ASCE-INDOT
STRUCTURAL SUBCOMMITTEE
MEETING NO. 58 MINUTES
March 15, 2013

The meeting was called to order at 9:00 a.m. by Anne Rearick. Those in attendance were:

Anne Rearick	INDOT, Bridge Division
Elizabeth Phillips	INDOT, Bridge Division
Naveed Burki	INDOT, Bridge Division
Mahmoud Hailat	INDOT, Bridge Division
Merril Dougherty	INDOT, Structural Services
Mike Wenning	GAI Consultants, Inc.
Mike McCool	Beam Longest & Neff, LLC.
Pete White	R. W. Armstrong
Mike Halterman	USI Consultants, Inc.
Michael Matel	Butler, Fairman and Seufert, Inc.
Burleigh Law	HNTB Corp.
Kurt Heidenreich	Engineering Resources, Inc.

In addition to the attendees, these minutes will be sent to the following:

Keith Hoernschmeyer	Federal Highway Administration
Jason Yeager	Gohmann Asphalt Company
Jim Reilman	INDOT, Construction Management
Tom Harris	INDOT, Construction Management
Celeste Spaans	Prestress Services, Inc.
Troy Jessop	R. W. Armstrong
Michael Eichenauer	Butler, Fairman and Seufert, Inc.

A meeting agenda had previously been distributed and the following items were discussed:

1. The November 8, 2012, meeting minutes were approved as written, and have been placed on the INDOT website.

2. No action was taken on the PTFE plates A task committee consisting of Mike Wenning*, Mike Eichenauer and Mahmoud Hailat has been asked to investigate this and report back to the committee. Kenny Anderson will be included for input and material.

3. Troy Jessop* and Celeste Spaans were asked to investigate whether elastomeric bearing pads need to be vulcanized to shim plates to keep the bearing assembly from “walking” and report back at the next meeting. The task `committee is still working on this.

4. No action was taken on the pavement ledge details.

5. No action was taken on the prestressed beam notch issue. Troy Jessop* and Celeste Spaans will investigate revising the details to show that the beam notch should not be the designers’ first choice.

6. The R.C. Bridge approach detail revision will require a Design Manual change. Elizabeth Phillips will pursue these changes. Mike Wenning will help if standard drawings are created.

7. INDOT would like to develop a standard beam detail sheet to be used in the plans similar to that used by Kentucky and other states. These are still being developed.

8. The committee would like to collect and share practice pointers dealing with commonly used design programs. The committee has 2 weeks to add items then it will be posted and announced via the listserve.

9. Stay-in-place metal form attachment requirements were pulled from the Standards Committee due to industry comments. Elizabeth Phillips will reintroduce the changes clarifying that this only affects angle connections in the "L" position.

10. Steel Diaphragm details need to be updated to include Hybrid Girders. Elizabeth Phillips will provide details for review at the next meeting.

11. Steel Diaphragm details need to be updated for rolled beam sections. The 2012 AASHTO allows less than full width bolted connections for rolled beams. Mike McCool will develop details and have them reviewed by Burleigh Law and Mahmoud Hailat. This will reintroduce Fig. 8-405.08A from the old Design Manual.

12. Stainless steel beveled plates should be used for elastomeric bearings under joints. All other locations may be galvanized.

13. Connection Plate Details – Elizabeth brought suggested details changes that agree with IDM 407-503.1 text. She will send markups to the committee so they can review and comment. Elizabeth will then present revised details and ext at the next meeting for approval.

14. It was decided to remove the 25% restriction for field splices in the IDM section 407-1.02(05)

15. The date for the next Bridge Conference will be set between Sept. 10-12, 2013. Anne Rearick, Mike McCool, Kurt Heidenreich and Mike Halterman will work on this.

16. The vertical clearance requirements apply to all components within the required shoulder width as opposed to the provided width. These include pier caps, wall copings and any other portion of the bridge that may extend.

* indicates the person primarily responsible for the activity.

The next meeting for the INDOT Structural Committee is scheduled for 9:00 a.m. on May 24, 2013, in room N642. Mike McCool will distribute an agenda prior to the meeting. This meeting was adjourned at 11:00 a.m.

Respectfully submitted,
GAI Consultants, Inc.

