PART 4

Structural

NOTE: Users should consult Design Memoranda listed on the title sheet of each affected chapter for details related to revisions.
CHAPTER 402

Structure Size and Type

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CHAPTER 402

STRUCTURE SIZE AND TYPE

402-1.0 DEFINITIONS

402-2.0 NOTATIONS

402-3.0 INTRODUCTION

The basic objective of this Chapter is to select the most appropriate structure type and configuration for the given project site conditions. This selection process is a critical event in the project development. The decision made in this process will impact the detailed structure design phase, construction costs, and maintenance costs over the life of the structure. The designer shall perform the structure size and type analysis based on the information provided in this Chapter, available resources and through the use of sound engineering theory, practice, and judgment. The results from the analysis will permit final design of the structure through the rest of the project development.

This Chapter describes available resources, abbreviated submission requirements for Stage 1 Review, as it relates to the structure size and type process, and the use of evaluation factors and design criteria to determine the most appropriate structure size and type for the given project site. This information is provided throughout this Chapter while referencing applicable figures, design references, and other Manual Chapters. The design memoranda can include additional applicable information for use in the structure size and type analysis.

Design factors shown in this Chapter will depend primarily on project classification as 3R or 4R. The Engineer’s Assessment will be based on the direct use of 3R or 4R criteria and the appropriate geometric design tables. See Chapters 53 and 55 for additional information.

402-4.0 SUBMISSION REQUIREMENTS

The structure size and type analysis is performed as part of the Stage 1 design phase. The Stage 1 design phase shall be concurrent with or following the design phase for the roadway. It is critical for the structure design to coordinate with the roadway design during the structure size and type process.
The Stage 1 design phase is described in Chapter 14. A brief overview of the specific requirements of the Stage 1 Submission as it relates to the structure size and type analysis is described below.

402-4.01 Design Information

Applicable project and design information shall be presented in the form of a report and included as part of the structure size and type analysis within the Stage 1 Submission. The structure size and type report shall provide a brief description of the project and include narratives discussing the following.

402-4.01(01) Discussion of Design Factors

The design factors that contribute to the structure size and type analysis shall be discussed within the report narrative. All existing conditions, such as existing structure, natural obstacles, utilities, unusable soil conditions, stream characteristics, traffic maintenance, hydraulic parameters, and clearance requirements shall be known at the time of the structure size and type selection process.

Design and evaluation factors are explained below. These factors are not all inclusive of the factors encountered. Each project is unique and dependent on specific considerations, restraints, and conditions that shape the development of the project. Such factors can include geometry and hydraulic considerations, environmental restrictions, right-of-way restraints, corridor consistency, aesthetics, construction costs, life cycle costs, maintenance of traffic, geotechnical considerations, and others. These applicable and specific factors that are relevant to the structure size and type analysis performed shall be discussed within the report narrative.

402-4.01(02) Deviation from the Initial Engineer’s Report

An initial Engineer’s Assessment is completed prior to the structure size and type phase. Deviation from the original Assessment will require an analysis to substantiate the need for the change. The structure size and type narrative will include the justification for changes to the project cost.
402-4.01(03) Discussion of Alternates

A structure size and type analysis consists of investigating pertinent and logical structure types and sizes that fit the specific project site and its parameters. Investigation into multiple alternates is encouraged so that a true best alternate can be chosen to be carried through final design. Typical alternates that can be investigated are a spill-through type configuration versus a bridge utilizing retaining walls at the end bents, a large-girder single-span bridge versus a three-span bridge, a small single-span bridge versus a three- or four-sided structure, or other possible configuration comparisons. Superstructure-type alternatives such as, but not limited to, reinforced concrete slab, prestressed-concrete beams, steel girders, post-tensioned structures, steel U-tubs, or a post-tensioned slab shall be considered. It is not necessary that all above types be evaluated, but a reasonable alternative shall be included.

These different structure alternates shall be compared using the applicable evaluation factors, with the primary consideration being cost. The advantages and disadvantages of each alternate shall be discussed, indicating the primary reasons for the selection of the recommended structure size and type.

402-4.02 Economic Analysis

An economic analysis shall be performed as part of the structure size and type analysis in order to determine the initial construction cost, the life cycle cost, and other costs associated with each alternate investigated. This economic analysis shall be included as part of the structure size and type analysis within the Stage 1 submission.

The purpose of this section is to provide the process to be used in evaluating the economics of various structural alternatives with the goal of selecting the most suitable alternative to proceed to the final design phase. Cost comparisons required at the Structure Size and Type phase shall not be completed with only the initial capital cost considerations. The lowest initial capital cost does not always lead to lowest cost for the owner. Cost comparisons for structural alternatives shall, in addition to initial capital costs, include costs associated with long-range considerations. Cost comparisons for each alternative shall consider all aspects that can impact initial and future costs such as;
1. the cost associated with the complexity of future inspections;
2. future maintenance and life cycle costs;
3. operating costs;
4. the availability and familiarity of the structure type with local contractors, fabricators and suppliers;
5. the impacts of the structure alternative to the roadway approaches and retaining walls;
6. the impacts to utilities;
7. costs associated with right-of-way requirements;
8. the costs required for additional environmental mitigation for a specific alternate; and
9. the costs associated with unusual site conditions or constraints.

All of these factors shall be calculated and included in the cost estimate for each structure alternative in order to properly identify the correct alternative to be chosen for the final design phase.

402-4.02(01) Construction Cost

The economic analysis shall compare the estimated construction cost to complete the project for each alternate investigated. To determine relative construction costs of each alternate, all quantities independent of the alternatives shall be computed. Quantities that are considered equal for each alternate need not be considered, as they do not contribute to the comparative construction cost computed.

Current construction prices in materials and construction methods shall be obtained in order to obtain accurate costs.

402-4.02(02) Life-Cycle Cost

Long-term life-cycle costs of each alternate shall be considered in the overall structure size and type analysis. Different structure types and elements have different rehabilitation cycles or replacement schedules. These factors can affect the overall cost of the structure and therefore the selection of the recommended alternate.

402-4.02(03) Summary

The economic analysis performed will yield the respective construction costs, life-cycle costs, and overall costs of each alternate investigated. This economic analysis shall be included as part of the structure size and type analysis. A discussion of the recommended alternate, largely based on this analysis, shall be provided within the structure size and type report.
402-4.03  Level One Checklist and Computations

A Level One Checklist, including computations, shall be developed for the roadway and bridge elements. The apparent Level One and Level Two design exceptions shall be indicated. See Section 40-8.02(01) for additional information regarding Level One Checklist requirements.

402-4.04  Plans

Stage 1 plans shall be submitted that show the recommended alternate determined from the structure size and type analysis. See Section 14-2.01(03) for additional information regarding Stage 1 Plans requirements.

402-4.05  Preliminary Cost Estimate for Selected Alternate

A preliminary cost estimate shall be submitted for the recommended alternate determined from the structure size and type analysis. At this stage of development, for the computation of the initial capital cost, the recommended alternative shall have approximately 70 to 85% of the major-quantities pay items identified, including the pay item-numbers. The remainder of the items shall be included as a contingency item.

402-4.06  Miscellaneous Forms

A Quality Assurance form and a Scope/Environmental Compliance Certification/Permit Application form shall be provided with the submission. See Section 14-2.01(03) for additional information regarding necessary forms required with the Stage 1 Submission.

402-4.07  Computations

The necessary calculations performed during the structure size and type analysis shall be submitted. The following calculations are those that shall be included as part of the structure size and type analysis.
402-4.07(01) Structure-Sizing Calculations

The design computations for determining the structure size and geometrics for the alternates investigated shall be included. For a structure spanning a waterway, the waterway opening and freeboard calculations shall be included, along with the hydraulics-approval letter following the Hydraulics Review Submission. All applicable and necessary drawings and sketches shall be submitted to supplement the structure-alternate sizing calculations.

402-4.07(02) Structural Calculations

The structural calculations performed during the structure size and type analysis will be included with the submittal. These can include preliminary structural-member calculations performed to obtain the required structure depth of the structure. If computer software is used, only the pertinent input and output shall be included in the submittal.

402-4.07(03) Quantities Calculations

The quantities computations for each alternate investigated during the economic analysis will be submitted, so that the results of the economic analysis can be verified. The preliminary quantities for the recommended alternate used to derive the preliminary cost estimate will be included with the submission.

402-5.0 PRIMARY EVALUATION FACTORS

402-5.01 Document Resources

402-5.01(01) Engineer’s Assessment

The Engineer’s Assessment is developed to establish the minimum parameters for the project as follows:

1. 3R or 4R criteria;
2. project alternatives and recommended alternate;
3. traffic maintenance;
4. cost estimates;
5. traffic data;
6. crash data;
7. survey requirements; and
8. right-of-way impacts.

402-5.01(02) Inspection Report

The National Bridge Inspection Standards dictate that every bridge is to be inspected at a frequency not to exceed 24 months. Inventory and condition data of all bridges is updated during biennial inspections in accordance with FHWA’s Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges.

The inspection report includes inventory data such as:
1. location description,
2. bridge type,
3. geometric dimensions,
4. year built,
5. year reconstructed,
6. estimated remaining life,
7. condition ratings and comments,
8. pictures,
9. deficiencies identified, and
10. proposed improvements.

This information included within a bridge-inspection report can be a useful piece of information for a structure size and type analysis being performed for the replacement structure at a project location.

402-5.01(03) Site Reconnaissance

Site reconnaissance provides field information, changes from surveyed information, and project discussion with personnel from other offices and divisions.

402-5.01(04) Survey

Aerial and topographical surveys provide ground information of all physical features including utilities.
402-5.01(05) Hydraulics

Based on topographic-survey information, the Office of Hydraulics provides recommendations for the structure sizing. It provides $Q_{100}$ and $Q_{500}$ elevations, design flow, velocity, waterway opening requirements, and scour information. See Section 203-3.04 for a typical hydraulics report.

402-5.02 Constraints

402-5.02(01) Environmental

The Environmental Services Division’s Environmental Assessment Team will perform the environmental studies. A bridge over a waterway will likely require an IDEM 401 permit, ACE 404 permit, and Rule 5 permit. The following shall be considered in the analysis for structure-type selection.

1. **Waterway Crossing.** Number and location of piers.

2. **Sensitive Area.** Environmental impacts shall be minimized in a sensitive area, e.g., near wetlands.

3. **Discharge of Fill.** Discharge of fill below the Ordinary High Water elevation will require a U.S. Army Corps of Engineers Section 404 permit. The need for this type of permit will depend upon the amount and type of fill discharged.

4. **Environmental Commitments.** Project specific criteria and commitments.

402-5.02(02) Historic and Archeological Resources

Integrity of all residences, churches, bridges, or barns that are on or eligible to be on National Register of Historic Places must be maintained. *The Archeological Resources Protection Act (ARPA)* prohibits the excavation of archeological resources, or anything of archeological interest, on federal or Native American lands. Therefore, the project parameters may have to be altered to avoid impacting historic or archeological resources.
402-5.03 Costs

Costs of various alternates will determine the final recommendation. Initial and-life cycle costs are a truer indicator of the final cost.

Figure 402-5A approximates the most economical structure for various span lengths. Factors such as vicinity of fabricators, availability, reliability of materials, and cost of labor shall also be considered.

402-5.04 Constructability

Temporary falsework may be a consideration in the structure alternate analysis and can be a substantial item of the construction cost. A superstructure system can include cast-in-place concrete. Therefore, it can require elaborate temporary supports and formwork. Such a system derives its economic feasibility from the relative simplicity of construction, or from the highly effective monolithic nature of the finished superstructure. If the bridge is over a waterway or will have a high finished elevation, the cost of the falsework may become prohibitive, and therefore eliminate this alternate.

A reduction below the minimum vertical highway clearance during construction is not permissible without a design exception. For a 3R non-freeway project, the minimum vertical clearance value is documented in the appropriate geometric design criteria table shown in Chapter 55, under Existing Overpassing Bridge. For another type of project, coordination is required with the appropriate district traffic engineer. A design-exception request shall be processed.

A cofferdam or temporary causeway can be another substantial item of the construction cost. These are often necessary for the contractor to build the substructure and foundation units. These shall be considered during the permitting process, and can be a limiting factor in the obtaining of the permits. These factors shall be considered in the structure size and type selection process.

402-5.05 Railroad [Rev. Aug. 2013]

Coordination with the railroad company should begin as early as possible in the project development process. See Chapter 105 for the railroad-coordination process.

402-5.06 Utilities [Rev. Aug. 2013]
The bridge design shall be consistent with the Utility Accommodation Policy, documented in Chapter 104.

402-5.07 Other Considerations

402-5.07(01) Maintainability

Open or inadequately-sealed deck joints have been identified as the foremost reason for structural corrosion of structural elements by permitting the percolation of salt-laden water through the deck. To address this, a continuous deck, integral end bents, improvements in drainage, epoxy coatings, and concrete admixtures shall be considered. The LRFD Specifications also requires that reasonable access be provided where other means of inspection are not practical.

402-5.07(02) Adaptability

Nearly every superstructure type can be widened, but not with the same level of ease. A slab, deck on beams or girders, or system consisting of prefabricated concrete or wood elements each lends itself to such reconstruction. However, a large concrete box, through-type superstructure, or that with substantial wings does not. If a definite need for future widening exists, these latter structural types shall not be considered.

402-5.07(03) Context-Sensitive Solutions and Aesthetics

Transportation professionals and communities are working together to develop engineering solutions that fit the project setting. Identifying potential issues and opportunities in the preliminary scoping phase, early in the process, can help develop the best project solution possible within the available budget. Engaging stakeholders throughout the project-development process allows their input to be considered during the appropriate stages of the project, and helps to gain their cooperation and support. A successful project will satisfy the purpose and need while preserving the scenic, aesthetic, historic, and environmental resources native to the project location.

Each project shall employ a context-sensitive approach, while the resulting project solutions will vary according to the context. The INDOT policy and definition of CSS, and additional information about CSS appears on the website at http://www.in.gov/indot/div/projects/indianacss/index.html.
LRFD Specifications Article 2.5.5 promotes uninterrupted lines, contours that follow the flow of forces, and the avoidance of cluttered appearances. The requirements regarding aesthetics have been prompted because many bridges have been exclusively selected and designed on the basis of construction cost or engineering simplicity with disregard for their appearance and for their conformance with the environment. The bridge design shall integrate the basic elements of efficiency, economy, and appearance. Regardless of size and location, the quality of the structure, its aesthetic attributes, and the resulting impact on its surroundings shall be considered.

402-6.0 DESIGN FACTORS

402-6.01 Structure Location

The sizing of a structure is dependent on the features being crossed, roadways, railroads, waterways, or a combination of these. The key features that shall be addressed for each type of crossing are described below.

402-6.01(01) Stream Crossing

Approval by the Office of Hydraulics will be required prior to the submittal of Stage 1 plans for each waterway crossing. The requirements for the hydraulics submittal are defined in Section 203-3.04.

Chapter 203 provides criteria for the hydraulic design of a bridge waterway opening. This will have an impact on the size and elevation of the structure. Chapter 203 also discusses hydraulic policies on maximum backwater, freeboard, bridge sizing policy, maximum velocity, hydraulic scour and the use of analysis methodologies. The structural considerations relative to waterway opening are described as follows.

1. **Substructure Displacement.** An allowance has already been made in the required waterway opening provided by the Office of Hydraulics Team for the area displaced by the substructure. Therefore, the area of piers and bents below the $Q_{100}$ elevation shall not be deducted from the gross waterway area provided. The Office of Hydraulics shall be contacted if thicker substructure units, e.g. drilled shafts, or if more substructure units are proposed than anticipated by the Office of Hydraulics so that adjustments can be made to the required waterway-opening value.
2. **Existing Substructure Elements.** Removing existing pier or abutment footings can be a major expense. Therefore, where practical at a stream crossing, the span lengths shall be adjusted, or the entire structure shall be shifted so that new foundations or piles for the replacement bridge will avoid existing substructure elements.

3. **Interior Supports.** For a major waterway crossing, and if the foundation conditions allow, a single round, hammerhead-type pier supported by a deep foundation is preferred. Multiple round columns may be used, but they can require a solid wall between columns to avoid the collection of debris. A single-wall pier aligned parallel to the flood flow direction can be a more suitable alternative.

For a meandering river or stream, the most desirable pier type is normally a single, circular pier column.

4. **Freeboard.** Where practical, a minimum clearance of 2 ft shall be provided between the design water-surface $Q_{100}$ elevation and the low-structure elevation to allow for passage of ice and debris. Where this is not practical, the clearance shall be established based on the type of stream and level of protection desired. For example, 1 ft shall be adequate for a small stream that normally does not transport drift. An urban bridge with grade limitations may not provide freeboard. A freeboard of 3 ft is desirable for a major river which is known to carry large ice floes or debris. Coordination with the Office of Hydraulics is essential.

5. **Low Channel-Clearing Elevation.** The low channel-clearing elevation shall be set as described in Section 203-3.04, normally at 1 ft above the Ordinary High-Water elevation. The OHW elevation shall be obtained from the survey or determined from U.S. Army Corps of Engineers procedures.

6. **Span Lengths.** The minimum span length for a bridge with more than 3 spans shall be 100 ft for those spans over the main channel. A three-span bridge shall have the center span length maximized where debris may be a problem. A two-span bridge shall be avoided at a stream crossing where the pier will be located in the center of the main channel. The Office of Hydraulics shall be contacted if a two-span structure is necessary.

**402-6.01(02) Grade Separation**
The geometrics of an underpass have an impact on the size of the overhead structure. Figures 402-6A, 402-6B, and 402-6C provide schematics of a bridge underpass. The underpass shall be designed to satisfy the geometric design criteria described in Chapter 53 and as discussed in Section 402-6.02(01). The geometric design of a bridge underpass is summarized as follows.

1. The full-approach-roadway section, including the median width, shall be provided through the underpass section.

2. The roadside clear-zone width applicable to the approaching roadway section and auxiliary lanes will be provided through the underpass.

3. Other roadside safety criteria may apply. See Chapter 49.

4. A collision wall to protect the bridge substructure from vehicular impact may be warranted through the underpass. The LRFD Bridge Design Specifications discusses both the warrants for and the design of a collision wall. See LRFD 3.6.5.

5. In determining the cross-section width, the likelihood of future roadway widening shall be considered. Widening an existing underpass in the future can be expensive, so it may be warranted, if some flexibility is available, to allow for possible future roadway expansion.

402-6.01(03) Railroad Grade Separation [Added Aug. 2013]

The roadway should desirably be placed over the railroad for a new railroad grade separation. Placing a railroad over a roadway is less desirable due to the following:

1. railroad operations are potentially slowed due to underpass construction;

2. future widening of the roadway becomes difficult and costly; and

3. a temporary runaround track is typically required during construction of a railroad bridge. This significantly increases the construction cost and will increase the amount of temporary right of way required.

The AREMA Manual for Railway Engineering Chapter 8, Section 2.1.5 requires a reinforced concrete crash wall for piers supporting bridges over railways located within 25 feet from the
centerline of the track, measured perpendicular to the track, unless the size of the pier satisfies the criteria for heavy construction.

The typical railroad horizontal and vertical clearances are shown in Figure 402-6 O. The values required for a specific railroad company (Railroad) may vary from those shown in the figure. The designer should document the required horizontal and vertical clearances during Stage 1 plan development.

The FHWA limits their fiscal participation to a horizontal clearance up to 20 feet. The Railroad must submit justification to INDOT for a horizontal clearance of more than 20 feet.

The Railroad may request accommodation for the construction of a future track. The location and spacing between existing and future tracks, typically 15 ft, should be established early in the design process so that the appropriate span length can be provided.

The Railroad may also request additional clearance to accommodate an access road for maintenance. The Railroad is responsible for providing justification for their request for this additional clearance.

The Department railroad coordinator will review the documentation provided by the Railroad for justification in accordance with 23 CFR 646.212 and 646.214. If the Department concurs with the justification, FHWA will participate in the total bridge cost. If not, FHWA funds will not be applied to additional portion of the bridge used to accommodate the additional horizontal clearance or access road.

402-6.02 Structure Sizing

The sizing of a structure requires the evaluation of other factors in addition to structural considerations. These include bridge and underpass geometrics, abutment dimensioning, and waterway opening. Together, they will determine the overall size of the structure for analysis and design. Each structure of longer than 20 ft in total span length is considered a bridge, and must have a structure file number and a separate Des. number.

Chapter 53 provides criteria for roadway geometrics. The road-design criteria will determine the proper geometric design of the roadway, and the bridge design will accommodate the roadway design across each structure within the project limits. This will provide full continuity of the
roadway section for the entire project. This process will, of course, require proper communication between the road designer and bridge designer to identify and resolve problems.

The bridge geometrics will be determined in the project scope of work. For a new or reconstructed bridge on a 4R project, the criteria provided in Chapter 53 will determine the geometric design of the bridge. For a bridge within the limits of a 3R project, the criteria provided in Chapter 55 will determine the bridge geometrics. Chapter 53 provides project scope-of-work definitions and a map of the State highway system with designated 3R and 4R routes.

402-6.02(01) Cross Sections

Figures 402-6D, 402-6E, 402-6F, and 402-6G each provide schematics of the bridge cross section for a specific highway type. The following will apply to the bridge cross section.

1. **Bridge Clear-Roadway Width.** The geometric design criteria figure in Chapter 53 for the appropriate functional classification provides this information for a new or reconstructed bridge within the limits of a 4R project. The geometric design criteria figure in Chapter 55 for the appropriate functional classification provides this information for a bridge within the limits of a 3R project. Figure 402-6H shows the relationship between the bridge-railing and approach-guardrail offsets.

Where a bridge clear-roadway width is permitted to be narrower than the travel lanes plus the usable shoulder width on each side, a guardrail transition, collinear with the bridge railing, shall be provided. Thereafter, the guardrail shall be flared at an appropriate barrier flare rate until the guardrail length satisfies the length-of-need requirement or it intersects the approach guardrail. However, a continuous straight, without flare, run of guardrail is preferred for driving comfort and aesthetics. For this situation, the bridge clear-roadway width will nearly match the face-to-face guardrail width of the approach road section.

Chapter 53 discusses the design of a median for a long bridge with a sufficiently narrow median. Increased safety benefits can be realized in construction of a single structure. Depending on site conditions, a single structure shall be used rather than twin structures where the median width is approximately 30 ft or less on a freeway, or 20 ft or less elsewhere. The median width at an overpass shall match the median width on the approach.

For the median shoulders of a divided facility with two or more lanes in each direction, each bridge shall have a 5'-8” median-shoulder width where a type FC, FT, or TF-2 railing is used, or a 6'-0” median-shoulder width where another bridge-railing type is used.
auxiliary lane may be required across a structure where warranted. See Chapter 53 for the requirements.

2. **Cross Slope.** Each new or reconstructed bridge on a tangent section will be constructed with a cross slope of 2% sloping away from the crown. The 2% applies to the entire width from the crown to the face of railing or curb. The crown across the bridge will be in the same location as the approaching roadway crown. A tangent-section cross slope may be increased to 3 to 4%, with only one slope break in the deck, if roadway geometrics require it.

3. **Sidewalk.** The sidewalk on a bridge is often poured monolithically with the curb and the width dimensioned from the front face of the curb. The sidewalk width is measured exclusive of the curb, i.e. measured from the back face of the curb. Guidance provided by the U.S. Access Board indicates that when there is no defined back face of curb, a 6-in curb width should be assumed. See Figure 402-6P, Bridge Sidewalk Width. Where a bridge includes a sidewalk, the bridge length should be reviewed in accordance with the passing space and sidewalk width criteria in section 51-1.03(02). Section 45-1.06 provides guidelines for sidewalk warrants and sidewalk accessibility criteria. Placement of a sidewalk on a bridge will impact the selection or location of the bridge railing. Section 404-4.02(03) provides criteria for bridge and pedestrian railing.

4. **Bridge Width for Traffic Maintenance.** The figures in Chapter 53 provide criteria for the bridge width. Additional permanent bridge width may be provided solely for the purpose of placing one lane of traffic across the bridge during construction. This can eliminate the need for a detour or runaround, or the use of a local road to re-route traffic during construction. See Chapter 83 for more information on maintenance and protection of traffic during construction.

5. **Bridge Width on Flat or Short Horizontal Curve.** Railings and copings on a bridge within a horizontal curve are built concentric with the roadway centerline. However, where the bridge is on a flat curve, or if the bridge is short, it may be more practical to build the railing and coping parallel to the long chord if the curved roadway plus shoulders and barrier offsets is within the inner faces of the railings, and it is economically feasible to construct a wider tangent bridge deck. It is considered economical if the bridge-deck width is increased by not more than 1 ft. However, it can be increased if it is determined to be more economical. Figure 402-6 I illustrates these criteria.
402-6.02(02) Alignment [Rev. Mar. 2017]

The horizontal and vertical alignment will be determined for the overall roadway within the project limits, and the bridge will be designed consistent with the roadway alignment. See Chapter 53 for geometric-design criteria. The desirable horizontal and vertical alignment objectives are as follows.

1. **Grade.** A minimum longitudinal grade of 0.5% on the bridge is desirable. A flatter grade will be permitted where it is not physically or economically desirable to satisfy this criterion.

2. **Vertical Clearance.** The vertical clearance requirements are shown in Figure 402-6J. This clearance shall be provided for the elevation and alignment of the overhead structure. The vertical clearance is determined at the low-steel or -concrete member elevation. Figures 402-6A, 402-6B, and 402-6C illustrate where the clearance is measured. Clearance shall be maintained across both the traveled way and the shoulders. The same minimum vertical clearance in the traveled way and shoulders is not required to be maintained in the clear zone. However, a separate minimum vertical clearance is often necessary within the clear zone. For economy, the minimum vertical clearance shall not be exceeded by more than 6 in. unless project constraints require a higher clearance.

   Consideration of the vertical and horizontal clearance during construction phases shall be considered in setting the profile of the bridge. See Chapter 83 for requirements during construction.

3. **End Bent.** The end-bent configuration impacts the required structure length and shall be accounted for in the sizing of the structure. The following will apply.

   a. The clearance from the top of the berm to the bottom of the superstructure shall be at least 6 in., with a maximum of 1’-8”. The minimum berm width is 3 ft. See Figure 402-6K.

   b. Wingwalls will be required for each beam structure.

   c. The spillslope for a water crossing is limited to a maximum of 2:1, except for a structure located within the backwaters of the Ohio River, where the spillslope is 3:1. For an overpass structure, the required crossed-roadway-section clear-zone width shall be considered in the setting of spill slopes.
Where utilizing an MSE retaining wall at an end bent, a minimum distance of 3 ft. is required between the back of the wall panel and the edge of the pile sleeve or the pile (where sleeves are not required). For determining preliminary structure span length, a 24-in pile sleeve should be assumed. The need for pile sleeves will be determined by the Office of Geotechnical Services. LRFD 11.10.11 provides additional information regarding the placement of obstructions in the reinforced soil zone.

402-6.02(03) Structure Width

Structure width, or out-to-out coping, derives from providing a bridge clear-roadway width as outlined in Section 402-6.02(01). Bridge railings, sidewalks, median, etc. shall be considered toward determining the required structure width.

402-6.02(04) Superelevation

If practical, a horizontal curve or superelevation transition shall be avoided on a bridge. A bridge may be superelevated if this results in a more desirable alignment on either roadway approach. If properly designed and constructed, a bridge will function adequately where this occurs.

On a superelevated roadway section, a break may be provided between the traveled way and high-side shoulder. However, on a superelevated bridge section, a constant cross slope shall be provided across the entire curb-to-curb or railing-to-railing width. If the bridge is within the normal superelevation-transition length where the pavement slope varies on either side of the profile grade, the superelevation-transition diagram shall be modified to provide a constant cross slope. See Figure 402-6L.

The approach roadway will include a shoulder with a cross slope different from that on the bridge. For example, the typical roadway-shoulder cross slope on tangent is 4%. It will be necessary to transition the roadway shoulder slope to the bridge deck slope in the field. Plan details are not required for this transition.

402-6.02(05) Structure Length

Structure length shall be determined by considering the vertical elevations and horizontal dimensions at the high coping. This is applicable to a superelevated bridge. See figures 402-6M and 402-6N for a structure-length calculation method.
402-6.02(06) Clear Zone

The geometrics of an underpass have an impact on the size of the overhead structure. The roadside clear-zone width applicable to the approaching roadway section will be provided through the underpass. Chapter 49 provides the clear-zone criteria, which are a function of design speed, traffic volume, highway alignment, and side slope. If an auxiliary lane is provided through the underpass, this impacts the clear-zone-width determination.

A collision wall to protect the bridge substructure from vehicular impact may be warranted through the underpass. The AASHTO LRFD Bridge Design Specifications discusses both the warrants and design of a collision wall.

402-6.02(07) Three Sided or Box Structure

The bridge definition outlined in Section 402-6.02 also applies to each three-sided structure, oversize box culvert, set of multiple box culverts, or set of multiple pipe structures. A large culvert having an opening width of 20 ft or less can also qualify as a bridge if the skew results in the span’s measurement along the centerline of the roadway to be greater than 20 ft. If a three-sided-structure span length for either the flat-top or arch alternate is longer than 20 ft as described above, it shall be regarded as a bridge.

402-7.0 SUBSTRUCTURE AND FOUNDATION

This Section discusses types of substructure and foundation systems, and it provides their general characteristics. This information shall be considered with the intent to select the combination of substructure and foundation which is suitable at the site to economically satisfy the geometric requirements of the bridge and to safely use the strength of the soil or rock present at the site.

The demarcation line between substructure and foundation is not always clear, especially for extended piles or drilled shafts. The foundation includes the supporting rock or soil and parts of the substructure which are in direct contact with, and transmit loads to, the supporting rock or soil. This definition will be used to the greatest extent possible.

A similar difficulty exists in separating substructure and superstructure where these parts are integrated. This Section will refer to each component or element located above the soffit line as part of the superstructure.
Chapters 408 and 409 discuss the design of foundations and substructure elements.

**402-7.01 Foundations**

The most economical design shall be established that accounts for structural criteria and intended function of the structure. Whether it is for shallow or deep foundations, the foundation support cost shall be defined as the total cost of the foundation system divided by the load the foundation supports in tons. The cost of a foundation system shall be expressed in terms of dollars per ton load that will be supported.

Most currently-used systems can be categorized into the groups illustrated in Figure 402-7A. These groups are discussed below.

For interior supports at a stream crossing, extended piles, piles with a pile cap, or drilled shafts are used. Where scour is not expected and quality load-bearing soil is close to the surface, the use of spread footings shall be considered, provided that the geometric limitation as discussed in Section 402-6.01(02) is satisfied.

**402-7.01(01) Pier or Frame Bent Supported with Spread Footing**

Limiting the applied stress for a specified amount of settlement is the most controlling factor in the design of a spread footing. The *LRFD Bridge Design Specifications* provides no dimensional restrictions for substructure settlement for a spread footing. However, the design shall satisfy the geotechnical-report recommendations which are based on a specific amount of settlement. Unlimited settlement can impair the serviceability of the structure and can cause problems as follows.

1. The superstructure can intrude into the required vertical clearance. This can be prevented by increasing the as-built clearance to be in excess of the specified settlement value.

2. Rideability can be impaired by introducing angular rotations in the longitudinal profile of the roadway due to differential settlement among the individual substructure portions. The substructure design shall limit such angular rotations to 0.005 rad. This value shall be applied to the cumulative rotations between two adjacent spans. Differential settlement shall be determined by means of assuming alternating maximum and minimum values of the calculated settlement range between adjacent supports. Because settlement is a deciding factor, this calculation shall be made during the structure type and size determination. The limit of 0.004 rad in relative rotation shall be applied to either a simply-supported or continuous superstructure. For a fixed value of permissible rotation, the larger the span, the larger the settlement that can be accommodated.
3. In a continuous superstructure, differential settlement results in force effects which are in addition to those due to gravity loads. The *LRFD Specifications* incorporates both force and geometric effects of settlement in a number of load combinations which are mandated for investigation. It does not prohibit the inelastic redistribution of the resulting force effects.

4. The larger the span and the lesser its rigidity, the smaller are the force effects due to settlement. Where settlement causes negative moments in the superstructure, the problems that can arise are related to cracking and ductility, rather than to strength.

The *LRFD Specifications* address the danger of scour for a pier located in a waterway. *LRFD Specifications* Section 2 lists methods of minimizing this catastrophic potential, due to the large number of bridges that wash away each year. A spread footing requires a quality foundation material close to the ground surface. The bottom of a spread footing on soil shall be below the deepest frost level or at least 4 ft below the flow line. See Chapter 408 and its applicable figures for more information.

**402-7.01(02) Pier or Frame Bent Supported with Deep Foundations**

Where conditions are not present which favor or permit the application of a spread footing, a deep foundation, such as drilled shafts or piles, shall be considered. Prefabricated piles made of concrete, steel, or a combination of these, are driven into position by means of hammers. Drilled shafts and drilled concrete piles are constructed with the same technique requiring specific skills. Drilled shafts, especially those with bell-shaped bottoms, can carry extremely large loads.

The *LRFD Specifications* provides a two-level approach for the design of a deep foundation, in which the structural resistance of the pile or shaft and the structural resistance of the supporting soil or rock are investigated separately.

**402-7.01(03) Extended-Pile Bent**

Under certain conditions, the economy of a substructure can be enhanced by means of extending the deep foundation above ground level to the soffit of the superstructure. These conditions exclude the presence of large horizontal forces which can develop due to seismic activity, collision by vessels or vehicles, ice, or stream flow intensified by accumulated debris. Longitudinal braking forces, which are increased in the *LRFD Specifications*, shall be resisted at the abutment.
The extended piles require a cap-beam for structural soundness. This cap-beam may be an integral part of the superstructure. An extended drilled shaft placed directly beneath each beam line can eliminate the use of a cap-beam. Sufficient space shall be provided at the top of the shaft to allow for future jacking operations.

**402-7.02 End Bent or Abutment**

**402-7.02(01) Usage**

The types of end supports and their usage are as follows.

1. **Integral End Bents.** These, a subset of spill-through end bents, shall be used for a structure which is in accordance with the geometric limitations provided in Figure 409-2A. Integral end bents may be utilized where the structure configuration has the end bent behind and acting independently from a retaining wall such as a mechanically-stabilized-earth retaining wall.

2. **Non-Integral End Bents.** These shall be used where spill-through end bents or end bents independently placed behind a mechanically-stabilized-earth retaining wall are desirable, but integral end bents are not appropriate. These include semi-integral end bents and shallow end bents utilizing an expansion joint.

3. **Abutments and Wingwalls.** For soil conditions or bridge geometric dimensions not suitable for spill-through end bents, abutments and wingwalls of the cantilever type, or a mechanically-stabilized-earth wall or other type of earth-retaining system, shall be used.

See Chapter 409 for more information.

**402-7.02(02) Spill-Through End Bent**

A spill-through end bent, either integral or non-integral, by its nature is supported by an individual deep foundation, which the fill flows through. The end bent consists of a cap-beam and a non-integral mudwall which provides partial retaining for the fill at its top. With this type of end bent, the fill is largely self-supporting. Therefore, for the same fill slope, it requires more space in plan geometry and results in longer spans.
**402-7.02(03) Integral End Bent**

The integral end bent eliminates the deck joint between the superstructure and the end bent by the structural integration of the two. The vertical dimension of the cap beam can be minimized as the mudwall becomes a composite part thereof.

Components of the deep foundation shall be flexible to accommodate the longitudinal movement of the pile bent. Such flexibility can be provided with steel H-piles or steel-encased-concrete piles.

The reinforced-concrete bridge approach shall be attached to the end bent. The longitudinal bridge movements shall be accommodated at the outer end of the reinforced-concrete bridge approach by using a terminal joint of 2 ft width or a pavement relief joint if a portion of the adjacent pavement section is concrete. No such joint is required if the entire adjacent pavement section is asphalt.

The *LRFD Specifications* encourages minimizing the number of deck joints. This end-bent type satisfies that requirement. If the superstructure is fully continuous, no deck joints remain.

Because of the difference in construction costs between an integral end bent and an abutment, and the less-than-desirable performance of bridge-deck joints, an integral end bent shall be used where possible. See Chapter 409. Limitations of continuous superstructure length are related to the flexural stresses caused in the piles by the expansion and contraction of the deck due to temperature, creep, and shrinkage.

If the maximum distance from the zero point to the integral end bent does not exceed the criteria shown in Figure 409-2A, the effects of deck expansion and contraction may be neglected in the analysis of the bridge. The piles are designed only for axial loads to satisfy specified stress limits. If the continuous deck length exceeds these limits, or if a better understanding of the behavior of the end bent is desired, an in-plane frame analysis shall be performed and the components designed as specified in the *LRFD Specifications*.

To minimize deformation-induced force effects, only one row of vertical piles is permitted in an end bent. If the resistance of the surrounding soil is larger than a specified value, the piles shall be driven into predrilled holes, which will be filled later with uncrushed granular material as described in the INDOT *Standard Specifications*. This latter measure can be used effectively as the stiffness of the pile, hence the stresses are inversely proportional to the third power of the free-pile length.

Unless approved by the Director of Bridges, temperature movement shall not exceed the predetermined deflection limit at either end of a bridge supported with integral end bents.
402-7.02(04) Non-Integral End Bent

This consists of a semi-integral end bent or an end bent utilizing an expansion joint. This end bent type shall be used where an integral end bent is not feasible based on the criteria shown in Figure 409-2A. Further explanations of semi-integral-end-bent and expansion-end-bent usage are described in Chapter 409.

402-7.02(05) Abutment

A concrete abutment may be supported with either a spread footing or a deep foundation. It consists of a vertical stem which supports the superstructure by means of bearings with or without pedestals, or a mudwall which retains the embankment fill in the longitudinal direction of the bridge. It can support the end of a reinforced-concrete bridge approach. Wingwalls are usually required to retain the fill in the transverse direction. Continuity of the riding surface between the abutment and the superstructure is provided by means of a deck joint.

For restricted geometry, a tall superstructure, or large relative longitudinal movement between the superstructure and the substructure, the abutment may be the only feasible alternative. It is, however, expensive to construct. For a small bridge, its cost can be out of proportion with respect to other components of the bridge. With large abutments located close to the edge of roadway or waterway below, superstructure spans can be reduced. Large abutments, however, can result in poor aesthetics of the bridge, and can impair visibility at an overpass.

An abutment is affected by the bridge geometry and site conditions. Therefore, it can be designed in an infinite variety of shapes and sizes. Figure 402-7B indicates the parts of a typical cantilever abutment of rectangular layout supported with a spread footing. If the wingwalls are large, they can be directly supported with spread footings or footings with piles.

402-7.03 Pier or Frame Bent

The above-ground portion of a substructure can be categorized as illustrated in Figures 402-7C and 402-7D. These are discussed below. See Section 409 for more information on interior supports.

402-7.03(01) Pier

A pier is made of reinforced concrete. Where piers are directly exposed to public view, their appearance may be improved as discussed in Section 402-5.07.
The round column shown in Figure 402-7C detail (a) is the most economical, because it is structurally efficient and easy to construct.

The single, narrow wall shown in Figure 402-7C detail (d) is most suitable if its structural height is relatively small and the superstructure is a concrete slab; if the superstructure is made of longitudinally placed, precast concrete components; or if it includes closely spaced, longitudinal beams. For a greater structural height, a hammerhead pier, as shown in Figure 402-7D detail (b), either with a rectangular or rounded stem, is more suitable.

The use of twin walls shown in Figure 402-7C detail (e) permits the segmental construction of a medium-span superstructure made from longitudinal precast concrete components without falsework. A larger pier located in a waterway susceptible to ice accumulation may be fitted with a sharp icebreaker nose as shown in Figure 402-7C detail (f). A medium- or large-span, single-box superstructure may be supported by means of aesthetically-pleasing flared piers, as illustrated in Figure 402-7D detail (c). In a debris-prone stream, the wall-type pier is preferred.

402-7.03(02) Frame Bent

A frame bent, as shown in Figure 402-7E can be constructed from steel, concrete, or a combination of these materials. Steel is used only for a temporary structure due to of problems associated with corrosion, the environmental impact of repainting, vulnerability to collision, and the difficulty in providing an appropriate pier head. Steel is not the most competitive material for resisting force effects which are primarily compressive.

A concrete frame bent shall instead be used to support steel or concrete structural members. The columns of the bent can be either circular or rectangular in cross section. Circular columns are usually more economical. The columns shall be directly supported by the slab portion of a spread footing or by the pile cap.

Figure 402-7E detail (a) illustrates the most common type of concrete bent consisting of vertical columns and a cap beam, used in an overpass structure. Figure 402-7E detail (b) depicts a tall concrete bent which can be used in a cable-stayed bridge. Concrete can provide an economical and visually attractive substructure.

402-8.0 SUPERSTRUCTURE

This Section discusses the considerations in the selection of the superstructure type.
402-8.01 General

The State’s geography is relatively flat with predominately small waterways, therefore, the largest of the available bridge types is rarely appropriate. The bridge types provided in Figure 402-8A are those which are either traditional or which may have an application resulting from the introduction of the AASHTO LRFD Specifications.

A minimum of four beam lines is required for a multi-beam application on a State route. The minimum deck thickness is 8 in., including a 1/2-in. sacrificial wearing surface.

The following provides guidance in selecting the bridge-superstructure type that is most appropriate for the highway geometry, span lengths, and site conditions.

1. **Span Lengths.** Figure 402-8B indicates the typical ranges of span lengths for which each superstructure type will apply.

2. **Superstructure Depth.** See LRFD Table 2.5.2.6.3-1 for the traditional minimum depth for constant-depth superstructure for each structure type.

3. **Superstructure Characteristics:** Figure 402-8C tabulates basic characteristics of the superstructure types shown in Figure 402-5A.

402-8.02 Superstructure Type

402-8.02(01) Type A: Reinforced Cast-in-Place Concrete Slab

The reinforced cast-in-place concrete slab is used because of its suitability for short spans and its insensitivity to skewed or curved alignments. It is the simplest among all superstructure systems, as it is easy to construct. Structural continuity can be achieved without difficulty.

Haunching is used to decrease maximum positive moments in a continuous structure by means of attracting increased negative moments to the haunches and providing adequate resistance at the haunches for the increased negative moments. It is a simple, effective, and economical way to maximize the resistance of a thin concrete slab. As illustrated in Figure 402-8D, there are three ways of forming the haunch. The parabolic shape (a) is the most natural in terms of stress flow, and the most aesthetic. It is preferred where the elevation is frequently in view. The parabolic haunch, however, is difficult to form and, as alternatives, the straight haunch (b) and the drop panel (c) shall be considered where appropriate. The narrow pile cap (d), used in conjunction with an extended-pile substructure, does not qualify as an effective haunch.
Figure 402-8E depicts the elevation of a three-span, continuous-haunched slab bridge. The preferable ratio between interior span and end spans is approximately 1.25 to 1.33 for economy, but the system permits considerable freedom in selecting span ratio. The ratio between the depths at the centerlines of interior piers and at the point of maximum positive moment shall be between 2.0 and 2.5. Except for aesthetics, the length of the haunch shall not exceed the $kL$ values indicated in Figure 402-8D, where $L$ is the end span length. Longer haunches may be unnecessarily expensive or structurally counterproductive.

402-8.02(02) Type B: Longitudinally Post-Tensioned, Cast-in-Place Concrete Slab

The distinction between the type A and type B superstructures is the difference in how they are reinforced. Therefore, most of the information described above for type A is applicable.

A shallow post-tensioned slab can be a feasible structural system for a given situation. Structural analysis shows that if the haunch ratio is about 2.5, the ratio between maximum negative and positive moments is also approximately 2.5. This indicates that the amount of post-tensioning steel, as determined for positive moment, will be consistent with the requirements for negative moment, producing a balanced design. As an alternative, the right-hand side of the elevation in Figure 402-8F is shown with a constant-depth soffit. The constant-depth soffit does not produce a balanced design; therefore additional negative-moment reinforcement is required. This results in a reduction in span range and increases the probability of spalling by providing a large amount of reinforcing steel close to the surface.

By increasing the span-to-depth ratio to a maximum of 1:30 for simple spans and 1:40 for continuous spans, cost savings can be obtained in both superstructure and substructure. The use of the potentially-extreme ratio shall be made with consideration, as appropriate, for deflection performance and dynamic response.

There are two alternatives for transverse steel. One is with normal reinforcement, for which the requirements are the same as for a type A deck system. The second alternative is shown in Figure 402-8G which incorporates transverse post-tensioning. The cross section can be with or without cantilever overhangs. In the latter situation, two levels of post-tensioning instead of one can be used. As illustrated in Figure 402-8F, transverse tendons shall be fanned in the end zones of a skewed bridge. If the bridge railing is attached to a cantilever overhang at isolated points such as posts, both longitudinal and transverse reinforcement shall be provided therein. The problem that often rises with transverse post-tensioning on a deck of width of less than approximately 30 ft is the control of excessive seating losses, which makes the reinforced alternative preferable.
402-8.02(03) Type C: Longitudinally Post-Tensioned, Cast-in-Place Concrete Box Girders

This is a variation of type B, in which the deck system is considerably lighter and, therefore, more economical due to large, rectangular, and rhombic voids. This creates a multicell box-type superstructure, as illustrated in Figure 402-8H. This is often referred to as the California-type box girder. To facilitate the forming of a thin-walled box, the majority of these structures have a straight soffit. Consequently, considering longitudinal post-tensioning, this system is also unbalanced, requiring additional negative-moment steel. Full diaphragms are required at all interior piers and abutments. The preferred substructure type is the flared, rounded pier, which provides direct support for the two internal webs and provides the potential for a solid transverse moment connection, if required for seismic force effects.

For type A, B, or D, concrete is placed to full depth in a single operation. For type C, it is poured in three stages. First, the bottom slab is placed with dowels for connecting the web reinforcement as shown in Figure 402-8H. Next, the web steel is assembled, to which the rigid tendon ducts are attached, and then the web concrete is placed between removable forms. The last step is to construct a form for the top slab, assemble its reinforcement, and pour the concrete. Thus, a large structure can be built without the need for expensive machinery, if it is close to firm ground.

The system is suitable for an alignment with moderate curvature and skew. The structure is analyzed with the piers as a framed spine beam to obtain moment, shear, and torsion. For the latter, this system offers excellent resistance.

402-8.02(04) Type D: Two-Way Post-Tensioned, Cast-in-Place Concrete Spine-Beam with Cantilevers

Type D is most suitable for an excessively curved or skewed alignment, and is insensitive to the location of its piers. The cross section is a variation of type B in which the application of large, cantilever overhangs reduces the weight of the superstructure. Above a certain structural depth, it becomes economical to further reduce the structure weight with round voids which are formed by means of stay-in-place steel pipes.

The voided deck has a tendency to crack at the top near the centerline of the voids. To prevent the formation of cracks in the bridge, the top is transverse post-tensioned. This transverse post-tensioning lends itself to the formation of large cantilever overhangs and, thus, a dual use. This improved version, with or without voids, is illustrated in Figure 402-8I.
Considering span range, the type D system is a transition between the type B solid slab and type C cellular deck. Its cross section is not as effective as that of the cellular deck, but because its whole depth can be placed in one operation, it is less labor-intensive.

The largest void used is approximately 4 ft diameter, providing for a structural depth of approximately 5 ft. A narrow bridge, as illustrated in Figure 402-8 I, requires a minimum of two voids. As the core widens or the structural depth decreases, the voids will be more numerous but of lesser diameter. If the core-void ratio, with the area of the wings excluded, does not exceed approximately 30%, a solid cross section, as shown in Figure 402-8 I, shall be used. To be economical, the void ratio shall be approximately 35%. If the ratio exceeds 40%, the LRFD Specifications considers the deck as a cellular, or box, construction.

For the piers, slender, round columns may be used. For a short bridge such as an overpass, the columns may be framed into the superstructure. For a longer bridge, sliding bearings shall be applied. A long structure with flat horizontal curvature requires intermittently-located wide piers with two bearings to provide torsional stability. A sharply-curved structure has a high degree of inherent stability, therefore, stabilizing by means of two bearings or a line support, is required only at the abutments.

402-8.02(05) Type E: Prestressed, Precast Concrete Beams

Precast, prestressed concrete I-beams were initially adopted as AASHTO types II, III, and IV. Later, types I, V, and VI were added to extend their span range at both the lower and upper ends of the spectrum.

Currently, the AASHTO I-beam types I, II, III, and IV are used, along with the bulb-tee beams for longer spans. Prestressed, precast concrete box beams are also acceptable for a shallow construction depth. However, they shall not be placed either partially or entirely below the $Q_{100}$ elevation.

Figure 402-8J illustrates a typical superstructure cross section with prestressed, precast concrete I-beams.

For a preliminary selection of an I-beam size and spacing, see Figure 402-8K, Prestressed Concrete I-Beam Selection Chart. The slab overhang shall be as wide as possible but shall be in accordance with the overhang criteria provided in Chapter 404.

For a wide beam spacing, if foundation conditions permit, the beams can be individually supported by means of drilled shafts, as shown in Figure 402-7A detail (c), instead of a continuous pier cap.
Figure 402-8L illustrates three variations of prestressed, precast concrete box beams. Alternative (a) is an open-box cross section. Its advantage over a closed box is in the forming of the beam. Alternative (b) is a closed, or spread, box, with a constant depth, cast-in-place deck. Alternative (c) is a keyed-in design. For transverse continuity of the deck, it shall have the top reinforcement shown, but not dedicated shear connectors. All three alternatives can have beam spacings of up to approximately 15 ft.

Precast beams shall be continuous in the longitudinal direction for transient loads. In this arrangement, the beams retain their individual bearings, but their ends are incorporated in a common diaphragm which is cast monolithically with the deck.

The reinforcement for flexural continuity is located in the deck. The strands are extended into the diaphragm to prevent separation at the bottom that can occur as a result of the upward bowing of the prestressed beam due to creep. The tendency for creep can be minimized by permitting the beam concrete to mature prior to placing the deck. The extended strands also increase the shear resistance of the prestressed beams.

This system is adaptable without regard to skew. Horizontal curvature cannot easily be matched with a continuous structure, but only by means of a series of chorded spans laid out in a segmental form.

402-8.02(06) Type F: Bulb-Tee Beams Made Continuous by Means of Post-Tensioning

Precast beams can be made continuous for both permanent and transient gravity loads by means of the application of longitudinal post-tensioning. However, the scheme is provided herein in conjunction with the bulb-tee because it appears to offer the best structural efficiency for this type of construction with reference to its large bottom flange required to resist high negative moments in compression. This efficiency is achieved by means of a certain level of sophistication in construction technology which is within current practice, but not uniformly practiced nationwide.

Figure 402-8M illustrates the cross section of the Indiana bulb-tee beam. In addition to making the beam structurally effective, the wide top flange provides lateral stability, reduces the deck area to be formed, and furnishes a safe and comfortable walkway for the construction crew. The limits of practical hauling shall be considered from the plant to the work site in selecting span length and beam type and size. Fabricators shall be contacted early in project development for information regarding the feasibility of hauling to a specific site.
The trajectory of the tendons follows that shown in Figure 402-8N. The draped tendons shall be as close as practical to the outer fibers of the beam, as the structural effectiveness of a tendon is directly proportional to the vertical distance between its highest and lowest points. If the sidespan is identical with the internal one and if the end is butted by another structure, the tendon anchorages may be located in the top of the beam. This increases tendon efficiency to avoid congestion. See Figure 402-8N. In either situation, an end-block at the anchorage end of the end beam will be required. One structure shall not include more than four continuous spans.

Figure 402-8 O illustrates an intricate but convenient system of relatively large spans built from transportable, precast bulb-tee elements. The span element is the same as discussed above. The pier element is also the bulb-tee cross section but with a haunched soffit for improved negative-moment effectiveness. To avoid temporary falsework, thin-walled twin piers, as shown in Figure 402-7C detail (e) may be used. For temporary stability, each set of pier elements shall be joined together with four diaphragms and the deck. The span elements and the deck above them shall be constructed with two-stage post-tensioning. Both systems accept unlimited skew but no curvature.

402-8.02(07) Type G: Deck System with Prestressed, Precast Longitudinal Elements

Prestressed, precast concrete longitudinal members of various cross sections have been used to create a bridge deck. The performance of these deck systems has not always been desirable due to the disintegration of longitudinal shear keys between the members. The keys, unprotected by transverse pressure, become vulnerable due to shortening and warping of the prestressed members, and they fail to transfer live-load shear. This results in potential overloading of the members and in an irresolvable maintenance problem for the deck.

The beneficial effect of keeping the keys under transverse pressure had been recognized previously, and third-point transverse post-tensioning had been introduced as an option. Unfortunately, the combination of low prestress, the unmatched side surfaces of the members, and the quality of the keys, has rendered this improvement ineffective. The grout in the key is impossible to inspect due to the way the key is formed.

LRFD Specifications Article 5.14.4.3 requires a keyway joint of not less than 7 in. depth between the members. In lieu of the traditional key, the LRFD Specifications prefers a V-shaped joint which is easy to fill and convenient to inspect. The post-tensioning ducts shall be located at the mid-depth of the joint, and not at the mid-depth of the beam. The minimum transverse prestress across the joint is 250 psi, or 20 kip/ft of length, which is a force nearly twice the traditional value. If the deck is not transversely post-tensioned, it requires a structural overlay depth of not less than 4.5 in.
The current practice is as follows.

1. Precast members have a composite structural-concrete overlay with a minimum depth of 5 in.
2. Precast members are transversely post-tensioned whether or not a structural overlay is used.
3. A traditional trapezoidal key is used instead of a V-shaped key.
4. The wet-joint depth between elements is 8 in.

As illustrated in Figure 402-8P, there are four precast, prestressed concrete members which can be economically assembled into a simply-supported deck system. In descending order of span length, these are the single tee, the double tee, the box, and the solid slab. The *LRFD Specifications* also includes channel sections, but these will herein be considered as a double tee with truncated cantilever overhangs. The precast members either serve as the finished roadway or provide an uninterrupted formwork for a structural-concrete overlay.

The sections’ depth of the top flange, $d$, shall not be less than 6.5 in. if post-tensioned, and not less than 4 in. if overlaid. If the member is transported by truck, its width shall not exceed 8 ft. There is an incentive to decrease the number of joints in the deck, but this results in larger members. The transportability and erectability of the members shall be investigated considering both weight and geometry, early in project development, with potential contractors. Although the double tee has a less than perfect cross section considering structural efficiency, this is offset due to ease of forming.

For simplicity, only the box alternative is shown for the two methods of assembly as outlined above. Figure 402-8Q is applicable to all four sections. For a box or slab section, end diaphragms are not required. For a single- or double-tee section, end diaphragms are required. Since it is nearly impossible to manufacture perfectly matched precast members, the surface of the grouted and post-tensioned deck shall be ground, where necessary. For this, the minimum specified depth of the top flange shall be increased by 0.5 in.

No variation of this system is applicable to a curved alignment. Skew is possible, but forming and casting the ends of each member with an angle of other than 90 deg will cause difficulty in manufacturing. The desirable limit for skew is 30 deg. A skew angle of greater than 45 deg not permitted. This system can also be made continuous in the longitudinal direction by using a monolithic diaphragm and continuity steel or longitudinal post-tensioning similar to precast-concrete beams discussed above. A double tee, however, which lacks an effective bottom flange, requires other measures to improve the compressive strength of the stems at the point of junction.
402-8.02(08) Type H: Segmental Concrete Box

The use of a segmental concrete structure may be considered for the following:

1. a bridge with long spans;
2. a long bridge with medium-length spans and limited vertical and horizontal curvature designed with essentially identical precast segments; or
3. a sharply-curved bridge where cast-in-place operations are not permitted.

The metal formwork, especially built for each construction, is expensive, and one of the above considerations shall be used to justify the use of this structure type.

Figure 402-8R illustrates a typical cross section with a single cell. This type of superstructure has its own technical literature with reference to long spans. A summary appears in the AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges. The segments can be assembled by means of either span-by-span or balanced-cantilever methods, using either precast or cast-in-place concrete segments. The preferred method of longitudinal post-tensioning is by means of internal bonded tendons. However, longitudinal post-tensioning with unbonded tendons is also permitted. These are threaded through deviation blocks and anchored in the diaphragms of the adjacent spans. To avoid longitudinal cracking, the top slab shall be transversely post-tensioned.

The segmental interfaces shall be match-cast and shall include shear keys, which shall be bonded with epoxy adhesive.

402-8.02(09) Type I: Composite Steel Rolled Beam

Low structural depth, thick webs, and wide flanges characterize most of the steel beams rolled today. Therefore, most of these beams are considered compact sections that do not require intermediate web stiffeners and require minimum lateral support. They are not economical based only on least steel weight. Instead, economy is derived from low fabrication costs and minimal or sometimes nonexistent wind or sway bracing. Spacing of diaphragms shall be in accordance with Chapter 407.

The depth-to-span ratio for the beam plus slab shall not exceed 1:24 for simple spans or 1:33 for continuous spans. For continuous spans, the span is the distance between points of dead-load contraflexure.
Cover plates for rolled beams are prohibited on the flanges. Weathering, or unpainted, steel shall be used to lower the future maintenance cost. See Chapter 407 for steel-weight curves, which can be used to provide a preliminary estimate of steel weight. With proper diaphragms, this structure type is suitable for a skewed or horizontally-curved alignment. See Chapter 407 for more information.

See Figure 402-8S for a typical steel rolled beam section.

402-8.02(10) Type J: Composite Steel Plate Girder

Steel plate girders may be used in lieu of steel rolled beams for spans that are uneconomical, or not feasible, for that type of structure. The depth-to-span ratio for the girder plus slab shall not exceed 1:20 for simple spans or 1:28 for continuous spans. For continuous spans, the span is the distance between points of dead-load contraflexure. Weathering, or unpainted, steel shall be used to lower the future maintenance cost. See Chapter 407 for steel-weight curves, which can be used to provide a preliminary estimate of steel weight.

It is seldom economical to use the thinnest web plate permitted by LRFD. The use of a thicker web and few or no intermediate transverse or longitudinal stiffeners shall be investigated. For appearance, transverse stiffeners shall be located on the inside of the outside girders. With proper diaphragms, this structure type is suitable for a skewed or horizontally-curved alignment. See Chapter 407 for more information.

See Figure 402-8T for a typical composite steel plate girder section.

402-8.02(11) Type K: Composite Open Steel Box Girder

Single steel box girders, usually closed on the top due to an orthotropic steel deck, are used for large spans which are rarely required. As illustrated in Figure 402-8U, the lack of fatigue-prone sway and wind-bracing connections make the open steel box less susceptible to fatigue damage, and the use of two girders as a minimum is permitted. This system is adaptable to a curved alignment and where the available structural depth is limited. Moderate skew causes no problems.

During transportation and construction, the open box can require sway and internal, or wind, bracings. The mechanically-connected sway bracings shall be removed after construction. If too much torsional rigidity is provided to the boxes, they can cause longitudinal cracking in the slab, especially if the slab had been empirically designed. A minimum of two bearings per girder shall
be used to provide adequate torsional resistance. A solid diaphragm with an access hole shall be placed at all bearing points.

Transverse web stiffeners may be used, but the use of longitudinal web stiffeners shall be avoided.

402-8.02(12) Type L: Wood Superstructure

The use of a wood superstructure is limited to a low-volume, local road, and is subject to the approval of the Director of Bridges. See Chapter 413. A wood bridge can be an attractive alternative for a small span or a temporary bridge. Wood can be used either as a deck and or can be directly supported by wood trestles, or piers.

As illustrated in Figure 402-8V, there are two variations for use as a deck unsupported by other components. The deck may be constructed from wood panels, prefabricated either by gluing or spiking, which are of full-span length and are connected together by means of bolting spreader beams to the underside of the panels at intervals not exceeding 8 ft. The LRFD Specifications permits the use of this type of deck without spreader beams, but its use is not recommended. The deck shall have an asphaltic wearing surface. Because the use of this deck is limited to simply-supported spans, it is recommended for a rural or secondary road where a rectangular layout can easily be achieved.

Alternatively, the deck can be constructed from prebored longitudinal laminates which are laid out in a staggered design and assembled by means of transverse post-tensioning. The laminates are held together by means of interface friction, and no other fasteners are required. To improve rideability and surface friction, the use of an asphaltic surface treatment is mandatory. The system lends itself to continuous construction with limited curvature, but with unlimited skew.

Two other alternatives are also available for a transversely post-tensioned wood deck. One includes glued wood ribs, by which the flat deck is transformed into a series of tee-beams. The second is an extension of the first by the addition of a post-tensioned bottom flange by which a cellular cross section is created.

Either flat deck can also be used as a transversely-positioned slab supported with longitudinal beams. The post-tensioning will run longitudinally in the laminated deck.

As illustrated in Figure 402-8W, wood can be used also as a primary longitudinal component as either sawn or native, in closely-spaced stringers or widely-spaced glue-laminated beams. For composite construction, the concrete slab shall be keyed into the top of the wood component and
secured with spikes. Both alternatives permit skew, but neither permits curved alignment or continuous construction.

**402-8.02(13) Type M: Structure Under Fill**

This type of structure can be an attractive alternative for a small stream or ditch crossing, a minor highway or street crossing, or a pedestrian or animal crossing. This type of structure may be made of steel, aluminum, or concrete. The most common configurations used are the three-sided concrete or steel structure, four-sided precast concrete box structure, structural plate pipe arch, or circular pipe.

The structure-sizing process is performed in accordance with a priority system. This system consists of six trials where specific installations are considered prior to evaluating other structure types, such as a reinforced cast-in-place concrete slab. The design priority system is as follows:

Trial 1: single circular-pipe installation.
Trial 2: single deformed-pipe installation.
Trial 3: single specialty-structure installation.
Trial 4: multiple circular-pipe installation.
Trial 5: multiple deformed-pipe installation.
Trial 6: multiple specialty-structure installation.

The principles of the priority system are summarized below.

1. A pipe structure is preferred to a precast concrete box section, precast concrete three-sided structure, or structural plate arch.
2. A circular pipe is preferred to a deformed pipe.
3. A single-cell installation is preferred to a multiple-cell installation.

See Chapter 203 for more information on the culvert-sizing process. Additional considerations and design criteria for each type of buried-structure system are provided in *LRFD Bridge Design Specifications*, Section 12.

If a specialty-structure installation is selected, manufacturers of such structures shall be contacted. See Section 203-2.05 for more information.

**402-9.0 ALTERNATIVE DESIGN PROCESS**
For a bridge project with an estimated construction cost of over $10 million, the project manager and the designer shall investigate the possibility of performing a dual set of contract documents for two different structure types. The additional design fees for performing dual designs are minor in comparison to the results of a competitive contract letting. Designs for alternative structures shall be of equal safety, serviceability, and aesthetic value. Similarly, for foundations whose construction costs are expected to exceed $2 million, the project manager and the designer shall investigate the possibility of performing a dual set of foundation contract documents.

