number of cycles the material can withstand before succumbing to fatigue cracking increases. In steel, there is a point at which the stress range is so low that fatigue cracking will not initiate no matter how many cycles are applied. The level of stress range necessary for crack initiation is known as the *fatigue threshold* (F_{TH}) and varies depending on the severity of surface defects and the connection geometry.

Fatigue fractures are easily distinguished from other types of connections failures by appearance. The failure surface typically has two visibly distinct regions that are created during the crack propagation and failure stages. The crack propagation stage is distinguished by a smooth, burnished appearance, a result of microscopic striations arranged in parallel. These *beach marks* are a visual record of the numerous steps in crack formation caused by individual cycles of loading.

In a double-tee connection, fatigue failure occurs through the weld throat. The configuration of the connections creates a severe notch at the root of the weld that is the precipitating point for crack initiation. Fatigue fractures of double-tee connections typically have the following distinctive features:

- The crack is very straight and uniform in the crack propagation zone;

- Beach marks are parallel to the axis of the weld creating a burnished appearance with a directional sheen when viewed by the naked eye;

- Crack initiation occurs at the root of the weld, propagation through the throat, and fracture at the face. This creates a thin ridge along the face of the weld where the abrupt failure has occurred.

Figure 9 shows a flat bar-type flange connection that failed due to fatigue. In this photo (noting that erection bar is shown upside down), we see a straight and uniform crack propagation zone, a smooth burnished surface on the failed weld surface, and a thin ridge where abrupt fracture occurred at the face.

Figure 9 Fatigue fracture at flat erection bar (shown upside-down). Note uniform and burnished crack propagation zone and linear striations at face of weld fracture (red arrow) and abrupt fracture at face of weld (black arrow).

Figures 10A and 11A show failed round dowel and flat bar-type double-tee flange connections, respectively. Close review of the fracture surface reveals a burnished appearance with distinguishable beach marks. Figures 10B and 11B are scanning electron micrographs of fracture surfaces for each of the two connection types. At the microscopic level, numerous parallel striations, or beach marks, are clearly visible, each representing the point to which the metal fractured in tension before stress reversal placed the cross section in compression.

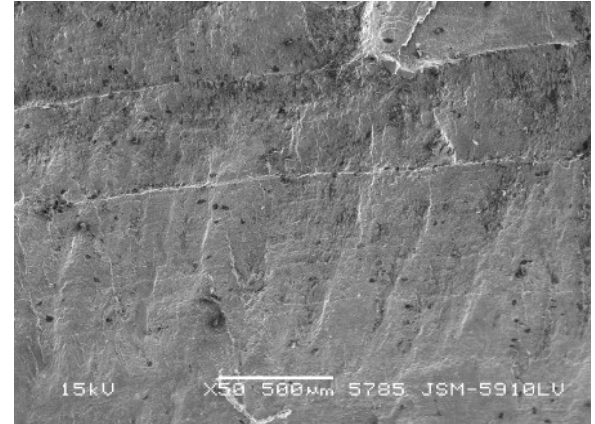**Figure 10A** Fatigue failure of a dowel-type connection.

Figure 10B Fatigue failure of a dowel-type connection; scanning electron micrograph.

Figure 11A Fatigue failure of a flat bar-type connection.

Figure 11B Fatigue failure of a flat bar-type connection; scanning electron micrograph.

CODE REQUIREMENTS AND INDUSTRY STANDARDS

This paper assumes the International Building Code (IBC) to be the governing Code for design of buildings.[1] In use or adopted by all 50 states and the District of Columbia, the IBC

provides requirements for loading, design, and construction of various building construction types, such as steel, concrete, masonry, etc. However, precast concrete is a hybrid type of construction for which not all IBC requirements are easily applied.

Precast concrete is manufactured as a concrete product, and the IBC requirements regarding concrete design are directly applicable. However, it is delivered and erected as discrete, individual pieces, utilizing methods and materials beyond the scope of concrete sections of the IBC and the concrete code referenced therein. The following discussion therefore encompasses a review of IBC, referenced model Codes, and industry standards with specific regard to stainless steel double-tee flange connections.

For concrete construction, the IBC provides specific requirements but largely defers to the American Concrete Institute's ACI 318, *Building Code Requirements for Structural Concrete*.[2] However, these concrete standards provide no reference to welded connections other than for welded reinforcement steel. For steel construction, IBC likewise provides specific requirements, but largely defers to the American Institute of Steel Construction's AISC 360, *Specifications for Structural Steel Buildings*.[3] However, AISC 360 applies to a very limited and specific range of structural steels, for which all stainless steel varieties are excluded. As virtually all double-tee deck connections are currently constructed of stainless steel, this reference likewise does not directly apply.

In fact, the IBC offers no direct mention of Code requirements for the use of stainless steel in construction. For such 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.[4]

For accepted engineering practice, we look to industry guidance for both design standards and Code requirements. For the construction of welded stainless steel connections, the following sources are considered herein:

1. *PCI Design Handbook*, by the Precast/Prestressed Concrete Institute (PCI);

2. *PCI Connections Manual for Precast and Prestressed Concrete Construction*, by the Precast/Prestressed Concrete Institute (PCI);

3. *Precast Prestressed Concrete Parking Structures: Recommended Practice for Design and Construction*, by the Precast/Prestressed Concrete Institute (PCI);

4. *AWS D1.1, Structural Welding Code – Steel,* by the American Welding Society (AWS);

5. *AWS D1.6, Structural Welding Code – Stainless Steel,* by the American Welding Society (AWS);

6. *Design Manual for Structural Stainless Steel,* by The Steel Construction Institute;

7. *Steel Design Guide 27, Structural Stainless Steel* by the American Institute of Steel Construction (AISC);

8. AISC 360, *Specifications for Structural Steel Buildings,* by the American Institute of Steel Construction (AISC); and,

9. *AISC Steel Construction Manual,* by the American Institute of Steel Construction (AISC).

General requirements of each of the above referenced standards as they specifically apply to the design of welded stainless steel connections for fatigue are summarized as follows:

PCI Design Handbook: Code and industry requirements for fatigue resiliency are not listed as design criteria or as durability considerations. However, according to Section 6.7.2, "Weld Design," the limits on weld stress are based on AWS D1.1, while the design strength of welds should be determined from the provisions of AISC *Steel Construction Manual*, which contains the AISC 360 (both discussed below).[5]

Design of flange-to-flange double-tee connections is not directly discussed. However, Example 5.12.1.1 is provided which indicates that where a 3 kip concentrated load is applied to the deck the 3-kip load may be distributed to two adjacent double tees (1.5 kips per double tee) because of the flange to flange connection. (Figure 12 below); [6]
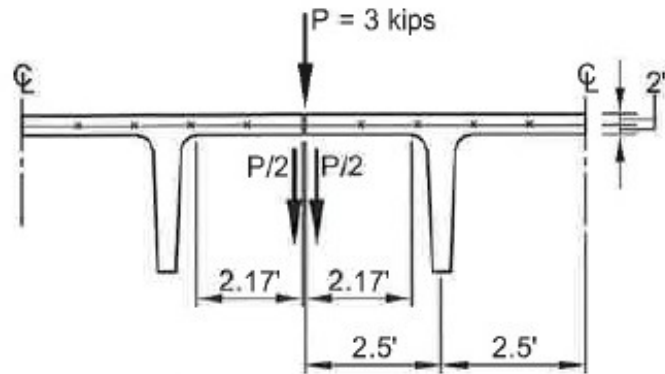
Figure *12* Excerpt from Example 5.12.1.1, Flexural Strength of Double-Tee Flange in Transverse Direction. Credit: Precast/Prestressed Concrete Institute, Force, Greg, et all, PCI Design Handbook, 7th ed., 2010.

This illustration indicates that the design load transferred across the joint is equal to one half the applied load. Further the design example assumes that half the load is transferred across a single connection at the point at which the load is applied.