Michael Wenning, P.E.
m.wenning@gaiconsultants.com

Attachments

Phillips, Elizabeth

From: Phillips, Elizabeth
Sent: Wednesday, February 20, 2013 4:45 PM
To: McCool, Mike
Subject: FW: Proposed Spec Change - Stay in Place Deck Forms

Mike,
Limiting the projection of the support angle to ½" is on the Standards Committee agenda for tomorrow. Below is some feedback I got today from the construction industry. I'm not seeing the issue as we already require a minimum ¾" fillet on the low side. Would you help me with some explanation to alleviate their concerns or explain to me why this is a bigger issue than I thought.
Thanks,
Elizabeth

There is considerable industry concern about the proposal to limit the projection of the form support (angle) to ½-inch above the top flange. Contractors see that this would create delays and add considerable costs to projects. There is also concern that this could result in thicker decks, increasing the dead load of the structure.

Designers and suppliers would have to work together on camber, decking and profile grade. The contractor has little control over these factors.

Until the distance between the top of the beam and the finish grade of the deck is determined, the installer and supplier may not be able to determine the angle sizes. The installer may wait until fillet depths are established to order the support angles. I'm told this could take 4-6 weeks. A tolerance of only ½-inch does not allow for much tolerance if the top of the beams are not consistent.

Contractors see that they would have to either cut the angle (form support) after placement to grade or set to the half-inch and place a thicker deck.

These are some of the concerns that a number of contractors have expressed about this proposed change. I'm expecting one of these firms will be at the meeting Thursday to elaborate on these concerns.

Paul Berebitsky

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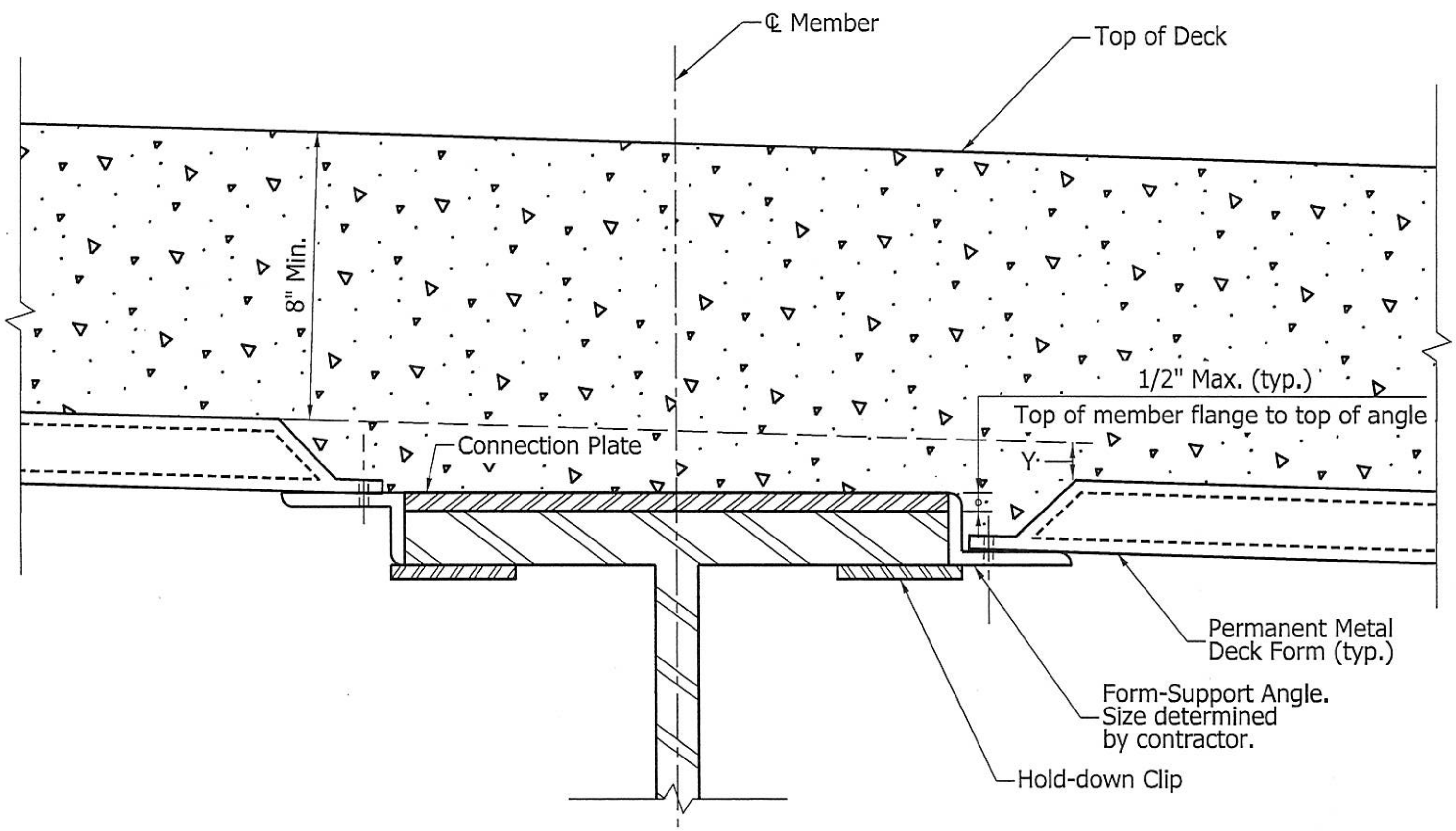

