10.1.1 GENERAL

In addition to AASHTO Standard Specifications for Highway Bridges, with current interims, the following references are to be used when applicable for the design of steel highway bridges:

- AASHTO Guide Specification for Fracture Critical Non-redundant Steel Bridge Members.
- AASHTO Guide Specification for Horizontally Curved Highway Bridges.
- ANSI/AASHTO/AWS D1.5 Bridge Welding Code.
- AASHTO Standard Specifications for Seismic Design of Highway Bridges.

Structural steel railroad bridges shall be designed in accordance with the current AREA Specifications.

The 509 Special Provisions shall be reviewed by CDOT Staff Materials on jobs with Fracture Critical Members, jobs requiring unusual fabrication or materials, and on jobs utilizing existing structural steel. Additionally, on jobs utilizing existing steel, the District should be notified early in the project to determine if the existing paint contains hazardous materials and what associated Project Special Provisions will be required.

All girders shall be designed to be fully composite with the deck. Longitudinal reinforcing steel in the top mat, within the effective deck width, shall be used when calculating section

C1: Generally, the reinforcing steel stress limitation is an issue for shored girders. The 27 ksi was originally chosen to be consistent with the probable allowable tensile stress in the girder. It has been suggested that 24 ksi should be used to be consistent with the Working Stress Design reinforced concrete allowables. This could excessively penalize the maximum stress in grade 50 top flanges. Another suggestion was to use .55(60) ksi for grade 60 reinforcing steel.

Using reinforced concrete Load Factor Design criteria, the serviceability requirements control for common dead to live load ratios with a crack control allowable stress of 29 ksi (for #11's at 6" spacing and 2" cover -- note, a revision to 2" maximum cover for this calculation by AASHTO is anticipated) and an allowable fatigue stress range of 20 ksi. These results indicate that the 27 ksi should result in adequate strength, serviceability, and economy. Designers may use lower values where they feel necessary.

C2: In general, for primary and secondary members and member components, rolled shapes have lower fabrication costs and better fatigue characteristics than customized welded plate and bent plate members. Additionally, they generally do not require as much quality control inspection as fabricated shapes do. Consequently, where rolled shapes are otherwise sufficiently practical and economical, they are preferable to fabricated shapes.

properties in negative moment regions. The stress in the deck reinforcing steel shall not exceed 27,000 psi. (C1)

Steel girders shall be made of rolled beams or welded plates. (C2)

Occasionally bent plates may be needed for attachments, connections, or secondary members. The AASHTO Standard Construction Specifications, and CDOT Standard Specifications, specify that plates may only be bent about an axis that is perpendicular to the direction of the plates' mill rolling. The designer shall consider the consequences of this requirement when using bent plates. (C3)

Uplift at supports and girder stresses due to the deck pouring sequence shall be considered during design. For additional requirements regarding bridge decks, see CDOT Bridge Design Manual Subsection 8.2.

10.1.2 MATERIALS

Generally, ASTM A36 should be used for members and components where a higher yield strength steel would not appreciably reduce the required sections. ASTM A572 Grade 50 should generally be used for girder webs and flanges. ASTM A588 shall be used for weathering steel applications and shall be used in place of A572 for plates 3" and greater in thickness. Where A572 is used, the plans should allow A588 to be substituted for A572 at no additional cost to the project. (C4)

However, for girder members, welded shapes generally are the optimum solution for most of our steel girder applications.

C3: Bending plates parallel to the primary direction of rolling can introduce cracks along the outside of the bend, and is therefore disallowed. However, bending normal to the rolling can significantly effect the economy of long bent plate members. For example, a 10 foot long bent plate bracing member would need to be cut from a 10 foot wide plate, or cut from smaller width plates and spliced to obtain the necessary length. Also, this normal bending can result, depending on the member orientation, in the member's primary working stresses acting perpendicular to the rolling.

C4: In most cases ASTM A36 is less expensive than ASTM A572 Grade 50, and ASTM A588 is more expensive than A572. However, the toughness characteristics of A572 steel plates thicker than 2" can be unreliable. Consequently, in order to meet AASHTO welding and toughness requirements, A572 can be more expensive than A588 for these plates. This is especially true of fracture critical members where A572 plates over 1" or 1.5" may be more expensive than A588. The 3" requirement here, a thickness where the distinction between costs is more clear, is from the Staff Bridge Engineer's 1/24/91 Technical Memorandum #2. Permitting A588 to be substituted for A572 in the plans allows the fabricator toselect the least expensive and most convenient material.

