AASHTO
Craig Thompson, *President*
Secretary, Wisconsin Department of Transportation
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Errata for *Guide for Design and Construction of Near-Surface Mounted Titanium Alloy Bars for Strengthening Concrete Structures*, First Edition (NSMT-1)

March 2024

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to *Guide for Design and Construction of Near-Surface Mounted Titanium Alloy Bars for Strengthening Concrete Structures*, First Edition. Please scroll down to see the full erratum.

In the event that you need to download this file again, please download from AASHTO's online bookstore at: https://downloads.transportation.org/NSMT-1-Errata.pdf.

The new changes in this erratum are detailed in the table under the "March 2024" heading. Pages with the new changes have a gray box in the page header reading as follows:

March 2024 Errata

AASHTO staff sincerely apologizes for any inconvenience to our readers. Please feel free to contact us if you have questions or need any additional information.

Sincerely,
AASHTO Publications

Summary of Errata Changes for NSMT-1, March 2024

Page	Existing Text/Notes	Corrected Text
Chapter 7		
8	Yield strains for TiABs are approximately 8,400 με for Class 130 material with insensitive bonding materials and 6,580 με for Class 120 material with sensitive bonding materials.	Yield strains for TiABs are approximately 8,400 με for Class 130 material with insensitive bonding materials and 7,700 με for Class 120 material with sensitive bonding materials.
Chapter 9		
13	$\varepsilon_s = \dfrac{\dfrac{\lvert M_u \rvert}{d_v} + \lvert V_u - V_p \rvert - A_{ps} f_{po}}{E_s A_s + E_p A_{ps} + \alpha_E E_{Ti} A_{Ti}}$ (9.4-5)	$\varepsilon_s = \dfrac{\dfrac{\lvert M_u \rvert}{d_v} + \lvert V_u - V_p \rvert - A_{ps} f_{po}}{E_s A_s + E_p A_{ps} + E_{Ti} A_{Ti}}$ (9.4-5)
14	if $V_u < 0.125\sqrt{f'_c} b_v d_v$, then $s_{\max} \leq 0.4 d_v \leq 12.0$ in. (9.6-2)	if $V_u \geq 0.125\sqrt{f'_c} b_v d_v$, then $s_{\max} \leq 0.4 d_v \leq 12.0$ in (9.6-2)

Page	30
Chapter 11	
Existing Text/ Notes	$\varepsilon_s = \dfrac{\dfrac{\lvert M_u \rvert}{d_v} + \lvert V_u \rvert}{E_s A_s + \alpha_E E_{Ti} A_{Ti}} = \dfrac{\dfrac{\lvert 507.2 \text{ kip ft} \rvert (12 \text{ in./ft})}{33.3 \text{ in.}} + \lvert 112.5 \text{ kips} \rvert}{29{,}000 \text{ ksi}(4.68 \text{ in.}^2) + 0} = 0.002175 \text{ in./in}$ (11.3.1-2a)
Corrected Text	$\varepsilon_s = \dfrac{\dfrac{\lvert M_u \rvert}{d_v} + \lvert V_u \rvert}{E_s A_s + E_{Ti} A_{Ti}} = \dfrac{\dfrac{\lvert 507.2 \text{ kip-ft} \rvert (12 \text{ in./ft})}{33.3 \text{ in.}} + \lvert 112.5 \text{ kips} \rvert}{29{,}000 \text{ ksi}(4.68 \text{ in.}^2) + 0} = 0.002175 \text{ in./in}$ (11.3.1-2a)
Existing Text/ Notes	$117.4 \text{ kips} \geq 0.125\sqrt{3.3 \text{ ksi}}\,(13 \text{ in.})33.3 \text{ in.} = 9.83 \text{ kips}$ (11.3.1-5)
Corrected Text	$117.4 \text{ kips} \geq 0.125\sqrt{3.3 \text{ ksi}}\,(13 \text{ in.})33.3 \text{ in.} = 98.3 \text{ kips}$ (11.3.1-5)
Page	34
Existing Text/ Notes	$T_{\text{demand}} = \dfrac{\lvert M_{ui} \rvert}{\phi_b d_v} + \left[\dfrac{V_{ui}}{\phi_v} - \dfrac{V_s}{2} - \dfrac{V_{Ti}}{2} - \left(\dfrac{\gamma_{DC} w_{DC}}{2} + \dfrac{\gamma_{DW} w_{DW}}{2} + \dfrac{\gamma_{LL} w_{\text{lane}} DF_M}{2} \right) d_v \cot\theta \right] \cot\theta$ $= \dfrac{359.1 \text{ kip-ft}(12 \text{ in./ft})}{(0.9)33.31 \text{ in.}} +$ $\left[\dfrac{98.3 \text{ kips}}{0.9} - \dfrac{63.7 \text{ kips}}{2} - \dfrac{27.9 \text{ kips}}{2} - \left(\dfrac{1.25(1\ 18 \text{ kip/ft})}{2} + \dfrac{1.5(0.33 \text{ kip/ft})}{2} + \dfrac{1.75(0.64 \text{ kip/ft})0.78}{2} \right) \dfrac{33.31 \text{ in.}}{12 \text{ in./ft}} \cot 34.87 \right] \cot 34.87$ $= 226.6 \text{ kips}$ (11.3.3-3)