FILLET TREATMENT FOR
STRUCTURAL-STEEL MEMBER

Figure 404-2B

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Meeting 58, Item 9 Attachment

These rigid load paths are required to preclude the development of significant secondary stresses that could induce fatigue crack growth in either the longitudinal or the transverse member (Fisher et al., 1990).

6.6.1.3.1—Transverse Connection Plates

Except as specified herein, connection plates shall be welded or bolted to both the compression and tension flanges of the cross-section where:

- Connecting diaphragms or cross-frames are attached to transverse connection plates or to transverse stiffeners functioning as connection plates,
- Internal or external diaphragms or cross-frames are attached to transverse connection plates or to transverse stiffeners functioning as connection plates, and
- Floorbeams or stringers are attached to transverse connection plates or to transverse stiffeners functioning as connection plates.

In the absence of better information, the welded or bolted connection should be designed to resist a 20.0-kip lateral load for straight, nonskewed bridges.

Where intermediate connecting diaphragms are used:

- On rolled beams in straight bridges with composite reinforced decks whose supports are normal or skewed not more than 10 degrees from normal and
- With the intermediate diaphragms placed in contiguous lines parallel to the supports.

less than full-depth end angles or connection plates may be bolted or welded to the beam web to connect the diaphragms. The end angles or plates shall be at least two-thirds the depth of the web. For bolted angles, a minimum gap of 3.0 in. shall be provided between the top and bottom bolt holes and each flange. Bolt spacing requirements specified in Article 6.13.2.6 shall be satisfied. For welded angles or plates, a minimum gap of 3.0 in. shall be provided between the top and bottom of the end-angle or plate welds and each flange; the heel and toe of the end angles or both sides of the connection plate, as applicable, shall be welded to the beam web. Welds shall not be placed along the top and bottom of the end angles or connection plates.

C6.6.1.3.1

These provisions appear in Article 10.20 of the AASHTO *Standard Specifications* "Diaphragms and Cross Frames" with no explanation as to the rationale for the requirements and no reference to distortion-induced fatigue.

These provisions apply to both diaphragms between longitudinal members and diaphragms internal to longitudinal members.

The 20.0-kip load represents a rule of thumb for straight, nonskewed bridges. For curved or skewed bridges, the diaphragm forces should be determined by analysis (Keating et al., 1990). It is noted that the stiffness of this connection is critical to help control relative displacement between the components. Hence, where possible, a welded connection is preferred as a bolted connection possessing sufficient stiffness may not be economical.