PCI Connections Manual for Precast and Prestressed Concrete Construction: One example is given for the design of double tee flange-to-flange connections. Design Example 3.12 demonstrates the design of a dowel type flange connection (Figure 13 below). [7]
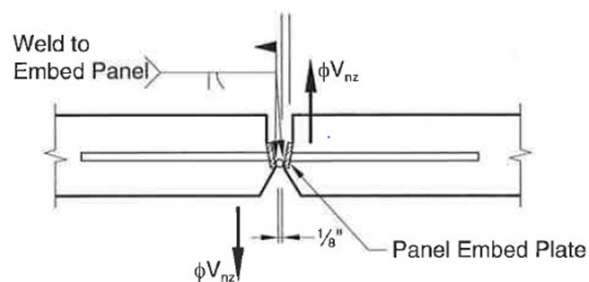
Shear Capacity:
$$\phi V_{nz} = \phi(0.6F_{exx})t_w \ell_w$$
$$= 0.75\left[0.6(70)\right](0.075)(4)$$
$$= 9.5 \text{ kip}$$

– Figure *13* Excerpts from Example 3.12, Double-Tee Flange to Flange. Precast/Prestressed Concrete Institute, Sennour, Larbi et all, PCI Connections Manual for precast and prestressed concrete construction, 1st ed., 2008.

The flare bevel groove weld of the connection is assumed to be in pure shear; the width of the joint and the stress intensification effects of the eccentricity of the connection geometry are not considered. Code and industry requirements for fatigue

resiliency are not listed as design considerations.

The calculation for the effective throat (t_w) of the flare bevel groove weld used in Example 3.12 (Figure 13) is erroneously assumed to be the same as that for a flare bevel groove weld on rebar, which is 0.2 times the bar diameter. As the effective throat (t_w) of the flare bevel groove weld is actually equal to 5/16R (where R = bar radius) this assumption is incorrect and overestimates the strength of the weld in the example by 28%.

Precast Prestressed Concrete Parking Structures: Chapter 3.0 "Durability Concerns" includes Section 3.6, "Joint Connections", which indicates that a properly detailed flange-to-flange connection is made with the minimum size weld, to minimize cracking of the concrete.[8] Fatigue loading is not listed as a durability concern.

Figure 3.7, "Pretopped Flange Connector", depicting proprietary bent plate embedments welded with fillet welds to a flat connection plate, is provided as a properly detailed connection (Figure 14 below).[9]
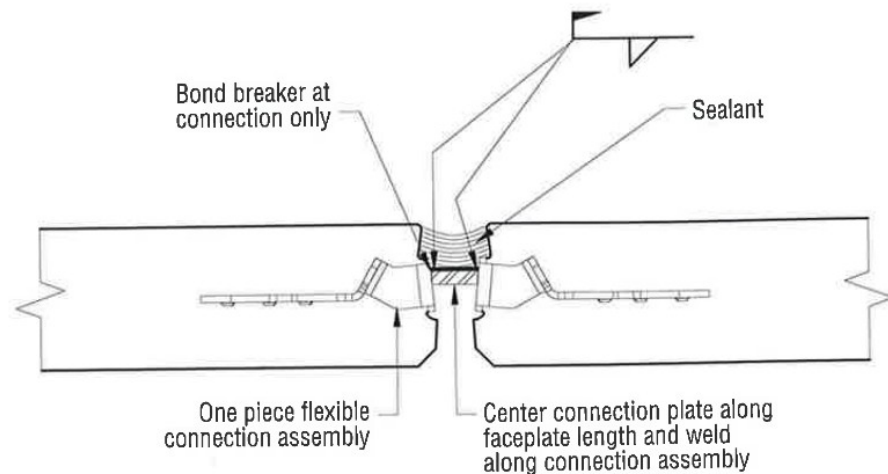

Figure 3-7 Pretopped Flange Connector

Figure *14* Figure 3-7, Pretopped Flange Connector. Precast/Prestressed Concrete Institute, Cleland, Ned et all, Precast Prestressed Concrete Parking Structures: Recommended Practice for Design and Construction, 3rd ed.,

2015.

Chapter 6.0, "Connections" states that "some connections used in precast parking structures are subject to significant and cyclic movement."[10] However, the connection type to which this refers is not stated and there is no further mention of cyclic loading. Code requirements with regard to cyclic loading are not mentioned as a design consideration.

AWS D1.1, Structural Welding Code – Steel: Per Section 1.1, "Scope," the AWS D1.1 is intended to be applied to welding of carbon or low-alloy steel structures but may be used for other types of steel construction based upon the engineer's evaluation of its suitability. However, the AWS Structural Welding Committee encourages that *AWS D1.6, Structural Welding Code – Stainless Steel* be considered.[11]

Per Section 2, Parts B and D, calculated stress shall include those stresses due to eccentricity.[12] Therefore, moment applied to the weld due to the eccentricity of joint width of the connection must be considered.

Section 2, Part C, "Specific Requirements for Design of Nontubular Connections (Cyclically Loaded)," requires that fatigue be considered and lists requirements for fatigue-resistant connection design. Allowable Stress Range (F_{SR}) is defined as the numerical sum of the maximum repeated tensile and compressive stress. It is based upon the number of cycles (N) and the constant (C_f) relating to the Stress Category, which is obtained by application of pictorial representations and descriptive narrative of connection configurations and geometries shown in Table 2.5, "Fatigue Stress Design Parameters".[13] Notably, there is no pictorial representation suitable in configuration to act as an analogue for the welds used at flat bar type double-tee connections in this Table;[14] therefore these requirements do not anticipate a connection of this geometry and provisions of this Code cannot be applied.

Commentary Section C-8.4, "Fatigue Life Enhancement", states that fatigue crack initiation from the root area "…should not be overlooked."[15]

With regard to cyclically loaded dowel type double-tee connections, Section 2, Part C, Subsection 2.18, "Prohibited Joints and Welds" states that groove welds, "made from one side", are prohibited unless both the welders and the Weld Procedure Specifications are "qualified", as demonstrated through laboratory testing, in conformance with Chapter 4.[16] As such welder qualification is specific to the individual performing the welds, and since this connection type is generally no longer used, this type of connection will not be considered further herein.

AWS D1.6, Structural Welding Code – Stainless Steel: Per Section 2, Part A, 2.2.2, "Bending Stress": "Corner and T-joints that are to be subjected to bending about an axis parallel to the joint shall have welds arranged so as to avoid concentration of tensile stress at the root of any weld."[17] AWS D1.6 therefore explicitly prohibits the flat bar-type and dowel-type connections discussed herein, as tensile stress in these connection configurations is directly concentrated at the root of the weld. The fact that such loading is prohibited under static conditions underscores the poor overall design of these connections to withstand the more severe cyclical loading.

Per Section 2, Part A, 2.3.3, "Fatigue Provisions," consideration shall be given to the "stress intensification effects" of weld details subject to cyclical loading.[18]

Design Manual for Structural Stainless Steel: Per Section 6.4.2, "Fillet Welds," "A fillet weld should not be used in situations which produce bending moment about the longitudinal axis of the weld if this causes tension at the root of the weld."[19] Similar to that of AWS D1.6 discussed above, this passage explicitly prohibits the use of flat bar-type and dowel-type connections discussed herein, even under static loading conditions, as tensile stress in these connection configurations is directly concentrated at the root of the weld. Furthermore, per Section 8, "Fatigue," fatigue assessment is required for rolling loads and the following should be avoided to eliminate fatigue problems: [20]

1. Sharp changes in cross-section and stress concentrations;

2. Eccentricities;

3. Partial penetration groove welds; and,

4. Fillet welds.

Round and flat bar type double-tee connections typically have all of these characteristics underscoring the poor overall design of these connections to withstand cyclical loading.

AISC Steel Design Guide 27, Structural Stainless Steel: Per Chapter 9, Section 9.2, "Design of Welded Connections," welding should be carried out in accordance with AWS D1.6, and a reduced resistance factor or increased factor of safety should be used for stainless steel.[21]

Per Chapter 11, "Fatigue", fatigue strength of welded stainless steel connections may be estimated in accordance with AISC 360, *Specifications for Structural Steel Buildings,* discussed below. However, as noted in the Design Manual for Structural Stainless Steel above, it is cautioned that the following should be avoided to eliminate fatigue problems: stress concentrations, eccentricities, partial penetration groove welds, and fillet welds. [22] As above, this underscores the poor overall design of these connections to withstand the more severe cyclical loading.