Bracing, stiffeners, and secondary members are examples of where yield strength oftentimes has a

Weathering steel may not be used unless approved by the CDOT Staff Bridge Engineer. Requests to use weathering steel need to be made early in the project. (C5)

Material in tension in primary members (referred to as "main members" by CDOT Standard Specifications) shall meet the longitudinal Charpy V-notch impact test requirements. Either the plans, Project Special Provisions, or Standard Specifications shall designate the structural steel "main members" and the tensile portions of these members. Fracture Critical Members shall be clearly identified on the plans. The plans shall also show the limits of tension flanges.

10.1.3 COVER PLATES

Cover plates shall not be used for new construction. Larger rolled beams or welded plate girders shall be used in lieu of cover plates. This is to avoid potential fatigue problems at cover plate termini.

minimal effect on the required sections, because stiffness and stability usually control their design. In which case, A36 should be used. A572 is commonly used for box girder interior pier and abutment diaphragms, and is occasionally needed for bearing stiffeners. Longitudinal flange stiffeners should satisfy allowable bending requirements. Consequently these stiffeners are usually made the same grade of steel as the flange. Although using A36 webs with A572 flanges can provide greater economy on some girders, this Subsection currently disallows hybrid girders. Therefore, A572 webs, matching the flanges, are used.

Note, Grand Junction Steel has found using "bars" (see AISC Manual of Steel Construction for definitions of "bar" and "plate") for stiffeners is usually less expensive than cutting them from plates. Therefore, calling for A572 stiffeners because they use the same size plate as the girder web or flanges will probably increase, instead of decrease, cost and inconvenience. This is probably true of other plate members or components where "bar" could be used.

Designers should keep in mind that small quantities of a given A572 and A588 rolled shape can be very expensive. For example, on a weathering steel bridge, reducing quantities by using several different sizes of A588 bracing members may actually increase costs. Although minimizing the number of different parts is an important rule for structural design in general, it deserves additional attention here.

10.1.4 WELDED GIRDERS

When designing structural steel elements, conservation of material shall not receive unwarranted emphasis. Simplification of details, reduction of fabricating operations, and ease of erection are often the best means for achieving minimum cost and maximum quality. Changes in plate sizes and the use of stiffeners should be avoided unless the savings in material is significant enough to offset the increased fabrication costs. (C6)

The minimum web plate thickness shall be 3/8 inch. The minimum flange thickness shall be 5/8 inch. The minimum flange width, except box girder bottom flanges, shall be 12 inches. For handling efficiency, the b/t ratio for tension flanges, except box girder bottom flanges, should not exceed 24. For steel box girders, the b/t ratio for the bottom flange in tension shall not exceed 120. Before using plates greater than 8 feet wide, the designer shall check their availability and the costs associated with their use. (C7)

C5: Weathering steel is not typically used in Colorado. Experiences with areas of adjacent concrete becoming stained and with uneven rusting giving non-uniform coloration and texture, as well as concerns about the potential for progressive deterioration in areas of continual moisture and/or high salt exposure, have led to its use being discouraged in the past.

C6: Less material represents economy. But, minimizing the number of stiffeners, different rolled members, and different plate thicknesses does too. Overall savings is achieved with a balance between the two, keeping in mind that as a percentage of total costs, labor costs can readily exceed material costs.

A change in flange plate size that introduces a welded splice should save 700 pounds, or [300+25(flange area)] pounds to be cost effective (per a Bethlehem and USS publication, respectively). These are older guidelines. Higher values may now be appropriate.

For bridges with typical girder lines, the cost of welded flange plate splices can be reduced when the two flanges at the splice are the same width. This allows the weld to be completed before the flange plates are cut. However, this can work contrary to minimizing the number of different plate thicknesses. Again, a balance must be found.

C7: Staff Bridge has historically established minimum plate sizes to help insure efficient handling, and to provide the boundary below which rolled shapes should be used to obtain an assumed highest quality for the least cost. However, given the subsequent prohibition of cover plates, and

POLICY	COMMENTARY

On box girders, the preferred distance from exterior face of web to edge of bottom flange is 1.25". (C8)

The web and flanges of a welded girder shall be of the same grade of steel; i.e., hybrid girders may not be used.

10.1.5 FATIGUE

Except for bridges on interstate and primary highways, fatigue design shall be based on the 20 year projected ADTT as derived from the final Form 463 or as reported by Staff Traffic. (C9)

the difficulty in splicing rolled shapes of different sizes, these restrictions now make efficient utilization of material more difficult for continuous steel girder bridges in the smaller span lengths. If these restrictions excessively affect the cost of a project, alternative solutions may be submitted to the Staff Bridge Engineer for approval.

The b/t limit of 120 was taken from the FHWA Report Number FHWA-TS-80-205, Proposed Design Specifications for Steel Box Girder Bridges, January 1980, by Wolchek and Mayrbaurl Consulting Engineers.