402.10 ACCELERATED CONSTRUCTION

The concept of accelerated construction is increasing in popularity and frequency of use. If it is determined that a project can benefit from an accelerated-construction method, this shall be coordinated with the project manager. This type of construction shall be evaluated individually for each project, and shall not be considered for each structure size and type analysis.
<table>
<thead>
<tr>
<th>Material</th>
<th>Superstructure Type</th>
<th>Typical Span Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precast Concrete</td>
<td>3-Sided Structure</td>
<td>12 – 48</td>
</tr>
<tr>
<td>Cast-in-Place Concrete</td>
<td>Continuous Reinforced Slab</td>
<td>20 – 45</td>
</tr>
<tr>
<td>Prestressed Concrete</td>
<td>Box Beams, Depth 12 in. through 27 in.</td>
<td>30 – 60</td>
</tr>
<tr>
<td></td>
<td>Box Beams, Depth 27 in. through 42 in.</td>
<td>60 – 85</td>
</tr>
<tr>
<td></td>
<td>I-Beams, AASHTO Type I</td>
<td>35 – 50</td>
</tr>
<tr>
<td></td>
<td>I-Beams, AASHTO Type II</td>
<td>40 – 65</td>
</tr>
<tr>
<td></td>
<td>I-Beams, AASHTO Type III</td>
<td>55 – 85</td>
</tr>
<tr>
<td></td>
<td>* I-Beams, AASHTO Type IV</td>
<td>70 – 110</td>
</tr>
<tr>
<td></td>
<td>Bulb-T Beams, Top-Flange Width 48 in. or 60 in.</td>
<td>80 – 140</td>
</tr>
<tr>
<td></td>
<td>Bulb-T Beams, Top-Flange Width 49 in. or 61 in.</td>
<td>65 – 165</td>
</tr>
<tr>
<td></td>
<td>Post-Tensioned Bulb-T Beams</td>
<td>140 – 200</td>
</tr>
<tr>
<td></td>
<td>Post-Tensioned Slab</td>
<td>50 – 80</td>
</tr>
<tr>
<td>Structural Steel</td>
<td>Steel Rolled Beams</td>
<td>&lt; 100</td>
</tr>
<tr>
<td></td>
<td>Steel Built-Up Plate Girders</td>
<td>&gt; 70</td>
</tr>
</tbody>
</table>

* These are generally used only in rehabilitating a structure. Bulb-T beams are preferred for a new or replacement structure.

**ECONOMICAL STRUCTURE-TYPE SELECTION**

*Figure 402-5A*
NOTE: Median pier protection should be provided.
BRIDGE UNDERPASS CROSS SECTION
NEW CONSTRUCTION / 4R PROJECT

Figure 402-6B
NEW BRIDGE UNDERPASS CROSS SECTION
3R PROJECT

Figure 402-6C
TWO-LANE, TWO-WAY HIGHWAY BRIDGE CROSS SECTION

Clear Roadway Width

Traveled Way

Shoulder width plus railing offset or curb offset
(See Fig. 402-6H)

Inside face of railing or curb

2 %

Inside face of railing or curb

Shoulder width plus railing offset or curb offset
(See Fig. 402-6H)

BRIDGE CROSS SECTION
TWO-LANE, TWO-WAY HIGHWAY

Figure 402-6D
A raised island or CMB may be warranted. See Chapter 55 for criteria on median width for a single structure.

BRIDGE CROSS SECTION
DIVIDED HIGHWAY - SINGLE STRUCTURE

Figure 402-6E
BRIDGE CROSS SECTION
TWIN STRUCTURES - FOUR LANES

Figure 402-6F
NOTE: For three lanes in one direction, the crown will be between the middle travel lane and the travel lane adjacent to the median. For four lanes in one direction, the crown will be in the center of the traveled way.

BRIDGE CROSS SECTION
TWIN STRUCTURES - SIX OR MORE LANES

Figure 402-6G
GUARDRAIL TRANSITION TO BRIDGE RAILING

BRIDGE-RAILING OFFSET

GUARDRAIL TRANSITION TO BRIDGE RAILING

Figure 402-6H
(Page 1 of 2)
BRIDGE RAILING TRANSITION TYPE TFC OR TFT

BRIDGE RAILING TRANSITION TYPE TPF, TPS, OR TTX

BRIDGE RAILING TRANSITION TYPE TTF-2

Bridge Railing Offset = Guardrail Offset or Reduced Guardrail Offset in Restricted Condition + Offset Gain (+) or Offset Loss (-)

Example: Guardrail Offset of 2'-0" on the bridge approach, and Bridge Railing Type FC. 4" of Railing Offset is lost through the Bridge Railing Transition Type TFC.

Bridge Railing Offset = (2'-0") + (- 4") = 1'-8"

BRIDGE-RAILING OFFSET
GUARDRAIL TRANSITION TO BRIDGE RAILING

Figure 402-6H
(Page 2 of 2)
FLAT OR SHORT HORIZONTAL CURVE

Clear Roadway Width

\[ \leq A + 1'0" \]

* Additional width in excess of 1'-0" is allowed if shown to be economical.

NOTE: See Chapter 55 for bridge width.

BRIDGE WIDTH

FLAT OR SHORT HORIZONTAL CURVE

Figure 402-6 I
<table>
<thead>
<tr>
<th>Type</th>
<th>Minimum Vertical Clearance (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Railroad Under Roadway</td>
<td>23’-0” (1)</td>
</tr>
<tr>
<td>Roadway Under Pedestrian Bridge</td>
<td>17’-6” (2)</td>
</tr>
<tr>
<td>Freeway Under Roadway</td>
<td>16’-6” (2) (3)</td>
</tr>
<tr>
<td>Arterial Under Roadway</td>
<td>16’-6” (2) (4)</td>
</tr>
<tr>
<td>Collector Under Roadway</td>
<td>14’-6” (2)</td>
</tr>
<tr>
<td>Local Facility Under Roadway</td>
<td>14’-6” (2)</td>
</tr>
<tr>
<td>Non-Motorized-Vehicle-Use Facility Under Bridge</td>
<td>10’-0” (5)</td>
</tr>
</tbody>
</table>

**Notes:**

(1) *See Chapter 413 for additional information on railroad under roadway.*

(2) *Value allows 6 in. for future resurfacing.*

(3) *A 14’-6” clearance, which includes provision for future resurfacing, may be used in an urban area where an alternative freeway facility with a 16’-0” clearance is available.*

(4) *In a highly-urbanized area, a minimum clearance of 14’-6”, which includes provision for future resurfacing, may be provided if there is at least one route available with a 16’-0” clearance.*

(5) *Value allows for clearance of a maintenance or emergency vehicle.*

**VERTICAL CLEARANCE**

*Figure 402-6J*
LOW SIDE

Deck

Shoulder Line

6" min.

2:1 max.

Berm

3'-0"

HIGH SIDE

Deck

Shoulder Line

1'-8" max.

2:1 max.

Berm

3'-0"

Deck

END-BENT BERM

Figure 402-6K
STANDARD SUPERELEVATION TRANSITION FOR TWO-LANE ROADWAY
NOT DESIRABLE

MODIFIED SUPERELEVATION TRANSITION FOR TWO-LANE ROADWAY

ALTERNATIVE MODIFIED SUPERELEVATION TRANSITION FOR TWO-LANE ROADWAY

SUPERELEVATION TRANSITION DIAGRAM FOR BRIDGE

Figure 402-6L
SECTION A-A

PERPENDICULAR TO CHANNEL

A  = One half of the cap width
B  = Width of berm
C  = Anticipated thickness of reinforced concrete slab
D  = Distance from bottom of slab to berm elevation
E  = (2) (El. A - C - D - El. C)
F  = (2) (El. B - C - D - El. C)
W  = Width of channel perpendicular to channel centerline
El. A = Elevation of top of slab
El. B = Elevation of top of slab
El. C = Bottom of channel elevation

NOTE: Waterway Area Required will be determined in the waterway opening analysis. Interior supports are not shown.

STRUCTURE LENGTH FOR STREAM CROSSING
REINFORCED CONCRETE SLAB STRUCTURE

Figure 402-6M
SECTION A-A
PARALLEL TO θ STRUCTURE

A = (Distance from bearing to front face of cap) / cos θ
B = (Width of berm) / cos θ
C = Construction depth plus height of bearing pad
D = Distance from top of cap to berm elevation
E = (2) (El. A - C - D - El. C) / cos θ
F = (2) (El. B - C - D - El. D) / cos θ
W = Width of traveled way plus width of obstruction-free or clear zone
El. A = Elevation of top of slab
El. B = Elevation of top of slab
El. C = Elevation of toe of slope
El. D = Elevation of toe of slope

NOTE: Interior supports are not shown.

STRUCTURE LENGTH FOR HIGHWAY CROSSING BEAM-TYPE SUPERSTRUCTURE

Figure 402-6N
NOTES:
1. FHWA limits their fiscal participation to horizontal clearance up to 20 ft. When a Railroad requests additional horizontal clearance for the accommodation of a planned future track, maintenance access roadway, or for drainage purposes, the railroad must provide justification to INDOT for approval in accordance with 23 CFR 646.212 and 646.214.
2. 23'-0" minimum vertical clearance measured 6'-0" from centerline of existing or future track.
3. A crashwall is required if horizontal clearance is less than 25 ft to face of pier or MSE wall.
4. Provide 15'-0" between centerlines for future track(s).
5. Horizontal dimensions are measured perpendicular to the track and may vary by Railroad.

TYPICAL HORIZONTAL AND VERTICAL CLEARANCES FOR RAILROAD GRADE SEPARATION

Figure 402-6 O
1. Where a sidewalk and curb are poured monolithically, the sidewalk width for the purpose of determining ADA compliance will be the monolithic sidewalk and curb width minus an assumed curb width of 6 inches.

2. See Section 51-1.03(02) for sidewalk width, sidewalk cross slope and passing space criteria.

BRIDGE SIDEWALK WIDTH

Figure 402-6P
BASIC BENT TYPES

Figure 402-7A

(a) PIER OR FRAME BENT SUPPORTED WITH SPREAD FOOTING

(b) PIER OR FRAME BENT SUPPORTED WITH DEEP FOUNDATIONS

(c) EXTENDED PILE BENT
CANTILEVER ABUTMENT

Figure 402-7B
PIER STEM AND COLUMN CONFIGURATIONS PLAN VIEWS

Figure 402-7C
STEM TYPES FOR PIERS

(a) SINGLE WALL PIER
(b) HAMMERHEAD PIER
(c) FLARED PIER

Figure 402-7D
FRAME BENTS

Figure 402-7E

(a) LOW CONCRETE BENT

(b) TALL CONCRETE BENT
<table>
<thead>
<tr>
<th>Type</th>
<th>Structure Description</th>
<th>Cross Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Reinforced, Cast-in-Place Concrete Slab</td>
<td>![Diagram A]</td>
</tr>
<tr>
<td>B</td>
<td>Longitudinally Post-Tensioned, Cast-In-Place Concrete Slab</td>
<td>![Diagram B]</td>
</tr>
<tr>
<td>C</td>
<td>Longitudinally Post-Tensioned, Cast-In-Place Concrete Box Girders</td>
<td>![Diagram C]</td>
</tr>
<tr>
<td>D1</td>
<td>Two-Way Post-Tensioned, Cast-In-Place, Solid Concrete Spine-Beam with Cantilevers</td>
<td>![Diagram D1]</td>
</tr>
<tr>
<td>D2</td>
<td>Two-Way Post-Tensioned, Cast-In-Place, Voided Concrete Spine-Beam with Cantilevers</td>
<td>![Diagram D2]</td>
</tr>
<tr>
<td>E1</td>
<td>Prestressed Precast Concrete I-Beams or Bulb-Tees</td>
<td>![Diagram E1]</td>
</tr>
<tr>
<td>E2</td>
<td>Prestressed Precast Concrete Open or Closed Box Beams</td>
<td>![Diagram E2]</td>
</tr>
<tr>
<td>Type</td>
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</tr>
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<td>------</td>
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<td>---------------</td>
</tr>
<tr>
<td>F</td>
<td>Post-Tensioned Concrete Bulb-Tee Beams</td>
<td><img src="image1.png" alt="Cross Section" /></td>
</tr>
<tr>
<td>G1</td>
<td>Jointed, Prestressed, Precast Longitudinal Concrete Single Tees</td>
<td><img src="image2.png" alt="Cross Section" /></td>
</tr>
<tr>
<td>G2</td>
<td>Jointed, Prestressed, Precast Longitudinal Concrete Double Tees</td>
<td><img src="image3.png" alt="Cross Section" /></td>
</tr>
<tr>
<td>G3</td>
<td>Jointed, Prestressed, Precast Longitudinal Concrete Boxes</td>
<td><img src="image4.png" alt="Cross Section" /></td>
</tr>
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<td>G4</td>
<td>Jointed, Prestressed, Precast Longitudinal Concrete Slabs</td>
<td><img src="image5.png" alt="Cross Section" /></td>
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<td>H</td>
<td>Segmental Concrete Box Girders</td>
<td><img src="image6.png" alt="Cross Section" /></td>
</tr>
<tr>
<td>I</td>
<td>Composite Steel Rolled Beams</td>
<td><img src="image7.png" alt="Cross Section" /></td>
</tr>
<tr>
<td>Type</td>
<td>Structure Description</td>
<td>Cross Section</td>
</tr>
<tr>
<td>------</td>
<td>------------------------------------------</td>
<td>---------------</td>
</tr>
<tr>
<td>J</td>
<td>Composite Steel Plate Girders</td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>Composite Steel Boxes</td>
<td></td>
</tr>
<tr>
<td>L1</td>
<td>Wood Panel Decks with Spreader Beams</td>
<td></td>
</tr>
<tr>
<td>L2</td>
<td>Stressed Wood Decks</td>
<td></td>
</tr>
<tr>
<td>L3</td>
<td>Composite Native Wood Stringers</td>
<td></td>
</tr>
<tr>
<td>L4</td>
<td>Glulam Beams</td>
<td></td>
</tr>
<tr>
<td>M</td>
<td>Structure Under Fill</td>
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</table>

**SUPERSTRUCTURE TYPES**

Figure 402-8A

(Page 3 of 3)
<table>
<thead>
<tr>
<th>Type</th>
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<th>Subgroup</th>
<th>Range (ft)</th>
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<td></td>
<td>&lt; 30</td>
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<td>A</td>
<td>Reinforced, Cast-in-Place Concrete Slab</td>
<td>Straight</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Haunched</td>
<td>X</td>
</tr>
<tr>
<td>B</td>
<td>Longitudinally Post-Tensioned, Cast-in-Place Concrete Slab</td>
<td>Straight</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Haunched</td>
<td>X</td>
</tr>
<tr>
<td>C</td>
<td>Longitudinally Post-Tensioned, Cast-in-Place Concrete Box Girder</td>
<td>n/a</td>
<td>X</td>
</tr>
<tr>
<td>D</td>
<td>Two-Way Post-Tensioned, Cast-in-Place Concrete Spine-Beams with Cantilevers</td>
<td>1. Solid</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Voided</td>
<td>X</td>
</tr>
<tr>
<td>E</td>
<td>Prestressed, Precast Concrete Beams</td>
<td>1. I-Beams</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Bulb-Tee Beams</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3. Boxes</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Straight</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Haunched</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>Post-Tensioned, Bulb-Tee Beams</td>
<td>1. Single Tees</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Double Tees</td>
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<td>3. Boxes</td>
<td>X</td>
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<td></td>
<td></td>
<td>4. Solid Slabs</td>
<td>X</td>
</tr>
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<td>G</td>
<td>Jointed Prestressed Precast Longitudinal Concrete Elements</td>
<td>1. Panel Deck</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Stressed Deck</td>
<td>X</td>
</tr>
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<td></td>
<td></td>
<td>3. Stringers</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4. Glulam Beams</td>
<td>X</td>
</tr>
<tr>
<td>H</td>
<td>Segmental Concrete Box Girders</td>
<td>n/a</td>
<td>X</td>
</tr>
<tr>
<td>I</td>
<td>Composite Steel Rolled Beams</td>
<td>n/a</td>
<td>X</td>
</tr>
<tr>
<td>J</td>
<td>Composite Steel Plate Girders</td>
<td>n/a</td>
<td>X</td>
</tr>
<tr>
<td>K</td>
<td>Composite Steel Box Girders</td>
<td>n/a</td>
<td>X</td>
</tr>
<tr>
<td>L</td>
<td>Wood Structure</td>
<td>1. Panel Deck</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Stressed Deck</td>
<td>X</td>
</tr>
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<td></td>
<td>3. Stringers</td>
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</tr>
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<td></td>
<td></td>
<td>4. Glulam Beams</td>
<td>X</td>
</tr>
<tr>
<td>M</td>
<td>Structure Under Fill</td>
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<td>X</td>
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**SPAN LENGTHS**

*Figure 402-8B*
<table>
<thead>
<tr>
<th>Type</th>
<th>Structure Description</th>
<th>Subgroup</th>
<th>For Skew</th>
<th>For Horiz. Curve</th>
<th>Aesthetics</th>
<th>False-work</th>
<th>Speed of Construction</th>
<th>Maintenance</th>
<th>Widening</th>
</tr>
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<tbody>
<tr>
<td>A</td>
<td>Reinforced, cast-in-place concrete slab</td>
<td>Straight</td>
<td>Good</td>
<td>Good</td>
<td>OK</td>
<td>Yes</td>
<td>Slow</td>
<td>Good</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Haunched</td>
<td>Good</td>
<td>Good</td>
<td>OK</td>
<td>Yes</td>
<td>Slow</td>
<td>Good</td>
<td>OK</td>
</tr>
<tr>
<td>B</td>
<td>Longitudinally post-tensioned cast-in-place concrete slab</td>
<td>Straight</td>
<td>Good</td>
<td>Good</td>
<td>OK</td>
<td>Yes</td>
<td>Slow</td>
<td>Good</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Haunched</td>
<td>Good</td>
<td>Good</td>
<td>OK</td>
<td>Yes</td>
<td>Slow</td>
<td>Good</td>
<td>OK</td>
</tr>
<tr>
<td>C</td>
<td>Longitudinally post-tensioned cast-in-place concrete slab</td>
<td>N/A</td>
<td>OK</td>
<td>OK</td>
<td>Good</td>
<td>Yes</td>
<td>Slow</td>
<td>Good</td>
<td>No</td>
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<td>D</td>
<td>2-way post-tensioned, cast-in-place concrete spine-bms.</td>
<td>1. Solid</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td>Yes</td>
<td>Slow</td>
<td>Good</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>with cantilevers</td>
<td>2. Voided</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
<td>Yes</td>
<td>Slow</td>
<td>Good</td>
<td>No</td>
</tr>
<tr>
<td>E</td>
<td>Prestressed precast concrete beams</td>
<td>1. I-Bms, Bulb-Ts</td>
<td>OK</td>
<td>Poor</td>
<td>OK</td>
<td>No</td>
<td>OK</td>
<td>Good</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Boxes</td>
<td>OK</td>
<td>Poor</td>
<td>OK</td>
<td>No</td>
<td>OK</td>
<td>Good</td>
<td>OK</td>
</tr>
<tr>
<td>F</td>
<td>Post-tensioned bulb-tee beams</td>
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<td>No</td>
<td>OK</td>
<td>No</td>
<td>OK</td>
<td>Good</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Haunched</td>
<td>OK</td>
<td>No</td>
<td>Good</td>
<td>No</td>
<td>OK</td>
<td>Good</td>
<td>OK</td>
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<td>G</td>
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<td>1. Single tees</td>
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<td>Poor</td>
<td>OK</td>
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<td>Good</td>
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<tr>
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<td>longitudinal concrete elements</td>
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<td>Poor</td>
<td>OK</td>
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<td>Good</td>
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<td>3. Boxes</td>
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<td>Poor</td>
<td>OK</td>
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<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
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<td></td>
<td>4. Solid slab</td>
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<td>Good</td>
<td>Good</td>
</tr>
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<td>Segmental concrete box girders</td>
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<td>OK</td>
<td>Yes/No</td>
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<td>Good</td>
<td>No</td>
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<tr>
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<td>Composite steel rolled beams</td>
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<td>OK</td>
<td>No</td>
<td>OK</td>
<td>Expensive*</td>
<td>OK</td>
</tr>
<tr>
<td>J</td>
<td>Composite steel plate girders</td>
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<td>OK</td>
<td>Good</td>
<td>OK</td>
<td>No</td>
<td>OK</td>
<td>Expensive*</td>
<td>OK</td>
</tr>
<tr>
<td>K</td>
<td>Composite steel box girders</td>
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<td>OK</td>
<td>Good</td>
<td>No</td>
<td>OK</td>
<td>Expensive*</td>
<td>OK</td>
<td></td>
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<tr>
<td>L</td>
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<td>1. Panel deck</td>
<td>Good</td>
<td>No</td>
<td>OK</td>
<td>No</td>
<td>Good</td>
<td>OK</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2. Stressed deck</td>
<td>Good</td>
<td>Poor</td>
<td>OK</td>
<td>No</td>
<td>Good</td>
<td>OK</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3. Stringers</td>
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<td>No</td>
<td>Good</td>
<td>OK</td>
<td>OK</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4. Glulam beams</td>
<td>Good</td>
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<td>OK</td>
<td>No</td>
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<td>OK</td>
<td>OK</td>
</tr>
<tr>
<td>M</td>
<td>Structure under fill</td>
<td>N/A</td>
<td>OK</td>
<td>Good</td>
<td>OK</td>
<td>No</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
</tbody>
</table>

* Expensive if painted. Good if unpainted.
TYPE A, REINFORCED CONCRETE SLAB SUPERSTRUCTURE

Figure 402-8D
This configuration should not be used as a structural haunch.

TYPE A, HAUNCH CONFIGURATIONS FOR REINFORCED CONCRETE SLAB BRIDGE

Figure 402-8E
TYPE B, ALTERNATIVES FOR TRANSVERSE POST-TENSIONING

Figure 402-8F
Anchorage steel cage not shown for clarity

Additional reinforcement

Draped Tendons

Steel support for ducts (typ.)

ELEVATION

PLAN

RECTANGULAR, SKEWED, OR CURVED LAYOUT

TYPE B, POST-TENSIONED CONCRETE SLAB

Figure 402-8G
TYPE C, CALIFORNIA-TYPE BOX GIRDER

Figure 402-8H
Transverse Post-Tensioning

Cantilever reinforcement is not shown for clarity

Draped Tendons

Steel Void Former

Deck System

Core Width

TYPE D1 TYPE D2

Figure 402-8 I

TYPE D, TWO-WAY POST-TENSIONED CAST-IN-PLACE CONCRETE SPINE-BEAM WITH CANTILEVER
TYPE E1, COMPOSITE DECK WITH PRESTRESSED, PRECAST CONCRETE BEAMS

Figure 402-8J
PRESTRESSED CONCRETE I-BEAM SELECTION CHART

Figure 402-8K
TYPE E2, COMPOSITE DECK WITH PRESTRESSED, PRECAST CONCRETE BOX BEAMS

Figure 402-8L
TYPE F, INDIANA BULB-TEE BEAM

Figure 402-8M
TYPE F, TOP ANCHORAGE FOR LONGITUDINAL POST-TENSIONING

Figure 402-8N
TYPE F, LARGE-SPAN BRIDGE
CONSTRUCTED FROM LONGITUDINAL PRECAST-CONCRETE BEAM ELEMENTS

Figure 402-8 O
TYPE G, ALTERNATIVE SECTIONS FOR PRECAST CONCRETE MEMBERS

Figure 402-8P
TYPE G, ASSEMBLY OF PRECAST-CONCRETE MEMBERS

Figure 402-8Q
TYPE H, TYPICAL CROSS SECTION FOR SEGMENTAL CONSTRUCTION

Figure 402-8R
TYPE I, TYPICAL CROSS SECTION WITH COMPOSITE STEEL ROLLED BEAMS

Figure 402-8 S
TYPE J, TYPICAL CROSS SECTION WITH COMPOSITE STEEL PLATE GIRDERS

Figure 402-8T
TYPE K, TYPICAL CROSS SECTION WITH COMPOSITE STEEL BOX GIRDERS

Figure 402-8U
TYPE L, TYPICAL CROSS SECTION FOR LAMINATED WOOD DECK

Figure 402-8V
Figure 402-8W

**TYPE L, TYPICAL CROSS SECTION WITH WOOD BEAMS**
CHAPTER 70

Bridge Design Operational Information

NOTE: This chapter is currently being re-written and its content will be included in Chapter 402 in the future.
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  70-1.03 Construction, Maintenance, and Inspection Manual .................................................... 4

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<tr>
<th>Figure</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>70-1A</td>
<td>Sample Outline of a Construction, Maintenance, and Inspection Manual</td>
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</tbody>
</table>
CHAPTER 70

BRIDGE DESIGN
OPERATIONAL INFORMATION

70-1.0 MAJOR OR COMPLEX BRIDGE

70-1.01 Major Bridge

A Major INDOT Bridge consists of one of the structure types as follows:

1. cable-stayed;
2. moveable;
3. owned by others (i.e., private, DNR, etc.) but inventoried by INDOT;
4. that which includes pin-and-hanger connections;
5. that which includes post-tensioned members or elements;
6. single structure with deck area greater than 25,000 ft$^2$;
7. state-line crossing at Ohio River or Wabash River; or
8. twin structure with combined deck area greater than 50,000 ft$^2$.

70-1.02 Complex Bridge

A Complex INDOT / County Bridge consists of one of the structure types as follows:

1. cable-stayed;
2. that which includes curved steel beams or girders;
3. moveable;
4. open-spandrel arch;
5. that which includes pin-and-hanger connections;
6. that which includes post-tensioned members or elements;
7. state-line crossing at Ohio River or Wabash River;
8. that which includes steel box girders or pier caps;
9. suspension;
10. thru-truss with four or more main spans; or
11. truss with pin-and-eyebar connections.
70-1.03 Construction, Maintenance, and Inspection Manual

A Construction, Maintenance, and Inspection Manual may need to be developed or updated for each Major Bridge, or Complex Bridge, when built or if undergoing a major rehabilitation. The lists of existing Major Bridges and Complex Bridges are available from the Planning Division’s Office of Technical Services bridge inspection program manager. Such a manual should be developed unless otherwise instructed by the Planning Division’s Office of Technical Services bridge inspection program manager, or the Production Management Division’s Office of Structural Services manager.

A draft version of the manual should be developed prior to the structure’s opening to unrestricted traffic. The manual should include similar information as listed in Figure 70-1A, Sample Outline of a Construction, Maintenance, and Inspection Manual, but tailored to the specific bridge type or complex details. Work on the manual should be started during the early stages of the design of the bridge so that all required items are included and the long-term inspection and maintenance can be considered and incorporated into the design and construction.

A bridge inspection should be conducted by the designer, the project engineer and his or her construction inspectors, the district bridge inspection engineer, and the county bridge engineer if a local-agency project, prior to the structure’s opening to unrestricted traffic. The designer should make all of the arrangements for this joint inspection. A draft copy of the manual should be complete and provided for review to each attendee, prior to the inspection. All comments from the inspection along with minutes should be included in the final manual. The initial NBIS inspection should also be incorporated into a final version of the manual.

The Planning Division’s Office of Technical Services bridge inspection program manager will approve the final manual as complete. Once finalized, one copy each of the manual should be transmitted to the Planning Division’s Office of Technical Services bridge inspection program manager, the INDOT district bridge inspection engineer, and the county bridge engineer if a local-agency project.
Post-Tensioned Concrete Slab Bridge

1.0 Bridge Description
   1.1 Superstructure Slab and Post-Tensioning
   1.2 Railings
   1.3 Substructure
   1.4 Bearings

2.0 Design Records
   2.1 Superstructure
   2.2 Substructures
   2.3 Design Records
   2.4 Post-Tensioning Shop Drawings

3.0 Construction Records
   3.1 Falsework Design Calculations
   3.2 As-Built Plans
   3.3 Material Records
      3.3.1 Concrete compressive strength
      3.3.2 Admixtures
      3.3.3 Post-Tensioning ducts
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CHAPTER 403

LOAD ANALYSIS AND APPLICATION

403-1.0 GENERAL [REV. FEB. 2018]

The Department’s practice is to design new and replacement structures using the current edition of the AASHTO LRFD Bridge Design Specifications (LRFD). For design related to bridge preservation, see Chapter 412.

403-1.01 Introduction

This Chapter describes the loads to be applied to bridge structures.

The following summarizes the discussion of structural loads and force effects.

1. Permanent Loads. This consists of the application of alternative sets of load factors specified for permanent loads and for imposed deformations.

2. Gravitational Live Load. This Chapter provides a treatment on vehicular live loads with reference to the following:

   a. the live-load regime as a tandem or truck coincident with a uniformly distributed load, as specified in the LRFD; and

   b. a description of heavy vehicles permitted to operate in the State for which certain bridges shall be investigated.

3. Creep, Shrinkage, and Temperature. The use of alternative load factors, introduced by the LRFD for the effects of creep, shrinkage, and uniform temperature, is discussed. See Section 403-5.02.

4. Earthquake. Section 403-3.05 discusses earthquake effects.

5. Ice. Section 403-3.06 discusses ice forces on a pier.
403-1.02 Limit States

LRFD 1.3.1 states that bridges shall be designed for specified limit states to achieve the objectives of constructibility, safety and serviceability, with due regard to issues of inspectability, economy and aesthetics. Through this requirement, the Specifications expand the traditional family of design objectives of constructibility, safety, and economy by means of concerns for maintenance and social issues. The first relates to the Specifications’ requirement of 75 years for a reasonably trouble-free service life, and the second reflects the pleasure and comfort of the highway user. LRFD Section 2 provides guidance on how the 75-year target service life can be achieved, and thus emphasizes the significance of non-strength issues.

For the purpose of this Chapter, the extreme-limit state applies to both maximum and minimum loads. Components and connections of a bridge are designed for strength, or derivatives of strength, at various limit states. The basic design relationship between load effects and structural performance for all limit states is as follows:

\[
\sum \eta_i \gamma_i Q_i \leq \phi R_n
\]  
(Equation 403-1.1)

Where:

- \( \gamma_i \) = load factor
- \( Q_i \) = load or force effect
- \( \phi \) = resistance factor
- \( R_n \) = nominal resistance

For a load for which a maximum value of \( \gamma_i \) is appropriate,

\[
\eta_i = \eta_D \eta_R \eta_I \geq 0.95
\]

For a load for which a minimum value of \( \gamma_i \) is appropriate,

\[
\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0
\]

where \( \eta_D, \eta_R, \eta_I \) are load modifiers relating to ductility, redundancy, and operational importance, respectively.

The left-hand side is the sum of the factored load, or force, effects of a type of effect acting on a component, and the right-hand side is the factored nominal resistance of the component for the type of effect. Where various types of force effects interact at a section of a component, e.g., shear and moment in a concrete beam, or where a load produces both force effect and resistance,
e.g., fill behind a retaining wall, either special interaction formulas are provided in the *LRFD Specifications* or the effects are artificially separated for design.

The strength-limit-state factors to be used are as follows:

\[
\begin{align*}
\eta_D &= 1.05 \text{ for components subject to brittle failure} \\
\eta_D &= 1.00 \text{ for conventional design in accordance with the *LRFD Specifications*} \\
\eta_R &= 1.05 \text{ for a simple span with non-integral supports or non-redundant structure} \\
\eta_R &= 1.00 \text{ for other type of bridge} \\
\eta_I &= 1.05 \text{ for a National Highway System bridge, or a bridge which provides single access to a military base, medical facility, generating station, or a considerable population} \\
\eta_I &= 0.95 \text{ for a highway classified as a local road or street} \\
\eta_I &= 1.00 \text{ for a bridge on another type of highway}
\end{align*}
\]

In addition to the *LRFD Specifications*, the following shall apply to the application of limit states.

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Investigation</th>
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<tr>
<td>Strength II</td>
<td>If special-permit vehicles, such as trucks carrying large transformers are anticipated, they shall be analyzed under this limit state. Wind load need not be considered.</td>
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</table>

### 403-2.0 PERMANENT LOADS

#### 403-2.01 General Requirements

*LRFD* 3.5 specifies the types of permanent loads, which are either direct gravity loads or those caused by gravity loads. These now include downdrag, *DD*, which is the result of soil consolidation around a deep foundation. Prestressing is considered part of resistance and has been omitted from the list of permanent loads shown in *LRFD* 3.5. However, in designing anchorage blocks and evaluating shear resistance, the prestressing force is contributing to load effects. It can sometimes be the dominating load.

As shown in *LRFD* Table 3.4.1-2, there are two sets of load factors for permanent loads. They shall be applied where the sum of force effects can be both positive and negative. For example, this situation can occur in the end bearing design of a continuous superstructure with relatively
shortened spans. Where the transient live load is in the end span, it causes compression and, if in the second span, uplift. The following combinations shall be considered in this situation.

1. If dead-load reaction is compressive, for extreme compression use the maximum load factor. For extreme uplift, use the minimum load factor.

2. If dead-load reaction is tensile, for extreme compression use the minimum load factor. For extreme uplift, use the maximum load factor.

The load factor for a given loading situation shall be the same for all spans.

403-2.02 Uplift

Uplift had been formerly treated as a separate loading situation. With the introduction of variable load factors, uplift has been reduced to one of the load combinations.

403-2.03 Concrete Deck Slab

If a concrete deck slab is placed on stay-in-place corrugated metal formwork as described in LRFD 9.7.4, the specified net concrete design section shall be taken from the top of the form. The design dead load shall include 15 psf of deck area for a deck formed with permanent metal forms to accommodate the weight of the forms and of the concrete in the valleys of the forms’ corrugations.

Clear spans between girders or beams exceeding 9.5 ft will require metal forms with the corrugations closed off, which prohibit concrete from entering the valleys of the corrugations. The girders or beams, however, shall still be designed for 15 psf of deck area.

Although permanent metal deck forms, which provide a dead load of less than 15 psf, are available, the 15 psf shall be retained as the minimum design load. In addition to the dead load of the initial structure, the design dead load shall be increased by 35 psf for a future wearing surface or overlay.

403-2.04 Utilities

Information shall be obtained concerning the weight and location of utilities that may be attached to the bridge.
403-2.05 Dead-Load Values

Figure 403-2A lists typical dead-load values.

403-2.06 Dead-Load Distribution

Typical practices for distributing dead load to beams or girders are as follows.

1. Future-wearing-surface load shall be applied equally to all beams.

2. Dead loads due to barrier railings, curbs, sidewalks, or other attachments (structural or aesthetic) placed after the deck has set, shall be distributed with 60% of the load applied to the exterior beam and 40% of the load applied to the first adjacent interior beam. The beams shall also be checked with the loads distributed equally to all beams.

3. Concrete dead load of deck applied to the outside girder shall be in accordance with the lever rule described in LRFD 4.6.2.2.1.

4. The capacity of outside beams shall not be less than the capacity of interior beams. All interior beams shall be equally sized.

5. For utilities, the lever rule can be used to compute the load to the adjacent beams.

403-3.0 TRANSIENT LOADS

403-3.01 Toll Road and Michigan-Train Design Trucks

Each bridge shall be designed for the HL-93 vehicular live load described above. In addition, a series of design truck loads shall be used as described as follows.

1. Toll Road Live Load. In addition to the Specifications’ live-load regime, the Toll Road live load shall be applied to each State-highway bridge located within 15 mi of an Indiana Toll Road gate. A single truck with design lane load shall be used in each design lane. This loading shall be investigated under Strength II Limit State. The configurations of the Toll Road live-load vehicles are shown in Figures 403-3A, 403-3B, and 403-3C.
Factors for multiple presence and dynamic load allowance shall be the same as those used for regular design trucks.

2. **Michigan Truck-Train Live Load.** In addition to the Specifications’ live load regime, the Michigan Truck-Train live load shall be applied to each bridge located on the Indiana Extra-Heavy-Duty-Highway System. A complete list of the locations of these highways appears at [http://codes.lp.findlaw.com/incode/9/20/5/9-20-5-4](http://codes.lp.findlaw.com/incode/9/20/5/9-20-5-4). The configurations of the Michigan Truck-Train vehicles are shown in Figures 403-3D and 403-3E. A single truck with design lane load shall be limited to one design lane located so as to cause extreme force effects, while the other design lanes are occupied by regular design loads. This loading combination shall be investigated under the Strength II Limit State. Factors for multiple presence and dynamic load allowance shall be the same as those used for regular design trucks.

These design trucks shall not be considered for fatigue considerations, but they may be used for centrifugal and braking forces.

### 403-3.02 Centrifugal and Braking Forces, and Wind Pressure on Vehicle

Centrifugal forces, braking forces, and wind pressure on a vehicle shall be applied at 6 ft above the profile grade at the centerline of the pier or bent.

### 403-3.03 Stream Pressure

A drag coefficient, $C_D$, shall be used as described in LRFD 3.7.3.1.

### 403-3.04 Forces Due to Friction

Chapter 409 discusses friction forces within the context of bearings.

### 403-3.05 Earthquake Effects [Rev. Oct. 2012]

Earthquake Effects, EQ, should be determined in accordance with AASHTO Guide Specifications for LRFD Seismic Bridge Design 2nd Edition. A structure longer than than 500 ft located in an area in Seismic Design Category greater than A will be analyzed using elastic dynamic analysis. Integral structures 500 feet in length or less will not require seismic analysis.
provided that they are detailed in accordance to the details provided in Chapter 409. Figure 403-3F shows the requirements for seismic analysis according to structure type and length.

### 403-3.06 Ice Forces on Pier

The following describes criteria for determining ice forces on a pier.

1. Effective ice crushing strength, \( p = 165 \) psi.
2. Ice thickness, \( t = 1 \) ft.
3. The horizontal force shall be applied midway between the \( Q_{100} \) elevation, i.e., the water elevation at the 100-year frequency flood event, and the low-water elevation.
4. If the low-water channel width is less than 60 ft, a reduction factor, \( K_1 \), of 0.75, and \( p = 55 \) psi, shall be used.
5. *LRFD Specifications* 3.9.3, 3.9.4, 3.9.5, and 3.9.6 do not apply.


Unless the structure is protected as specified in *LRFD* 3.6.5.1, an abutment or pier located within 30 ft of the edge of a roadway shall be designed for loads in accordance with *LRFD* 3.6.5.2. Requirements for train collision load have been removed from the 2012 *LRFD*.

A mechanically-stabilized-earth-wall bridge abutment placed adjacent to a roadway need not be checked for vehicle-collision forces as described in *LRFD* 3.6.5. However, if the wall must be placed inside the clear zone, roadway safety shall be addressed as described in Chapter 49.

### 403-3.08 Vessel Collision with Structure

In a navigable waterway, a merchant ship or barge can collide with a bridge substructure. The bridge structure shall be designed using load combination Extreme Event II to prevent collapse of the superstructure by considering the size and type of the vessel, available water depth, vessel speed, and structure response in accordance with *LRFD* 3.14. For additional information, see the AASHTO *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges*. The design water depth shall be computed from the bottom of the waterway to the annual mean high-water level.
403-3.09 Pedestrian Live-Load Distribution

Pedestrian live loads shall be evaluated for each of the conditions as follows.

1. Based on the assumption that all sidewalks can be removed in the future; vehicles are permitted to travel from face-of-barrier to face-of-barrier. A pedestrian live load is not applied to the structure.

2. Pedestrian live load shall be distributed with 60% of the load applied to the exterior beam and 40% of the load applied to the first adjacent interior beam. Vehicles are permitted to travel from face-of-curb to face-of-curb.

403-4.0 CONSTRUCTION LOADINGS

403-4.01 General Requirements

Construction loadings shall be evaluated in accordance with *LRFD* 3.4.2.

Construction loadings are not accurately known during the project-design stage. The magnitude and location of the assumed construction loadings used for design, however, shall be shown in the contract documents.

For a beam or girder structure, the information shown in Figure 403-4A shall be shown on the General Plan. The exterior beam or girder shall be checked for the specified construction loadings:

All additional rotational resistance measures required as a result of the construction loading check shall be shown on the plans.


1. Component Loads, DC.
   a. DC1, Stay-in-place Formwork = 15 psf
   b. DC2, Concrete = 150 pcf

2. Construction Dead Loads, CDL.
   a. CDL1, Removable Coping Deck Forms = 15 psf
   b. CDL2, Temporary Walkway = 15 psf, applied over a 2-ft width platform on outside of coping
3. **Construction Live Loads, CLL.**
   a. CLL1, Construction Live Load = 20 psf extended the entire bridge width plus 2 ft outside of bridge coping over 30 ft longitudinal length centered on screed-machine load
   b. CLL2, Screed Machine = 4500 lb over 10 ft longitudinal length applied 6 in. outside the bridge coping
   c. CLL3, Vertical Railing and Walkway Load = 75 plf applied 6 in. outside the bridge coping over 30 ft longitudinal length centered on screed-machine load

4. **Wind Load, WS.** Structure designed for 70 mph horizontal wind loading in accordance with *LRFD* 3.8.1.

The angled bracket shall be assumed to extend from the flange/web intersection to 6 in. outside the edge of coping to maximize the horizontal force. Figures 403-4B, 403-4C, and 403-4D depict the application of these loads and the resultant girder rotation.

### 403-5.0 ELASTIC STRUCTURAL ANALYSIS

#### 403-5.01 General

This Section discusses the effects of imposed deformations such as elastic shortening, creep, shrinkage, temperature, and settlement.

#### 403-5.02 Superimposed Deformations

Superimposed deformations include the following:

1. elastic shortening;
2. creep;
3. shrinkage;
4. temperature; and
5. settlement.

With the exception of settlement, all of these deformations are internally generated. More discussion on sectional, i.e., internal, effects of these imposed deformations is provided in Chapter 406.
The LRFD Specifications specify various load factors for these effects. The 1.20 factor relates to the fact that the movement calculated on the basis of specified values may occasionally be exceeded and helps to avoid the undersizing of joints, expansion devices, and bearings.

The poor performance of many deck joints and expansion bearings can be traced to an underestimate of extreme movements of retaining walls and abutments due to earth pressure or pavement expansion which can be cumulative with the effects of the other three. Deck joints frozen due to substructure movements are often reported. A pavement relief joint is provided at the end of each reinforced-concrete bridge-approach pavement, and the effect of pavement expansion can be neglected.

The governing combinations of the effects of creep, shrinkage, and uniform temperature shall be determined in accordance with the LRFD Specifications. The substructure displacement shall be determined considering strain or relative structural movement, whichever applies, and multiplying it by 1.20. If a calculated force effect is a direct response to creep, shrinkage, and uniform temperature, a load factor of 0.50 for the strength-limit state, and one of 1.00 for the service-limit state shall be used. In theory, a load factor of less than 1.00 signifies that the effects of superimposed deformation tend to dissipate at the strength-limit state due to inelastic action. This may be further reduced if so justified by inelastic analysis. If the calculated force effect is an indirect response, such as for altering eccentricity of gravitational or other loads, the load factors specified for these loads shall be applied, but the eccentricity caused by the deformation shall be upgraded by a factor of 1.2.

Indiana is considered to be in a cold climate. A setting temperature of 60 °F shall be used for the installation of expansion bearings and expansion deck joints.
<table>
<thead>
<tr>
<th>Dead Load</th>
<th>Value</th>
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<tr>
<td>Future wearing surface</td>
<td>35 lb/ft²</td>
</tr>
<tr>
<td>Permanent metal deck forms</td>
<td>15 lb/ft²</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>150 lb/ft³</td>
</tr>
<tr>
<td>Earth</td>
<td>120 lb/ft³</td>
</tr>
<tr>
<td>Water</td>
<td>62.4 lb/ft³</td>
</tr>
<tr>
<td>Lateral soil pressure</td>
<td>32.5 lb/ft³</td>
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<tr>
<td>(Equivalent fluid pressure)</td>
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**TYPICAL DEAD LOADS**

**Figure 403-2A**
TOLL ROAD LOADING NO. 1
(W = 90.0 kip)

Figure 403-3A
TOLL ROAD LOADING NO. 2
(W = 90 kip)

Figure 403-3B
SPECIAL TOLL ROAD TRUCK  
(W = 126 kip)

Figure 403-3C
MICHIGAN TRUCK TRAIN NO. 5
(W = 134 kip)

Figure 403-3D
MICHIGAN TRUCK TRAIN NO. 8
(W = 134.2 kip)

Figure 403-3E
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<th>Bridge Length</th>
<th>Integral Structure</th>
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<tr>
<td>A</td>
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<td>Detail in accordance with CH 409</td>
<td>In accordance with AASHTO Guide Spec</td>
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<tr>
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<td>&gt; 500’</td>
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**Seismic Analysis Requirements for Integral and Non-Integral Structures**

**Figure 403-3F**
CONSTRUCTION LOADING

The exterior girder has been checked for strength, deflection, and overturning using the constructions loads shown below. Cantilever overhang brackets were assumed for support of the deck overhang past the edge of the exterior girder. The finishing machine was assumed to be supported 6 in. outside the vertical coping form. The top overhang brackets were assumed to be located 6 in. past the edge of the vertical coping form. The bottom overhang brackets were assumed to be braced against the intersection of the girder bottom flange and web.

Deck Falsework Loads: Designed for 15 lb/ft² for permanent metal stay-in-place deck forms, removable deck forms, and 2-ft exterior walkway.

Construction Live Load: Designed for 20 lb/ft² extending 2 ft past the edge of coping and 75 lb/ft vertical force applied at a distance of 6 in. outside the face of coping over a 30-ft length of the deck centered with the finishing machine.

Finishing-Machine Load: 4500 lb distributed over 10 ft along the coping.

Wind Load: Structure designed for 70 mph horizontal wind loading in accordance with LRFD 3.8.1.

CONSTRUCTION-LOADINGS INFORMATION TO BE SHOWN ON GENERAL PLAN

Figure 403-4A
CONSTRUCTION LOADS

Figure 403-4B
CONSTRUCTION LOADS

Figure 403-4C
BEAM ROTATION

Figure 403-4D
CHAPTER 404

Bridge Deck

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<td>16-33</td>
<td>Sep. 2016</td>
<td>Figure 404-4D</td>
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<td>17-16</td>
<td>Aug. 2017</td>
<td>404-2.02(02), 404-2.02(03), 404-2.03(02), Figs. 404-2B and -2C</td>
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The applicable design memorandum revision date is date the memo was issued and is noted in brackets next to each section heading below. The date the revision becomes effective may be different and is identified in the design memo.
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404-1.0 STRIP METHOD

This chapter addresses the design of typical bridge decks using approximate methods of analysis commonly referred to as the equivalent strip method.

404-1.01 Description

For the design of the deck, LRFD 4.6.2 provides guidance on the use of an approximate method of analysis which is analogous to those described in past LRFD Specifications. The strip method is based on a structural simplification in which the deck is replaced by a parallel set of continuous beams running in the primary direction of the deck, supported by unyielding supports with the span lengths taken as the beam spacing. The vehicular loading is represented by a single line of wheel loads acting on this beam. The effective width of this beam is determined from LRFD Table 4.6.2.1.3-1.

In analyzing a transverse strip, the following procedure may be applied.

1. Determine the number of design lanes in accordance with LRFD 3.6.1.1.1.

2. Position the loads on the strip in accordance with the following:
   a. only full axles of 32 kip (16 kip per wheel) are to be used;
   b. the center-to-center distance of the wheels is 6 ft;
   c. the center of the wheel does not become closer than 1 ft to the front face of the bridge railing or curb;
   d. the axle is positioned to obtain extreme moments irrespective of the position of the design or traffic lanes; and
   e. the wheel loads may be modeled as concentrated loads.

Calculate the maximum moments by considering dynamic load, multiple lane loads, and multiple-presence factors. For negative moment, see LRFD 4.6.2.1.6 for determination of the critical design section.

Live-load moments for the strip method may be obtained from LRFD Table A4-1 in lieu of calculating them.
404-1.02 Application of the Strip Method to a Composite Concrete Deck

The strip method is appropriate for each type of supporting members, including AASHTO I-beams, spread box beams, steel beams, or concrete beams with T-shaped flanges. The following will apply to the application of the strip method of analysis.

1. **Reinforcing Bars.** It is not necessary to use the same bar size or spacing in the top and bottom of the bridge deck in the primary direction.

2. **Shear Effects.** By using the strip method, an 8-in. deck is designed for flexure, and shear effects can be neglected.

Figure 404-1A illustrates the cross section of a typical beam-slab bridge with four beams spaced at 10 ft, a minimum-depth 8-in. concrete deck, and concrete railings. The deck-overhang width of 4.5 ft shown in Figure 404-1A is intended only as an example. For criteria for deck-overhang-width limitations, see Section 404-3.02. A dead load of 35 lb/ft² shall be considered as a future wearing surface. The 36-ft width clear roadway accepts three 12-ft width travel lanes.

404-2.0 BRIDGE-DECK DESIGN

404-2.01 General Requirements

1. **Load Modifier, \( \eta \).** Due to the conservative deck design produced by the strip method, the \( \eta \) value for a deck shall be 1.

2. **Thickness.** The depth of a reinforced-concrete deck shall not be less than 8 in.

3. **Reinforcement.** The bottom-reinforcement cover shall be 1 in. The top-reinforcement cover shall be 2½ in. The primary reinforcement shall be on the outside and shall be a #5 bar or larger.

4. **Maximum Bar Spacing.** The maximum bar spacing for primary, distribution, and temperature reinforcement is 8 in. This maximum bar spacing is used to control cracking.

5. **Sacrificial Wearing Surface.** The top ½ in. of the bridge deck shall be considered sacrificial and shall not be included in the structural design or as part of the composite section.
6. **Class of Concrete.** Class C concrete shall be used.

7. **Concrete Strength.** The specified 28-day compressive strength of concrete shall not be less than 4 ksi.

8. **Reinforcing-Steel Strength.** The specified yield strength shall not be less than 60 ksi.

9. **Epoxy Coating.** All reinforcing steel in both top and bottom layers shall be epoxy coated for a bridge deck supported on beams.

10. **Sealing.** All exposed roadway surfaces, concrete railings, and outside copings shall be sealed from drip bead to drip bead. The underside of the copings and the exterior face of outside concrete beams shall also be sealed.

11. **Length of Reinforcing Steel.** The maximum length of individual reinforcing bars shall be 40 ft. All reinforcing-bar splice lengths shall be shown on the plans.

12. **Truss Bars.** Truss bars shall not be used in a concrete deck supported on longitudinal stringers or beams.

### 404-2.02 Dimensional Requirements for Concrete Deck

#### 404-2.02(01) Screed Elevations for Cast-in-Place Concrete Deck

Screed elevations shall be furnished to ensure that the gutters, or the edges of deck on a bridge without curbs, will be at the proper final elevations. Screed elevations are required for a beam or girder bridge, or a continuous reinforced-concrete slab bridge on a superelevation transition. Screed elevations, if not shown on the plans, shall be provided in tabular form on letter-size sheets.

This information shall include a diagram or table showing the elevations at the top of the concrete deck that are required before the concrete is placed. Elevations shall be shown for both curblines, or sidewalk gutter lines, and crown of the roadway, and above all beam or girder lines for the full length of the bridge, at all bearings, and at a maximum of 10-ft intervals. Elevations at mid-span are optional and need be shown only for short spans where the nearest 10-ft station may be some distance from the point of maximum deflection. Elevations at splice points are required.
A structure on a horizontal curve or in a superelevation transition will require additional elevation points to define the concrete-deck screed elevations. A sufficient number of screed elevations shall be provided so that the contractor is not forced to interpolate or make assumptions in the field.

All elevation points shall be furnished so as to allow the proper construction and finishing of the deck.

Figure 404-2A illustrates the locations of screed elevations for a bridge deck with curbs and sidewalks. Screed elevations shall be determined using the following criteria.

1. Screed lines shall be established parallel to the skew and at approximately 10-ft intervals longitudinally within each span.

2. Screed elevations shall be provided transversely at both copings and curb lines, at the centerline of each beam or flange tip for wide-flange beams, at the profile grade, crown line if not coincident with profile grade, and longitudinal construction joints.

3. Deflections shall be computed on the basis of beam continuity at the time of deck placement.

4. Screed elevations shall be rounded to the nearer 0.005 ft.

404-2.02(02) Fillet Dimensions for Steel Beams or Girders [Rev. Aug. 2017]

Figure 404-2B illustrates fillet dimensions for steel beams or girders. The following will apply to the use of this Figure. Let dimension $Y$ equal the total depth of the deck and fillet measured to the top of the web.

1. Control dimension $Y$ shall be established so that the theoretical bottom of deck clears the thickest and widest top flange plate by $\frac{3}{4}$ in. to compensate for the allowed tolerance for beam camber, or so that the bottom reinforcement clears the field-splice plate by $\frac{1}{2}$ in., whichever controls.

2. Dimension $Y$ shall be shown on deck details and rounded up to the nearer 1/8 in.

3. Control dimension $Y$ shall be calculated after the vertical curve, top-flange plate and slice-plate thickness and cross slope have been determined.
4. Dimension $Y$ shall be held constant at each beam or girder, where possible, throughout the structure.

5. Once established, dimension $Y$ shall be used for all elevation computations such as bridge seats, top-of-splice elevations, etc.

404-2.02(03) Fillet Dimensions for Concrete Beams [Rev. Aug. 2017]

Figure 404-2C illustrates fillet dimensions for concrete beams. The basic requirement is to have the top of beam not higher than 3/4 in. below the bottom of slab at the center of the span. This allows the actual beam camber to exceed the calculated value up to 1 3/4 in. before the top of the beam can begin interfering with the deck steel.

Dimension $A$ at the center of the span represents an input item required for prestressed-beam design software, and can be determined as follows:

$$A = 0.75 + e \times (W / 2)$$

Where:

$W$ = beam top-flange width, in.

$e$ = cross slope (deck crown or superelevation rate)

The following should be considered:

1. The critical location of the ¾-in. minimum fillet can occur at the center of each span.

2. The critical location of the ¾-in. minimum fillet can also occur at the ends of the beam. For example, where either the residual beam camber is negative or where the residual beam camber allowance is less than the effect of the crest vertical curve.

404-2.03 Forms for Concrete Deck [Rev. May 2013]

Contractor options regarding the use of permanent and removable forms are provided in the INDOT Standard Specifications. The following selection and exception criteria apply to forms for a concrete deck.
404-2.03(01) Precast Deck Panels [Rev. May 2013]

Precast prestressed-concrete deck panels are no longer permitted as an alternative forming method.

404-2.03(02) Permanent Metal Forms [Rev. May 2013, Rev. Aug. 2017]

Metal stay-in-place forms may be used to support the deck between beams. Designers should assume the use of metal forms and account for dead load as described in Section 403-2.03.

Plan details should include only the minimum fillet dimensions. The design of permanent metal forms is the responsibility of the contractor. Recurring Special Provision (RSP) 702-B-304, Permanent Deck Form Angles, should be included in contracts with applicable bridge work until such time as the RSP is incorporated into the Standard Specifications.

404-2.03(03) Removable Forms [Rev. May 2013]

Removable forms shall be used to support a deck overhang, and may be used to support the deck between girders.

404-2.04 Skewed Deck

Skew is defined as the angle between the end line of the deck and the normal drawn to the longitudinal centerline of the bridge at that point. The two end skews can be different. In addition to skew, the behavior of the superstructure is also affected by the span-length-to-bridge-width ratio. The *LRFD Specifications* implies that the effects of a skew angle not exceeding 25 deg can be neglected, but only for a bridge with a relatively large span-length-to-bridge-width ratio. Figure 404-2E shows four combinations of skew angles 25 deg and 50 deg, and length-to-width ratios of 3:1 and 1:3. Both the 50-deg skew and the 1:3 length-to-width ratio are considered extreme values, but this often occurs where the deck constitutes the top slab of a culvert. It can be judged visually that both combinations with 25-deg skew may be orthogonally modeled for design.

*LRFD* C9.7.1.3 provides valid arguments supporting the limit of 25 deg concerning the direction of transverse reinforcement. It suggests that placing the transverse reinforcement parallel to a skew larger than 25 deg will create a structurally undesirable situation in which the deck is
essentially un-reinforced in the direction of principal stresses. For a skew larger than 25 deg, the transverse reinforcement shall be placed perpendicular to the beams.

The combination of 50-deg skew and length-to-width ratio of 1:3, as indicated in Figure 404-2E, produces a layout such that if the deck is a cast-in-place concrete slab without beams, the primary direction of structural action is one that is perpendicular to the end line of the deck. Because of the geometry of the layout, consideration shall be given to placing the primary reinforcement in that direction and fanning it as appropriate in the side zone. With that arrangement, the secondary reinforcement can then be placed parallel to the skew, thus regaining the orthogonality of the reinforcement as appropriate for this layout.

404-2.05 Shear Connectors and Vertical Ties

Based on the LRFD Specifications, composite action between the deck and its supporting components shall be ensured where it is technically feasible. The design of stud or channel shear connectors for steel sections and vertical ties for concrete beams or girders is discussed in the LRFD Specifications. Shear connectors and vertical ties between the deck and its supporting members shall be designed for force effects calculated on the basis of full composite action, whether or not that composite action is considered in proportioning the primary members.

404-2.06 Deck Joints

404-2.06(01) Longitudinal Open Joint

A longitudinal open joint is not required in a concrete bridge deck with a width of 90 ft or less. If a deck of wider than 90 ft is required, a longitudinal open joint may be used, or a longitudinal closure pour not less than 2 ft wide, may be used. Transverse-steel lap splices shall be located within the longitudinal closure pour. Such a joint shall remain open as long as the construction schedule permits transverse shrinkage of the deck concrete to occur. The bearings supporting a superstructure that has a deck width exceeding 50 ft shall be capable of allowing movement in the transverse direction due to temperature and shrinkage movements.

404-2.06(02) Construction Joint

A construction joint creates planes of weakness that frequently cause maintenance problems. The use of deck construction joints is discouraged and their number shall be minimized. The
contractor, however, has the option of requesting additional joints if the number or locations shown on the plans are too restrictive.
1. **Longitudinal Construction Joint.**

   a. **Usage.** Construction joints need not be used on a deck having a constant cross section where the pour width is less than 65 ft. This applies if the constant cross section is rotated along the length of the deck, and the angular breaks within the cross section remain constant. Where the angular breaks within the cross section are variable, as in the runout length of a superelevation transition, a construction joint shall be provided. Longitudinal construction joints will also be required on a deck with phased construction.

   b. **Location.** The following applies.

      (1) Where a construction joint is required, it shall preferably be placed along the edge of a traffic lane. A joint which is close to a curb may be placed up to 1 ft outside the traffic lane.

      (2) A joint shall not be located over a beam flange, unless phased construction dictates otherwise.

      (3) The joint locations shall limit the maximum pour width to 50 to 55 ft.

   c. **Transverse Reinforcing Steel.** The lengths of transverse reinforcing bars shall be selected so bar laps do not appear within a longitudinal construction joint.

2. **Transverse Construction Joint.**

   a. **Steel Beam or Girder Structure.** Concrete may be placed continuously on a deck requiring less than 260 yd$^3$ of concrete. A bridge deck that is poured integrally with the end bents may usually be placed with one pour.

   For a longer structure that exceeds the pour-volume limitation of 260 yd$^3$, an alternative may be considered in which the deck length is subdivided into segments near the points of final dead load contraflexure, with segments in positive flexure placed first and those in negative flexure last.

   b. **Prestressed-Concrete Structure.** A prestressed-concrete beam bridge made continuous for live load only shall be treated such that transverse construction joints located 2.5 ft on each side of the pier centerline shall be shown on the plans. The short deck segment and diaphragm over the support provide continuity for
live load in the superstructure after the previously-poured center regions of the deck have been poured as simple-span loads.

c. Location. Where used, transverse joints shall be placed parallel to the transverse reinforcing steel.

3. **Diaphragms.** For a prestressed-concrete beam bridge with a cast-in-place deck, the *LRFD Specifications* requires diaphragms at the bearing points. These diaphragms shall be poured at the same time as the deck.

4. **Steel Structure.** A steel superstructure with short end spans relative to the adjacent interior span can be subject to uplift at the end bent during the deck pour. This can occur where the far end span is 60% or less of the adjacent interior span. Where this occurs, and if objectionable, a required transverse construction joint shall be placed in the far end span and a terminal portion of the end span poured first to counterbalance the uplift. The deck may then be poured from the opposite end forward in the usual manner. The effects of the deck pouring sequence shall be investigated, including its effect on camber, screed elevations, and stresses.

5. **End Bents.** The simply-supported end of a short end span can experience uplift under live load. A counterweight may be poured near the end of the span to counterbalance the uplift, or positive hold-down devices may be installed. The details of counterweights or tie-downs shall be shown on the plans. Integral end bent concrete shall be considered as a counterweight.

6. **Pour Diagrams.** Figure 404-2F illustrates the pour diagrams for a continuous, prestressed-concrete beam structure. The plans shall include a note similar to that shown on Figure 404-2F, revised as necessary. Figure 404-2G illustrates the pour diagrams for a continuous steel beam or girder structure.

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**404-2.06(03) Expansion Joints [Rev. May 2013]**

Indiana is considered to have a cold climate for the purpose of expansion-joint design. See *LRFD* Table 3.12.2.1-1 for temperature-range values to use to calculate joint movements due to temperature.

The following provides criteria for the use of expansion joints.
1. **Compression Seal Type BS.** This type of seal has not performed well in the past and shall not be used as an expansion joint.

2. **Expansion Joint Sealing System.** This joint may be used on a bridge to be rehabilitated.

3. **Strip Seal Type SS.**
   a. **Details.** The details are shown on the INDOT *Standard Drawings*.
   b. **Expansion Length.** This expansion joint may be used for an expansion length up to 400 ft. The plans shall designate the expansion length for the contractor’s use of the Joint Setting Table shown in the INDOT *Standard Drawings*, which is dependent upon the ambient temperature while the deck is being poured (see item 3.c.). Therefore, the expansion length shall be computed in feet for each joint location. This value shall be shown on the General Plan at each joint location.
   c. **Width of Opening.** This joint is designed by the manufacturer to accommodate a maximum of 4 in. of movement. The width of the joint opening at installation depends upon the ambient temperature while the deck is being poured, and the expansion length of the structure at the joint location. This joint-opening width is shown in the Joint Setting Table in the *Standard Drawings* for a range of ambient temperatures and expansion lengths.

4. **Modular Type M.**
   a. **Details.** Figures 404-2H and 404-2 I illustrate typical schematic details for this joint.
   b. **Expansion Length.** This joint is used only where the anticipated expansion movement exceeds the length that can be accommodated by an expansion joint type SS. For an expansion movement of greater than 4 in., a modular expansion joint is recommended.
   c. **Splices.** Where practical, a modular joint shall be full length with no field splices across the roadway width. If a field splice is required for traffic continuity, the support beams shall be spaced at a maximum of 2 ft. See Figure 404-2 I, Section A-A.
d. Elastomeric Seal. The elastomeric seal will be one piece across the roadway width, regardless of traffic continuity considerations and the presence of a field splice in the steel armor. See the INDOT Standard Specifications for more information.

404-2.07 Drainage Outlets

Chapter 203 discusses the hydrological and hydraulic analyses for a bridge deck. This includes the methodologies for calculating the flow of water on the bridge and for determining the necessary spacing of drainage outlets to prevent the spread of water on the deck from exceeding the acceptable limits. Additional design criteria and details of drainage outlets on a bridge deck and closed drainage systems are also provided in Chapter 203.

To make deck-drainage facilities operationally effective and insensitive to blocking by debris or ice, they shall be large in size and few in number as suggested by LRFD 2.6.6. Where practical, the use of roadway drain type SQ is preferred to roadway drain type OS because it does not accumulate debris as easily. Locations of roadway drains types SQ and OS shall be checked to verify that they clear the top flange of the outside beam or girder. The large drainage facility, however, creates a discontinuity in the deck which shall be considered. A deck acting monolithically is not too sensitive to this, and for a drainage facility whose maximum dimension does not exceed 1.33 ft, no additional deck reinforcement is required. If the facility interrupts a main bar of a steel-grid deck, the facility shall be framed, and the frame shall support the interrupted element.

