ASCE-INDOT
STRUCTURAL SUBCOMMITTEE
MEETING NO. 51 MINUTES
April 19, 2011

The meeting was called to order at 2:05 p.m. by Steve Weintraut. Those in attendance were:

Randy Strain	INDOT, Structural Services
Anne Rearick	INDOT, Structural Services
Tony Uremovich	INDOT, Structural Services
Mike Wenning	American Structurepoint, Inc.
Burleigh Law	HNTB Corp.
Mike McCool	Beam Longest & Neff, LLC.
Troy Jessup	R. W. Armstrong
Celeste Spaans	Prestress Services, Inc.
Jason Yeager	Gohman Asphalt Company
Mike Halterman	USI Consultants, Inc.
Steve Weintraut	Butler, Fairman and Seufert, Inc.
Michael Eichenauer	Butler, Fairman and Seufert, Inc.

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

Ron McCaslin	INDOT, Structural Services
Tony Zander	INDOT, Materials and Tests Division
Bill Dittrich	INDOT, Program Development
Jim Reilman	INDOT, Construction Management
Brian Harvey	INDOT, Program Development
Keith Hoernschmeyer	Federal Highway Administration

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

1. The January 19, 2011, meeting minutes were approved as written, and have been placed on the INDOT website.

2. A semi-light weight concrete specification has been approved by the INDOT Standards Committee. It includes the use of both expanded slate and expanded shale. It should be ready in 2-3 months for this group to review.

3. No further action is being done on self consolidating concrete and it will be removed from the agenda for future meetings.

4. Bridge Design Conference will be on July 26. Conference Room B in the South Government Building has been reserved. Subcommittee has not met yet but will meet soon to come up with topics and an agenda. Various topics were tossed around at the meeting for the conference.

5. Mike McCool is in the process of developing a proposal for a research project at Purdue with Professor Frosch concerning mild reinforcement in prestressed beams.

6. Steve passed out a handout (Attachment #1) on welded wire fabric in prestressed beams. Randy has a group writing a specification for this. The welded wire

reinforcement would be an option for the prestressed beam manufacturer. They can use all WWF in beams, some WWF in beams or all regular reinforcement in beams. Celeste will discuss with her company on details and comments.

7. Randy is still developing a post tension manual but the timeline on completion is up in the air.

8. The special provision for high strength concrete is a low priority at this time.

9. Skewed end approach slabs may have issues with integral end bents. This topic will be removed from the agenda until more research is done. Mike Wenning would like INDOT to revisit their approach slab standards and update them as appropriate.

10. Rehabilitated bridges need to be checked for construction loading unless the scope is for minor repairs or an overlay. However, since the current construction loading information is for LRFD, a new memo will be issued to cover LFD rehabilitation.

11. Deflections on concrete bridge beam, particularly long term deflections, are known to be an issue. Research is being done on deflections to better represent these beams. New factors are expected to be available within a year.

12. PCI and ACI recommend $6d_b$ for stirrups. However, for #3, #4, or #5 rebars, you can use $4d_b$. Celeste stated that there are cracking issues not only in the epoxy coating but in the rebar itself when the bend diameter is less than $6d_b$. Mike McCool suggested to bundle the #5 stirrup bars (3d bar and #5 U shaped bar) at the ends of the beam. Also, on draped strands, he suggested analyzing the force for the top draped strands and the bottom straight strands, not the total strand force as most computer programs output.

13. Jim was not present to discuss articulating mats.

The next meeting for the INDOT Structural Subcommittee is scheduled for 2:00 p.m. on July 7, 2011, in a room to be determined.

This meeting was adjourned at 4:00 p.m.

Respectfully submitted,
BUTLER, FAIRMAN and SEUFERT, INC.

Michael H. Eichenauer

Michael Eichenauer, P.E.
meichenauer@bfsengr.com

ME:me

Attachments

<u>PROPOSAL FOR USE OF WWF IN PRECAST CONCRETE BEAMS</u>

At the option of the contractor, welded deformed wire reinforcement as shown in the "alternate beam section with welded wire and bar", may be used in lieu of the conventional deformed bars shown in standard section. The welded wire reinforcement shall be of the size shown on the alternate section and shall be spaced the same as the conventional bar reinforcement shown for the standard section.

The WWF that extends out of the top of the beam into the composite deck shall be epoxy coated. The beam manufacturer shall provide shop drawings for approval by the engineer which show the wire/bar reinforcement in the beam.

ADVANTAGES OF WELDED WIRE FABRIC

Time savings primarily come from improved field labor productivity which translates into money savings.
Other advantages include reduced equipment costs, lower insurance premiums, and in some cases lower material costs due to optimal use of material and its higher yield strength.

Other Advantages

- Because bars are welded in a mat, the bars do not move when concrete is placed, insuring bars are in their proper position.
- Since WWR is prefabricated and welded in plants, the risk of missing a bar is minimized.
- The chance of improper bending of bars is reduced by using WWR since bending machines bend the mat as a single unit.
- The use of WWR minimizes the chance of misplacement since only one type of mat is used on a given section.
- Provides the exact size of reinforcement where needed through variable bar size and spacing, thereby reduces steel waste.
- Any wire size can be used from, starting at W 1.4 up to W 46.5 in increments of 0.1.
- WWR can reduce construction time.
- Less field labor needed.
- Less crowding in the precast beam due less because of higher yield strength and elimination of hooked bars at the top and bottom of the beam vertical web reinforcement.

- Less epoxy coated steel, since only the top bars that are exposed above the top of the beam need be coated.

Almost any place that contractors have been using rebar, WWR will do the job. Because of the ability to shape and bend the wire and the potential for thin slabs, WWR has also been a large part of the concrete pipe and box culvert industries in the past. In fact, welded wire accounts for nearly 80% of all concrete pipe reinforcement and is gaining momentum in the box culvert industry.

For bridge structures, WWR structural shear reinforcement is seeing its way into more precast/pre-stressed girders, beams, boxes and bulb-tees. In the past 10 years that have precast/pre-stressed concrete spans of over 150 feet in which most of the bridge components have WWR shear reinforcement for the complete length of the spans. The NU 2000, 150-foot "I" girder, with a depth of 7 feet and a top flange width of four feet was developed at the University of Nebraska by Dr. Maher Tadros and his graduate students.