Corrected Text	$$T_{\text{demand}} = \frac{\lvert M_{ui} \rvert}{\phi_b d_v} + \left[\frac{V_{ui}}{\phi_v} - \frac{V_s}{2} - \frac{V_{Ti}}{2} - \left(\frac{\gamma_{DC} w_{DC}}{2} + \frac{\gamma_{DW} w_{DW}}{2} + \frac{\gamma_{LL} w_{\text{lane}} DF_M}{2}\right) d_v \cot\theta\right]\cot\theta$$ $$= \frac{359.1\text{ kip-ft }(12\text{ in./ft})}{(0.9)33.31\text{ in.}} +$$ $$\left[\frac{98.3\text{ kips}}{0.9} - \frac{63.7\text{ kips}}{2} - \frac{27.9\text{ kips}}{2} - \left(\frac{1.25(1.18\text{ kip/ft})}{2} + \frac{1.5(0.33\text{ kip/ft})}{2} + \frac{1.75(0.64\text{ kip/ft})0.78}{2}\right)\frac{33.31\text{ in.}}{12\text{ in./ft}}\cot 34.87\right]\cot 34.87$$ $$= 226.6\text{ kips}$$ (11.3.3-3)

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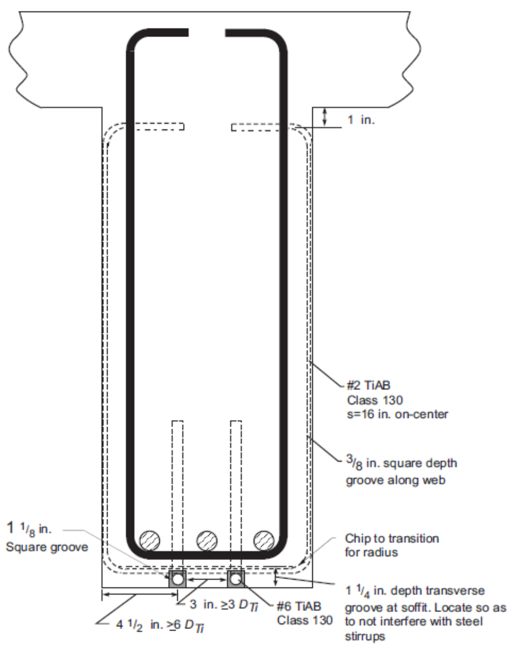
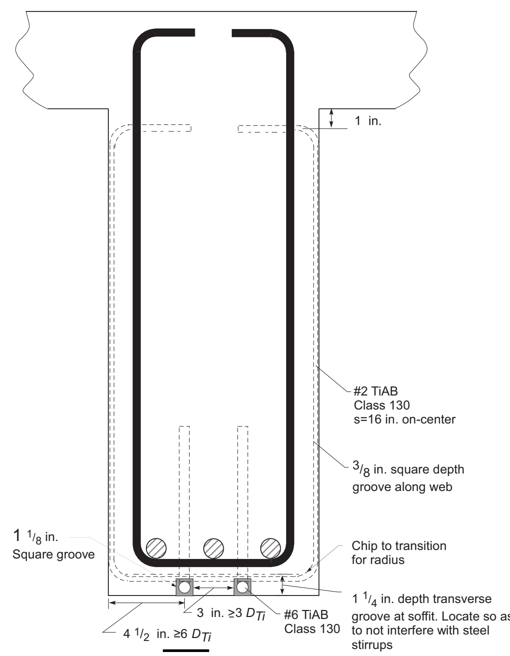
36		

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the fatigue truck including 15 percent impact. The existing structure should be able to meet the following condition:

$$\phi R_n \geq 1.05DC + 1.1DW + 0.75LL \qquad (7.2\text{-}1)$$

where ϕR_n is the design strength of the existing structure without TiABs and *DC, DW,* and *LL* are the load effects from weight of components, weight of wearing surface, and live load (with 15 percent impact), respectively.

If future loads are not to be increased and when calculations show the existing structure does not satisfy Equation 7.2-1, demonstrated in-service performance can be used as evidence of sufficient underlying operational strength.

TiABs have been shown in the laboratory not only to fully restore the strength of a failed structural girder, but to increase the strength above the expected unstrengthened capacity (Higgins, Amneus, and Barker, 2015a). Thus, it is possible to design TiABs to strengthen structures that do not meet the conditions of Equation 7.2-1. For such conditions, additional care shall be exercised by the designer, and inspection procedures and intervals shall be established to ensure long-term performance.

7.3 FIRE ENDURANCE

No specific fire endurance limits are proposed at present. The use of hooked ends that are anchored in the core concrete provide some additional robustness for TiAB-strengthened systems compared to other surface-only bonded materials.

If TiABs are used in cases that do not satisfy Equation 7.2-1, fire endurance should be considered by the designer.