404-3.0 MISCELLANEOUS STRUCTURAL ITEMS

404-3.01 Longitudinal Edge Beam

The LRFD Specifications does not require the presence of side longitudinal edge beams but permits the utilization of solid concrete barriers as structural parts of the deck or deck system. Railing-type barriers consisting of wood, steel, or concrete beams and posts offer only negligible structural contribution, so the usable advantages relate exclusively to solid-concrete wall and safety-shape types. However, the structural contribution of concrete appurtenances to the deck shall not be considered for the strength or extreme-event limit states per LRFD 9.5.1, because other than immediate repair of an accidentally-damaged railing will result in temporary, yet unacceptable, understrength of the bridge. The structural contribution of the concrete railing shall only be considered for the service or fatigue-and-fracture limit states. If the bridge railing is structurally discontinuous, i.e., beam and post, a longitudinal edge beam may be required.
404-3.02 Deck Overhang

Additional top reinforcement to resist the collision load transmitted through a railing may be required in a large overhang. The width in inches of the primary strip for a cast-in-place concrete deck is $45 + 10X$, where $X$ is the distance from load to point of support, as described in LRFD 4.6.2.1.3.

404-3.02(01) Design Methods

Bridge-deck overhang-width restrictions apply only to a multi-girder type superstructure, as follows:

1. Overhang width is defined as the distance from the centerline of the exterior beam to the face of the deck coping. The suggested overhang-width criteria are as follows:
   a. not more than 0.45 times the beam spacing; and
   b. not more than 5 ft.

2. The overhang width for a prestressed-concrete box-beam bridge shall not exceed 2 ft from the edge of the outside beam.

If deck drains type OS are to be used on a beam or girder structure, the minimum overhang width shall be 1.75 ft plus one-half the flange width. The deck-drain locations shall always be checked to verify that the drains clear the top flange.

404-3.02(02) Deck Overhang

For curved deck copings with reference to segmentally spaced straight beams, the above limits on maximum width shall be interpreted as the average width within a span. A ¾-in., half-round drip bead shall be placed 6 in. in from the edge of the coping. The depth of the outside coping shall be a minimum of 8 in. Once a coping depth is selected, it shall be maintained over the full length of the superstructure along that coping.
Because of the geometry of construction jacks used to support an overhang, the bottom of the coping shall be made flush with the underside of the top flange of a steel structure. The bottom of the coping may be sloped to match the slope of the top of deck, or at least be made level.

The depth of the outside coping for a superelevated deck shall be greater than the minimum 8-in. deck thickness. The coping depth will be dimension $Y$ minus the superelevation rate times half the flange width at the low side, or, plus the superelevation rate times half the flange width at the high side. See Figure 404-3A. The slope of the bottom of the overhang will then be parallel to the top of the deck. The coping depth selected at each coping shall be maintained over the full length of the superstructure along that coping.

For a curved-deck layout, the distance from the centerline of the girder to the face of the coping along both copings shall be shown in a coping offset diagram. These offsets shall be shown at 10-ft intervals measured along the centerline of the girder. The offset at all break points of the girders shall also be shown.

For the design of a deck overhang with a cantilever not exceeding 6 ft from the centerline of the exterior girder to the face of a structurally-continuous concrete barrier, LRFD 3.6.1.3.4 permits replacement of concentrated wheel loads with a line load of 1 kip/ft intensity, located 1 ft from the face of the railing. Thus, an effective longitudinal distribution of wheel loads and an ease of design is realized. If other railings are used, the strip method may be applied.

The design approach, as described in LRFD CA13-4.2, is that the vehicular-collision loads are not specified and that the overhangs are designed for the maximum inelastic force effects which can be generated and transmitted by the railing resisting the vehicular impact. LRFD CA13.4.2 states, the crash testing program is oriented toward survival and not necessarily the identification of the ultimate strength of the railing system. This could produce a railing system that is significantly over-designed leading to the possibility that the deck overhang is also over-designed. The computed railing capacity will be far greater than the required capacity. Designing the overhang for full railing capacity will result in an extremely conservative section that is not in accordance with observed field behaviors. Based on observations of impacted bridge railings, an overhang designed according to previous AASHTO bridge-design specifications shows the desired behavior that the overhang does not fail if a railing failure occurs due to a collision. Accordingly, the overhang shall be designed for a collision force of 25% greater than the required capacity, which results in a design approximating present satisfactory practice.

There is a normal concentration of force effects in the end zone of the railing, and the deck may need strengthening therein. An extension of the end beam (hidden or otherwise) to the railing may be necessary to strengthen the overhang. LRFD Equations A13.3.1-3 and A13.3.1-4 can be
used to check the railing strength for impact near the end of the railing and to compute the magnitude of the loads to be transferred to the deck overhang and the need for extra top-deck reinforcement.

The deck overhang is a cantilever slab that supports the barrier railing and can be designed independently of the deck spans. Sufficient negative moment reinforcement must be provided for the design conditions. The reinforcement must be adequately anchored into the first deck span.

Two limit states are used for design, as described in LRFD A13.4.1 and A13.2. Strength I and Extreme Event II requires the deck overhang to be designed for the vertical forces specified in the Extreme Event limit state. However, for a continuous concrete railing, this never controls. The strength-limit state considers vertical gravity loads and will only govern the design if the cantilever span is very large. The extreme-event-limit state considers horizontal forces caused by the collision of a vehicle with the railing, and will usually govern the deck-overhang design.

**404-3.03 Transverse Edge Beam**

LRFD 9.7.1.4 requires the end zone of the deck to have adequate resistance against the increase of force effects due to the structural discontinuity at the deck joint. This additional resistance is provided by the diaphragms that are commonly used for an integral end bent superstructure. In the absence of diaphragms, a transverse edge beam, hidden or exposed, is acceptable. The beam shall be designed as described in LRFD 4.6.2.1.4.

Typical configurations of transverse edge beams are shown in Figures 404-3B through 404-3F for various types of superstructures. For the positive-moment reinforcement, crankshaft bars shall be considered (see Figure 404-3F) instead of straight bars between beams, unless the requirements for tension development length provided in LRFD 5.11.2.1 can be satisfied by placement of straight bars between beams. Crankshaft bars that extend from coping to coping can be difficult to handle and install. They shall preferably consist of two bars that are lapped above the beam at the centerline of the roadway or above a beam that is adjacent to the centerline of roadway. As shown in Figure 404-3F, past designs have included a 45-deg fillet along the inside face of the edge beam. The 45-deg fillet is available as an option for additional resistance to force effects or assurance of a smooth flow of stress at a location where there is a significant change in section depth.

For a deck with no skew or with a skew angle not exceeding 25 deg, the transverse deck reinforcement shall be placed parallel to the skew, but for a greater skew angle, it shall be placed perpendicular to the supporting beams. For a greater skew angle, the transverse deck reinforcement...
reinforcement shall be terminated at points encompassed by the edge beam. Hence, it shall not be counted as part of the resistance. For a skew angle not exceeding 25 deg, the top transverse deck steel may be included in the design of the edge beam. For a skew angle exceeding 25 deg, the negative-moment reinforcement in the edge beam shall be placed below the top longitudinal deck steel.

Although LRFD 4.6.2.4c specifies a strip width for positive and negative moment for the edge member, it is acceptable to use the thickened edge beam or diaphragm width. The edge beam shall be assumed to carry one truck axle for each vehicle considered.

**404-3.04 Bridge Railing**

Section 404-4.0 discusses the types of bridge railings that may be used. Section 404-3.04 discusses the structural evaluation of concrete or steel railings at the copings of a bridge.

A concrete barrier railing is built monolithically and continuous with no contraction joints at either mid-span or over the interior supports. Full-depth open joints shall be provided only between the end of the structure and the reinforced-concrete bridge approach and at expansion joints on a structure composed of two or more units.

The INDOT Standard Drawings illustrate concrete railings with the preferred arrangement of reinforcement. This arrangement of railing reinforcement is a departure from traditional design in which the railing has essentially been considered as a vertical flexural element and the longitudinal reinforcement, as secondary, being compatible with frequent relief joints. The uninterrupted railing becomes a primary longitudinal flexural element, playing a significant role in resisting both gravitational and impact loading. The longitudinal steel is appropriate for this role, in addition to controlling shrinkage-caused cracking.

Stirrups connecting continuously-placed, whether or not structurally continuous, concrete railing, curb, parapet, sidewalk, or median to the concrete deck shall be determined assuming full composite action at the interface, in accordance with LRFD 5.8.4.

**404-4.0 BRIDGE-RAILING DESIGN**

**404-4.01 Test-Level Selection**

The basic parameter for bridge-railing selection is the Test Level required at the site. This is a function of the following:
1. highway design speed;
2. average annual daily traffic and percent trucks;
3. bridge-railing offset;
4. highway geometry (grades and horizontal curvature);
5. height of deck; and
6. type of land use below deck.

The detailed methodology for determining a bridge railing’s Test Level is described herein. The methodology has been adapted from the AASHTO publication *Guide Specifications for Bridge Railings*. The performance-level designations in the *Guide Specifications* have been converted to the Test-Level designations in National Cooperative Highway Research Program Report 350 (NCHRP 350) *Recommended Procedures for the Safety Performance Evaluation of Highway Features*. The *Guide Specifications* methodology is based on a benefit/cost analysis which considers occupant safety, vehicular types, highway conditions and costs. The overall objective is to match the bridge railing’s Test Level, and therefore costs, to site needs.

The Performance Level (PL-__) terminology applies to the AASHTO *Guide Specifications for Bridge Railings*. Under the NCHRP 350 criteria, performance of each bridge railing and associated transition is measured in terms of Test Levels (TL-__). A bridge-railing equivalency table for converting PL-1, PL-2 and PL-3 railings to TL-2, TL-4 and TL-5 railings is provided in Figure 404-4A, Bridge Railing Level Equivalency.

NCHRP 350 identifies six Test Levels. To limit the number of necessary bridge railings, three of these Test Levels have been selected, and warrants have been developed for their use.

The Test Level is selected based on the following.

**404-4.01(01) TL-2**

A TL-2 bridge railing is appropriate on a bridge which satisfies the following:

1. the bridge is located on a route not on the State highway system, and the adjusted AADT in the construction year appears within the TL-2 range shown in Figure 49-6D(30), 49-6D(40), 49-6D(45), 49-6D(50), or 49-6D(55), Median Barrier and Bridge Railing Test Level Selection, for the appropriate design speed; or
2. the bridge is located on a State-highway-system route with a design speed of 45 mph or lower and the adjusted AADT in the construction year appears within the TL-2 range shown in Figure 49-6D(30), 49-6D(40), or 49-6D(45), for the appropriate design speed.

404-4.01(02) TL-4

A TL-4 bridge railing is appropriate on a bridge which satisfies the following:

1. the criteria for TL-2 are not satisfied; and

2. the adjusted AADT in the construction year appears within the TL-4 range shown in Figure 49-6D(30), 49-6D(40), 49-6D(45), 49-6D(50), 49-6D(55), 49-6D(60), or 49-6D(65), for the appropriate design speed.

404-4.01(03) TL-5

A TL-5 bridge railing is appropriate on a bridge where the adjusted AADT in the construction year appears within the TL-5 range shown in Figures 49-6D(30) through 49-6D(65), whichever applies.

404-4.01(04) TL-6

A TL-6 bridge railing is intended to reduce to almost zero the probability that a large van-type, semi-trailer truck will penetrate the railing. The TL-6 bridge railing is intended to contain and redirect a tanker trailer truck, which has a high point of contact with a bridge railing.

The decision to use a TL-6 bridge railing is a policy decision based on a site-by-site evaluation; therefore no numerical thresholds are provided. As an example, a TL-6 bridge railing may be selected on a highway with a high volume of large trucks, or tanker trucks, where rollover or penetration beyond the barrier will result in severe consequences.

404-4.01(05) Making Test-Level Determination

Test-Level determination applies directly to a level roadway on tangent, with a bridge deck surface approximately 35 ft above the under-structure ground or water surface, and with low-
occupancy land use or shallow water under the structure. The traffic volume used to determine the Test Level is the construction-year AADT.

For highway conditions that differ from those described above, the AADT shall be adjusted by the correction factors shown in Figure 49-9B, Grade Traffic Adjustment Factor, $K_g$, and Curvature Traffic Adjustment Factor, $K_c$; and Figure 49-9C, Traffic Adjustment Factor, $K_s$, for Deck Height and Under-Structure Shoulder Height Conditions. The high-occupancy land use referred to in Figure 49-9C applies to a site where there is a relatively high probability for injury or for loss of human life. The low-occupancy land use applies to a site where the probability for injury or loss of human life is relatively low.

Once the adjusted AADT is determined, the appropriate Test Level can be determined from Figure 49-6D(30), 49-6D(40), 49-6D(45), 49-6D(50), 49-6D(55), 49-6D(60), or 49-6D(65), Median-Barrier and Bridge-Railing Test-Level Selection, for the design speed shown in the figure designation.

The following procedure will apply to the determination of the appropriate Test Level.

1. Determine adjustment factors $K_g$ and $K_c$ from Figure 49-9B, and $K_s$ from Figure 49-9C.

2. Calculate the adjusted AADT by multiplying the construction-year AADT, total for both directions, by the three adjustment factors, as shown below.

   \[
   \text{Adjusted AADT} = (\text{Construction-year AADT}) (K_g) (K_c) (K_s).
   \]

3. Determine the figure in the 49-6D figures series which is appropriate for the design speed. If the design speed is 35 mph, a straight-line interpolation between Figures 49-6D(30) and 49-6D(40) shall be used to determine the adjusted AADT range.

4. Locate the appropriate line in such figure under the Site Characteristics column.

5. Move across to the columns corresponding to the appropriate Highway Type.

6. Determine which of the three columns, TL-2, TL-4, or TL-5, includes the adjusted AADT value calculated in Step 2 to identify the appropriate Test Level.

Each side of a bridge shall be checked against these criteria, especially if the bridge is on a horizontal curve. The higher Test Level warrant shall be used for both sides of a structure.

See Section 49-9.03 for example calculations for the selection of a TL-4 or TL-5 bridge railing.
For a minor bridge rehabilitation project which does not include bridge-deck replacement or deck widening and the bridge currently has a crashworthy TL-4 bridge railing, the existing railing need not be upgraded to a TL-5 railing, though the warrants for the TL-5 railing are satisfied. If there is no significant history of truck accidents, the installation of the TL-5 bridge railing shall be deferred until the time of deck replacement or deck widening.

### 404-4.02 Bridge-Railing-Type Selection

#### 404-4.02(01) INDOT Standard Railings

Once the Test Level has been determined, a bridge-railing type shall be selected to match the required Test Level and other considerations, e.g., aesthetics, owner preference.

Figure 404-4B, Bridge-Railing Types, summarizes the Department-standardized bridge-railing types for each Test Level. Figure 404-4C summarizes the Department standardized bridge-railing pay items.

#### 404-4.02(02) FHWA-Approved Non-INDOT-Standard Railings

There are other bridge railings which have passed NCHRP 350 crash tests for specified Test Levels, in addition to those which the Department has standardized. These are identified on FHWA’s website, [http://safety.fhwa.dot.gov/](http://safety.fhwa.dot.gov/). If one of these devices is desired to be used for a specific project, the required documentation to be downloaded from the website and provided is as follows:

1. an acceptance letter from FHWA that approves the device for use; and

2. details for the device as successfully crash tested.

The device may be modified for specific-project use. However, the shape, strength, and performance requirements cannot be changed. If the device is to be modified, the additional required documentation to be provided is as follows:

1. details showing the modifications; and

2. calculations showing that the modified version still satisfies the strength and performance requirements of the crash-tested version.
The appropriate transition or end treatment shall be determined. This can be done by further investigating the bridge-railing details. Such details shall be provided, along with documentation that the transition or end treatment is appropriate for the bridge railing. If an appropriate transition or end treatment cannot be found, the bridge railing cannot be used.

404-4.02(03) Considerations if Sidewalk Present

Including a sidewalk on a bridge can impact the selection or location of the bridge railing. Once a vehicle strikes a curb, it can become airborne. Depending upon the lateral offset of the bridge railing, the vehicle can impact the railing while airborne and thus, can interfere with the proper vehicle/bridge railing interaction.

The following will apply to the selection and location of a bridge railing in combination with a sidewalk.

1. **Design Speed of 45 mph or Lower.** Only a railing shown to be crashworthy in the presence of a sidewalk may be used. The bridge-railing type can be selected based on the Test Level required at the site as described above, or, a vertical concrete wall of the appropriate height may be provided at the back of the sidewalk. The Test Level of such wall shall match that of a concrete shape F bridge railing.

2. **Design Speed of 50 mph or Higher.** The bridge railing cannot be placed at the coping side of the sidewalk; therefore it must be placed between the roadway and the sidewalk. A pedestrian- or bicycle railing shall then be placed at the coping side of the sidewalk as described below. The height of the vehicular bridge railing between the roadway and the sidewalk shall satisfy or exceed the minimum height requirement of a pedestrian railing, 42 in., or a bicycle railing, 54 in., whichever applies. Where the vehicular bridge railing is placed between the roadway and the sidewalk, the sidewalk need not be raised; i.e., the roadway surface and sidewalk surface may be at the same elevation. However, the sidewalk drainage pattern shall be reviewed. The guardrail transition and bridge-railing transition shall be connected to the pedestrian railing. An impact attenuator type R1 shall be connected to the bridge railing.
404-4.03 Bridge-Railing-Design Considerations

404-4.03(01) Superelevated Bridge Deck

The INDOT Standard Drawings illustrate the orientation of concrete shape F bridge railing with the bridge-deck surface for a bridge on a superelevated roadway section.

404-4.03(02) Barrier Delineators

Barrier delineators are to be placed on each bridge railing. However, barrier delineators are not to be placed on a bridge railing at the coping side of a sidewalk. The location of the delineators along the bridge railing shall be as described in the INDOT Standard Specifications. Barrier delineators shall be placed on each roadway face of a bridge-railing transition.

404-4.04 Bridge-Railing Transition

A steel-element roadside barrier will deflect upon impact, but a rigid bridge railing normally will not. Therefore, where a steel-element roadside barrier approaches a rigid bridge railing, a transition is necessary to gradually strengthen the steel-element roadside barrier as it approaches and connects to the bridge railing. The following will apply to such transitions.

404-4.04(01) Type

The preferred transition for each bridge-railing type is shown in Figure 404-4B. Most systems include both a guardrail transition and a bridge-railing transition. The details are shown on the INDOT Standard Drawings identified in Figure 404-4B.

A transition is typically used at each location, except where an intersecting road or driveway prevents the placement of a proper device. To use the bridge-railing transition listed, there shall be space to place at least 25 ft of roadside barrier between a curved W-beam guardrail connector terminal system or curved W-beam guardrail system and the beginning of a guardrail transition type TGB.

Bridge-railing transition type WBC is not identified in Figure 404-4B. It may be used with concrete bridge railing type FC, only where the proximity of an intersecting road or driveway prevents the proper installation of the preferred transition. Where at least one bridge-railing transition type WBC is required, such transition shall be used for all bridge-railing ends.
404-4.04(02) Location

The following will apply to the location of a bridge-railing transition.

1. **Reinforced-Concrete Bridge Approach (RCBA).** The ideal treatment is to locate a bridge-railing transition along the RCBA. This will keep the deck drainage not collected in the deck drains from flowing down the spill slopes at the bridge corners, which can cause erosion at the end bents. Placing the transition on the RCBA will require moving the connection between the bridge-railing transition and the guardrail transition a sufficient distance from the wing to allow placement of the posts required with the transition.

2. **Bridge Corner.** A transition shall be used at each bridge corner for each bridge-railing type, including the trailing end of a bridge railing on a one-way roadway, such as a ramp, or one roadway of a divided highway.

3. **Bridge Deck.** If it is necessary to locate the transition on the bridge deck, the wings shall be extended laterally a sufficient distance to provide a minimum clearance of 6 in. between the roadside face of the wing and the backs of the guardrail-transition posts.

4. **Intersecting Road or Drive.** The presence of an intersecting road or drive close to the bridge can complicate the location of the transition. Where practical, the intersecting road or drive shall be relocated to allow placement of the bridge-railing transition on the RCBA. Where this is not practical, the consideration of the bridge-railing transition shall be determined in the order of preference as follows:

   a. it shall be placed on the bridge deck if the structure has integral or semi-integral end bents;

   b. a modified version of the bridge-railing transition that may be used with guardrail transition type WGB shall be placed on the RCBA;

   c. a modified version of the bridge-railing transition that may be used with guardrail transition type WGB shall be placed on the bridge deck if the structure has integral end bents;

   d. an impact attenuator shall be used at the end of the bridge railing; or
e. since standard details for modified versions of bridge-railing transitions that may be used with the guardrail transition type WGB are not available, details of a modified version of the appropriate concrete-bridge-railing transition shall be included in the plans if an intersecting drive or public road approach cannot be relocated away from the end of the structure.

5. **Expansion Joint.** The bridge-railing transition may not be located on the bridge deck if a deck expansion joint is located between the bridge deck and the mudwall.

6. **Alternative Location.** In a situation with severe space restrictions, transition location or design modifications which do not comply with the above criteria may be necessary.

### 404-4.05 Pedestrian Railing

If a sidewalk is to be placed on a bridge, and the design speed is 50 mph or higher, a bridge railing shall be used to separate vehicular traffic from pedestrians, and a pedestrian railing shall be placed on the outside edge of the sidewalk.

If the design speed is 45 mph or lower, the need for protection of pedestrians by means of a combination vehicular bridge railing/pedestrian railing shall be considered on a site-by-site basis. Additional considerations to be made are as follows:

1. design speed;
2. vehicular-traffic volume;
3. pedestrian-traffic volume;
4. accident history;
5. geometric impacts (e.g., sight distance);
6. practicality of providing proper end treatments;
7. construction costs; and
8. local preference.

Figure 404-4D shows the typical reinforcement requirements for a bridge sidewalk.

### 404-4.06 Bicycle Railing

If bicyclists are permitted to use a bridge, a bicycle railing may be warranted. The following will apply.
404-4.06(01) Bicycle Path

This is defined as a bikeway physically separated from motorized vehicular traffic by an open space or barrier and either within the highway right of way or within an independent right of way. Each bridge which is a part of a bicycle path will require bicycle railing of 42-in. minimum height.

404-4.06(02) Other Facility

The need for combination vehicular bridge railing/bicycle railing to protect bicyclists shall be considered on a site-by-site basis. Additional considerations to be made are as follows:

1. motor-vehicular traffic design speed;
2. motor-vehicular traffic volume;
3. bicycle traffic volume;
4. accident history;
5. geometric impacts (e.g., sight distance);
6. practicality of providing proper end treatments;
7. construction costs; and
8. local preference.

404-5.0 BRIDGE APPURTENANCES

404-5.01 Outside Curbs

Except for the base of a two-tubed curb-mounted bridge railing, outside curbs are only used on a bridge in conjunction with a sidewalk. The typical curb height is 8 in. where used in conjunction with a sidewalk, unless a vehicular bridge railing is placed between the roadway and the sidewalk. For this situation, the roadway and sidewalk will be at the same elevation.

404-5.02 Center Curb or Median Barrier

A center curb or a median barrier on a structure shall be a continuation of the curb or median barrier on the approaching roadway. The use of a roughened construction joint between the bridge deck and the center curb or median barrier shall be provided. Curb or median barrier shall be anchored to the bridge deck with #5 bars spaced at 8 in. maximum.
404-5.03 Lighting

The Highway Management Division’s Office of Traffic is responsible for determining warrants for highway lighting. The district traffic office can offer input regarding lighting requirements for an INDOT project in an urban area. Warrants for, and the design of, lighting for a local-public-agency project shall be determined by the designer and the LPA. Section 502-4.02(07) discusses warrants for lighting on a bridge structure. Where lighting will be provided on an INDOT bridge, the Traffic Review Team will determine the pole size and spacing and submit this information to the project manager. The INDOT Standard Drawings provide the details for attaching standard luminaire supports to a concrete bridge railing.

404-5.04 Traffic Signal

A traffic signal can currently be, or be proposed to be, located near a bridge. Conduits may be provided across the bridge for the electrical service. The following will apply.

1. **Bridge Location.** For each bridge located within an urbanized area, or within 2 mi of its boundary, the designer shall contact the Office of Traffic. The Office will determine the need for conduits across the bridge.

2. **Junction Boxes.** Where conduits will be provided, junction boxes shall be located at approximately 250-ft intervals. The Office of Traffic will provide details for the junction boxes for incorporation into the plans.

404-5.05 Utilities Located on an INDOT Bridge

The Office of Utilities and Railroads’ Utilities Team and the designer shall work together to determine the proper accommodation of utilities across a highway structure. Chapter 104 describes the policy regarding the attachment of utility lines to a bridge. Chapter 104 also discusses procedures regarding permit applications and cost reimbursement. For approved utility attachments to a bridge, the following procedure applies.

1. **Preparation.** The utility company must prepare the attachment details for the utility. These are submitted to the Utilities Team.
1. **Review and Approval.** The Utilities Team reviews the attachment details and submits these to the designer for comment. The Utilities Team will notify the utility company of needed revisions. Once all changes have been made, the Utilities Team will approve the utility-attachment details and process the Utility Agreement.

2. **Plans Incorporation.** The Utilities Team will submit the approved utility-attachment details to the designer for incorporation into the final contract plans.
CROSS SECTION OF MULTI-BEAM BRIDGE

Figure 404-1A
LOCATION OF SCREED ELEV. (TYP.)

Note: 10'-0" spacing pattern may begin at left support or may be centered in span.

PLAN OF SCREEDS

Figure 404-2A
FILLET DIMENSIONS FOR STEEL BEAM

Figure 404-2B
FILLET DIMENSION FOR PRE-STRESSED BEAMS

Figure 404-2C
This figure deleted.

PRECAST DECK PANELS ON BRIDGE WITH SAG VERTICAL CURVE
Figure 404-2D
COMBINATION OF SKEW ANGLE AND SPAN LENGTH/BRIDGE WIDTH RATIO

Figure 404-2E
The following note, revised as necessary, should be shown on the plans for a continuous prestressed concrete I-beam, bulb-T, or box beam structure in which the composite slab over the interior supports is designed for the live load:

POUR NUMBERS INDICATE SEQUENCE OF POURS. POURS OVER INTERIOR SUPPORTS SHALL BE MADE LAST TO REDUCE THE EFFECT OF THE SLAB DEAD LOAD IN THE NEGATIVE MOMENT AREA. POUR #3 WILL INCLUDE THE DIAPHRAGM AT THE SUPPORT AND SHALL BE HELD TO A 5'-0" LENGTH. INTERIOR DIAPHRAGMS WILL BE POURED BEFORE SLAB IS POURED.

TYPICAL POUR DIAGRAM
(Continuous Prestressed Concrete Beams)

Figure 404-2F
Note: Pour numbers indicate sequence of pours. Avoid placing construction joints at same location as the beam or girder splice.

TYPICAL POUR DIAGRAMS
(Continous Steel Beams or Plate Girders)

Figure 404-2G
PRECOMPRESSED SPRING SUPPORT BOX SUPPORT BAR SLIDE BEARING

CENTER BEAM EDGE BEAM

CONTINUOUS NEOPRENE STRIP SEAL

5/8" X 4" STUD ANCHOR PLACE NON-SHRINK GROUT PAD UNDER SUPPORT BOX

PRECOMPRESSED SPRING SUPPORT BOX SUPPORT BAR SLIDE BEARING

REMOVABLE PLATE ASSEMBLY

MODULAR EXPANSION JOINT

Figure 404-2H
1. The elastomeric seal will be one piece across the roadway width and will not be spliced at the construction joint.

2. Where practical, a modular joint should be a one-piece rail across the roadway width.

3. The support box will not be less than 9" nor more than 1'-0" from the construction joint.

4. Where modular joints are placed in the roadway, the following will apply:

   1. The edge of the support box will not be less than 9" nor more than 1'-0" from the construction joint.
   2. The elastomeric seal will be one piece across the roadway width and will not be spliced at the construction joint.
   3. Where practical, a modular joint should be a one-piece rail across the roadway width.

MODULAR EXPANSION JOINT
(Field Splice)

Figure 404-2 I
SUPERELEVATION = e

Y = CONTROL DIMENSION (in.)
FLG = FLANGE WIDTH (in.)
e = SUPERELEVATION RATE (%)
SUGGESTED TRANSVERSE EDGE BEAM DETAILS
(For Bulb-Tee Beams)

Figure 404-3B
(Page 1 of 2)
SUGGESTED TRANSVERSE EDGE BEAM DETAILS
(For Bulb-Tee Beams)

Figure 404-3B
(Page 2 of 2)
SUGGESTED TRANSVERSE EDGE BEAM DETAILS
(For AASHTO I-Beam)

Figure 404-3C
(Page 1 of 2)
SUGGESTED TRANSVERSE EDGE BEAM DETAILS
(For AASHTO I-Beam)

Figure 404-3C
(Page 2 of 2)

NOTE: The edge beam shall extend from coping to coping.

NOTE: All reinforcing steel shall be epoxy coated.

#4 @ 1'-0" Max. spa. between beams

Bars between beams as required by design

Additional bars in top as required by design for negative moment

Drip Bead

from coping to coping.

All reinforcing steel shall be epoxy coated.
NOTE: All reinforcing steel shall be epoxy coated.

SUGGESTED TRANSVERSE EDGE BEAM DETAILS
(For Steel Plate Girder)

Figure 404-3D
(Page 1 of 2)
SUGGESTED TRANSVERSE EDGE BEAM DETAILS
(For Steel Plate Girder)

Figure 404-3D
(Page 2 of 2)
SUGGESTED ALTERNATE TRANSVERSE EDGE BEAM DETAILS
(For Steel Plate Girder)

Figure 404-3E

NOTE: All reinforcing steel shall be epoxy coated.
SUGGESTED ALTERNATE TRANSVERSE EDGE BEAM DETAILS
(For Bulb-Tee Beam, AASHTO I-Beam, and Steel Plate Girder)

Figure 404-3F
(Page 1 of 2)
SUGGESTED ALTERNATE TRANSVERSE EDGE BEAM DETAILS
(For Bulb-Tee Beam, AASHTO I-Beam, or Steel Plate Girder)

Figure 404-3F
(Page 2 of 2)
<table>
<thead>
<tr>
<th>TESTING CRITERIA</th>
<th>ACCEPTANCE EQUIVALENCIES</th>
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<tbody>
<tr>
<td>NCHRP Report 350</td>
<td>TL-1 TL-2 TL-3 TL-4 TL-5 TL-6</td>
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<tr>
<td>AASHTO Guide Specifications for Bridge Railings</td>
<td>--- PL-1 --- PL-2 PL-3 ---</td>
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**BRIDGE-RAILING LEVEL EQUIVALENCY**

Figure 404-4A
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<tr>
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<th>TS-1 *</th>
<th>PF-2</th>
<th>PS-2</th>
<th>TX **</th>
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<td>Common</td>
<td>Pedestrian</td>
<td>Pedestrian</td>
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<tr>
<td>Mounting Location</td>
<td>On bridge coping</td>
<td>Flush with bridge deck</td>
<td>Atop sidewalk of minimum 5 ft width</td>
<td>Flush with bridge deck</td>
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<tr>
<td>Railing Elements</td>
<td>Thrie-beam with steel posts</td>
<td>2 steel tubes with steel posts on concrete parapet</td>
<td>4 steel tubes with steel posts on concrete parapet</td>
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<tr>
<td>Total Height</td>
<td>2'-9”</td>
<td>3'-6”</td>
<td>3’-6”</td>
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* Bridge railing type TS-1 may be used only on a local-public-agency collector or local road. Details for the bridge railing and transition are shown in INDOT Recurring Plan Detail 706-B-140d.

** Bridge railing type TX should be considered for an aesthetically-sensitive area.

**BRIDGE-RAILING TYPES**

**TEST LEVEL 2**

Figure 404-4B

(Page 1 of 3)
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<tr>
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<th>PF-1</th>
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<td>Mounting Location</td>
<td>Flush with bridge deck</td>
<td>On existing concrete parapet</td>
<td>Atop sidewalk of minimum 5 ft width</td>
<td>Flush with bridge deck</td>
</tr>
<tr>
<td>Railing Elements</td>
<td>Concrete, shape F</td>
<td>Thrie beam with steel posts</td>
<td>2 steel tubes with steel posts on concrete parapet</td>
<td>1 steel tube with steel posts on concrete parapet</td>
</tr>
<tr>
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<td>2’-10”</td>
<td>3’-6”</td>
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<td>706-BRTR-01, through -04</td>
<td>706-BRPP-03, and -05, -06</td>
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<td>706-TTPP-05 and -06</td>
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<td>706-BRTR-05 and -06</td>
<td>601-TTGB-01 through -05</td>
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*** Bridge-railing type TR should be used only to replace existing aluminum bridge railing where no other modifications to a bridge are to be made, either as a spot improvement or within the limits of a 3R or 4R project.

BRIDGE-RAILING TYPES
TEST LEVEL 4

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<tr>
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<td>TTF-2</td>
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BRIDGE-RAILING TYPES
TEST LEVEL 5

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<td>CYS</td>
<td>LFT</td>
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<td>EACH</td>
<td>LBS</td>
<td>Guardrail Transition, TGB</td>
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<td>LBS</td>
<td>Reinforcing Steel, Epoxy Coated</td>
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<tr>
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<td>CYS</td>
<td>LFT</td>
<td>Concrete Bridge Railing Transition, TPS-2</td>
<td>EACH</td>
<td>LBS</td>
<td>Guardrail Transition, TGB</td>
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<td>LBS</td>
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</table>

**Test Level 2 Railings**

| FC                  | Railing, Concrete, FC | CYS      | Concrete Bridge Railing Transition, TFC * | EACH      | LBS       | Guardrail Transition, TGB *            | EACH      |
|                     | Reinforcing Steel, Epoxy Coated | LBS | Reinforcing Steel, Epoxy Coated           |           |           |                                        |           |
| TR                  | Railing, Steel, TR    | LFT      | none                                     | n/a       |           | Guardrail Transition, TGR              | EACH      |
| PF-1                | Railing, Concrete, PF-1 | CYS      | Concrete Bridge Railing Transition, TPF-1 | EACH      | LBS       | Guardrail Transition, TGB              | EACH      |
|                     | Railing, Steel, PF-1 | LFT      | Reinforcing Steel, Epoxy Coated           |           |           |                                        |           |
| PS-1                | Railing, Concrete, PS-1 | CYS      | Concrete Bridge Railing Transition, TPS-1 | EACH      | LBS       | Guardrail Transition, TGB              | EACH      |
|                     | Railing, Steel, PS-1 | LFT      | Reinforcing Steel, Epoxy Coated           |           |           |                                        |           |

**Test Level 4 Railings**

| FT                  | Railing, Concrete, FT | CYS      | Concrete Bridge Railing Transition, TFT   | EACH      | LBS       | Guardrail Transition, TGB              | EACH      |
|                     | Reinforcing Steel, Epoxy Coated | LBS | Reinforcing Steel, Epoxy Coated           |           |           |                                        |           |
| TF-2                | Railing, Concrete, TF-2 | CYS      | Concrete Bridge Railing Transition, TTF-2  | EACH      | LBS       | Guardrail Transition, TGB              | EACH      |
|                     | Railing, Steel, TF-2 | LFT      | Reinforcing Steel, Epoxy Coated           |           |           |                                        |           |

**Test Level 5 Railings**

* May be WFC where appropriate. If bridge-railing transition is WFC, guardrail transition is WGB.

**BRIDGE-RAILING PAY ITEMS**

*Figure 404-4C*
TYPICAL REINFORCEMENT IN BRIDGE SIDEWALK

Figure 404-4D
CHAPTER 405

Reinforced-Concrete Structure

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CHAPTER 405

REINFORCED CONCRETE

The LRFD Bridge Design Specifications Section 5 specifies the design requirements for concrete in all structural elements. This Chapter provides supplementary information specifically regarding the general properties of concrete and reinforcing steel and the design of reinforced concrete.

References shown following section titles are to the AASHTO LRFD Bridge Design Specifications.

405-1.0 GENERAL DESIGN CONSIDERATIONS

405-1.01 Material Properties

The minimum yield strength for reinforcing steel shall be taken as 60 ksi.

Figure 405-1A provides criteria for concrete materials in structural elements.

405-1.02 Flexure [Rev. Apr. 2017]

To facilitate design, LRFD 5.7.2.2 provides a simplified sectional stress distribution for the Strength Limit state, the application of which is limited to an under-reinforced rectangular section. Stresses in both top and bottom steel mats are taken at yield, while the concrete stress block is assumed to be rectangular with an intensity of $\alpha_1 f'_c$ and a height as described by the equation as follows:

$$a = \frac{A_v f_y - A'_v f'_c}{\alpha_1 f'_c b}$$

Location of the neutral axis is calculated as follows:

$$c = \frac{a}{\beta_1}$$

The factors $\beta_1$ and $\alpha_1$ shall be as defined in LRFD 5.7.2.2.
The nominal flexural resistance will be as follows:

\[ M_n = A_s f_y (d_s - 0.5a) - A'_s f'_{y} (d'_s - 0.5a) \]

### 405-1.03 Limits for Reinforcing Steel

The minimum reinforcement shall be checked in accordance with LRFD 5.7.3.3.2 at a section to be certain that the amount of prestressed and non-prestressed reinforcement is enough to develop a factored flexural resistance, \( M_r \), at least equal to the lesser of at least 1.2 times the cracking moment, \( M_{cr} \), or 1.33 times the factored moment required by the applicable strength load combinations. Most often, 1.2\( M_{cr} \) controls in the maximum positive-moment regions. In the region located approximately within the end one-third of the beam or span, 1.33 times the factored moment will control.

The cracking moment is computed by means of LRFD Equation 5.7.3.6.2-2 as follows:

\[ M_{cr} = \frac{f_r I_g}{y_t} \]

Where

\( M_{cr} = \) cracking moment, kip-in.

\( f_r = \) modulus of rupture of concrete

\( y_t = \) distance from the neutral axis to the extreme tension fiber, in.

### 405-1.04 Shear and Torsion

LRFD 5.8 allows two methods of shear design for prestressed concrete, the strut-and-tie model and the sectional-design model. The sectional-design model is appropriate for the design of a typical bridge girder, slab, or other region of components where the assumptions of traditional beam theory are valid. This theory assumes that the response at a particular section depends only on the calculated values of the sectional force effects such as moment, shear, axial load, and torsion, but it does not consider the specific details of how the force effects were introduced into the member.
In a region near a discontinuity, such as an abrupt change in cross-section, opening, coped, or dapped, end, deep beam, or corbel, the strut-and-tie model shall be used. See *LRFD* 5.6.3 and 5.13.2.

*LRFD* 5.8.3 discusses the sectional-design model. Subsections 1 and 2 describe the applicable geometry required to use this technique to design web reinforcement.

The nominal resistance is taken as the lesser of the following:

\[ V_n = V_c + V_s + V_p \]  
\[ LRFD \text{ Equation 5.8.3.3-1} \]

or

\[ V_n = 0.25 f'_c b_v d_v + V_p \]  
\[ LRFD \text{ Equation 5.8.3.3-2} \]

For a non-prestressed section, \( V_p = 0 \).

*LRFD* Equation 5.8.3.3-2 represents an upper limit of \( V_n \) to ensure that the concrete in the web will not crush prior to yield of the transverse reinforcement.

The nominal shear resistance provided by tension in the concrete is computed as follows:

\[ V_c = 0.0316 \beta \sqrt{f'_c b_v d_v} \]

The contribution of the web reinforcement is computed as follows:

\[ V_s = \frac{A_v f_s d_v (\cot \theta + \cot \alpha) \sin \alpha}{s} \]  
\[ LRFD \text{ Equation 5.8.3.3-4} \]

Where angle \( \theta \) represents the inclination of the diagonal compressive forces measured from the horizontal beam axis, and angle \( \alpha \) represents the web reinforcement relative to the horizontal beam axis, respectively.

For where the web shear reinforcement is vertical, \( \alpha = 90 \text{ deg} \), \( V_s \) simplifies to the following:

\[ V_s = \frac{A_v f_s d_v \cot \theta}{s} \]  
\[ LRFD \text{ Equation 5.8.3.3-3} \]

Both \( \theta \) and \( \beta \) are functions of the longitudinal steel strain, \( \varepsilon_x \), which, in turn, is a function of \( \theta \). Therefore, the design process is an iterative one. A detailed methodology, along with the design
tables, is provided in *LRFD* 5.8.3.4.2. For a section including at least the minimum amount of transverse reinforcement specified in *LRFD* 5.8.2.5, the values of $\beta$ and $\theta$ shall be taken from *LRFD* Table 5.8.3.4.2-1. For a section that does not include the minimum transverse reinforcement requirements, *LRFD* Table 5.8.3.4.2-2 shall be used to determine $\beta$ and $\theta$.

This process can be considered as an improvement in accounting for the interaction between shear and flexure and attempting to control cracking at the Strength Limit state.

For a non-prestressed concrete section not subjected to axial tension and including at least the minimum amount of transverse reinforcement specified in *LRFD* 5.8.2.5, or having an overall depth of less than 16 in., a value of 2 may be taken for $\beta$ and a value of 45 deg may be taken for $\theta$.

Transverse shear reinforcement shall be provided if the following applies.

\[
V_u > 0.5 \phi (V_c + V_p)
\]

*LRFD* Equation 5.8.2.4-1

Where transverse reinforcement is required, the area of steel shall not be less than the following:

\[
A_v = 0.0316 \sqrt{\frac{f_c'}{f_y}} 
\]

*LRFD* Equation 5.8.2.5-1

If the reaction introduces compression into the end of the member, the critical section for shear is taken as the larger of $0.5d_v\cot\theta$ or $d_v$, measured from the face of the support (see *LRFD* 5.8.3.2).

Torsion is most often not a major consideration. Where torsion effects are present, the member shall be designed in accordance with *LRFD* 5.8.2 and 5.8.3.6. A situation that can require a torsion design includes the following:

1. cantilever brackets connected perpendicular to a concrete beam, especially if a diaphragm is not located opposite the bracket; or

2. concrete diaphragms used to make precast beams continuous for live load where the beams are spaced differently in adjacent spans.

**405-1.05 Strut-and-Tie Model**

Members, if loaded, indicate the presence of definite stress fields which can individually be represented by tensile or compressive resultant forces as their vectoral sums. The load paths taken by these resultants form a truss-like pattern which is optimum for the given loading and...
that the resultants are in reasonable equilibrium, especially after cracking. The compressive concrete paths are the struts, and the reinforcing steel groups are the ties. The model is shearless.

The model has application for bridge components and parts such as pier caps, beam ends, post-tensioning anchorage zones, etc. A presentation of the model appears in the *PCI Precast Prestressed Concrete Bridge Design Manual*, Chapter 8, *Design of Highway Bridges based on AASHTO LRFD Design Specifications*, and in *Towards a Consistent Design of Structural Concrete*, PCI Journal, Vol. 32, No. 3, 1987. LRFD 5.6.3 provides adequately for design. If the model is not used for actual proportioning, it provides a fast check to ensure that all considerations are made in the design, especially for the appropriate anchorage of the steel.

Application of the model for a hammerhead pier is demonstrated in Figure 405-1B. There are five beams supported by the pier, of which two affect the design of a cantilever. The truss geometry selected here ensures that the struts, being parallel, are independent from each other. The scheme is indicative of the significance of a well-proportioned haunch. This design will yield approximately the same amount of steel in both ties. The steel in both ties is extended to the boundaries of their respective struts, then hooked down. The 90-deg hook of Tie No. 1 is further secured to the concrete by secondary steel, and the hook of Tie No. 2 is positioned in, and normal to, Strut No. 1.

This example was selected because of the potential excessive cracking of a pier head invariably designed as a beam. Normal beam design is not conservative for this application, due to the following:

1. early discontinuity of steel;
2. an erroneous estimate for the location of maximum moment, usually taken at the face of the pier-column; and
3. anchoring the steel in cracked zones.

Cracking is associated with at least partial debonding. Thus, the bonding capacity of cracked concrete cannot be considered completely reliable. Improperly-anchored steel is a design consideration in which mistakes are made, and the *LRFD Specifications* requires that steel shall not be anchored in cracked zones of concrete.

The model can also be used for the approximate analysis of the beam end. Figure 405-1C detail (a) shows a convenient method for checking the adequacy of reinforcement in the end zone and
the magnitude of compressive stresses in the web. In lieu of refined calculations, the angle $\theta$ can be assumed as 30 deg.

Figure 405-1C detail (b) indicates an application of the model to estimate the transverse forces in the bearing area to be resisted by the cage.

**405-1.06 Fatigue**

The constant-amplitude fatigue threshold for straight reinforcement is taken as follows:

$$(\Delta F)_{TH} = 24 - 0.33 f_{\min} \quad LRFD \text{ Equation 5.4.3.2-1}$$

Assuming $r/h = 0.3$, *LRFD* Equation 5.5.3.2-1 can be rearranged for easier interpretations as follows:

$$f_f + 0.33 f_{\min} \leq 23.4 \text{ ksi}$$

*LRFD* 5.5.3 shows a change in computing $f_f$. It is the stress range due to 75% of a single truck per bridge, lane load excluded, with reduced impact of 15%, and with the major axles of the truck at a constant spacing of 30 ft, instead of all contributing lanes being loaded. Also, *LRFD* 5.5.3 specifies that, if the bridge is analyzed by means of the approximate distribution method, live-load distribution factors for one design lane loaded shall be used.

**405-1.07 Crack Control**

Crack-control design shall be as described in *LRFD* 5.7.3.4.

**405-2.0 REINFORCING STEEL**

**405-2.01 Grade**

The yield strength of reinforcing bars shall be taken as 60 ksi. The modulus of elasticity, $E_s$, shall be taken as 29,000 ksi.

**405-2.02 Sizes**
Reinforcing bars are referred to by number, and they vary in size from #3 to #18. Figures 405-2A and 405-2B show the sizes, bar spacings, and properties of the types of bars used.

To avoid damage due to handling, the minimum bar size shall be #4. Longitudinal ties in compression members may be #3. See Section 409-7.03(07).

405-2.03 Concrete Cover

See Figure 405-2C for criteria for minimum concrete cover for various applications. The values shown in Figure 405-2C are based on $0.40 \leq w/c \leq 0.50$. All clearances to reinforcing steel shall be shown on the plans.

405-2.04 Spacing of Reinforcement

For minimum spacing of bars, see LRFD 5.10.3 and Figure 405-2D.

Fit and clearance of reinforcement shall be checked by means of calculations and large-scale drawings. Skews will tend to aggravate problems of reinforcing fit. Tolerances normally allowed for cutting, bending, and locating reinforcement shall be considered.

The distance from the face of concrete to the center of the first bar shall be shown. Where the distance between the first and last bars is such that the number of bars required results in spacings that are not to the nearer of ¼ in., the bars shall be shown to be equally spaced. Alternatively, one odd spacing may be used with spacings that are to the nearer of ¼ in.

405-2.05 Fabrication Lengths

See Figure 405-2A for maximum and normal bar lengths for fabrication. For ease of hauling and handling, the maximum length shall be reduced where the location of the splice is arbitrary. The maximum length of bars extending above a horizontal joint, e.g., from a footing into a wall, shall be 10 ft.

405-2.06 Development of Reinforcement

Development of reinforcement shall be as described in LRFD 5.11.2.
405-2.06(01) Development Length in Tension

Development length, \( l_d \), or anchorage of reinforcement, is required on both sides of a point of maximum stress at each section of a member.

Development of bars in tension involves calculating the basic development length, \( l_{db} \), which is modified by factors to reflect bar spacing, cover, enclosing transverse reinforcement, top-bar effect, type of aggregate, epoxy coating, and the ratio of required area to provide the area of reinforcement to be developed.

The development length, \( l_d \), including all applicable modification factors, must not be less than 1'-0".

Figures 405-2E through 405-2H show the tension development length for both uncoated and epoxy coated bars for normal weight concrete with specified 28-day strength of 3 ksi or 4 ksi. For class A concrete with \( f'_{c} = 3.5 \) ksi, use the development lengths shown for \( f'_{c} = 3 \) ksi unless calculated independently.

Development lengths shown in the figures for both uncoated and epoxy-coated bars must be multiplied by a factor of 2 for bars with a cover equal to the bar diameter, \( d_b \), or less, or with a clear spacing between bars of \( 2d_b \) or less. Development lengths shown for epoxy-coated bars may be multiplied by a factor of 0.8, if the cover is \( 3d_b \) or more and the clear spacing between bars is \( 6d_b \) or more.

405-2.06(02) Development Length in Compression

The standard procedure is to use tension development lengths for bars in either tension or in compression. This ensures that an adequate development length will be provided in a compression member that will be primarily controlled by bending.

405-2.06(03) Standard End Hook Development Length in Tension

A standard end hook, utilizing a 90-deg or 180-deg bend, is used to develop a bar in tension where space limitations restrict the use of a straight bar. End hooks on compression bars are not effective for development-length purposes. The values shown in Figures 405-2I and 405-2L, and LRFD Figure C5.11.2.4.1 show the tension development lengths for both uncoated and epoxy-coated hooked bars for normal weight concrete with specified strength of 3 ksi or 4 ksi.
For class A concrete with $f'_c = 3.5$ ksi, use development lengths shown for $f'_c = 3$ ksi unless calculated independently.

405-2.07 Splices

Splice-length determination shall be as described in LRFD 5.11.5.

405-2.07(01) General

Lap splices or mechanical splices can be used to splice reinforcing bars: Lap splicing is the most common method. The plans shall show the locations and lengths of all lap splices. Due to splice lengths required, lap splices are not permitted for #11 bars or larger. However, if #11 bars or larger are necessary, mechanical bar splices shall be used. Mechanical splices shall also be considered in lieu of lap splices in a highly-congested area. Mechanical splices are required for tension tie members.

Lap splices, for either tension or compression bars, shall not be less than 12 in. See the INDOT Standard Specifications for additional splice requirements.

If transverse reinforcing steel in a bridge deck is to be lapped near a longitudinal construction joint, show the entire lap splice on the side of the construction joint that will be poured last.

405-2.07(02) Lap Splices in Tension

Many of the same factors which affect development length affect splices. Consequently, tension lap splices are a function of the bar development length, $l_d$. Tension lap splices are classified as A, B, or C. Bars shall be spliced at points of minimum stress.

For a tension splice, the length of a lap splice between bars of different sizes shall be governed by the smaller bar.

Figures 405-2M through 405-2X show tension lap splices for both uncoated and epoxy-coated bars for normal weight concrete with specified strength of 3 ksi or 4 ksi. For class A concrete with $f'_c = 3.5$ ksi, use splice lengths shown for $f'_c = 3$ ksi unless calculated independently.

Splice lengths for spacing $\geq 6$ in., shown in the Figures for both uncoated and epoxy-coated bars, must be multiplied by a factor of 2 for bars with a cover of equal to bar diameter, $d_b$, or less, or
with a clear spacing between bars of $2d_b$ or less. Splice lengths shown for epoxy-coated bars can also be multiplied by a factor of 0.8 if cover is $3d_b$ or more and clear spacing between bars is $6d_b$ or more.

405-2.07(03) Lap Splices in Compression

Lap splices in a compression member are sized for tension lap splices. The design of a compression member, such as a column, pier wall, or abutment wall, involves the combination of vertical and lateral loads. Therefore, the policy of requiring a tension lap splice accounts for the possibility that the member design is primarily controlled by bending. Also, the increase in cost of additional splice-reinforcement material is small.

405-2.07(04) Mechanical Splices

A mechanical splice is a proprietary splicing mechanism. The requirements for mechanical splices are described in LRFD 5.11.5.2.2, 5.11.5.3.2, and 5.11.5.5.2. All mechanical connectors shall develop not less than 125% of the specified yield strength of the bar regardless of the stress level in the bar.

405-2.07(05) Welded Splices

Splicing of reinforcing bars by means of welding is not permitted.

405-2.08 Hooks and Bends

See LRFD 5.11.2.4 and Figure 405-2Y for standard hook or bend diameters. The value of $A$ shall be used for a standard 90-deg hook for longitudinal reinforcement with an end hook, and transverse reinforcement with a stirrup or tie hook. For transverse reinforcement where the bar size is #3 or #4 and shorter tail lengths are required for constructability, a non-standard hook may be used. Dimensions and bend diameters of non-standard hooks shall be shown on the plans and shall be in accordance with the CRSI Manual of Standard Practice. The total length of each bent bar shall be rounded up to the next 1 in. The legs of the bar shall add up to this total. The difference must be added to a leg of the bar.
405-2.09  Epoxy-Coated Reinforcement

Epoxy-coated reinforcement shall be used in accordance with LRFD 2.5.2.1.1 and 5.12.4, at the locations as follows:

1. the bridge deck;
2. the top 12 in. of a reinforced-concrete slab bridge;
3. the end bents and wingwalls of an integral end bent beam and deck-type structure;
4. the end bents and wingwalls of a beam and deck-type structure where deck expansion joints are located at the ends of the structure;
5. above the footing of each interior substructure unit that is located below a deck expansion joint. For a tall pier or bent, engineering judgment shall be used;
6. concrete bridge railing;
7. bars extending into the deck from the beams or substructure; or
8. reinforced-concrete bridge approaches.

For all other locations, use uncoated bars. These include the following:

1. piers, bents, or abutments that are located adjacent to a pavement surface; or
2. a reinforced-concrete retaining wall.

405-2.10  Reinforcement Detailing

405-2.10(01)  Standard Practice

The following provides the standard practice for detailing reinforcing bars.

1. Reinforcing bars shall be called out in the plan, elevation, and sections to indicate the size, location, and spacing of the individual bars. The number of reinforcing bars shall be called out in only one view, usually the plan or elevation view. In other views, only the bar size and length, or bar mark, shall be called out.

2. In a plan or elevation view, only the first bar and the last bar of a series of bars shall be drawn, and the number of bars indicated between. In a section view, all bars shall be shown.

3. All dimensions on details are measured on centerlines of bars, except where cover, e.g., 2 in. cl., is indicated.
4. Straight bars will be designated by size and length, e.g., #4 x 15’-0”.

5. Straight-bar lengths shall be in 3-in. multiples, except for short vertical bars in a railing or a parallel wing, which shall be in 1-in. multiples.

6. Bent bars are assigned a bar mark of which the first one or two numbers indicate the size of the bar, and the last two numbers, 01 to 99, indicate the mark. Each bar mark may be assigned a lower-case-letter suffix to indicate the location of the bar in the proper element of the structure (e.g., 801a, 802a). The following letters may be used as suffixes:

   a, b, c, d, f, h, k, m, n, p, r, s, t, u, v, w, x, y, and z.

7. Assign letters in sequence with superstructure first and substructure last. For the substructure, assign letters in sequence for each abutment or bent except where these are detailed in pairs. The one letter is to apply to both.

8. Epoxy-coated bars will be suffixed by the letter E (e.g., #6E x 15’-0”, 801aE). If all bars are epoxy-coated, a note will suffice.

The following shall be considered when selecting and detailing reinforcing steel.

1. Where possible, make similar bars alike to result in as few different bars in a structural element as practical.

2. If rounding off lengths of bars, one length shall not encroach upon the minimum clearances.

3. Consideration shall be given to ease of placement of bars. A bar shall not have to be threaded through a maze of other bars. The bars shall be located so that they can be easily supported or tied to other reinforcement.

4. It may be more practical to lap two bent bars than to have a bar with five or six bends.

**405-2.10(02) Bars in Section**

Figure 405-2Z provides a section through a hypothetical member showing some of the accepted methods for detailing reinforcing steel. The following list describes some of the concerns and observations that shall be considered in detailing reinforcing steel.
A section view shall be drawn to a large-enough scale to show reinforcing details.

1. Stirrups or other bars not shown end-on shall be drawn as single broken or unbroken lines for a scale smaller than 1:10, or as double unbroken lines for a scale of 1:10 or larger.

2. Bends of standard hooks and stirrups need not be dimensioned. However, all bends shall be drawn to scale.

3. Bars shown end-on shall be shown as small circles. The circles may be left open or may be shown as a dot. However, the symbol used shall be consistently applied on the drawing. If bars and holes will be shown, the bars shall be shown as solid.

4. An arrowhead pointing to the bar or a circle drawn around the bar are the acceptable methods of detailing for a bar shown end-on. An arrowhead shall point directly to the bar.

5. Sections cut at specific locations along a member are preferred to a typical section for a complex reinforcing pattern.

6. Corner bars enclosed by stirrups or ties shall be shown at the corner of the bend (see Figure 405-2Z).

**405-2.11 Bending Diagrams**

The following is the standard practice for detailing bending diagrams.

1. All dimensions are measured out-to-out of bars.
2. All bent-bar partial dimensions shall be shown to the nearer ¼ in.
3. The overall length of a bent bar shall be rounded up to the next 1 in.

See Figure 405-2AA for information on bending diagrams.

**405-2.12 Cutting Diagrams**

Two methods of showing cutting diagrams are provided. Other methods may be used at the discretion of the designer. The first is used where two sets of the same size bars are required and the second is used where only one bar of each size is required. Cutting diagrams are given a bar mark like bent bars. The first method is shown in an example of a skewed deck with the same
bars in the top and bottom mats. Figure 405-2BB applies to the transverse steel in a bridge deck. The pertinent information shall be determined as follows:

1. Determine the longest, $B$, and shortest, $A$, bars required to the nearer 1 in.
2. Determine the number of bars required.
3. Divide the number of spaces, the number of bars minus 1, by the difference in length between the longest and shortest bars to obtain the increment. Round the increment to the nearer inch.
4. The length $L$ is the sum of $A + B$.

The second method shall be used such as in an asymmetric widening of a hammerhead pier. An even number of bars will be provided by this cutting group. Figure 405-2CC shows the cantilevered portion of a hammerhead pier.

1. Determine the longest, $B$, and shortest, $A$, bars required to the nearer 1 in.
2. Determine the number of bars required.
3. Divide the number of spaces, the number of bars minus 1, by the difference in length between the longest and shortest bars to obtain the increment $N$. Round the increment to the nearer inch.
4. Determine dimensions $B$ and $C$ as follows:

$$B \text{ or } C = \frac{A + D}{2 \pm 0.5N}$$

5. The length $L = A + D = B + C$. Adjust dimensions as necessary to make them fit this equation.

### 405-2.13 Bill of Materials

The following applies to the Bill of Materials.

1. The bars shall be listed in descending order of size.
2. For each bar size, bent bars shall be listed sequentially by number first followed by straight bars.
3. Straight bars shall be listed in descending order of length.
4. Subtotals of the weight shall be provided for each bar size.
5. Plain and epoxy-coated bars shall be billed separately with totals for each.
6. There shall be a separate Bill of Materials shown on the appropriate plan sheet for each structural element.
7. If two structural elements are very similar in dimension and reinforcement, it is permissable to combine the quantities into one Bill of Materials.

Figure 405-2DD illustrates a typical Bill of Materials for a reinforced-concrete bridge approach.

405-3.0 REINFORCED CAST-IN-PLACE CONCRETE SLAB SUPERSTRUCTURE

405-3.01 General

The reinforced cast-in-place concrete slab superstructure is frequently used due to its suitability for short spans and its ease of construction. It is the simplest among all superstructure systems.

This Section provides information for the design of a reinforced cast-in-place concrete slab superstructure that amplifies or clarifies the requirements of the LRFD Bridge Design Specifications.

405-3.01(01) Materials

Class C concrete shall be used. See LRFD 5.4 and Figure 405-1A for concrete properties.

405-3.01(02) Cover

LRFD 5.12.3 and Figure 405-2C provides criteria for minimum concrete cover for all structure elements. All clearances to reinforcing steel shall be shown on the plans.

405-3.01(03) Haunches

Straight haunches are preferred to parabolic haunches because straight haunches are relatively easy to form yet result in relatively proper stress flow.

Haunching is used to decrease maximum positive moments in a continuous structure by attracting more-negative moments to the haunches and to provide adequate resistance at the haunches for the increased negative moments. It is a simple, effective, and economical way to enhance the resistance of a thin concrete slab. As illustrated in Figure 405-3A, there are three ways of forming the haunch. The parabolic shape shown in detail (a) is the most natural in terms of stress flow, and certainly the most aesthetic. It is preferred for where the elevation is
frequently in view. The parabolic haunch, however, is not the easiest to form and, as alternatives, the straight haunch shown in detail (b), and the drop panel shown in detail (c), shall be considered where appropriate.

Figure 405-3B depicts the elevation and plan of a three-span, continuous haunched slab bridge with an extensive skew. The preferable ratio between interior and end span is approximately 1.25 to 1.33 for economy, but the system permits considerable freedom in selecting span ratio. The ratio between the depths at the centerlines of the interior piers and at the point of maximum positive moment shall be between 2.0 and 2.5. Except for aesthetics, the length of the haunch need not exceed the $kL$ value indicated in Figure 405-3A, where $L$ is the end span length. Longer haunches may be unnecessarily expensive or structurally counterproductive.

405-3.01(04) Substructures

The following describes the practice for types of substructures used.

1. **End Supports.** Where possible, use integral end bents. Their use is not restricted by highway alignment or skew. The maximum bridge length is 200 ft for the use of integral end bents without a special analysis.

2. **Interior Supports.** See Section 402-6.03 for practices regarding the selection of the type of interior support (e.g., piers, frame bents).

405-3.01(05) Minimum Reinforcement

In both the longitudinal and transverse directions, at both the top and bottom of the slab, the minimum reinforcement shall be determined in accordance with LRFD 5.7.3.3.2 and 5.10.8. The first is based on the cracking flexural strength of a component, and the second reflects requirements for shrinkage and temperature. In a slab superstructure, the two articles provide for nearly identical amounts of minimum reinforcement.

According to LRFD 5.14.4.1, bottom transverse reinforcement, with the minimum requirements described above as notwithstanding, may be determined either by means of a two-dimensional analysis or as a percentage of the maximum longitudinal positive moment steel in accordance with the following:

$$\frac{100}{L} \leq 50\%$$
The span length, \( L \), in the equation shall be taken as that measured from the centerline to centerline of the supports. For a heavily skewed or curved bridge, the analytical approach is recommended.

Section 405-3.05 provides a simplified approach for shrinkage and temperature steel requirements.

405-3.02 Computation of Slab Dead-Load Deflections

For a concrete-deck-on-girder-type superstructure, the screed elevations shall be provided in accordance with LRFD 5.7.3.6.2 and Section 404-2.02(01). For a simple span or a continuous-spans reinforced-concrete slab superstructure, a dead-load deflection diagram showing the quarter-point deflections shall be shown on the plans. The contractor uses this information to develop screed elevations that will enable it to place the concrete slab at the proper final elevations. If a concrete-slab superstructure is located within a superelevation transition, or if other geometric complications are present, screed elevations are to be provided at 5-ft intervals.

The following criteria shall be used in developing a dead-load deflection diagram.

1. Compute dead-load deflections due to the weight of the concrete slab at the span quarter points or at a closer spacing if more accuracy is desired.
2. Compute instantaneous deflections by the usual methods using formulas for elastic deflections.
3. For determining deflections, use the gross moment of inertia and modulus of elasticity shown in Figure 405-1A.
4. Round off deflections values to the nearer 0.1 in.
5. The deflection of the concrete slab caused by the weight of a concrete railing is insignificant and may be ignored in developing the slab dead-load deflection diagram.
6. Do not include the effects of form settlement or crushing. This is the contractor’s responsibility.