Dr. Tadros' girder has over two tons of shear reinforcement in the web and flanges. Similar 'I' girders are being designed by the State of Nebraska and other states with spans to 200 feet. The typical cast in place bridge decks or precast/pre-stressed replacement deck panels can also utilize mats of WWR. With WWR manufacturing capability or WWR up to 3/4" diameter, typical bridge reinforcing can be substituted with WWR.

One of the greatest things to happen to WWR technology in the past few years has been the ever-increasing wire diameter and materials that manufacturers have been able to weld together. Not only are some manufacturers selling welded wire that is 3/4" in diameter, but there are also zinc-coated and epoxy-coated products available to resist corrosion. These advancements have allowed WWR to move from just road and slab reinforcement to structural components in bridges.

WWR material cost is usually higher than traditional rebar due to a higher fabrication cost. However, even with a higher material cost, WWR installed prices is on average around 20% lower.

WWR vs Traditional Rebar...

This table shows the total cost (material and labor) for each option. Notice WWR material cost is higher than rebar. Even with higher material costs it is more economical than rebar.

Cost Items	Placement Method Traditional Rebar	WWR D12 Manual (8 x 18 ft)	WWR D12 Mechanical (8 x 40 ft)
Material			
Quantity (t)	45.7	44.5	43.0
Unit cost (dollars)	600.00	750.00	750.00
Total base cost (dollars)	27,420.00	33,375.00	32,250.00
Total cost including 10% O&P (dollars)	30,162.00	36,712.00	35,475.00
Labor			
Productivity (man-hours/t)	18.8	4.0	1.4
Crew			
Foreman	1 at $22.00	1 at $22.00	1 at $22.00
Skilled laborers	4 at $20.00	5 at $20.00	2 at $20.00
Unskilled laborers	4 at $15.00	8 at $15.00	2 at $15.00
Crew cost (dollars/hr)	162.00	242.00	92.00
Total crew time (hrs)	96	13	12
Total bare installation (dollars)	15,552.00	3,146.00	1,104.00
Total labor cost including 20% O&P ($)	18,662.00	3,775.00	1,325.00
Equipment			
Crane (for placement only)	0	0	1
Cost including operator ($/day)	0.00	0.00	1,182.00
Total equipment cost including 20% O&P ($)	0.00	0.00	2,128.00
Total work days (work days)	12.0	1.5	1.5
Total cost ($)	48,824.00	40,487.00	38,928.00
Cost savings (%)		18	21

...with inflated WWR costs

This table shows the cost comparison for each option adding WWR material waste cost and a higher labor cost. *WWR is still more economical with both adjustments.*

Cost Items	Placement Method		
	Traditional Rebar	WWR D12 Manual (8 x 18 ft)	WWR D12 Mechanical (8 x 40 ft)
Material			
Original total cost including 10% O&P ($)	30,162.00	36,712.00	35,475.00
5% waste for WWR		1,836.00	1,774.00
Adjusted total cost including 10% O&P ($)	30,612.00	38,548.00	37,249.00
Labor			
Original average crew cost ($/mhr)	21.60	21.74	22.10
Original total labor cost including 20% O&P ($)	18,662.00	3,775.00	1,325.00
Adjusted average crew cost ($/mhr)	36.70	36.70	36.70
Adjusted labor productivity (mhrs/t)	15.40		
Adjusted total labor cost including 20% O&P ($)	25,829.00	6,679.40	2,202.00
Equipment			
Total equipment cost including 20% O&P ($)			2,128.00
Total cost ($)			
Original total cost ($)	48,824.00	40,487.00	38,928.00
Total cost with 5% WWR waste ($)	48,824.00	42,323.00	40,702.00
Total cost with adjusted labor rates ($)	55,991.00	43,391.40	39,805.00
Total cost with combined effect ($)	55,991.00	45,227.40	41,579.00
Cost savings (%)			
Original cost savings (%)		18%	21%
Savings with 5% WWR waste (%)		13%	17%
Savings with adjusted labor rates (%)		23%	29%
Savings with combined effect (%)		19%	26%

WWR Time Advantage: It Installs Quickly

Two independent studies performed time comparison between WWR and reinforcement steel installation in different applications.
For flat slabs, productivity increase was 15 man hours per ton. This represents labor time savings on the order of 4 to 11 times faster for slab reinforcement. Greater time savings occurred for columns and walls. Using WWR for beam cages reduces installation time by half.

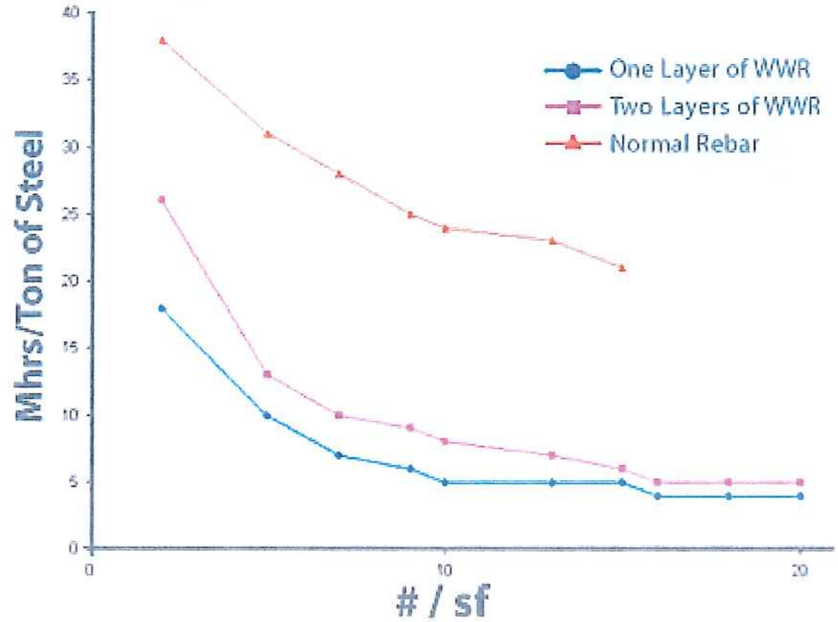
Study #1 Potential Gains Through WWR

For all rebar and WWR sizes, WWR is significantly faster to install, even for two layers of WWR. Productivity on some flat slabs can be increased by 250%.