Previously, plate widths exceeding 8’ were prohibited by this Subsection. This was changed because wider plates are available from some steel mills. However, their availability in the length and thickness desired, the plate cost, and shipping costs, need to be determined and considered by the designer. By using longitudinal welded splices, girder webs deeper than 8’ have been used. However, the cost of making this splice, and the costs associated with using a girder over 8’deep, need to be considered.

C8: This distance has been requested (and verified on 10/91) by Grand Junction Steel to provide the necessary riding surface for their welding machine.

C9: This paragraph assumes use of the AASHTO Standard Specifications for fatigue design. The AASHTO Guide Specifications for Fatigue Design of Steel Bridges offers several alternative for determining design truck volumes, but these alternatives are for when the guide specification is used.

POLICY	COMMENTARY

Fatigue design for all bridges on interstate and primary highways shall be based on the Case I stress cycles in the AASHTO Standard Specifications. (C10)

Non-redundant members are defined as members whose failure would be expected to result in collapse of the structure.

10.1.6 STIFFENERS

Transverse (vertical) web stiffeners and longitudinal web and flange stiffeners shall be 5/16 inch minimum thickness and shall be welded to the girder with a minimum 1/4 inch continuous fillet weld.

Longitudinal web stiffeners shall not be used, except for girder spans exceeding 165 feet between points of zero dead load moment. (C11)

Transverse stiffeners shall be normal to the top flange and placed on the non-visible side (inside) of exterior girders. The minimum spacing for the first transverse stiffener from the centerline of bearing shall be equal to one-half the depth of the web. The preferred minimum spacing at all other locations is equal to the depth of web. For longitudinally stiffened girders, use the maximum sub-panel depth, instead of the total web depth, in determining these minimum spacings. (C12)

C10: Bridge designers need to be thorough when considering fatigue. Under normal loading conditions, fatigue failure in steel girders is apparently more common than failure due to member load capacity. Unfortunately, the consequences of current fatigue design procedures will not be known for many years, well into the fatigue design life. Taking this into consideration, it was decided to conservatively use Case I fatigue for all interstate and primary highway bridges. In order to monitor the consequences of this requirement, projects where it has a heavy influence on costs should be reported to the Staff Bridge Engineer.

C11: The previous version had a 300 foot span (center to center of bearing) limitation. The current 165 feet between points of zero moment translates to 165 foot simple spans (c/c bearing) and approximately 300 foot interior spans of multi-span continuous girder bridges. It would be preferable to make the stiffeners a function of percent of total material weight saved instead of span length. Or to provide a weighted cost factor for stiffeners. However, until this matter is pursued further, the existing requirement will be used.

C12: Theses limitations on transverse stiffeners pacings, along with the preceding limitations on longitudinal stiffeners, mandate the use of fewer stiffeners and thicker webs. The intent is to establish a practice of pursuing economy by simplifying and reducing fabrication rather than just reducing the total weight of structural steel used. The quality of fabrication is also positively influenced by increased simplicity.

Shop splices of stiffeners, if any, shall be made with full penetration groove welds. These welds shall be completed before the stiffeners are welded to the girder. (C13)

Rectangular sections are preferred over T-sections for bottom flange longitudinal stiffeners. To facilitate welding operations during fabrication the minimum clear distance between the longitudinal stiffener and girder web, or between adjacent longitudinal stiffeners, should preferably be 2’-4". (C14)

To facilitate fabrication, when T-sections are used for bottom flange longitudinal stiffeners, the ratio of the stiffener depth to one-half the stiffener flange width should be greater than or equal to 1.7. (C15)

10.1.7 BEARING STIFFENERS

Bearing stiffeners shall be placed with a tight fit against the top flange, or be connected to it by fillet welds. When the top flange is in tension, the tight fit is preferred. When the stiffener is used to connect a diaphragm, the fillet welded, or to flange connection is required.

Where this intent is otherwise satisfied, stiffener spacings less than the depth of web may be used where required for coordination with diaphragm spacing details. This is often needed on heavily curved or skewed I-girder bridges which have tight and inflexible diaphragm spacings.

Spacing stiffeners at one-half the web depth from the centerline of bearings is allowed to give greater flexibility in these high shear areas. This allowance also accommodates the current AASHTO curved girder guide specification requirement for the end of girder stiffener.

C13: CDOT has had problems getting full penetration welds and good workmanship at longitudinal stiffener splices. These welds are often not adequately addressed by the plans or the specifications. The design engineer is to ensure that they are. This applies to longitudinal web and flange stiffeners. It also applies to transverse web stiffeners, although it is unlikely they would require splicing.

C14: Welded and bolted splices are more difficult to make on T-sections than on rectangular sections. The cost of cutting and straightening a W-shape to make a WT-shape can readily exceed the costs of using a rectangular section of "bar" stock or of cut and straightened "plate".

The 2’-4’ is based on requests made by Grand Junction Steel.

C15: This ratio ensures good access to the stiffener web and to the stiffener to girder weld.