AISC 360, Specifications for Structural Steel Buildings: Per AISC 360, fatigue shall be considered in accordance with Appendix 3, "Design for Fatigue".[23] Requirements of Appendix 3 are largely identical to that of AWS D1.1, with the exceptions that no evaluation of fatigue resistance is required unless the number of cycles exceeds 20,000[24] and that no provisions are given for flare bevel groove welds.

Similar to AWS D1.1 above, the Stress Category is obtained by application of pictorial representations and descriptive narratives of connection configurations and geometries shown in Table A-3.1, "Fatigue Design Parameters".[25] Notably, as with AWS D1.1, there is no pictorial representation suitably analogous to the welds used at typical double tee connections; these requirements do not anticipate a connection of this geometry and so cannot be applied.

Review of the AISC 360 Appendix 3 Commentary reveals that for fatigue design: [26]

1. Stress range and notch severity are the dominant stress variables for welded details;

2. Other variables such as minimum stress, mean stress and maximum stress are not significant for design purposes; and,

3. For cases involving cyclic reversal of stress, the calculated stress range must be taken as the sum of the compressive stress and the tensile stress.

AISC Steel Construction Manual: According to AISC, Part 8, "Design Consideration for Welds", in the case of one sided fillet welds, which are the type used in double-tee construction, "…the unwelded side has no strength in tension and a notch may form from the unwelded side." A figure is provided (Figure 15 below) indicating a crack initiation point at the root of the weld. [27]
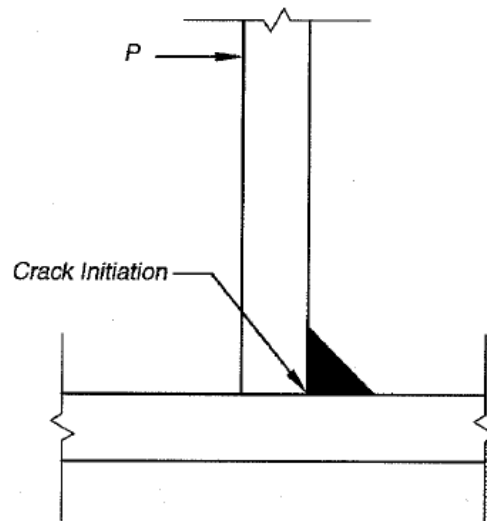

Figure 8–8. Notch effect at one-sided weld.

Figure *15* Figure 8-8, AISC Steel Construction Manual, Thirteenth Edition", American Institute of Steel Construction, 2005, p. 8-15

The similarities of the Figure 8-8 to the common precast double-tee connection are striking. It is important then to note that the recommendations given in this section,

and the figure provided, apply to static loading of welds, which is less severe than fatigue loading.

Precast concrete industry sources cited above do not specifically mention cyclical vehicular fatigue loading of double-tee flange-to-flange connections as either a design or durability consideration. However they do reference AWS and AISC as the authoritative Codes for design of welded connections, both of which require cyclical fatigue loading be considered and provide requirements for fatigue resistant design.

AWS and AISC requirements for the design of welds for fatigue resistance are, for practical purposes, identical. Both sources require that fatigue analysis be performed. Both sources also use pictorial and narrative representations to determine the stress category, but neither offer a weld configuration similar to that used in double-tee connections. Neither recommends such connection loading geometry be used, even under much less severe static loading conditions; AISC warns that a crack will form at the root of the weld, and AWS explicitly prohibits that stainless steel welds being used in this manner.

In summary, there is no support outside of the precast concrete parking garage industry for the use of such connections. All Code sources cited herein either recommend against such a configuration or explicitly prohibit it.

It is also apparent, from the limited design information available within the precast concrete industry that forces applied to this type of connection from vehicular loading and the stresses imparted on welds due to the connection geometry have not been fully considered. The assumption that one half of a point load applied to the deck is transferred across an individual connection is erroneous and overestimates the forces in the connection. Conversely, disregarding the bending moment applied to the welds from the joint eccentricity grossly underestimates the stresses in the welds.

Bending moment within the weld due to the joint eccentricity during loading can be substantial. For the connection in Figure 16, the allowable design strength of the fillet weld is calculated to be 11,100 lbs. when acted upon only in static shear. When static bending is considered the strength is reduced to only 656 lbs. (assuming that such weld configurations were permissible) using basic static analysis. This equates to a 94% reduction in strength. Therefore the connection geometry has a substantial stress intensification effect that must be considered. When fatigue loading is taken into account these effects become far more significant.
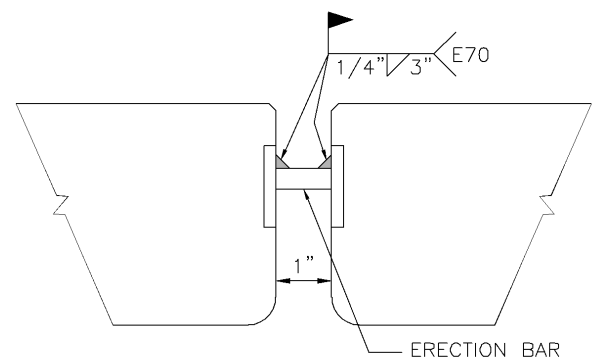

Figure *16* Welded double-tee connection example.

DESIGN EXAMPLE

The following example of a typical precast, pre-topped, double-tee connection shall be analyzed to demonstrate the use of Code requirements, to the extent that they can be applied. Applicable Codes do not allow connections of this geometry, consequently the Stress Category necessary for determining fatigue strength is not available. Instead, the Stress Range will be determined to demonstrate the magnitude of the stress intensification effect of the connection geometry and to ascertain if such a stress range is significant.

For this exercise, requirements of AWS D1.1, *Structural Welding Code – Steel*[28] shall be applied, although it is emphasized that requirements of AISC 360 *Specifications for Structural Steel Buildings* are largely identical.

EXAMPLE: Determine the Stress Range of the flat bar-type welded connection shown in Figures 17 and 18.

Given:

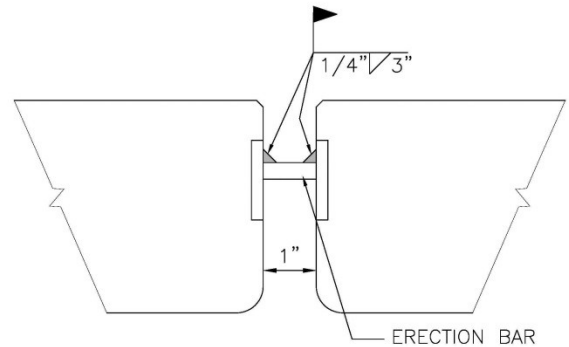|	
Double tee length	= 62'-0"
No. of connections	= 11
Connection spacing	= 6'-0"
Joint width	= 1"
Fillet Weld	= ¼" x 3"
Number of vehicles per day	= 500
Design life	= 30 years

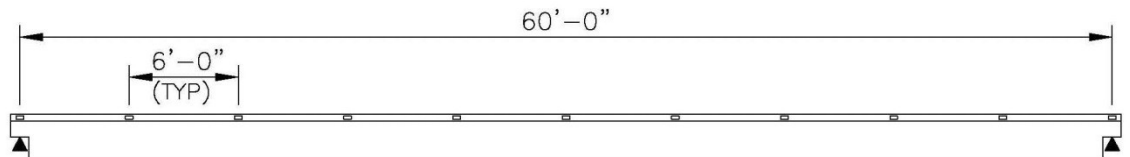**Figure 17** Example, flat bar-type connection.