On average, the productivity of one-way slab increases by 15 man-hours/t of steel when using WWR instead of traditional rebar. (1:2 in slabs, 1:3 in columns, 1:4 in walls)

This represents labor time savings on the order of 4 to 11 times faster for slab reinforcement.

Standard Laying Times For WWR and Traditional Rebar

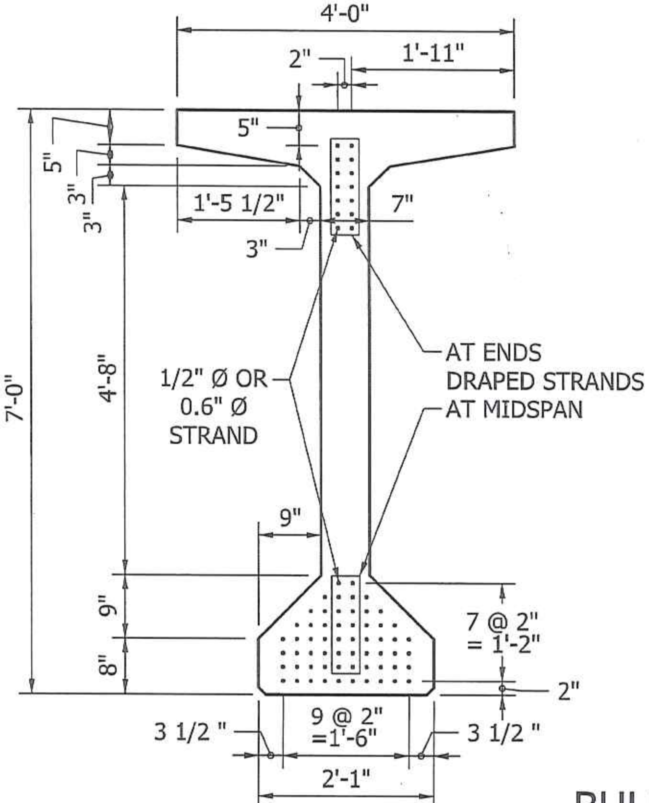


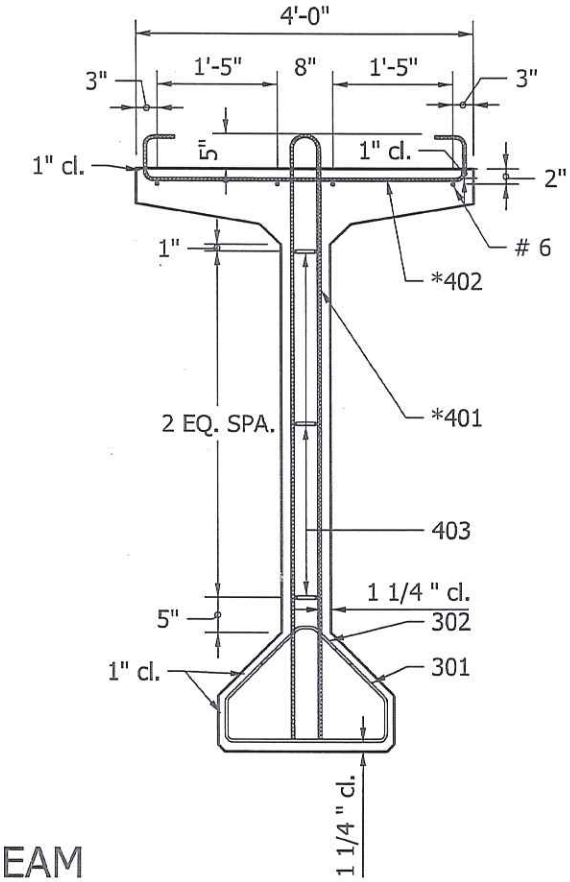
This was reported in the Journal of Construction Engineering and Management, June, 1992, in a paper titled "Potential gains through welded-wire fabric reinforcement", authored Bernold and Peter Chang

BEAM PROPERTIES

A_B = 1100 in^2
I_B = 1,048,921 in^2
S_{TB} = 26,733 in^3
S_{BB} = 23,433 in^3
Y_{TB} = 39.2 in
Y_{BB} = 44.8 in
Wt. = 1146 lb/lf