Bearing stiffeners shall be ground to bear against the bottom flange. When used to connect a diaphragm, the stiffener shall be fillet welded to the bottom flange after grinding to bear. Or, in all cases, the stiffener may be attached to the bottom flange with a full penetration groove weld. However, to prevent their potential warping effect on bottom flanges, the full penetration welds are discouraged. (C16)

The angle between bearing stiffeners and the web shall not be less than 60 degrees. Where necessary to connect diaphragms at larger skews, bent plates shall be used. Plates separate from the bearing stiffeners may be used to connect diaphragms, but the 60 degree limitation also applies to these plates. (C17)

When the final grade along centerline of girder is less than 2%, bearing stiffeners may be set perpendicular to the flanges. For 2% grades and larger, bearing stiffeners shall be set plumb. (C18)

10.1.8 SPLICES

All splices shall be normal to the top flange and normal to the longitudinal axis of the girder. Field splices shall preferably be located at or near the points of dead load contraflexure.

The preferred maximum length between field splices is 100feet for steel girders. Difficult haul routes and/or limited access to the bridge site may require reducing this length. Piece weights for handling during construction should also be considered when locating splices. (C19)

C16: There have been problems with warping in the bottom flange of box girders. Welds to the bottom flange, especially large welds near the center of a box girder flange, can contribute to, or cause, this warping. Although this experience has been with box girders, placing these large welds across I-girder flanges is similarly discouraged.

C17: For stiffness, bearing stiffeners are most efficient when placed perpendicular to the web. However, when connecting diaphragms, or obtaining the optimum orientation to a bearing device, it may be desirable to skew them. The maximum skew is limited by the AWS requirements for fillet welds. Welds at angles less than 60 degrees (the angle between the web and the stiffener) qualify as partial penetration groove welds and they are not be used where there may be tension perpendicular to the weld length. Note, this applies to all fillet welded t-joints, and not just those at stiffeners.

This limitation also ensures adequate access to the weld. However, the designer should watch for other obstacles to access, for example, adjacent stiffeners or diaphragm connection plates.

When placing stiffeners on skews the designer also needs to remember to calculate the required moment of inertia along the skewed girder’s axis, and not an axis perpendicular to the stiffener.

C18: Placing bearing stiffeners normal to flanges can sometimes simplify fabrication. CDOT has used bearing stiffeners up to 2% out of plumb in the past. This practice constitutes the current policy.

To facilitate fabrication, where filler plates are used in bolted splices, a note shall be added to the plans permitting the use of oversized holes in the filler plates. The applicable diameter, from AASHTO, shall be given in the note. (C20)

Flange thickness transition ratios shall not exceed 2:1 at welded splices.

The full penetration welds at girder splices shall not be made with backing. The plans shall use the following weld symbol for these connections.

Missing figure

The designer shall review the shop drawings to ensure that full and complete weld details are shown, and that the welds selected by the fabricator are acceptable. (C21)

10.1.9 CONNECTIONS

Generally, all shop connections shall be welded and all field connections shall be made with high strength bolts. Shop bolted connections should be used when welding would cause difficulty with fabrication or fatigue.

All full penetration welds shall be ground flush for testing. Ultrasonic testing shall be performed on full penetration welds in accordance with the frequency established in the construction Standard Specifications.

C19: Previously, 100 feet was the maximum length allowed by this Subsection. Since then several steel girder bridges have had shipping lengths between 100 and 122 feet. But these lengths represent maximums which may not be practical or economical on other projects. Note, precast concrete girders up to 150 feet long have been used in the state. But again, as maximums, these lengths are not possible on all shipping routes.

Grand Junction Steel has indicated that both railroad and highway shipping costs can jump higher at lengths greater than about 85’. For instance, they found that in some cases it cost them less to make welded splices than to order plates greater than 85’ long.

C20: The procedure used to drill a stack of splice plates by several fabricators requires the splice filler plates to be drilled separately from the splice plates. This can lead to fit-up problems if tolerance on the filler plate hole is not provided. This policy is taken from a 5/16/91 memorandum from the Staff Bridge Engineer.

C21: Full penetration welds made with backing have a relatively high repair rate. The repairs are necessary to eliminate cracks which result from a fusion type of defect between the backing and the base metal. The crack continues to propagate as subsequent weld passes are made. Using the weld symbol shown allows the fabricator to select the full penetration weld details which best suit the associated plate sizes and his means and methods of fabrication. This policy is from the Staff Bridge Engineer’s 8/7/91 Technical Memorandum #9.

POLICY	COMMENTARY

Slip-critical connections shall be made with 3/4" or 7/8" diameter ASTM A325 bolts using Class A friction surfaces. Where special consideration is necessary, requests to use 1" diameter bolts or Class B friction surfaces may be submitted to the Staff Bridge Engineer for approval. (C22)

When Class B friction surfaces are used, the plans shall specify the connection surface conditions that must be present at the time of bolting.