Figure 18 *Beam elevation.*

First it must be determined if fatigue analysis is required. From the information given, there are 500 vehicles entering and exiting each day over a 30 year life. We assume that the vehicles enter and exit from the same location necessitating that each joint be traversed twice

per car. Cyclical loading may be calculated as follows:

500 cars x 2 axles x 2 passes	=	2,000 cycles/day	(1)
2,000 cycles/day x 365 days/year	=	730,000 cycles/year	(2)
730,000 cycles/year x 30 years	=	<u>21.9 Million cycles</u>	(3)

Per AISC requirements the threshold for requiring fatigue analysis (20,000 cycles) is reached within 10 days. The joints within this garage are therefore subject to very high cyclical loading and must be designed accordingly.

Environmental Protection Agency (EPA) statistics show that the average vehicle weight for the 2012 model year was 3,977 pounds, not including occupants.[29] The design loading for the average laden vehicle shall therefore be assumed to be 4,250 lbs. with a corresponding load of 2,125 lbs. per axle. The design load transferred across the joint will be one half the axle load, with each connection supporting a portion of the load. The load transferred across each connection is computed as follows.

As load is applied to the center of the beam, the beam deflects. Load is transferred across the joint through each connection. The magnitude of load transferred across an individual connection is proportional its deflection (Figure 19).
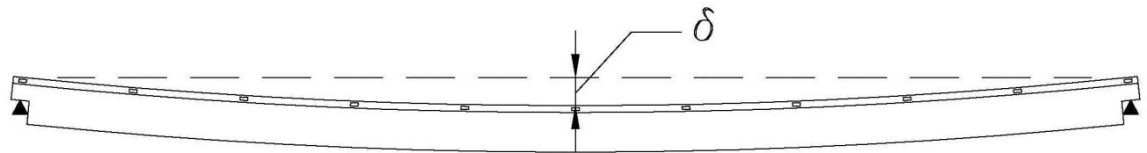

Figure 19 *Exaggerated beam deflection.*

From the deflected shape, note that the end connections have no deflection and thus do not share the vertical load. The load is resisted by the remaining nine connections, with the largest load resisted by the center connection. This connection will receive the highest stress range and will therefore be the subject of this design exercise. Failure of this connection will precipitate failure of the adjacent connections through stress redistribution; therefore its design will dictate the fatigue resiliency of the entire joint.

The load transferred across an individual connection is:

$$P_{(connection)} = (P_{(axel)} \times \mu)/2 \qquad (4)$$

where μ is a distribution factor based upon relative deflection of each connection. The distribution factor μ is calculated proportional to the sum of the deflection resisted by all of the connections. The deflection δ at each connection is calculated as follows:

$$\delta \; = \qquad\qquad\qquad\qquad\qquad (5)$$

where x is the distance from the support and l is the distance between supports.

The distribution factor μ for each connection is proportional to the sum of the deflections of each connection; therefore:

$$\mu \; = \; = \; = \qquad\qquad\qquad\qquad (6)$$

As the distribution factor μ is calculated as the relative deflection of each connection, it is not dependent on the stiffness of the beam, and therefore the modulus of elasticity E and the moment of inertia I cancel out of the equation.

We now calculate the distribution factor for the center connection, where deflection is greatest:

$$\mu \; = \; = \; = 0.161$$

The center connection, therefore, carries half of 16.1% of the cyclical load. Thus given an axle load of 2,125 lbs and a distribution factor of 0.161 we calculate the cyclical load V that is transmitted across the center connection:

$$V_{(connection)} \; = \; (P_{(axel)} \; x \; \mu)/2 \; ; \qquad\qquad\qquad (7)$$
$$V_{(connection)} \; = \; (2{,}125 \text{ lbs} \; x \; 0.161)/2$$
$$V_{(connection)} \; = \; \textbf{171 lbs}$$

While this load may seem small to those unfamiliar with fatigue design, it must be emphasized that maximum stress is not important for design purposes. Given millions of cycles, stress range and notch severity are the dominant stress variables in design of fatigue-resistant welded details. Also, the poor geometry of the connection has a stress intensification effect, creating significant stress at the root of the weld. This stress is considered next.

As load is applied to one side of the connection, half of the load is transmitted across the connection to the adjacent flange. As shown in Figure 20, both shear and bending act simultaneously on each weld. However, in determining the allowable load on the connection, the shear force shall be considered incidental for the time being. This assumption will be revisited later for confirmation.
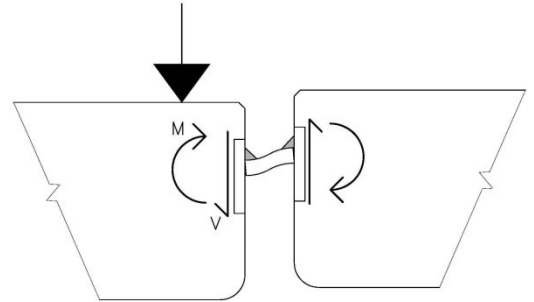

Figure 20 Shear and bending forces on welded connection.

To determine the moment, we note that the erection bar is placed in reverse curvature. As such, moment = 0 at the point of inflection, which occurs at the center of the connection (Figure 21).
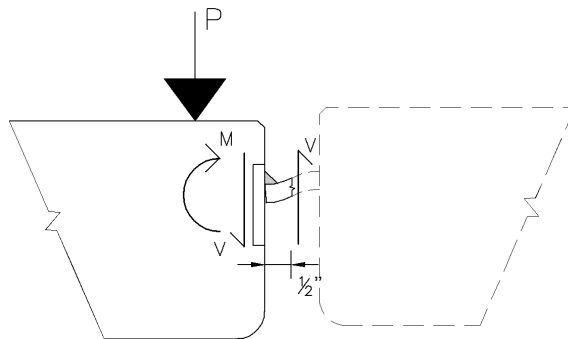

Moment at the weld is calculated as follows:

$$M = V x \text{ distance to inflection point;} \qquad (8)$$

therefore,

$$M = V (½”) \text{ for a 1” joint} \qquad (9)$$

Figure 21 Moment at the weld.

To determine the stress range in the weld, we consider loading conditions before and after load reversal. The first loading condition considered will place the root of the weld in tension; The load will then be reversed such that the root of the weld will be placed in compression (Figure 22).
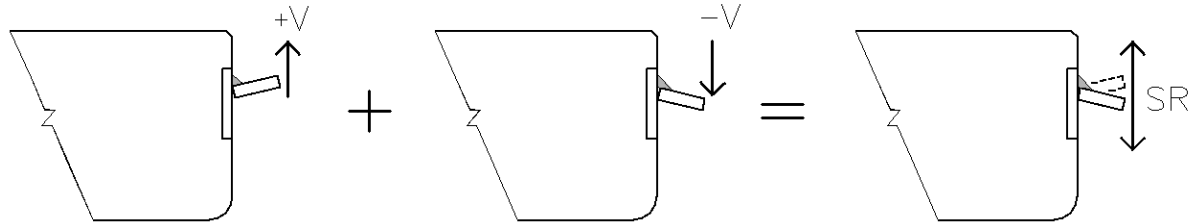
Figure 22 Stress Range pictorial.

As depicted in Figure 23, the external force V is resisted by the couple of compression and tension stresses within the weld (shown graphically as resultant forces C and T).

Stress within the weld will be greatest at the minimum cross-sectional area, which in this case is at the throat of the weld. Since fatigue occurs below the elastic limit, the stress within the weld is assumed to have a linear distribution (Figure 24).
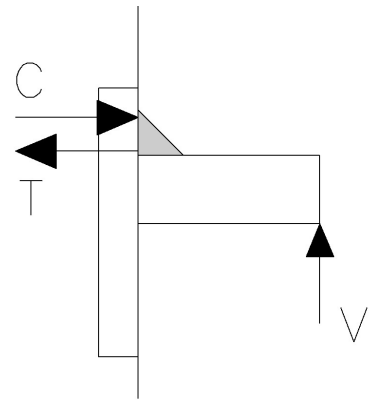

Figure 23 Resultant forces acting on the weld.