NOTES:
1. BARS 301 AND 302 COMBINED TO FORM ONE STIRRUP.
2. ▩ *DENOTES EPOXY-COATED BARS

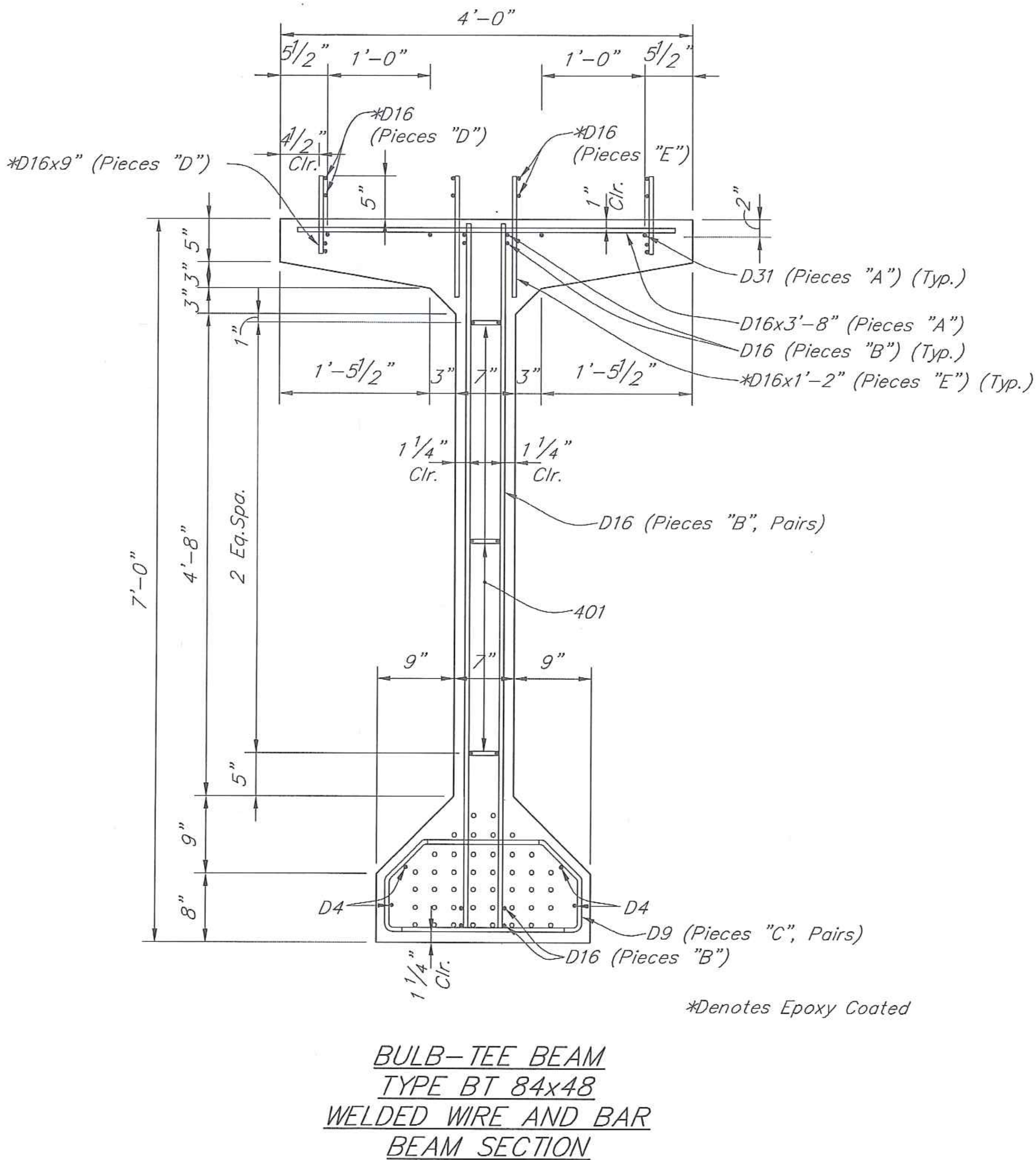




BULB - TEE BEAM
TYPE BT 84 x 48

Figure 63-14F(1)

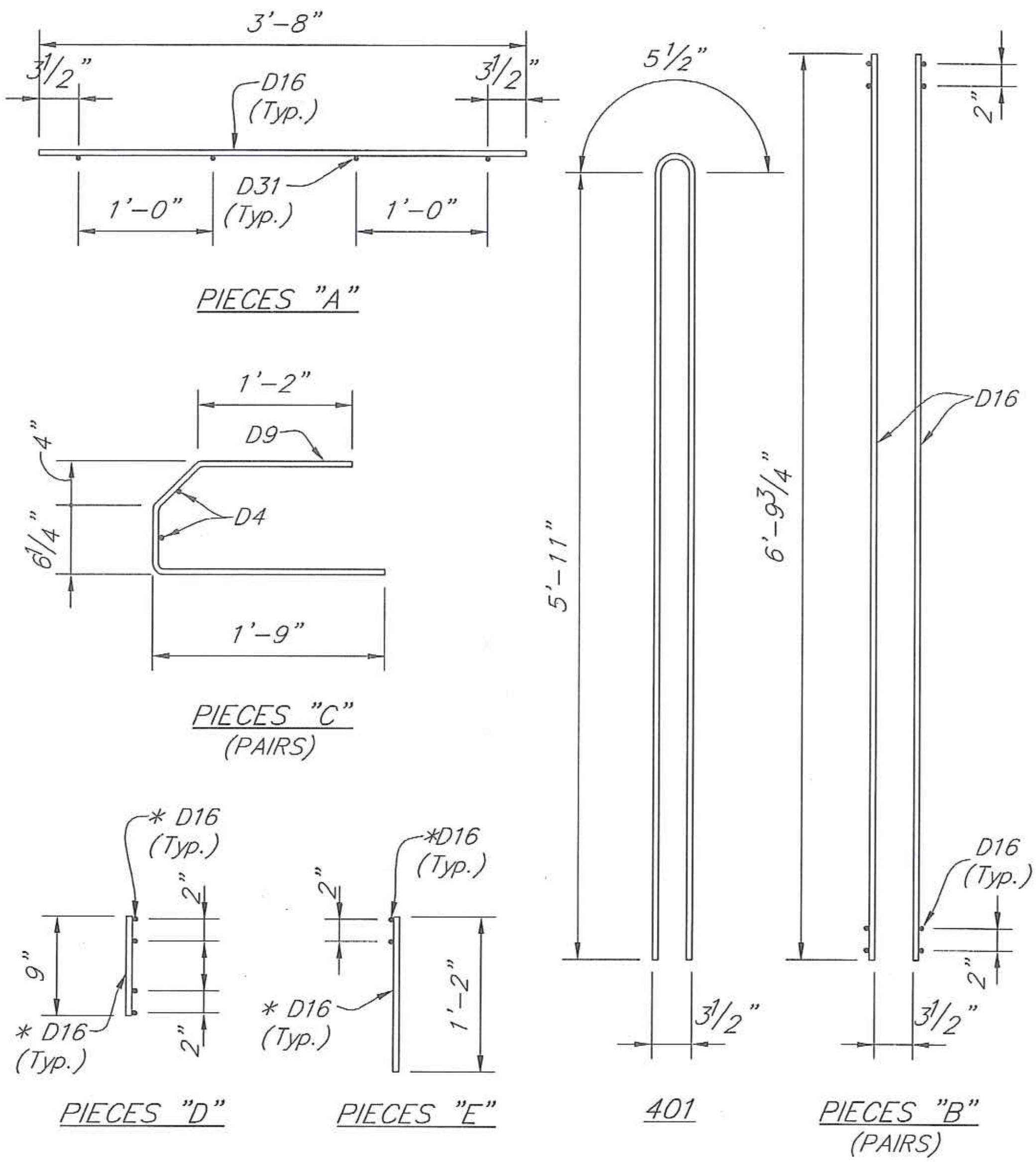
STANDARD BEAM SECTION



*Denotes Epoxy Coated

<u>BULB–TEE BEAM</u>
<u>TYPE BT 84x48</u>
<u>WELDED WIRE AND BAR</u>
<u>BEAM SECTION</u>

Scale: ¾" = 1'–0"