7.4 PREFERRED FAILURE MODES

Application of supplemental strengthening materials can shift or alter failure modes. It can also result in failures at unstrengthened locations in the member. Ductile failure modes are preferred. When possible, failures should be controlled by ductile flexural modes rather than diagonal tension (shear).

7.5 ENVIRONMENTAL DURABILITY

TiABs are not sensitive to degradation from environmental exposure. However, the materials used to bond TiABs to the concrete substrate can be sensitive to such exposure. Some bonding materials are less sensitive than others to different environmental conditions. No bonding materials should be used that exhibit marked degradation in bond performance after exposure testing.

To account for reduced bond strength along the NSM length due to environmental exposure, exposure factors, α_E, are applied that depend on the environmental sensitivity of the bonding material. The factors are shown in Table 7.5-1. The slightly larger exposure factors compared to FRP systems are due to the beneficial hooked ends that are anchored in the concrete core and are better protected from surface environmental conditions.

Table 7.5-1 Environmental Exposure Factors

Exposure Sensitivity	Environmental Exposure Factor, α_E
Routine	0.85
Insensitive	1.00

7.6 DESIGN MATERIAL PROPERTIES

The material properties to be used in design are the minimum yield stress for the prescribed class of TiAB (f^*_{yTi}; Section 3.3), modified by the environmental exposure factor (α_E; Table 7.5-1) as follows:

$$f_{y_{Ti}} = \alpha_E f^*_{y_{Ti}} \tag{7.6-1}$$

where f_{yTi} is the nominal design yield stress of TiABs modified by α_E (ksi).

In the elastic range, the material follows Hook's Law so that the elastic TiAB strain, ε_{Ti}, is described by

$$\varepsilon_{Ti} = f_{Ti}/E_{Ti} \tag{7.6-2}$$

where f_{Ti} is the stress in the TiAB, and E_{Ti} is the modulus of elasticity prescribed in Section 3.3.

Yield strains for TiABs are approximately 8,400 με for Class 130 material with insensitive bonding materials and 7,700 με for Class 120 material with sensitive bonding materials.

CHAPTER 8—FLEXURAL STRENGTHENING

Strengthening of members for flexure requires installation of TiABs in the flexural tension zone of the element considered. The TiABs can be placed in the soffit of beams or along the sides of beams and beam columns, or along both the soffit of beams and the sides of beams and beam columns. Although having a reduced moment arm, placement of TiABs along the beam web can overcome potential interference with the embedded flexural steel.

8.1 STRENGTH

For flexural design, the design strength, $\phi_b M_n$, must exceed the factored moment demands, M_u, as follows:

$$\phi_b M_n \geq M_u \tag{8.1-1}$$

If loads are to be increased on the structure after flexural strengthening, members should be checked to ensure there is adequate diagonal tension strength for the increased demands.

8.2 DESIGN ASSUMPTIONS

For strength design in flexure, the following assumptions, idealizations, and simplifications are made: the flexural tensile strength of the concrete is neglected; plane sections remain plane; the reinforcing steel and TiABs are assumed to have idealized elastic–plastic behavior; there is no relative slip between the concrete and steel or TiABs; and there is live-load strain compatibility between the steel and TiABs. Strain in the concrete at ultimate, ε_{cu}, is assumed to be 0.003. The concrete stress can be idealized as an equivalent rectangular stress block (Whitney stress block) whose height depends on the concrete compressive stress (β_1 as prescribed in *AASHTO LRFD Design,* Section 5.6.2.2). An example cross section, assumed flexural strain distribution, and stress distribution are shown in Figure 8.2-1.

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s + A_{Ti} \alpha_E f^*_{yTi} d_{Ti}}{A_{ps} f_{ps} + A_s f_y + A_{Ti} \alpha_E f^*_{yTi}} \quad (9.4\text{-}3)$$

For sections meeting minimum requirements for embedded transverse steel as defined in Section 9.7, β is computed as

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} \quad (9.4\text{-}4)$$

where ε_s is the strain in the longitudinal tension reinforcement. The strain in the longitudinal tension reinforcement for a conventionally RC member can be computed as

$$\varepsilon_s = \frac{\dfrac{|M_u|}{d_v} + |V_u - V_p| - A_{ps} f_{po}}{E_s A_s + E_p A_{ps} + E_{Ti} A_{Ti}} \quad (9.4\text{-}5)$$

where:

E_s (ksi) and A_s (in.^{2}) = modulus of elasticity and cross-sectional area of the longitudinal tension steel, respectively;

E_p (ksi) and A_{ps} (in.^{2}) = modulus of elasticity and cross-sectional area of prestressing steel on the flexural tension side of the member where applicable;

f_{po} = a parameter taken as the modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel and the surrounding concrete (ksi);

E_{Ti} (ksi) and A_{Ti} (in.^{2}) = modulus of elasticity and cross-sectional area of longitudinal TiABs if used concurrently with shear strengthening, respectively; and

M_u (kip-in.) and V_u (kips) = factored moment and shear demands in the section, respectively. The contribution of longitudinal TiABs can conservatively be neglected in Equation 9.4-5.