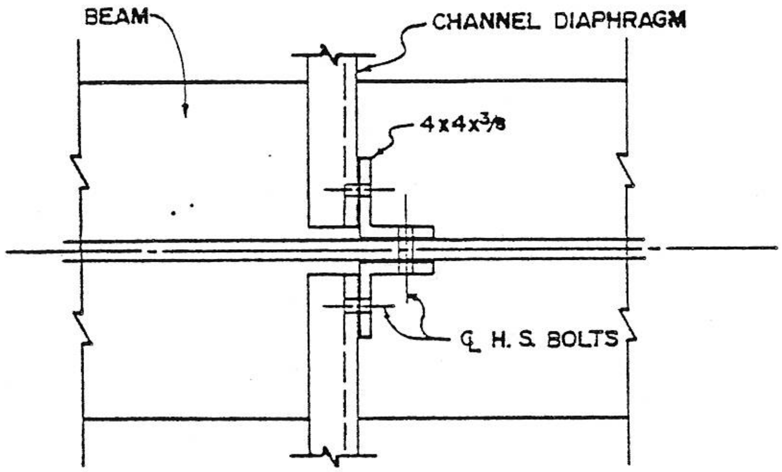
For box sections, webs are often joined to top flanges and cross-frame connection plates and transverse stiffeners are installed, and then these assemblies are attached to the common box flange. In order to weld the webs continuously to the box flange inside the box section, the details in this case should allow the welding head to clear the bottom of the connection plates and stiffeners. A similar detail may also be required for any intermediate transverse stiffeners that are to be attached to the box flange. Suggested details are shown in AASHTO/NSBA (2003). The Engineer is advised to consult with fabricators regarding the preferred approach for fabricating the box section and provide alternate details on the plans, if necessary.

6.6.1.3.2—Lateral Connection Plates

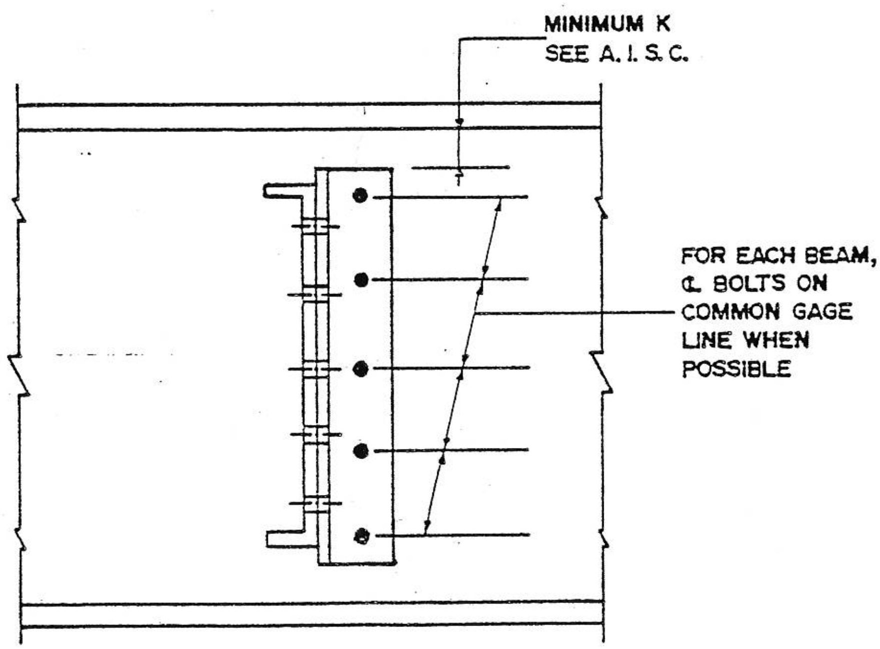
If it is not practical to attach lateral connection plates to flanges, lateral connection plates on stiffened webs should be located a vertical distance not less than one-half the width of the flange above or below the flange. Lateral connection plates attached to unstiffened webs should be located at least 6.0 in. above or below the flange but not less than one-half of the width of the flange, as specified above.

C6.6.1.3.2

The specified minimum distance from the flange is intended to reduce the concentration of out-of-plane distortion in the web between the lateral connection plate and the flange to a tolerable magnitude. It also provides adequate electrode access and moves the connection plate closer to the neutral axis of the girder to reduce the impact of the weld termination on fatigue strength.