405-3.03 Construction Joints

Transverse construction joints are not permitted. The INDOT Standard Specifications provide construction requirements where transverse construction joints are unavoidable if concrete placement is interrupted due to rain or other unavoidable event.
Longitudinal construction joints are also undesirable. However, the method of placing concrete, rate of delivery of concrete, and the type of finishing machine used by the contractor dictate whether or not a slab must be placed in one or more placements. An optional longitudinal keyway construction joint shall be shown on the plans at the centerline of roadway. The contractor may request permission to eliminate the construction joint by providing information specific to the proposed method of placing concrete and equipment to be used.

Where phased construction is not anticipated, transverse reinforcing steel may be lapped at the optional longitudinal construction joint. If the structure will be built in phases, show the entire lap splices for all transverse reinforcing steel on the side of the construction joint that will be placed last.

405-3.04 Longitudinal Edge-Beam Design

An edge beam must be provided along each slab edge. Structurally-continuous barriers are considered effective only for the Service Limit state, and not the Strength or Extreme-Event Limit state. An edge beam can be a thickened section or a more heavily-reinforced section composite with the slab. The width of the edge beam may be taken to be the width of the equivalent strip as specified in LRFD 4.6.2.1.4b.

405-3.05 Shrinkage and Temperature Reinforcement

Evaluating the redistribution of force effects as a result of shrinkage, temperature change, creep, and movements of supports is not necessary.

The required shrinkage and temperature reinforcement, as a function of slab thickness, is provided in Figure 405-3C.

405-3.06 Reinforcing Steel and Constructibility

The following practices for reinforcing-steel placement shall be considered to improve the constructability.

1. The maximum reinforcing-bar size shall be #11.
2. The minimum spacing of reinforcing bars shall preferably be 6 in.
3. Longitudinal steel shall be detailed in a 2-bar alternating pattern, with one of the bars continuous through the slab. The maximum size difference shall be two standard bar sizes.

Vertical steel, other than that required to keep the longitudinal negative-moment reinforcement floating, may not be required. **LRFD** 5.11.1.2 provides requirements for the portion of the longitudinal positive-moment reinforcement that must be extended to the next support point in excess of that required by the factored maximum moment diagram. Similarly, there is a more-stringent requirement addressing the location of the anchorage for the longitudinal negative-moment reinforcement.

### 405-3.07 Drainage Outlets

**LRFD** 2.6.6, Chapter 202, and Section 203-4.03(07) discuss the hydrological and hydraulic analyses for a bridge deck. The following specifically applies to inlet selection.

The deck drains shown on the INDOT *Standard Drawings* shall be specified. The deck drains are designed for a reinforced-concrete slab bridge only. The drain is a PVC pipe, 6 in. dia., set into the deck. The small deck drains have limited hydraulic capacity. Therefore, the standard spacing is approximately 6 ft. A 1/2-in. depression, which extends 12 in. transversely from the face of the curb, slightly increases the capacity. The PVC pipe must clear the bent-cap face by 2 ft.

### 405-3.08 Distribution of Concrete-Railing Dead Load

Dead load due to barrier railings placed after the deck has set, shall be distributed with 60% of the load applied to the exterior beam and 40% of the load applied to the first adjacent interior beam. The beams shall also be checked with the loads distributed equally to all beams.

### 405-3.09 Shear Resistance

The moment design in accordance with **LRFD Specifications** Article 4.6.2.3 may be considered satisfactory for shear.
405-3.10 Minimum Thickness of Slab [Rev. Apr. 2017]

The minimum slab thickness shall be in accordance with LRFD Table 2.5.2.6.3-1. In using the equations in the LRFD Table, the assumptions are as follows.

1. $S$ is the length of the longest span.
2. The calculated thickness includes the 1/2-in. sacrificial wearing surface.
3. The thickness used may be greater than the value obtained from the Table.

405-3.11 Development of Flexural Reinforcement

LRFD 5.11.1.2 provides requirements for the portion of the longitudinal positive-moment reinforcement that must be extended beyond the centerline of support. Similarly, LRFD 5.11.1.2.3 addresses the location of the anchorage, or embedment length, for the longitudinal negative-moment reinforcement.

405-3.12 Skewed Reinforced-Concrete Slab Bridge

For a skew angle of less than 45 deg, the transverse reinforcement is permitted to be parallel to the skew, providing for equal bar lengths. For a skew angle of 45 deg or greater, the transverse reinforcement shall be placed perpendicular to the longitudinal reinforcement. This requirement concerns the direction of principal tensile stresses as they develop in a greatly-skewed structure and is intended to prevent excessive cracking.

Special slab-superstructure design or modifications to the integral end supports are not required for a greatly-skewed or -curved structure. The requirements are based upon performance of relatively small span structures constructed to date. Such slab superstructures have included skews in excess of 50 deg and moderate curvatures. A significant deviation from successful past practice shall be reviewed. See LRFD Table 2.6.2.6.3-1 and Figure 405-3B.
405-3.13 Transverse Shrinkage and Temperature Reinforcement in the Top of the Slab at the Bent Caps

Top longitudinal cap flexural reinforcement cannot be considered effective reinforcement for transverse shrinkage and temperature stresses described in *LRFD* 5.10.8 if this steel is located significantly below the surface of the concrete slab.

405-3.14 Fatigue-Limit State

The design shall be as described in *LRFD* 5.5.3.
### Concrete Yield Strength, $f'_c$ (psi), Modulus of Elasticity, $E_c$ (ksi), Modulus of Rupture, $f_r$ (psi)

<table>
<thead>
<tr>
<th>Concrete</th>
<th>$f'_c$ (psi)</th>
<th>$E_c$ (ksi)</th>
<th>$f_r$ (psi)</th>
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</thead>
<tbody>
<tr>
<td>Class C</td>
<td>4000</td>
<td>3645</td>
<td>480</td>
</tr>
<tr>
<td>Class A</td>
<td>3500</td>
<td>3410</td>
<td>450</td>
</tr>
<tr>
<td>Class B</td>
<td>3000</td>
<td>3155</td>
<td>415</td>
</tr>
</tbody>
</table>

**Notes:**

1. Thermal coefficient of expansion = $6.0 \times 10^{-6}/^\circ F$
2. Shrinkage coefficient = 0.0002 after 28 days
   = 0.0005 after 1 year
3. Normal-weight-concrete density = 150 lb/ft$^3$ for computing loads
   = 145 lb/ft$^3$ for computing properties

**MATERIAL PROPERTIES OF CONCRETE**

**Figure 405-1A**
STRUT-AND-TIE MODEL FOR HAMMERHEAD PIER

Figure 405-1B
STRUT-AND-TIE MODEL FOR BEAM ENDS

Figure 405-1C
<table>
<thead>
<tr>
<th>Bar-Size Designation</th>
<th>Nominal Dimensions</th>
<th>Maximum Bar Length for Fabrication (ft)</th>
<th>Preferred Maximum Bar Length for Detailing (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Weight (lb/ft)</td>
<td>Diameter (in)</td>
<td></td>
</tr>
<tr>
<td>#3*</td>
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<td>0.375</td>
<td>35</td>
</tr>
<tr>
<td>#4*</td>
<td>0.668</td>
<td>0.500</td>
<td>35</td>
</tr>
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<td>0.625</td>
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<td>0.750</td>
<td>45</td>
</tr>
<tr>
<td>#7</td>
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<td>0.875</td>
<td>45</td>
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<tr>
<td>#8</td>
<td>2.670</td>
<td>1.000</td>
<td>45</td>
</tr>
<tr>
<td>#9</td>
<td>3.400</td>
<td>1.128</td>
<td>45</td>
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<td>#10</td>
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<td>45</td>
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<td>1.410</td>
<td>45</td>
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<td>7.650</td>
<td>1.693</td>
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<tr>
<td>#18</td>
<td>13.600</td>
<td>2.257</td>
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</tr>
</tbody>
</table>

*Maximum bar length does not apply to spiral bars.

REINFORCING-BAR SIZES

Figure 405-2A
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Area (in.²)</th>
<th>4</th>
<th>5</th>
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<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>17</th>
<th>18</th>
</tr>
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<tbody>
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<td>#3</td>
<td>0.11</td>
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<td>0.26</td>
<td>0.22</td>
<td>0.19</td>
<td>0.17</td>
<td>0.15</td>
<td>0.13</td>
<td>0.12</td>
<td>0.11</td>
<td>0.10</td>
<td>0.09</td>
<td>0.09</td>
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<td>0.08</td>
<td>0.07</td>
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<td>0.48</td>
<td>0.40</td>
<td>0.34</td>
<td>0.30</td>
<td>0.27</td>
<td>0.24</td>
<td>0.22</td>
<td>0.20</td>
<td>0.18</td>
<td>0.17</td>
<td>0.16</td>
<td>0.15</td>
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<td>0.47</td>
<td>0.41</td>
<td>0.37</td>
<td>0.34</td>
<td>0.31</td>
<td>0.29</td>
<td>0.27</td>
<td>0.25</td>
<td>0.23</td>
<td>0.22</td>
<td>0.21</td>
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<tr>
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<td>0.66</td>
<td>0.59</td>
<td>0.53</td>
<td>0.48</td>
<td>0.44</td>
<td>0.41</td>
<td>0.38</td>
<td>0.35</td>
<td>0.33</td>
<td>0.31</td>
<td>0.29</td>
</tr>
<tr>
<td>#7</td>
<td>0.60</td>
<td>1.80</td>
<td>1.44</td>
<td>1.20</td>
<td>1.03</td>
<td>0.90</td>
<td>0.80</td>
<td>0.72</td>
<td>0.65</td>
<td>0.60</td>
<td>0.55</td>
<td>0.51</td>
<td>0.48</td>
<td>0.45</td>
<td>0.42</td>
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<tr>
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<td>0.79</td>
<td>2.37</td>
<td>1.90</td>
<td>1.58</td>
<td>1.35</td>
<td>1.19</td>
<td>1.05</td>
<td>0.95</td>
<td>0.86</td>
<td>0.79</td>
<td>0.73</td>
<td>0.68</td>
<td>0.63</td>
<td>0.59</td>
<td>0.56</td>
<td>0.53</td>
</tr>
<tr>
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<td>1.00</td>
<td>3.00</td>
<td>2.40</td>
<td>2.00</td>
<td>1.71</td>
<td>1.50</td>
<td>1.33</td>
<td>1.20</td>
<td>1.09</td>
<td>1.00</td>
<td>0.92</td>
<td>0.86</td>
<td>0.80</td>
<td>0.75</td>
<td>0.71</td>
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<td>2.54</td>
<td>2.18</td>
<td>1.91</td>
<td>1.69</td>
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<td>1.39</td>
<td>1.27</td>
<td>1.17</td>
<td>1.09</td>
<td>1.02</td>
<td>0.95</td>
<td>0.90</td>
<td>0.85</td>
</tr>
<tr>
<td>#11</td>
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<td>4.68</td>
<td>3.74</td>
<td>3.12</td>
<td>2.67</td>
<td>2.34</td>
<td>2.08</td>
<td>1.87</td>
<td>1.70</td>
<td>1.56</td>
<td>1.44</td>
<td>1.34</td>
<td>1.25</td>
<td>1.17</td>
<td>1.10</td>
<td>1.04</td>
</tr>
<tr>
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<td>2.25</td>
<td>5.40</td>
<td>4.50</td>
<td>3.86</td>
<td>3.38</td>
<td>3.00</td>
<td>2.70</td>
<td>2.45</td>
<td>2.25</td>
<td>2.08</td>
<td>1.93</td>
<td>1.80</td>
<td>1.69</td>
<td>1.59</td>
<td>1.50</td>
<td>1.50</td>
</tr>
<tr>
<td>#18</td>
<td>4.00</td>
<td>9.60</td>
<td>8.00</td>
<td>6.86</td>
<td>6.00</td>
<td>5.33</td>
<td>4.80</td>
<td>4.36</td>
<td>4.00</td>
<td>3.69</td>
<td>3.43</td>
<td>3.20</td>
<td>3.00</td>
<td>2.82</td>
<td>2.67</td>
<td>1.50</td>
</tr>
</tbody>
</table>

**REINFORCING BARS**

Area (in.²) Per One-Foot Section

*Figure 405-2B*
<table>
<thead>
<tr>
<th>Item</th>
<th>Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck or Reinforced-Concrete Slab:</td>
<td></td>
</tr>
<tr>
<td>Top Bars</td>
<td>2½ *</td>
</tr>
<tr>
<td>Bottom Bars</td>
<td>1</td>
</tr>
<tr>
<td>Ends of Slab</td>
<td>2</td>
</tr>
<tr>
<td>Faces of Copings</td>
<td>2</td>
</tr>
<tr>
<td>Footing:</td>
<td></td>
</tr>
<tr>
<td>General</td>
<td>3</td>
</tr>
<tr>
<td>Bottom Bars</td>
<td>4</td>
</tr>
<tr>
<td>Columns, Ties, and Stirrups</td>
<td>1½</td>
</tr>
<tr>
<td>All Other Structural Elements</td>
<td>2</td>
</tr>
</tbody>
</table>

* Includes a ½-in. sacrificial wearing surface.

**MINIMUM CONCRETE COVER (in.) FOR DESIGN AND DETAILING**

Figure 405-2C
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Minimum Center-to-Center Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unspliced Bars</td>
</tr>
<tr>
<td>#3</td>
<td>2</td>
</tr>
<tr>
<td>#4</td>
<td>2</td>
</tr>
<tr>
<td>#5</td>
<td>2¼</td>
</tr>
<tr>
<td>#6</td>
<td>2½</td>
</tr>
<tr>
<td>#7</td>
<td>2½</td>
</tr>
<tr>
<td>#8</td>
<td>2½</td>
</tr>
<tr>
<td>#9</td>
<td>3</td>
</tr>
<tr>
<td>#10</td>
<td>3¼</td>
</tr>
<tr>
<td>#11</td>
<td>3¾</td>
</tr>
<tr>
<td>#14</td>
<td>4¼</td>
</tr>
<tr>
<td>#18</td>
<td>5¾</td>
</tr>
</tbody>
</table>

**Note:** Minimum spacing value, rounded up to the nearest 1/4 in., should be based on LRFD Specifications Articles 5.10.3.1.1 and 5.10.3.1.4. The maximum size of coarse aggregate used in cast-in-place concrete is 1 in.

**MINIMUM CENTER-TO-CENTER SPACING OF BARS** (in.)

**Figure 405-2D**
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Area (in.²)</th>
<th>Top Bars (1)</th>
<th>Others (2)</th>
<th>Top Bars (3)</th>
<th>Others (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>0.11</td>
<td>1'-1”</td>
<td>1’-0”</td>
<td>1’-0”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#4</td>
<td>0.2</td>
<td>1’-5”</td>
<td>1’-0”</td>
<td>1’-2”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#5</td>
<td>0.31</td>
<td>1’-9”</td>
<td>1’-3”</td>
<td>1’-5”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#6</td>
<td>0.44</td>
<td>2’-3”</td>
<td>1’-8”</td>
<td>1’-10”</td>
<td>1’-4”</td>
</tr>
<tr>
<td>#7</td>
<td>0.6</td>
<td>3’-1”</td>
<td>2’-2”</td>
<td>2’-6”</td>
<td>1’-9”</td>
</tr>
<tr>
<td>#8</td>
<td>0.79</td>
<td>4’-0”</td>
<td>2’-11”</td>
<td>3’-3”</td>
<td>2’-4”</td>
</tr>
<tr>
<td>#9</td>
<td>1</td>
<td>5’-1”</td>
<td>3’-8”</td>
<td>4’-1”</td>
<td>2’-11”</td>
</tr>
<tr>
<td>#10</td>
<td>1.27</td>
<td>6’-5”</td>
<td>4’-7”</td>
<td>5’-2”</td>
<td>3’-8”</td>
</tr>
<tr>
<td>#11</td>
<td>1.56</td>
<td>7’-11”</td>
<td>5’-8”</td>
<td>6’-4”</td>
<td>4’-7”</td>
</tr>
<tr>
<td>#14</td>
<td>2.25</td>
<td>10’-11”</td>
<td>7’-10”</td>
<td>8’-9”</td>
<td>6’-3”</td>
</tr>
<tr>
<td>#18</td>
<td>4</td>
<td>14’-2”</td>
<td>10’-2”</td>
<td>11’-4”</td>
<td>8’-1”</td>
</tr>
</tbody>
</table>

Notes:

(1) $1.4 \times l_d$
(2) $l_d$
(3) $1.4 \times 0.8 \times l_d$
(4) $0.8 \times l_d$

MODIFICATION FACTORS FOR NORMAL-WEIGHT CONCRETE

1.4, for top horizontal or nearly-horizontal reinforcement, so placed that more than 12 in. of fresh concrete is cast below the reinforcement

0.8, for reinforcement being developed in the length under consideration spaced laterally not less than 6 in. center-to-center, with not less than 3 in. cover measured in the direction of the spacing

DEVELOPMENT LENGTH FOR UNCOATED BARS IN TENSION

$f_{c'} = 3$ ksi

Figure 405-2E
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Area (in.²)</th>
<th>Top Bars (1)</th>
<th>Others (2)</th>
<th>Top Bars (3)</th>
<th>Others (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>0.11</td>
<td>1’-1”</td>
<td>1’-0”</td>
<td>1’-0”</td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td>0.2</td>
<td>1’-5”</td>
<td>1’-0”</td>
<td>1’-2”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#5</td>
<td>0.31</td>
<td>1’-9”</td>
<td>1’-3”</td>
<td>1’-5”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#6</td>
<td>0.44</td>
<td>2’-2”</td>
<td>1’-6”</td>
<td>1’-9”</td>
<td>1’-3”</td>
</tr>
<tr>
<td>#7</td>
<td>0.6</td>
<td>2’-8”</td>
<td>1’-11”</td>
<td>2’-2”</td>
<td>1’-6”</td>
</tr>
<tr>
<td>#8</td>
<td>0.79</td>
<td>3’-6”</td>
<td>2’-6”</td>
<td>2’-10”</td>
<td>2’-0”</td>
</tr>
<tr>
<td>#9</td>
<td>1</td>
<td>4’-5”</td>
<td>3’-2”</td>
<td>3’-6”</td>
<td>2’-6”</td>
</tr>
<tr>
<td>#10</td>
<td>1.27</td>
<td>5’-7”</td>
<td>4’-0”</td>
<td>4’-6”</td>
<td>3’-3”</td>
</tr>
<tr>
<td>#11</td>
<td>1.56</td>
<td>6’-10”</td>
<td>4’-11”</td>
<td>5’-6”</td>
<td>3’-11”</td>
</tr>
<tr>
<td>#14</td>
<td>2.25</td>
<td>9’-6”</td>
<td>6’-9”</td>
<td>7’-7”</td>
<td>5’-5”</td>
</tr>
<tr>
<td>#18</td>
<td>4</td>
<td>12’-3”</td>
<td>8’-9”</td>
<td>9’-10”</td>
<td>7’-0”</td>
</tr>
</tbody>
</table>

Notes:

(1) $1.4 \times l_d$
(2) $l_d$
(3) $1.4 \times 0.8 \times l_d$
(4) $0.8 \times l_d$

MODIFICATION FACTORS FOR NORMAL-WEIGHT CONCRETE

1.4, for top horizontal or nearly-horizontal reinforcement, so placed that more than 12 in. of fresh concrete is cast below the reinforcement

0.8, for reinforcement being developed in the length under consideration spaced laterally not less than 6 in. center-to-center, with not less than 3 in. cover measured in the direction of the spacing

DEVELOPMENT LENGTHS FOR UNCOATED BARS IN TENSION

$f'_{c} = 4$ ksi

Figure 405-2F
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Area (in.²)</th>
<th>Top Bars (1)</th>
<th>Others (2)</th>
<th>Top Bars (3)</th>
<th>Others (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>0.11</td>
<td>1'-4''</td>
<td>1'-2''</td>
<td>1'-1''</td>
<td>1'-0''</td>
</tr>
<tr>
<td>#4</td>
<td>0.2</td>
<td>1'-9''</td>
<td>1'-6''</td>
<td>1'-5''</td>
<td>1'-3''</td>
</tr>
<tr>
<td>#5</td>
<td>0.31</td>
<td>2'-2''</td>
<td>1'-11''</td>
<td>1'-9''</td>
<td>1'-6''</td>
</tr>
<tr>
<td>#6</td>
<td>0.44</td>
<td>2'-9''</td>
<td>2'-5''</td>
<td>2'-2''</td>
<td>1'-11''</td>
</tr>
<tr>
<td>#7</td>
<td>0.6</td>
<td>3'-9''</td>
<td>3'-3''</td>
<td>3'-0''</td>
<td>2'-8''</td>
</tr>
<tr>
<td>#8</td>
<td>0.79</td>
<td>4'-11''</td>
<td>4'-4''</td>
<td>3'-11''</td>
<td>3'-6''</td>
</tr>
<tr>
<td>#9</td>
<td>1</td>
<td>6'-2''</td>
<td>5'-5''</td>
<td>4'-11''</td>
<td>4'-4''</td>
</tr>
<tr>
<td>#10</td>
<td>1.27</td>
<td>7'-10''</td>
<td>6'-11''</td>
<td>6'-3''</td>
<td>5'-6''</td>
</tr>
<tr>
<td>#11</td>
<td>1.56</td>
<td>9'-7''</td>
<td>8'-6''</td>
<td>7'-8''</td>
<td>6'-10''</td>
</tr>
<tr>
<td>#14</td>
<td>2.25</td>
<td>13'-4''</td>
<td>11'-9''</td>
<td>10'-8''</td>
<td>9'-5''</td>
</tr>
<tr>
<td>#18</td>
<td>4</td>
<td>17'-3''</td>
<td>15'-2''</td>
<td>13'-9''</td>
<td>12'-2''</td>
</tr>
</tbody>
</table>

Notes:

1. 1.7 x $l_d$
2. 1.5 x $l_d$
3. 1.7 x 0.8 x $l_d$
4. 1.5 x 0.8 x $l_d$

MODIFICATION FACTORS FOR NORMAL-WEIGHT CONCRETE

1.5, for epoxy-coated bars with cover less than $3d_b$, or with clear spacing between bars less than $6d_b$

0.8, for reinforcement being developed in the length under consideration spaced laterally not less than 6 in. center-to-center, with not less than 3 in. cover measured in the direction of the spacing

The product obtained in combining the factor for top reinforcement with the applicable factor for epoxy-coated bars need not be taken as greater than 1.7.

DEVELOPMENT LENGTHS FOR EPOXY-COATED BARS IN TENSION

$f_{c'} = 3$ ksi

Figure 405-2G
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Area (in.²)</th>
<th>Top Bars (1)</th>
<th>Others (2)</th>
<th>Top Bars (3)</th>
<th>Others (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>0.11</td>
<td>1'-4”</td>
<td>1'-2”</td>
<td>1'-1”</td>
<td>1'-0”</td>
</tr>
<tr>
<td>#4</td>
<td>0.2</td>
<td>1'-9”</td>
<td>1'-6”</td>
<td>1'-5”</td>
<td>1'-3”</td>
</tr>
<tr>
<td>#5</td>
<td>0.31</td>
<td>2'-2”</td>
<td>1'-11”</td>
<td>1'-9”</td>
<td>1'-6”</td>
</tr>
<tr>
<td>#6</td>
<td>0.44</td>
<td>2'-7”</td>
<td>2'-3”</td>
<td>2'-1”</td>
<td>1'-10”</td>
</tr>
<tr>
<td>#7</td>
<td>0.6</td>
<td>3'-3”</td>
<td>2'-10”</td>
<td>2'-7”</td>
<td>2'-3”</td>
</tr>
<tr>
<td>#8</td>
<td>0.79</td>
<td>4'-3”</td>
<td>3'-9”</td>
<td>3'-5”</td>
<td>3'-0”</td>
</tr>
<tr>
<td>#9</td>
<td>1</td>
<td>5'-4”</td>
<td>4'-9”</td>
<td>4'-3”</td>
<td>3'-9”</td>
</tr>
<tr>
<td>#10</td>
<td>1.27</td>
<td>6'-9”</td>
<td>6'-0”</td>
<td>5'-5”</td>
<td>4'-10”</td>
</tr>
<tr>
<td>#11</td>
<td>1.56</td>
<td>8'-4”</td>
<td>7'-4”</td>
<td>6'-8”</td>
<td>5'-11”</td>
</tr>
<tr>
<td>#14</td>
<td>2.25</td>
<td>11'-6”</td>
<td>10'-2”</td>
<td>9'-3”</td>
<td>8'-2”</td>
</tr>
<tr>
<td>#18</td>
<td>4</td>
<td>14'-11”</td>
<td>13'-2”</td>
<td>11'-11”</td>
<td>10'-6”</td>
</tr>
</tbody>
</table>

Notes:
1. 1.7 x l_d
2. 1.5 x l_d
3. 1.7 x 0.8 x l_d
4. 1.5 x 0.8 x l_d

MODIFICATION FACTORS FOR NORMAL-WEIGHT CONCRETE

1.5, for epoxy-coated bars with cover less than 3d_b, or with clear spacing between bars less than 6d_b

0.8, for reinforcement being developed in the length under consideration spaced laterally not less than 6 in. center-to-center, with not less than 3 in. cover measured in the direction of the spacing

The product obtained in combining the factor for top reinforcement with the applicable factor for epoxy-coated bars need not be taken as greater than 1.7.

DEVELOPMENT LENGTHS FOR EPOXY-COATED BARS IN TENSION

\[ f_c' = 4 \text{ ksi} \]

Figure 405-2H
### Hooked Uncoated-Bar Development Lengths, Tension

\( f_c' = 3 \text{ ksi} \)

Figure 405-2 I

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>( l_{dh} ) Side Cover &lt; 2.5 in., or Cover on Hook &lt; 2 in.</th>
<th>( l_{dh} ) Side Cover ≥ 2.5 in., or Cover on Hook ≥ 2 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>9” (( l_{dh} = l_{hb} ))</td>
<td>6” (( l_{dh} = 0.7 l_{hb} ))</td>
</tr>
<tr>
<td>#4</td>
<td>11”</td>
<td>8”</td>
</tr>
<tr>
<td>#5</td>
<td>1’-2”</td>
<td>10”</td>
</tr>
<tr>
<td>#6</td>
<td>1’-5”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#7</td>
<td>1’-8”</td>
<td>1’-2”</td>
</tr>
<tr>
<td>#8</td>
<td>1’-10”</td>
<td>1’-4”</td>
</tr>
<tr>
<td>#9</td>
<td>2’-1”</td>
<td>1’-6”</td>
</tr>
<tr>
<td>#10</td>
<td>2’-4”</td>
<td>1’-8”</td>
</tr>
<tr>
<td>#11</td>
<td>2’-7”</td>
<td>1’-10”</td>
</tr>
<tr>
<td>#14</td>
<td>3’-2”</td>
<td>n/a</td>
</tr>
<tr>
<td>#18</td>
<td>4’-2”</td>
<td>n/a</td>
</tr>
<tr>
<td>Bar Size</td>
<td>$l_{dh}$ Side Cover $&lt; 2.5$ in., or Cover on Hook $&lt; 2$ in.</td>
<td>$l_{dh} = l_{hb}$</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>#3</td>
<td>8”</td>
<td>6”</td>
</tr>
<tr>
<td>#4</td>
<td>10”</td>
<td>7”</td>
</tr>
<tr>
<td>#5</td>
<td>1'-0”</td>
<td>9”</td>
</tr>
<tr>
<td>#6</td>
<td>1'-3”</td>
<td>10”</td>
</tr>
<tr>
<td>#7</td>
<td>1'-5”</td>
<td>1'-0”</td>
</tr>
<tr>
<td>#8</td>
<td>1'-7”</td>
<td>1'-2”</td>
</tr>
<tr>
<td>#9</td>
<td>1'-10”</td>
<td>1'-4”</td>
</tr>
<tr>
<td>#10</td>
<td>2'-1”</td>
<td>1'-5”</td>
</tr>
<tr>
<td>#11</td>
<td>2'-3”</td>
<td>1'-7”</td>
</tr>
<tr>
<td>#14</td>
<td>2'-9”</td>
<td>n/a</td>
</tr>
<tr>
<td>#18</td>
<td>3'-7”</td>
<td>n/a</td>
</tr>
</tbody>
</table>

**HOOKED UNCOATED-BAR DEVELOPMENT LENGTHS, TENSION**

$f'_{c} = 4$ ksi

**Figure 405-2J**
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>$l_{dh}$ Side Cover $&lt; 2.5$ in., or Cover on Hook $&lt; 2$ in.</th>
<th>$l_{dh}$ Side Cover $\geq 2.5$ in., or Cover on Hook $\geq 2$ in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>10” $l_{dh} = l_{hb}$</td>
<td>7” $l_{dh} = 0.7 l_{hb}$</td>
</tr>
<tr>
<td>#4</td>
<td>1’-2”</td>
<td>10”</td>
</tr>
<tr>
<td>#5</td>
<td>1’-5”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#6</td>
<td>1’-8”</td>
<td>1’-2”</td>
</tr>
<tr>
<td>#7</td>
<td>2’-0”</td>
<td>1’-5”</td>
</tr>
<tr>
<td>#8</td>
<td>2’-3”</td>
<td>1’-7”</td>
</tr>
<tr>
<td>#9</td>
<td>2’-6”</td>
<td>1’-9”</td>
</tr>
<tr>
<td>#10</td>
<td>2’-10”</td>
<td>2’-0”</td>
</tr>
<tr>
<td>#11</td>
<td>3’-2”</td>
<td>2’-2”</td>
</tr>
<tr>
<td>#14</td>
<td>3’-9”</td>
<td>n/a</td>
</tr>
<tr>
<td>#18</td>
<td>5’-0”</td>
<td>n/a</td>
</tr>
</tbody>
</table>

**HOOKED EPOXY-COATED-BAR DEVELOPMENT LENGTHS, TENSION**

$fe’ = 3$ ksi

*Figure 405-2K*
### Hooked Epoxy-Coated-Bar Development Lengths, Tension

\[ f' = 4 \text{ ksi} \]

**Figure 405-2L**

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>( l_{dh} ) Side Cover &lt; 2.5 in., or Cover on Hook &lt; 2 in.</th>
<th>( l_{dh} ) Side Cover ≥ 2.5 in., or Cover on Hook ≥ 2 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>( l_{dh} = l_{hb} ) 9”</td>
<td>( l_{dh} = 0.7 l_{hb} ) 6”</td>
</tr>
<tr>
<td>#4</td>
<td>1’-0”</td>
<td>8”</td>
</tr>
<tr>
<td>#5</td>
<td>1’-3”</td>
<td>10”</td>
</tr>
<tr>
<td>#6</td>
<td>1’-6”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#7</td>
<td>1’-8”</td>
<td>1’-2”</td>
</tr>
<tr>
<td>#8</td>
<td>1’-11”</td>
<td>1’-4”</td>
</tr>
<tr>
<td>#9</td>
<td>2'-2”</td>
<td>1’-7”</td>
</tr>
<tr>
<td>#10</td>
<td>2’-5”</td>
<td>1’-9”</td>
</tr>
<tr>
<td>#11</td>
<td>2’-9”</td>
<td>1’-11”</td>
</tr>
<tr>
<td>#14</td>
<td>3’-3”</td>
<td>n/a</td>
</tr>
<tr>
<td>#18</td>
<td>4’-4”</td>
<td>n/a</td>
</tr>
</tbody>
</table>
### CLASS A SPLICE LENGTH FOR UNCOATED BARS IN TENSION

\[ f_{c'} = 3 \text{ ksi} \]

**Figure 405-2M**

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing (&lt;6\text{ in.}, \text{ or Cover} &lt;3\text{ in.})</th>
<th>Center to Center Spacing (\geq6\text{ in.}, \text{ or Cover} \geq3\text{ in.})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1’-1”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#4</td>
<td>1’-5”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#5</td>
<td>1’-9”</td>
<td>1’-3”</td>
</tr>
<tr>
<td>#6</td>
<td>2’-3”</td>
<td>1’-8”</td>
</tr>
<tr>
<td>#7</td>
<td>3’-1”</td>
<td>2’-2”</td>
</tr>
<tr>
<td>#8</td>
<td>4’-0”</td>
<td>2’-11”</td>
</tr>
<tr>
<td>#9</td>
<td>5’-1”</td>
<td>3’-8”</td>
</tr>
<tr>
<td>#10</td>
<td>6’-5”</td>
<td>4’-7”</td>
</tr>
<tr>
<td>#11</td>
<td>7’-11”</td>
<td>5’-8”</td>
</tr>
</tbody>
</table>

**Notes:**
1. All splice lengths in feet and inches
2. \(d_b < \text{Cover}\)
3. \(2d_b < \text{Clear Spacing}\)
4. Values are for normal weight concrete.
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1’-1”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#4</td>
<td>1’-5”</td>
<td>1’-0”</td>
</tr>
<tr>
<td>#5</td>
<td>1’-9”</td>
<td>1’-3”</td>
</tr>
<tr>
<td>#6</td>
<td>2’-2”</td>
<td>1’-6”</td>
</tr>
<tr>
<td>#7</td>
<td>2’-8”</td>
<td>1’-11”</td>
</tr>
<tr>
<td>#8</td>
<td>3’-6”</td>
<td>2’-6”</td>
</tr>
<tr>
<td>#9</td>
<td>4’-5”</td>
<td>3’-2”</td>
</tr>
<tr>
<td>#10</td>
<td>5’-7”</td>
<td>4’-0”</td>
</tr>
<tr>
<td>#11</td>
<td>6’-10”</td>
<td>4’-11”</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b < \text{Cover}$
2. $2d_b < \text{Clear Spacing}$
3. Value is for normal-weight concrete.

CLASS A SPLICE LENGTH FOR UNCOATED BARS IN TENSION

$$f_c' = 4 \text{ ksi}$$

Figure 405-2N
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1’-4”</td>
<td>1’-2”</td>
</tr>
<tr>
<td>#4</td>
<td>1’-9”</td>
<td>1’-6”</td>
</tr>
<tr>
<td>#5</td>
<td>2’-2”</td>
<td>1’-11”</td>
</tr>
<tr>
<td>#6</td>
<td>2’-9”</td>
<td>2’-5”</td>
</tr>
<tr>
<td>#7</td>
<td>3’-9”</td>
<td>3’-3”</td>
</tr>
<tr>
<td>#8</td>
<td>4’-11”</td>
<td>4’-4”</td>
</tr>
<tr>
<td>#9</td>
<td>6’-2”</td>
<td>5’-5”</td>
</tr>
<tr>
<td>#10</td>
<td>7’-10”</td>
<td>6’-11”</td>
</tr>
<tr>
<td>#11</td>
<td>9’-7”</td>
<td>8’-6”</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b \leq \text{Cover} < 3d_b$
2. $2d_b \leq \text{Clear Spacing} < 6d_b$
3. Value is for normal-weight concrete.

**CLASS A SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION**

$f_c’ = 3\text{ ksi}$

**Figure 405-2 O**
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1’-4”</td>
<td>1’-2”</td>
</tr>
<tr>
<td>#4</td>
<td>1’-9”</td>
<td>1’-6”</td>
</tr>
<tr>
<td>#5</td>
<td>2’-2”</td>
<td>1’-11”</td>
</tr>
<tr>
<td>#6</td>
<td>2’-7”</td>
<td>2’-3”</td>
</tr>
<tr>
<td>#7</td>
<td>3’-3”</td>
<td>2’-10”</td>
</tr>
<tr>
<td>#8</td>
<td>4’-3”</td>
<td>3’-9”</td>
</tr>
<tr>
<td>#9</td>
<td>5’-4”</td>
<td>4’-9”</td>
</tr>
<tr>
<td>#10</td>
<td>6’-9”</td>
<td>6’-0”</td>
</tr>
<tr>
<td>#11</td>
<td>8’-4”</td>
<td>7’-4”</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b \leq \text{Cover} < 3d_b$
2. $2d_b \leq \text{Clear Spacing} < 6d_b$
3. Value is for normal-weight concrete.

CLASS A SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION

\[ f_c' = 4 \text{ ksi} \]

Figure 405-2P
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1'-5”</td>
<td>1'-0”</td>
</tr>
<tr>
<td>#4</td>
<td>1'-10”</td>
<td>1'-4”</td>
</tr>
<tr>
<td>#5</td>
<td>2'-4”</td>
<td>1'-8”</td>
</tr>
<tr>
<td>#6</td>
<td>2'-11”</td>
<td>2'-1”</td>
</tr>
<tr>
<td>#7</td>
<td>4'-0”</td>
<td>2'-10”</td>
</tr>
<tr>
<td>#8</td>
<td>5'-3”</td>
<td>3'-9”</td>
</tr>
<tr>
<td>#9</td>
<td>6'-7”</td>
<td>4'-9”</td>
</tr>
<tr>
<td>#10</td>
<td>8'-5”</td>
<td>6'-0”</td>
</tr>
<tr>
<td>#11</td>
<td>10'-3”</td>
<td>7'-4”</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b < \text{Cover}$
2. $2d_b < \text{Clear Spacing}$
3. Value is for normal-weight concrete.

**CLASS B SPLICE LENGTH FOR UNCOATED BARS IN TENSION**

$f_{c'} = 3 \text{ ksi}$

Figure 405-2Q
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1'-5&quot;</td>
<td>1'-0&quot;</td>
</tr>
<tr>
<td>#4</td>
<td>1'-10&quot;</td>
<td>1'-4&quot;</td>
</tr>
<tr>
<td>#5</td>
<td>2'-4&quot;</td>
<td>1'-8&quot;</td>
</tr>
<tr>
<td>#6</td>
<td>2'-9&quot;</td>
<td>2'-0&quot;</td>
</tr>
<tr>
<td>#7</td>
<td>3'-5&quot;</td>
<td>2'-6&quot;</td>
</tr>
<tr>
<td>#8</td>
<td>4'-6&quot;</td>
<td>3'-3&quot;</td>
</tr>
<tr>
<td>#9</td>
<td>5'-9&quot;</td>
<td>4'-1&quot;</td>
</tr>
<tr>
<td>#10</td>
<td>7'-3&quot;</td>
<td>5'-2&quot;</td>
</tr>
<tr>
<td>#11</td>
<td>8'-11&quot;</td>
<td>6'-5&quot;</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b < \text{Cover}$
2. $2d_b < \text{Clear Spacing}$
3. Value is for normal-weight concrete.

**CLASS B SPLICE LENGTH FOR UNCOATED BARS IN TENSION**

\[ f_{c'} = 4 \text{ ksi} \]

*Figure 405-2R*
## Class B Splice Length for Epoxy-Coated Bars in Tension

### $f'_{c} = 3 \text{ ksi}$

**Figure 405-2 S**

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing $&lt; 6$ in., or Cover $&lt; 3$ in.</th>
<th>Center to Center Spacing $\geq 6$ in., or Cover $\geq 3$ in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1’-8”</td>
<td>1’-6”</td>
</tr>
<tr>
<td>#4</td>
<td>2’-3”</td>
<td>2’-0”</td>
</tr>
<tr>
<td>#5</td>
<td>2’-10”</td>
<td>2’-6”</td>
</tr>
<tr>
<td>#6</td>
<td>3’-7”</td>
<td>3’-2”</td>
</tr>
<tr>
<td>#7</td>
<td>4’-10”</td>
<td>4’-3”</td>
</tr>
<tr>
<td>#8</td>
<td>6’-4”</td>
<td>5’-7”</td>
</tr>
<tr>
<td>#9</td>
<td>8’-0”</td>
<td>7’-1”</td>
</tr>
<tr>
<td>#10</td>
<td>10’-2”</td>
<td>9’-0”</td>
</tr>
<tr>
<td>#11</td>
<td>12’-6”</td>
<td>11’-0”</td>
</tr>
</tbody>
</table>

**Notes:**

1. $d_b \leq \text{Cover} < 3d_b$
2. $2d_b \leq \text{Clear Spacing} < 6d_b$
3. Value is for normal-weight concrete.
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1’-8”</td>
<td>1’-6”</td>
</tr>
<tr>
<td>#4</td>
<td>2’-3”</td>
<td>2’-0”</td>
</tr>
<tr>
<td>#5</td>
<td>2’-10”</td>
<td>2’-6”</td>
</tr>
<tr>
<td>#6</td>
<td>3’-4”</td>
<td>3’-0”</td>
</tr>
<tr>
<td>#7</td>
<td>4’-2”</td>
<td>3’-8”</td>
</tr>
<tr>
<td>#8</td>
<td>5’-6”</td>
<td>4’-10”</td>
</tr>
<tr>
<td>#9</td>
<td>6’-11”</td>
<td>6’-2”</td>
</tr>
<tr>
<td>#10</td>
<td>8’-10”</td>
<td>7’-9”</td>
</tr>
<tr>
<td>#11</td>
<td>10’-10”</td>
<td>9’-7”</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b \leq \text{Cover} < 3d_b$
2. $2d_b \leq \text{Clear Spacing} < 6d_b$
3. Value is for normal-weight concrete.

CLASS B SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION

$f_{c'} = 4 \text{ ksi}$

Figure 405-2T
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
</tr>
<tr>
<td>#3</td>
<td>1’-10”</td>
<td>1’-4”</td>
</tr>
<tr>
<td>#4</td>
<td>2’-5”</td>
<td>1’-9”</td>
</tr>
<tr>
<td>#5</td>
<td>3’-0”</td>
<td>2’-2”</td>
</tr>
<tr>
<td>#6</td>
<td>3’-10”</td>
<td>2’-9”</td>
</tr>
<tr>
<td>#7</td>
<td>5’-2”</td>
<td>3’-9”</td>
</tr>
<tr>
<td>#8</td>
<td>6’-10”</td>
<td>4’-11”</td>
</tr>
<tr>
<td>#9</td>
<td>8’-8”</td>
<td>6’-2”</td>
</tr>
<tr>
<td>#10</td>
<td>10’-11”</td>
<td>7’-10”</td>
</tr>
<tr>
<td>#11</td>
<td>13’-5”</td>
<td>9’-7”</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b < \text{Cover}$
2. $2d_b < \text{Clear Spacing}$
3. Value is for normal-weight concrete.

**CLASS C SPLICE LENGTH FOR UNCOATED BARS IN TENSION**

$f_c' = 3 \text{ ksi}$

*Figure 405-2U*
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
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<td>#3</td>
<td>1’-10”</td>
<td>1’-4”</td>
</tr>
<tr>
<td>#4</td>
<td>2’-5”</td>
<td>1’-9”</td>
</tr>
<tr>
<td>#5</td>
<td>3’-0”</td>
<td>2’-2”</td>
</tr>
<tr>
<td>#6</td>
<td>3’-7”</td>
<td>2’-7”</td>
</tr>
<tr>
<td>#7</td>
<td>4’-6”</td>
<td>3’-3”</td>
</tr>
<tr>
<td>#8</td>
<td>5’-11”</td>
<td>4’-3”</td>
</tr>
<tr>
<td>#9</td>
<td>7’-6”</td>
<td>5’-4”</td>
</tr>
<tr>
<td>#10</td>
<td>9’-6”</td>
<td>6’-9”</td>
</tr>
<tr>
<td>#11</td>
<td>11’-8”</td>
<td>8’-4”</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b < \text{Cover}$
2. $2d_b < \text{Clear Spacing}$
3. Value is for normal-weight concrete.

**CLASS C SPLICE LENGTH FOR UNCOATED BARS IN TENSION**

$f_{c'} = 4 \text{ ksi}$

**Figure 405-2V**
<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Bars</td>
<td>Others</td>
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<tr>
<td>#3</td>
<td>2’-3”</td>
<td>1’-11”</td>
</tr>
<tr>
<td>#4</td>
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<td>4’-8”</td>
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<td>#7</td>
<td>6’-4”</td>
<td>5’-7”</td>
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<td>#8</td>
<td>8’-3”</td>
<td>7’-4”</td>
</tr>
<tr>
<td>#9</td>
<td>10’-6”</td>
<td>9’-3”</td>
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<tr>
<td>#10</td>
<td>13’-3”</td>
<td>11’-9”</td>
</tr>
<tr>
<td>#11</td>
<td>16’-4”</td>
<td>14’-5”</td>
</tr>
</tbody>
</table>

Notes:
1. \(d_b \leq \text{Cover} < 3d_b\)
2. \(2d_b \leq \text{Clear Spacing} < 6d_b\)
3. Value is for normal-weight concrete.

**CLASS C SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION**

\[ f_c' = 3 \text{ ksi} \]

**Figure 405-2W**
<table>
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<th>Bar Size</th>
<th>Center to Center Spacing &lt; 6 in., or Cover &lt; 3 in.</th>
<th>Center to Center Spacing ≥ 6 in., or Cover ≥ 3 in.</th>
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</thead>
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<td>Top Bars</td>
<td>Others</td>
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<td>#3</td>
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<tr>
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<tr>
<td>#5</td>
<td>3'-8”</td>
<td>3'-3”</td>
</tr>
<tr>
<td>#6</td>
<td>4'-5”</td>
<td>3'-10”</td>
</tr>
<tr>
<td>#7</td>
<td>5'-6”</td>
<td>4'-10”</td>
</tr>
<tr>
<td>#8</td>
<td>7'-2”</td>
<td>6'-4”</td>
</tr>
<tr>
<td>#9</td>
<td>9'-1”</td>
<td>8'-0”</td>
</tr>
<tr>
<td>#10</td>
<td>11'-6”</td>
<td>10'-2”</td>
</tr>
<tr>
<td>#11</td>
<td>14'-2”</td>
<td>12'-6”</td>
</tr>
</tbody>
</table>

Notes:
1. $d_b \leq \text{Cover} < 3d_b$
2. $2d_b \leq \text{Clear Spacing} < 6d_b$
3. Value is for normal-weight concrete.

CLASS C SPLICE LENGTH FOR EPOXY-COATED BARS IN TENSION

\[ f_e' = 4 \text{ ksi} \]

Figure 405-2X
HOOKS AND BENDS

Figure 405-2Y
COVER ——— 2"
STIRRUP ——— 5/8"
½ BAR Ø ——— 19/32"
ALLOWANCE FOR STIRRUP BEND ——— 1/4"

DIMENSION "A" = 3 7/16" MIN.

NOTE: IF SCALE IS 1:10 OR SMALLER, THE STIRRUP SHOULD BE SHOWN AS A SINGLE BROKEN LINE.

BARS IN SECTION
Figure 405-2Z
EXAMPLE NO. 1

802c X 36'-9"

EXAMPLE NO. 2

502c X 9'-9"

BENDING DIAGRAM EXAMPLES

Figure 405-2AA
CUTTING DIAGRAM
(Transverse Steel in Bridge Deck)

Figure 405-2BB
NOTE:
A CUTTING DIAGRAM CAN ALSO BE USED WHEN STIRRUPS ARE PLACED AT TWO DIFFERENT SPACINGS WITH TWO SEPARATE BAR MARKS. "NO. OF BARS" AND CUTTING DIAGRAM DIMENSIONS FOR EACH BAR MARK CAN BE SHOWN IN A TABLE.

CUTTING DIAGRAM
(Hammerhead Stem Pier)

Figure 405-2CC
### Plain Reinforcing Steel

<table>
<thead>
<tr>
<th>Size and Mark</th>
<th>No. of Bars</th>
<th>Length</th>
<th>Weight (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>503</td>
<td>69</td>
<td>19'-10&quot;</td>
<td></td>
</tr>
<tr>
<td>591</td>
<td>144</td>
<td>20'-7&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>48</td>
<td>31'-0&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>54</td>
<td>29'-4&quot;</td>
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</tr>
<tr>
<td>#5</td>
<td>2</td>
<td>26'-0&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>1</td>
<td>24'-8&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>2</td>
<td>22'-0&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>1</td>
<td>20'-8&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>2</td>
<td>18'-4&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>49</td>
<td>16'-8&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>2</td>
<td>14'-4&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>1</td>
<td>12'-8&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>2</td>
<td>10'-4&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>1</td>
<td>9'-0&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>2</td>
<td>6'-4&quot;</td>
<td></td>
</tr>
<tr>
<td>#5</td>
<td>1</td>
<td>5'-0&quot;</td>
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</table>

Total No. 5 8853

<table>
<thead>
<tr>
<th>Size and Mark</th>
<th>No. of Bars</th>
<th>Length</th>
<th>Weight (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>401</td>
<td>14</td>
<td>3'-7&quot;</td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td>2</td>
<td>19'-8&quot;</td>
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</table>

Total No. 4 63

Total Plain Reinforcing Steel 8916

### Epoxy-Coated Reinforcing Steel

<table>
<thead>
<tr>
<th>Size and Mark</th>
<th>No. of Bars</th>
<th>Length</th>
<th>Weight (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#8</td>
<td>4</td>
<td>19'-8&quot;</td>
<td>210</td>
</tr>
<tr>
<td>#7</td>
<td>4</td>
<td>19'-8&quot;</td>
<td>161</td>
</tr>
<tr>
<td>502</td>
<td>5</td>
<td>20'-2&quot;</td>
<td></td>
</tr>
<tr>
<td>581</td>
<td>51</td>
<td>6'-9&quot;</td>
<td></td>
</tr>
<tr>
<td>591a</td>
<td>31</td>
<td>5'-7&quot;</td>
<td></td>
</tr>
<tr>
<td>593</td>
<td>62</td>
<td>4'-2&quot;</td>
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</table>

Total No. 5 914

Total Epoxy-Coated Reinforcing Steel 1285

### Concrete

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<tr>
<th>Material Description</th>
<th>Quantity</th>
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<tr>
<td>Reinf.-Conc. Bridge Appr., 15 in.</td>
<td>2950 ft²</td>
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<tr>
<td>Concrete Railing Class C</td>
<td>2.5 yd²</td>
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REINFORCED-CONCRETE BRIDGE APPROACH BILL OF MATERIALS

Figure 405-2DD
HAUNCH CONFIGURATIONS FOR REINFORCED CONCRETE SLAB SUPERSTRUCTURES

Figure 405-3A

(a) PARABOLIC

(b) STRAIGHT

(c) DROP PANEL

(d) CAP BEAM *

* THIS CONFIGURATION SHOULD NOT BE USED AS A STRUCTURAL HAUNCH
TYPICAL REINFORCED CONCRETE SLAB SUPERSTRUCTURE

Figure 405-3B
<table>
<thead>
<tr>
<th>Slab Thickness (in.)</th>
<th>Reinforcement, Top and Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 18</td>
<td>#5 @ 1’-6” or #4 @ 1’-0”</td>
</tr>
<tr>
<td>18 ≤ Thick. ≤ 28</td>
<td>#5 @ 1’-0” or #4 @ 8”</td>
</tr>
<tr>
<td>&gt; 28</td>
<td>Design per LRFD Article 5.10.8.2</td>
</tr>
</tbody>
</table>

SHRINKAGE AND TEMPERATURE REINFORCEMENT FOR SLAB SUPERSTRUCTURE

Figure 405-3C
NOTE: Entire typical section to be detailed on plans.

1. Bar spacing and number of spaces to be determined to facilitate a constant bar spacing in remainder of slab.

2. Design edge beam in accordance with articles in the LRFD Specifications, but use as a minimum the same area of steel per foot as in slab.

3. For depth of keyway, use on-third the slab thickness.

INTEGRAL CAP AT SLAB SUPERSTRUCTURE
(Typical Half-Section)

Figure 405-3D
INTEGRAL CAPS AT SLAB SUPERSTRUCTURE
(Half Longitudinal Section)

Figure 405-3E
INTEGRAL CAP AT SLAB SUPERSTRUCTURE
(Section Through End Bent)

Figure 405-3F
INTEGRAL CAP AT SLAB SUPERSTRUCTURE
(Section Through Interior Bent)

Figure 405-3G
CHAPTER 406

Prestressed-Concrete Structure

<table>
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<tr>
<th>Design Memorandum</th>
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<th>Sections Affected</th>
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<td>LIST OF FIGURES</td>
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<td>406-4.05 Post-Tensioning Anchorage and Couplers</td>
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<td>406-12B</td>
<td>Dimensioning Prestressed-Concrete Beam on Slope</td>
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<td>Bulb-Tee Beam Type BT 36 x 49 Sections Showing Prestressing and Mild</td>
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<td>406-14H</td>
<td>Bulb-Tee Beam Type BT 42 x 49 Sections Showing Prestressing and Mild Reinforcing Steel</td>
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<td>406-14V</td>
<td>Wide Bulb-Tee Beam Type BT 54 x 61 Sections Showing Prestressing and Mild Reinforcing Steel</td>
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CHAPTER 406

PRESTRESSED CONCRETE

406-1.0 GENERAL [REV. MAY 2013]

The requirements of this Chapter will apply to each bridge designed with normal or lightweight concrete reinforced with prestressed or post-tensioned strands. Partial prestressing is not permitted. The requirements described herein are based on a 28-day concrete strength, \( f'_c \), of 4 to 8 ksi.

406-2.0 DEFINITIONS

See LRFD 5.2.

406-3.0 NOTATIONS

See LRFD 5.3.

406-4.0 MATERIAL PROPERTIES

406-4.01 General

The material properties cited herein are based on the construction materials specified in LRFD 5.4. The minimum acceptable properties and test procedures shall be specified in the contract documents.


The minimum \( f'_c \) for prestressed or post-tensioned concrete components shall be shown on the plans. Such a strength outside the range shown in Section 406-1.0 is not permitted without written approval of the Director of Bridges. For lightweight concrete, the air dry unit weight shall be shown on the plans as 119 lb/ft\(^3\). The modulus of elasticity will be calculated using the 119 lb/ft\(^3\) value. The unit weight of the lightweight concrete will be taken as 124 lb/ft\(^3\). The
additional weight is to account for the mild reinforcing steel and the tensioning strands. See \textit{LRFD} 5.4.2.2 for the coefficient of linear expansion.

The following will apply to concrete.

1. The design compressive strength of normal-weight and lightweight concrete at 28 days, \( f'_c \), shall be in the range as follows:
   
   a. prestressed box beam: 5 to 7 ksi  
   b. prestressed I-beam: 5 to 7 ksi  
   c. prestressed bulb-tee beam: 6 to 8 ksi  

   An exception to the range shown above will be allowed for a higher strength if the higher strength can be documented to be of significant benefit to the project, it can be effectively produced, and approval is obtained from the Director of Bridges.

2. At release of the prestressing strands, \( f'_c \) shall not be less than 4 ksi, and shall be determined during the beam design. The specified concrete compressive strength at release shall be rounded to the next higher 0.1 ksi.

\textbf{406-4.02(01) Shrinkage and Creep}

Losses due to shrinkage and creep, for other than a segmentally-constructed bridge, that require a more-precise estimate including specific materials, structural dimensions, site conditions, construction methods, and age at various stages of erection, can be estimated by means of the methods specified in \textit{LRFD} 5.4.2.3.2 and 5.4.2.3.3. Other acceptable methods are those described in the CEB-FIP 1978 / 1990 code. The annual average ambient relative humidity shall be taken as 70%.

\textbf{406-4.02(02) Modulus of Elasticity, Poisson’s Ratio, and Modulus of Rupture}

The modulus of elasticity shall be calculated as specified in \textit{LRFD} Eqn. 5.4.2.4-1. Poisson’s ratio shall be taken as 0.2. See \textit{LRFD} 5.4.2.6 for modulus-of-rupture values depending on whether the concrete is normal weight or lightweight, and whether the intended application is control of cracking, deflection, camber, or shear resistance.

The use of lightweight concrete, with normal-weight sand mixed with lightweight coarse aggregate, is permitted with a specified density of 119 lb/ft³. The use of lightweight concrete shall be demonstrated to be necessary and cost effective during the structure-size-and-type study.

The modulus of elasticity will be less than that for normal-weight concrete. Creep, shrinkage, and deflection shall be appropriately evaluated and accounted for if lightweight concrete is to be used. The formula shown in LRFD 5.4.2.6 shall be used in lieu of physical test values for modulus of rupture. The formula for sand-lightweight concrete shall be used for lightweight concrete.

406-4.04 Prestressing Steel

Prestressing strands shall be of the low-relaxation type with a minimum tensile strength of 270 ksi. Unless there is a reason to do otherwise, only the following three-strand diameters shall be used.

1. Nominal 3/8 in., $A_s = 0.085$ in², for use in a stay-in-place deck panel.

2. Nominal 1/2 in., $A_s = 0.167$ in², for use in an I, bulb-tee, or box beam, or post-tensioned member.

3. Nominal 0.6 in., $A_s = 0.217$ in², for use in a bulb-tee beam or post-tensioned member.

See LRFD Table 5.4.4.1-1 for values of yield strength, tensile strength, and modulus of elasticity of prestressing strands or bars.

Prestressing threadbars are used for grouted construction. If the bars are used for permanent non-grouted construction, the bars shall be epoxy coated.

406-4.05 Post-Tensioning Anchorage and Couplers

See LRFD 5.4.5 regarding the use of anchorages or couplers. Tendons, anchorages, end fittings, and couplers shall be protected against corrosion. If couplers are used to connect bars, they shall be enclosed in duct housings long enough to permit the necessary movement.
406-4.06 Ducts

See LRFD 5.4.6 for types of ducts, radius-of-curvature limits, general considerations for tendons, and size requirements. Polyethylene ducts shall be used in a corrosive environment such as in a bridge deck or in a substructure element under a joint. The contract documents shall indicate the type of duct material to be used.

Ducts for a post-tensioned bulb-tee beam shall be of round, semi-rigid, galvanized-metal. The wall thickness shall not be less than 28 gage. A radius that requires prebending shall be avoided if possible.

If the bridge is to be constructed by means of post-tensioning precast components together longitudinally or transversely by use of a cast-in-place concrete joint, the end of each duct shall be extended beyond the concrete interface for not less than 3 in. and not more than 6 in. to facilitate joining the ducts. If necessary, the extension can be in a local blockout at the concrete interface. Joints between sections of ducts shall be positive metallic connections, which do not result in angle changes at the joints.

The plans shall show all pertinent geometry required for development of working drawings with the correct tendon alignment in both elevation and plan views. This includes anchorage regions and areas where there are tight or reverse curvatures of the the tendons.

Curved ducts that run parallel to each other or around a void or re-entrant corner shall be encased in concrete and reinforced as necessary to avoid radial failure, or pullout into another duct or void.

Upon completion of post-tensioning, the ducts shall be grouted.

Ducts or anchorage assemblies for post-tensioning shall be provided with a pipe or other suitable connection at each end for the injection of grout after prestressing. A duct of over 200 ft in length shall be in accordance with the recommendations of the PTI. Vents shall be ½-in. minimum diameter standard pipe or suitable plastic pipe. Connections to the ducts shall be made with metallic or plastic fasteners. Plastic components, if selected and approved, shall not react with the concrete or enhance corrosion of the prestressing steel, and shall be free of water-soluble chlorides. The vents shall be mortar tight, taped as necessary, and shall provide means for injection of grout through the vents and for positive sealing of the vents. Ends of steel vents shall be removed at least 1 in. below the concrete deck surface, if appropriate, after the grout has set. Ends of plastic vents shall be removed to the surface of the concrete after the grout has set.
Grout injection pipes shall be fitted with positive mechanical shut-off valves. Vents and ejection pipes shall be fitted with valves. Caps shall not be removed or opened until the grout has set.

406-5.0 LIMIT STATES

406-5.01 General

In addition to service, fatigue, strength, and extreme-event limit states, prestressed or post-tensioned components shall be investigated for stresses and deformations for each critical stage during construction, stressing, handling, transportation, and erection.

406-5.02 Service-Limit State

See LRFD 5.7.3.4 for control of cracking, 5.7.3.6 for deformations, and 5.9.4 for stress limits for concrete.

406-5.03 Fatigue-Limit State

See LRFD 5.5.3.1 to determine if fatigue shall be considered. Use the stress ranges for prestressing tendons provided in LRFD 5.5.3.3 if necessary.

406-5.04 Strength-Limit State

See LRFD 5.5.4.2 and Table 5.5.4.2.2-1 for resistance factors for tension-controlled, compression-controlled, or transition prestressed concrete sections, shear, torsion, or anchorage zones. For segmental construction, the resistance factors shall be modified for flexure and shear accordingly. See LRFD 5.5.4.3 for required stability checks.

406-5.05 Extreme-Event-Limit State

See LRFD 5.4.6.2 and Table 3.4.1-1.
See *LRFD* 5.6.3 and 5.10.9.4 for a strut-and-tie model overview that can be used in the design of an anchorage zone, deep beam, bracket, or corbel.

The assumptions for service, fatigue, strength, and extreme-event-limit states are described in *LRFD* 5.7.1. Strength equations provided in *LRFD* 5.7.2 are based on the rectangular-stress-block approach.

For stress calculations in prestressing steel at nominal flexure resistance, the equations for components with bonded and unbonded tendons provided in *LRFD* 5.7.3.1.1 and 5.7.3.1.2 are acceptable. For components with both bonded and unbonded tendons, the simplified analysis provided in *LRFD* 5.7.3.1.3b is acceptable.

The flexural resistance may be computed with the equations provided in *LRFD* 5.7.3.2, or a more-precise calculation can be used as described in *LRFD* 5.7.3.2.5.

The amount of prestressed tensile reinforcement is limited by the minimum reinforcement requirements provided in *LRFD* 5.7.3.3.2. There is not a limitation in *LRFD* 5.7.4.2 for maximum prestressed tensile reinforcement for flexural members. For compression members, the maximum and minimum reinforcement requirements provided in *LRFD* 5.7.4.2 are applicable. Slenderness effects, computation of factored axial resistance, and biaxial flexure effects provided in *LRFD* 5.7.4.4 are applicable to prestressed concrete columns.

Instantaneous deflections, long-term deflections, and cambers shall be computed with the modulus of elasticity, moment of inertia, and cracking moment specified in *LRFD* 5.7.3.6.2. The methods provided therein to obtain long term deflections from instantaneous deflections are acceptable.

### 406-7.0 SHEAR AND TORSION

#### 406-7.01 General

*LRFD* 5.6.3 allows the strut-and-tie model and the sectional-design model for shear design of prestressed concrete. In a region near a discontinuity, the strut-and-tie model shall be used. The sectional-design model is appropriate for the design of a girder, slab, or other region of components where the assumptions of traditional beam theory are valid. See *LRFD* 5.8.3 and *PCI Bridge Design Manual* Section 8.12 for more information regarding the strut-and-tie model.
Torsional effects shall be investigated only if the condition in *LRFD* 5.8.2.1 for normal weight concrete, or slightly modified for lightweight concrete, is satisfied. If necessary, the torsional resistance obtained with the sectional-design model is provided in *LRFD* 5.8.2.2.

Transverse reinforcement, minimum and maximum requirements, types of transverse reinforcement, and shear stresses of concrete are provided in *LRFD* 5.8.2.4. They shall be satisfied in the design. These are complemented by *LRFD* 5.8.2.9 for a segmental post-tensioned concrete box-girder bridge.

406-7.02 Sectional-Design Model

*LRFD* 5.8.3 discusses the sectional-design model. The general formulas used to calculate the nominal shear resistance are shown in *LRFD* 5.8.3.3-1. The parameters used to evaluate these expressions depend on the adopted approach. For a prestressed-concrete structure, the use of either the modified-compression field theory or the simplified procedure is acceptable.

In order to account for the tension caused by shear, the longitudinal-reinforcement requirements shown in *LRFD* 5.8.3.6.3 shall be satisfied for sections subjected to combined shear and torsion.