The shear contribution from vertical-leg steel stirrups, V_s, is taken as

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \quad (9.4\text{-}6)$$

where:

A_v = cross-sectional area (in.^{2}) of the transverse reinforcing steel,

f_y = yield stress of the transverse reinforcing steel (ksi),

s = spacing of the transverse reinforcing steel (in.), and

θ = the angle of inclination of the diagonal compressive stresses (degrees), computed as

$$\theta = 29 + 3{,}500\varepsilon_s \quad (9.4\text{-}7)$$

The TiAB contribution to shear strength, V_{Ti}, is taken as

$$V_{Ti} = \frac{A_{vTi}\,\alpha_E\, f^*_{yTi}\; d_v\; \cot\theta}{s_{Ti}} \tag{9.4-8}$$

where A_{vTi} is the cross-sectional area (in.^{2}), and s_{Ti} is the spacing (in.) of the TiABs.

9.5 SHEAR RESISTANCE FACTOR

The shear resistance factor, ϕ_v, is taken as 0.9.

9.6 MAXIMUM SPACING OF TRANSVERSE TiABS

Where required for strength, the spacing of transverse TiAB reinforcement depends on the factored shear demands in the section and available strength from the concrete and steel stirrups. Where required for strength, spacing of the transverse reinforcement, including the combined presence of reinforcing steel and TiABs, shall not exceed the maximum permissible spacing, s_{max}, which is determined as

$$\text{if } V_u < 0.125\sqrt{f'_c}b_v d_v,\ \text{then } s_{\max} \le 0.8d_v \le 24.0 \text{ in.} \tag{9.6-1}$$

$$\text{if } V_u \ge 0.125\sqrt{f'_c}b_v d_v,\ \text{then } s_{\max} \le 0.4d_v \le 12.0 \text{ in.} \tag{9.6-2}$$

If the spacing of the existing transverse reinforcing steel exceeds the maximum spacing limit, then transverse TiAB reinforcement shall be added such that the spacing between any two adjacent transverse reinforcing bars is less than s_{max} along the span. Alternatively, an effective spacing can be used to satisfy this requirement based on the relative strength and spacing of the existing steel and supplemental TiAB transverse reinforcing.

9.7 MINIMUM AMOUNT OF TRANSVERSE REINFORCEMENT

Where transverse reinforcement is required as noted in Section 9.3, all TiAB shear strengthening designs must provide total shear reinforcement (existing steel and TiABs) that satisfies the following minimum amount of transverse reinforcement:

$$\frac{A_v f_y}{b_v s} + \frac{\alpha_E f^*_{yTi} A_{Ti}}{b_v S_{Ti}} \ge 0.0316\sqrt{f'_c} \tag{9.7-1}$$

9.8 CHECK OF FLEXURAL TENSION REINFORCING IN PRESENCE OF DIAGONAL CRACK

The flexural tension reinforcing (steel and TiABs) must be able to resist the demands from flexure in the presence of shear acting on a presumed diagonally cracked section. The free-body diagram is shown in Figure 9.8-1.

11.3 SHEAR STRENGTHENING EXAMPLE

Structural analysis of the existing bridge established the factored shear demands for the Strength I load combination for the HL93 load model. The existing shear strength according to *AASHTO LRFD Design* (2017) is shown along with the factored strength demands in Figure 11.3-1. As seen in Figure 11.3-1, the existing shear capacity cannot meet the owner's objective of achieving the inventory-level load rating near the support location. To achieve the objective, the bridge must be strengthened.

To strengthen the bridge with NSM-TiABs, the structure must have adequate strength to carry the self-weight of the structure and an operational load level according to Equation 7.2-1. The existing design strength (dashed, blue line) has sufficient strength to meet the requirements of Equation 7.2-1 (bottommost, solid orange line) as seen in Figure 11.3-1.

To meet the owner's objective, TiABs will be used to increase the shear strength to achieve an inventory-level load rating.