BULB-TEE BEAM
TYPE BT 84x48
WELDED WIRE AND BAR
BENDING DETAILS
Scale: 3/4" = 1'-0"

* Denotes Epoxy Coated Bars

SPECIFICATION FOR WELDED WIRE REINFORCEMENT

GENERAL

SUMMARY:

A. Work includes the furnishing and installation of welded wire reinforcement and miscellaneous welded wire reinforcement accessories as shown on the plans or shop drawings.

REFERENCES:

A. American Society for Testing and Materials (ASTM)

A82	Standard Specifications for Steel Wire, Plain, for Concrete Reinforcement
A123	Standard Specifications for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products
A185	Standard Specifications for Steel Welded Wire Reinforcement, Plain, for Concrete
A496	Standard Specifications for Steel Wire Reinforcement, Deformed, for Concrete
A497	Standard Specifications for Welded Wire Reinforcement, Deformed, for Concrete
A525	Specification for General Requirements for Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process
A884	Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement

B. CRSI, Manual of Standard Practice

C. Wire Reinforcement Institute, Manual of Standard Practice

D. American Road Builders Association, Technical Bulletin No. 265, 1968, Effects of Rust on Bond of Welded Wire Fabric

DEFINITIONS

A. Welded Wire Reinforcement - Welded Wire Reinforcement (WWR) designates a material composed of cold-worked steel wire, fabricated into sheets by the process of electric resistance welding.

B. Wire Size - Individual wire size designations are based on the cross-sectional area of a given wire. The "W" and "D" number represents the cross-sectional area of the wire multiplied by 100. The "W" represents a plain wire and the "D" represents a deformed wire. A D10 wire would indicate a deformed wire of 0.10 square inch.

C. Wire Spacing - The centerline-to-centerline distance between parallel wires.

D. Sheet Width - Center to center distance between outside longitudinal wires. This dimension does not include side overhangs.

E. Sheet Length - Tip to tip dimension of longitudinal wires (the length dimension always includes end overhangs).

F. Side Overhang - Extension of transverse wires beyond centerline of outside longitudinal wires (side overhangs are not included in the sheet width dimension).

G. Overall Width - Tip to tip dimension of transverse wires (this dimension is the sheet width plus both side overhangs).

H. End Overhangs - Extension of longitudinal wires beyond centerline of first and last traverse wires (end overhangs are included in the sheet length dimension).

SUBMITTALS

A. Submit the following in accordance with conditions of Contract and Section 01330, "Submittal Procedures".

1. Product Data: Submit product data for welded wire reinforcement, accessory, and other manufactured product indicated.

2. Shopdrawings for fabrication, bending, and placement of welded wire reinforcement, and additional reinforcing bars as required.

3. Conversion and lap calculations detailing the replacement of conventional reinforcing bars with welded wire reinforcement (include only if project is designed in conventional reinforcing steel).

4. Material certificates for the following, signed by manufacturer and Contractor certifying that each material complies with requirements established by this specification.

a. Certification shall include copies of material test reports certifying compliance with the governing ASTM specifications, submitted before or upon delivery of the welded wire reinforcement.

b. Deformed steel welded wire reinforcement shall be in accordance with ASTM A 497, except as follows.

1. The wire used in manufacturing the welded wire reinforcement shall be in accordance with ASTM A 496.

2. Weld shear tests of welded wire reinforcement shall be performed by the manufacturer on the test specimens obtained for testing tensile properties in

accordance with the Frequency Manual. If there is shear failure, additional test specimens shall be tested in accordance with ASTM A 497.

QUALITY ASSURANCE

A. Materials shall conform to governing ASTM specifications.

B. Welded Wire Reinforcement Tolerances

1. Sheet Width: The permissible variation shall not exceed ± 1/2 inch, center-to-center distance between outside longitudinal wires.

2. Overall Width: The permissible variation shall not exceed ± 1 inch of the overall width (tip-to-tip length of transverse wire).

3. Sheet Length: The overall length may vary by ± 1 inch or 1% whichever is greater.

4. Side Overhang: The permissible variation shall not exceed ± 1/2 inch.

5. Deformed Wire Weight: The weight of any deformed wire is ± 6%.

6. Plain Wire Diameter: The allowable variation in diameter of plain wire is as follows:

Wire Size	Diam. Variation
Smaller than W5.0	± 0.003 inch
W5.0 to W12.0	± 0.004 inch
W12.1 to W20.0	± 0.006 inch
Over W20	± 0.008 inch

C. Prior to placement of concrete, all welded wire reinforcement shall be free of contaminants that may adversely affect or reduce bond, such as oil or grease. Rust, surface irregularities, mill scale or a combination of all three will not be cause for rejection. Material will be accepted as being satisfactory without cleaning or brushing provided the dimensions, cross sectional area, and tensile properties of the reinforcement meet the requirements of these specifications.

D. Prior to placement of concrete, the Engineer, or the Engineer's representative, will inspect all reinforcing materials before and/or after installation to ensure compliance with the Contract Documents. All concrete placed in violation of these provisions shall be rejected and removed.

E. The Owner can conduct additional tests on the welded wire reinforcement to assure compliance with these specifications. Noncompliance demonstrated by these tests shall be cause for rejection of the material represented by the test samples.

DELIVERY, STORAGE, AND HANDLING

A. Welded wire reinforcement shall be shipped in bundles, tagged and marked in accordance with the Wire Reinforcement Institute's "Manual of Standard Practice".

B. Deliver welded wire reinforcement to project in undamaged condition. Lift welded wire reinforcement using a crane, forklift, or other lifting equipment.