PLAN

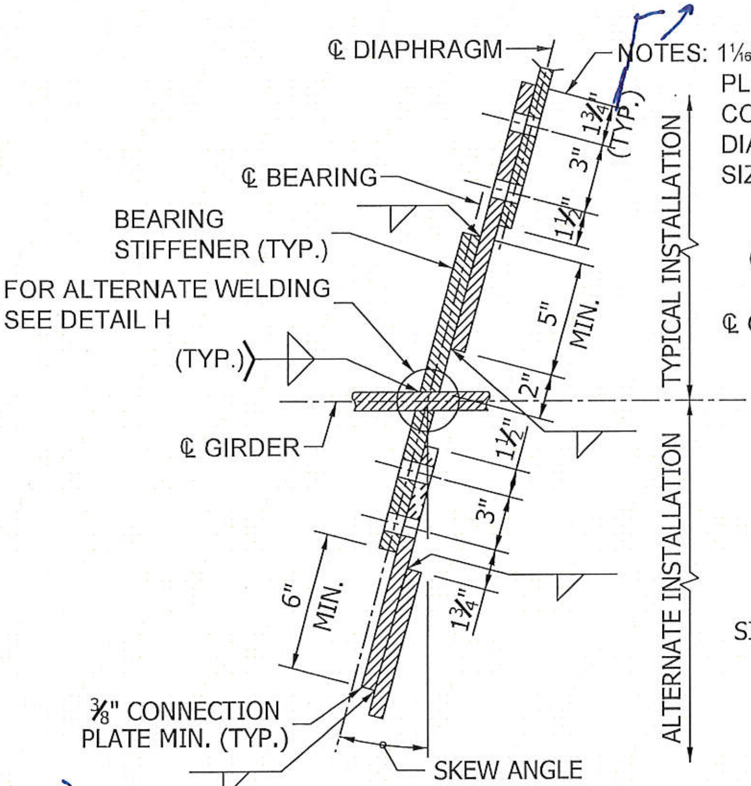


ELEVATION

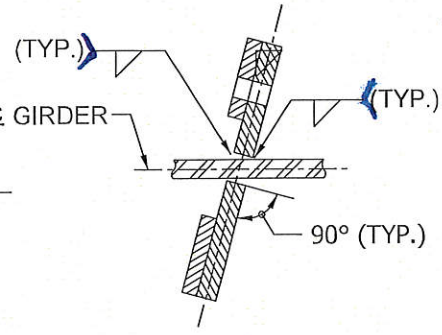
BEAM	DIAPHRAGM	HIGH STRENGTH BOLTS
W 36	C 18x42.7 – Not readily Avail 10/23/01	5-7/8" ø
W 33	C 18x42.7	5-7/8" ø
W 30	C 15x33.9	4-7/8" ø
W 27	C 15x33.9	4-7/8" ø
W 24	C 12x20.7	3-3/4" ø

TYPICAL DIAPHRAGM
CONTINUOUS SUPPORTS
ITERMEDIATE SPAN POINTS

Figure 8-405.08A



SKEW 0º TO 20º



DETAIL H

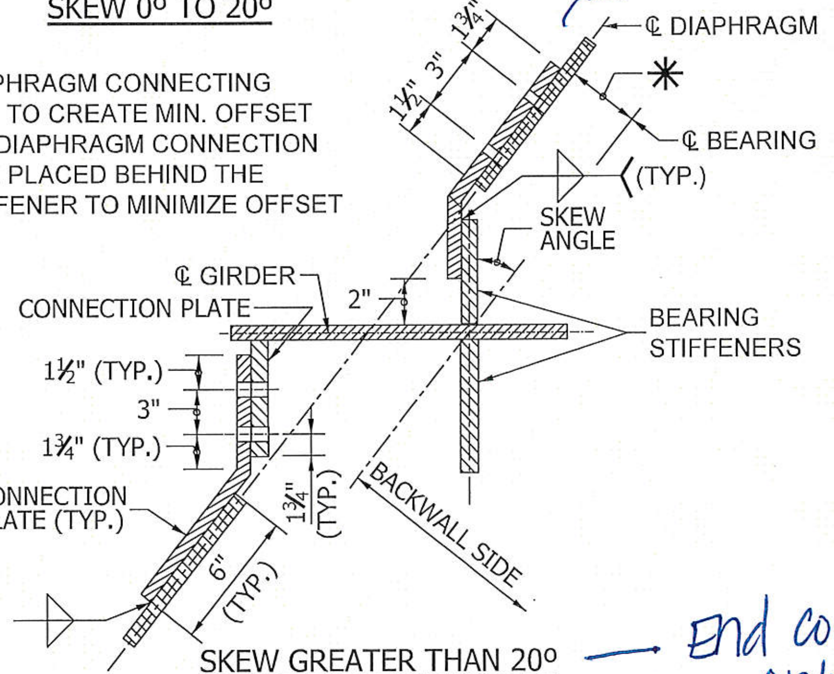
SIMILAR TO NORMAL CONNECTION EXCEPT AS NOTED

same as 5D lower right.