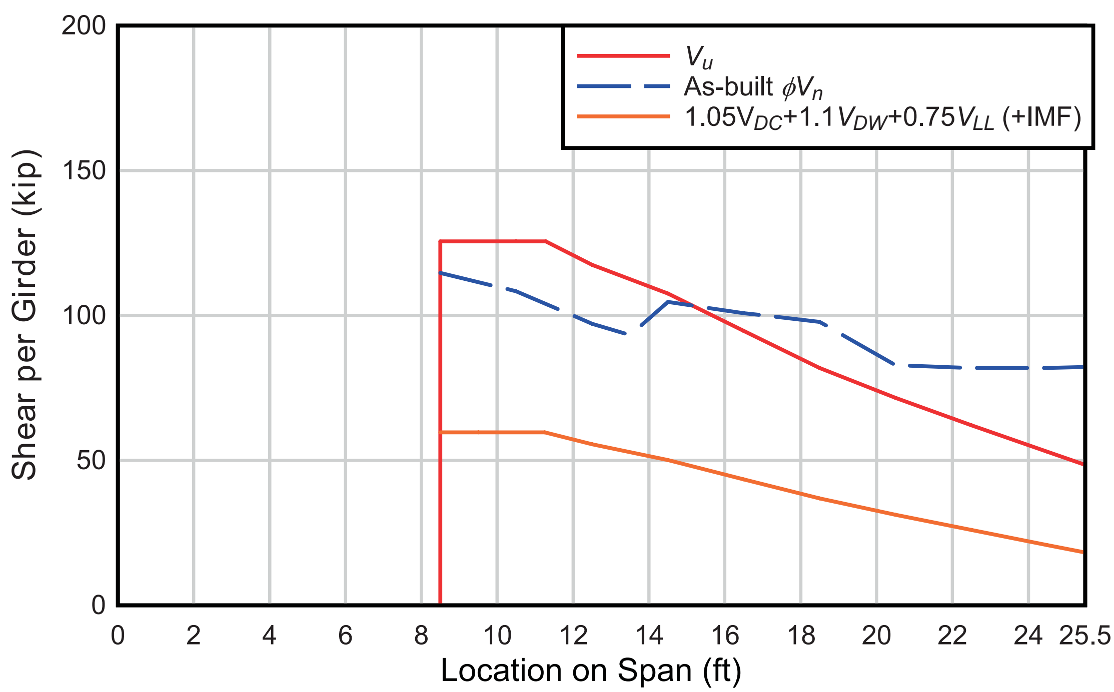

Figure 11.3-1. Factored Ultimate Shear Envelope for Strength-Level, As-Built Design Shear Strength without TiABs, and the Minimum Factored Shear Strength Required to Permit Strengthening

11.3.1 Shear Strength

The nominal shear strength of the girder is computed along the region where TiABs are to be used as transverse reinforcing to achieve the design object. The nominal shear strength of the girder is computed as the summation of the concrete, reinforcing steel, and TiAB contributions:

$$V_n = V_c + V_s + V_{Ti} \tag{11.3.1-1}$$

The concrete, steel, and TiAB contributions vary depending on the factored moment and shear demands along the span as well as the geometry and material properties. The factored moment and shear vary along the span. In this design example, the factored moment and shear are taken at their envelope maximums rather than coincident values. A sample calculation is performed for the location 12.5 ft on the span.

First, the strain in the longitudinal tension reinforcement is computed using d_v for the section of interest. For this example, the section in the high shear region near the support includes the addition of flexural reinforcing TiABs. Here the cross section has three #11 steel reinforcing bars and two #6 TiABs. The dimension d_v for this section is 33.3 in. (taken as the nominal moment capacity divided by the compression force resultant). In the strain calculation example, the contribution of the flexural TiABs in reducing the reinforcing steel flexural strain is conservatively neglected (set equal to zero). The steel flexural strain is computed as

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} + |V_u|}{E_s A_s + E_{Ti} A_{Ti}} = \frac{\frac{|507.2 \text{ kip-ft}|(12 \text{ in./ft})}{33.3 \text{ in.}} + |112.5 \text{ kips}|}{29{,}000 \text{ ksi}(4.68 \text{ in.}^2) + 0} = 0.002175 \text{ in./in} \quad (11.3.1\text{-}2a)$$

If the flexural TiAB contribution were included in Equation 11.3.1-2a, the strain in the longitudinal tension reinforcement would reduce to 0.00203 in./in.

The concrete efficiency parameter, β, and the diagonal crack angle are computed from the steel strain. The section satisfies the minimum stirrup requirements as defined in *AASHTO LRFD Design* (2017) such that β is computed as

$$\beta = \frac{4.8}{1 + 750\varepsilon_s} = \frac{4.8}{1 + 750(0.002175)} = 1.82 \quad (11.3.1\text{-}2b)$$

and θ is computed as

$$\theta = 29 + 3{,}500\varepsilon_s = 29 + 3{,}500(0.002175) = 36.6° \quad (11.3.1\text{-}2c)$$

The concrete capacity, V_c, is determined as

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d = 0.0316(1.82)\sqrt{3.3 \text{ ksi}}(13 \text{ in.})33.3 \text{ in.} = 45.2 \text{ kips} \quad (11.3.1\text{-}3)$$

The shear contribution from the #4 Intermediate Grade, double-legged vertical steel stirrups with 12-in. spacing, V_s, is computed as

$$V_s = \frac{A_v f_y d_v (\cot\theta)}{s} = \frac{2(0.2 \text{ in.}^2)(40 \text{ ksi})33.3 \text{ in.}(\cot 36.6)}{12 \text{ in.}} = 59.8 \text{ kips} \quad (11.3.1\text{-}4)$$

The TiABs are unknown at design and a trial design is required. First, the maximum spacing is determined. The factored shear demand at the section equals 117.4 kips. The demand is compared to the threshold values as follows:

$$\begin{aligned} &\text{If } V_u < 0.125\sqrt{f'_c}b_v d_v, \text{ then } s_{\max} \le 0.8 d_v \le 24 \text{ in.} \\ &117.4 \text{ kips} \ge 0.125\sqrt{3.3 \text{ ksi}}(13 \text{ in.})33.3 \text{ in.} = 98.3 \text{ kips} \\ &\text{thus } s_{\max} \le 0.4 d_v = 0.4\ (33.3 \text{ in.}) \cong 13 \text{ in.} \le 12 \text{ in.} \end{aligned} \quad (11.3.1\text{-}5)$$

The maximum spacing limit of 12 in. controls for this case. The existing transverse reinforcing steel is spaced at 12 in. in this region and thus satisfies this requirement.