406-7.03 Interface Shear Transfer – Shear Friction

A cast-in-place concrete deck designed to act compositely with precast-concrete beams shall be able to resist the horizontal shearing forces at the interface between the two elements. *LRFD* 5.8.4.1 discusses the requirements for shear transfer. The nominal shear resistance of the interface and the factored interface shear force are provided in *LRFD* 5.8.4.2. The most appropriate condition of the interaction between concrete-slab and concrete-girder surfaces shall be selected individually from the six situations of cohesion and friction factors provided in *LRFD* 5.8.4.3. The requirement for minimum area of interface shear reinforcement provided in *LRFD* 5.8.4.4 shall be satisfied.

406-7.04 Segmental Concrete Bridge

*LRFD* 5.8.5 states that principal stresses determined using classic beam theory and the principles of Mohr’s circle and local tensions in the web from anchorage of tendons shall be investigated. The principal tensile stress shall not exceed the tensile stress limits shown in *LRFD* Table 5.9.4.2.2-1.
The shear and torsion requirements shown in \textit{LRFD} 5.8.3.6 supersede those for the design of a segmental post-tensioned concrete box girder bridge. \textit{LRFD} 5.8.2 applies, but with the modifications shown in \textit{LRFD} 5.8.3 through 5.8.5.

\textbf{406-8.0 PRESTRESSED CONCRETE}

\textbf{406-8.01 General Considerations and Stress Limitations}

General requirements for design, concrete strength, buckling, section properties, crack control, location of tendon relative to the duct, and stresses due to imposed deformations are provided in \textit{LRFD} 5.9.1. Contrary to the \textit{LRFD} requirements, INDOT does not allow a transformed section for a pretensioned member.

The stress limitations for prestressing tendons, deformed high-strength bars, and concrete before and after losses have occurred, are provided in \textit{LRFD} 5.9.3.

\textbf{406-8.02 Loss of Prestress}

Total prestress losses in a member constructed and prestressed in a single stage are provided in \textit{LRFD} 5.9.5.1. The losses are defined as instantaneous losses such as anchorage set, friction, elastic shortening, and time-dependent losses. The acceptable methods provided for determining the time-dependent losses are an approximate and a refined method. The refined method can be used for the final design of a nonsegmental prestressed concrete member. For a post-tensioned concrete member with multistage construction or prestressing, the prestress losses shall be computed by means of a time-dependent-analysis method such as that described in CEB-FIP (1978 / 1990). The approximate lump-sum estimate method shall be used for preliminary design only.

The values of the wobble and curvature friction coefficients, and the anchor-set loss assumed for the design shall be shown on the plans.

\textbf{406-9.0 PRESTRESSING-REINFORCEMENT REQUIREMENTS}

\textbf{406-9.01 Spacing of Prestressing Tendons and Ducts}
The minimum and maximum spacing of pretensioned strands, and curved and straight post-tensioning ducts in the horizontal plane are provided in LRFD 5.10.3.3. However, bundling of ducts will not be permitted. LRFD 5.10.3.5 provides requirements for longitudinal post-tensioning couplers.

406-9.02 Tendon Confinement and Effects of Curved Tendons

LRFD 5.10.4 provides general requirements for tendon confinement, and in-plane and out-of-plane force effects. Shear resistance of the concrete cover against pullout shall be satisfied, or fully-anchored tiebacks shall be provided. Local confining reinforcement shall be provided if out-of-plane forces exceed the factored shear resistance of concrete cover. LRFD 5.10.5 provides the maximum unsupported length of external tendons.

406-9.03 Post-Tensioned and Pretensioned Anchorage Zone

LRFD 5.10.9 provides general requirements for the anchorage zones at the end and intermediate anchorages. Requirements for design of general zones are provided in LRFD 5.10.9.3. The strut-and-tie, elastic stress analysis, and approximate method are acceptable. The design requirements for a local zone including special anchorage devices are provided in LRFD 5.10.9.7. For a pretensioned anchorage zone, LRFD 5.10.10 shall be followed.

406-10.0 DEVELOPMENT OF PRESTRESSING STRANDS AND DEBONDING

The transfer length of pretensioned components is shown in LRFD 5.11.4.1. The specific requirements for development length and variation of pretensioned stress in strands for bonded or debonded strands are provided in LRFD 5.11.4.2 and 5.11.4.3. Where debonded, or shielded, strands are used, the following apply.

1. In a bulb-tee beam, not more than 25% of the total number of strands and not more than 40% in each horizontal row shall be debonded. The allowable percentage of debonded strands for an AASHTO I-beam or a box beam shall be not more than 50% of the total number of strands and of the strands in each horizontal row. Strands placed in the top flange of the beam shall not be included in the percentages shown above.

2. Exterior strands in each horizontal row shall not be debonded.

3. Bonded and debonded strands shall preferably alternate both vertically and horizontally.
4. Debonding termination points shall be staggered at intervals of not less than 3 ft.

5. Not more than four strands, or 40% of the total debonded strands, whichever is greater, shall be terminated at one point.

See LRFD 5.11.4.3 for additional guidelines.

6. Two strands shall be considered in the top of a box beam, 2 or 4 strands in the top flange of an I-beam, or up to 6 strands in a bulb-tee beam. This can significantly reduce the need for debonded strands in the bottom of the beam, and it facilitates the placement of the top mild reinforcement. Where strands are placed in the top flange, a note shall be shown on the plans indicating that these strands are to be cut at the center of the beam after the bottom strands are released and the pocket is then to be filled with grout. The top strands may not need to be cut if ultimate moment controls the number of strands in the bottom flange.

7. Top strands in a concrete box beam shall be placed near the sides of the box.

Minimum concrete cover for prestressing strands and metal ducts, and the general protection requirements for prestressing tendons are described in LRFD 5.12.

**406-11.0 DIAPHRAGMS**

Design considerations for diaphragms are provided in LRFD 5.13.2.2.

**406-11.01 General Requirements**

A multi-girder bridge, except for one with adjacent box beams, shall have diaphragms provided at the abutments, end bents, and interior piers or bents, to resist lateral forces and transmit loads to points of support. For certain span lengths, permanent intermediate diaphragms shall be provided to stabilize the beams during construction.

To simplify the bill of materials, the longitudinal and transverse reinforcing bars in concrete diaphragms and transverse edge beams, except the #6 threaded bars, may be epoxy coated.

**406-11.02 Intermediate Diaphragms**
Intermediate diaphragms shall be provided for an I-beam or bulb-tee beam superstructure as follows.

1. For a span greater than 80 ft but less than or equal to 120 ft, provide diaphragms at the midspan.

2. For a span greater than 120 ft, provide diaphragms at the span third points.

A spread-box-beam superstructure having an inside radius of curvature of less than 800 ft shall have intermediate diaphragms between the individual boxes. The required spacing will depend upon the radius of curvature and the proportions of the webs and flanges. The diaphragms shall be placed on the radial lines. Other box-beam superstructures do not require intermediate diaphragms.

**406-11.03 Structural-Steel and Reinforced-Concrete Interior Diaphragms**

Structural-steel interior diaphragms shall be specified if interior diaphragms are required. This use of structural steel instead of concrete does not affect the bridge design. Structural-steel interior diaphragms shall be shown on the plans. The quantities in pounds shall be shown in the superstructure bill of materials and on the Bridge Summary sheet.

If it is determined that cast-in-place concrete interior diaphragms shall be used, the Director of Bridges shall be provided with a written justification for the concrete diaphragms. Once the Director concurs in the justification, such diaphragms shall be shown on the plans. The required quantities of concrete and reinforcing steel shall be incorporated into those for the bridge deck.

A note shall also be placed on the plans that states the following:

*Concrete in the intermediate diaphragms shall attain a compressive strength of 3 ksi before the deck concrete is poured.*

**406-11.04 End Diaphragms**

End diaphragms or edge beams are mandatory, though not on an adjacent precast-concrete box-beam superstructure. Integral end bents function as full-depth diaphragms. An end diaphragm serves the purposes as follows:

1. as a perimeter beam for the deck;
2. supports the deck-joint device; and
3. transfers lateral loads to the end bent.

For typical details of an end diaphragm, or transverse edge beam, see Section 404-3.03. For typical details of integral end bents, see Section 409-2.01.

406-11.05 Interior Pier or Bent Diaphragms

End diaphragms are mandatory at each interior pier or bent, except for an adjacent precast concrete-box-beam superstructure. They serve the purposes as follows:

1. transfer lateral loads to the piers or bents, and
2. for beams made continuous for live load, strengthen the cast-in-place closure placement by providing lateral restraint.

The minimum diaphragm width for bulb-tee beams shall be 36 in., for I-beams 30 in., or for spread box beams 24 in. The clear distance between beam ends shall be 6 in. unless otherwise approved. This dimension shall always be determined parallel to the longitudinal centerline of the beam.

See Section 406-16.0 for typical details of cast-in-place concrete pier and bent diaphragms. The information illustrated in the figures therein is as follows:

1. diaphragm widths;
2. diaphragm reinforcement;
3. cap-keyway details;
4. clear distance between adjacent beam ends;
5. bearing-pad location details;
6. cap-sizing details; and
7. beam threaded-bar-hole/insert location details.

The figures in Section 406-16.0 also show bearing layouts for a skewed structure with I-beams, bulb-tee beams, or box beams. For the same skew angle, the bearing pads are oriented differently in these figures. The ideal orientation of the pads, between the direction of the beams and the normal drawn to the bearing line, is a function of the skew, the length-to-width ratio, the component rigidities, and the position of loads. The structural significance of the orientation is small, therefore geometric requirements shall govern.
406-12.0  ADDITIONAL DESIGN FEATURES

406-12.01  General

Design requirements are provided for beams and girders, segmental construction, arches and slab superstructures in LRFD 5.14.1 through 5.14.4.

406-12.02  Prestressed-Concrete-Member Sections

406-12.02(01)  General

The standard prestressed-concrete-member sections used are as follows:

1. AASHTO I-beam type I, II, III, or IV;
2. Indiana bulb-tee beams; and
3. Indiana composite and non-composite box beams.

To ensure that the structural system has an adequate level of redundancy, a minimum of four beam lines shall be used.

An alternative prestressed-concrete-beam section may be considered if its use can be justified. The use of a beam section not available through local producers will be more expensive if the forms must be purchased or rented for a small number of beams. One or more beam fabricators shall be contacted early in project development to determine the most practical and cost-effective alternative beam section for a specific site.

406-12.02(02)  AASHTO I-Beam Type I, II, III, or IV

See Figures 406-13A through 406-13D for details and section properties. I-beam type IV shall not be used unless widening of an existing bridge is required. The 54-in.-depth beam shall be used for a new structure where this member depth and span length is required.

See Figures 406-14A through 406-14F, and 406-14M through 406-14R for details and section properties. For a long-span bridge, bulb-tee beams with a top-flange width of 60 in. shall be considered for improved stability during handling and transporting. Draped strands may be considered for use in a bulb-tee beam, but shall only be considered if tensile stresses in the top of the beam near its end are exceeded if using straight strands. The maximum allowable compressive strength, tensile strength, extent of strand debonding, and number of top strands shall be considered in evaluating the need for draped strands. If draped strands are used, the maximum allowable hold-down force per strand shall be 3.8 kip, with a maximum total hold-down force of 38 kip. For additional information on draped strands, see Section 406-12.03. Semi-lightweight concrete may be used for this type of beam if it is economically justified. See Section 406-4.03. Lightweight concrete may be used for this type of beam if it is economically justified. See Section 406-4.03.

Prestressed-concrete bulb-tee members identified as wide bulb-tees have been approved for use. One of these sections shall be considered if it is deemed to be more economical or structurally adequate than an Indiana bulb-tee member. See Figures 406-14G through 406-14L, and 406-14S through 406-14X for details and section properties.

406-12.02(04) Indiana Composite or Non-Composite Box Beam

See Figures 406-15A through 406-15L for details and section properties of composite members. See Figures 406-15M through 406-15R for details and section properties of non-composite members. It is not acceptable to use non-composite box beams for a permanent State highway bridge. The use of non-composite box beams is limited to a non-Federal-aid local public agency bridge, or a temporary bridge. The desirable limit for the end skew is 30 deg. An end skew of over 30 deg shall be avoided unless measures have been considered for potential warping or cracking of the beam at its ends and congested reinforcement in the acute angle corner of the beam.

For a spread-box-beam structure, diaphragms of 8-in. thickness shall be placed within the box section for increased stability and torsion resistance during delivery and erection of the beams. The maximum spacing of the diaphragms is 25 ft.

For an adjacent-box-beams structure, interior diaphragms shall be provided to accommodate the transverse tension rods or tendons. Effective means for transferring shear between the box beams shall be provided (see Section 406-12.06). Because the longitudinal joints between
adjacent box beams have shown a tendency to leak, use of adjacent box beams shall be limited to where maintaining a thin construction depth is critical, where construction time is critical, or where substantial life-cycle cost savings can be demonstrated.

Each void in a box beam shall be equipped with a vertical drainage pipe to prevent accumulation of water and ice therein. The inside diameter of the pipe shall be approximately 5/8 in. It shall be located at the lowest point of the void in the finished structure.

If the cost of a superstructure using precast-concrete AASHTO I-beams or bulb-tees is close to the cost of precast concrete spread box beams, the I-beam or bulb-tee superstructure is preferred unless other factors such as a thin structure depth are critical.

406-12.03 Strand Configuration and Mild-Steel Reinforcement

406-12.03(01) General

Mild reinforcing steel shall be detailed to allow its placement after the strands have been tensioned. If the reinforcement is a one-piece bar to be placed around the strands, it requires that the strands be threaded through the closed bars. By using two-piece bars that can be placed after the strand is tensioned, the fabrication process is simplified.

In specifying concrete cover and spacing of strands and bars, reinforcing-bar diameters and bend radii shall be considered to avoid conflicts. Beam producers prefer to locate at least two strands in the top of each I-beam or bulb-tee beam below the top transverse bars and between the vertical legs of the web reinforcement to support the reinforcing-steel cage. This will also reduce the need for debonded strands.

406-12.03(02) Prestressing-Strands Configuration

See Sections 406-13.0, 406-14.0, and 406-15.0 for typical strand patterns for standard prestressed beam sections. Other strand patterns may be used if there is reason for deviation from the standard pattern, and the LRFD criteria for spacing and concrete cover are followed. If 11 strands are placed in a horizontal row in the bottom of a bulb-tee beam, the bending diagram for the vertical stirrup must be modified. The strand pattern shown may be used for nominal ½-in. or 0.6-in. diameter strands. Section 406-4.03 provides criteria for the strand diameters used.

The strand-pattern configurations shown in Sections 406-13.0, 406-14.0, and 406-15.0 were developed in accordance with the following.
1. Minimum center-to-center spacing of prestressing strands equal to 2 in.

2. Minimum concrete cover for prestressing strands shall be 1½ in., which includes the modification factor of 0.8 for a water/cement ratio equal to or less than 0.40 as described in LRFD 5.12.3.

3. Minimum concrete cover to stirrups and confinement reinforcement shall be 1 in.

The strand pattern has been configured so as to maximize the number of vertical rows of strands that can be draped. Due to the relatively thin top flange of a bulb-tee beam, strands placed in the top of the beam shall be at least 6 in. from the outside edge of the flange.

**406-12.03(03) Mild-Steel Reinforcement**

See Sections 406-13.0, 406-14.0, and 406-15.0 for typical mild-steel reinforcement configurations for the standard prestressed beam sections. The vertical shear reinforcement shall be #4 stirrup bars where possible. To fully develop the bar for shear, the ends of the stirrup bar shall include a standard 90-deg stirrup hook. The maximum spacing of the vertical stirrups shall be in accordance with LRFD 5.8.2.7. The maximum longitudinal spacing of reinforcement for interface shear transfer shall be in accordance with LRFD 5.8.4.1.

A minimum of three horizontal U-shaped #4 bars shall be placed in the web of each bulb-tee at the ends of the beam. See Section 406-14.0 for location and spacing of these bars. This reinforcement will help reduce the number and size of cracks, which can appear in the ends of the beams due to the prestress force. LRFD 5.10.10.1 requires that vertical mild reinforcement shall be placed in the beam ends within a distance of one fourth of the member depth. This is to provide bursting resistance of the pretensioned anchorage zone. Enough mild reinforcing steel shall be provided to resist not less than 4% of the prestress force at transfer. The end vertical bars shall be as close to the ends of the beam as possible. The stress in the reinforcing steel shall not exceed 20 ksi.

Confinement reinforcement in accordance with LRFD 5.10.10.2 shall be placed in the bottom flange of each I-beam or bulb-tee as shown in Section 406-13.0 or 406-14.0, respectively. The reinforcement shall be #3 bars spaced at 6 in. for a minimum distance of 1.5 times the depth of the member from the end of the beam or to the end of the strand debonding, whichever is greater.
406-12.04 Stage Loading for Pretensioned Construction

406-12.04(01) Strands Tensioned in the Stressing Bed

It is the fabricator’s responsibility to consider seating losses, relaxation of the strands, and temperature changes in the strands prior to placement of the concrete during the fabrication of the beam and to make adjustments to the initial strand tension to ascertain that the tension prior to release satisfies the design requirements.

406-12.04(02) Strands Released and Force Transferred to the Concrete

The region near the end of the member does not receive the benefit of bending stresses due to dead load, and can develop tensile stresses in the top of the beam large enough to crack the concrete. The critical sections for computing the critical temporary stresses in the top of the beam shall be near the end and at all debonding points. If the transfer length of the strands is chosen to be at the end of the beam and at the debonding points, the stress in the strands shall be assumed to be zero at the end of the beam or debonding point, and shall vary linearly to the full transfer of force to the concrete at the end of the strand transfer length.

The accepted methods to relieve excessive tensile stresses near the ends of the beam are as follows:

1. debonding, wherein the strands are kept straight but wrapped in plastic over a predetermined distance;

2. adding additional strands in the top of the beam, debonding them in the middle third, and releasing them at the center of the beam; or

4. draping some of the strands to reduce the strand eccentricity at the end of the beam.

406-12.04(03) Camber Growth and Prestress Losses

This condition occurs several weeks to several months after strand release. If a cast-in-place composite deck is placed, field adjustments to the haunch-fillet thickness are needed to provide the proper vertical grade on the top of deck and to keep the deck thickness uniform. Reliable estimates of deflection and camber are needed to prevent excessive fillet thickness or to avoid significant encroachment of the top of beam into the bottom of the concrete deck. Stresses at this stage are not critical.
Unless other more-accurate methods of determining camber are utilized (see *PCI Bridge Design Manual*, Section 8.7), the beam camber at the time of placement of the composite concrete deck shall be assumed to be the initial camber due to prestress minus the deflection due to the dead load of the beam times a multiplier of 1.75.

### 406-12.04(04) Maximum Service Load, Minimum-Prestress Stage

At this stage, all prestress losses have occurred and loads are at their maximum. The tensile stress in the bottom fibers of the beam at mid-span will likely control the design.

### 406-12.05 Continuity for Superimposed Loads

A multi-span bridge using composite beams shall be made continuous for live load if possible. The design of the beams for a continuous structure is approximately the same as that for simple spans except that, in the area of negative moments, the member is treated as an ordinary reinforced-concrete section. The members shall be assumed to be fully continuous with a constant moment of inertia in determining both the positive and negative moments due to superimposed loads.

The traditional method of making simply-supported beams continuous is to construct a closure joint between the adjacent beam ends over the pier, conveniently as part of the diaphragm, and to place extra longitudinal steel in the deck over the pier support to resist the negative moment. Spans made continuous for live load are assumed to be treated as prestressed members in the positive-moment zone between supports, and as conventionally-reinforced members in the negative-moment zone over the support. The reinforcing steel in the deck shall carry all of the tension in the composite section due to the negative moment. The longitudinal reinforcing steel in the deck that makes the girder continuous over an internal support shall be designed in accordance with *LRFD* 5.14.1.4.8.

Continuity diaphragms shall be designed in accordance with *LRFD* 5.14.1.4.10 based on the compressive strength of the precast girder regardless of the strength of the cast-in-place concrete.

No allowable tension limit is imposed on the top-fiber stresses of the beam in the negative-moment region. However, crack width, fatigue, and ultimate strength shall be checked. If partial-depth precast, prestressed concrete stay-in-place forms are to be used, such as for an AASHTO I-beam superstructure, only the top mat of longitudinal steel reinforcement shall be used to satisfy the negative-moment requirements.
406-12.06 Effect of Imposed Deformations

Potential positive moments at the piers shall also be considered in the design of a precast, prestressed-concrete beam structure made continuous for live load. Creep of the beams under the net effects of prestressing, self-weight, deck weight, and superimposed dead loads will tend to produce additional upward camber with time. Shrinkage of the deck concrete will tend to produce downward camber of the composite system with time. Loss of prestress due to creep, shrinkage, or relaxation will result in downward camber. Depending on the properties of the concrete materials and the age at which the beams are erected and subsequently made continuous, either positive or negative moments can occur over the continuous supports.

Where beams are made continuous at the relatively young age of less than 120 days from time of manufacture, it is more likely that positive moments will develop with time at the supports. These positive restraint moments are the result of the tendency of the beams to continue to camber upward as a result of ongoing creep strains associated with the transfer of prestress. Shrinkage of the concrete deck, loss of prestress, or creep strains due to self-weight, deck weight, or superimposed dead loads all have a tendency to reduce this positive moment.

For a span of over 150 ft or for concrete whose creep behavior is known to be poor, a time-dependent analysis shall be made to predict positive restraint moments at the piers. The PCI Bridge Design Manual, Section 8.13.4.3, describes two methods to evaluate restraint moments at the piers. Positive-moment connections at the piers that have proven successful in the past shall be used based on experience with similar spans and concrete-creep properties.

Unless positive-moment-connecting steel calculations are made, the minimum number of strands to be used for the positive-moment connection over the pier shall be one-half the number of strands in the bottom row of the bottom flange of a bulb-tee or an I-beam. The minimum is 5 strands for a bulb-tee or I-beam type IV, 4 strands for an I-beam type II or III, or 3 strands for an I-beam type I.

The strands shall be extended and bent up without the use of heat to make the positive moment connection. For a box beam, the minimum number of strands to be extended into the positive-moment connection and bent up shall be 6 strands for a beam deeper than 27 in., or 4 strands for a beam depth equal to or less than 27 in.

The strands extended into the positive-moment connection between beams shall not be debonded. The strands that are not used for the positive-moment connection shall be trimmed back to the beam end to permit ease of beam and concrete placement.
The prestressing-strand and concrete strengths shall be as shown in Section 406-4.0. The tensile and compressive stress limits shall be as shown in LRFD 5.9.4. LRFD requires that only 80% of the live-load moment is to be applied in checking the tensile stress at service condition.

406-12.07 Transverse Connection of Precast Box Beams

The shear keys on adjacent, precast, prestressed box beams tend to crack and leak with a thin concrete deck placed composite with the beams. The following methods shall be considered toward minimizing cracking in the shear keys between the beams.

1. Use epoxy grout due to its high bond strength.

2. Use a full-depth shear key to stop the joint from performing like a hinge and prevent the joint from opening. In the past, the area below the key was open and free to rotate. With this area grouted, the movement of the joint will be reduced.

3. Apply compression across the joint by means of transverse tensioning rods. This will help prevent opening of the joint.

Figure 406-12A illustrates a method to minimize cracking in this type of structure.

The joints between the beam shear keys, and the recesses for the transverse tensioning rods on the exterior face of the beam, shall be grouted with an epoxy grout as shown in the figure.

After the joints between the beams are grouted, a preliminary tightening of the transverse tensioning rods shall be performed. Once this is completed, a final tensioning of the rods shall be performed to yield 20 ksi.

406-12.08 Segmental Construction

Prestressed concrete beam lengths in the range of 100 ft to 120 ft are common. For a continuous structure, the girders are fabricated in lengths to span from support to support. A closure pour is then made over the piers to provide continuity for live load and superimposed dead loads. This type of construction is cost effective because the girders can be erected in one piece without falsework. However, if girders are too long or too heavy to be shipped in lengths to accommodate the spans, spliced girders or segmental construction are options. Construction techniques have been developed that reduce the cost and can make concrete girders competitive.
with steel girders for spans in excess of 250 ft. The most commonly-used techniques are as follows:

1. segmental post-tensioned box girders erected on temporary falsework or by means of the balanced cantilever method; or

2. precast-concrete girders spliced at the construction site. These girders can either be supported on temporary falsework or spliced on the ground and lifted into place onto the supports.

Most spliced-girder bridges have bulb-tee beams with post-tensioning. Cambers, deflections, stresses, and end rotations of the structural components shall be calculated during the stages of construction.

For further information, see publications of the Precast/Prestressed Concrete Institute, Post-Tensioning Institute, and the Segmental Concrete Bridge Institute.

406-12.09 Dimensioning Precast Beams

If a precast beam is to be placed on a longitudinal slope, its manufactured dimensions shall be modified to accommodate the geometric consequences of the grade. The casting bed is always horizontal. Consequently, the out-to-out beam length becomes $L_{CL}$, as follows:

$$L_{CL} = L / \cos \theta$$

where:

- $\theta$ = arctan ($S/100$) = angle of slope of the beam
- $L$ = length of beam as it appears in plan view, ft
- $S$ = slope of beam as shown in elevation view, percent

As shown in Figure 406-12B, the plans and the working drawings shall identify dimensions $L$, $L_{CL}$, $a$, and $b$. The seat surfaces are always horizontal and the end surfaces are always vertical once the beam is in place.

Maintaining vertical end surfaces of the in-place beams often has only a minimal effect on the constructability of this type of superstructure, and need be considered only where dimension $b$ in Figure 406-12B exceeds 1½ in.
If the slope of the beam between supports is more than 1.0%, a beveled recess will be required in the bottom of the beam at the supports. For an integral end bent, a steel sole plate cast into the beam recess will be required. The recess in the bottom flange of the beam shall have a minimum recess dimension of ¼ in. The minimum concrete cover over the prestressing strands at the opposite end of the recess shall be 1 in. as shown in Figure 406-12B. For a severe grade where use of the minimum ¼-in. recess results in less than 1 in. of cover, either the beam seat shall be sloped, or the bottom strand clearance shall be increased in ¼-in. increments until the 1-in. cover is achieved.

For a beam length in excess of 80 ft, the length of the prestressing strands prior to release shall be increased due to the elastic shortening, creep, and shrinkage anticipated to occur prior to casting the deck slab. Due to variables beyond the designer’s control, the beam fabricator is responsible for making this change.

To avoid sharp corners which can be damaged during construction due to a skew of 15 deg or greater, a chamfer of at least 3 in. width shall be placed at each acute corner of a prestressed box beam.

### 406-12.10 Other Design Features

**406-12.10(01) Skew [Rev. Apr. 2017]**

Although normal flexural effects due to live load tend to decrease as the skew angle increases, shear does not. There is a considerable redistribution of shear forces in the end zone due to the development of involuntary negative moments therein. For a skew angle of less than 30 deg, the skew may be ignored, and the bridge may be analyzed as a square structure whose span lengths are equal to the skewed span lengths.

*LRFD* 4.6.2.2.2e and 4.6.2.2.3c provide tabulated assistance to roughly estimate these live-load effects. The factors shown in the tables can be applied to either a simple span or a continuous-spans skewed bridge. The correction factors for shear apply only to support shears at the obtuse corner of an exterior beam. Shear in portions of the beam away from the end supports need not be corrected for skew effects.

To obtain a better assessment of skewed-structure behavior and to utilize potential benefits in reduced live-load moments, more sophisticated methods of analysis are required. The refined methods most often used to study skewed-structure behavior are the grillage analysis and the finite element method. The finite element analysis requires the fewest simplifying assumptions.
in accounting for the greatest number of variables that govern the structural response of the bridge. However, input preparation time and derivation of overall forces for a composite beam can be tedious. Data preparation for the grillage method is simpler, and integration of stresses is not needed.

406-12.10(02) Shortening of Superstructure

For a long continuous structure, the shortening of the superstructure due to creep, shrinkage, temperature, and post-tensioning, if applicable, shall be considered in the design of the beam supports and the substructure.

406-13.0 AASHTO I-BEAMS

Figures 406-13A through 406-13D show details and section properties for these beams.

406-14.0 INDIANA BULB-TEE BEAMS

Figures 406-14A through 406-14Z show details and section properties for these beams.

406-15.0 INDIANA COMPOSITE AND NON-COMPOSITE BOX BEAMS

Figures 406-15A through 406-15R show details and section properties for these beams.

406-16.0 MISCELLANEOUS DETAILS

Figures 406-16A through 406-16X show details for diaphragms, closure pours, support cap sizing, and bearing pad layouts for I beams, bulb-tees, and box beams.
OUT-TO-OUT COPINGS = 
REQ'D BEAM WIDTHS + 
REQ'D JOINT WIDTHS.

NON-COMPOSITE

COMPOSITE

1" Ø rod through 3" Ø holes

ADJACENT BOX BEAMS WITH TRANSVERSE TENSIONING RODS
(Section View)

Figure 406-12A
NOTES:

1. BEAM FABRICATOR IS RESPONSIBLE FOR ADJUSTING THE CASTING LENGTH TO ACCOMMODATE AN INCREASE IN BEAM LENGTH DUE TO DIMENSION b. IF DIMENSION b IS LESS THAN 1 1/2 in., THE END OF THE BEAM SHALL NOT BE ADJUSTED.

2. THE LENGTH OF THE RECESS SHALL PROVIDE A BEAM OVERHANG OF AT LEAST 2 in. PAST THE FACE OF THE BEARING PAD FOR A BULB-T BEAM OR 1 in. FOR AN I-BEAM OR A BOX BEAMS.

DIMENSIONING PRESTRESSED-CONCRETE BEAM ON SLOPE

Figure 406-12B
BEAM PROPERTIES

\[ A_B = 276 \text{ in.}^2 \]
\[ I_B = 22,744 \text{ in.}^4 \]
\[ S_{TB} = 1,476 \text{ in.}^3 \]
\[ S_{BB} = 1,807 \text{ in.}^3 \]
\[ Y_{TB} = 15.4 \text{ in.} \]
\[ Y_{BB} = 12.6 \text{ in.} \]
\[ \text{Wt.} = 288 \text{ lb/lf} \]

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

I - BEAM TYPE I

Figure 406-13A
(page 1 of 3)
Figure 406-13A
I - BEAM TYPE I
BAR BENDING DETAILS

* DENOTES EPOXY-COATED BAR
I - BEAM TYPE I

ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-13A
(page 3 of 3)
**BEAM PROPERTIES**

- $A_B = 369 \text{ in.}^2$
- $I_B = 50,979 \text{ in.}^4$
- $S_{TB} = 2,527 \text{ in.}^3$
- $S_{BB} = 3,221 \text{ in.}^3$
- $Y_{TB} = 20.2 \text{ in.}$
- $Y_{BB} = 15.8 \text{ in.}$
- Wt. = 384 lb/lf

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

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**Figure 406-13B**

* I - BEAM TYPE II

(page 1 of 3)
I - BEAM TYPE II
BAR BENDING DETAILS

Figure 406-13B
(page 2 of 3)
I - BEAM TYPE II
ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-13B
(page 3 of 3)
BEAM PROPERTIES

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_B$</td>
<td>560 in.$^2$</td>
</tr>
<tr>
<td>$I_B$</td>
<td>125,390 in.$^4$</td>
</tr>
<tr>
<td>$S_{TB}$</td>
<td>5,071 in.$^3$</td>
</tr>
<tr>
<td>$S_{BB}$</td>
<td>6,185 in.$^3$</td>
</tr>
<tr>
<td>$Y_{TB}$</td>
<td>24.7 in.</td>
</tr>
<tr>
<td>$Y_{BB}$</td>
<td>20.3 in.</td>
</tr>
<tr>
<td>Wt.</td>
<td>583 lb/ft</td>
</tr>
</tbody>
</table>

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

I - BEAM TYPE III

Figure 406-13C
(page 1 of 3)
I - BEAM TYPE III
BAR BENDING DETAILS

Figure 406-13C
(page 2 of 3)
Figure 406-13C
ELEVATIONS SHOWING END REINFORCEMENT

I - BEAM TYPE III

*DENOTES EPOXY-COATED BAR

SHOP BEND AS SHOWN. DO NOT HEAT.
BEAM PROPERTIES

\[ A_B = 789 \text{ in.}^2 \]
\[ I_B = 260,741 \text{ in.}^4 \]
\[ S_{TB} = 8,909 \text{ in.}^3 \]
\[ S_{BB} = 10,542 \text{ in.}^3 \]
\[ Y_{TB} = 29.3 \text{ in.} \]
\[ Y_{BB} = 24.7 \text{ in.} \]
\[ \text{Wt.} = 822 \text{ lb/lf} \]

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

I - BEAM TYPE IV

Figure 406-13D
(Page 1 of 3)
I - BEAM TYPE IV
BAR BENDING DETAILS

Figure 406-13D
(Page 2 of 3)
SHOP BEND AS SHOWN. DO NOT HEAT.

CUT BONDED STRANDS IN BOTTOM ROW WITH 1'-10" PROJECTION AND SHOP BEND AS SHOWN. DO NOT HEAT.

*DENOTES EPOXY-COATED BAR

Figure 406-13D
(Page 3 of 3)
BULB - TEE BEAM
TYPE BT 54 x 48

Figure 406-14A
(Page 1 of 2)
BULB - TEE BEAM
TYPE BT 54 x 48
BAR BENDING DETAILS

Figure 406-14A
(Page 2 of 2)
BULB - TEE BEAM
TYPE BT 60 x 48

Figure 406-14B
(Page 1 of 2)
BULB - TEE BEAM
TYPE BT 60 x 48
BAR BENDING DETAILS

Figure 406-14B
(Page 2 of 2)
Figure 406-14C

TYPE BT 66 x 48

BULB - TEE BEAM

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

BEAM PROPERTIES

A_B = 974 in^2
I_B = 572,338 in^4
STB = 18,794 in^3
S_BB = 16,101 in^3
Y_TB = 30.5 in
Y_BB = 35.5 in
Wt. = 1015 lb/ft
* DENOTES EPOXY-COATED BARS

BULB - TEE BEAM
TYPE BT 66 x 48
BAR BENDING DETAILS

Figure 406-14C
(Page 2 of 2)
**BEAM PROPERTIES**

- $A_B = 1016 \text{ in}^2$
- $I_B = 712,670 \text{ in}^2$
- $STB = 21,354 \text{ in}^3$
- $S_{BB} = 18,451 \text{ in}^3$
- $Y_{TB} = 33.4 \text{ in}$
- $Y_{BB} = 38.6 \text{ in}$
- Wt. = 1058 lb/lf

**BULB - TEE BEAM**

**TYPE BT 72 x 48**

Figure 406-14D

*NOTES:*

1. BARS 301 AND 302 COMBINED TO FORM ONE STIRRUP.

2. *DENOTES EPOXY-COATED BARS*
* DENOTES EPOXY-COATED BARS

BULB - TEE BEAM
TYPE BT 72 x 48
BAR BENDING DETAILS

Figure 406-14D
(Page 2 of 2)
BULB - TEE BEAM
TYPE BT 78 x 48

Figure 406-14E
(Page 1 of 2)
BULB - TEE BEAM
TYPE BT 78 x 48
BAR BENDING DETAILS

Figure 406-14E
(Page 2 of 2)
BEAM PROPERTIES

A_g = 1100 in^2
I_g = 1,048,921 in^4
S_TB = 26,733 in^3
S_BB = 23,433 in^3
Y_TB = 39.2 in
Y_BB = 44.8 in
W_T = 1146 lb/ft

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BULB - TEE BEAM
TYPE BT 84 x 48

Figure 406-14F
(Page 1 of 2)

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

BULB - TEE BEAM
TYPE BT 84 x 48
BAR BENDING DETAILS

Figure 406-14F
(Page 2 of 2)
BULB-TEE BEAM TYPE BT 36 x 49
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14G
(Page 1 of 2)

NOTES:
1. *DENOTES EPOXY-COATED BARS.
2. LOCATE HOLDDOWNS 5'-0" EACH SIDE OF CENTER LINE OF BEAM.
3. BARS 301 AND 302 COMBINE TO FORM ONE STIRRUP.
BULB-TEE BEAM TYPE BT 36 x 49
BAR BENDING DETAILS
BULB-TEE BEAM TYPE BT 42 x 49
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14H
(Page 1 of 2)

NOTES:
1. *DENOTES EPOXY-COATED BARS.
2. LOCATE HOLDDOWNS 5'-0" EACH SIDE OF CENTER LINE OF BEAM.
3. BARS 301 AND 302 COMBINE TO FORM ONE STIRRUP.
BULB-TEE BEAM TYPE BT 42 x 49
BAR BENDING DETAILS

Figure 406-14H
(Page 2 of 2)
BULB-TEE BEAM TYPE BT 48 x 49
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14 I
(Page 1 of 2)
BULB-TEE BEAM TYPE BT 48 x 49
BAR BENDING DETAILS

Figure 406-14 I
(Page 2 of 2)
BULB-TEE BEAM TYPE BT 54 x 49
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14J
(Page 1 of 2)
BULB-TEE BEAM TYPE BT 54 x 49
BAR BENDING DETAILS

Figure 406-14J
(Page 2 of 2)
BULB-TEE BEAM TYPE BT 60 x 49
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14K
(Page 1 of 2)
DENOTES EPOXY-COATED BARS

BULB-TEE BEAM TYPE BT 60 x 49
BAR BENDING DETAILS

Figure 406-14K
(Page 2 of 2)
NOTES:
1. *DENOTES EPOXY-COATED BARS.
2. LOCATE HOLDOWNS 5'-0" EACH SIDE OF CENTER LINE OF BEAM.
3. BARS 301 AND 302 COMBINE TO FORM ONE STIRRUP.

BULB-TEE BEAM TYPE BT 66 x 49
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14L
(Page 1 of 2)
BULB-TEE BEAM TYPE BT 66 x 49
BAR BENDING DETAILS
Figure 406-14M
(Page 1 of 2)
* DENOTES EPOXY-COATED BARS

BULB-TEE BEAM TYPE BT 54 x 60
BAR BENDING DETAILS

Figure 406-14M
(Page 2 of 2)
Figure 406-14N
(Page 1 of 2)
BULB - TEE BEAM
TYPE BT 60 x 60
BAR BENDING DETAILS

Figure 406-14N
(Page 2 of 2)
**BEAM PROPERTIES**

- \( A_B = 1018 \text{ in}^2 \)
- \( I_B = 608,591 \text{ in}^4 \)
- \( S_{TB} = 20,859 \text{ in}^3 \)
- \( S_{BB} = 16,527 \text{ in}^3 \)
- \( Y_{TB} = 29.2 \text{ in} \)
- \( Y_{BB} = 36.8 \text{ in} \)
- Wt. = 1060 lb/ft

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**NOTES:**

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

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**BULB - TEE BEAM**

**TYPE BT 66 x 60**

Figure 406-14 O

(Page 1 of 2)
BULB - TEE BEAM
TYPE BT 66 x 60
BAR BENDING DETAILS

Figure 406-14 O
(Page 2 of 2)
NOTES:
1. BARS 301 AND 302 COMBINED TO FORM ONE STIRRUP.
2. *DENOTES EPOXY-COATED BARS

Figure 406-14P
(Please find the full figure on the following page.)
BULB - TEE BEAM
TYPE BT 72 x 60
BAR BENDING DETAILS

Figure 406-14P
(Page 2 of 2)
NOTES:
1. BARS 301 AND 302 COMBINED TO FORM ONE STIRRUP.
2. *DENOTES EPOXY-COATED BARS

BULB - TEE BEAM
TYPE BT 78 x 60

Figure 406-14Q
(Page 1 of 2)
BULB - TEE BEAM
TYPE BT 78 x 60
BAR BENDING DETAILS

Figure 406-14Q
(Page 2 of 2)
**BEAM PROPERTIES**

- $A_g = 1144 \text{ in}^2$
- $I_g = 1.110.567 \text{ in}^2$
- $S_{TB} = 29.410 \text{ in}^3$
- $S_{BB} = 24.019 \text{ in}^3$
- $Y_{TB} = 37.8 \text{ in}$
- $Y_{BB} = 46.2 \text{ in}$
- $Wt. = 1192 \text{ lb/ft}$

**NOTES:**

1. BARS 301 AND 302 COMBINED TO FORM ONE STIRRUP.

2. *DENOTES EPOXY-COADED BARS

**BULB - TEE BEAM**

**TYPE BT 84 x 60**

Figure 406-14R

(Page 1 of 2)
* DENOTES EPOXY-COATED BARS

BULB - TEE BEAM
TYPE BT 84 x 60
BAR BENDING DETAILS

Figure 406-14R
(page 2 of 2)
BARS 301 AND 302 COMBINE TO FORM ONE STIRRUP.

1. *DENOTES EPOXY-COATED BARS.
2. LOCATE HOLDDOWNS 5'-0" EACH SIDE OF CENTER LINE OF BEAM.
3. BARS 301 AND 302 COMBINE TO FORM ONE STIRRUP.

NOTES:

WIDE BULB-TEE BEAM TYPE BT 36 x 61
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14S
(Page 1 of 2)
WIDE BULB-TEE BEAM TYPE BT 36 x 61
BAR BENDING DETAILS

Figure 406-14S
(Page 2 of 2)
WIDE BULB-TEE BEAM TYPE BT 42 x 61
SECTIONS SHOWING PRESTRESSING AND MILD REINFORCING STEEL

Figure 406-14T
(Page 1 of 2)
DENOTES EPOXY-COATED BARS

WIDE BULB-TEE BEAM TYPE BT 42 x 61
BAR BENDING DETAILS

Figure 406-14T
(Page 2 of 2)
WIDE BULB-TEE BEAM TYPE BT 48 x 61
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14U
(Page 1 of 2)
*402

END VIEW
*401

SIDE VIEW
*401

WIDE BULB-TEE BEAM TYPE BT 48 x 61
BAR BENDING DETAILS

Figure 406-14U
(Page 2 of 2)
BARS 301 AND 302 COMBINE TO FORM ONE STIRRUP.

3. LOCATE HOLDDOWNS 5'-0" EACH SIDE OF CENTER LINE OF BEAM.

2. *DENOTES EPOXY-COATED BARS.

1. NOTES:
   1. *DENOTES EPOXY-COATED BARS.
   2. LOCATE HOLDDOWNS 5'-0" EACH SIDE OF CENTER LINE OF BEAM.
   3. BARS 301 AND 302 COMBINE TO FORM ONE STIRRUP.

WIDE BULB-TEE BEAM TYPE BT 54 x 61
SECTIONS SHOWING Prestressing
AND MILD REINFORCING STEEL

Figure 406-14V
(Page 1 of 2)
WIDE BULB-TEE BEAM TYPE BT 54 x 61
BAR BENDING DETAILS

Figure 406-14V
(Page 2 of 2)
WIDE BULB-TEE BEAM TYPE BT 60 x 61
SECTIONS SHOWING PRESTRESSING AND MILD REINFORCING STEEL

Figure 406-14W
(Page 1 of 2)
WIDE BULB-TEE BEAM TYPE BT 60 x 61
BAR BENDING DETAILS

Figure 406-14W
(PAGE 2 OF 2)
WIDE BULB-TEE BEAM TYPE BT 66 x 61
SECTIONS SHOWING PRESTRESSING
AND MILD REINFORCING STEEL

Figure 406-14X
(Page 1 of 2)
WIDE BULB-TEE BEAM TYPE BT 66 x 61
BAR BENDING DETAILS

Figure 406-14X
(Page 2 of 2)
CUT BONDED STRANDS IN BOTTOM ROW WITH 1'-10" PROJECTION AND SHOP BEND AS SHOWN. DO NOT HEAT.

END BENT

INTERIOR BENT OR PIER

NOTES:
1. THE 302 AND 403 BARS REQUIRED IN SOME SECTIONS ARE NOT SHOWN.
2. * DENOTES EPOXY-COATED BARS

BULB - TEE BEAM (36 DEPTH)
ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-14Y
(Page 1 of 9)
NOTES:
1. THE 302 AND 403 BARS REQUIRED IN SOME SECTIONS ARE NOT SHOWN.
2. * DENOTES EPOXY-COATED BARS

END BENT

CUT BONDED STRANDS IN BOTTOM ROW WITH 1'-10" PROJECTION AND SHOP BEND AS SHOWN. DO NOT HEAT.

INTERIOR BENT

OR PIER

BULB - TEE BEAM (42 DEPTH)
ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-14Y
(Page 2 of 9)
NOTES:
1. THE 302 AND 403 BARS REQUIRED IN SOME SECTIONS ARE NOT SHOWN.
2. * DENOTES EPOXY-COATED BARS

BULB - TEE BEAM (48 DEPTH)
ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-14Y
(Page 3 of 9)
NOTES:
1. THE 302 AND 403 BARS REQUIRED IN SOME SECTIONS ARE NOT SHOWN.
2. * DENOTES EPOXY-COATED BARS

BULB - TEE BEAM (54 DEPTH)
ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-14Y
(Page 4 of 9)
END BENT

4 - #6 x 4'-9"
(2 e.f.)

CUT BONDED STRANDS IN BOTTOM ROW WITH 1'-10" PROJECTION AND SHOP BEND AS SHOWN. DO NOT HEAT.

301

*401

BULB - TEE BEAM (60 DEPTH)
ELEVATIONS SHOWING END REINFORCEMENT

NOTES:
1. THE 302 AND 403 BARS REQUIRED IN SOME SECTIONS ARE NOT SHOWN.
2. * DENOTES EPOXY-COATED BARS

INTERIOR BENT OR PIER

4 - #6 x 4'-9"
(2 e.f.)

4 1/2"

Figure 406-14Y
(Page 5 of 9)
NOTES:

1. THE 302 AND 403 BARS REQUIRED IN SOME SECTIONS ARE NOT SHOWN.

2. * DENOTES EPOXY-COATED BARS

BULB - TEE BEAM (66 DEPTH)
ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-14Y
(Page 6 of 9)
BULB - TEE BEAM (72 DEPTH)
ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-14Y
(Page 7 of 9)
BULB - TEE BEAM (78 DEPTH)
ELEVATIONS SHOWING END REINFORCEMENT

Figure 406-14Y
(Page 8 of 9)
NOTES:
1. THE 302 AND 403 BARS REQUIRED IN SOME SECTIONS ARE NOT SHOWN.
2. * DENOTES EPOXY-COATED BARS

**BULB - TEE BEAM (84 DEPTH)**

**ELEVATIONS SHOWING END REINFORCEMENT**

Figure 406-14Y

(Page 9 of 9)
BULB - TEE BEAM
SECTION AT END SHOWING DRAPED STRANDS

Figure 406-14Z
BOX BEAM
TYPE CB 12 x 36

Figure 406-15A
BEAM PROPERTIES

\[ A_B = 471 \text{ in}^2 \]
\[ I_B = 14,169 \text{ in}^4 \]
\[ S_{TB} = 1,653 \text{ in}^3 \]
\[ S_{BB} = 1,681 \text{ in}^3 \]
\[ Y_{TB} = 8.57 \text{ in} \]
\[ Y_{BB} = 8.43 \text{ in} \]
\[ Wt. = 491 \text{ lbs/ft} \]

NOTES:

1. *DENOTES EPOXY-COADED BAR

BOX BEAM
TYPE CB 17 x 36

Figure 406-15B
BEAM PROPERTIES

\[ A_B = 515 \text{ in}^2 \]
\[ I_B = 27,619 \text{ in}^4 \]
\[ S_{TB} = 2,417 \text{ in}^3 \]
\[ S_{BB} = 2,463 \text{ in}^3 \]
\[ Y_{TB} = 10.60 \text{ in} \]
\[ Y_{BB} = 10.40 \text{ in} \]
\[ Wt. = 536 \text{ lbs/ft} \]

NOTE:
1. *DENOTES EPOXY-COATED BAR

NOTES:

402 x 3'-8"

5 1/2 "

2'-8"

3/8 "

3/4 "

4"

1 1/4 " cl.

1 1/4" Chamfer

3/4" Chamfer

1/2" Ø Strand

402

6 - #5

2 @ 2" = 4"

5 1/2"

1" Chamfer

14 @ 2" = 2'-4"

3'-0"

Figure 406-15C

BOX BEAM
TYPE CB 21 x 36

Figure 406-15C
**BEAM PROPERTIES**

\[
\begin{align*}
A_B &= 581 \text{ in}^2 \\
I_B &= 50,627 \text{ in}^4 \\
S_{TB} &= 3,712 \text{ in}^3 \\
S_{BB} &= 3,789 \text{ in}^3 \\
Y_{TB} &= 13.64 \text{ in} \\
Y_{BB} &= 13.36 \text{ in} \\
Wt. &= 605 \text{ lbs/ft}
\end{align*}
\]

**NOTES:**

1. *DENOTES EPOXY-COATED BAR

**BOX BEAM**

**TYPE CB 27 x 36**

**Figure 406-15D**
**BEAM PROPERTIES**

- $A_B = 647$ in$^2$
- $I_B = 86,092$ in$^4$
- $S_{TB} = 5,168$ in$^3$
- $S_{BB} = 5,269$ in$^3$
- $Y_T = 16.66$ in
- $Y_B = 16.34$ in
- Wt. = 674 lbs/ft

**NOTES:**
1. *DENOTES EPOXY-COATED BAR

**BOX BEAM**

**TYPE CB 33 x 36**

Figure 406-15E
FIGURE 406-15F

BOX BEAM
TYPE CB 42 x 36

BEAM PROPERTIES

\[
\begin{align*}
A_B &= 746 \text{ in}^2 \\
I_B &= 161,496 \text{ in}^4 \\
S_{TB} &= 7,618 \text{ in}^3 \\
S_{BB} &= 7,764 \text{ in}^3 \\
Y_{TB} &= 21.20 \text{ in} \\
Y_{BB} &= 20.80 \text{ in} \\
Wt. &= 777 \text{ lbs/ft}
\end{align*}
\]

NOTES:
1. *DENOTES EPOXY-COATED BAR
NOTES:
1. ■■■*DENOTES EPOXY-COATED BAR

BOX BEAM
TYPE CB 12 x 48

Figure 406-15G
BOX BEAM
TYPE CB 17 x 48

Figure 406-15H
BEAM PROPERTIES

- $A_B = 647 \text{ in}^2$
- $I_B = 33,881 \text{ in}^4$
- $S_{TB} = 3,202 \text{ in}^3$
- $S_{BB} = 3,252 \text{ in}^3$
- $Y_{TB} = 10.58 \text{ in}$
- $Y_{BB} = 10.42 \text{ in}$
- Wt. = 674 lbs/ft

NOTES:

1. *DENOTES EPOXY-COATED BAR

BOX BEAM
TYPE CB 21 x 48

Figure 406-15 I
**BEAM PROPERTIES**

- $A_B = 713 \text{ in}^2$
- $I_B = 66,216 \text{ in}^4$
- $S_{TB} = 4,865 \text{ in}^3$
- $S_{BB} = 4,945 \text{ in}^3$
- $Y_{TB} = 13.61 \text{ in}$
- $Y_{BB} = 13.39 \text{ in}$
- Wt. = 743 lb/ft

**NOTES:**

1. *DENOTES EPOXY-COATED BAR

---

**BOX BEAM**

**TYPE CB 27 x 48**

Figure 406-15J
**BEAM PROPERTIES**

- $A_B = 779 \text{ in}^2$
- $I_B = 111,384 \text{ in}^4$
- $S_{TB} = 6,694 \text{ in}^3$
- $S_{BB} = 8,808 \text{ in}^3$
- $Y_{TB} = 16.64 \text{ in}$
- $Y_{BB} = 16.36 \text{ in}$
- Wt. = 811 lbs/ft

**NOTES:**

1. □**DENOTES EPOXY-COATED BAR**

---

**BOX BEAM**  
**TYPE CB 33 x 48**  

**Figure 406-15K**
**BEAM PROPERTIES**

- $A_B = 878 \text{ in}^2$
- $I_B = 205,798 \text{ in}^4$
- $S_{TB} = 9,721 \text{ in}^3$
- $S_{BB} = 9,880 \text{ in}^3$
- $Y_{TB} = 21.17 \text{ in}$
- $Y_{BB} = 20.83 \text{ in}$
- Wt. = 915 lbs/ft

**NOTES:**

1. *DENOTES EPOXY-COATED BAR*

---

**BOX BEAM**

**TYPE CB 42 x 48**

Figure 406-15L
BOX BEAM
TYPE WS 12 x 48

Figure 406-15M
BEAM PROPERTIES

A_B = 564 in^2
I_B = 18,467 in^4
S_{TB} = 2,212 in^3
S_{BB} = 2,135 in^3
Y_{TB} = 8.35 in
Y_{BB} = 8.65 in
Wt. = 588 lbs/ft

BOX BEAM
TYPE WS 17 x 48

Figure 406-15N
BOX BEAM
TYPE WS 21 x 48

Figure 406-15 O
BEAM PROPERTIES

$A_B = 713 \text{ in}^2$

$I_B = 66,216 \text{ in}^4$

$S_{TB} = 4,865 \text{ in}^3$

$S_{BB} = 4,945 \text{ in}^3$

$Y_{TB} = 13.61 \text{ in}$

$Y_{BB} = 13.39 \text{ in}$

Wt. = 743 lbs/ft

---

BOX BEAM

TYPE WS 27 x 48

Figure 406-15P
BOX BEAM
TYPE WS 33 x 48

Figure 406-15Q
**BEAM PROPERTIES**

- $A_B = 878 \text{ in}^2$
- $I_B = 205,798 \text{ in}^4$
- $S_{TB} = 9,721 \text{ in}^3$
- $S_{BB} = 9,880 \text{ in}^3$
- $Y_{TB} = 21.17 \text{ in}$
- $Y_{BB} = 20.83 \text{ in}$
- Wt. = 915 lbs/ft

---

**Figure 406-15R**

**BOX BEAM**

**TYPE WS 42 x 48**
I-BEAM PIER DIAPHRAGM
SECTION BETWEEN BEAMS

Figure 406-16A
I-BEAM PIER DIAPHRAGM
SECTION AT BEAMS

Figure 406-16B
I-BEAM
INTERMEDIATE DIAPHRAGM

Figure 406-16C
INTERMEDIATE

I-BEAM
DIAPHRAGMS

Figure 406-16D
A = (1'-4") sec $\Theta$ + 10" + 0.5 (L + W tan $\Theta$)  
E = B + D  
B = A cos $\Theta$  
ACTUAL D = E - B  
C = 0.5 (L + W tan $\Theta$) + sec $\Theta$  
F = B - (0.5E)  
D = C cos $\Theta$  
G = F sec $\Theta$

* USE FOR SIZING CAP ONLY. ROUND UP TO AN INCREMENT OF 3"

© CHECK SEISMIC MINIMUM SUPPORT LENGTH FOR EXPANSION BENT.

© THIS DIMENSION SHOULD BE INCREASED FOR LARGE EXPANSION LENGTH.

I-BEAM: END BENT CAP SIZING AND BEARING LAYOUT DETAILS

Figure 406-16E
\[ A = 9'' \cos \Theta \]
\[ B = 0.5 \left( L + W \tan \Theta \right) + 4'' \sec \]
\[ C = B \cos \Theta \]

CAP WIDTH = 2AC

ACTUAL C = 1/2 CAP WIDTH - A

* USE FOR SIZING CAP ONLY. ROUND UP TO AN INCREMENT OF 3"

** USE 6" FOR PIER BELOW EXPANSION JOINT.

I BEAM: PIER CAP SIZING AND BEARING LAYOUT DETAILS

Figure 406-16F
A = a + b
B = a + c
C = a - c
D = a - b

NOTES:
1. IF D < 2" USE LARGER DIAPHRAGM.
2. A MINUS B AND C MINUS D SHOULD BE MADE EQUAL

* THIS DIMENSION WILL INCREASE OR DECREASE SLIGHTLY IF ENDS OF BEAMS ARE NOT VERTICAL. SEE SECTION 63-12.0 FOR ADDITIONAL INFORMATION.

I-BEAM
HOLES AT PIER DIAPHRAGM

Figure 406-16G
BULB-TEE PIER DIAPHRAGM
SECTION BETWEEN BEAMS

Figure 406-16H
BULB-TEE PIER DIAPHRAGM
SECTION AT BEAMS

Figure 406-16 I
BULB-TEE
INTERMEDIATE DIAPHRAGM

Figure 406-16J
BULB-TEE

DIAPHRAGM

Figure 406-16K
BULB-TEE: END BENT CAP SIZING AND BEARING LAYOUT DETAILS

Figure 406-16L

A = (1'-4") sec\( \Theta \) + 9" + 0.5 (L + W tan\( \Theta \))
B = A cos\( \Theta \)
C = 0.5 (L + W tan\( \Theta \)) + sec\( \Theta \)
D = C cos\( \Theta \)
E = B + D
F = B - (0.5E)
G = F sec\( \Theta \)

* USE FOR SIZING CAP ONLY. ROUND UP TO AN INCREMENT OF 3"
\( \Theta \) CHECK SEISMIC MINIMUM SUPPORT LENGTH FOR EXPANSION BENT.
\( \Theta \) THIS DIMENSION SHOULD BE INCREASED FOR LARGE EXPANSION LENGTH.
A = \cos \Theta

B = 0.5 (L + W \tan \Theta) + 4 \sec \Theta

C = B \cos \Theta

CAP WIDTH = 2(A+C)

ACTUAL C = 1/2 CAP WIDTH - A

* USE FOR SIZING CAP ONLY. ROUND UP TO AN INCREMENT OF 3''

© USE 6'' FOR PIER BELOW EXPANSION JOINT

BULB TEE: PIER CAP SIZING AND BEARING LAYOUT DETAILS

Figure 406-16M
\[
a = (1\text{-}0\ 1/2\text{"}) (\tan \Theta) \\
b = (3\ 3/4\text{") (tan \Theta)} \\
c = (1\text{-}6\text{") sec \Theta)} \\
d = (5\text{") (sec \Theta)} \\
e = (1\text{-}1\text{") sec \Theta - 3\text{")}
\]

**A** = \( E + a \)  
**B** = \( E + b \)  
**C** = \( E - b \)  
**D** = \( E - a \)

**NOTE:**  
If D or d is < 2" use larger diaphragm.

* THIS DIMENSION WILL INCREASE OR DECREASE SLIGHTLY IF ENDS OF BEAMS ARE NOT VERTICAL. SEE SECTION 63-12.0 FOR ADDITIONAL INFORMATION.

**BULB-TEE**  
HOLES AT PIER DIAPHRAGM

Figure 406-16N
EXPANDED POLYSTYRENE (SAME HEIGHT AS BEARING PADS)

EXPANDED POLYSTYRENE (1/2" BOTTOM, 1" EACH SIDE AND 1" EACH END OF KEYWAY) (BONDED TO THE CONCRETE)

5" (Typ.)
2" Cl. (Typ.)

3" x 1'-0" KEYWAY BETWEEN BEAMS

#6 THREADED BAR FROM BEAM @1'-0" MAX.

#5 BTWN. BEAMS

#4 @ 1'-0" MAX.

*THIS IS A MINIMUM DIMENSION.

BOX BEAM PIER DIAPHRAGM FOR SPREAD BEAMS
SECTION BETWEEN BEAMS

Figure 406-16 O
THIS IS A MINIMUM DIMENSION.

BOX BEAM PIER DIAPHRAGM FOR SPREAD BEAMS
SECTION AT BEAMS

Figure 406-16P
BOX BEAM DIAPHRAGM
AT PIER

Figure 406-16Q
BEARING PAD
ELASTOMERIC
BEARING PADS)
(SAME HEIGHT AS
EXPANDED POLYSTYRENE
#8
(BONDED TO THE CONCRETE)
AND 1" EACH END OF KEYWAY)
(1/2" BOTTOM, 1" EACH SIDE
AND 1" EACH END OF KEYWAY)
(BONDED TO THE CONCRETE)

BOX BEAM
CLOSURE POUR AT PIER
FOR ADJACENT BEAMS

Figure 406-16R
CHECK SEISMIC MINIMUM SUPPORT LENGTH FOR EXPANSION BENT.

USE FOR SIZING CAP ONLY. ROUND UP TO AN INCREMENT OF 3".

CHECK SEISMIC MINIMUM SUPPORT LENGTH FOR EXPANSION BENT.

A = (1'-4") sec Θ + 6"
B = A cos Θ
C = 0.5 L + 6"
D = B + C

ACTUAL C = D - B
E = B - 0.5 D
F = E sec Θ

USE FOR SIZING CAP ONLY. ROUND UP TO AN INCREMENT OF 3"

Box Beam: End Bent Cap Sizing and Bearing Layout Details

Figure 406-16S
BEARING PAD
ELASTOMERIC
USE FOR SIZING CAP ONLY.
ROUND UP TO AN INCREMENT OF 3"
CAP WIDTH = 2AB
ACTUAL B = 1/2 CAP WIDTH - A

A = 9" cos Θ
B = 0.5L + 4"
CAP WIDTH = 2AB
ACTUAL B = 1/2 CAP WIDTH - A

* USE FOR SIZING CAP ONLY. ROUND UP TO AN INCREMENT OF 3"
Θ USE 0.5 ft. FOR PIER BELOW EXPANSION JOINT.

BOX BEAM: PIER CAP SIZING AND BEARING LAYOUT DETAILS FOR SPREAD BEAMS

Figure 406-16T
USE FOR SIZING CAP ONLY. ROUND UP TO AN INCREMENT OF 3"

USE 0.5 ft. FOR PIER BELOW EXPANSION JOINT

A = 9" cos θ
B = 0.5 L + 4"
CAP WIDTH = 2AB
ACTUAL B = 1/2 CAP WIDTH - A

BOX BEAM: PIER CAP SIZING AND BEARING LAYOUT

Figure 406-16U
A = (7" / \cos \Theta) - 3" = __________

BOX BEAM
INSERTS AT PIER DIAPHRAGM

Figure 406-16V
1 1/4" CLR. OFF HEADBOARD
PLACE PARALLEL TO BEAM END.
2-404 PER BEAM END,
1'-6"

#5
1 1/2 " Cl.
MINUS 5"
BEAM DEPTH

TYPICAL END REINFORCEMENT SECTION

404 BAR

3'-0"

2'-5"

403 BAR

MILD REINFORCEMENT FOR 36 WIDTH
SKEWED-BEAM END (45-deg Skew Shown)

Figure 406-16W
MILD REINFORCEMENT FOR 48 WIDTH
SKEWED-BEAM END (45-deg Skew Shown)

Figure 406-16X
CHAPTER 407

Steel Structure

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CHAPTER 407

STEEL STRUCTURE

This chapter addresses structural-steel requirements in the LRFD Bridge Design Specifications, hereafter referred to as LRFD, which may require amplification, clarification, or an improved application. This chapter is intended to provide general guidance in LRFD design and detailing practices. The chapter is structured as follows:

1. Section 407-1.0 provides general information for which there is not a direct reference in LRFD Section 6.

2. Sections 407-2.0 through 407-9.0 provide information which augments and clarifies LRFD Section 6. To assist in using these Sections, references to LRFD are provided where applicable.

The discussion in this chapter is restricted to a multi-beam or multi-girder steel superstructure. Unless stated otherwise, the terms beam and girder are used interchangeably. This reflects the popularity of these systems because of their straightforward design, ease of construction, and the potential for aesthetic appearance.

407-1.0 GENERAL

407-1.01 Economy

Factors that influence the cost of a steel-girder bridge include, but are not limited to, the number of girders, the type of material, type and number of substructure units, amount of material, fabrication, transportation, and erection. The cost of these changes periodically, in addition to the cost relationship among them. Therefore, the guidelines used to determine the most economical type of steel girder on one bridge must be reviewed and modified as necessary for another bridge.