Intermediate end interior support

✳ POSITION DIAPHRAGM CONNECTING COMPONENTS TO CREATE MIN. OFFSET FROM ℄ BRG. DIAPHRAGM CONNECTION PLATE MAY BE PLACED BEHIND THE BEARING STIFFENER TO MINIMIZE OFFSET

=skewed.



SKEW GREATER THAN 20º

End condition only

Note: All member sizes shown are minimum.
Designer shall verify actual sizes by design.

CONNECTION PLATE DETAILS

Figure 407-5H

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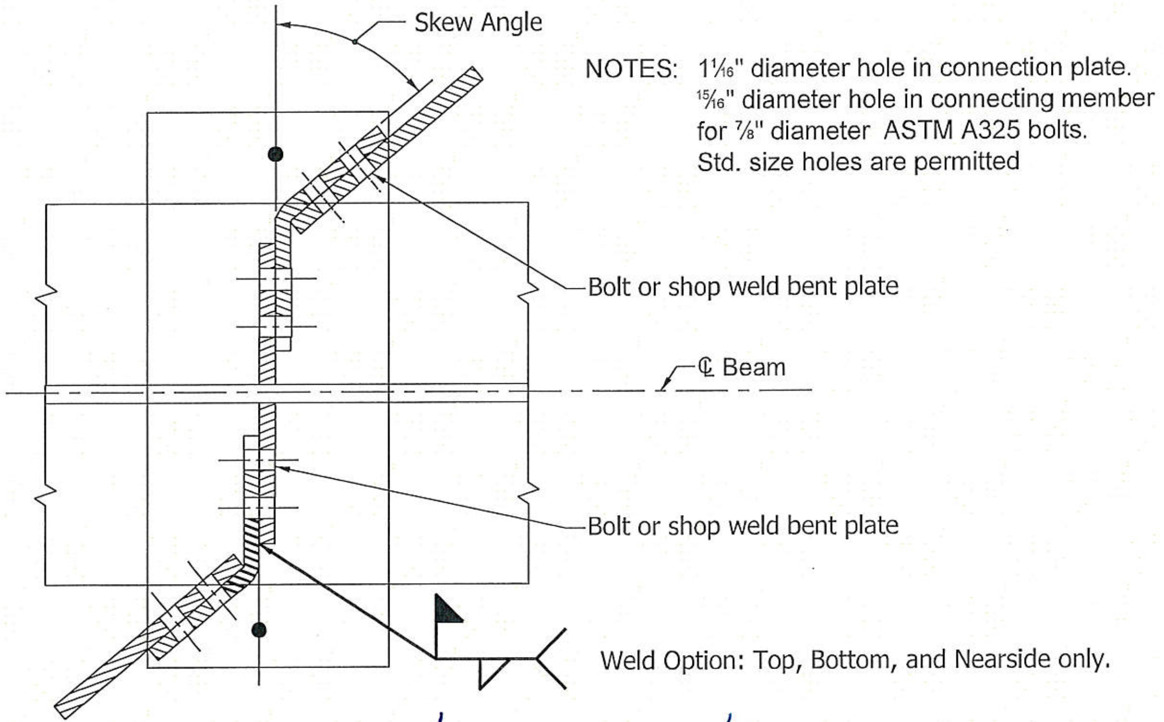
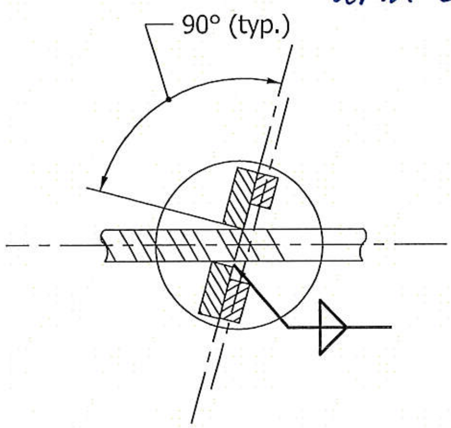
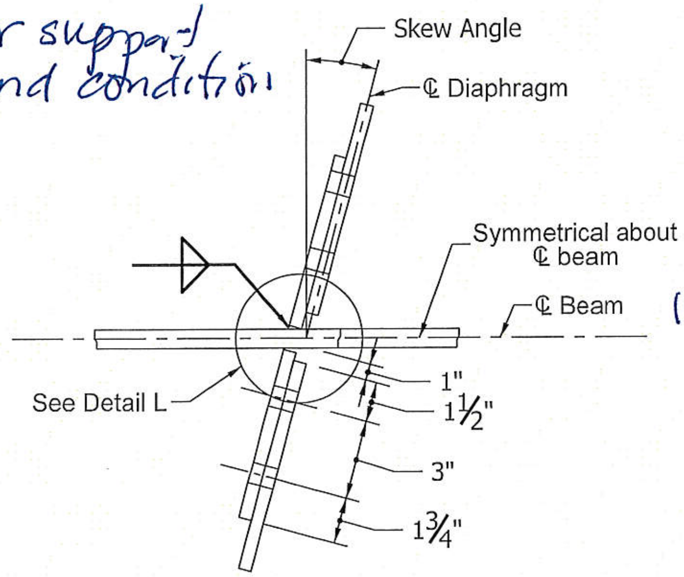
intermediate, end, interior support