This is a conservative approach and suitable for new designs in which the maximum factored moment and shear demands are combined and assumed to occur at a vertical section where T is computed. This approach has little impact on the cost and provides designs with an additional reserve capacity. However, for existing structures, this approach is excessively conservative and many girders would not be satisfactory. Existing structures tend to have less excess capacity, hence the need for strengthening in the first place. Thus, the factored moment and shear demands need to be computed based on a coherent model that properly incorporates the axle-load position and the permanent and lane loads acting on the section that includes the presumed diagonal crack. The proper free-body diagram necessary for establishing the flexural tension demand from combined bending with shear in the presence of diagonal crack is shown in Figure 11.3.3-1.

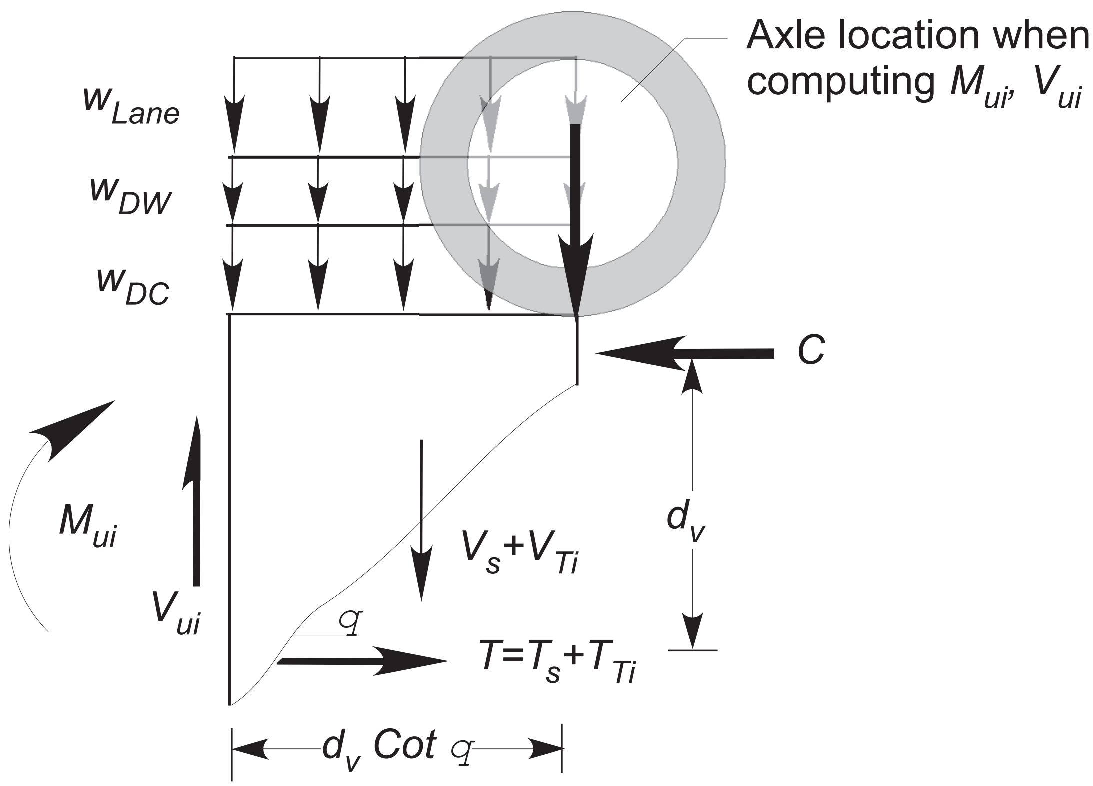

Figure 11.3.3-1 Free-Body Diagram for Computing Flexural Tension Demand in a Section with a Diagonal Crack

The tension demand in the flexural reinforcement (both steel and TiAB) is computed for the free-body diagram of Figure 11.3.3-1 as

$$T_{\text{demand}} = \frac{|M_{ut}|}{\phi_b d_v} + \left[\frac{V_{ut}}{\phi_v} - \frac{V_s}{2} - \frac{V_{Ti}}{2} - \left(\frac{\gamma_{DC} w_{DC}}{2} + \frac{\gamma_{DW} w_{DW}}{2} + \frac{\gamma_{LL} w_{\text{lane}} DF_M}{2}\right) d_v \cot\theta\right] \cot\theta \quad (11.3.3\text{-}2)$$

where:

M_{ui} and V_{ui} = factored moment and shear from the Strength I load combination, respectively, that occur coincidently and produce the largest magnitude moment at the vertical section, but when the truck axle is positioned at the tip of the projected diagonal crack (a distance $d_v \cot\theta$ from the vertical section);

V_s = maximum force in the steel stirrups;

V_{Ti} = maximum force in the TiAB stirrups acting across the diagonal crack;

w_{DC} and w_{DW} = distributed weight of components and wearing surface per girder, respectively;

γ_{DC} and γ_{DW}= Strength I load combination load factors for weight of components and weight of wearing surface, respectively;

w_{lane} = distributed lane load (0.64 kip/ft);

DF_M = live-load distribution factor for moment; and

γ_{LL} = Strength I load combination live-load factor.