1. Loading and unloading material with a crane requires lifting eyes. Lifting eyes are lengths of wire passing completely through to the underside of the bundle and brought back up to the top and twisted around 3 to 4 times to form an eye. Lifting eyes are located in the bundle to limit deflecting and bending in the center of the bundle. Typically there are four lifting eyes per bundle located along the outside edge of the sheet.

2. Loading and unloading material with a forklift requires the welded wire reinforcement to be supported by dunnage.

C. Store welded wire reinforcement in a protected area to limit the potential for injury and surface deterioration caused by prolonged exposure to conditions that accelerate the oxidation of steel.

D. Store welded wire reinforcement accessories to prevent corrosion and accumulation of dirt and oil.

E. Protect reinforcement, ties, and metal accessories form permanent distortion and store them off the ground.

MATERIAL

WELDED WIRE REINFORCEMENT:

A. Plain Welded Wire Reinforcement: ASTM A185, and as follows:

1. WWR Yield Strength: Provide minimum yield strength of 75,000 psi

2. Welded wire reinforcement shall be manufactured from domestic steel. No foreign steel or foreign billets used in manufacturing process will be permitted.

3. Wire Spacing and Size: Provide wire spacing and size, to maintain the specified area of steel as indicated on the contract documents.

4. Wire used in the manufacturing of welded wire reinforcement shall conform to ASTM A82.

5. Welded Wire Reinforcement shall be furnished in flat sheets or fabricated into bent sheets as indicated in the contract documents.

B. Deformed Welded Wire Reinforcement: ASTM A497, and as follows:

1. WWR Yield Strength: Provide minimum yield strength of 75,000 psi

2. Welded wire reinforcement shall be manufactured from domestic steel. No foreign steel or foreign billets used in manufacturing process will be permitted.

3. Wire Spacing and Size: Provide wire spacing and size, as indicated on the contract documents.

4. Wire used in the manufacturing of welded wire reinforcement shall conform to ASTM A496.

5. Welded Wire Reinforcement shall be furnished in flat sheets or fabricated into bent sheets as indicated in the contract documents.

SUPPORTS

A. Plastic or Wire Chairs, Bolsters, Bar Supports, Spacers: Sized and shaped for strength and support of reinforcement during concrete placement and construction loading conditions. Items shall conform to industry practice as described in the Wire Reinforcement Institute's, "Manual of Standard Practice" or "TF 702 – Supporting Welded Wire Reinforcement".

1. Metal bolsters or chairs which bear against the forms for exposed surfaces shall be equipped with snug fitting, high density, polyethylene tips which provide 1/2" minimum clearance between the metal and any exposed surface.

2. For epoxy-coated reinforcement, all wire/ bar supports and bar clips shall be epoxy or plastic coated.

3. For galvanized reinforcing, chair and bar supports shall be hot-dipped galvanized, after fabrication, in accordance with ASTM A123.

4. Spacing of slab bolster rows and high chair rows for deck slabs shall be as noted in Wire Reinforcement Institute's "Manual of Standard Practice" and "TF 702, Supporting Welded Wire Reinforcement" unless otherwise directed by the Engineer.

B. Concrete Blocks: Concrete blocks shall have a minimum bearing area of 2"X2" and equal in quality to the concrete specified. Concrete blocks can be used for supporting welded wire reinforcement in footings and slab-on-grades only.

OTHER REINFORCEMENT

A. Steel Reinforcing Bars: Reinforcing bars shall consist of deformed bars meeting the requirements of ASTM A615, Grade 60.

B. Tie Wire: Shall be 16-gauge, black soft-annealed wire conforming to ASTM A641

COATINGS

A. Galvanizing for welded wire reinforcement shall be in accordance with ASTM A525

B. Epoxy Coatings for welded wire reinforcement shall be in accordance with ASTM A884. Epoxy powders are electrostatically spray applied to a sandblasted near-white steel finish (fusion bonded epoxy resin). Ties, supports, and inserts used in conjunction with epoxy coated steel welded wire reinforcement shall be similarly coated.

CONSTRUCTION

EXAMINATION

A. Examine conditions for compliance with requirements for installation tolerances and other specific conditions, and other conditions affecting performance of welded wire reinforcement.

B. Examine rough-in and built-in construction to verify actual locations of concrete penetrations prior to installation.

C. Do not proceed until unsatisfactory conditions have been corrected.

PREPARATION

A. Clean all reinforcement by removing mud, oil, or other materials that will adversely affect or reduce bond at the time concrete is placed. Reinforcement with minor rust and/or mill scale will be accepted, provided the dimensions and weights, including heights or depth of deformations, are not less than required by the ASTM specification covering this reinforcement in this Specification.

B. Prior to placing welded wire reinforcement, remove laitance, loose materials, and anything that would prevent or hinder the installation of welded wire reinforcement.

INSTALLATION

A. General:

1. Place supports to secure welded wire reinforcement against displacement caused by construction loads or placing of concrete. Concrete blocks shall be used for supporting welded wire reinforcement in footings and slab-on-grades. For other concrete work, metal or plastic supports, hangers, or spacers maybe used. Layers of welded wire reinforcement shall be separated by chairs or bolsters. Stones, wood blocks, brick chips, etc., shall not be used to support reinforcement.

2. Place welded wire reinforcement to obtain the minimum coverage indicated in the construction documents.

3. Do not displace or damage vapor barrier, if one is required.

4. Welded wire reinforcement sheets shall have side lap and end laps as called for on the shopdrawings. Laps shall be calculated in accordance with ACI 318 code.

5. Tie any added rebar with annealed iron wire of not less than 16 gauge or suitable clips. Wire ties shall be cut back so that no metal is within minimum clearance required for the concrete.

CUT & REPAIR COATINGS

A. Field Cutting

1. Coated welded wire reinforcement shall not be field cut, unless permitted by the Engineer.

2. Field cutting of coated reinforcement should be performed using hydraulic-powered or friction cutting tools to minimize coating damage and field touch-up.

3. Flame cutting of coated reinforcement will not be permitted.

4. Field cut coated reinforcement shall be repaired immediately with compatible patching material and suitable for repairs in the field.