Fastener spacing and edge distances shall satisfy the requirements for bearing capacity, mill and fabrication tolerances, bolt entering and tightening clearances, and AASHTO minimum spacing and edge distance criteria. (C23)

The minimum clearance for entering and tightening high strength bolts shall be determined from the AISC Manual of Steel Constriction. Special evaluation will be required for non-orthogonal planes which are not covered by the AISC manual. The overall dimensions of the bolting gun, and the length of tensile control control bolts with their break-off tips attached, need to be considered for non-orthogonal planes and other obstructions.

The designer should assume that tensile control bolts, assembled with a large installation tool, will be used. Where clearances will not allow this, locations where tensile control bolts cannot be used shall be clearly noted in the plans. Tensile control bolts must be used with unpainted A588 steel. (C24)

10.1.10 SHEAR STUDS

The plans shall specify the stud length and diameter used in design. To provide for construction tolerances in the

C22: ASTM A490 bolts are excluded due to potential problems with ductility and obtaining proper tension. These concerns are based on the May 1987 FHWA/R8-87/088 report, High Strength Bolts for Bridges, by the University of Texas at Austin. The construction specifications for structural steel connections submitted by the FHWA, and adopted by CDOT in 1989, similarly exclude A490 bolts.

To facilitate fabrication and construction, CDOT prefers the most commonly used high strength bolt diameters.

The policy on Class A friction surfaces is from the Staff Bridge Engineer’s 5/22/90 Policy Letter #4. This letter reported that out of 15 states surveyed, all responded that they did not routinely use Class B slip critical connections.

C23: The minimum spacings and edge distances given by the AASHTO Standard Specifications are currently being interpreted by CDOT as absolute minimums with no tolerance permitted. Rather than calculating the actual total mill and fabrication tolerances needed (which can be found in the CDOT and AASHTO Standard Specifications for Construction, the AISC Manual of Steel Construction, and AWS D1.5), Grand Junction Steel has recommended calling out 3" spacing and 1.75" (2" preferably) edge distance in the plans for 7/8" diameter bolts. To date, this recommendation has been widely accepted and used.

haunch depth, the minimum allowable cover from top of stud to top of deck, and from top of stud to bottom of deck, shall also be given. This cover shall not be less than the amount specified by AASHTO, and shall notbe less than the specified coveron the deck reinforcing steel.

10.1.11 CONTROL DIMENSION

The control dimension "Y" shall be measured from the top of the girder web to the top of the concrete deck (see attached sketch).

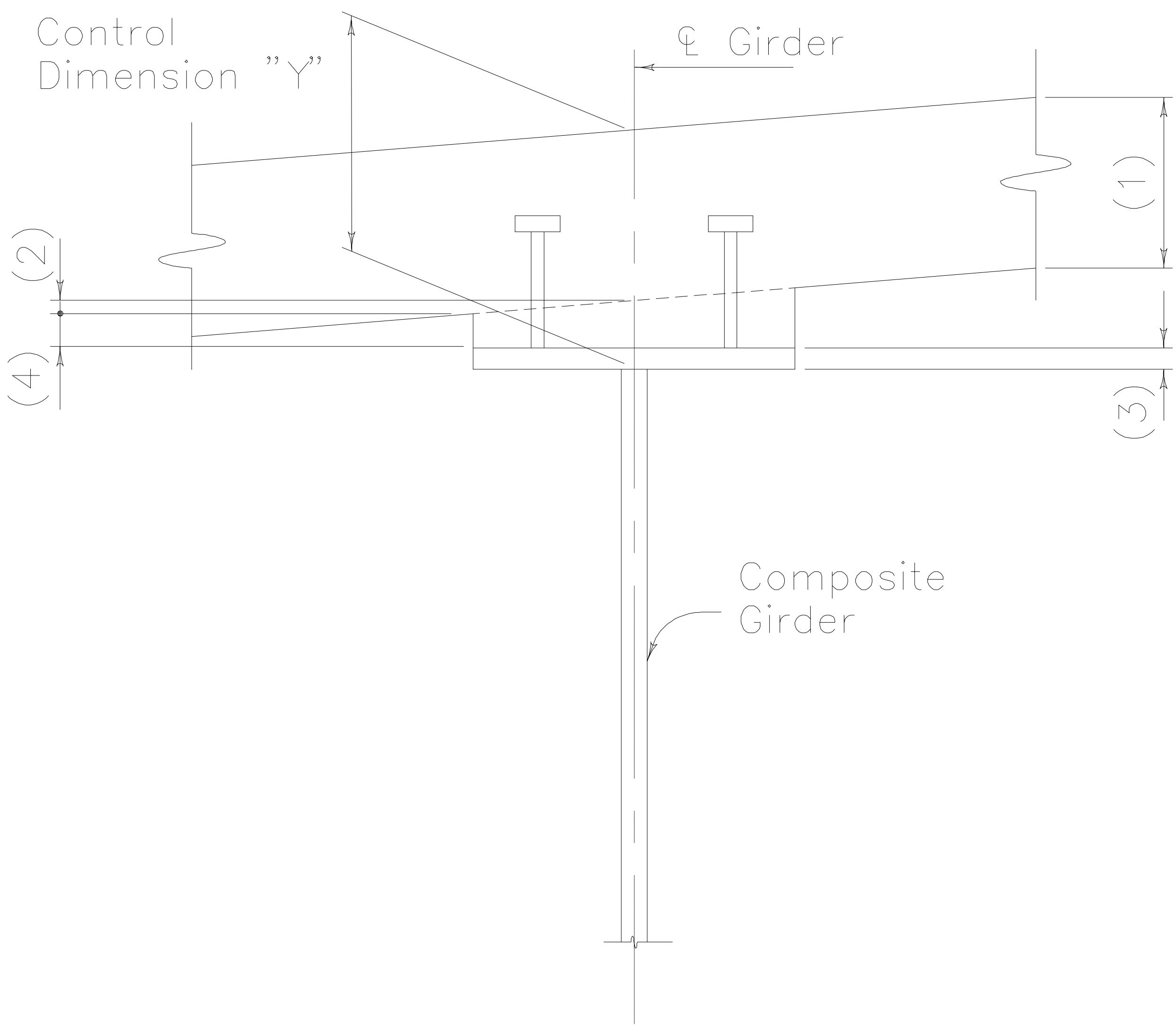
To calculate the dimension "Y", add together the following 4 factors:

1. Minimum design deck thickness.
2. Correction for roadway slope = 1/2 maximum flange width times roadway cross slope. This is not required for box girders placed parallel to the cross slope of the deck.
3. Maximum top flange thickness.
4. Excess haunch to allow for fabricating tolerance in girder camber; allow 1 inch for spans 100 feet or less. Allow 1-1/2 inch minimum for spans over 100 feet.

In multiple span structures, dimension "Y" should be constant. Item 4 may be increased as necessary to achieve this. Dimension "Y" should be shown on the Typical Section and designated at the distance from the top of the deck to the top of we at the centerline of the girder and at the centerline of bearing. The concrete portion of the haunch shall not be used to determine section properties for analyzing composite sections except in unusual cases where the haunch, including flange thickness, exceeds 4".

C24: Using high strength tensile control bolts has become standard practice with contractors. Contractors will usually assume they can use tensile control bolts unless directed otherwise. Therefore, designers need to note in the plans bolt locations where, due to clearances, tensile control bolts probably cannot be used.

Uncoated rust resistant load indicating washers are not available, and CDOT has not approved the use of coated washers. The coating can be scraped off during tightening. Therefore, for direct tension indication, only tension control bolts may be used with unpainted A588 steel.



Control
Dimension "Y"
℄ Girder
(1)
(2)
(3)
(4)
Composite
Girder

GENERAL

Cross frames and lateral bracing shall normally be composed of rolled angles, structural tees, or channels and not built up sections or bent plates. The smallest angle used in bracing shall be 3" by 2-1/2" by 5/16". (C1)

There shall not be less than 2 fasteners, or the equivalent weld, at each end connection of the bracing elements. Field connections shall be made by bolting. To facilitate fabrication and erection, oversized holes in gusset plates for diaphragm and lateral bracing connections are preferred. This is a minimum. Skew, curvature, or other considerations may require larger tolerances.

All gussets and connection plates shall be 3/8 inch minimum thickness.

Intermediate diaphragms and lateral bracing shall be ASTM A36 steel except as otherwise approved by the Staff Bridge Preconstruction Engineer.

DIAPHRAGMS

Unless noted otherwise, "diaphragms" is used by this Subsection to refer to both beam-type and truss-type transverse bracing for girders. AASHTO refers to these as diaphragms and cross frames, respectively.

Diaphragms for curved I-girders shall be designed as main members when the central angle due to curvature exceeds the limits of Table 1.4A, "Limiting Central Angle for Neglecting Curvature in Determining Moments", of the AASHTO

C1: For internal diaphragms of large box girders, larger minimum angles may be appropriate to improve handling.
4" by 4" by 3/8" minimum angle has been suggested.

C2: The guide specification states that diaphragms are to be "designed as main structural elements to distribute torsional forces to the longitudinal girders." To make the diaphragm design criteria consistent with the criteria for girders, table 1.4A is used to determine when these distributed torsional forces are negligible.