Additional structural analysis for the AASHTO truck and tandem-load models is required to compute the moment and shear at the vertical section when the load is located at the diagonal crack tip. The analysis cannot be performed without knowing the magnitude of $d_v \cot\theta$, and these values vary with the moment and shear along the span. As a first approximation, the values for d_v and $\cot\theta$ are used from the shear design previously. Then, for the given M_{ui} and V_{ui}, the values of d_v and $\cot\theta$ are updated. A single iteration should be adequate. The contributions from the permanent loads and lane load can conservatively be disregarded in Equation 11.3.3-2 (as the difference is small, as shown by the bottommost, purple, curve in Figure 11.3.3-2).

The adequacy of the flexural tension force is computed for the same location, 12.5 ft. Knowing the values of d_v = 33.31 in. and $\cot\theta = \cot(35.90) = 1.38$ from the shear design previously, the structural analysis is repeated to compute M_{ui} and V_{ui} at the location 12.5 ft, but when the controlling axle location for maximum moment is located $d_v \cot\theta$ = 3.83 ft farther on the span (i.e., located at 16.33 ft). The values for M_{ui} = 359.1 kip-ft and V_{ui} = 98.3 kips when the controlling axle load is at the crack tip. For these values of moment and shear, the shear analysis can be repeated to get updated values of ε_s= 0.00167709, θ = 34.87 degrees, V_{Ti} = 27.9 kips, and V_s = 63.7 kips. These values are reasonably close to the initial shear design values used in the second analysis, and no further iteration is required. The flexural demand is calculated as

$$T_{\text{demand}} = \frac{|M_{ui}|}{\phi_b d_v} + \left[\frac{V_{ui}}{\phi_v} - \frac{V_s}{2} - \frac{V_{Ti}}{2} - \left(\frac{\gamma_{DC} w_{DC}}{2} + \frac{\gamma_{DW} w_{DW}}{2} + \frac{\gamma_{LL} w_{\text{lane}} DF_M}{2}\right) d_v \cot\theta\right]\cot\theta$$

$$= \frac{359.1 \text{ kip-ft } (12 \text{ in./ft})}{(0.9)33.31 \text{ in.}} + \tag{11.3.3-3}$$

$$\left[\frac{98.3 \text{ kips}}{0.9} - \frac{63.7 \text{ kips}}{2} - \frac{27.9 \text{ kips}}{2} - \left(\frac{1.25(1.18 \text{ kip/ft})}{2} + \frac{1.5(0.33 \text{ kip/ft})}{2} + \frac{1.75(0.64 \text{ kip/ft})0.78}{2}\right)\frac{33.31 \text{ in.}}{12 \text{ in./ft}} \cot 34.87\right]\cot 34.87$$

$$= 226.6 \text{ kips}$$

The available flexural tension force at this location comes from three #11 steel reinforcing bars and two #6 TiABs. The available tension force is computed as

$$V_s = \frac{A_v f_y d_v (\cot\theta)}{s} = \frac{2(0.2 \text{ in.}^2)(40 \text{ ksi})33.3 \text{ in.}(\cot 36.6)}{12 \text{ in.}} = 59.8 \text{ kips} \tag{11.3.3-4}$$

As seen here, the flexural steel and TiABs are adequate to resist the additional demands from combined flexural tension and shear. This analysis is repeated for the other locations along the span. The results are shown in Figure 11.3.3-2, which shows the flexural tension reinforcing is adequate.

There is a location near the #11 cutoff where the demands are close to the available capacity, and it may be desirable to add an additional TiAB stirrup over the region where the #11 bars are being developed.

A design sketch is shown in Figure 11.3.3-3a and 11.3.3-3b.