Based upon market factors, the availability of steel can be an issue in satisfying the construction schedule. It is the responsibility of the bridge designer to verify the availability of the specified steel. The bridge designer should contact structural-steel producers to ensure the availability of rolled beams and plates.
A steel plate girder should be designed to optimize weight savings in correlation with fabrication and erection costs. The top flanges of a compositely-designed plate girder are typically smaller than their bottom flanges. The flange section is varied along the length of the bridge following the moment envelope to save cost by offsetting the increased fabrication costs of welded flange transitions with larger savings in material costs. To save in total costs, minimum web thicknesses are increased to avoid the use of stiffeners.

The load-carrying capacity of an exterior beam or girder shall not be less than that of the interior beams or girders as described in LRFD 4.6.2.2.1.

Weathering steel, unpainted Grades 50W and HPS70W should be used if possible to lower the initial construction costs and future maintenance costs. Aesthetic considerations limit the application of weathering steel in a high-visibility application, because the inherent staining of the substructures may not be desirable. See Section 407-2.01(01) for other factors limiting the use of weathering steel.

407-1.01(01) Rolled Beams vs. Welded Plate Girders

If rolled beams are specified, the selected sections shall be ensured to be available. Welded plate girders should be specified instead of rolled beams for the conditions as follows:

1. the bridge has a radius of less than 1200 ft. due to fabrication limitations;
2. the span lengths exceed the span capacity of rolled sections, or
3. the camber is too large to be accommodated in the natural camber of the beam.

407-1.01(02) Number of Beam or Girder Lines

The lowest number of beam lines in the cross section, as compatible with deck design requirements, provides the most economical bridge in the absence of girder-depth restrictions. In considering the economy of the bridge and number of beam or girder lines, the evaluations to be made are as follows:

1. the available depth of superstructure governed by vertical clearance requirements;
2. the increase or decrease in approach roadway costs;
3. the increase or decrease in deck thickness and reinforcement requirements; and
4. the cost of stiffeners versus increased web thickness.
An INDOT-route bridge requires a minimum of four steel-beams or plate-girders lines, or four web lines for steel box girders. Future maintenance and rehabilitation shall be considered in determining the number of beams or girders in a cross section.

407-1.01(03) Spacing

The beams or girders should be spaced uniformly across the width of the bridge. The maximum spacing of rolled beams should not exceed 11 ft. The maximum spacing of plate girders should not exceed 12.5 ft. The location of the exterior beam or girder is controlled by the minimum and maximum overhang widths that are specified in Section 404-3.02 and the space required for deck drains if required.

407-1.02 Plate-Girder Design Considerations

In addition to the information shown in LRFD, the following applies to the design of structural-steel plate girders.

407-1.02(01) General

Plate girders shall be made composite with the bridge deck and should be continuous over interior supports where applicable.

To achieve economy in the fabrication shop, all girders in a multi-girder bridge should be identical.

407-1.02(02) Haunched Girders

Where practical, constant-depth girders, or girders with constant web depths, shall be used. Haunched girders are generally uneconomical for spans of less than 300 ft. They may be used where aesthetics or other special circumstances are involved, but constant-depth girders will generally be more cost effective.

407-1.02(03) Longitudinally-Stiffened Web

A longitudinally-stiffened web shall not be used. In addition to being considered uneconomical, the ends of longitudinal stiffeners are fatigue sensitive if subject to applied tensile stresses. Therefore, where used, they must be ended in zones of little or no applied tensile stresses.
407-1.02(04) Flange-Plate Size and Transitions

The minimum flange-plate size for a plate girder is 12 in. by ¾ in. For curved girders, the minimum flange thickness is 1 in. As wide a flange plate as practical should be used, consistent with stress and flange width/thickness ratio, $b/t$, requirements. The wide flange contributes to girder stability and reduces the number of passes and weld volume at flange butt welds. The flange width should be in increments of 2 in. Typically, the maximum flange thickness is 3 in. Figure 407-1A provides commonly-specified plate thicknesses.

Within a single field section, i.e., an individual shipping piece, the width of each flange should be of constant width. Typically, only flange thicknesses, not widths, are varied within a field section. A design of multiple identical girders with constant-width flanges minimizes fabrication costs.

Flanges shall be proportioned so that the fabricator can economically cut them from structural plate steel between 60-in., preferably 72-in., and 96-in widths. Flanges should be grouped to provide an efficient use of the plates. Because structural-steel plate is most economically purchased in these widths, it is advantageous to repeat plate thicknesses as much as practical. Many of the plates of like width can be grouped by thickness to satisfy the minimum-width purchasing requirement, but this economical purchasing strategy may not be possible for thicker, less-used plates.

The most efficient method to fabricate flanges is to groove-weld together several wide plates of varying thicknesses received from the mill. After welding and non-destructive testing, the individual flanges are stripped from the full plate. This method of fabrication reduces the number of welds, individual runoff tabs for both start and stop welds, the amount of material waste, and the number of X-rays for non-destructive testing. The obvious objective, therefore, is for flange widths to remain constant within an individual shipping length by varying material thickness as required. Figure 407-1B illustrates the efficient fabrication of girders.

Constant flange width within a field section may not always be practical in a span of over 300 ft where a flange-width transition may be required in the negative bending region. Though not preferred, if a width transition must be provided, the butt splice shall be shifted a minimum of 3 in. from the transition into the narrower flange plate. This 3-in. shift makes it simpler to fit runoff tabs, weld, and test the splice and then grind off the run-off tabs.
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.
<table>
<thead>
<tr>
<th>WEB DEPTH, $d$</th>
<th>STIFFENER USAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 4'-4&quot;</td>
<td>None</td>
</tr>
<tr>
<td>4'-4&quot; $\leq d \leq$ 5'-6&quot;</td>
<td>Partial</td>
</tr>
<tr>
<td>&gt; 5'-6&quot;</td>
<td>Full</td>
</tr>
</tbody>
</table>

An unstiffened web is the thinnest web allowed by *LRFD* without transverse stiffeners. A partially-stiffened web is approximately 1/16 in. thinner than an unstiffened web. A fully-stiffened web is the thinnest web allowed by *LRFD* in combination with the maximum number of transverse stiffeners. A minimum web thickness of 1/2 in. shall be used.

For webs of at least 72 in. depth, safety handrails shall be considered for future inspection. See Figure 407-1D for details.

### 407-1.03 Continuous Structure

Span-by-span continuity enhances both the strength and rigidity of the structure. However, the most significant benefit of structural continuity is the reduction in the number of deck expansion joints. Open or leaking deck joints can cause damage to beam ends, diaphragms, bearings, bent caps, or pier caps. See Section 404-2.06 for more discussion on bridge-deck expansion joints.

### 407-1.04 Composite Action

Composite action enhances both the strength and rigidity of the beam. Composite action is mandatory in the positive-moment region. Composite action is preferred in the negative-moment regions.

Deck concrete should be considered effective in the negative-moment region for determining live-load deflection at the Service Limit state. For design, concrete in tension is ignored in checking the Strength Limit state. The deck reinforcing steel can be considered to act with the steel section if shear connectors are provided. The composite-section slab depth shall be reduced by 0.5 in. due to the wearing surface.
**407-1.05 Horizontally-Curved Steel Girder**

**407-1.05(01) General**

*LRFD* 6.10 includes horizontally curved girders as a part of the requirements for the resistance of I-shaped girders. Analysis methodologies that describe the required levels of analysis are also specified, including the following.

1. The span, radius, and skew of the girder determine whether the curvature must be considered in the analysis.

2. Curved steel girders are always considered noncompact in the positive-moment region. Therefore, the maximum nominal bending stress is $F_y$.

3. *LRFD* Appendix A or B shall not be used.

4. Lateral flange bending stresses due to torsion must be considered. As a result, a curved steel plate-girder bridge usually has wider flanges than a straight steel bridge.

5. Horizontal curvature causes a variable load distribution that increases from inside to outside of the curve. Theoretically, flange and web sizes can be different for each girder. The economic benefits associated with grouping plate sizes shall be considered. The girders shall be grouped, using identical flange sizes for the exterior and first interior girder, the second and third interior girder, etc. Incrementally increasing the web depth from inside to outside of curve shall be considered. This practice can cause the outside girders to become too stiff, drawing too much moment to the outside exterior girder.

**407-1.05(02) Details**

Cross frames and diaphragms shall be considered primary members. A curved steel simple-span or continuous-span bridge shall have its diaphragms directed radially, except for end diaphragms, which should be placed parallel to the centerline of bearings.

Diaphragms, including their connections to the girders, shall be designed to carry the total load to be transferred at each diaphragm location. Cross frames and diaphragms should be as close as practical to the full depth of the girders.

A bridge expands and contracts in all directions. For a bridge that is long in relationship to its width, the transverse expansion is ignored. For an ordinary geometric configuration where the bridge length is long relative to the bridge width and the curvature is moderate, the unique
expansion characteristics of a curved structure need not be considered. In an urban area, a wide sharply-curved structure may be required. For this situation, multi-rotational bearings shall be considered, and restraint shall selectively be provided either radially or tangentially to accommodate the thermal movement of the structure as the bridge tries to expand in all directions. See Figure 407-1E for additional geometric information.

Flange splices shall be designed to carry flange bending or lateral bending stresses and vertical bending stresses in the flanges.

**407-1.06 Integral or Semi-Integral End Bent**

Section 409-2.01 discusses the design of an integral or semi-integral end bent. The following applies to the use of integral or semi-integral end bents in combination with a steel superstructure.

1. **Bridge Length.** Integral end bents may be used with a structural-steel bridge where the superstructure expansion length from the superstructure point of no movement to the integral end bent does not exceed the value shown in Figure 409-2A.

2. **Deck Pour.** An interior diaphragm shall be placed within 10 ft of the end support to provide beam or girder stability during the deck pour.

3. **Anchorage.** Each steel beam or girder should have stiffener plates of 1/2-in. thickness welded to both sides of the web and the flanges over the supports to anchor the beam into the concrete cap. A minimum of three holes should be provided through the web to allow #6 bars to be inserted to further anchor the beam to the cap.

**407-1.07 Fracture-Critical Member**

A fracture-critical member is a steel component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function.

The member or member components which are Fracture Critical Members (FCM) shall be determined. Such members or member components should be identified on the plans. The INDOT Standard Specifications address non-fracture-critical members. If a structure includes fracture-critical members or steel main members with yield strength of greater than 50 ksi, a unique special provision must be prepared to specify the Charpy V-notch requirements. All member or member components designated as FCM shall be in accordance the ANSI/AASHTO/AWS D1.5 Bridge Welding Code, Section 12.
Indiana is in temperature-zone designation 2, minimum service temperature 0 °F to -30 °F, with respect to Charpy V-notch impact requirements.

### 407-1.08 Other Design Considerations

At the stress-limit state, a beam should be designed for the sum of the steel and concrete-slab dead loads acting on the beam alone, plus the superimposed dead load and live load acting on the composite section. Shrinkage shall be considered only for a long span or other unusual configurations. At the Strength Limit state for a compact section, large-scale inelastic activity is presumed to rearrange stress distributions in a section such that the history of stress build-up need not be considered. In a non-compact section where the factored flexural resistance is limited to the yield stress, the history of stress build-up must be considered.

*LRFD* Appendix D6 provides formulas for computing the plastic moment for both the positive- and negative-moment section. It also explains procedures for determining the yield moment of a composite section. *LRFD* Appendix C6 provides a step-by-step approach for the design of a steel-bridge superstructure.

### 407-2.0 MATERIALS

#### 407-2.01 Structural Steels

**407-2.01(01) Selection**

The most common steel-grade choice is unpainted ASTM A709 Grade 50W weathering steel. Its initial cost advantage compared to painted high-strength steel, e.g., A 709 Grade 50, can range up to 15%. If compared to painted ASTM A709 Grade 36 steel, the cost advantage is approximately 20%. If future repainting costs are considered, the cost advantage is more substantial. This reflects, for example, environmental considerations in the removal of paint, which can make the use of painted steel prohibitive. Grade 36 steel is becoming less used and thus less available. The higher-strength ASTM A 709 Grade HPS 70W often carries a cost premium of approximately 10% compared to Grade 50W. *AASHTO Guide Specification for Highway Bridge Fabrication with HPS 70W Steel* should be used as a reference. A new high-performance steel with a minimum specified yield strength of 100 ksi has been introduced. It has yet to be proven cost-effective for girder bridge applications.

Despite its cost advantage, the use of weathering steel is not appropriate in every environment or at every location. The application of weathering steel and its potential problems are discussed in
the FHWA *Technical Advisory: Uncoated Weathering Steel in Structures*, October 3, 1989. Also, the proceedings of the Weathering Steel Forum, July 1989, are available from the FHWA Office of Implementation, HRT-10. Weathering steel should not be used where the following adverse conditions exist.

1. **Environment.** Weathering steel should not be used in an industrial area where concentrated chemical fumes can drift onto the structure. Its suitability should be determined by a corrosion consultant.

2. **Location.** Weathering steel should not be used at a grade separation in a tunnel, which is produced due to a depressed roadway section with narrow shoulders between vertical retaining walls, with a shallow vertical clearance, with deep abutments adjacent to the shoulders, or with a wide bridge. This tunnel effect prevents roadway spray from being dissipated and spread by air currents. There is no evidence of salt-spray corrosion where the longitudinal extent of the vertical walls is limited to the abutment itself and roadway spray can be dissipated on both approaches.

3. **Low-Level Water Crossing.** Sufficient clearance over a body of water should be maintained so that water-vapor condensation does not result in prolonged periods of wetness on the steel. For weathering steel, clearance to the bottom flange should be at least 10 ft over sheltered, stagnant water and at least 8 ft above the average low-water level for a running stream.

Where unpainted weathering steel is inappropriate, and a concrete-members alternative is not feasible, the most economical painted steel is ASTM A709 Grade 50 steel in both webs and flanges.

FHWA *Technical Advisory: Uncoated Weathering Steel in Structures* is a source of information, but its recommendation for partial painting of the steel in the vicinity of deck joints should not be considered the first choice. The best solution is to eliminate the deck joints. In a shorter bridge, the end joint is replaced with an integral or semi-integral end bent (see Chapter 409).

**407-2.01(02) Hybrid Girder**

The use of HPS 70W steel in the top and bottom flanges of the negative flexure regions and the bottom flange of the positive moment regions, with Grade 50 steel in the top flange of the positive moment regions and in the web throughout is generally considered to be the most economical design.
407-2.01(03) Details for Unpainted Weathering Steel

For integral or semi-integral end bents, the girder shall be painted for a distance of 1 ft from the front face of the each concrete diaphragm as shown in Figure 407-2A detail 1. If a joint is used, all superstructure steel shall be painted within 10 ft of the joint or within 1.5 times the web depth, whichever is greater as shown in Figure 407-2A detail 2. For an end bent surrounded by MSE walls, all superstructure steel shall be painted for a distance of 10 ft beyond the MSE wall as shown in Figure 407-2A detail 3. For interior bents or piers supporting continuous spans, all superstructure steel shall be painted for a distance of 10 ft beyond each side of the centerline of bent or pier as shown in Figure 407-2A detail 4.

The following drainage treatments should be considered to avoid premature deterioration.

1. A drip bead should be provided at the end of each deck overhang.

2. The number of bridge-deck drains should be minimized, the drainage pipes should be generous in size, and they should extend below the steel soffit as specified in Chapter 203.

3. Water or debris traps shall be eliminated. Overlapping surfaces exposed to water shall be sealed or painted. This applies to non-slip-critical bolted joints. Slip-critical bolted joints or splices should not produce rust-pack where the bolts are spaced according to LRFD, and, therefore, do not require protection.

4. A drip bar shall be placed transversely across the top of the bottom flange and across the bottom of the bottom flange downslope of the top plate in front of the substructure elements to prevent water from running off the flange onto the concrete; see Figure 407-2B.

5. Piers and abutments shall be wrapped with polyethylene sheeting during construction to minimize staining while the steel is exposed to rainfall.

6. If an expansion joint is used, the superstructure steel shall be painted within 10 ft of the joint.

7. Cold joints shall be located away from the beams or girders.

407-2.02 Bolts, Nuts, and Washers

High-strength bolts should be specified as follows.

1. Weathering Steel: 7/8 in. dia., A 325 Type 3. Open holes of 15/16 in. dia.
2. **Painted Steel**: 7/8 in. dia., A 325 Type 1. Open holes of 15/16 in. dia.

For a large structure, A 325 or A 490 bolts may be used of 1 in. diameter, with open holes of 1 1/16 in. diameter. A 490 bolt cannot be plated, galvanized, or mechanical zinc coated.

**407-2.03 Stud Shear Connectors**

Stud shear connectors shall be designed in accordance with *LRFD 6.4.4*.

**407-2.04 Weld Metal**

Weld metal shall be designed in accordance with *LRFD 6.4.5*.

**407-2.05 Other Elements**

If painted, ASTM A 709 Grade 50 steel is to be used in the web and flanges, all steel for stiffeners, secondary members, connections, and diaphragms should be ASTM A 709 Grade 36, unless a higher strength is justified due to significant forces in these members. Grade 36 steel shall not be used for these secondary members if unpainted weathering steel is used in the webs and flanges. Steel for all splices should be the same material as used in the web and flanges of the controlling section in a built-up girder.

For a steel bridge to be painted, the color shall be shown on the plans. See the INDOT *Standard Specifications* for a table of colors. For unpainted weathering steel, the color shall be as specified in Section 407-2.01(03).
407-3.0 LOADS AND LIMIT STATES

407-3.01 Limit States

See Section 403-1.02 for a discussion on limit states.

407-3.02 Distribution of Dead Load

See Section 403-2.0 for a discussion on the distribution of dead load.

407-3.03 Live Load Deflection [Rev. Apr. 2017]

Live load deflection consideration is as described in LRFD 2.5.2.6.2. For vehicular bridges, live load plus dynamic load allowance deflection is limited to 1/800 of the span length for the design of a steel beam or plate girder structure of simple or continuous spans. For a structure in an urban area used by pedestrians or bicyclists, vehicular and pedestrian live load deflection should be limited to the 1/1000 of the span length. The span length used to determine deflection should be assumed to be the distance between centers of bearings or other points of support.

Live load deflection should be evaluated in accordance LRFD 2.5.2.6.2. The deflection of the girders should be based on the stiffness of the short-term composite section, assuming the entire concrete deck to be fully effective over the entire span length. In effect, the distribution of live loads is the number of loaded lanes divided by the number of girders. The concrete deck should be considered to act compositely with the girder, though sections of the girder may not be designed as composite. If multiple lanes are loaded, multiple presence factors shall be applied as shown in LRFD Table 3.6.1.1.2-1.

For horizontally-curved girders, uniform participation of the girders should not be assumed. Instead, the live load should be placed to produce the maximum deflection in each girder individually in the span under consideration.
407-4.0  FATIGUE CONSIDERATIONS

407-4.01  Load-Induced Fatigue

LRFD 6.6.1.2 provides the framework of analysis to evaluate load-induced fatigue. This Section provides additional information on the implementation of LRFD 6.6.1.2 and defines interpretation of its requirements.

Load-induced fatigue is determined by the following:

1. the stress range induced by the specified fatigue loading at the point under consideration;

2. the number of repetitions of fatigue loading a steel component will experience during its 75-year design life. This is determined by using anticipated truck volumes; and

3. the nominal fatigue resistance for the Detail Category being investigated.

The use of Fatigue I or Fatigue II load combinations shall be in accordance with LRFD 6.6.1.2.3. Fatigue I load combination is utilized for a higher traffic volume and provides for infinite life. Fatigue II load combination is used in designing for a finite life. Components of fracture-critical members should be designed for an infinite life using Fatigue I load combination.

407-4.01(01)  Fatigue Stress Range

1. **Range.** The fatigue stress range is the difference between the maximum and minimum stresses at a point subject to a net tensile stress, as described in LRFD 3.6.1.4. The stress range is caused by a single design truck that can be placed on the deck within the boundaries of a design lane. If a refined-analysis method is used, the design truck shall be positioned to maximize the stress at the point under consideration. The design truck should have a constant 30-ft spacing between the 32-kip axles. The dynamic load allowance is 0.15, and the fatigue load factor is 0.75.

2. **Regions.** Fatigue should be considered only in those regions of a steel member that experience a net applied tensile stress, or where the compressive stress of the unfactored permanent load is less than twice the maximum fatigue tensile stress.

3. **Analysis.** Unless a refined analysis method is used, the single-design-lane load distribution factor in LRFD 4.6.2.2 should be used to determine fatigue stresses. These tabularized distribution-factor equations incorporate a multiple presence factor of 1.2 that should be removed by dividing either the distribution factor or the resulting fatigue stresses by 1.2.
This division does not apply to distribution factors determined using the lever rule.

For a flexural member with shear connectors provided throughout its entire length, and with slab reinforcement satisfying *LRFD* 6.10.1.7, the live-load stress range may be computed using the short-term composite section assuming the concrete slab to be fully effective for both positive and negative flexure.

407-4.01(02) Fatigue Resistance

*LRFD* 6.6.1.2.5 groups the fatigue resistance of structural details into categories A through E′, which include two categories for axial tension in bolts. Detail Categories A, B, and B′ are seldom critical. Investigation of a detail with a fatigue resistance greater than Category C may be only occasionally appropriate. For example, Category B applies to base metal adjacent to slip-critical bolted connections and should be evaluated if thin splice plates or connection plates are to be used.

The fatigue resistance for a Fatigue I load combination and infinite life appears as *LRFD* Equation 6.6.1.2.5-1, as follows:

\[(\Delta F)_n = (\Delta F)_{TH}\]

The fatigue resistance for a Fatigue II load combination and finite life appears as *LRFD* Equation 6.6.1.2.5-2, as follows:

\[(\Delta F)_n = (A/N)^{1/3}\]

*LRFD* 6.6.1.2.5 should be referenced for additional fatigue checks for welds and base metal.

Fatigue resistance is independent of the steel strength. The application of a higher-grade steel causes the fatigue stress range to increase, but the fatigue resistance remains the same. This implies that fatigue can become a controlling factor where a higher-strength steel is used.

407-4.01(03) Stress Cycles

*LRFD* defines the number of stress cycles, \(N\), in Equation 6.6.1.2.5-3, as follows:

\[N = (365)(75) n \ (ADTT)_{SL}\]

where:
\[ n \] = number of stress range cycles per truck passage. As defined in LRFD 6.6.1.2.5, for a simple or continuous span of less than 40 ft, \( n = 2.0 \). For a span of 40 ft or longer, \( n = 1.0 \). For a location within 0.1 of the span length from a continuous support, \( n = 1.5 \).

\[(ADTT)_{SL} = \text{Average Daily Truck Traffic in a single lane} = (p)(ADTT), \text{which is LRFD Equation 3.6.1.4.2-1.}\]

\[ p \] = the fraction of truck traffic in a single lane. This is defined in LRFD 3.6.1.4.2. If one direction of traffic is restricted, \( p \) should be taken as follows:

- 1 lane, \( p = 1.00 \)
- 2 lanes, \( p = 0.85 \)
- 3 or more lanes, \( p = 0.80 \)

\[ \text{ADTT} = \text{the number of trucks per day in one direction averaged over the design life of the structure.} \]

The portion of LRFD Equation 6.6.1.2.5-2 that is (365)(75)(ADTT)SL represents the total accumulated number of truck passages in a single lane during the 75-year design life of the structure. If site-specific values for the fraction of truck-traffic data are unavailable from the Planning Division, the values provided in LRFD Table C3.6.1.4.2-1 may be used.

Figure 407-4A provides the annual growth rate based on recommendations of the Planning Division’s Traffic Monitoring Team. The designer should contact the Traffic Monitoring Team for more-specific and up-to-date annual growth rates which can be available.

**407-4.02 Distortion-Induced Fatigue**

LRFD 6.6.1.3.1 and 6.6.1.3.2 provide detailing practices for transverse and lateral connection plates intended to reduce significant secondary stresses which can induce fatigue-crack growth.

**407-4.03 Other Fatigue Considerations**

The designer is responsible for ensuring compliance with fatigue requirements for all structural details, e.g. stiffeners, connection plates, lateral bracing, etc., shown on the plans.

During construction, field personnel will desire to field-weld attachments, either permanent or temporary, to the top flange to facilitate setting deck forms and other appurtenances. The plans for a continuous structure should include a detail showing the location of compression, reversal,
and tension regions along the girder top flange. The length of each stress region shall be shown. Each shall be referenced to the point of support. Figure 407-4B illustrates the information required. This detail will provide the field personnel with the necessary information to prevent welding in tension or reversal zones which can be detrimental to the fatigue life of the structure.

The fatigue requirements described elsewhere in LRFD Chapter 6 should be considered. They include the following:

1. fatigue due to out-of-plane flexing in web of plate girder, LRFD 6.10.5.3.
2. fatigue in shear connectors, LRFD 6.10.10.2 and 6.11.10; and
3. bolts subject to tensile fatigue, LRFD 6.13.2.10.3.

407-5.0 DIMENSIONING AND DETAILING REQUIREMENTS

407-5.01 Dead-Load Camber

Dead-load camber shall be designed in accordance with LRFD 6.7.2.

407-5.01(01) General

Steel beams or girders shall be cambered to compensate for the profile grade and offset the deflections due to applied dead loads. Camber shall be displayed in inches with a precision of two decimal places. Dead load should include the weight of the steel, and deck and railing, but not the future wearing surface. The effects of superelevation, where applicable, should also be considered. Unfactored force effects shall be used to determine the deflections.

407-5.01(02) Diagram

The plans should include a diagram and an optional table showing total camber due to the effects listed in Section 407-5.01(01). Figure 407-5A illustrates an example of the no-load camber and reaming diagram for bolted field splices and the table of cambers. Camber should be computed assuming the girder is lying on its side; i.e., not loaded. This information is required for fabrication. The detail should reference dimensions to the bottom edge of the web; i.e., assuming that flanges have not been attached.

The basic reference line should extend as a straight line from the two end supports along the centerline of the girder. At each support, a blocking dimension shall be provided from this
reference line to the bottom of the web. This dimension is numerically equal to the offset of the profile grade from a straight line extending through the end bent stations along the profile-grade line. These are control dimensions for assembling the girder sections for reaming.

Within each span, another reference line shall be established extending between supports. Camber ordinates shall be referenced to this line. Camber is cut into the webs of plate girders using these dimensions. Camber ordinates are shown in inches at a minimum of the each span’s tenth points and at each splice location.

**407-5.02 Minimum Thickness of Steel**

The thicknesses of steel elements shall be as described in *LRFD* 6.7.3, and should not be less than the following.

1. Plate girder webs, ½ in.
2. Plate-girder flange, 3/4 in., or 1 in. for horizontally-curved girders.
3. Rolled beam or channel webs, 1/4 in.
5. Connection plates, ½ in.
6. All other structural-steel elements, 5/16 in.

**407-5.03 Diaphragms and Cross Frames**

The use of diaphragms and cross-frames shall be considered as described in *LRFD* 6.7.4 and 6.6.1.3.1. Their purpose is to stabilize the beams during and after construction and to distribute gravitational, centrifugal, and wind loads.

**407-5.03(01) General**

1. **Location.** Diaphragms or cross-frames shall be placed at each support and throughout the span at an appropriate spacing. The location of the field splices should be planned to avoid conflict between the connection plates of the diaphragms or cross-frames and the splice material.

2. **Spacing.** The maximum spacing of diaphragms and cross-frames shall be based on an analysis as outlined in *LRFD* Article 6.7.4. Where integral or semi-integral end bents are
used, the first interior diaphragm shall be placed within 10 ft of the centerline of bearing to provide beam stability prior to and during the deck pour.

3. **Skew.** All intermediate diaphragms and cross-frames shall be placed perpendicular to the beams or girders. For a skew of less than 20°, the intermediate diaphragms and cross-frames should be continuous and not staggered.

4. **Ends.** End diaphragms and cross-frames should be placed along the centerline of bearing. The top of the diaphragm shall be set below the top of the beam or girder to accommodate the joint and the thickened slab at the end of the superstructure deck, where applicable. The end diaphragms should be designed to support the edge of the slab including live load plus impact.

5. **Curved Structure.** Diaphragms or cross-frames connecting curved girders are considered primary members and should be oriented radially.

6. **Design.** LRFD refers to the AISC Specification for Load and Resistance Factor Design of Single Angle Members, included in its Specification for Load and Resistance Factor Design Manual of Steel Construction. Therefore, design of single angle members shall be in accordance with the AISC LRFD Specifications.

**407-5.03(02) Diaphragm Details**

For a span composed of rolled beams, diaphragms at continuous supports and at intermediate span points may be detailed as illustrated in Figure 407-5B. Figure 407-5C illustrates the typical end-diaphragm-connection details for rolled beams. Plate girders with web depths of 42 in. or less should have the same diaphragm details. For plate girders with webs deeper than 42 in., use cross-frames or K-frames as detailed on Figure 407-5D at continuous supports and at intermediate span points. Figure 407-5E and Figure 407-5F illustrates the typical end cross-frame or K-frame details.

Intermediate diaphragms should be designed and detailed as non-load bearing. Diaphragms at points of support should be designed as a jacking frame, if needed.
### 407-5.03 Cross-Frame Details

Figure 407-5D illustrates typical intermediate cross-frame details for plate-girder webs of more than 42-in. depth. The X-frame at the top of the figure is more cost effective than the K-frame at the bottom. However, the K-frame should be used instead of the X-frame where the girder spacing becomes much greater than the girder depth, for example, where the angle of the diaphragm is less than 30 deg, and the X becomes too shallow. A solid bent-plate diaphragm with a depth equal to \( \frac{3}{4} \) the girder depth is an option for a plate girder of less than 48-in. depth.

The rolled angles that comprise the cross frames are of minimum sizes based upon the limiting slenderness ratios shown in *LRFD* 6.8.4 and 6.9.3.

Cross-frame transverse connection plates, where used, shall be welded to both the tension and compression flanges. The connection-plate welds to the flanges should be designed to transfer the cross frame forces into the flanges.

The width of the connection plates should be sized to use bar stock, and shall be not less than 5 in. Where the connection plate also acts as a transverse stiffener, it shall satisfy *LRFD* 6.10.11.1.

If the design permits, gusset plates shall be eliminated by means of bolting cross frame members directly to the stiffeners.

Figures 407-5G through 407-5I illustrate the stiffener and connection plate details that can be used.

### 407-5.04 Jacking

Jacking shall be as described in *LRFD* 3.4.3. The plans shall show jack-point locations and design loads. The beam, girder, or jacking frame shall be capable of resisting 1.3 times the dead-load reactions at those points. A slender beam may require web stiffeners at the jacking point. Such stiffeners may be fastened to the girder if jacking is required. Jacking frames will not be required at the supports unless there is insufficient clearance between the bottom of beam and top of cap to place a jack. If insufficient clearance is provided for the jack, it shall be determined whether the jack can be supported by means of temporary falsework. If temporary falsework is not feasible, a jacking frame should be provided, or the cap widened and the bearings placed on pedestals to provide sufficient space for a jack to be placed under the beam. Other locations where jacking may be required are as follows:

1. at supports under the expansion joints where joint leakage can deteriorate the girder bearing areas; or
2. at large-displacement expansion bearings where deformation-induced fatigue is possible.

If a jacking frame is not provided, the cross-frame at the support shall be capable of transferring lateral wind forces to the bearings. For a continuous structure with integral or semi-integral end bents, providing jacking frames at interior supports should not be considered.

407-5.05 Lateral Bracing

Lateral bracing should be eliminated where practical. The measures which may eliminate lateral bracing for straight I-beams are reducing the cross-frame spacing and increasing the flange width.

LRFD 6.7.5 requires that the need for lateral bracing be investigated for all stages of assumed construction procedures. If the bracing is included in the structural model used to determine force effects, it should be designed for all applicable limit states.

LRFD 4.6.2.7 provides for alternatives relative to lateral wind distribution in a multi-beam bridge.

407-5.06 Heat-Curved Rolled Beam and Welded Plate Girder

These shall be designed in accordance with LRFD 6.7.7.

407-5.07 Shims

Shims shall be included and the minimum thickness shall be 1/8 in. The shim packs shall be 1/2 in. minimum in total thickness, with multiple shim plates in a pack.
407-6.0  I-SECTIONS IN FLEXURE

407-6.01  General

407-6.01(01)  Negative Flexural Deck Reinforcement

LRFD 6.10.1.7 requires that, in the negative-moment region where the longitudinal tensile stress in the slab, due to factored construction loads or the Service II load combination, exceeds the factored modulus of rupture, the total cross-sectional area of the longitudinal steel should not be less than 1% of the total cross-sectional area of the deck slab excluding the 1/2-in. sacrificial wearing surface. However, sufficient negative-moment steel shall be provided for the applied loads.

407-6.01(02)  Stiffness in Negative-Moment Areas

LRFD 6.10.1.5 permits assuming uncracked concrete in the negative-moment areas for member stiffness. This is used to obtain continuity moments due to live load, future wearing surface, and barrier weights placed on the composite section.

The transformed-section properties should be calculated based on three times the modular ratio for composite dead loads of railing, future wearing surface, utilities, etc., and one times the modular ratio for composite live loads.

For the Service Limit state control of permanent deflections described in LRFD 6.10.4.2 and the Fatigue Limit state described in LRFD Article 6.6.1.2, the concrete slab may be considered fully effective for both positive and negative moments for a member with shear connectors throughout its full length and satisfying LRFD 6.10.1.7.

407-6.02  Strength Limit State

Moment redistribution will be permitted for continuous spans of I-section members, if \( F_y \leq 70 \text{ ksi} \) and if members satisfy the requirements of LRFD Appendix B6.2.

407-6.03  Service Limit State Control of Permanent Deformations

Moment redistribution as described in LRFD 6.10.4.2 and Appendix B6 is permitted for the investigation of permanent deformations.
**407-6.04 Shear Connectors**

Shear connectors shall be designed in accordance with LRFD 6.10.10. They shall consist of welded studs, with a preferred diameter of 7/8 in., and a minimum diameter of 3/4 in. The minimum height is 4 in. Shear connectors should have a minimum 2.5-in. concrete cover and should penetrate at least 2 in. above the bottom of deck slab. The stud length should be increased in 1-in. increments where necessary to maintain a 2-in. minimum penetration of the stud into the deck slab. The stud height may need to be modified at splice locations for minimum cover requirements. The minimum longitudinal shear connector pitch is six stud diameters. The maximum such pitch is 2 ft. See Figure 407-6A for additional information.

The minimum number of studs in a group is two in a single transverse row. The transverse spacing, center to center, of the studs should be not less than four stud diameters. The minimum clear distance between the edge of the beam flange and the edge of the nearest stud should be 1 in. Details and spacing of stud shear connectors shall be shown on the plans.

If the structure is skewed 5 deg or less, the rows of shear connectors shall be placed perpendicular to the centerline of the roadway. If the skew is greater than 5 deg but less than or equal to 25 deg, the rows of shear connectors shall be placed along the skew. If the skew is greater than 25 deg, the rows of shear connectors shall be placed perpendicular to the centerline of the roadway, which will be the same as that of the transverse reinforcing steel in the deck.

**407-6.05 Stiffeners**

**407-6.05(01) Transverse Intermediate Stiffeners**

A straight girder may be designed without intermediate transverse stiffeners, if economical, or with intermediate transverse stiffeners placed on one side of the web plate. Due to the labor intensity of welding stiffeners to the web, the unit cost of stiffeners by weight is approximately nine times that of the web. It is seldom economical to use the thinnest web plate permitted. Therefore, the use of a thicker web and fewer intermediate transverse stiffeners, or no intermediate stiffeners, should be investigated.

Intermediate transverse stiffeners should be welded near side and far side to the compression flange. A tight fit shall be used for the tension flange including stress-reversal areas. This exceeds the requirements of LRFD 6.10.11.1. See Figure 407-6B for details.

Transverse stiffeners, except at diaphragm or cross-frame connections, should be placed on only one side of the web. The width of the projecting stiffener element, moment of inertia of the
transverse stiffener, and stiffener area should be in accordance with LRFD 6.10.11.1.2 and 6.10.11.1.3.

Longitudinal stiffeners used in conjunction with transverse stiffeners on a longer span with deeper webs should be placed on the opposite side of the web from the transverse stiffener. Where this is not practical, e.g., at an intersection with cross-frame connection plates, the longitudinal stiffener should not be interrupted for the transverse stiffener.

407-6.05(02) Bearing Stiffeners

Bearing stiffeners are required at the bearing points of each rolled beam or plate girder. Bearing stiffeners at integral end bents may be designed for dead and construction loads only.

The bearing stiffeners shall be designed as columns. The stiffeners shall be extended to the outer edges of the bottom flange plate. LRFD 6.10.11.2 does not specify an effective column length for the design of bearing stiffeners. Because the reaction load applied at one end of the stiffener pair is resisted by forces distributed to the web instead of by a force concentrated at the opposite end, as in columns, it is not necessary to consider the stiffeners as an end-hinged column where the flanges are free to rotate. An effective column length of ¾ of the web depth shall be used.

The weld connecting the stiffener to the web should be designed to transmit the full bearing force from the stiffener to the web due to the factored loads.

The bearing stiffeners may be either milled to bear plus fillet welded against the flange through which they receive their reaction, or welded to the flange with full-penetration groove welds. See Figure 407-5G for details. All bearing stiffeners under full dead load shall be vertical to within applicable fabrication and construction tolerances.

407-6.06 Cover Plates

Cover plates will not be permitted for a new, rehabilitated, or widened bridge.

LRFD 6.10.12.1 requires that partial-length cover plates should not be used with flange plates whose thickness exceeds 3/4 in. in a non-redundant-load-path structure. According to LRFD 1.3.4, those elements and components whose failure is not expected to cause collapse of the bridge should be designated as not failure-critical, and the associated structural system as redundant. The thickness of a single cover plate should not exceed twice the thickness of the flange plate. Multiple cover plates should not be used. The width of the cover plate should be different from that of the
flange plate to allow for proper placement of the weld. The ends of partial-length cover plates should be terminated with a bolted connection according to LRFD 6.10.12.2.3.

407-6.07 Constructability

LRFD 6.10.3 and its commentary provide additional information regarding constructability of a steel I-girder bridge.

See Chapter 403 for additional guidance for construction loading.

407-7.0 BOX SECTIONS IN FLEXURE

407-7.01 General

LRFD 6.11 addresses most aspects of steel-box-girder design, fabrication, inspection, and maintenance. However, additional designing and detailing issues to be considered are described below and in the AASHTO/NSBA Steel Bridge Collaboration, Task Group 1.4, Guidelines for Design Details.

Due to the high torsional rigidity and resistance associated with a closed section, box girders are particularly suitable for a curved bridge in which torsional moments resulting from curvature can be high.

Steel box girders are economical only if their total number of webs is fewer than those in a comparable steel-plate-girder bridge. Box girders should have a constant trapezoidal or rectangular shape and should be rotated with the cross slope. The top of slab to top of web dimension shall be kept constant. See Figure 407-7A for preferred horizontal geometry. Departure from the shown geometry can result in difficulties in generating shop drawings. The profile-grade line and horizontal-control line locations shown in Figure 407-7A are for demonstration purposes only. The centerline-of-bearing offset shall be considered in detailing substructure elements.

407-7.02 Flanges

In addition to the requirements of LRFD 6.11.2.2, the top flanges for box girders should be in accordance with the requirements for plate girder flanges.
For the bottom flange, plate distortion during fabrication and erection can be a problem. If using a bottom tension flange plate of less than 1-in. thickness, a fabricator shall be contacted to determine whether practical stiffness needs are satisfied. A bottom tension flange should not have a thickness of less than ¾ in. The bottom tension flange should have a $b/t$ ratio of 80 or less. The bottom flange’s edges should extend at least 1½ in. beyond the web centerline to facilitate automated welding.

A wide box can have thin bottom flanges in the tension region resulting in a visible deflection in the transverse direction due to the weight of the plates. If the deflection affects the appearance of the bridge, it may be reduced to an acceptable limit by transversely stiffening the plate. If using longitudinal stiffeners, a clear distance shall be maintained between longitudinal stiffeners of not less than 24 in. to accommodate automated welding equipment. Therefore, the minimum flange width between webs is 48 in. if using one stiffener, or 72 in. if using two stiffeners. More than two stiffeners shall not be used per flange. If not using longitudinal stiffeners, the minimum width should be 48 in. to facilitate welding for the web-to-bottom-flange connection.

For straight girders, plates or bars shall be used instead of WT shapes for longitudinal stiffeners as long as they satisfy LRFD criteria. Plate and bar sections are less expensive and easier to splice than WT sections. For curved girders, WT sections shall be used. If WT sections are used, the ratio of depth to one-half the WT flange width should be greater than 1.5 to provide welding access. Termination of longitudinal stiffeners shall be at a bolted field splice so that fatigue is not a concern at the stiffener’s end.

### 407-7.03 Webs

A minimum web depth of 60 in. should be used for the purposes of maintenance and inspection. Other LRFD 6.11.2.1 requirements for plate girders are applicable to box girders.

### 407-7.04 Stiffeners

Stiffeners shall be designed in accordance with LRFD 6.11.11. For stiffeners and connections plates for internal cross frames, an option to provide fabricators is shown in Figure 407-7B. Cutting stiffeners short of the bottom flange facilitates automated welding of the web to the bottom flange. After this welding is complete, the stiffener can then be attached to the bottom flange with an additional plate.
Complete-penetration groove welds shall not be specified to connect bearing stiffeners to bottom flanges. Weld-induced flange distortion is more of a problem with box-girder flanges than those for plate girders.

**407-7.05 Top-Flange Horizontal Lateral Bracing**

Lateral bracing shall be used in straight or curved box girders. The top lateral bracing should be designed to resist force effects caused by the dead-load torsional moments. The section shall be analyzed and designed as a closed section by using the deck slab as the top member in considering live load and superimposed dead loads. Stresses due to flexure and torsion should be combined in the design.

Lateral bracing shall be bolted directly to the top flange. Enough slab haunch shall be provided so that formwork does not interfere with the bracing. Shims or fill plates shall not be used between the lateral bracing and girder top flange which will increase eccentricity of the connection.

Single-laced lateral bracing is preferred over double-laced bracing. The angle between the girder flange and the bracing should be at least 35 deg. An angle closer to 45 deg is desirable.

**407-7.06 External Diaphragms and Cross Frames between Piers**

External diaphragms shall be designed in accordance with *LRFD 6.7.4.3*. Cross frames shall be designed in accordance with *LRFD 6.7.5.3*. External diaphragms or cross frames are used to control relative displacement and twist of girders during slab placement. Once the slab has sufficiently cured, they may be removed, which is done primarily for aesthetic reasons. If they are to remain in place, they should complement the overall structural aesthetics and should include fatigue-resistant features.

For curved box girders, external diaphragms or cross frames at the span quarter or third points is usually sufficient, so, adding more is unnecessary. With straight box girders, one external cross frame or diaphragm at mid-span should be sufficient. External diaphragms or cross frames shall be backed up with an internal diaphragm or cross frame.
407-7.07 Internal Diaphragms and Cross Frames Between Piers

Internal diaphragms and cross frames are used to control cross-section distortion. For curved box girders, an internal cross frame or diaphragm shall be placed at every other lateral-bracing point, which should result in a spacing of 14 to 18 ft. Horizontal struts, usually angle sections, shall be placed at the lateral brace point between internal cross frames to control horizontal bending of the flange during concrete placement. Like lateral bracing, they should be attached directly to the flanges.

For straight box girders, internal diaphragms or cross frames can be spaced at every third or fourth lateral-bracing point.

407-7.08 Pier Diaphragms and Cross Frames

Assuming one bearing per girder, diaphragms at a pier should be plate-girder sections that are approximately the same depth as the girders. They should connect to the box-girder flanges and web if their span-to-depth ratio is 3 or more. With two bearings, a cross frame is preferred at the pier. Bearing assemblies shall not interfere at the bottom-flange connection. Constructability of diaphragms shall be considered at the girder ends, as the presence of abutment backwalls or other girders can complicate bolting the diaphragms in place. An inspection-access hole shall be provided through the diaphragm web plate at the intermediate supports. See Section 407-7.11 for recommended opening sizes. See Figure 407-7C for a typical pier diaphragm between girders with one bearing per girder.

407-7.09 Field Splices

Field splices shall be bolted. The requirements for plate girders also apply. Overall girder width, including sweep, should be not more than 14 ft for ease of shipping.

407-7.10 Bearings

One bearing per girder is preferred. A girder may not bear evenly on both bearings or, on only one of the bearings with two bearing supports. This is true for skewed piers or curved spans. An uplift force can occur at one of the two bearings of an individual box, so a separate investigation should be made to determine if uplift does occur.
High-Load Multi-Rotational (HLMR) or pot bearings may be necessary. Neoprene elastomeric bearings are preferred over HLMR bearings. HLMR bearings shall be used for reactions of more than 1200 kip, or for a large-span curved bridge.

Bearings without anchor bolts, or with anchor bolts through masonry plates only, are preferred. Anchor bolts shall not pass through the girder’s bottom flange. An external alignment device shall be used for restraint that is flexible in terms of placement after girder erection.

Bearings should be designed to accommodate bearing replacement with a minimal amount of lifting.

407-7.11 Electric Service and Inspection Access

Electric-service access should be provided in accordance with LRFD 6.11.1.4, on the insides of the girders, with outlets spaced at not more than 100 ft to facilitate maintenance and inspection during the life of the bridge. The long girder length between access holes or doors necessitates this requirement.

An access hole shall be provided with a lockable door or cover in the bottom flange near each end support for inspector access. The door or cover shall weigh 25 lb or less, to be usable by an inspector.

At the girder ends, in the bottom flange, a minimum 30-in. diameter access door shall be placed. In a pier diaphragm at an intermediate support, the minimum access-hole opening shall be 36 in. by 18 in. This opening shall desirably be 36 in. by 32 in. The holes shall be placed at mid-depth and concentric with the box. The opening corners should have a minimum radius of 6 in.

407-7.12 Constructability

Stay-in-place forms are used inside a steel box due to the difficulty of removing forms. Stay-in-place forms, however, have a limiting span length which can be exceeded in a wide box. For this situation, the plans should provide for an intermediate support. The intermediate support is at the option of the contractor and need not be detailed. However, an optional diaphragm to carry the reaction of the intermediate-form support should be designed and shown on the plans.

See Chapter 403 for additional guidance on construction-loading criteria.
407-8.0 CONNECTIONS AND SPLICES

407-8.01 Bolted Connections

Bolted connections shall be designed in accordance with LRFD 6.13.12. The following also applies.

1. **Type.** High-strength bolts shall be used. See LRFD 6.4.3. For unpainted weathering steel, ASTM A 325 Type 3 bolts should be used. For other than weathering steel, A 325 Type 1 bolts should be used. For a large or curved-girder structure, A 490 bolts shall be used.

2. **Design.** A bolted connection should be designed as slip-critical at the Service II Limit state and for construction loading, except for secondary bracing members.

3. **Slip Resistance.** LRFD Table 6.13.2.8-3 provides values for the surface condition. Class B surface condition shall be used for the design of a slip-critical connection.

407-8.02 Welded Connections

LRFD 6.13.3 specifies fatigue and welding requirements. The AASHTO/AWS D1.5/D1.5 Bridge Welding Code specifies requirements for welded connections. Welding symbols shall be as specified in AWS Publication A2.4.

The following requirements apply to welding.

1. **Accessibility.** Accessibility shall be provided to welded joints. Sufficient clearance shall be provided to enable a welding rod to be placed at the joint. An isometric drawing of the joint can reveal difficulties in welding or indicate where critical weld stresses shall be investigated.

2. **Minimum Fillet Weld.** The weld should be designed economically, but its size should not be less than 1/4 in., and not less than that shown in LRFD 6.13.3.4 for the thicker of the two parts joined.

3. **Field Welding.** Field welding is prohibited for all splices.

4. **Intersecting Welds.** These should be avoided. There should be a gap equal to four times the web thickness, $4t_w$, or 2 in., whichever is larger, between vertical and horizontal welds.
A bridge that includes intersecting vertical and horizontal welds or that has gaps of less than \(4\ell_w\) or 2 in. is prone to fatigue cracks.

5. **Intermittent Fillet Welds.** These are prohibited.

6. **Partial-Penetration Groove Welds.** These are prohibited except as permitted in *LRFD* 9.8.3.7.2.

### 407-8.03 Splices

In addition to the requirements of *LRFD* 6.13.6, the following will apply.

1. **Location.** Field splices should be located at low-stress areas and near the points of dead-load contraflexure for continuous spans. Numerous butt welds or butt welds located in high-stress regions are not desirable. The location of shop butt splices is normally dependent upon the length of plate available to the fabricator. This length varies depending upon the rolling process. The maximum length of normalized and quenched and tempered plates is 50 ft. Other plates can be obtained in lengths of greater than 80 ft depending on thickness. Shop availability should be verified for lengths and thicknesses of plate material. The cost of adding a shop-welded splice instead of extending a thicker plate should be considered in designing members. Discussion with a fabricator or the NSBA during the design is recommended.

2. **Field Splices.** For a girder longer than 100 ft, additional field splices may need to be shown on the plans.

3. **Swept Width.** For a curved girder, the swept width between splices should be limited to 10 ft to accommodate the shipment of the steel.

4. **Bolts.** Bolt loads should be calculated by means of an elastic analysis method. Not less than two lines of bolts shall be provided on each side of the joint for both the web and flange splice.

5. **Composite Girder.** If a composite girder is spliced at a section where the moment can be resisted without composite action, the splice may be designed as noncomposite. If composite action is necessary to resist the loads, the splice should be designed for the forces due to composite action.
6. **Top-of-Field-Splice Elevations.** A table showing the erection elevation of the top-flange splice plate should be provided for all field splices. This elevation is determined by the following:

\[(\text{Top of slab elevation immediately above centerline splice}) - (\text{distance from top of slab to top of splice plate}) + (\text{concrete and superimposed dead-load deflection at the splice})\]

The table will allow the erector to properly position the girder sections at the time of erection to maintain the vertical alignment.

7. **Design.** A bolted splice shall be slip-critical under Service II and construction loads and shall be designed in accordance with *LRFD* 6.13.2.2 for the Strength Limit state.

8. **Welded Splice.** Figure 407-1C illustrates welded-splice details. See *LRFD* 6.13.6.2 for more information regarding splicing different widths of material using butt welds.

### 407-9.0 TRUSSES AND ARCHES

A truss shall be designed in accordance with *LRFD* 6.14.2. A solid-web arch shall be designed in accordance with *LRFD* 6.14.4.
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<td>1 7/8</td>
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<td>7/8</td>
<td>2 1/2</td>
</tr>
<tr>
<td>1</td>
<td>---</td>
</tr>
</tbody>
</table>

Notes:
1. For thickness of < 1 in., use 1/16-in. increments.
2. For thickness of ≥ 1 in., use 1/8-in. increments.
3. Based on ANSI Standard B32.3. Mills can produce a plate size upon request.

PLATE THICKNESSES

Figure 407-1A
FLANGE GROUPING FOR FABRICATION

Figure 407-1B
TYPICAL GIRDER DETAIL

WEB THICKNESS TRANSITION

FLANGE WIDTH TRANSITION
(MUST USE RADIUS FOR STEEL STRENGTHS ≥ 70KSI)

GIRDER WELD SPLICE DETAILS

Figure 407-1C
NOTE "A": USE CLIP ANGLE BETWEEN STIFFENERS. 8'-6" IS MAXIMUM SUPPORT SPACING.

NOTE "B": HOLE SIZE IN CONN. STIFFENERS MUST BE THE SAME AS OTHER HOLES IN THE STIFFENERS (MIN. 1 13/16"

NOTE "C": USE SUPPORT BETWEEN STIFFENERS. 8'-6" IS MAXIMUM SUPPORT SPACING.

"Ø HOLE HOLES FOR %" PLAIN BARS. BARS TO BE MADE CONTINUOUS THROUGH USE OF WELDED SPLICES.

Figure 407-1D

SAFETY HANDRAIL DETAILS
BEARING RESTRAINTS

Figure 407-1E
BEGINNING OR ENDING OF BRIDGE

1. End of Girder

2. End of Girder

3. MSE WALL

4. INTERIOR BENTS

BEGINNING OR ENDING OF BRIDGE

1. INTEGRAL END BENT

2. JOINED END BENT

3. MSE WALL

4. INTERIOR BENTS

WEATHERING STEEL
(PAINT LIMITS)

Figure 407-2A
TYPICAL DRIP BAR DETAIL (END BENT)

TYPICAL DRIP BAR DETAIL (PIER)

SECTION B-B

Note: Drip Bars shall be located on the upward slope of all exterior girders adjacent to End Bent or Piers.

Drip Bars shall be caulked with dark brown caulking against flange, web and fillet welds.

DRIP BAR DETAILS

Figure 407-2B
<table>
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<tr>
<th>Facility</th>
<th>Annual Growth Rate</th>
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<tbody>
<tr>
<td>Rural or Urban Freeway</td>
<td>3.07 %</td>
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<tr>
<td>Divided Rural Non-Freeway</td>
<td>1.51 %</td>
</tr>
<tr>
<td>Divided Urban Non-Freeway</td>
<td>1.32 %</td>
</tr>
<tr>
<td>Undivided Rural Arterial</td>
<td>1.51 %</td>
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<tr>
<td>Rural Collector or Local Road</td>
<td>2.45 %</td>
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<tr>
<td>Undivided Urban Facility</td>
<td>1.32 %</td>
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**ANNUAL TRAFFIC GROWTH RATE**

Figure 407-4A
NOTE: LENGTH OF COMPRESSION, TENSION AND REVERSAL LIMITS TO BE SHOWN TO NEAREST INCH.

SCHEMATIC OF TOP FLANGE STRESS

Figure 407-4B
Dead Load - Steel
Dead Load - Railing
Subtotal - Dead Load
Geometric Camber
Total Camber

NO LOAD CAMBER AND REAMING DIAGRAM
(SKETCH SHOWN FOR CREST VERTICAL CURVE)

TABLE OF CAMBERS (in) EACH SPAN

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>1</th>
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<th>3</th>
<th>4</th>
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<td>Subtotal - Dead Load</td>
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BLOCKING DIMENSIONS (in)

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<td>Girder Line #</td>
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BEAM CAMBER & BLOCKING DETAILS

Figure 407-5A
Designer shall verify actual sizes by design.

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

### Intermediate Diaphragm for Rolled Sections

<table>
<thead>
<tr>
<th>Connection Detail</th>
</tr>
</thead>
</table>

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

### Table 1

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<th>Stringer Size</th>
<th>Diaphragm Size</th>
<th>No. of Bolts</th>
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<tr>
<td>( \geq 27'' ) Depth</td>
<td>C 15 x 33.9</td>
<td>8</td>
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<tr>
<td>Up to 24'' Depth</td>
<td>C 12 x 25</td>
<td>6</td>
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### Rolled Beam Intermediate Diaphragm Details

Figure 407-5B
END DIAPHRAGM FOR ROLLED SECTIONS

TABLE 1

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<tr>
<th>BEAM SIZE</th>
<th>DIAPHRAGM SIZE</th>
<th>NO. OF BOLTS</th>
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<tr>
<td>≥ 27&quot; DEPTH</td>
<td>C 15 x 33.9</td>
<td>8</td>
</tr>
<tr>
<td>UP TO 24&quot; DEPTH</td>
<td>C 12 x 25</td>
<td>6</td>
</tr>
</tbody>
</table>

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

ROLLED BEAM END DIAPHRAGM DETAILS

Figure 407-5C
INTERMEDIATE DIAPHRAGM DETAIL

ALTERNATE INSTALLATION

3/8" PLATE

3/8" CONNECTION PLATE

WELD NOT REQUIRED FOR ANGLE LEGS 6" OR LESS

3" (TYP.)

6" MIN. (TYP.)

INTERMEDIATE DIAPHRAGM DETAIL

ALTERNATE INSTALLATION

3/8" PLATE

TYPICAL INTERMEDIATE CROSS FRAME DETAILS

Figure 407-5D

Note: All member sizes shown are minimum. Designer shall verify actual sizes by design.
TYPICAL END CROSS FRAME DETAILS

Figure 407-5E
ALTERNATE END CROSS FRAME DETAILS

Figure 407-5F
SAME DIMENSION - BOTH SIDES

1. MILL TO BEAR PLUS FILLET WELD OR 2. CJP WELD

BEARING AREAS

CONNECTION PLATE

(TYP.)

STIFFENER WELDING DETAIL
(FOR SKewed STIFFENERS)

SAME DIMENSION - BOTH SIDES

WELD (WHERE INDICATED ON DETAIL)

STIFFENER AND CONNECTION
PLATE DETAILS

ALTERNATE DETAIL @ TENSION FLANGE
WHERE STRESS RANGE EXCEEDS CATEGORY C

SHOW STIFFENER PLATE AND FILLET WELD SIZE ON THE PLAN

X = 1/4" ± 1/8"

Y = 1/2" ± 1/4"

Z =

2 1/2" FOR 1/2" WEB
3" FOR 9/16" WEB
3 1/2" FOR 5/8" WEB
4" FOR 3/4" WEB

* = 0" FOR GROOVE WELD

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

Figure 407-5G
CONNECTION PLATE DETAILS

Figure 407-5H
CONNECTION PLATE DETAILS

NOTES:
1 1/4" diameter hole in connection plate.
3/8" diameter hole in connecting member for 1/4" diameter ASTM A325 bolts.
Std. size holes are permitted

SKEWS > 20°

90° (typ.)

SKEW 0° TO 20°

Symmetrical about Ø beam

See Detail L

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

CONNECTION PLATE DETAILS

Figure 407-5 I
STUD AND BOLTED SPLICE DETAILS

Figure 407-6A
STIFFENER PLATE DETAILS

Figure 407-6B
In addition to AASHTO requirements, top flanges for box girders should follow the suggestions for plate girder flanges in Section 407-1.02.

For bottom flanges, plate distortion during fabrication and erection can be a problem. Check with fabricators when using bottom tension flange plates less than 1 inch thick to determine whether practical stiffness needs are met. Bottom tension flanges should never be less than ¾ inch thick. In addition, the bottom tension flanges should have a w/t ratio of 80 or less.

BOX GIRDER HORIZONTAL CONTROL

Figure 407-7A
Do not specify complete penetration groove welds to connect bearing stiffeners to bottom flanges. Weld-induced flange distortion is even more of a problem with box girder flanges than with plate girder.
Bolt to top flange to provide load path for top lateral bracing forces, see note.

No shear studs on splice plate

Access hole

External Diaphragm

One bearing per girder

Longitudinal Stiffener

Check bottom splice plate for interference with bearing components

AT END BENT

AT INTERIOR BENT

Note: May not be possible to bolt splice plate to girder flange if piers are skewed and / or diaphragms are plumb with girders on a vertical guide.

MOMENT - CONNECTED PIER DIAPHRAGM

Figure 407-7C
CHAPTER 408

Foundation

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CHAPTER 408

FOUNDATION

A consideration for the satisfactory performance of a structure is the proper selection and design of its foundations that will provide adequate bearing resistance, tolerable lateral and vertical movements, and aesthetic compatibility. This Chapter discusses criteria for the design of structural foundations relative to spread footings, driven piles, and drilled shafts.

408-1.0 GENERAL

The design of a bridge foundation is best accomplished by an interdisciplinary team of structural, hydraulic, and geotechnical engineers. Foundations shall be designed for the specified limit states described in LRFD.

The following summarizes the concepts described in LRFD.

408-1.01 Introduction

Loads generated as a result of earth pressures can be determined with assistance from LRFD Section 11. Then the nominal and factored resistance of the substructure is computed in accordance with LRFD Section 10, the INDOT Geotechnical Manual (IGM), and the INDOT Load and Resistance Factor Design Policy for Structural Foundations.

408-1.02 Informational Needs

The expected geotechnical-exploration project requirements shall be developed in accordance with LRFD and IGM.

408-1.03 Selection of Foundation Type

A spread footing is typically the most cost effective, with the right set of conditions. It functions best in hard or dense soils that have adequate bearing resistance and exhibits tolerable settlements under load. A spread footing is most appropriate where shallow bedrock is encountered.
A spread footing shall be avoided where loss of lateral support and lack of overall stability is a possibility. A spread footing shall be avoided on soft soil or weak rock to support design loads that can create excessive settlements or loss of stability.

If a spread footing is unsuitable or uneconomical for foundation support, driven piles shall be considered.

Drilled shafts may be considered where a deep foundation is required but piles are unsatisfactory due to obstructions, noise, vibrations, voids, or steeply dipping rock. Drilled shafts can be an economical alternative to driven piles where the use of cofferdams is anticipated.

Chapter 402 discusses those types of foundations and the criteria which influence the selection of a foundation type. Other factors to be evaluated in choosing the type of foundation are discussed in Chapter 402.

408-1.04 Load Factors and Load Combinations

The LRFD loads, load combinations and load factors for use in foundation design shall be in accordance with LRFD Sections 3 and 4.

408-1.05 Limit States and Resistance Factors

The limit state shall be that specified in LRFD 1.3.2. The foundation-specific requirements are provided in LRFD 10.5 and herein. For the Service and Extreme Limit states, the resistance factor shall be taken as 1.0. A foundation shall be proportioned so that the factored resistance is not less than the effect of the factored loads specified in LRFD Section 3.

408-1.05(01) Service Limit State

Foundation design shall be in accordance with LRFD 10.5.2.

As indicated in LRFD 10.5.5.1, the resistance factors for the service limit states shall be taken as 1.0, except as provided for overall stability in LRFD 11.6.2.3.

A resistance factor of 1.0 shall be used to assess the ability of the foundation to satisfy the specified deflection criteria after scour due to the design-flood event, $Q_{100}$. 
408-1.05(02) **Strength Limit State**

The design of a foundation element shall include consideration of the nominal geotechnical and structural resistances. Deformations required to mobilize the nominal resistance need not be considered.

The resistance factors for each type of foundation system shall be taken as specified in *LRFD* 10.5.5.2.2, 10.5.5.2.3, 10.5.2., or 10.5.5.2.5, and the Indiana-specific values shown in Figure 408-1A.

The design of a foundation shall consider structural resistance, which includes axial, lateral, and flexural consideration in addition to loss of lateral and vertical support due to scour at the design-flood event, $Q_{100}$.

Foundation design shall be in accordance with *LRFD* 10.5.3.

408-1.05(03) **Extreme Limit State**

The design of a foundation shall be consistent with the expectation that structure collapse is prevented and that life safety is protected. It shall be checked as specified in *LRFD* 10.5.4.

408-1.06 **Foundation Approval**

The procedure and guidelines for a foundation review, the Foundation Review Form, and pile-tip-elevation guidelines are described below.

408-1.06(01) **Guidelines for Foundation Review**

A foundation review shall be conducted by the designer for each bridge replacement, bridge reconstruction, box structure that can be classified as a bridge, or three-sided structure including that which cannot be classified as a bridge. It shall be conducted once actual loads are available, but not later than the Stage 3 submittal.
The guidelines for conducting a foundation review are as follows.

1. Minimum pile-tip elevations for scour for the interior substructure shall be determined in accordance with the method outlined in Figure 408-3D, the Pile Tip Elevation Guidelines flowchart.

2. The minimum pile-tip elevation for a pile footing shall be determined using the check flood, $Q_{500}$ scour elevation.