B. Epoxy Coating Repair

1. All visible damage (i.e., scratches, nicks, cracks) to the epoxy coating of the welded wire reinforcement, caused during shipment, storage or placement shall be repaired by the Contractor at the job site with approved patching material.

2. Ends of reinforcement that have been sheared sawed, or cut by other means shall be coated with approved patching material.

3. Patching of damaged areas shall be performed in accordance with the patching material manufacturer's recommendations.

4. Damaged surface area (prior to repair with approved patching material) shall not exceed 10% of the total sheet surface area, unless otherwise directed by the engineer. Should this limit be exceeded the sheet shall be removed and replaced with an acceptable sheet

5. Patching material shall be fully cured prior to placing concrete. The patching material shall be compatible with the epoxy coating, inert in concrete, and suitable for repairs in the field

C. Galvanized Coating Repair

1. All visible damage (i.e., scratches, nicks, cracks) to the galvanized coating of the welded wire reinforcement, caused during shipment, storage or placement shall

be repaired by the Contractor at the job site in accordance with appropriate ASTM specifications.

2. Ends of reinforcement that have been sheared, sawed, or cut by other means shall be coated.

3. Field coating of damaged areas shall be performed in accordance with the manufacturer's recommendations.

4. Zinc coating shall conform to contract specifications and shall be applied to achieve a dry film equal to or exceeding that designated in the contract documents. All touchup shall be cured fully prior to placing concrete.

APPLICABLE PROVISIONS FROM THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

For fully prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of effective prestress and permanent loads shall not exceed $0.40f'_c$ after losses.

The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads and prestress, and the Fatigue I load combination is tensile and exceeds $0.095\sqrt{f'_c}$.

5.5.3.2—Reinforcing Bars

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for straight reinforcement and welded wire reinforcement without a cross weld in the high-stress region shall be taken as:

$$(\Delta F)_{TH} = 24 - 0.33 f_{\min} \quad (5.5.3.2\text{-}1)$$

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for straight welded wire reinforcement with a cross weld in the high-stress region shall be taken as:

$$(\Delta F)_{TH} = 16 - 0.33 f_{\min} \quad (5.5.3.2\text{-}2)$$

where:

f_{min} = minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)

The definition of the high-stress region for application of Eqs. 5.5.3.2-1 and 5.5.3.2-2 for flexural reinforcement shall be taken as one-third of the span on each side of the section of maximum moment.

C5.5.3.2

Bends in primary reinforcement should be avoided in regions of high stress range.

Structural welded wire reinforcement has been increasingly used in bridge applications in recent years, especially as auxiliary reinforcement in bridge I- and box beams and as primary reinforcement in slabs. Design for shear has traditionally not included a fatigue check of the reinforcement as the member is expected to be uncracked under service conditions and the stress range in steel minimal. The stress range for steel bars has existed in previous editions. It is based on Hansen et al. (1976). The simplified form in this edition replaces the (r/h) parameter with the default value 0.3 recommended by Hansen et al. Inclusion of limits for WWR is based on recent studies by Hawkins et al. (1971, 1987) and Tadros et al. (2004).

Since the fatigue provisions were developed based primarily on ASTM A615 steel reinforcement, their applicability to other types of reinforcement is largely unknown. Consequently, a cautionary note is added to the Commentary.

5.5.3.3—Prestressing Tendons

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for prestressing tendons shall be taken as:

- 18.0 ksi for radii of curvature in excess of 30.0 ft, and
- 10.0 ksi for radii of curvature not exceeding 12.0 ft.

A linear interpolation may be used for radii between 12.0 and 30.0 ft.

C5.5.3.3

Where the radius of curvature is less than shown, or metal-to-metal fretting caused by prestressing tendons rubbing on hold-downs or deviations is apt to be a consideration, it will be necessary to consult the literature for more complete presentations that will allow the increased bending stress in the case of sharp curvature, or fretting, to be accounted for in the development of permissible fatigue stress ranges. Metal-to-metal fretting is not normally expected to be a concern in conventional pretensioned beams.

Transverse reinforcement shall be detailed such that the shear force between different elements or zones of a member are effectively transferred.

Torsional reinforcement shall consist of both transverse and longitudinal reinforcement. Longitudinal reinforcement shall consist of bars and/or tendons. Transverse reinforcement may consist of:

- Closed stirrups or closed ties, perpendicular to the longitudinal axis of the member, as specified in Article 5.11.2.6.4,
- A closed cage of welded wire reinforcement with transverse wires perpendicular to the longitudinal axis of the member, or
- Spirals or hoops.

5.8.2.7—Maximum Spacing of Transverse Reinforcement

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.125 f'_c$, then:

$$s_{max} = 0.8d_v \leq 24.0 \text{ in.} \qquad (5.8.2.7\text{-}1)$$

- If $v_u \geq 0.125 f'_c$, then:

$$s_{max} = 0.4d_v \leq 12.0 \text{ in.} \qquad (5.8.2.7\text{-}2)$$

where:

v_u = the shear stress calculated in accordance with 5.8.2.9 (ksi)

d_v = effective shear depth as defined in Article 5.8.2.9 (in.)

For segmental post-tensioned concrete box girder bridges, spacing of closed stirrups or closed ties required to resist shear effects due to torsional moments shall not exceed one-half of the shortest dimension of the cross-section, nor 12.0 in.

C5.8.2.7

Sections that are highly stressed in shear require more closely spaced reinforcement to provide crack control.

5.8.2.8—Design and Detailing Requirements

Transverse reinforcement shall be anchored at both ends in accordance with the provisions of Article 5.11.2.6. For composite flexural members, extension of beam shear reinforcement into the deck slab may be considered when determining if the development and anchorage provisions of Article 5.11.2.6 are satisfied.

C5.8.2.8

To be effective, the transverse reinforcement should be anchored at each end in a manner that minimizes slip. Fatigue of welded wire reinforcement is not a concern in prestressed members as long as the specially fabricated reinforcement is detailed to have welded joints only in the flanges where shear stress is low.