POLICY	COMMENTARY

Guide Specification for Horizontally Curved Highway Bridges. All other intermediate diaphragms usually need to be designed for kL/r requirements only. (C2)

It is preferable to place intermediate diaphragms perpendicular to the girders (radially to curved girders). (C3)

It shall be noted in the plans when the intermediate diaphragms between two adjacent girders need to be different lengths for proper fit-up. Preferably, the distances between workpoints for each diaphragm should be given. (C4)

The diaphragms at the ends of girders should preferably be placed near and parallel to the centerline of bearing, and set parallel to and 1'- 0" below the top of deck. The slab shall be haunched down and supported by the diaphragm, and connected to it with shear connectors. In lieu of these requirements, when girder ends are cast in concrete, provide minimal bracing to restrict girder movement during concrete placement, and to accommodate other loads that may be encountered during construction.

When girder to substructure skews are greater than 20 degrees, gusset plates for intermediate diaphragms (except those inside boxes) shall, as a minimum, have short slotted holes to allow for differential deflection. The following note shall be added to the plans:

> Holes in gusset plates to be slotted vertically 1-1/8" x 15/16" for 7/8" diameter H.S. Bolts.

Use 1" x 13/16" for 3/4" diameter high strength bolts in the above note. This is a minimum requirement.

C3: Previously, skewed intermediate diaphragms were prohibited by this Subsection. The current writing allows skewed diaphragms in deference to the AASHTO allowance for intermediate diaphragms skewed up to 20 degrees.

It appears that the primary reason for prohibiting skewed diaphragms in the past was to alleviate fabrication difficulties. Namely, having to skew diaphragm connection plates to the web, and having to fabricate diaphragms of different lengths when the bridge is on a vertical curve. The latter concern is now addressed separately in this Subsection.

Skewed diaphragms provide some degree of restraint to girder rotation about the girder's primary bending axis. This restraint should be kept in mind by designers, especially when skewed diaphragms are used with torsionally rigid girders.

C4: In many instances changes in superelevation, or not having girders parallel to the horizontal control line, can cause the length of intermediate diaphragms to vary. Changes in grade (e.g. when bridge is on a vertical curve) can have the same effect on skewed intermediate diaphragms. In areas of superelevation transition, steel box girders should be made non-parallel to the horizontal control line, as necessary, to obtain typical diaphragm dimensions. CDOT's Bridge Geometry program can compute this varying offset due to superelevation transition.

C5: Previously this Subsection disallowed using transverse stiffeners, that were otherwise required for girder shear, and bearing stiffeners to connect diaphragms. In most applications of full depth diaphragms, the

POLICY

For all bridges, especially when there is skew or horizontal curvature, actual differential deflection should be investigated, and the corresponding requirements for diaphragm fit-up satisfied. Vertical connection plates for connecting intermediate diaphragms to webs shall be rigidly connected to the top and bottom flanges. This may be done by shop welding, or where economical due to fatigue considerations, by bolting. (C5)

ADDITIONAL REQUIREMENTS FOR BOX GIRDERS

In order to avoid problems during construction and erection, and to maintain geometric integrity, lateral bracing and cross frames shall be provided within steel box girders.

Single laced lateral bracing is preferred. Lateral bracing shall be located at, or as near as practical to, the top flange. The connection of lateral bracing to the girder web, or flange, shall be made by bolting. (C6)

The lateral bracing equations for equivalent plate thickness and required stiffness from the AASHTO Guide Specifications for Horizontally Curved Highway Bridges shall not be used. The Kollbrunner Basler equations shall be used for determining equivalent plate thickness. The required area and radius of gyration for bracing members shall be computed using standard analytical methods. (C7)

Temporary external diaphragms between boxes will be required at every other internal intermediate cross frame. When the radius of curvature, R, is less than 1000 feet, temporary external diaphragms shall be provided at every internal cross frame. (C8)

COMMENTARY

stiffeners should adequately handle the dual functions of transmitting diaphragm loads to the girder, and stiffening the web. However, the effect of this dual usage should be considered when designing the stiffeners, especially when partial depth diaphragms are used. When desired, separate connection plates for the diaphragms may be used.

C6: The closer lateral bracing is to the top flange, the more efficient it is. However, the clearance needed for forming the deck must be provided. If lateral bracing is connected to the top flange, precast panel deck forms or steel stay-in-place deck forms may be required. If they are, it shall be noted, and adequate haunch depth provided, in the plans.

C7: The lateral bracing equations for equivalent thickness and required stiffness in the current version of the AASHTO curved girder guide specification appear to be in error and therefore may not be used. The Kollbrunner and Basler equations can be found in the 1976 FHWA Curved Girder Workshop manual, and in the 1979 textbook, "Design of Modern Steel Highway Bridges" by Heins and Firmage. The FHWA workshop manual provides an example of computing the required lateral bracing section properties.