3. Where the bottom of a pile footing is located above the design flood, $Q_{100}$ scour elevation, the piling shall be designed as free-standing for the unsupported pile length above the design flood, $Q_{100}$ scour elevation. The resistance factors shall be as described in Section 408-5.0. The piling shall also be checked for the same criteria using the check flood, $Q_{500}$ scour elevation, with the resistance factors described in Section 408-5.0.

4. The minimum pile-tip elevation for scour shall not be confused with the estimated pile-tip elevation theoretically required to obtain the required bearing. The estimated pile-tip elevation appears in the Geotechnical Report. The lower of these two pile-tip elevations shall be used for determining the pay quantity.

5. Proposed top and bottom footing elevations shall be determined in accordance with the procedure described in Figure 408-3D.

6. The mudsill of approximately 12-in. thickness for a wall pier that has a single row of piles can be considered as an open pile bent with a deep cap. Hence, the mudsill need not be placed below the scour elevation.

7. A pier in a floodplain shall be designed as a river pier. Its foundation shall be located at the appropriate depth if there is a likelihood that the stream channel will shift during the life of the structure, or that channel cutoffs are likely to occur. For a structure or portion thereof that qualifies as an overflow structure, contact the Office of Hydraulics.

8. Engineering judgment shall be used in conjunction with Figure 408-3D in recommending pile-tip and footing elevations.

**408-1.06(02) Foundation-Review Procedure**

The designer will receive the Geotechnical Report. Once actual loads are available and before Stage 3 submittal, the designer shall propose the foundation using Figure 408-1B, the Foundation Review form.
1. The information to be provided is as follows.
   
a. Spread footing.
   Type: Spread Footing on Rock, or Spread Footing on Soil
   Size: N/A
   Factored Design load: The factored bearing resistance, $q_R$, shall be shown
   Nominal Design Load: The nominal bearing resistance, $q_n$, shall be shown
   Minimum pile tip elevation: n/a
   Use pile tips: n/a
   Bottom of Footing Elevation
   Top of Footing Elevation

b. Footing supported on piles or pile bent.
   Type: Bent or Pier
   Size: dimensions of H or pipe piles, and the shell thickness of pipe piles
   Factored Design load: The factored bearing resistance, $R_R$, shall be shown
   Nominal Design Load: The nominal bearing resistance, $R_n$, shall be shown
   Minimum pile tip elevation: As recommended in the geotechnical report
   Use of pile tips: As recommended in the geotechnical report
   Bottom of Footing Elevation
   Top of Footing Elevation

2. The designer shall then transmit the calculated foundation loads and the form to the
   geotechnical engineer who developed the Geotechnical Report.

3. If the geotechnical engineer approves, he or she signs, dates, and returns the form to the
   designer. If the geotechnical engineer disagrees with the recommendations, the marked-
   up form is returned to the designer for resubmission.

4. Once the geotechnical engineer approves the form, the designer transmits a request for a
   foundation review, with the Stage 3 submittal, which includes the following:
   a. Geotechnical Report’s summary of bridge-related items;
   b. General Plan and Layout sheets;
   c. Foundation Review form; and
   d. Scour Review memorandum.
5. The project reviewer reviews the Foundation Review form and signs and dates it once he or she concurs.

6. The project reviewer submits the form to the appropriate Bridge Design supervisor, who transmits it to the Division director. If the Division director concurs with the recommendations, he or she signs and dates the form.

7. The Bridge Design supervisor transmits the completed Foundation Review form to the project manager. The Office of Geotechnical Services shall receive a copy of the approved Foundation Review form.

408-2.0 SPREAD FOOTING

Reference: LRFD 5.8, 5.13, 10.5, 10.6

408-2.01 General

A spread footing is considered early on in the design process as a possible economical foundation option if the foundation conditions are suitable. The design of a spread footing is usually an interactive process between the geotechnical and structural designers.

A spread footing shall not be constructed on soils that can liquefy under earthquake loading.

If a spread footing is recommended, the geotechnical engineer will provide the following design recommendations in the geotechnical report:

408-2.01(01) Footing Elevations

The elevations of the proposed footing will be provided along with a description of the foundation materials that the footing is to be constructed on.

408-2.01(02) Nominal and Factored Bearing Resistances

The nominal and factored bearing resistances will be provided for each effective footing width likely to be used.
408-2.01(03) Bearing Resistance and Settlement

Bearing resistance corresponding to 0.5 in. and 1 in. of settlement at the Service Limit state shall also typically be provided unless other settlement limits are established by the designer. The designer shall communicate all footing-settlement limits to the geotechnical engineer.

For soil conditions, the bearings resistance provided assumes that the footing pressures are uniform loads acting over effective footing width, $B'$, and length, $L'$, with $(B \text{ or } L - 2e)$ as determined by the Meyerhof method. For a footing on rock, the resistances provided assume triangular or trapezoidal stress distribution and maximum toe bearing conditions.

Minimum footing setback on a slope or embedment depth shall be provided.


The soil parameters shall be provided for calculating frictional sliding resistance and active and passive earth pressures as follows:

- Soil Unit Weight, $\gamma$, for soil above footing base;
- Soil Friction Angle, $\varphi$, for soil above footing base;
- Active Earth Pressure Coefficient, $K_a$;
- Passive Earth Pressure Coefficient, $K_p$; and
- Coefficient of Sliding, $\tan \delta$.

The eccentricity of loading at the Strength Limit state, evaluated based on factored loads shall not exceed the following:

1. $1/3$ of the corresponding dimension $B$ or $L$ for a footing on soil; or
2. $0.45$ of the corresponding dimensions $B$ or $L$ for a footing on rock.

408-2.01(05) Overall Stability

The designer shall evaluate overall stability and provide the maximum footing load which can be applied to the design slope. A resistance factor of 0.65 shall be used if a structure is supported over the slope. This shall be as described in LRFD 11.6.2.3.
408-2.02 Minimum Dimensions and Materials

408-2.02(01) Spread Footing

The minimum thickness is 1.5 ft.

408-2.02(02) Class of Concrete

The concrete shall be class B.

408-2.02(03) Concrete Strength

The specified 28-day compressive strength, \( f'_c \), is 3000 psi.

408-2.02(04) Reinforcing Steel

The specified minimum yield strength, \( f_y \), is 60 ksi.

408-2.03 Footing Thickness and Shear Design

The footing thickness may be governed by the development length of the footing dowels from the footing to the wall or column, or by concrete-shear requirements. Shear reinforcement shall be avoided. If concrete shear governs the thickness, it is usually more economical to use a thicker footing unreinforced for shear instead of a thinner footing with shear reinforcement. Requirements for determining the shear resistance are provided in LRFD 5.8.3 and 5.13.3.6.

408-2.04 Depth and Cover

The vertical footing location shall satisfy the following criteria.

408-2.04(01) Bottom of Footing

The bottom of a footing on soil shall be set below the deepest frost level which is approximately 4 ft.
Where a footing is founded on rock, the bottom of the footing shall be embedded a minimum of 2 ft below the top of the rock. However, if the rock surface slopes more than 1 ft, the minimum embedment shall be 1 ft at the low end and 2 ft at the high end of the footing. Lesser minimum embedments may be used if recommended in the Geotechnical Report.

The bottom of a spread footing shall be at least 4 ft below the flowline of the streambed unless non-erodible bedrock is present. The bottom of a spread footing shall also be below the estimated depth of the check flood scour, $Q_{500}$ scour elevation.

408-2.04(02) Top of Footing

The top of the footing shall have a minimum permanent earth cover of 1 ft.

Where the footing is founded in a rock streambed, the top of the footing shall not protrude above the top of the rock.

The top of the spread footing shall be below the estimated depth of design flood scour, $Q_{100}$ scour elevation.

At a stream crossing where stream-bed materials are susceptible to scour, the top of a pile footing shall be set below what is defined by as the $Q_{100}$ contraction scour.

The top of the footing shall be set sufficiently low to avoid conflicts with the pavement section, including subbase or underdrains.

408-2.05 Soil Pressure

The resultant of triangularly vertical pressures between the footing and the foundation shall be within the middle one-half and middle three-fourths of a footing on either soil or rock, respectively. The soil pressures for such distributions may be calculated according to the formulas provided in LRFD 10.4 and 10.6.

The soil-pressure formulas can be used for a footing loaded eccentrically about one axis, e.g., retaining wall or wingwalls. LRFD 10.4 and 10.6 provide additional information on the treatment of a footing loaded eccentrically.

The factored bearing resistance and the nominal bearing resistance shall be shown on the General Plan sheet.
408-2.06 Settlement

If varying conditions exist, settlement will be addressed in the Geotechnical Report and the following effects shall be considered.

408-2.06(01) Structural

The differential settlement of the substructure causes the development of force effects in a continuous superstructure. These force effects are directly proportional to structural depth and inversely proportional to span length, indicating a preference for a shallow, large-span structure. They are normally smaller than expected and tend to be reduced in the inelastic phase. Nevertheless, they are considered in the design, especially negative movements which can either cause or enlarge existing cracking in the concrete deck slab.

408-2.06(02) Joint Movements

A change in bridge geometry, especially for a deep superstructure, due to settlement causes movement in deck joints which shall be considered in their detailing.

408-2.06(03) Profile Distortion

Excessive differential settlement can cause a distortion of the roadway profile that may be undesirable for a vehicle traveling at high speed.

408-2.06(04) Appearance

Excessive settlement can create an appearance of decay, neglect, or lack of safety.

408-2.07 Reinforcement

Reinforcement shall be as described in Chapter 405. Bar-development lengths shall be as shown in Figure 408-2A.
408-2.08 Joints

A footing shall not require expansion joints. Footing construction joints shall be offset 2 ft from expansion joints or construction joints in a wall.

408-2.09 Stepped Footing

The difference in elevation of adjacent stepped footings shall not be less than 0.5 ft. The lower footing shall extend 2 ft under the adjacent higher footing, or an approved anchorage system may be used.

408-2.10 Addition to an Existing Footing

At the interface between an existing footing and a new one, existing concrete shall be removed as necessary to provide adequate development length for lap splicing of existing reinforcement or an approved anchorage system may be used.

Where the substructure of an existing structure is extended, the old footing with respect to the new footing shall be shown on the New Footing Details sheet.

408-2.11 Cofferdam

The purpose of a cofferdam is to provide a protected area within which an abutment or a pier can be built. A cofferdam is a structure consisting of steel or wooden sheathing driven into the ground below the bottom of the footing elevation and braced to resist pressure. It shall be practically watertight and be capable of being dewatered. The sheeting used shall be wood or steel depending upon the depth and the pressure encountered. For more information, see the INDOT Standard Specifications. A cofferdam is designed and detailed by the contractor.

408-2.12 Concrete Foundation, or Tremie, Seal

A bridge with foundations located in water requires sheet-pile cofferdams to provide dry conditions for construction of the pier foundations. Under certain conditions, such as loose granular soil, the cofferdam cannot be pumped dry due to high-infiltration flows through the bottom of the excavation. A foundation seal must therefore be placed inside the cofferdam and below the proposed bottom of footing to reduce or eliminate the water infiltration.
At the preliminary field check, the designer shall check with the district construction representative and the geotechnical engineer to determine if a foundation seal shall be investigated for the foundation in question. The geotechnical engineer shall determine the need for a seal and include the recommendation in the Geotechnical Report.

Because the unreinforced seal slab is primarily to provide dry working conditions, its design is based upon the uplift force due to the amount of water displaced by the cofferdam. If a seal is specified as part of the design, the assumed water-surface elevation during foundation construction shall be shown on the plans. This elevation is assumed to be approximately 2 ft above the normal water-surface elevation.

The seal thickness shall be determined such that the weight of the concrete in the seal plus friction, or bond, on the steel foundation piling is equal to 100% of the weight of the water displaced. The minimum thickness of the seal slab shall be 2 ft.

The assumed weight of the concrete shall be 140 lb/ft³. The resistance force due to friction on the pile shall be equal to $F_b D p$, if $D < d$, or $F_b d p$, if $D \geq d$, where $F_b$ is the allowable friction, or bond, stress, $d$ is the H-pile-section depth or the pipe-pile diameter, $p$ is the perimeter, and $D$ is the depth of the seal slab. The allowable service-load bond stress between the steel H-pile or pipe pile and the seal concrete shall be taken as 36 psi.

Tension shall be checked in the concrete seal due to bending moments induced by the force of the water pressing upward on the bottom of the slab minus the weight of the seal concrete. The piles shall be treated as the points of support for the slab. The concrete slab shall be treated as an unreinforced-concrete beam. The maximum service load tension in the seal concrete shall be 25% of $7.5(f'_{c})^{1/2}$.

### 408-2.13 Proof Testing of Rock

Excavation for a spread footing on rock shall be proof tested to check the integrity of the rock. See the INDOT *Standard Specifications* for the proof-testing procedure.

### 408-3.0 PILES

Reference: *LRFD* 5.13, 6.9, 6.12, 10.7
408-3.01 General [Rev. Aug. 2018]

If underlying soils cannot provide adequate bearing capacity, scour resistance, or tolerable settlements, piles may be used to transfer loads to deeper suitable strata through friction or end bearing. The selected type of pile is determined by the required bearing capacity, length, soil conditions, and economic considerations. Steel pipe piles and steel H-piles are most commonly used. Other pile types, such as auger-cast piles or timber, may be considered.

If a spread footing is unsuitable or uneconomical for foundation support, driven piles should be considered. The geotechnical engineer should be contacted to determine the most appropriate pile type, size and nominal resistance to support the desired pile loads.

A cost-effective pile foundation should consider both the number of piles driven, as well as the pile driving equipment necessary for installation. Increasing the nominal driving resistance may reduce the total number of piles, but increase the installation costs when a contractor must rent a larger hammer and/or crane to achieve the higher driving resistance.

Designers should limit the nominal driving resistance, $R_{ndr}$, to 426 kips for routine bridge projects. This value correlates to commonly owned pile hammers with maximum energy ratings of 69,000 - 75,000 ft-lbs. Limiting $R_{ndr}$ also reduces the risk of pile damage during installation.

Higher $R_{ndr}$ values may be feasible for site-specific conditions where higher installation costs are offset by a significant reduction in the number of piles. Where higher $R_{ndr}$ values are appropriate, the designer should coordinate with the project geotechnical engineer.

Typical pile types, sizes, and nominal resistances for piles terminating in soil, soft rock, or shale, are listed in Figure 408-3A.

The factored design soil resistance may be increased if the field method of resistance/capacity determination is by static load test and if approved by the Office of Geotechnical Engineering.

For piles seated on hard rock, such as limestone, etc., the maximum nominal geotechnical resistance shall be less than or equal to $0.65A_sF_y$. The maximum factored geotechnical resistance shall be less than or equal to $0.46A_sF_y$. This does not apply to piles seated in soft rock, such as shale, weathered rock, etc.

For friction piles, the geotechnical resistance factors for pile analysis shown in Figure 408-1A shall be reduced by 20% if the number of piles in a pile group is 4 or less.
408-3.02 Types

408-3.02(01) Steel-Pipe Piles

LRFD 5.13.4.5, and 5.13.4.6 for a seismic zone, provide requirements for steel-pipe piles. Additional information appears in LRFD 6.9.5 and 6.12.2.3. The following will apply to steel-pipe piles.

1. **Usage.** These are best suited as friction piles. Depending on the subsurface conditions, the geotechnical engineer shall anticipate that such piles will achieve their capacity through a combination of skin friction and end bearing.

2. **Diameter.** This shall normally be 14 in. However, 16 in. is also used.

3. **Class of Concrete.** Pile shells shall be filled with class A concrete.

4. **Material Strength.** The specified 28-day compressive strength of concrete, $f'_c$, is 3500 psi. Pile shells shall have a minimum yield strength of 45 ksi for Grade 3.

5. **Nominal Axial Capacity and Wall Thickness.** The maximum nominal axial geotechnical capacity is provided in Figure 408-3A. The minimum wall thickness for a 14-in. pile shall be 0.25 in. For a 16-in. pile it shall be 0.312 in.

The designer is expected to perform a preliminary feasibility analysis where a higher bearing capacity is desired, and to notify the Office of Geotechnical Services of the desired bearing capacity prior to the beginning of the soils investigation, which is usually at the preliminary field check stage. This shall also be documented in the field check minutes.

If requested by the designer, the Office of Geotechnical Services may allow nominal geotechnical capacities greater than those shown in Figure 408-3A based on drivability and static-load-test requirement.

The designer shall use a single steel-shell wall thickness where the piling for the different substructure elements is in different nominal bearing-capacity ranges. The recommended wall thicknesses to be used shall be based on assurance of availability.

6. **Protection for Exposed Piles.** Only fusion-bonded, or powdered epoxy resin, epoxy coating shall be used. The epoxy coating shall be extended to 2 ft below the flow-line elevation. The epoxy coating is vulnerable to handling and driving. Because of the vulnerability of the epoxy coating near the flowline, reinforcing steel shall be included in the top part of the pile. See the INDOT *Standard Drawings.*
7. **Construction.** The designer shall consider the drivability of pipe piles.

### 408-3.02(02) Steel H-Piles

The following will apply to steel H-piles.

1. **Usage.** These are used either where the pile obtains most of its bearing capacity from end bearing on rock or as recommended in the Geotechnical Report. These are also used where there are hard layers that must be penetrated in order to reach an adequate point-bearing stratum.

   A steel H-pile can act efficiently as friction piling due to its large surface area. However, steel H piling shall not be used where the soil consists of only moderately dense material. In such conditions, it can be difficult to develop the friction capacity of the H-piles. Excessive pile length can result.

2. **Size.** Pile size designations may be HP10, HP12, or HP14. HP12 is used most often.

3. **Protection for Exposed Piles.** Only reinforced-concrete encasement shall be used. The concrete encasement shall be extended a minimum of 2 ft below the flow-line elevation or as specified in the Geotechnical Report.

4. **Steel Strength.** The yield strength, $F_y$, shall be a minimum of 50 ksi.

5. **Nominal Resistance and Bearing Capacity.** The maximum nominal structural resistance or bearing capacity for steel H-piles shall be as described in *LRFD* Section 6, and based on the structural resistance factors allowed as described in *LRFD 6.5.5*. However, this shall be less than or equal to the maximum nominal geotechnical resistance as shown in Figure 408-3A or in the geotechnical report.

### 408-3.03 Pile Length

#### 408-3.03(01) Minimum Length

The minimum pile length for an integral end bent shall be that shown in Figure 408-3B. If the minimum length shown in the figure cannot be attained, the designer shall provide calculations to support the use of a shorter length. A minimum core depth of 3 ft into scour-resistant rock shall be used. Pedestals shall not be used.
408-3.03(02) Tip Elevation for Friction Piles

The minimum pile-tip elevation shall be shown on the General Plan sheet’s elevation view, based on the scour requirements or the minimum pile-tip elevation requirements specified in Figure 408-3D.

408-3.03(03) Tip Elevation for Point-Bearing Piles

The approximate rock elevation shall be shown at each support location on the General Plan’s elevation view.

408-3.03(04) Pile-Tip Elevation for Billed Length

The minimum pile-tip elevation shown on the General Plan for a stream crossing is established to provide adequate penetration to protect against scour. It does not necessarily indicate the penetration needed to obtain the required bearing, which is shown only in the Geotechnical Report. Therefore, the billed length of piling shall be computed based on the lower of the minimum tip elevations shown on the General Plan or the estimated bearing elevation shown in the Geotechnical Report. For a spill-through end bent, the billed length of piling will be based upon the estimated bearing elevation shown in the Geotechnical Report.

408-3.03(05) Pile-Tip-Elevation Guidelines

Figure 408-3D lists pile-tip-elevation guidelines for setting piles for an interior substructure in a body of water. Minimum pile-tip elevations are not shown for the end bents unless recommended in the Geotechnical Report, e.g., due to voids in the bedrock or soft-soil strata located below where the pile capacity is reached.

408-3.04 Design Requirements

408-3.04(01) Battered Piles

Piles may be battered to a maximum of 4 vertical to 1 horizontal. For the outside row of piles in a footing, a batter shall be provided on alternating piles. Where closely-spaced battered piles are used, the pile layout shall be checked to ensure that battered piles do not intersect. Battered piles in a bent cap or a footing shall be centered on the bottom of the cap or footing. Therefore, the tops of such piles will be off-center.
Battered piles shall not be used where extensive downdrag load is expected because such a load causes flexure in addition to axial-force effects. Approximately one-half of the piles in a non-integral end bent cap shall be battered.

408-3.04(02) Spacing and Side Clearance

For H-piles, the enter-to-center spacing shall not be less than the greater of 3 ft or 3 times the pile width. For pipe piles, it shall be not less than the greater of 3.5 ft or 3 times the pile diameter or width. For H-piles used as friction piles in cohesive soil, the center-to-center spacing shall not be less than the greater of 2.5 ft or 3 pile widths. This requirement does not apply to piles driven into shale. A larger pile spacing 6 ft or 6 times the pile width is required. The distance from the side of a pile to the nearest edge of a footing shall be greater than 0.75 ft.

The maximum pile spacing shall not exceed 10 ft. However, if the cap or footing is properly designed for a larger spacing, this restriction need not apply. At a pile end bent, at least one pile shall be placed beneath each beam. The need for this requirement lessens with the increase in depth of the pile cap.

408-3.04(03) Embedment

LRFD 10.7.1.5 specifies that pile tops shall project not less than 1.5 ft into the footing after all damaged pile material has been removed. Embedment of piles into the stem of a wall pier with a single row of piles shall be a minimum of 5 ft.

408-3.04(04) Downdrag, DD, Load

Where a pile penetrates a soft layer subject to settlement, the force effects of downdrag or negative loading on the foundations shall be evaluated. These force effects are fully mobilized at relative movements of approximately 1/4 in. to 1/2 in. Downdrag acts as an additional permanent axial load on the pile. If the force is of sufficient magnitude, structural failure of the pile or a bearing failure at the tip is possible. At a smaller magnitude of downdrag, the pile can cause additional settlement. For piles that derive their resistance mostly from end bearing, the structural resistance of each pile must be adequate to resist the factored loads including downdrag. Battered piles shall be avoided where downdrag loading is possible due to the potential for bending of the pile. If the downdrag force is too large to be included as part of the pile load, the force shall be reduced or eliminated through the use of predrilled holes, special coatings, etc.
408-3.04(05) Uplift Forces

Uplift forces can be caused by lateral loads, buoyancy, or expansive soils. Piles intended to resist uplift forces shall be checked for resistance to pullout and structural resistance to tensile loads. The connection of the pile to the footing shall also be checked. Piles may be designed for uplift if specified in the Geotechnical Report. Pile-construction methods that require preboring, jetting, or spudding will reduce uplift capacity.

408-3.04(06) Laterally-Loaded Piles

The capacity of laterally-loaded piles shall be estimated according to approved methods. Investigations are waived if a sufficient number of battered piles are used to resist the lateral loads.

408-3.04(07) Reinforcing Steel for Pile Footing

Reinforcing steel shall be placed within a minimum of 4 in. cover from the bottom of the pile cap.

408-3.04(08) Pile Tips

To minimize damage to the end of the pile, cast-in-one-piece steel H-pile tips shall be used and shown on the General Plan sheet if recommended in the Geotechnical Report or if recommended during the Foundation Review.

408-3.04(09) Pile-Loads Table

The nominal resistance, $R_n$, and the nominal driving resistance, $R_{ndr}$, shall be shown in a table on the Soil Borings sheet. See Figure 408-3C. This information will help ensure that pile-driving efforts during the construction process will result in a foundation adequate to support the design loads. The information to be included in the table is as follows.

1. Factored Design Load, $Q_F$. This is the factored load from the design computations.
2. Factored Design Soil Resistance, $R_R$. This is the factored geotechnical soil resistance.
3. Resistance Factor, $\varphi_{dyn}$. This shall be based on the method of field construction selected to drive the piles. It shall be as described in Figure 408-1A unless otherwise instructed by the Office of Geotechnical Services.
4. Downdrag Load, DD. This is the downdrag load per pile.

5. Nominal Soil Resistance, \( R_n \). This is the long-term nominal pile-bearing resistance.

6. Downdrag Friction, \( R_{sdd} \). This is the skin friction that must be overcome during pile driving. It is obtained from the Geotechnical Report.

7. Scour-Zone Friction, \( R_{s scour} \). This is the skin friction due to scour that must be overcome during pile driving. It is obtained from the Geotechnical Report.

8. Relaxation, \( R_{relax} \). This is the relaxation in shale that must be overcome during pile driving. It is obtained from the Geotechnical Report.

9. Nominal Driving Resistance, \( R_{ndr} \). This is the nominal pile driving resistance including all geotechnical losses, or bearing.

10. Testing Method. This is the recommended method reported in the Geotechnical Report and as described in the INDOT Standard Specifications.

The nominal driving resistance, \( R_{ndr} \), shall be shown on the General Plan’s elevation view using a notation similar to the following: *Piling driven to ______ kip nominal driving resistance, \( R_{ndr} \).* The notation shall match the nominal driving resistance shown in the Soil Borings sheet table. It will not be necessary to show the nominal driving resistance on the other detail sheets.

H-piles are not to be driven to refusal. They are instead to be driven to the required ultimate bearing in bedrock. If the Geotechnical Report shows the elevation of the top of the bedrock, it shall be shown on the General Plan’s elevation view.

The information regarding piles shall be shown on the plans in the example format shown in Figure 408-3C.

**408-3.04(10) Pile-Load Test**

The method of pile testing will be specified in the Geotechnical Report. The designer shall contact the Office of Geotechnical Services in specifying the level of pile testing. The locations of the pile-load test shall be shown in the plans.
408-3.04(11) Pile Footing

The minimum thickness under a pier, frame bent, abutment, or retaining wall is 2.5 ft.

408-3.05 Pile Design for End Bent

Chapter 409 discusses the design of piles for an end bent.

408-4.0 DRILLED SHAFTS

Drilled shafts shall be designed as described in LRFD 5.7.4, 10.8

408-4.01 Usage

Drilled shafts may be considered where a deep foundation is required but piles are unsatisfactory due to obstructions, noise, vibrations, voids, or steeply-dipping rock. They can be an economical alternative to driven piles where the use of cofferdams is anticipated. They shall also be considered to resist large lateral or uplift loads where deformation tolerances are relatively small. Drilled shafts derive load resistance either as end-bearing shafts transferring load by tip resistance or as floating, or friction, shafts transferring load due to side resistance.

408-4.02 Socketed Shaft

A schematic drawing of a rock-socketed shaft is shown in Figure 408-4A. Where casing through the overburden soils is required, the socket diameter shall be at least 6 in. less than the inside diameter of the casing. Reinforcement is required within the rock sockets. For a shaft not requiring casing, the socket diameter may be equal to the shaft diameter.

408-4.03 Belled Shaft

Figure 408-4A also shows a belled section. In stiff, cohesive soil, an enlarged base, bell, or underarm may be used to increase the tip-bearing area to reduce unit end-bearing pressure or resistance to uplift. Where practical, extension of the shaft to a greater depth shall be considered to avoid the difficulty and expense of the belled shaft.
408-4.04  Column Design

Because soft soils provide sufficient support to prevent lateral buckling of the shaft, it may be designed according to the criteria for short columns described in LRFD 5.7.4.4. If the drilled shaft is extended above ground to form a pier or part of a pier, it shall be analyzed and designed as a column. The diameter of the column supported by a shaft shall be smaller than the diameter of the shaft. The effects of scour around the shaft shall be considered in the analysis. LRFD 10.7.3.13.4 provides criteria for determining the depth to fixity below the ground line for a shaft that extends for a portion of its length through water or air.

408-4.05  Reinforcement

Reinforcement shall satisfy the requirements of LRFD 10.8.3.9, 5.7.4, 5.10.11, and 5.13.4.6.

408-4.06  Acceptance Testing [Rev. May 2013]

For a drilled shaft foundation with an outside diameter larger than 5.0 ft, the designer should work with the Office of Geotechnical Services to develop a Unique Special Provision for acceptance of the drilled draft and consideration of mass pour concrete.

408-5.0  SCOUR AND FOUNDATION CONSIDERATION

An assessment shall be made of the bridge’s vulnerability to undermining due to potential scour. Section 203-3.0 discusses the hydraulic design of a bridge, including the hydraulic scour calculations that will significantly impact the design of its foundations. It discusses scour types, e.g., contraction, local, scour-resistant materials, analytical methods for scour evaluation, and countermeasures for alleviating potential scour. These calculations shall be approved by the Office of Hydraulics.

Bridge-foundation scour shall be designed for considering the 100-year event and the 500-year event that generate the maximum scour depth.

For scour conditions, the following resistance factors, $\varphi$, shall be used unless otherwise justified.

1. Design flood, 100-yr scour or overtopping flood: $\varphi = 0.70/0.55$.
2. Check flood, 500-yr scour or overtopping flood: $\varphi = 1.0$. 
3. Extreme Event Limit I, earthquake loading: $\varphi = 1.0$.

408-6.0 STRUCTURAL CONSIDERATIONS

Reference: *LRFD* 2.6.4.4.2, 3.7.5, 10.6, 10.7.3.13, 10.8.3.9.2

Scour is not a limit state in the context of *LRFD*. It is a change in foundation condition. All of the applicable *LRFD* limit states shall be satisfied for both the as-built and scoured bridge-foundation conditions.

The consequences of the change in foundation conditions resulting from the design flood for scour shall be considered at all applicable strength- and service-limit states. The design flood for scour is the more severe of the 100-year flood or an overtopping flood of lesser recurrence. The consequences of the change in foundation conditions resulting from the check flood for scour shall be considered at the Extreme Event limits. The check flood for scour shall not exceed the 500-year flood or an overtopping flood of lesser recurrence.

A spread footing shall be used only where the stream bed is extremely stable below the footing, and where the spread footing is founded at a depth below the maximum scour computed in Section 203-3.03(03). A footing may be founded above the scour elevation where it is keyed into non-erodible rock.

The pile cap for a deep foundation, driven-open-pile bent, or drilled shaft, shall be located such that the top of the cap is below the estimated contraction-scour depth. A lower elevation shall be considered where erosion or corrosion can damage the piles or shafts. Where the cap cannot be located below the maximum scour depth, soil loss surrounding the deep foundation results in piles or shafts with unbraced lengths. The unbraced length is equal to the length of the pile or shaft exposed by the scour, plus an estimated depth to fixity. The depth to fixity shall be determined as specified in *LRFD* 10.7.3.13 for driven piles, or *LRFD* 10.8.3.9.4 for drilled shafts. The piles or shafts exposed due to scour shall be designed structurally as unbraced-length columns in accordance with *LRFD* Section 5 for a concrete foundation, or *LRFD* Section 6 for a steel foundation. Unscoured piles or shafts can be considered in structural design as continuously-braced columns.

408-6.01 Pile Lateral Design

The Strength Limit state for lateral resistance is only structural, though the determination of pile fixity is the result of soil-structure interaction.
408-6.02 Battered Piles

Battered piles may be used to resist static lateral loads. However, battered piles shall not be used to resist dynamic lateral loads for a new-bridge foundation. Battered piles shall be avoided where extensive downdrag loads are expected.

408-6.03 Pile-Tip Elevations and Quantities

Pile-length quantities provided are based on the estimated tip elevation shown in the Geotechnical Report, or the depth required for design, whichever is greater. If the estimated tip elevation shown in the Geotechnical Report is greater than the design tip elevation, overdriving the pile will be required. The geotechnical engineer shall be contacted to evaluate driving conditions. A unique special provision may be required to alert the contractor of the additional effort required to drive these piles.

Minimum pile-tip elevations provided in the Geotechnical Report may need to be adjusted lower depending on the results of the lateral, axial, and uplift analysis. This becomes the minimum pile-tip-elevation requirement for the contract specifications. If adjustment in the minimum tip elevations is necessary, or if the required pile diameter is different than what was assumed for the Geotechnical Report, the Office of Geotechnical Services shall be informed so that pile drivability can be re-evaluated.

Lateral loading and uplift requirements can influence the number of piles required in the group if the capacity available at a reasonable minimum tip elevation is not adequate. This will depend on the soil conditions and the loading requirements.

The total factored pile axial loading shall be less than the geotechnical factored resistance, $R_n$, i.e., $\phi R_n$, for a simple situation with no geotechnical losses.

408-6.04 Other Pile-Design Considerations

1. **Pile Resistance.** Nominal pile resistances shall be provided according to LRFD design procedures. The resistance factor shall be provided according to the construction quality-control method recommended in the Geotechnical Report, i.e., dynamic formula, wave equation, pile-driving analyzer, etc. The pile types and sizes selected shall optimize the design, and hence take advantage of the available geotechnical and structural resistances.

2. **Downdrag Loads.** Pile downdrag loads, due to soil settlement other than that caused by dynamic, or seismic, loading, are added to the factored vertical dead loads on the
foundation in the Strength Limit state. Load Factors for downdrag loads will be those recommended by the designer. Transient loads shall not be included with the downdrag loads in either the Strength or Service Limit state calculations. Downdrag loads resulting from liquefaction or earthquake-induced soil settlement shall be considered in the Extreme Event limit state pile design. Downdrag loads resulting from soil liquefaction are different than those caused due to static loading. They shall not be combined in the Extreme Limit state analysis.

Where downdrag conditions exist, the pile must overcome the frictional resistance in the downdrag zone during installation. Such resistance shall not be included in the calculation of available factored resistance, since, after installation, it reverses over time becoming the static downdrag load.

3. **Uplift Capacity.** The uplift resistance is the same as the pile friction, or side, resistance. Resistance factors and factored uplift resistances will be provided in the Geotechnical Report. Friction resistance in a downdrag zone shall be considered available for uplift resistance. The geotechnical engineer shall be consulted regarding the ability of the piles to resist uplift forces under static or dynamic loading conditions.

4. **Minimum Pile-Tip Elevation.** Minimum pile-tip elevations are required to satisfy one or more of the design requirements as follows:

   a. lateral load;
   b. scour;
   c. liquefaction;
   d. uplift loads;
   e. settlement or downdrag; or
   f. required soil/rock bearing strata.

The required pile-tip elevations shall be shown on the plans and labeled as Required Pile Tip Elevations. Large lateral loads or other conditions can result in the need for additional piling, or larger piles, in order to satisfy lateral deflection criteria or other requirements. This may result in individual axial pile loads being much less than the maximum factored resistances available, either geotechnical or structural. If pile-tip elevations are needed to satisfy scour, uplift, or other requirements, the piles may need to be driven through dense materials to nominal resistances much higher than needed for supporting only the axial loads. The designer shall contact the geotechnical engineer to determine the most economical foundation design under these conditions.
5. **Pile-Group Settlement.** Pile-group settlement shall be compared to the maximum allowable settlement. Pile depths or layout shall be adjusted if necessary to reduce the estimated settlement to an acceptable level.


7. **Pile Spacing.** For H-piles in soil, use a minimum spacing of 3 times the pile width. For piles placed in shale bedrock, use 6 times the pile width.

8. **Pile-Tip Treatment.** Where pile-tip reinforcement is required, specify commercial-cast steel points or shoes, or conical pile tips.

9. **Pile-Foundation-Design Recommendations.** The geotechnical engineer will provide final geotechnical recommendations in the Geotechnical Report, or earlier in the design process as needed. The following minimum recommendations will be provided.

   a. **Pile Resistance.** The maximum nominal pile resistance, $R_{n_{max}}$, will be provided along with estimated pile lengths for one or more pile types. A modified $R_{n_{max}}$ value will be provided as necessary to account for scour or liquefaction conditions. The resistance factor will be provided along with the recommended method of construction control, i.e., dynamic formula, wave equation, etc. Downdrag loads, if present, will be provided along with an explanation of the cause of such loads. The depth or thickness of the downdrag zone will be provided.

   b. **Nominal Pile-Uplift Resistance.** This will be provided either as a function of depth or for a given pile length, typically associated with the minimum tip elevation. The pile-uplift resistance will be provided for normal static conditions and for a reduced-capacity condition such as scour or liquefaction. The resistance factor will be provided.

   c. **P-Y Curves.** The geotechnical engineer shall provide the geotechnical design parameters to develop p-y curves for lateral load analysis using a p-y analysis computer program. Two sets of data may be required, one for static conditions and one for dynamic, or liquefied-soil, conditions.
d. Required Pile-Tip Elevations. These will be provided along with an explanation of their basis. The tip elevations, or minimum pile embedments, shall be checked to see if they shall be modified to satisfy other design requirements, such as:

1. lateral loading requirements,
2. settlements,
3. liquefaction,
4. scour,
5. seismic and
6. axial uplift condition.

Changes to the recommended required tip elevations shall be reviewed by the geotechnical engineer.

e. The following geotechnical-related items will be provided, as necessary.

1. Wave Equation input, if drivability is performed;
2. pile tip treatment;
3. pile-restrike time after initial pile drive; or
4. recommendations regarding pile setup, relaxation, jetting, preboring, precoring, etc.

408-7.0 SEISMIC DESIGN

Seismic design of each foundation shall be in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design, except as otherwise indicated below.

The effects of wall inertia and probable amplification of active earth pressure or mobilization of passive earth masses due to an earthquake shall be considered.

The effects of geotechnical seismic hazard shall be considered. A liquefaction-potential assessment shall be conducted for soils that have been screened to be potentially liquefiable. The liquefaction potential shall be evaluated according to the procedures described in the IGM.

The effect of liquefaction-induced down drag can have an effect on the substructure. The additional load on the pile or column elements shall be considered in the design.

The designer shall contact the geotechnical engineer regarding minimization of the effect of soil liquefaction and adaptation of mitigation measures, such as ground modification, utilization of different substructure types, etc., to achieve an economical design.
The settlement of end bents and interior bents shall be considered along with the lateral motions accompanying the design event. The vertical settlement of the substructure can result in additional load on substructure elements, increased rotation of bearing elements, uplift of continuous structures, or excessive superstructure grade changes.

The geotechnical engineer is responsible for evaluating earthquake-induced soil settlement in accordance with the *IGM*.

The bridge-approach embankment is defined as 150 ft from the beginning or end of the bridge in the longitudinal direction. Slope failure of the embankment due to earthquake loads can lead to damage to end-bent components or complete bridge failure. Global stability of the embankment shall be determined utilizing the procedures prescribed in the *IGM*.

Piles may be used to resist both axial and lateral loads. The minimum depth of embedment, together with the axial and lateral pile capacities required to resist seismic loads, shall be determined by means of the design criteria established in the site investigation report. The nominal capacity of the piles shall be used in designing for seismic loads.

Where reliable uplift pile capacity from skin friction is present, and the pile/footing connection detail and structural capacity of the piles are adequate, uplift at a pile footing is acceptable, provided the magnitude of footing rotation will not result in unacceptable performance. Friction piles may be considered to resist intermittent, but not sustained, uplift. For seismic loads, uplift resistance of piles, and shafts, the resistance factor shall be taken as 0.80 or less. The uplift shall not exceed the weight of material, with buoyancy considered, surrounding the embedded portion of the pile.
<table>
<thead>
<tr>
<th>Condition / Resistance Determination Method</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Resistance of Single Pile in Axial Compression, $R_n$</td>
<td>Driving criteria established by INDOT Dynamic Formula at the end of initial drive condition (EOID) only.</td>
</tr>
<tr>
<td>Nominal Resistance of Single Pile in Axial Compression, $R_n$</td>
<td>Driving criteria established by dynamic test with signal matching at the beginning of re-drive (BOR) INDOT Dynamic Pile Load Test, PDA with CAPWAP</td>
</tr>
<tr>
<td>Nominal Resistance of Single Pile in Axial Compression, $R_n$</td>
<td>Driving criteria established by Static Load Test in combination with INDOT Dynamic Pile Load Test, PDA with CAPWAP</td>
</tr>
<tr>
<td>For all other conditions, as described in LRFD 10.5</td>
<td>---</td>
</tr>
</tbody>
</table>

Notes:

1. The resistance factors shown above are higher than those included in the AASHTO LRFD Specifications.

2. The resistance factors were calibrated based on data developed under the ASD methodology. Since no reliability analysis has been performed on the test data used to develop these factors, the actual reliability obtained by using the factors is unknown, and can be less than what was used to develop the resistance factors included in the AASHTO LRFD Specifications.

3. The bases for the calibrated resistance factors are as follows:
   a. INDOT Final Report of the Pile Driving Analysis Demonstration Projects, November 1995;
   b. INDOT’s long-term successful use of the resistance factors since being adopted as standard practice in 1996;
   c. exclusive use of the Gates driving formula; and
   d. INDOT Standard Specifications Section 701.05(a), which requires a re-strike at each bent or pier.

RESISTANCE FACTORS FOR DRIVEN PILES

Figure 408-1A
TO:
Manager, Office of Geotechnical Services

FROM:

Route:
Structure No.:
Des. No.:
Construction Project No.:
Over:

It is recommended that the following foundations be used for the structure identified above.

<table>
<thead>
<tr>
<th>Support</th>
<th>No. 1</th>
<th>No. 2</th>
<th>No. 3</th>
<th>No. 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Size, incl. Shell Thickness</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Factored Design Load, $Q_F$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nominal Design Load, $Q_N$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min. Pile Tip Elev. for Scour</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile Tips</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td></td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Bottom of Footing Elevation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top of Footing Elevation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The structure is on piles, so the Summary of Pile Loading for Geotechnical Testing is completed as shown below.  Yes ☐ No ☐ n/a ☐
### SUMMARY OF PILE LOADING FOR GEOTECHNICAL TESTING

<table>
<thead>
<tr>
<th>Support</th>
<th>No. 1</th>
<th>No. 2</th>
<th>No. 3</th>
<th>No. 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile Size, Type, and Grade</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Factored Design Load, $Q_F$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Factored Design Soil Resistance, $R_F$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance Factor $\phi_{dyn}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Downdrag Load, $DD$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nominal Soil Resistance, $R_n$ (kip) *</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Downdrag friction, $R_{s,dd}$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Scour Zone Friction, $R_{s,scour}$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Relaxation of Tip in Shale, $R_{relax}$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nominal Driving Resistance, $R_{ndr}$ (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Testing Method</td>
<td>Standard Specifications Section 701.05( )</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The MSE-wall or modular-block-wall factored applied pressure shown on the wall envelope is less than the factored bearing resistance. Yes ☐ No ☐ n/a ☐

Notes:

* In Calculation of $DD$, $\gamma_p = 1.4$

\[
\frac{Q_F}{Q_F^{max}} \leq \frac{Q_F}{R_F}
\]

To calculate $R_n$:

\[
R_n = \frac{R_F + \gamma_p \Phi D}{\phi_{dyn}}
\]

To calculate $R_{ndr}$:

\[
R_{ndr} = R_n + \text{(Geotechnical Losses)} \ (R_{s,scour} \text{ or } R_{s,dd} \text{ or } R_{s,liq})
\]

Other:

Approved by: ________________________________ Date: ______________________

(Signed) Geotechnical Engineer

Reviewed by: ________________________________ Date: ______________________

(Signed) Reviewer, ☐ INDOT ☐ Consultant,

Reviewed by: ________________________________ Date: ______________________

(Signed) Director, Bridge
TIMES STANDARD TENSION DEVELOPMENT LENGTH (MIN.)

A = STANDARD TENSION DEVELOPMENT LENGTH (MIN.)
B = 1 ½ TIMES STANDARD TENSION DEVELOPMENT LENGTH (MIN.)

TYPICAL FOR WING
OR STANDARD ABUTMENT

BAR DEVELOPMENT LENGTH

Figure 408-2A
Notes:

1. The resistance factor, $\Phi_{dyn}$, should be used for calculating a pile’s geotechnical capacities by means of the field methods. For PDA, $\Phi_{dyn} = 0.70$. For Gates’ formula, $\Phi_{dyn} = 0.55$.

2. The maximum nominal capacity and the maximum factored capacity should be dependent on drivability and shell thickness. For a pipe pile of outside diameter 14 in., the minimum shell thickness should be 0.25 in. For a pipe pile of outside diameter 16 in., the minimum shell thickness should be 0.312 in.

3. $R_{n_{max}}$ should be taken from the above table. From this value, the maximum factored design soil resistance, $R_{R_{max}}$, should be back-calculated with applicable geotechnical losses.

4. The maximum nominal driving resistance, $R_{ndr_{max}}$, should be calculated from $R_{n_{max}}$ with the applicable geotechnical losses included.

5. Factored design load, $Q_F$, should be less than the factored design soil resistance, $R_R$. $R_R$ should be less than or equal to $R_{R_{max}}$. Nominal soil resistance, $R_n$, should be less than or equal to $R_{n_{max}}$. Nominal driving resistance, $R_{ndr}$, should be less than or equal to $R_{ndr_{max}}$.

### MAXIMUM NOMINAL SOIL RESISTANCE

**Figure 408-3A**

<table>
<thead>
<tr>
<th>Pile Type</th>
<th>Section Area, in.²</th>
<th>Maximum Nominal Soil Resistance, $R_{n_{max}}$, kip</th>
</tr>
</thead>
<tbody>
<tr>
<td>HP 10x42</td>
<td>12.4</td>
<td>341</td>
</tr>
<tr>
<td>HP 10x57</td>
<td>16.8</td>
<td>462</td>
</tr>
<tr>
<td>HP 12x53</td>
<td>15.5</td>
<td>426</td>
</tr>
<tr>
<td>HP 12x63</td>
<td>18.4</td>
<td>506</td>
</tr>
<tr>
<td>HP 12x74</td>
<td>21.8</td>
<td>600</td>
</tr>
<tr>
<td>HP 12x84</td>
<td>24.6</td>
<td>677</td>
</tr>
<tr>
<td>HP 14x73</td>
<td>21.4</td>
<td>589</td>
</tr>
<tr>
<td>HP 14x89</td>
<td>26.1</td>
<td>718</td>
</tr>
<tr>
<td>HP 14x102</td>
<td>30.0</td>
<td>825</td>
</tr>
<tr>
<td>HP 14x117</td>
<td>34.4</td>
<td>946</td>
</tr>
<tr>
<td>Pipe pile, 14 in.</td>
<td>n/a</td>
<td>420</td>
</tr>
<tr>
<td>Pipe pile, 16 in.</td>
<td>n/a</td>
<td>480</td>
</tr>
<tr>
<td>Pile Size</td>
<td>Minimum Length, ft</td>
<td></td>
</tr>
<tr>
<td>-----------</td>
<td>-------------------</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Clay</td>
<td>Sand</td>
</tr>
<tr>
<td>HP 10</td>
<td>30</td>
<td>25</td>
</tr>
<tr>
<td>HP 12</td>
<td>35</td>
<td>25</td>
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<td>HP 14</td>
<td>40</td>
<td>30</td>
</tr>
<tr>
<td>CFT 14</td>
<td>50</td>
<td>35</td>
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</tbody>
</table>

MINIMUM PILE LENGTH FOR INTEGRAL END BENTS

Figure 408-3B
<table>
<thead>
<tr>
<th>Support</th>
<th>No. 1</th>
<th>No. 2</th>
<th>No. 3</th>
<th>No. 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile Size, Type, and Grade</td>
<td>HP 12 x 53</td>
<td>HP 12 x 53</td>
<td>HP 12 x 53</td>
<td>HP 12 x 53</td>
</tr>
<tr>
<td>Factored Design Load, $Q_F$ (kip)</td>
<td>120</td>
<td>160</td>
<td>200</td>
<td>120</td>
</tr>
<tr>
<td>Factored Design Soil Resistance, $R_R$ (kip)</td>
<td>120</td>
<td>160</td>
<td>200</td>
<td>120</td>
</tr>
<tr>
<td>Resistance Factor $\phi_{dyn}$</td>
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<td>0.55</td>
<td>0.55</td>
<td>0.55</td>
</tr>
<tr>
<td>Downdrag Load, $DD$ (kip)</td>
<td>12</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Nominal Soil Resistance, $R_n$ (kip) *</td>
<td>240</td>
<td>290</td>
<td>363</td>
<td>218</td>
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<tr>
<td>Downdrag friction, $R_{s,dd}$ (kip)</td>
<td>12</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Scour Zone Friction, $R_{s,scour}$ (kip)</td>
<td>0</td>
<td>7</td>
<td>7</td>
<td>0</td>
</tr>
<tr>
<td>Relaxation in Shale (kip)</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Nominal Driving Resistance, $R_{ndr}$ (kip)</td>
<td>311</td>
<td>348</td>
<td>421</td>
<td>268</td>
</tr>
<tr>
<td>Testing Method</td>
<td>Standard Specifications Section 701.05(a)</td>
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<td></td>
</tr>
</tbody>
</table>

**SAMPLE SUMMARY OF PILE LOADING**

*Figure 408-3C*
**PILE TIP ELEVATION GUIDELINES**  
(For Body of Water)

**Figure 408-3D**
DRILLED SHAFTS

Figure 408-4A
CHAPTER 409

Abutment, Bent, Pier, and Bearing

<table>
<thead>
<tr>
<th>Design Memo</th>
<th>Revision Date</th>
<th>Sections Affected</th>
</tr>
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<tbody>
<tr>
<td>13-11</td>
<td>May 2013</td>
<td>409-7.03(03), Figure 409-7F</td>
</tr>
<tr>
<td>17-03</td>
<td>Mar. 2017</td>
<td>409-2.04(02), 409-3.03, Figure 409-2G</td>
</tr>
<tr>
<td>19-03</td>
<td>May 2019</td>
<td>409-2.04(01), Figure 409-2F (deleted)</td>
</tr>
</tbody>
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CHAPTER 409

ABUTMENTS, BENTS, PIERS, AND BEARINGS

References shown following section titles are to the AASHTO LRFD Bridge Design Specifications.

LRFD Section 11 discusses the design requirements for bents, piers, and abutments. Section 14 discusses the design requirements for bearings. This Chapter describes supplementary information on the design of these structural components. See Chapter 402 for more information on substructure types and their selection.

409-1.0 LIMIT STATES, RESISTANCE FACTORS, AND LOADS

409-1.01 Limit States

The design of abutments, bents, piers, and bearings shall be in accordance with LRFD.

409-1.01(01) Service-Limit State

Abutment, bents, and piers shall be investigated for excessive vertical and lateral displacement, and overall stability, at the service-limit state.

LRFD 10.6.2.2, 10.7.2.2, and 10.8.2.2 apply to the investigation of vertical movements.

409-1.01(02) Strength-Limit State

Abutments, bents, and piers shall be investigated at the strength-limit state using LRFD Equation 1.3.2.1-1 for bearing resistance failure, lateral sliding or excessive loss of base contact, pullout failure of anchors or soil reinforcements, or structural failure.
409-1.01(03) Extreme-Event-Limit State

Substructure elements for seismic loading shall be designed in accordance with AASHTO Guide Specifications for LRFD Seismic Bridge Design. For all other extreme events, substructure elements shall be designed in accordance with AASHTO LRFD Bridge Design Specifications.

409-1.02 Resistance Factors

For abutments, bents, and piers, see LRFD 11.5.6. The resistance factor for bearings shall be taken as 1.0.

409-1.03 Load Combinations and Load Factors

See LRFD 3.4.1 and 11.5.5.

409-2.0 INTEGRAL END BENT [REV. OCT. 2012, SEP. 2016]


An integral end bent eliminates the expansion joint in the bridge deck, which reduces both the initial construction costs and subsequent maintenance costs.

Integral end bents shall be used for a new structure in accordance with the geometric limitations provided in Figure 409-2A. Minimum support-length requirements need not to be investigated for an integral end bent bridge. An integral structure of length of 500 ft or less will not require seismic analysis, provided the end bent is detailed in accordance with the information provided in this chapter. An integral structure of 500 ft or longer located in an area in a seismic design category greater than A will be analyzed using elastic dynamic analysis.

For additional information and research supporting INDOT’s integral end bent design philosophy, see the following publications:

1. Frosch, R.J., V. Chovichien, K. Durbin, and D. Fedroff. Jointless and Smoother Bridges: Behavior and Design of Piles. Publication FHWA/IN/JTRP-2004/24. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2006. This study investigates the fundamental principals affecting the integral end bent, gives recommendations concerning minimum pile depths, and recommends the limits of use be extended to 500 feet.
2. Frosch, R.J., Kreger, M.E., and A.M. Talbott. *Earthquake Resistance of Integral Abutment Bridges*. Publication FHWA/IN/JTRP-2008/11. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2009. This study investigates the seismic resistance of the integral abutment (end bent).

3. Frosch, R.J. and M.D. Lovell. *Long-Term Behavior of Integral Abutment Bridges*. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2011. This study extends the previous two studies to further investigate skew and detailing of the integral abutment (end bent).

409-2.02 Materials

Class C concrete and epoxy coated reinforcing bars are required.

The wingwalls concrete shall be Class C.

409-2.03 Design Criteria

Although each end of the superstructure is monolithically attached to an integral end bent, the rotation permitted by the piles is sufficiently high, and the attendant end moment is sufficiently low, to justify the assumption of a pinned-end condition for design. The following design assumptions shall be considered.

409-2.03(01) Ends

The ends of the superstructure are free to rotate and translate longitudinally.


The restraining effect of passive earth pressure behind the end bent may be neglected in considering superstructure longitudinal force distribution to the interior piers. Alternatively, the effect of passive earth pressure behind the end bent may be considered by distributing the longitudinal forces between the interior supports, end bent supports, and the soil behind the end bent.
409-2.03(03) Interior Pile Bent

All longitudinal forces from the superstructure shall be distributed among the interior supports, end bents, and soil behind the end bents based on relative stiffness in designing an interior pile bent or a thin-wall pier on a single row of piles.

409-2.03(04) Shear and Moment

Force effects in the cap beam may be determined on the basis of a linear distribution of vertical pile reactions. For minimum reinforcement, the cap shall be treated as a structural beam.

409-2.04 Design Requirements [Rev. May 2019]


The following requirements must be satisfied.

1. Backfill. Each integral end bent for a beam- or girder-type superstructure should be backfilled with aggregate for end bent backfill. Each end bent for a reinforced concrete slab bridge should be backfilled with removable flowable backfill. The INDOT Standard Drawings series E 211-BFIL provides backfill details for both concrete slab, beam, and girder structures.

2. Reinforced Concrete Bridge Approach (RCBA). A reinforced concrete bridge approach is utilized to span over the backfill placed behind a newly constructed end bent or mudwall. The RCBA should be anchored to the end bent with epoxy coated #5 threaded tie bar assemblies, spaced at 2’-0” centers. Two layers of polyethylene sheeting shall be placed between the reinforced-concrete bridge approach and the subgrade. INDOT Standard Drawings series E 609-RBCA provides additional details.

   Where an expansion joint or mudwall is used, the threaded tie bar anchoring system may not be practical and an alternate connection may be considered.
3. **Terminal Joint.** A bridge approach joint should be included where the approach pavement is PCCP. For a structure with an expansion length less than 300 ft, a terminal joint of 2 ft width, as shown on the INDOT *Standard Drawings* series E 503-BATJ should be placed at the end of the reinforced concrete bridge approach. An expansion joint should be considered for an integral structure having an expansion length from 300 ft to 500 ft to the terminal joint. A steel finger plate expansion joint is required for an integral structure with an expansion length greater than 500 ft to the terminal joint. The expansion joint shall be designed in accordance with the *LRFD*.

4. **Wingwalls Configuration.** Wingwalls shall extend parallel to the centerline of roadway. This configuration reduces the loads imposed upon the bridge structure due to passive earth pressure from the end-bent backfill. See Figure **409-5A** for suggested wingwall dimensioning details. The minimum thickness of a wingwall used with an integral end bent shall be 1 ft. The wingwall length shall not be greater than 10 ft. A longer wingwall will require additional analysis.

5. **Wingwall Connection.** Force effects in the connection between the wingwall and cap, and in the wingwall itself, shall be investigated, and adequate reinforcing steel shall be provided.

6. **Interior Diaphragms for Steel Structure.** Where steel beams or girders are used, an interior diaphragm shall be placed within 10 ft of the end support to provide beam stability prior to and during the deck pour.

7. **Intermediate Pier Details for Integral Structure Located in Seismic Area with Seismic Design Category Greater than A.** Intermediate piers should include concrete restrainers as shown in Figure **409-2B**.

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[Itemized plan detail information was removed from this section and added to new section 409-2.04(03). Remaining information is unchanged]
Regardless of the method used, the end bent shall be in accordance with the following.

1. **Width.** The width shall not be less than 2.5 ft. The width shall consider
   a. Beam extension and concrete cover as noted below;
   b. clearance from the side of any pile to the nearest vertical face of the pile cap. The minimum distance should not be less than 9 in. (*LRFD* 10.7.1.2), per plan pile locations;
   c. pile misalignment per the *Standard Specifications* without requiring modifications to the end bent reinforcing or spiral reinforcing.

2. **Depth.** The depth from the bottom of the beam or girder to the bottom of the integral end bent should not exceed 6’- 0”. Use of a deeper end bent must be approved by the Bridge Design Division.

3. **Cap Embedment.** For all span lengths, the pile shall be embedded 2 ft into the cap.

4. **Spiral Reinforcement.** For a bridge with an expansion length greater than 250 ft, the embedded portion of the pile shall be confined with spiral reinforcement. See Figure 409-2E for spiral reinforcement details.

5. **Beam Attachment.** The beams shall be physically attached to the piling if using Method A, or to the cast-in-place cap if using Method B.

6. **Beam Extension.** The beams shall extend at least 1.75 ft into the bent, as measured along the centerline of the beam.

7. **Concrete Cover.** Concrete cover beyond the farthest-most edge of the beam at the rear face of the bent shall be at least 4 in. This minimum cover shall also apply to the pavement-ledge area. The top flanges of structural-steel or prestressed concrete I-beams may be coped to satisfy this requirement. Where the 4-in. minimum cover cannot be maintained within a 2.5-ft cap, the cap shall be widened.
8. **Stiffener Plates.** Structural-steel members shall have stiffener plates welded to both sides of their webs and to the flanges over the supports to anchor the beams into the concrete.

9. **Reinforcement through the Webs of Beams.** A minimum of three holes shall be provided through the webs of steel members near the front face of the bent for #6 bars to be inserted through. Two holes shall be provided through prestressed concrete I-beam webs near the front face of the bent, to allow #6 bars to be inserted to further anchor the beam to the cap. Box beams shall have two threaded inserts placed in each side face for anchorage of #7 threaded bars.

10. **End-Bent Reinforcement.** The minimum size of stirrups shall be #6 spaced at a maximum of 1’-0”. Longitudinal cap reinforcement shall be at least #7 at 1’-0” maximum spacing along both faces of the bent. All reinforcing steel shall be epoxy coated.

11. **Corner Bars.** Corner bars shall extend from the rear face of the cap into the top of the deck at not more than 1’-0” spacing as shown in Figures 409-2B and 409-2C. The figures show suggested details for an integral end bent with a structural-members bridge. Other reinforcement and connection details shall be used where they are structurally sound and afford an advantage if compared to that shown in the figures. See Figures 409-2B and 409-2C for drainage-pipes placement behind an end bent. See LRFD 11.4.1 and 11.6.6 for additional drainage information.

12. **MSE Wall.** If placed behind an MSE retaining wall, the end bent should be configured as shown in Figure 409-2G. See LRFD 11.10.8 and Section 410-5.0(07) of this manual for MSE wall drainage information.

### 409-3.0 SEMI-INTEGRAL END BENT

#### 409-3.01 General

Semi-integral end bents shall be considered if integral end bents are not practical or feasible. For a skew angle of greater than 30 deg or an expansion length of 250 ft or longer, twisting or racking of the bridge shall be investigated.

Minimum support-length requirements shall be investigated for semi-integral end bent Method 2.
**409-3.02  Materials**

Semi-integral end bents and wingwalls will require the use of class C concrete and epoxy-coated reinforcing steel.

**409-3.03  Details [Rev. Mar. 2017]**

Figure 409-3A shows details for Method 1. Figure 409-3B shows details for Method 2. Figure 409-3C shows details for the joint-protection sheeting. Figure 409-3D shows details pavement-ledge details for integral and semi-integral end bents. All applicable information shown in the figures shall be shown on the plans.

Wingwalls details are similar to those for an integral end bent except for the connection method. The wingwall is connected to the bent below the seat elevation. See Figure 409-5A for suggested wingwall-dimensioning details. The minimum wingwall thickness of a wingwall shall be 1 ft.

See *LRFD* 11.4.1 and 11.6.6 for additional drainage information.

If placed behind an MSE retaining wall, the end bent should be configured as shown in Figure 409-2G. See *LRFD* 11.10.8 and Section 410-5.0(07) of this manual for MSE wall drainage information.

**409-4.0  PILES, DESIGN CONSIDERATIONS, AND DETAILS FOR END BENTS**

**409-4.01  Piles**

The following criteria apply to piling for an integral, semi-integral, or non-integral end bent.

**409-4.01(01)  Pile Spacing**

Pile spacing shall not exceed 10 ft. If the cap is properly analyzed and designed as a continuous beam, this restriction need not apply. If practical, one pile may be placed beneath each girder. See Chapter 408 for minimum pile spacing. For an integral end bent within the limits defined in Figure 409-2A, or for a non-integral end bent, the piles are considered to be free-ended and capable of resisting only horizontal and vertical forces.
409-4.01(02) Number of Piles

See Chapter 408 for the minimum number of piles.

409-4.01(03) Cap Overhang

The minimum cap overhang shall be 1.5 ft measured from centerline of pile.

409-4.01(04) Pile Overload

If an individual pile is overloaded due to the maximum beam or girder loads, the overload amount may be considered equally distributed to the two adjacent piles provided that this distribution of overloads does not cause either of the adjacent piles to exceed its allowable bearing capacity. This distribution of overload will be permitted only if the allowable bearing value for the pile is based upon the capacity of the soils and not on the structural strength of the pile, and if the pile cap has enough beam strength to distribute the overload to the adjacent piles.
409-4.01(05) Live-Load Distribution

The wheel loads located out in the span shall be distributed to the substructure in accordance with the live-load distribution factors shown in LRFD 4.6.2.2.2. For wheels located over the support, a simple-span transverse distribution shall be used.

409-4.02 Design Considerations

409-4.02(01) Integral End Bent

The following criteria apply specifically to piles and loads.

1. **Loads and Forces.** Only vertical loads shall be considered in designing end-bent piling for a structure which satisfies the requirements provided in Figure 409-2A. Force effects in the end-bent piles due to temperature, shrinkage, and creep may be neglected.

   An alternative analysis shall be used if the criteria in Figure 409-2A are not satisfied. The analysis to be made is as follows.

   a. The point of zero movement shall be established by considering the elastic resistance of all substructure elements, bearing devices, and passive earth pressure.

   b. The effects of creep, shrinkage, and temperature shall be considered.

   c. Movement at a point on the superstructure shall be taken as being proportional to its distance to the point of zero movement.

   d. Lateral curvature of the superstructure may be neglected if it satisfies LRFD 4.6.1.2.

   e. Vertical force effects in the end-bent piles shall be distributed linearly with load eccentricities properly accounted for.

   f. Lateral soil resistance shall be considered in establishing force effects and buckling resistance of piles. Force effects shall be combined in accordance with LRFD 3.4.1.
2. **Pile Type.** Only steel H-piles or pipe piles shall be used with an integral end bent. Steel H-pile webs shall be placed perpendicular to the centerline of the structure to minimize flexural forces in the piling. All end bent piling shall be driven vertically. Only one row of piling is permitted.

### 409-4.02(02) Semi-Integral End Bent

1. **Pile Spacing.** The minimum pile spacing shall be as specified in Chapter 408. For a structure with deep girders, two rows of piles with staggered pile spacing shall be considered.

2. **Batter.** Up to one-half of the piles may be battered to increase the resistance to horizontal movement of the structure.