Stress in the outer fibers of the weld is therefore determined by:

$$ f \quad = \quad M \ / \ S_x $$
(10)

As depicted in Figure 25, stress at the root of the weld during the compression cycle will greatly depend on whether the bottom of the erection bar is touching the embedded plate or not.
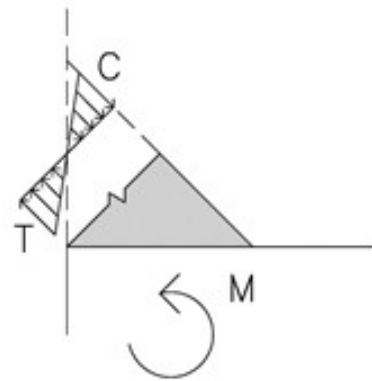

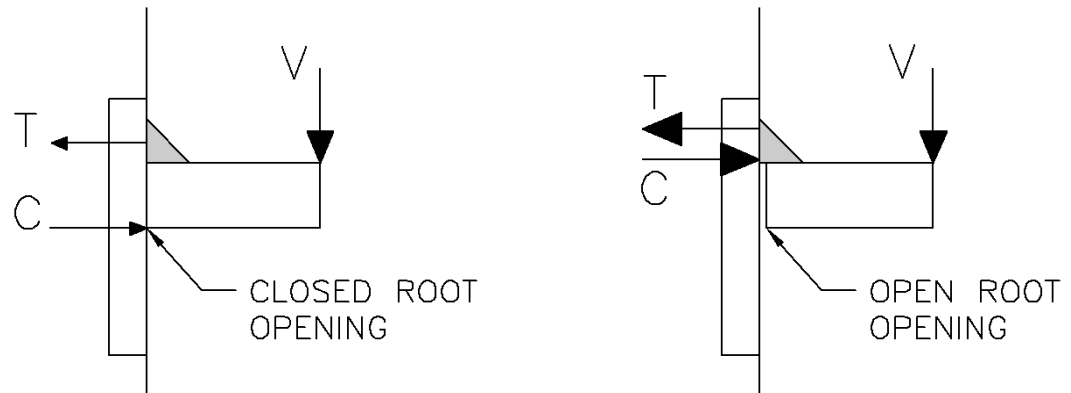**Figure 24** Internal stress at throat of weld.

Figure *25* Comparison of forces for open versus closed root openings.

If there is a gap, or *root opening,* then the bar is able to rotate, allowing bending forces to develop within the weld, equal but opposite those shown in Figure 20, above. However, if there is no root opening, then the erection bar will bear against embedded plate and the distance between the internal compression force C and tension force T increases, decreasing the internal stress (Figure 25).

A judgment must therefore be made as to whether it is likely that such a root opening will be closed while the connection is in service. 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 determine what the root opening width would need to be to remain open during fatigue loading and then to consider whether it is likely for such an opening to exist.

To determine this, we assume the weld to form a hinge under loading with rotation occurring through the centroid (Figure 26). The amount of rotation necessary to close the root opening R, therefore, is related to the change in length of the weld face W under stress.
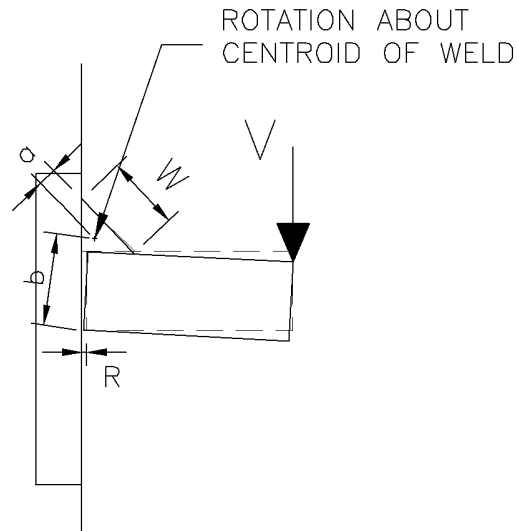
ROTATION ABOUT
CENTROID OF WELD

Figure 26 The weld forms a hinge under loading, with rotation about the centroid.

For a $^3/_8$" erection bar and ¼" weld, using simple geometry $b = 5a$; therefore:

$$R = 5(\varDelta_w) \qquad (11)$$

where $\varDelta_w$ is the change in length of the face of the weld W.

$$\varDelta_w \qquad (12)$$

For a ¼" weld, the face of the weld $W = 0.354$".

For stainless steel, we assume
$E = 28,000$ ksi.

The outer fiber stress is equal to one half the Allowable Stress Range (F_{SR}). As the required (F_{SR}) is not yet known, and finding its value is indeed the purpose of this exercise, we will conservatively assume 12 ksi for this calculation.

$$R \;=\; 5(\varDelta_w) \;=\; (5) \qquad\qquad\qquad\qquad (13)$$

$$= \;=\; 0.00038" \;\approx$$

The root opening, then, may be exceedingly small without closing under fatigue loading. Any potential benefit of a closed root opening must therefore be discounted and the bending moment within the connection, regardless of direction, must be assumed completely resisted by the weld alone.

The Stress Range (SR) of this connection can now be determined. As the stress range is below the elastic limit, the elastic section modulus S_x is used for the calculation of strength of the weld with respect to bending. For a ¼" fillet weld that is three inches long with moment applied about its longitudinal axis, the elastic section modulus of the weld throat is as follows:

$$S_x \; = \quad = \quad = 0.0156 \text{ in}^3 \qquad\qquad (14)$$

The stress in the weld due to bending (f_m) is therefore:

$$f_m \; = \quad M_F / S_x = 0.171(\tfrac{1}{2}) / 0.0156 = 5.48 \text{ ksi} \qquad\qquad (15)$$

Noting that the Stress Range for the stress reversal is additive:

$$SR_m \; = \; + \; - \quad = 5.48 + 5.48 \; = \; 10.96 \text{ ksi} \qquad\qquad (16)$$

However, as noted above, both bending and shear forces act simultaneously on the weld, so the combination of these forces must be considered. We note that the maximum fatigue stress in the weld due to bending is one half of the Stress Range, or 5.48 ksi. As this stress is reached, a vertical shear of 171 lbs is simultaneously applied over the cross-sectional area of the weld throat. The maximum shear stress is calculated thus:

$$f_s \; = \quad = \quad = 0.48 \text{ ksi} \qquad\qquad (17)$$

The stresses are then added vectorially:

$$f_r \; = \quad = 5.50 \text{ ksi} \qquad\qquad (18)$$

The Stress Range (SR) is therefore:

$$SR \; = \; f_r \; + \quad -f_r \quad = 5.50 + 5.50 \; = \; 11.00 \text{ ksi} \qquad\qquad (19)$$

It is worth noting that shear stress in the weld is insignificant in comparison to that due to bending, adding only 0.4% to the Stress Range value.

Having determined the Stress Range (SR) for this connection, we now look to Code requirements to determine the significance.