C8: These temporary frames serve to unify the overall action of the steel box girders during deck pouring while also providing additional restraint for temperature effects.

The 1000 foot radius requirement was added in the January 1988 edition of this Subsection.

This value was taken from the AASHTO Guide Specifications for

POLICY	COMMENTARY

Box girders 5 feet and greater in depth shall be made fully accessible for interior inspection. Refer to CDOT Bridge Design Manual Subsection 2.7, Access for Inspection, for additional requirements.

Horizontally Curved Highway Bridges. The specification's impact requirements were only applicable when the radius was less than 1000 feet. Until a resource more directly applicable to diaphragm loads is found, the existing value should be used as a minimum requirement.

STRUCTURAL STEEL FRACTURE CRITICAL MEMBERS

POLICY

Fracture critical members or member components (FCMs) are tension members or tension components of members whose failure would be expected to result in collapse of the bridge. (C1)

The responsibility for determining which, if any, bridge member or member component is in the FCM category shall rest with the bridge design engineer. (C1)

The bridge design engineer shall evaluate each bridge design to determine the location of any FCMs that may exist. The location of all FCMs shall be clearly delineated on the contract plans. The bridge design engineer shall review the shop drawings to assure that they show the location and extent of FCMs. (C1)

The bridge design notes shall contain the supporting calculations and evaluations as to which members are designated FCMs and why they are so designated. (C2)

On all projects with FCMs, the contract documents shall contain a Fracture Control Plan (FCP). This plan may be provided directly by the 1991 CDOT Standard Specifications, or by reference to the AASHTO specifications (AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members) in a project special provision. The CDOT Staff Materials Branch shall be consulted as to which method to use. The final specifications and special provisions selected shall be discussed with Staff Materials. (C3)

COMMENTARY

C1: These paragraphs are taken from Articles 2 and 3 of the AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members. This is not a design specification, but a construction specification for the fabrication of steel FCMs. To make our design policy consistent with this construction specification, the applicable portions have been used for this Subsection. The term "Engineer" in the guide specification has been revised here to refer specifically to the bridge designer.

C2: This paragraph is taken from the Staff Bridge Engineer’s 4/6/89 Technical Memorandum #2.

C3: It is anticipated that ANSI/AASHTO/AWS D1.5 will eventually contain a Fracture Control Plan. When it does, CDOT will probably refer to D1.5 for its Fracture Control Plan.

POLICY	COMMENTARY

The Staff Bridge BRIAR unit shall be notified of any new bridge containing FCMs. The bridge designer will provide half-size copies of the bridge plan sheets showing the FCMs and their details. These members and their details shall be highlighted. In addition, the form shown below shall be filled out. This form with the highlighted plans are to be submitted to BRIAR with the Rating Package for the bridge. (C4)

By definition, fracture critical members are non-redundant. The fatigue requirements for non-redundant members given by the AASHTO Standard Specifications shall be closely followed.

C4: This requirement, and the attached form, originated from a 2/21/90 memorandum from the Staff Bridge Construction Engineer. The attached form is presented to illustrate the requested format. This format is to be expanded where necessary to include additional elements, or to give more room for descriptions and sketches. The sketches are to show the fracture critical details that should be looked at. The highlighted plans are to identify the FCMs and the locations of the fracture critical details.

FRACTURE CRITICAL INSPECTIONS

STRUCTURE TYPE: ________________ STRUCTURE NO: ________________
NO OF SPANS: ________________ HIGHWAY NO: ________________
NO OF GIRDERS PER SPAN: ________________ DATE: ________________
YEAR BUILT: ________________

DETAILS THAT ARE FRACTURE CRITICAL:

DETAIL 1 ____

area to inspect: ____

DETAIL 2 ____

area to inspect: ____

DETAIL 3 ____

area to inspect: ____

SKETCH OF DETAIL:

DETAIL 1 DETAIL 2 DETAIL 3

Inspection Date: __________ Inspectors Initials: __________

AASHTO AND ASTM STRUCTURAL STEEL DESIGNATIONS

POLICY

Staff Bridge and bridge design consultants shall disregard the new ASTM and AASHTO materials designations in Table 10.2A of the 14th Edition of the AASHTO bridge specifications.

ASTM & AASHTO materials specifications A36, A572, A588 still exist as does AASHTO M183, M223 & M222 in the 1989 edition of the respective materials manuals.

COMMENTARY

ASTM & AASHTO elected to group all bridge steels together under A709 and M270 respectively. Colorado however, does not use the Quenched and Tempered steels. To eliminate the possibilities of substitutions and the perceived confusion that comes with change and to conform with the new "Bridge Welding Code" ANSI/AASHTO/AWS D1.5-88 which does not address the new M270 structural steel, we will stay with the old designations as long as they are available to us.