SKEWS > 20º

interior support
and end condition





same
as 5D
lower
left.

DETAIL L

SKEW 0º TO 20º

CONNECTION PLATE DETAILS

intermediate
interior support

NOTE: All member sizes shown are minimum.
Designer shall verify actual sizes by design.

CONNECTION PLATE DETAILS

Figure 407-5 I

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407-1.02(05) Field and Shop Splices

Field and shop splices shall be designed in accordance with *LRFD* 6.13.6. Field splices are expensive and their number should be minimized. Field splices are used to reduce shipping length. The flange cross-sectional area should be reduced by not more than approximately 25% of the area of the heavier flange plate at the splice location.

Not more than two shop flange splices, or three plate thicknesses, should be included in the top or bottom flange within a single field section. Constant flange widths should be maintained within a field section for economy of fabrication as specified in Section 407-1.02(04). In determining the points where changes in plate thickness occur within a field section, the cost of groove-welded splices should be compared against the extra plate area. The National Steel Bridge Alliance (NSBA) or local fabricators should be consulted if possible to ascertain current costs. The flange cross-sectional area should be reduced by not more than 50% of the area of the heavier flange plate at the shop splices to reduce the buildup of stresses at the transition. Typically 400 to 700 lb of steel must be saved to justify the cost of a groove-welded shop splice.

The thicker plate can often be continued beyond the theoretical step-down point to avoid the cost of the groove-welded splice.

To facilitate testing of the weld, flange shop splices shall be located at least 2 ft away from web splices. Flange and web shop splices shall be located at least 6 in. from transverse stiffeners. See Figure 407-1C for typical plate-girder welded-splice details.

407-1.02(06) Web Plate

Preliminary design services available through the NSBA and some steel companies can be used for the optimization of the web depth as a starting point. Preliminary line-girder analysis shall be performed to optimize the girder geometrics for cost. Other programs or methods can also be used if they are based upon material use and fabrication unit costs. Web depth should be in 1-in. or, preferably, 2-in. increments. Web thickness should be in 1/16-in. increments. The web of a plate girder is typically deeper and thinner than that of a rolled beam.

Web design can have a significant impact on the overall cost of a plate girder. Considering material costs, it is desirable to make the girder web as thin as design considerations will permit. However, this may not always produce the greatest economy, since fabricating and installing stiffeners is one of the more labor-intensive of shop operations.

The use of transverse stiffeners should be determined using the following guidelines and, except for diaphragm connections, should be placed on only one side of the web.