Per AWS D1.1, Section 2.16, the value of the Allowable Stress Range (F_{SR}) is determined by the number of cycles and the Stress Category. The Stress Category for a conforming connection would be determined by comparing the connection configuration to the pictorial representations and narrative that are provided in Table 2.5, "Fatigue Stress Design Parameters."[30] AWS D1.1 Figure 2.11 "Allowable Stress Range for Cyclically Applied Load (Fatigue) in Nontubular Connections" illustrates the relationship between these three factors (Figure 27). With 21.9 million cycles and an Allowable Stress Range (FSR) equal to 11.0 ksi, we find the Stress Category required to be B'.
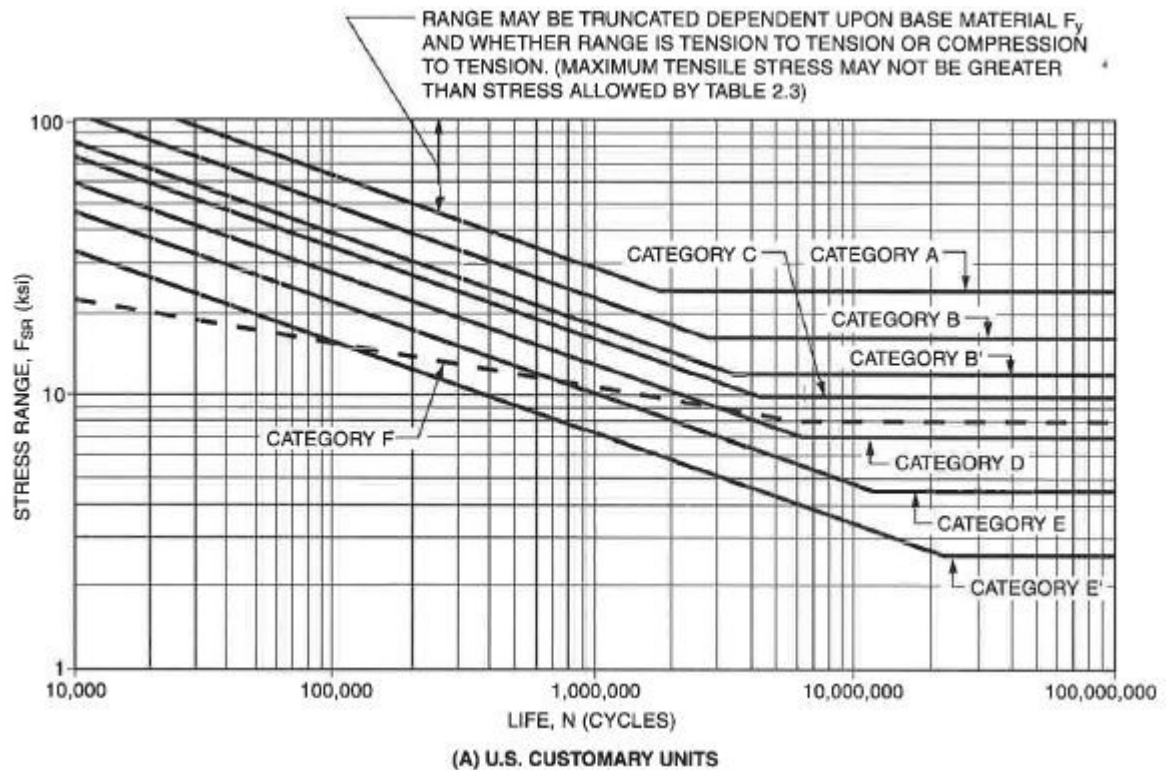

Figure *27* AWS D1.1 Figure 2.11 Allowable Stress Range for Cyclically Applied Load (Fatigue) in Nontubular Connections.[31]

Notably, Stress Category B' requires a highly fatigue-resistant weld configuration. Very few of the weld configurations recognized by either AWS or AISC meet this standard. For comparison, a fillet weld loaded in direct shear across the weld throat provides only a Stress Category of F, which would offer an Allowable Stress Range (FSR) of only 8 ksi, far below that required for this connection. In summary, even if the connection configuration common to precast double-tees were allowed, the stress range is far greater than that which could be provided under any of the Stress Category that may conceivably apply.

ARGUMENTS AGAINST THE ABOVE ANALYSIS

The analysis provided in this paper challenges current industry practice with regard to design of these connections. As the common detailing of this connection is used in virtually every pre-topped precast parking garage in America this analysis is potentially disruptive to standard practices and has met with great resistance within the precast industry. Early drafts of this paper submitted to the precast concrete industry for peer review prompted a number of critical comments. In the interest of furthering discussion and providing a thorough analysis of the concepts presented herein, the six major arguments against the conclusions of this paper are considered as follows:

Argument No. 1: "The connection model assumed in this analysis is an oversimplification that does not accurately represent flange-to-flange connections."

> *Discussion:* An exhaustive search of standards available within the precast industry reveals a single reference for the model of these connections. As noted above, the PCI Connections Manual for Precast and Prestressed Concrete Construction provides Example 3.12 which demonstrates the design of a dowel type flange connection (Figure 28). [32] It is notable that this model assumes the welds of the connection to be in shear; the eccentricity of the joint is not considered.
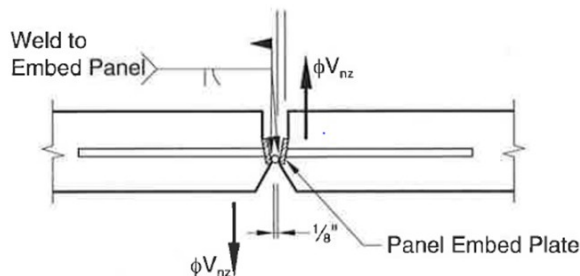
Shear Capacity:

$$\phi V_{nz} = \phi(0.6F_{exx})t_w \ell_w$$
$$= 0.75\left[0.6(70)\right](0.075)(4)$$
$$= 9.5 \text{ kip}$$

Figure *28* Excerpts from Example 3.12, Double-Tee Flange to Flange. Precast/Prestressed Concrete Institute, Sennour, Larbi et all, PCI Connections Manual for precast and prestressed concrete construction, 1st ed., 2008.

> Being the only published analysis model, the methodology advocated in this example arguably represents the most recent state of development of flange connection design within the precast industry.

> In contrast to the model cited above, the model proposed within this paper assumes basic engineering principles of *statics* apply. Specifically:

> > *Force x Distance = Moment*   (20)

> As load is applied static analysis requires that moment be developed in the welds due

to the width of the joint.

The disparity between the industry model and the model proposed within this paper are graphically depicted in Figure 29.
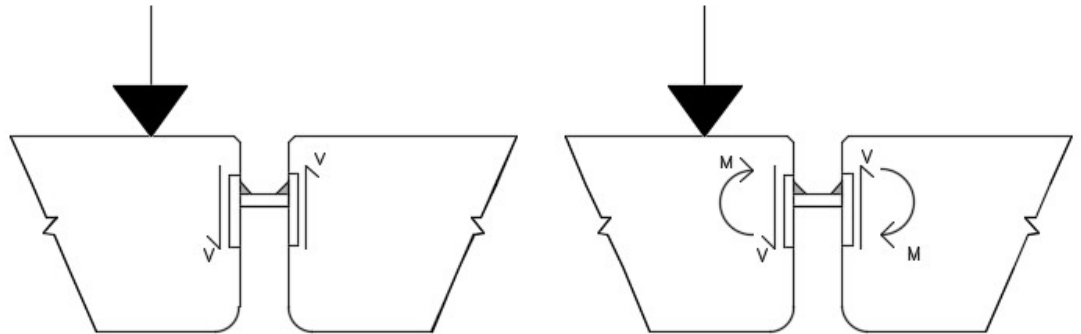

Figure *29* Model assumed by precast industry vs. model assumed within this paper.

It is apparent from Figure 29 that the precast industry is not aware that the model that they have used as basis for design to date has itself been an erroneous oversimplification of the flange-to-flange connections and grossly overestimates the strength of the connection. It is the intent of this paper that such obvious oversights be corrected.

Argument No. 2: "This analysis is not supported by test. The industry has full scale tests that demonstrate that this is not a vulnerability."

Discussion: This question provides insight to the misunderstanding within the precast industry with regard to the design of steel components and of these welded connections in particular. While testing of anchors embedded within concrete is commonplace for determining the anchor design strength, testing of welds has no bearing on the design strength of the welds.

Per the International Building Code, the design of welded connections is performed in accordance with AISC 360. Per AISC 360 the design strength of welds are determined through the application of Code provisions provided in Chapter J. As Chapter J provides prescriptive requirements that are not influenced by load testing, there is no amount of testing that can be performed to increase the design strength of

welds above the design values allowed by this Code. Nor is there any form of test that would allow prohibited weld configurations to be used.

This notwithstanding, exploration of this contention reveals no evidence within the available public record that these connections have been tested for vehicular fatigue loading. Further, there is no evidence within the industry that vehicular fatigue loading of these connections has been considered at all.

Argument No. 3: "This analysis assumes the connections deflect equally along the joints and do not account for the differential deflection along the joints"

Discussion: Procedures are provided within this paper for determining the load shared by each connection based upon the differential deflection of each connection along the joint. See, for example, page 17.

It is worth noting that there is no evidence within the available public record that such an analysis has previously been performed or that vehicular loading of individual connections has been reasonably considered within the precast industry. As noted in the review of Code Requirements and Industry Standards above, the only standard that provides reference to such loading is the *PCI Design Handbook,* which incorrectly assumes 50% of the load is transferred across a single connection for the purposes of designing the reinforced concrete flange of a double-tee.