The design yield strength of nonprestressed transverse reinforcement shall be taken equal to the specified yield strength when the latter does not exceed 60.0 ksi. For nonprestressed transverse reinforcement with yield strength in excess of 60.0 ksi, the design yield strength shall be taken as the stress corresponding to a strain of 0.0035, but not to exceed 75.0 ksi. The design yield strength of prestressed transverse reinforcement shall be taken as the effective stress, after allowance for all prestress losses, plus 60.0 ksi, but not greater than f_{py}.

WWF YIELD STRENGTH IS MEASURED AT 0.0035 STRAIN, SO IS NOT AN ISSUE.

When welded wire reinforcement is used as transverse reinforcement, it shall be anchored at both ends in accordance with Article 5.11.2.6.3. No welded joints other than those required for anchorage shall be permitted.

Components of inclined flexural compression and/or flexural tension in variable depth members shall be considered when calculating shear resistance.

Some of the provisions of Article 5.8.3 are based on the assumption that the strain in the transverse reinforcement has to attain a value of 0.002 to develop its yield strength. For prestressed tendons, it is the additional strain required to increase the stress above the effective stress caused by the prestress that is of concern. Limiting the design yield strength of nonprestressed transverse reinforcement to 75.0 ksi or a stress corresponding to a strain of 0.0035 provides control of crack widths at service limit state. For reinforcement without a well-defined yield point, the yield strength is determined at a strain of 0.0035 at strength limit state. Research by Griezic (1994), Ma (2000), and Bruce (2003) has indicated that the performance of higher strength steels as shear reinforcement has been satisfactory. Use of relatively small diameter deformed welded wire reinforcement at relatively small spacing, compared to individually field tied reinforcing bars results in improved quality control and improved member performance in service.

The components in the direction of the applied shear of inclined flexural compression and inclined flexural tension can be accounted for in the same manner as the component of the longitudinal prestressing force, V_p.

5.8.2.9—Shear Stress on Concrete

The shear stress on the concrete shall be determined as:

$$v_u = \frac{\left|V_u - \phi V_p\right|}{\phi b_v d_v} \qquad (5.8.2.9\text{-}1)$$

where:

ϕ = resistance factor for shear specified in Article 5.5.4.2

b_v = effective web width taken as the minimum web width, measured parallel to the neutral axis, between the resultants of the tensile and compressive forces due to flexure, or for circular sections, the diameter of the section, modified for the presence of ducts where applicable (in.)

d_v = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of 0.9 d_e or 0.72h (in.)

C5.8.2.9

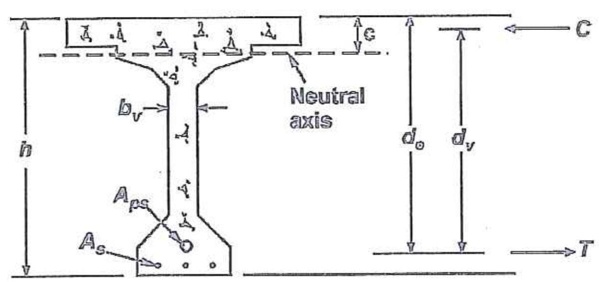


Figure C5.8.2.9-1—Illustration of the Terms b_v and d_v

For flexural members, the distance between the resultants of the tensile and compressive forces due to flexure can be determined as:

$$d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}} \qquad (C5.8.2.9\text{-}1)$$

In continuous members near the point of inflection, if Eq. C5.8.2.9-1 is used, it should be evaluated in terms of both the top and the bottom reinforcement. Note that other limitations on the value of d_v to be used are specified and that d_v is the value at the section at which shear is being investigated.

- For No. 6, No. 7 and No. 8 stirrups with f_y greater than 40.0 ksi:

  A standard stirrup hook around a longitudinal bar, plus one embedment length between midheight of the member and the outside end of the hook, ℓ_e shall satisfy:

$$\ell_e \geq \frac{0.44 d_b f_y}{\sqrt{f'_c}} \qquad (5.11.2.6.2\text{-}1)$$

5.11.2.6.3—Anchorage of Wire Fabric Reinforcement

C5.11.2.6.3

Each leg of welded plain wire fabric forming simple U-stirrups shall be anchored by:

- Two longitudinal wires spaced at 2.0 in. along the member at the top of the U, or
- One longitudinal wire located not more than $d/4$ from the compression face and a second wire closer to the compression face and spaced not less than 2.0 in. from the first wire. The second wire may be located on the stirrup leg beyond a bend or on a bend with an inside diameter of bend not less than $8d_b$.

For each end of a single-leg stirrup of welded plain or deformed wire fabric, two longitudinal wires at a minimum spacing of 2.0 in. and with the inner wire at not less than $d/4$ or 2.0 in. from middepth of member shall be provided. The outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

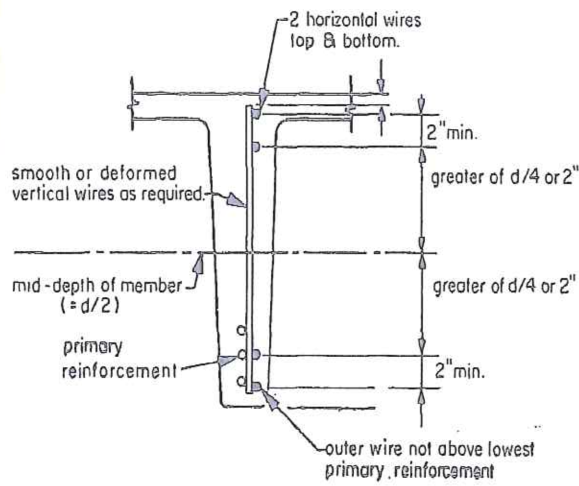


Figure C5.11.2.6.3-1—Anchorage of Single-Leg Welded Wire Fabric Shear Reinforcement, ACI (1989)

5.11.2.6.4—Closed Stirrups

Pairs of U-stirrups or ties that are placed to form a closed unit shall be considered properly anchored and spliced where length of laps are not less than 1.7 ℓ_d, where ℓ_d in this case is the development length for bars in tension.

In members not less than 18.0 in. deep, closed stirrup splices with the tension force resulting from factored loads, $A_b f_y$, not exceeding 9.0 kips per leg, may be considered adequate if the stirrup legs extend the full available depth of the member.