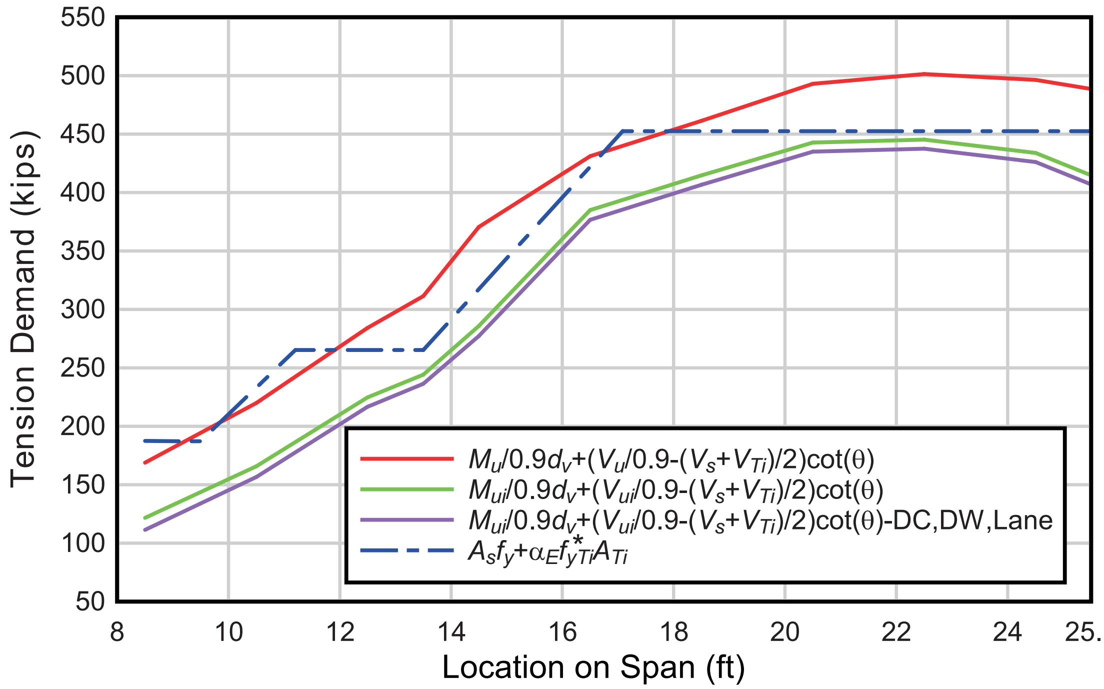

Figure 11.3.3-2. Flexural Reinforcing Demands: Excessively Conservative from Equation 11.3.3-1; Properly Estimated by Equation 11.3.3-2 with and without Permanent and Lane-Load Effects; and Available Tension Capacity from Reinforcing Steel and TiABs

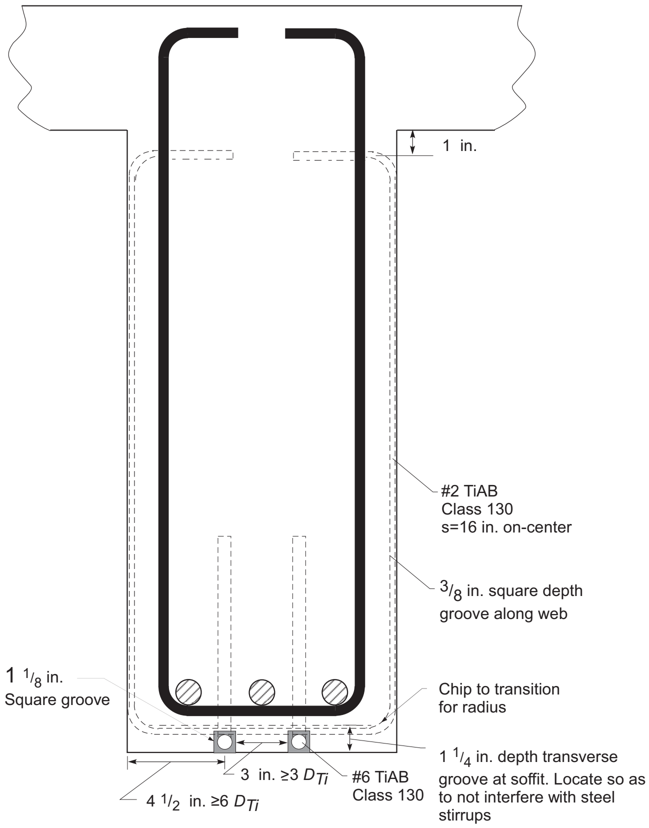

Figure 11.3.3-3a. Section View of Strengthening Design with TiABs

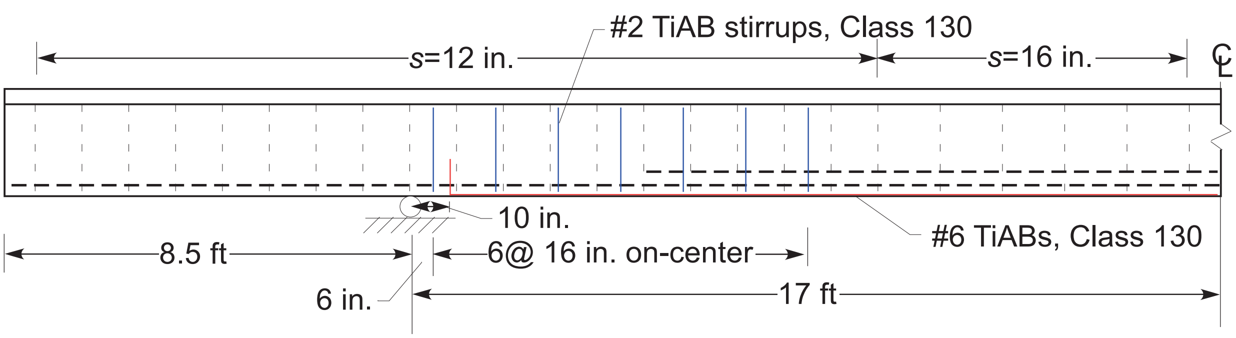

Figure 11.3.3-3b. Elevation View of Strengthening Design with TiABs