Argument No. 4: "This analysis does not account for the flexibility of the connections. The connections are the intentionally flexibility so the welds are not in a fixed-fixed condition."

Discussion: This theory assumes that there is sufficient flexibility within the embedded anchor plate to allow full release of moment developed within the weld due to the eccentricity of the joint. This would require that the embedded plates rotate freely as if they were pinned. Since the embedments are not hinges, rotation of the plates could only be created by moment applied through the welds. In this respect the embedments would act like torsional springs wherein rotational deflection would be proportional to the moment applied.

To illustrate this concept we observe that load is transferred from one double-tee flange across the connection to the adjacent double-tee flange in the form of shear. In accordance with the engineering principles of statics shear cannot be transferred across any distance without developing moment.

For the moment to be fully released the embedded anchors must rotate freely. This would require that the connection function as a perfect pinned/pinned condition (Figure 30). However it is apparent from this figure that release of moment is accompanied by simultanious release of shear. We therefore conclude that force cannot be transferred across this joint width in this configuration as the connection would deflect freely as if the connection were not welded at all.
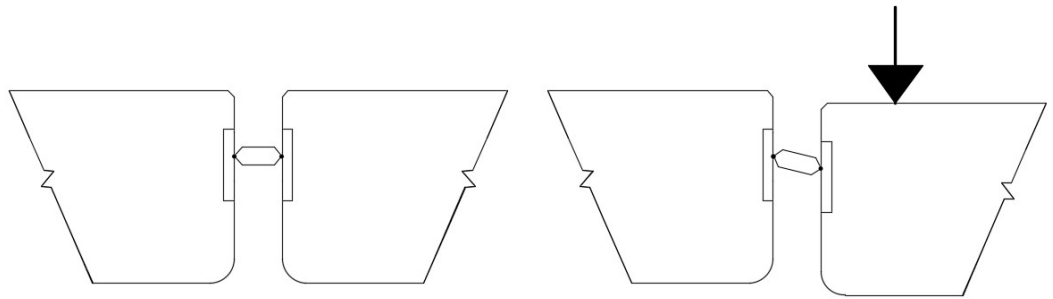

Figure *30* Connection in pinned/pinned condition.

To further illustrate this condition we observe the proposed pinned/pinned connection under loading graphically (Figure 31) and observed that such rotation of the anchorage could not occur without gross deformation of the anchor embedment and without conflict between the embedment and the concrete.
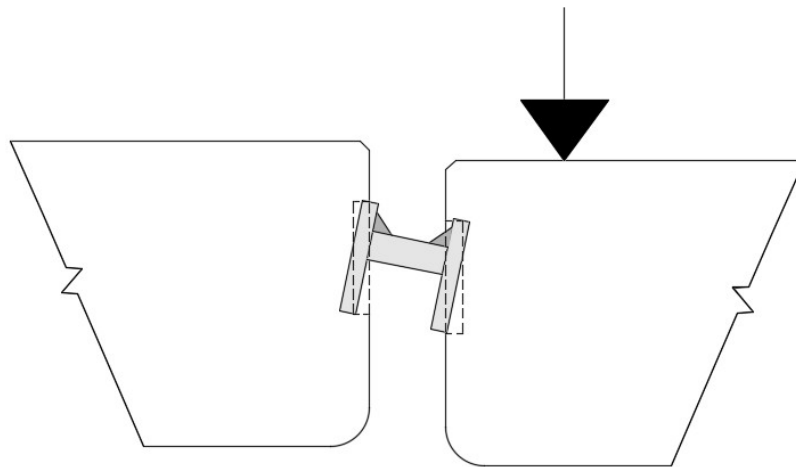

Figure *31* Connection in pinned/pinned condition under loading.

It may be argued that the deflected condition depicted in Figure 31 is at least in part accurate, and rotation of the embedded brackets releases a significant amount of the moment within the welds. To assess this possibility we first observe that the

differential deflection across a joint where the welds have fractured is at least ¼"
under vehicular loading. We then consider a joint where the connections are welded
and calculate (in similar fashion to that performed on page 20) the differential
deflection across the joint due to bending of connection under vehicular loading to be
approximately 1/1000". We then observe that the differential deflection of the joint
required to release moment in the weld by rotation of the anchor plates is
approximately 250 times more than the deflection of the joint observed under
vehicular loading and conclude that joint flexibility has no significant effect on the
moment applied to the welds.

*Argument No. 5: "This analysis assumes that double tees share load across the joint equally.
The load transferred across the joint may be much less than the 50% assumed."*

Discussion: This argument presumes that current connections are conservatively
designed based upon a loading assumption that is at odds with current precast
industry design standards. As noted previously, the only industry standard that
references the calculation of loading transferred across a connection is the *PCI
Design Handbook,* which assumes 50% of the load is transferred across a single
connection for the purposes of designing the reinforced concrete flange of a double-
tee. This argument, which states that less than 50% of the load is transferred across
the joint, would therefore require that the methodology for designing precast concrete
double-tee flanges that has been advocated by the precast industry for many years has
been incorrect and un-conservative.

To assess the load distribution across the joint we consider the following identical
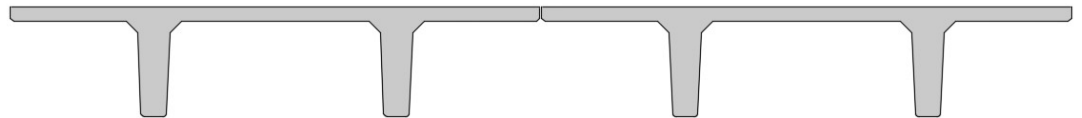beams that are not connected (Figure 32).

Figure *32* Double-tee beams without loading. Beams not connected.

As load is applied to the edge of the beam on the right the flange deflects downward
due to bending and torsion within the beam (Figure 33). The total deflection is
proportional to the quantity of load applied and the stiffness of the beam.
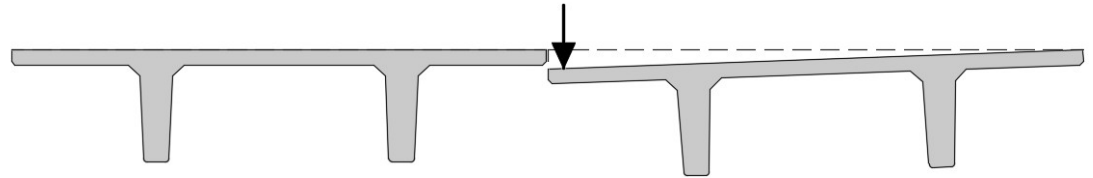
Figure *33* Double-tee beam with applied load. Beams not connected.

Now we consider this loading with the beams connected to each other. Since the beams are identical and are connected they deflect in an equal but opposite manner (Figure 34).
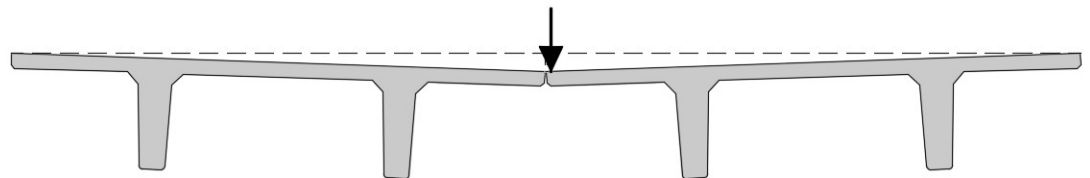

Figure *34* Double-tee beam with applied load. Beams connected.

In considering the deflected shapes of Figure 34 it is apparent that for each identical beam to deflect equally they must share the same amount of load. This requires that 50% of the load be transferred across the connections. The analysis provided in this paper is therefore in agreement with the industry standard suggested by the design example in the *PCI Design Handbook*, the only difference being that this paper assumes the load to be shared by all connections along the joint, not just one.

Argument No. 6: "These connections have been used for years and perform well. There is no basis to assume that they are defective in any way."

Discussion: This argument presumes that 1). These connection do not fail due to fatigue loading, and 2). Code requirements are not applicable based upon the perceived performance observed by the precast industry.

A small body of evidence, consisting of photos and scanning electron micrographs, has been provided within this paper to establish the basis for the claim that fractures occur due to fatigue loading. While it is the observation of the author that such fractures are commonplace it is not the purpose of this paper to provide exhaustive evidence but rather to reveal the cause of such fractures such that more resilient and safer structures can be constructed.

Further, adherence to Code design requirements is not predicated upon perceived

performance with regard to nonconforming construction in the field. These design requirements are absolute and cannot be mollified by claims to the contrary. In short, even if no fractures had ever occurred, prescriptive Code requirements for the design of these connections would remain in effect.

It is worth noting that the arguments against the analysis presented in this paper assume that it is incumbent upon the critic of the existing connection to prove that it is not adequate. This is antithetical to engineering practice wherein it is incumbent upon the designer to prove that the connection is in fact satisfactory for the applied loads and conditions. It is not prudent to assume that the connections behave in a suitable yet unverified manner or that loads are likely less than that calculated by rational analysis. Such claims must be proven by the design professional.

In addition, there is no indication that such arguments form the basis for the previous consideration and acceptance of the current connection design. There is no evidence to suggest that design of this connection for either static or fatigue loading from vehicles has been appropriately considered. The only published industry standards available for the design of this connection assume a loading distribution specifically at odds with the objections raised in the dissenting arguments presented above, and they provide but one single design example, which neglects basic considerations of statics. Moreover, current standards and practices disregard fatigue design requirements of the Building Code entirely.

RECOMMENDED AREAS OF STUDY

The analysis performed above was based upon application of rational structural analysis. Through this process, it is apparent that little empirical information exists specific to precast double-tee construction to guide such an analysis. The following areas would benefit from further study:

Loading information used herein assumes the static weight of an average vehicle. However, no information exists to suggest what impact load, if any, should be applied. The actual load imparted on the beam depends on the difference in elevation of the beam surfaces, the weight of the vehicle, its velocity, suspension type, and the pressure of air in the tires. As impact load has not been included in the above calculations, analysis results may be insufficiently conservative.

Further, applied load has been distributed to each connection based upon simplified rational analysis. Secondary considerations, such as localized deflection of the beam flanges, would act to increase the load across a single connection. As this variability in deflection was also not considered in the above analysis, results may not be conservative.

DISCUSSION

The use of flat bar type double-tee connections as discussed herein has considerable momentum within the precast industry. This type of connection is used in virtually all pre-topped, precast concrete garages currently manufactured. However, it is also clear that the design is at odds with Building Codes, industry standards, and sound engineering practice.

When a connection fails, load is redistributed to adjacent connections, which also subsequently fail. Failure of one connection will effectively cause the joint to "unzip" as one connection after another breaks. The most immediate and tangible result of this failure is that the sealant joint, stressed beyond its limit, begins to leak. The joint flanges will also noticeably deflect as a car traverses them, creating a curious if un-alarming bounce in the beam and vibration in the deck. This is a common occurrence within the precast parking garage industry and is considered little more than a maintenance problem and nuisance. It is important, then, to place the failure of double-tee connections within context of their function.

These are structural connections. It is difficult to imagine a connection used in any other building type or situation that would be allowed to fail with the regularity of precast flange connections. Further, these are *seismic* structural connections, which form an integral part of the buildings lateral force-resisting system. It cannot be overstated how serious the loss of connections along a single joint would be during a seismic event. The loss of one joint would render the entire diaphragm of which it is a part inoperable. As precast parking garages are not typically designed with redundant diaphragms, the loss connections along one joint could render the entire lateral force resisting system ineffective.

It is evident that re-development of the connection is required. The factors that have the greatest effect on the fatigue strength of the connection include the geometry of the connection, the weld size, and the effective joint width. However, there are tradeoffs that must be considered when varying these parameters.

Given the concerns raised above regarding the inadvisability of welded flat bar type connections, the configuration of the connection would appear to be of primary concern. Any design improvement should satisfy Building Code requirements. A welded connection must be configured to avoid concentration of tensile stress at the root of any weld.

The bending and fatigue resistance of the weld may be augmented by increasing the weld size. However, such a modification would not, by itself, satisfy Code requirements. Furthermore, additional welding creates additional heat, and it is well established[33] that the heat of welding is detrimental to the concrete in which the connections are embedded. Stainless steel has a high coefficient of thermal expansion, expanding 50% more than mild steel when exposed to heat. As welds are performed, the portion of the stainless steel connection embedded within the concrete expands, creating cracks and spalls (Figures 35 and 36). These cracks allow moisture intrusion to the portions of the connection embedded within the concrete, corroding non-stainless components and causing leaks at the precast

joints.

Figure 35 Spall due to welding at stainless steel embedment.

Figure 36 Cracking at concrete and corrosion of rebar due to large weld at stainless steel embedment.

Joint width may be varied, within limits. A smaller joint width between embedded plates will impart less moment on the connection, although such a modification would not, by itself, satisfy Code requirements. However, there are practical considerations. The smaller the joint, the more difficult it is to obtain the proper angle with a welding rod to properly perform the weld (Figure 37). As the joint width becomes smaller the welding rod must be held at a steeper angle, making it difficult to deposit weld metal on the vertical leg of the fillet. Importantly, as the joint approaches ½ inch, the toe of ¼ inch fillet welds touch, obscuring the reference for determining the height of the vertical leg and making the connection impossible to inspect.
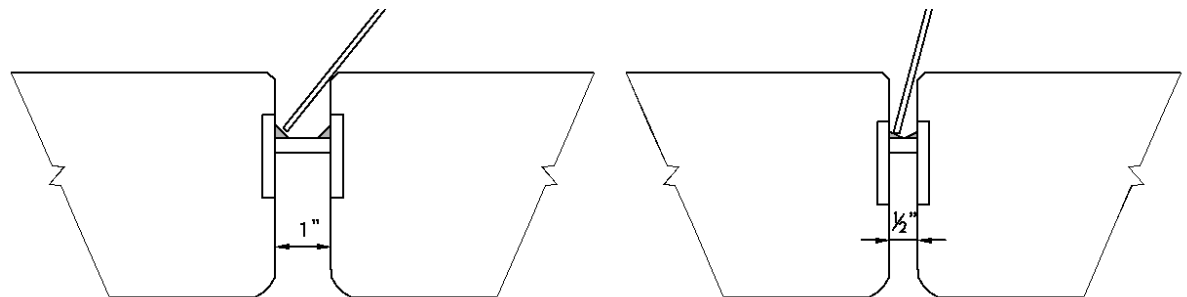

Figure 37 Weld angle for varying joint widths.

For tight joints widths it is possible to reduce the size of the weld; however, AWS D1.1 Section 5.14 and Table 5.8 specifically limit the minimum weld size that may be performed based upon the base material thickness.[34] In addition, per AWS D1.1, Section 2.18, fillet welds less than $^3/_{16}$ inch are prohibited for cyclically loaded connections.[35]

For dowel-type connections, a smaller joint size requires a smaller erection dowel, and the

effective weld size of a flare bevel groove weld is related to the dowel radius. Since the elastic section modulus is based upon the square of the throat depth, the strength of the weld is decreased exponentially as the joint width is decreased.

In summary, precast double-tee flange-to-flange connection details currently employed within the precast concrete industry are poorly configured to withstand cyclical vehicular loading and do not satisfy Code design requirements in this regard. Further development of these connections is warranted to increase the safety, to extend the serviceability, and to reduce maintenance costs of precast parking garages.

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33 **Gleich, Harry A. et al**, "Joints in Precast Parking Structures," PCI Committee Report, *PCI Journal*, September-October 2007, p. 130.

34 **Miller, D. K. et al**, "Structural Welding Code—Steel," *AWS D1.1/D1.1M:2010*, American Welding Society, October 2011, pp. 198, 209.

35 *Ibid*, p. 15.

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