3. **Overturning.** If the pile spacing is less than 10 ft and one-half of the piles are battered, overturning need not be investigated. If less than one-half of the piles are battered, or if the pile spacing is 10 ft or greater, the stability due to overturning pressures shall be investigated.

### 409-4.02(03) Wingwalls

With respect to a spill-through end bent, the following applies to wingwalls.

1. **Usage.** Each structural-steel or prestressed-concrete beam bridge requires wingwalls. A reinforced-concrete slab bridge usually does not require wingwalls.

2. **Dimensions.** Wingwalls shall be of sufficient length and depth to prevent the roadway embankment from encroaching onto the stream channel or clear opening. The slope of the fill shall not be steeper than 2:1, perpendicular to the skew. Wingwall lengths can be established on this basis. For more information, see *LRFD* 11.6.1.4 for more information. See Figure 409-5A for suggested wingwall-dimensioning details. The minimum thickness of a wingwall used with an end bent shall be 1 ft.

3. **Pile Support.** If the wingwalls for a non-integral or semi-integral end bent have a total length of more than 10 ft, pile support shall be investigated. Pile-supported wings shall not be used with an integral end bent.
4. **Design.** A non-pile-supported wingwall shall be designed as a horizontal cantilevered wall. Because the wingwalls are rigidly attached to the remainder of the bent, the bent is restrained from deflecting except laterally as a unit. Due to the lack of the usual retaining-structure rotation, the active-soil-pressure condition cannot develop, and the design soil pressure must be increased to a value between the active and at-rest condition. Therefore, the horizontal earth pressure to be used in design shall be equal to 150% of the value determined assuming an active-soil condition. Live-load surcharge shall be added to the soil loads in accordance with *LRFD* 3.11.6.2.

**409-4.03 Details**

**409-4.03(01) Construction Joint  [Rev. Oct. 2012]**

The following applies to a construction joint at a spill-through end bent.

1. **Type.** Construction joint type A shall be used for each horizontal construction joint. See the INDOT *Standard Drawings*.

2. **Integral.** See Figures 409-2C and 409-2D for construction-joint use at an integral end bent.

**409-4.03(02) Longitudinal Open Joint**

If the bridge deck includes a longitudinal open joint, an expansion joint shall also be placed in the end bent. Also, flashing shall be placed behind the joint in the end bent. See the INDOT *Standard Drawings*.

**409-5.0 CANTILEVER ABUTMENT AND WINGWALLS**

**409-5.01 General  [Rev. Oct. 2012]**

See Chapter 402 and *LRFD* 11.6 for more information on the selection and design of abutments.

An abutment functions as both an earth-retaining and vertical-load-carrying structure. A parapet abutment is designed to accommodate thermal movements with strip-seal expansion devices between the concrete deck and abutment end block. An integral end bent shall be designed to accommodate movements at the roadway end of the approach panel.
A mechanically-stabilized-earth-wall bridge abutment placed adjacent to a roadway need not to be checked for vehicle-collision forces as described in *LRFD* 3.6.5. However, if the wall must be placed inside the clear zone, roadside safety shall be addressed.

A mechanically-stabilized-earth-wall bridge abutment placed adjacent to a railroad track shall be in accordance with Section 409-6.03(03).

For soil conditions or bridge geometric dimensions not suitable for a spill-through end bent or mechanically-stabilized-earth abutment, an abutment with wingwalls of the cantilever type shall be used. Such a cantilever structural unit shall be founded on a spread footing, drilled shafts, or a driven-pile footing with a minimum of two rows of piles. The front row of piles may be battered a maximum of 1:4 to provide additional horizontal resistance.

409-5.02 Materials

For a mechanically-stabilized-earth abutment, the required materials are described in the INDOT Standard Specifications.

For an abutment or wingwall, class A concrete shall be used for all components above the footing. Class B concrete shall be used in the footing.

If an expansion joint is located directly over the abutment cap, all reinforcement in the abutment wall shall be epoxy coated.

409-5.03 Design Considerations

409-5.03(01) Integral End Bent

An integral end bent shall be designed to resist and absorb creep, shrinkage, and thermal deformations of the superstructure. Movement calculations shall consider temperature, creep, and long-term prestress shortening in determining potential movements. See *LRFD* 11.6.1.3 for more information.

409-5.03(02) Expansion Joints

Vertical expansion joints shall be considered for an abutment whose width exceeds 90 ft, as indicated in *LRFD* 11.6.1.6.
**409-5.03(03) Abutment-Wingwall Junction**

The junction of the abutment wall and wingwall is a critical design element, requiring the considerations as follows.

1. If the abutment wall and wingwall are designed using active earth pressure, the two elements shall be separated by a filled expansion joint of \( \frac{1}{2} \)-in. width to permit the expected deformations. If the abutment is designed using at-rest earth pressure, an expansion joint between the wingwall and abutment wall is not required.

2. If the wingwall is tied to the abutment wall with no joint, all horizontal steel reinforcement shall be developed into both elements such that full moment resistance can be obtained.

**409-5.03(04) Stem Batter**

Where a batter is used, it shall range from 1:10 through 1:15.

**409-5.03(05) Concrete Cover**

See *LRFD* Table 5.12.3-1 for more information.

**409-5.03(06) Keyway**

A keyway shall be used in each vertical expansion joints. See the INDOT *Standard Drawings* for details.

**409-5.03(07) Backfill**

The abutment and wingwalls shall be backfilled with structure backfill. The neat-line limits shall be shown on the Layout sheet.
409-5.03(08) Toe

The fill on the toe of footing shall be ignored in investigating sliding resistance.

409-5.03(09) Soil Weight

Only the weight of the soil which is vertically above the heel of the footing shall be included in the overturning-stability analysis and the structural design of the footing.

409-5.03(10) Minimum Footing Thickness

The minimum thickness shall be 1.5 ft.

409-5.03(11) Piles

A footing on piles shall be analyzed to consider the structural contribution of the concrete below the tops of the piles. Bottom-mat reinforcement shall be placed 4 in. above the bottom of the footing.

The pile type shall be based on the recommendations provided in the geotechnical report. Pile spacing shall be as described in Chapter 408. Pile embedment into the footing shall be at least 1.5 ft.

409-5.03(12) Loads

An abutment stem shall be designed for the imposed gravitational loads, weight of the stem, and horizontal loads. The static earth pressure shall be determined in accordance with LRFD 3.11 and 11.6.1.2. Passive earth pressure shall not be assumed to be generated by the prism of earth in front of the wall.

409-5.03(13) Details

Figure 409-5A shows typical wingwall details for integral, semi-integral, or non-integral end bents. Figure 409-5E illustrates the preferred methods for determining the geometrics for a flared wingwall for a square structure. Figures 409-5C and 409-5F illustrate this for a structure
skewed to the right. Figures 409-5D and 409-5G illustrate this for a structure skewed to the left. Figure 409-5B provides an example for determining a flared-wing length and elevations.

Figure 409-5H provides suggested typical abutment details.

409-5.03(14) Drainage

Positive drainage shall be provided behind each abutment or wingwall. See the INDOT Standard Drawings for a weephole detail. See LRFD 11.6.6 for more information. Drains shall be located in an abutment or wingwall as follows.

1. **Abutment with Wingwalls of 15 ft or Shorter.** Drains shall be spaced at 12 ft maximum in the abutment. Drains shall be omitted from the wingwalls.

2. **Abutment with Wingwalls of Longer Than 15 ft.** Drains shall be spaced at 12 ft maximum in the abutment, with a 12-ft maximum distance from the ends of the wingwalls.

3. **Location of Drain Outlet.** The outlet shall be placed 1 ft above the low-water elevation or the proposed ground-line elevation.

409-5.03(15) Construction Joints

A construction joint type A shall be used for all horizontal construction joints in both the abutment and wingwalls. See the INDOT Standard Drawings. Vertical construction joints shall be placed as follows.

1. **Abutment.** Preferably at 30 ft center to center, with a maximum of 40 ft.

2. **Wingwall of 20 ft or Longer.** At 20 ft center-to-center and one batter face cut.

3. **Wingwall Shorter than 20 ft.** In the abutment section so that the combined length of wingwall and abutment between joints is approximately 20 ft.

4. **Either the Wingwall or the Abutment.** Not less than 1.5 ft from the intersection of batter faces at the top of the footing.

Joints shall not be placed under bridge bearing areas.
The horizontal reinforcing steel shall continue through the construction joint. Vertical bars shall be placed at a minimum of 3 in. from the centerline of the joint.

409-6.0 INTERIOR SUPPORTS

409-6.01 General

409-6.01(01) Types of Interior Supports

1. Extended-Pile or Drilled-Shaft Bent. The economy of a substructure can be enhanced under certain conditions by means of extending a deep foundation, such as a single row of driven piles or drilled shafts above ground level to the superstructure. An extended-pile bent may be of the integral type or the non-integral type. See Figure 409-6A for details.

2. Stem-Type Pier. The types of stem piers are as follows.
   a. Single-Wall. This is a relatively thin wall, set on a single row of piles, a spread footing, or a pile cap with multiple rows of piles. The single-wall is most suitable if its structural height is less than 20 ft. See Figure 409-6B for a wall pier on a single row of piles.
   b. Hammerhead. For a larger structural height or pier width, a hammerhead pier, either with a rectangular or rounded stem, is often more suitable. See Figure 409-6C for a hammerhead pier.

3. Frame Bent. A concrete frame bent may be used to support a variety of superstructures. The columns may be either circular or rectangular in cross section. The columns may be directly supported by the footing or by a partial-height wall. Figures 409-6D and 409-6E illustrate a frame bent. If the columns rest directly on the footing, the footing shall be designed as a two-way slab. Construction joints may be required in the cap if the concrete-shrinkage moment introduced into the columns becomes excessive.

409-6.01(02) Usage

The selection of the interior-support type shall be based on the feature passing beneath the bridge, as follows.
1. **Major Water Crossing.** A hammerhead, wall, or single round column-type pier supported by a deep foundation or a spread footing on rock is preferred. Multiple round columns may be used, but they may require a solid wall between columns to avoid the collection of debris. This decision shall be coordinated with the Office of Hydraulics. A single-wall pier may be a more suitable alternative.

2. **Meandering River.** For a meandering river or stream, or where the high flow is at a different skew than the low flow, the most desirable pier type is normally a single, circular pier column.

3. **Highway- or Railroad-Grade Separation.** A thin-wall or frame bent with multiple columns shall be used. The aesthetics of the pier shall be considered. Solid wall piers under a wide superstructure can lead to a tunnel effect for a motorist passing under the structure, and may require the placement of a lighting system under the structure. Surface treatments using form liners or other means shall be investigated, especially for a wall pier.

**409-6.02 Materials**

**409-6.02(01) Epoxy-Coated Reinforcement Under Expansion Joint**

All reinforcing steel in the concrete above the footing, where an expansion joint is located directly over the cap shall be epoxy coated. This includes the stem, cantilevers, and cap. This applies only to a substructure which supports the ends of two superstructure units with an expansion joint located directly over the cap.

**409-6.02(02) Concrete**

Class A concrete shall be used above the footing. Class B concrete shall be used in the footing.

**409-6.03 General Design Considerations**

**409-6.03(01) Pier in Waterway**

A stem-type pier shall have a solid wall to an elevation of 1 ft above the $Q_{100}$ high-water level. Depending on aesthetics and economics, the remainder of the wall may be either solid or
multiple columns. The dimensions of the wall may be reduced by providing cantilevers to form a hammerhead pier. Round noses shall be considered for a pier in a waterway.


A new-bridge pier located within 30 ft of the edge of roadway shall be designed for a vehicular collision-static force of 600 kip, as indicated in LRFD 3.6.5.1.


A pier within 25 ft of a present-track or a future-track centerline shall be designed in accordance with the AREMA Manual for Railway Engineering.

409-6.03(04) Pier-Cap Reinforcement

Multiple layers of negative-moment reinforcement are permitted to minimize cap dimensions.

409-6.03(05) Column Reinforcement

The area of steel reinforcement provided across the interface between the base of the column or pier stem and the top of footing shall not be less than 0.5% of the gross area of column or stem as described in LRFD 5.13.3.8. According to LRFD 5.10.11.4.2, the minimum reinforcement ratio, both horizontally and vertically in a pier, shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio. The reinforcement spacing, either horizontally or vertically, shall not exceed 1’-6”.

409-6.03(06) Reinforcing-Steel Splicing

If a pier-stem height is less than 10 ft, the steel extending out of the footing shall not be spliced. See LRFD 5.11.5 for more information.
409-6.03(07) Compression Reinforcement

Compression steel tends to buckle once the concrete cover is gone or where the concrete around the steel is weakened by compression. The criteria shown in LRFD 5.7.4.2 and 5.7.4.6 for ties or spirals shall be used. See Figure 409-6G for suggested hammerhead- and wall-type-pier reinforcement in columns without plastic hinging capability. Ties may be #3 bars for longitudinal bars up to size #10.

Where column and pier-wall reinforcement is controlled by seismic requirements, see the AASHTO Guide Specifications for LRFD Seismic Bridge Design Articles 8.6 and 8.8 for limits of reinforcement.

409-6.03(08) Piles

For a pier on multiple rows of piles with a footing, pile embedment shall be at least 1.5 ft inside the footing. Bottom-mat reinforcement shall be placed 4 in. above the bottom of the footing.

For a pier on a single row of piles, pile embedment inside the wall shall be 5 ft.

409-6.04 Specific Design Considerations

409-6.04(01) Extended-Pile Bent

1. Limitations. This type of support has little resistance to longitudinal forces, particularly seismic forces, and shall not be used unless such forces are resisted by other substructure units such as integral end bents or abutments. This support shall also not be used if the stream carries large debris, heavy ice flow, or large vessels. If steel H-piles are used for support, they shall be encased in concrete. The concrete encasement shall be extended to 2 ft below the flow-line elevation. Encasement details are provided on the INDOT Standard Drawings. Scour shall be considered in establishing design pile lengths and for the structural design of the piles.

2. Cap Beam. Extended piles require a cap beam for structural soundness, which may be an integral part of the superstructure. Extended drilled shafts shall be arranged to support, for example, widely-spaced beams without the presence of a cap beam if sufficient space is provided at the top for mandatory jacking operations.
3. **Loads.** Girders may be fixed or semi-fixed at an extended pile bent. Because the piles are relatively flexible compared to the end bent or abutment, the force effects induced in the piles by lateral displacement is small. Where practical, one pile shall be placed beneath each girder. The vertical load carried by the piles shall be the girder reaction and the appropriate portion of the pile-cap dead load. Assuming the bent acts as a rigid frame in a direction parallel to the bent, force effects due to lateral displacement and lateral loads may be uniformly distributed among the extended piles.

4. **Cap Design.** The minimum reinforcement shall be #5 bars at 1’-0” spacing on all faces, and shall be in accordance with LRFD 5.7.3.3. The cap shall be designed as a continuous beam.

**409-6.04(02) Hammerhead Pier**

1. **Cofferdam.** If a cofferdam is anticipated to be required, the hammerhead portion of the pier shall be above the average low-water level of the stream.

2. **Bottom Elevation.** The bottom of the hammerhead portion shall be a minimum of 6 ft above the finished ground line at a stream crossing to help prevent debris accumulation.

3. **Effective-Length Factor.** LRFD Table 4.6.2.5-1 provides criteria for the effective length factor, $K$. For beams on rockers or sliding bearings, $K$ shall be taken as 2.1. For an expansion pier with beams on a single row of neoprene pads, $K$ shall be taken as 1.5. For prestressed-concrete beams on semi-fixed bearings on a fixed pier, $K$ shall be taken as 1.2. $K$ shall be taken as 1.0 for the strong or transverse direction.

4. **Pier Wall.** A pier wall shall be designed as columns for biaxial bending. See LRFD 5.7.4.5 for more information.

**409-6.04(03) Frame Bent**

1. **Column Fixity.** The columns founded on a spread- or multiple-piles footing shall be assumed to be fixed at the bottom.

2. **Cantilevered Cap.** The moments used for the cap design shall be calculated at the face of the support for a square or rectangular column, or at the theoretical face of a circular column.
3. **Effective-Length Factor.** The same $K$ factors shall be taken as described for a hammerhead pier in Section 409-6.04(02), in the weak, or longitudinal, direction. $K$ shall be taken as 1.0 for the strong, or transverse, direction. See *LRFD* 4.6.2.5 for more information.

4. **Structural Design.** If the number of columns is kept to a minimum, and the components are reasonably small, frame analysis is both appropriate and safe for a frame bent.

**409-6.04(04) Compression**

Reinforced-concrete piers, pier columns, and piles are referred to as compression members although their design is normally controlled by flexure. Tall, slender columns or pier shafts are relatively rare due to topography. The use of the moment magnification approach in *LRFD* 5.7.4.1 is most-often warranted. For exceptionally tall or slender columns or shafts, a refined analysis, as outlined in *LRFD* 5.7.4.1, shall be performed.

For limits of reinforcement in compression members, see *LRFD* 5.7.4.

**409-6.05 Details**

**409-6.05(01) Size**

Columns can be rectangular, square, or round, with a minimum diameter or thickness of 2 ft. Diameter increments shall be in multiples of 0.5 ft. A solid pier wall shall have a minimum thickness of 2 ft, and may be widened at the top to accommodate the bridge seat.

**409-6.05(02) Cap Extension**

The width of the cap shall project beyond the sides of the columns. The added width of the cap shall be a minimum of 1½ in. on the outside the columns. This width will reduce the reinforcement interference between the column and cap. The cap shall have cantilevered ends to balance positive and negative moments in the cap.
409-6.05(03) **Step Cap**

Where one end of the cap is on a considerably different elevation than the other, the difference shall be accommodated by means of increasing the column heights as shown in Figure 409-6F. The bottom of the cap shall be sloped at the same rate as the cross slope of the top of the bridge deck. The top of the cap shall be stepped to provide level bearing surfaces.

409-6.05(04) **Construction Joints**

A construction joint type A shall be used for all horizontal construction joints. See the INDOT Standard Drawings.

409-6.05(05) **Reinforcement Clearance**

The reinforcement clearances shall be checked to ensure that there is adequate space for the proper placement of the concrete during construction.

409-6.05(06) **Backfill**

An interior bent or pier at the base of a slopewall shall be backfilled with structure backfill as shown on the INDOT Standard Drawings. For an interior bent or pier adjacent to a railroad track, the area shall be backfilled with structure backfill to a point 1.5 ft outside the neat lines of the footing. Structure backfill shall not be provided as backfill material around a pier that is located in a stream.

409-7.0 **BEARINGS**

409-7.01 **General**

Bearings ensure the functionality of a bridge by allowing translation and rotation to occur while supporting the vertical loads. However, the use of integral end bents and possibly integral piers shall be considered prior to deciding upon the use of bearings to support the structure.
409-7.01(01) Movement

Movement shall be considered. Movement includes both translations and rotations. The sources of movement include bridge skew and horizontal-curvature effects, initial camber or curvature, construction loads, misalignment or construction tolerances, settlement of supports, thermal effects, creep, shrinkage, or traffic loading. Bearing pads on a skewed structure shall be oriented parallel to the principal rotation axis.

409-7.01(02) Effect of Bridge Skew and Horizontal Curvature

A skewed bridge moves both longitudinally and transversely. The transverse movement becomes significant on a bridge with a skew angle of greater than 20 deg and bearings not oriented parallel to the movement of the structure.

A curved bridge moves both radially and tangentially. These complex movements are predominant in a curved bridge with a small radius and with an expansion length of longer than 200 ft.

409-7.01(03) Thermal Effects

Thermal translation, $\Delta o$, is estimated as follows:

$$\Delta o = \alpha L \Delta T$$

where $L$ is the expansion length, $\alpha$ is the coefficient of thermal expansion of $6.0 \times 10^{-6}/^\circ F$ for normal-density concrete, or $6.5 \times 10^{-6}/^\circ F$ for steel, and $\Delta T$ is the change in the average bridge temperature from the installation temperature.

A change in the average bridge temperature causes a thermal translation. A change in the temperature gradient induces bending and deflections. The design temperature changes are specified in LRFD 3.12. Maximum and minimum bridge temperatures are defined depending upon whether the location is viewed as a cold or moderate climate. Indiana is considered a cold climate. See LRFD 3.12 for temperature-range values. An installation temperature of 60 °F shall be assumed. The change in average bridge temperature, $\Delta T$, between the installation temperature and the design extreme temperature is used to compute the positive and negative movements. A given temperature change causes thermal movement in all directions. This means that a short, wide bridge can experience greater transverse movement than longitudinal movement.
409-7.01(04) Loads and Restraint

Restraint forces occur if part of a movement is prevented. Forces due to direct loads include the dead load of the bridge and loads due to traffic, earthquakes, water, or wind. Temporary loads due to construction equipment and staging also occur. The majority of the direct design loads are reactions of the bridge superstructure on the bearing. Therefore, they can be estimated from the structural analysis. The applicable LRFD load combinations shall be considered.

409-7.01(05) Serviceability, Maintenance, and Protection Requirements

Bearings under a deck joint collect large amounts of dirt and moisture, which promotes problems of corrosion and deterioration. As a result, such bearings shall be designed and installed to have the maximum possible protection against the environment and to allow easy access for inspection.

The service demands on bridge bearings are severe and result in a service life that is typically shorter than that of other bridge elements. Therefore, allowances for bearing replacement shall be part of the design process. Lifting locations shall be provided to facilitate removal and reinstallation of bearings without damaging the structure. No additional hardware shall be necessary for this purpose. The primary requirements are to allow space suitable for lifting jacks based on the original design and to use devices that permit quick removal and replacement of the bearing.

409-7.01(06) Clear Distance

The minimum clear distance between the bottom shoe of a steel bearing and the edge of the bearing seat or cap shall be 3 in. For an elastomeric pad resting directly on the concrete bridge seat, the minimum edge distance shall be 6 in. under a deck expansion joint, or 3 in. with 4 in. desirable for all other locations. Seismic support lengths shall also be checked.

409-7.01(07) Bearing Selection

Bearing selection is influenced by factors such as loads, geometry, maintenance, available clearance, displacement, rotation, deflection, availability, policy, designer preference, construction tolerances, or cost.
Vertical displacements are prevented, rotations are allowed to occur as freely as possible, and horizontal displacements may be either accommodated or prevented. The loads shall be distributed among the bearings in accordance with the superstructure analysis.

Unless conditions dictate otherwise, conventional steel-reinforced elastomeric bearings shall be used for a girder bridge. Where the practical limits of an elastomeric bearing pad are exceeded, flat polytetrafluoroethylene (PTFE) slider plates shall be considered in conjunction with a steel-reinforced elastomeric bearing. See Figure 409-7A for a general summary of expansion-bearing capabilities. The values shown in the figure are for guidance only.

The final step in the selection process consists of completing a design of the bearing in accordance with LRFD 14.7. The resulting design will provide the geometry and other pertinent specifications for the bearing.

For a structure widening, bearing types shall not be mismatched. Yielding type bearings, such as elastomeric, shall not be used in conjunction with steel rockers or other non-yielding type bearings.

A steel-beam bridge without integral end bents must have at least one fixed bearing line. Due to the presence of the interior-diaphragm keyway, semi-fixed interior supports are allowed for a prestressed-concrete beams bridge. If integral end bents in accordance with the empirical design limits are used, interior fixed bearings are not required.

**409-7.01(08) Anchor Plates and Anchor Bolts**

Anchor plates shall be used only to attach the bottom steel shoe of an expansion bearing to the concrete beam seat. Anchor bolts shall be used to connect fixed steel bearings to the concrete beam seat.

**409-7.02 Elastomeric Bearing Pads and Steel-Reinforced Elastomeric Bearings**

Elastomers are used in both elastomeric bearing pads and steel-reinforced elastomeric bearings. The behavior of both pads and bearings is influenced by the shape factor, S, as shown in LRFD 14.7.5.1.

Elastomeric bearing pads and steel-reinforced elastomeric bearings have fundamentally different behaviors and, therefore, they are discussed separately. Elastomeric pads and bearings shall be
oriented so that the long side is parallel to the principal axis of rotation, as this facilitates the accommodation of rotation.

Holes shall not be placed in an elastomeric bearing pad due to increased stress concentrations around the hole. These increased stresses can cause tearing of the elastomer during an extreme event, such as an earthquake. If holes are placed in a steel-reinforced bearing, the steel-reinforcement thickness shall be increased in accordance with LRFD 4.7.5.3.7.

409-7.02(01) Elastomer

For details and material properties of elastomeric bearings, see the INDOT Standard Drawings, and INDOT Standard Specifications, respectively.

409-7.02(02) Steel-Reinforced Elastomeric Bearing Pad

For design requirements, see LRFD 14.7.6.

409-7.02(03) Elastomeric Bearing Pad

For design requirements for PEP, FGP, and CDP bearing pads, see LRFD 14.7.6.

409-7.03 Standardized Elastomeric Bearing Pads and Assemblies

Standardized elastomeric bearing pads and assemblies have been developed for use with AASHTO prestressed-concrete I-beams, Indiana prestressed-concrete bulb-tee beams, prestressed-concrete hybrid bulb-tee beams, prestressed-concrete spread and adjacent box beams, and structural-steel members. They have been designed based on LRFD 14.7.6, Design Method A.

409-7.03(01) Standard Pad and Assembly Types

1. AASHTO Prestressed-Concrete I-Beam. Elastomeric bearing pads are designated as type 1, 2, 3, or 4 for this type of member. The details are shown on the INDOT Standard Drawings.
2. **Prestressed-Concrete Box Beam.** Elastomeric bearing pads are designated as type 5, 6, or 7, and shape A or B, for this type of member. For a spread box beam, shape A or B may be used. For an adjacent interior box beam, shape A shall be used. For the outside edge under an adjacent exterior box beam, shape B shall be used. The details are shown on the INDOT Standard Drawings.

3. **Prestressed-Concrete Bulb-Tee Beam.** Elastomeric bearing pads are designated as type T, and shape 1, 2, 3, or 4, for this type of member. The details are shown on the INDOT Standard Drawings.

4. **Prestressed-Concrete Wide Flange Bulb-Tee Beam.** Elastomeric bearing pads are designated as type TH, and shape 5, 6, 7, or 8, for this type of member. The details are shown on the INDOT Standard Drawings.

5. **Steel Beam or Girder.** Elastomeric bearing assemblies are designated as type S, with bearing-area designation 1, 2, 3, 4, 5, 6, or 7, and effective-elastomer-thickness designations a or b, for this type of member. The details and designations are shown on the INDOT Standard Drawings.

The locations of elastomeric-bearing devices shall be shown on the plans with their type and shape designations. However, they are not separate pay items.

**409-7.03(02) Design Parameters**

The design of bearing devices is governed by the parameters as follows:

1. dead-load plus live-load reaction at service limit state, impact not included;

2. expansion length, or distance from fixed support to expansion support; and

3. grade percentage due to nonparallel surfaces, considering dead-load rotation, profile grade of member, and camber of member.

**409-7.03(03) Determining Standard Bearing-Device Type  [Rev. May 2013]**

The procedure for determining the applicable standard elastomeric bearing device is the same for each structural-member type.
Determine the dead-load plus live-load reaction, and calculate the maximum expansion length for the bridge at the support for which the device is located. Then enter Figure 409-7B, 409-7C, 409-7D, or 409-7E, Elastomeric Bearing Pad or Assembly Types, Properties, and Allowable Values, for the appropriate structural-member type, with the reaction and maximum expansion length. The required bearing-device size is that which corresponds to the reaction and expansion-length values shown in the figure which are less than or equal to those determined. If the reaction or expansion length is greater than the figure’s value, use the next larger device size. If the reaction or expansion length is greater than the maximum value shown on the figure, the pad must be properly resized and designed.

The maximum service limit state rotation due to total load, $\Theta_s$, shall be calculated in accordance with $LRFD$ 14.4.2.1.

The requirement for a tapered plate shall be determined in accordance with $LRFD$ 14.8.2. See Figure 409-7F for a typical elastomeric bearing pad with tapered steel plate. Stainless steel should be considered only when located beneath an expansion joint. When a stainless steel tapered plate is specified, the steel plate cast with the beam, steel stud, and welds must also be specified as stainless steel.

### 409-7.04 Nonstandardized Elastomeric Bearing Device

The design shall be based on $LRFD$ 14.7.6, Method A.

Each pad or assembly shall be sized according to the load capacities and expansion lengths that it can accommodate.

An elastomeric bearing device not shown on the INDOT Standard Drawings may be used if its parameters check, or its design is in accordance with $LRFD$ 14.7.6. $LRFD$ defines certain limitations in terms of allowable stresses, movements, or minimum dimensions. These limitations are as follows.

1. **Shear Modulus.** See $LRFD$ 14.7.6.2. The design of an elastomeric bearing pad shall include, but shall not be limited to, the consideration of increased $G$ at a temperature below 73 °F; see $LRFD$ 14.6.3.1.

2. **Design Shear Force.** The elastomer with the lowest temperature tolerance shall be used. The total elastomer thickness shall be sufficient to resist twice the design shear force.
3. **Relationship of Device Dimensions.** Both the width and the length of the device shall be at least three times the total thickness of the pad. For a circular pad, the diameter of the pad shall be at least four times the total thickness of the pad.

4. **Stress Due to Dead Load Plus Live Load without Impact.** This stress shall be less than or equal to the lesser of 1.25 ksi or 1.25 $GS$.

5. **Rotational Deflection.** Sufficient pad thickness or a beveled plate shall be provided to prevent a liftoff condition on the leading edges of the device.

6. **Anchorage.** The pad or assembly shall be secured against seismic or other extreme-event resistant anchorage to defy the horizontal movement in excess of that accommodated by shear in the pad, unless it is intended to act as a fuse as required by *LRFD* 14.7.6.3.8. The calculations are performed in the Strength-Limit state. The load modifiers for ductility (*LRFD* 1.3.3), redundancy (*LRFD* 1.3.4), and importance (*LRFD* 1.3.5) must be accounted for.

**409-7.05 Connections for Elastomeric Bearing or PTFE Bearing**

An elastomeric bearing or PTFE bearing shall be provided with adequate seismic-resistant anchorage to resist the transverse horizontal forces in excess of those accommodated by shear in the bearing. The restraint may be provided by one of the methods as follows:

1. steel side retainers with anchor bolts;

2. concrete shear keys placed in the top of the pier cap, or channel slots formed into the top of the cap or mudwall at the end bent; or

3. concrete channels formed in the top of the end bent cap or expansion pier cap.

Steel side retainers and anchor bolts shall be designed to resist the minimum transverse seismic force for the seismic category in which the bridge is located. The number of side retainers shall be as required to resist the seismic forces. They shall be placed symmetrically with respect to the cross section of the bridge. Side retainers will often be required on each side of the girder flange of each beam line. The strength of the beams and diaphragms shall be sufficient to transmit the seismic forces from the superstructure to the bearings. A minimum of two anchor bolts of 1 in. diameter shall be provided for each side retainer.

Concrete channels formed around each beam in the top of the end bent cap or expansion pier cap represent an acceptable alternative to steel side retainers. The top of the top shoe shall be set a
minimum of 4 in. below the top of the concrete channel. If a top shoe is not present, the bottom of the beam shall be placed 4 in. below the top of the channel. The minimum depth of the channel shall be 6 in. The horizontal clearance from the side of the top shoe or edge of the beam to the side wall of the channel shall be at least 1 in.

Integral end bents are an effective way of accommodating horizontal seismic forces. An integrally-designed end bent will inherently resist the transverse seismic forces.

**409-7.06 Shear Keys at Semi-Fixed Support**

Unreinforced shear keys shall be provided between the beams at each semi-fixed supports. The shear keys rest in recessed keyways of 1 ft width by 3 ft length by 3 in. depth, the edges of which are also unreinforced. Although the shear keys are not structurally designed, they are expected to adequately resist the anticipated horizontal seismic forces.

To ensure that the shear keys will function as intended, keyways shall be provided between each beam line at each semi-fixed support, and an expanded-polystyrene sheet, with a maximum thickness of 1/2 in., shall be provided in the bottom of the keyway resulting in a minimum shear-key extension of 2 1/2 in. into the keyway.

Seismic restraint for an adjacent-box-beams bridge shall be provided with retaining blocks at the ends of the pier caps and end bent caps. The blocks shall be designed as reinforced shear keys and shall be in accordance with *LRFD 5.8.4.*

**409-7.07 Fixed Steel Bearing**

The top shoe of a steel bearing shall be at least as wide as the beam flange, but not more than 2 in. wider. The maximum reaction is shown for each shoe type on the INDOT *Standard Drawings*. An independent design is required if the design reaction is greater than the maximum reaction shown, or if the beam or girder flange width is not in accordance with the *Standard Drawings*.

If the flexibility of tall, slender piers is sufficient to absorb the horizontal movement at the bearings due to temperature change without developing undue force in the superstructure, the bearings, one pier, or two or more piers, may be fixed to distribute the longitudinal force among the piers.
The connection between a fixed steel shoe and the pier cap shall be made with anchor bolts. The ultimate shear resistance in the anchor bolts, pintles, and high-strength bolts in the top shoe shall be verified that it is adequate to resist the calculated seismic forces. See *LRFD* 6.13.2.7 and Figure 409-7G for determining the nominal shear resistance of anchor bolts and pintles. The minimum connections shall be as shown in Figure 409-7I.

Masonry anchor bolts shall extend into the concrete a minimum of 1’-3”. Anchor bolts shall be in accordance with *LRFD* 14.8.3.

Anchor bolts shall be located beyond the limits of the bottom beam flange and interior diaphragm to ensure adequate clearance for anchor-bolt installations and impact wrenches. The grade of structural steel used for the anchor bolts or pintles shall be shown on the plans.

Where the pintles cannot be designed to accommodate the minimum seismic force of seismic category A, a hooded top shoe as shown in Figure 409-7I shall be provided. A hooded top shoe is also an acceptable seismic restrainer. If seismic forces are large, a restraining device will be required instead of the hooded shoe.

**409-7.08 Pot Bearing**

A fixed pot bearing shall be in accordance with the connection requirements for a fixed steel shoe. The top bearing plate and lower masonry plate shall be bolted to the beam flange and the pier cap respectively. Where welds are required between plates in the pot bearing, they shall be made continuous around the perimeter of the smaller plate.

**409-7.09 Miscellaneous Bearing-Connection Details**

The following figures provide suggested details for acceptable connections for bearing assemblies.

1. For a fixed-shoe assembly, see Figure 409-7I.
2. For an elastomeric bearing assembly, see Figures 409-7J, 409-7K, and 409-7L.
3. For a PTFE bearing assembly, see Figure 409-7M.

The suggested details may be revised as necessary for each project. Also, see the INDOT *Standard Drawings* for more bearing details.
409-8.0 BRIDGE-SEAT ELEVATIONS

In establishing bridge-seat elevations at both end and interior supports, the following shall be considered.

1. Bridge-deck depth.

2. Fillet of \( \frac{3}{4} \) in. The fillet distance is measured from bottom of the deck to the top of beam. This distance is included to allow for variation in beam camber.

3. Residual beam camber.

4. Vertical curve effect: + for sag vertical curve, - for crest vertical curve.

5. Beam depth.

6. Middle-span correction for curved bridge with straight beams. Due to the distance variation from the bridge centerline and beam centerline, this shall appear at the supports and at the middle of the span.

7. Bearing thickness, including shims and taper plate.

The accuracy for establishing bridge-seat elevations shall be to the nearer 0.01 ft.


NHI course No. 130082A, LRFD for Highway Bridge Substructures and Earth Retaining Structures, 2006, with all subsequent Revisions, US Department of Transportation, Federal Highway Administration.
Use of Integral End Bridge Length (ft) vs. Skew (deg)

**Figure 409-2A**

**NOTES:**

1. Integral end bents may be used in a curved-alignment or curved-girder structure with length of 500 feet or less, with a subtended angle in plan not greater than 30°.

2. Pile confinement spiral reinforcement required on integral end bents with expansion length greater than 250 ft.

**USE OF INTEGRAL END BENT**

**HP Piles Only**

Elastic Dynamic Analysis required for structures of length over 500 ft located in Seismic Design Category B.

**HP or Shells**
INTERMEDIATE PIER DETAIL FOR INTEGRAL STRUCTURE
LOCATED IN SEISMIC AREA WITH SEISMIC-DESIGN CATEGORY GREATER THAN A

Figure 409-2B
NOTES:

1. A pavement ledge greater than 6 in. may be considered for skewed structures or structures subject to significant truck traffic.

2. A depth greater than 6 ft requires approval from the Office of Bridge Design.

3. All reinforcing steel shall be epoxy coated.

SUGGESTED INTEGRAL END BENT DETAILS
Method A, Beams Attached Directly to Piling

Figure 409-2C
(Page 1 of 4)
NOTE: All reinforcing steel shall be epoxy coated.

Reinforcing details, backfill behind end bent and similar details are as shown on the prestressed concrete I-beam section unless otherwise noted.

Optional Type A Construction Joint

1/2" Min. Pl Anchorage Stiffener (Both Sides of Web)

Optional Type A Construction Joint

Spiral Reinforcement

Steel Beam or Girder

Prestressed Strand Extension (8 per Beam or 1/2 the Strands in the Bottom Row, Whichever is Less)

3/4" Ø Threaded Inserts for 3/4" Ø Threaded Bars Between Beams

H-Pile Bearing Beam (Min. Web Thickness = 1/2")

Steel H-Pile or Steel Pipe Pile

SUGGESTED INTEGRAL END BENT DETAILS
Method A, Beams Attached Directly to Piling

Figure 409-2C
(Page 2 of 4)
NOTE:
① Minimum distance from front or back face of end bent should be 9 in.

SUGGESTED INTEGRAL END BENT DETAILS
Method A, Beams Attached Directly to Piling

Figure 409-2C
(Page 3 of 4)
SUGGESTED INTEGRAL END BENT DETAILS
Method A, Beams Attached Directly to Piling

Figure 409-2C
(Page 4 of 4)
SUGGESTED INTEGRAL END BENT DETAILS  
Method B, Beams Attached to Concrete Cap  

Figure 409-2D  
(Page 1 of 4)
SUGGESTED INTEGRAL END BENT DETAILS
Method B, Beams Attached to Concrete Cap

Figure 409-2D
(Page 2 of 4)
NOTE:

1. Minimum distance from front or back face of end bent should be 9 in.

SUGGESTED INTEGRAL END BENT DETAILS
Method B, Beams Attached to Concrete Cap

Figure 409-2D
(Page 3 of 4)
SUGGESTED INTEGRAL END BENT DETAILS
Method B, Beams Attached to Concrete Cap

Figure 409-2D
(Page 4 of 4)
Steel Pipe Pile

Steel H-Pile or

Pitch = 2.5"

Outside Diameter

28.5"

2 2" Min.

Spiral Height, h

2" Clr.

1 1/2 Extra Turns of Spiral

1 1/2 Extra Turns of Spiral

Steel H-Pile or Steel Pipe Pile

L = Total Length of Spiral Reinforcement (in.)

\[ L = \left[ \frac{h}{2.5"} + 2(1 \frac{1}{2} \text{ turns}) \right] \pi 28.5" \]

SPIRAL REINFORCEMENT
FOR BRIDGES WITH EXPANSION LENGTH GREATER THAN 250 FT

Figure 409-2E
This figure deleted [May 2019]

Tooth Joint
Figure 409-2F
NOTE:
Coarse aggregate and 6" end-bent drain pipe are not required to be specified separately for an end bent placed behind an MSE Wall.

END BENT PLACED BEHIND MSE WALL

Figure 409-2G
(Sheet 1 of 2)
PREFERRED DETAIL
OPTION 1

PREFERRED DETAIL
OPTION 2

NOTE:
Where an MSE is placed parallel to the bridge approach roadway, it should be placed adjacent the outside face of the end bent or wingwall, but not cast against it. Sufficient clearance is needed to accommodate the thermal movement of the end bent. The MSE wall should not be placed abutting the back face of the end bent or wingwall.

END BENT PLACED BEHIND MSE WALL

Figure 409-2G
(Sheet 2 of 2)
SUGGESTED SEMI-INTEGRAL END BENT DETAILS
(Method 1)

Figure 409-3A
(Page 1 of 4)
SUGGESTED SEMI-INTEGRAL END BENT DETAILS
(Method 1)

Figure 409-3A
(Page 2 of 4)
ANCHOR PLATE DETAIL

Typ. 1/2" x 1'-0" (Cast in Beam)

Prepare End for Full Penetration Weld in Shop

1/8" x 6" long (Min.) Headed Automatic Welded Stud

Typ. 1/8"

3" Min. (typ.)

Constr. Joint, Type "A"

2" (typ.)

Anchor Plate (Bar 6" x 1")

SUGGESTED SEMI-INTEGRAL END BENT DETAILS
(Method 1)

Figure 409-3A
(Page 3 of 4)
NOTES:

1. 3 layers of medium weight roofing felt with grease between layers over 1/8" high-density plastic bearing strip with smooth side up.

2. Expanded polystyrene, size to be determined by designer.

3. Polychloroprene joint membrane attached to concrete. See Figure 409-3C.

4. Main cap reinforcing. Reinforce for dead and live loads. Stirrup size determined by designer, spaced at 1'-0" minimum.

5. Anchor plate. See Detail on Sheet 3 of 4.

6. Construction joint, type A.

7. 1" thickness expanded polystyrene, to be extended to 1/2" outside limits of beam, so that beam does not come in contact with construction-jointed concrete.

8. Plate 1/2" x 1'-0", full width of beam, cast in beam.

9. #6E x 6'-0" through 1" Ø holes cast in beams, lapped with #7E between beams.


11. #6 reinforcing bar set in 1'-0" depth field-drilled hole filled with epoxy grout, min. pullout 26,500 lb.

12. a. PVC sleeve, size determined by designer. Top of sleeve to be sealed before concrete is poured.

   b. Used only if uplift is expected, or if bridge is in Seismic Category B.

13. Minimum distance from front or back face of end bent to edge of pile should be 9".

SUGGESTED SEMI-INTEGRAL END BENT DETAILS

(Method 1)

Figure 409-3A

(Page 4 of 4)
SUGGESTED SEMI-INTEGRAL END BENT DETAILS
(Method 2)

Figure 409-3B
(Page 1 of 3)
SECTION BETWEEN BEAMS

SUGGESTED SEMI-INTEGRAL END BENT DETAILS
(Method 2)

Figure 409-3B
(Page 2 of 3)
NOTES:

1. 1/2" expanded polystyrene (horizontal face), 1" expanded polystyrene (vertical face).
2. Polychloroprene joint membrane attached to concrete. See Figure 409-3C.
3. Main cap reinforcing. Reinforce for dead and live loads. Stirrup size determined by designer, spaced at 1'-0" minimum.
4. Elastomeric bearing pad.
5. Optional construction joint, type A.
6. Expanded polystyrene cut to clear bearing pad by 1/2".
7. #6E x 6'-0" through 1" Ø holes cast in beams, lapped with #7E between beams.
8. Prestressed strand extension.
9. #6 reinforcing bar set in 1'-0" depth field-drilled hole filled with epoxy grout, min. pullout 26,500 lb.
10. a. PVC sleeve, size determined by designer. Top of sleeve to be sealed before concrete is poured.
    b. Used only if uplift is expected, or if bridge is in Seismic Category B.
11. Minimum distance from front or back face of end bent to edge of pile should be 9".

SUGGESTED SEMI-INTEGRAL END BENT DETAILS
(Method 2)

Figure 409-3B
(Page 3 of 3)
JOINT MEMBRANE DETAIL

Figure 409-3C
NOTES:

1. 10" if design year AADT < 1000. Otherwise, 1'-0" or the thickness of the concrete pavement, whichever is thicker.

2. Structure Backfill, Type 4 if slab bridge or Coarse Aggregate #8 or #9 for all other bridges with end bents.

PAVEMENT LEDGE DETAIL
FOR INTEGRAL AND SEMI-INTEGRAL END BENT
(Deck without Expansion Joint)

Figure 409-3D
TYPICAL WINGWALL DETAILS

Figure 409-5A
FLARED-WING LENGTHS AND END ELEVATIONS, SQUARE STRUCTURE

Figure 409-5B
FLARED-WING LENGTHS AND END ELEVATIONS, STRUCTURE SKEWED TO RIGHT

Figure 409-5C
FLARED-WING LENGTHS AND END ELEVATIONS,
STRUCTURE SKEwed TO LEFT

Figure 409-5D
FLARED-WING-CORNER DIMENSIONS, SQUARE STRUCTURE

Figure 409-5E
FLARED-WING-CORNER DIMENSIONS, STRUCTURE SKewed TO RIGHT

Figure 409-5F
TO ROADWAY

HALF ABUTMENT WIDTH PARALLEL TO SKEW

HALF SUPERSTRUCTURE WIDTH

HALF PVMT. WIDTH

EDGE PVMT. EL.

CR. RDWY EL.

SHOULDER WIDTH

EDGE OF PVMT.

SHOULDER SLOPE

BREAK-POINT EL.

TOP WING EL.

SLOPE OF FILL

6" MIN.

D = TOP WING EL. - BREAK-POINT EL.

∞ = ANGLE BETWEEN WING AND LINE ⊥ TO CR. RDWY.

Δ = SKEW ANGLE

L1 = \( \frac{T}{\cos \Delta} \)

T & M - TO BE AS DETERMINED BY DESIGNER.

D = TOP WING EL. - BREAK-POINT EL.

∞ = ANGLE BETWEEN WING AND LINE ⊥ TO CR. RDWY.

Δ = SKEW ANGLE

L1 = \( \frac{T}{\cos \Delta} \)

T & M - TO BE AS DETERMINED BY DESIGNER.

CROSS SECTION

HALF ABUTMENT WIDTH ⊥ TO CR. RDWY

HALF PVMT. WIDTH

EDGE PVMT. EL.

CR. RDWY EL.

SHOULDER SLOPE

BREAK-POINT EL.

TOP WING EL.

SLOPE OF FILL

6" MIN.

D = TOP WING EL. - BREAK-POINT EL.

∞ = ANGLE BETWEEN WING AND LINE ⊥ TO CR. RDWY.

Δ = SKEW ANGLE

L1 = \( \frac{T}{\cos \Delta} \)

T & M - TO BE AS DETERMINED BY DESIGNER.

FLARED-WING-CORNER DIMENSIONS,
STRUCTURE SKEWED TO LEFT

Figure 409-5G
TYPICAL ABUTMENT DETAILS

Figure 409-5H
(Page 1 of 2)
TYPICAL ABUTMENT DETAILS

Figure 409-5H
(Page 2 of 2)
Steel pipe pile or steel H-pile are used where steel H-piles are used.

Concrete encasement with steel H-pile with concrete encasement.

Epoxy-coated steel pipe pile or steel H-pile with concrete encasement.

Figure 409-6A
Note: Elevation of bottom of mudsill shall be below the contraction scour elevation but not more than 6'-0" below the flowline elevation.

WALL PIER ON SINGLE ROW OF PILES

Figure 409-6B
HAMMERHEAD PIER

Figure 409-6C

- Number and size of reinforcing bars to be determined by design.

- Hooks may be required by design.

- Width determined by design.

- As required (min. 2'-6")

- Approximate 20% to 50% of B

- Approximate 50% to 100% of C

- Approximately 30% to 50% of cap length

- Not to be more than length of footing.

- Round for stream crossing, rectangular for grade separations.
1. Minimum column spacing to be 9'-2".
3. Bent to be designed as frame bent.
4. Column Steel to extend into footing.
5. Construction joints in cap to be placed to miss bearings and columns.
6. Piles type and size as determined by geotechnical report.

NOTES

ABOVE PROPOSED GROUND LINE.

LINE
PROP.
GROUND
LINE

IF REQUIRED BY ANALYSIS
CONSTRUCTION JOINT

MIN.
MIN.
(TYP.)

CONSTRUCTION JOINT AT
30'-0" MAX. SPACING

TOP OF CRASHWALL TO BE 2'-10" MINIMUM
ABOVE PROPOSED GROUND LINE

ELEVATION

NUMBER AND SIZE OF REINFORCING
BARS TO BE DETERMINED BY DESIGN

SECTION "A-A"

SECCTION "A-A"

NOTES

1. Minimum column spacing to be 9'-2".
3. Bent to be designed as frame bent
4. Column Steel to extend into footing.
5. Construction joints in cap to be placed to miss bearings and columns.
6. Piles type and size as determined by geotechnical report.

GEOMETRICS FOR FRAME BENT WITH SOLID STUB WALL

Figure 409-6D
GEOMETRICS FOR FRAME BENT WITH INDIVIDUAL CRASHWALLS

Figure 409-6E

NOTES
1. MINIMUM OF 3 COLUMNS.
2. BENT TO BE DESIGNED AS FRAME BENT.
3. IF OPEN JOINT IN SUPERSTRUCTURE, CAP TO HAVE JOINT AT SAME POSITION.
4. INDIVIDUAL FOOTINGS MAY BE USED UNDER EACH COLUMN WHEN FOUNDATION IS ON ROCK.
5. MINIMUM COLUMN SPACING TO BE 6'-2".
6. PILES TYPE AND SIZE AS DETERMINED BY GEOFENICAL REPORT.

SECTION "A-A"

SECTION "B-B"

SECTION "C-C"

CONSTRUCTION JOINT IF REQUIRED BY ANALYSIS

TOP OF CRASHWALL TO BE 2'-12" MINIMUM ABOVE PROPOSED GROUND LINE.

NUMBER AND SIZE OF REINFORCING BARS TO BE DETERMINED BY DESIGN.

SAME WIDTH AS COLUMN

KEYWAY CONSTR. JOINT

HOOKS MAY BE REQUIRED BY DESIGN

WIDTH DETERMINED BY DESIGN

2'-6" MIN.

4'-6" CL.

2'-0" MIN.

2'-0" MIN.

PROP. GROUND LINE.

"A"

"B"

"C"

"A"

"B"

"C"

1/2" (TYP.)

2'-0" (TYP.)

612.0x792.0
STEP CAP

Figure 409-6F
NOTE:
CROSS TIES SHALL BE PLACED AT ALTERNATE VERTICAL BARS AND BE SPACED AT 2'-0" MAX. HORIZONTALLY AND 1'-0" MAX. VERTICALLY.

SUGGESTED REINFORCING DETAILS FOR WALL OR HAMMERHEAD PIER
Figure 409-6G
<table>
<thead>
<tr>
<th>Bearing Type</th>
<th>Load</th>
<th>Translation</th>
<th>Rotation</th>
<th>Costs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min. (kip)</td>
<td>Max. (kip)</td>
<td>Min. (in.)</td>
<td>Max. (in.)</td>
</tr>
<tr>
<td>Elastomeric Pad</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plain (PEP)</td>
<td>0</td>
<td>100</td>
<td>0</td>
<td>0.5</td>
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<td>Cotton Duck (CDP)</td>
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<td>315</td>
<td>0</td>
<td>0.25</td>
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<td>Fiberglass (FGP)</td>
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<td>135</td>
<td>0</td>
<td>1</td>
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<td>Steel Reinforced Elastomeric</td>
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<td>1007</td>
<td>0</td>
<td>4</td>
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<td>Flat Polytetrafluoroethylene (PTFE) Slider</td>
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<td>&gt; 2250</td>
<td>1</td>
<td>&gt; 4</td>
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<td>Curved Sliding Cylindrical</td>
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<td>0</td>
<td>0</td>
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<td>2250</td>
<td>0</td>
<td>0</td>
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<td>0</td>
<td>4</td>
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<td>Single Roller</td>
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<td>100</td>
<td>1</td>
<td>&gt; 4</td>
</tr>
<tr>
<td>Curved PTFE</td>
<td>270</td>
<td>1575</td>
<td>0</td>
<td>0</td>
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<tr>
<td>Multiple Rollers</td>
<td>112</td>
<td>2250</td>
<td>4</td>
<td>&gt; 4</td>
</tr>
</tbody>
</table>

SUMMARY OF EXPANSION-BEARING CAPABILITIES

Figure 409-7A
## Maximum DL + LL Reaction, (kip) Maximum Expansion Length, (ft) Bearing-Pad Type | W (in.) | L (in.) | Area (in.²) | Shape Factor, S | hₙ (in.) | Number of Internal Elastomeric Layers, n | Allowable Compressive Stress, σ₇L (psi)
---|---|---|---|---|---|---|---
124 | 230 | 1 | 14 | 10.5 | 147 | 6.00 | 2.0625 | 3 | 844
143 | 285 | 2 | 14 | 11.5 | 161 | 6.31 | 2.5625 | 4 | 887
190 | 285 | 3 | 18 | 11 | 198 | 6.83 | 2.5625 | 4 | 960
324 | 340 | 4 | 24 | 12 | 288 | 8.00 | 3.0625 | 5 | 1125

### ELASTOMERIC BEARING PAD TYPES, PROPERTIES, AND ALLOWABLE VALUES FOR AASHTO I-BEAMS

Figure 409-7B
<table>
<thead>
<tr>
<th>Maximum $DL + LL$ Reaction, (kip)</th>
<th>Maximum Expansion Length, (ft)</th>
<th>Bearing-Pad Type</th>
<th>W (in.)</th>
<th>L (in.)</th>
<th>Area (in.$^2$)</th>
<th>Shape Factor, $S$</th>
<th>$h_{rt}$ (in.)</th>
<th>Number of Internal Elastomeric Layers, $n$</th>
<th>Allowable Compressive Stress, $\sigma_{TL}$ (psi)</th>
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</thead>
<tbody>
<tr>
<td>249</td>
<td>285</td>
<td>5A</td>
<td>22</td>
<td>11</td>
<td>242</td>
<td>7.33</td>
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<td>1031</td>
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<td>122</td>
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<td>2.0625</td>
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ELASTOMERIC BEARING PAD TYPES, PROPERTIES, AND ALLOWABLE VALUES FOR BOX BEAMS

Figure 409-7C
<table>
<thead>
<tr>
<th>Maximum DL + LL Reaction, (kip)</th>
<th>Maximum Expansion Length, (ft)</th>
<th>Bearing-Pad Type</th>
<th>$W$ (in.)</th>
<th>$L$ (in.)</th>
<th>Area (in.$^2$)</th>
<th>Shape Factor, $S$</th>
<th>$h_{rt}$ (in.)</th>
<th>Number of Internal Elastomeric Layers, $n$</th>
<th>Allowable Compressive Stress, $\sigma_{TL}$ (psi)</th>
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<tbody>
<tr>
<td>306</td>
<td>341</td>
<td>T1</td>
<td>23</td>
<td>12</td>
<td>276</td>
<td>7.89</td>
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<td>394</td>
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<td>T2</td>
<td>23</td>
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<td>570</td>
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<td>547</td>
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</tbody>
</table>

Note: Bearing pads with $T$ designation are for Indiana bulb-tee members. Bearing pads with $TH$ designation are for wide bulb-tee members.

**ELASTOMERIC BEARING PAD TYPES, PROPERTIES, AND ALLOWABLE VALUES FOR INDIANA BULB-TEE AND WIDE BULB-TEE MEMBERS**

Figure 409-7D
<table>
<thead>
<tr>
<th>Maximum DL + LL Reaction, (kip)</th>
<th>Maximum Expansion Length, (ft)</th>
<th>Bearing-Assembly Type</th>
<th>W (in.)</th>
<th>L (in.)</th>
<th>Area (in.²)</th>
<th>Shape Factor, S</th>
<th>hₜ (in.)</th>
<th>Number of Internal Elastomeric Layers, n</th>
<th>Allowable Compressive Stress, σₜₜ (psi)</th>
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<tr>
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<td>174</td>
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<td>57</td>
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<td>S1-b</td>
<td>11</td>
<td>8</td>
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<td>651</td>
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<td>3.0625</td>
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<td>8.57</td>
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<td>1205</td>
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ELASTOMERIC BEARING ASSEMBLY TYPES, PROPERTIES, AND ALLOWABLE VALUES FOR STRUCTURAL-STEEL MEMBERS

Figure 409-7E
Taper top of steel plate to the nearest $\frac{1}{6}$" to correct for slope.

2. When stainless steel is specified, plate cast with beam, studs, and weld must all be specified as stainless steel.

**NOTES:**

**ELASTOMERIC BEARING PAD WITH TAPERED STEEL PLATE**

**Figure 409-7F**
<table>
<thead>
<tr>
<th>Grade of Steel</th>
<th>Minimum Tensile Strength, $F_{Uv}$ (ksi)</th>
<th>Nominal Shear Resistance, $R_n$, (kip) *</th>
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<tr>
<td></td>
<td></td>
<td>Anchor Bolts, threads included **</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 in.</td>
</tr>
<tr>
<td>A 307</td>
<td>60</td>
<td>17.9</td>
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<tr>
<td>A 325 High Strength</td>
<td>120</td>
<td>35.8</td>
</tr>
<tr>
<td></td>
<td>105</td>
<td>n/a</td>
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* One shear plane is assumed. Resistance value should be multiplied by number of shear planes.

** Value should be multiplied by 0.80 for a connection longer than 50 in.

NOMINAL SHEAR RESISTANCE OF ANCHOR BOLTS AND PINTLES

Figure 409-7G
<table>
<thead>
<tr>
<th>No. of Anchor Bolts</th>
<th>Diameter (in)</th>
<th>Span Length Range (ft)</th>
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<tr>
<td>4</td>
<td>1</td>
<td>20 ≤ Span &lt; 100</td>
</tr>
<tr>
<td>4</td>
<td>1 1/8</td>
<td>100 ≤ Span &lt; 150</td>
</tr>
<tr>
<td>4</td>
<td>1 3/8</td>
<td>Span ≥ 150</td>
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MINIMUM CONNECTIONS FOR FIXED STEEL SHOES

Figure 409-7H
FIXED SHOE ASSEMBLY

Figure 409-7 I
ELASTOMERIC BEARING ASSEMBLY

Figure 409-7J
ELASTOMERIC BEARING ASSEMBLY

Figure 409-7K
ELEVATION

C\ BEARING

A 325 BOLTS W/ WASHERS

SHIM PLATE
TOP PLATE

ELASTOMERIC PAD
BOTTOM PLATE
ANCHOR PLATE

C\ GIRDER

2" MIN.

1/2" MAX.

S = WELD SIZE IN INCHES

CROSS SECTION

ELASTOMERIC BEARING ASSEMBLY WITH BOTTOM PLATE

Figure 409-7L
PTFE ELASTOMERIC BEARING ASSEMBLY

ELEVATION

CIRCLICAL BEARING

1/8" TFE PLATE W/DIMPLED SURFACE
TOP PLATE

1/16" STAINLESS STEEL SLIDING PLATE (A240) TYPE 304 2B FINISH

ELASTOMERIC PAD

CROSS SECTION

GIRDER

A325 BOLTS W/WASHERS

1/4" CL. (TYP.)

SHIM PLATE
SIDE RETAINER

ANCHOR BOLTS (SWEDGED)

Figure 409-7M
CHAPTER 410

Earth-Retaining System

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<td>410-5.01(06)</td>
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<td>Mar. 2017</td>
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CHAPTER 410

EARTH-RETAINING SYSTEMS

410-1.0 INTRODUCTION

The intent of this chapter is to inform designers, and earth-retaining-system manufacturers and suppliers, of the procedures and responsibilities associated with the preparation of plans for an earth-retaining system.

410-1.01 Consideration of an Earth-Retaining System

An earth-retaining system shall be considered in the situations as follows.

1. Right-of-way is too limited for constructing side slopes.
2. There is a proximate live-load surcharge which must remain in place. Such surcharges can include buildings, highways, or railroads.

410-1.02 Aesthetics Considerations

An earth-retaining system or retaining wall is one of the key road-design elements. Along with the direct function of holding back earth, it provides opportunities for aesthetic enhancement of transportation systems. A retaining wall acts as a link between various highway structures and adjacent land forms. Where multiple walls exist along a corridor, repetition of a similar design will provide continuity throughout that corridor. Therefore, the designer shall be aware of the total impact of retaining walls within the roadway corridor and determine how to treat them aesthetically so that they blend into the surrounding environment. The designer shall be conscious of the traveler’s view of the wall as well as the view of those adjacent to the corridor.

Aesthetic elements surrounding a particular retaining wall are key to public acceptance of a wall project. Early in the wall-design process, all comments about the wall generated from public meetings shall be reviewed during the preliminary design and environmental-documentation process. Where possible, such comments shall be considered in the design.

There is often uncertainty associated with aesthetics and there is no universally accepted theory. Aesthetic qualities are the visual qualities that contribute to a perception of well-being and quality of life as defined by a cross-section of society.
Because of the uncertainties surrounding aesthetics, and the lack of a universally accepted theory of aesthetics, the following three-step process has been established toward full consideration of the aesthetic elements of a retaining wall. These steps shall be integrated and considered together, not as discrete individual actions. Aesthetics consists of a blending and balancing of materials such as wood, concrete, or steel, with design elements such as line, form, color, and texture, and architectural elements such as wall caps, parapets, fencing, etc.

410-1.02(01) Architectural Considerations

Determination shall be made as to whether to involve a landscape architect. Consideration shall be given to involving a landscape architect as follows.

1. The wall will exceed 10 ft in height.

2. Extenuating circumstances are present, regardless of wall height. For example, in a rural area, the public may request special aesthetic treatments to enhance a scenic area. Other examples include, but are not limited to, historic areas, tourist areas, or other public requests.

A landscape architect can provide important information, guidance, and early assistance with aesthetic considerations. Involving a landscape architect in the design process for an earth-retaining system will not only result in a more aesthetically-pleasing design, but it can also result in cost-savings. It is easier and more cost-effective to determine the real costs of a design rather than requiring expensive add-ons later.

410-1.02(02) Earth-Retaining Structure Adjacent to Sidewalk or Shared-Use Facility

A functional or decorative retaining wall can be placed where an existing retaining wall must be extended or replaced and matched. The need for such a wall will mostly likely occur in an urban or suburban area. Such a wall can be adjacent to a sidewalk, multi-use path, or trail that provides access to adjoining homes, businesses, or public-recreation facilities. Where the wall height is greater than 2.5 ft above the adjacent ground or walk, etc., a railing will be required to prevent pedestrians or children from falling from the wall to a lower level which can result in injury. A chain-link fence fabric shall be attached to the railing to prevent a person from crawling between or trapping their head between the horizontal railing members. The chain-link fence fabric shall be vinyl coated. The color of such coating shall be dark green, dark brown, or black to minimize the visibility of the fence fabric to a passer-by, property owner, etc.
410-1.02(03) Stairway Access through Earth-Retaining Structure to Property

A handrail shall be placed along each side of a stairway that provides pedestrian access to a property on the other side of a retaining wall. The width of the wall opening shall match the width of the existing stairway, but it shall not be less than 4 ft between handrails. The handrail-gripping surface shall be a minimum of 1½ in. from the adjacent surface of a wall, etc.

410-1.02(04) Use of Earth-Retaining Structure versus Steep but Maintainable Lawn

A retaining wall shall be considered along a street or highway to provide a more level lawn or to retain an embankment due to space restrictions, etc. Where the wall height is less than 2.8 ft above the adjacent ground, the property owner shall be consulted to determine if he or she prefers a steeper, but maintainable, lawn slope in lieu of a retaining wall. A maintainable slope shall not be steeper than 3:1. A stairway or, if necessary, an alternate ADA-compliant pedestrian-access route is still required.

410-1.02(05) Urban- or Rural-Area Considerations

Determination shall be made as to whether the wall will be placed in an urban or rural setting.

An urban setting is one generally dominated by structures with a variety of colors, textures, and architectural styles. The surrounding landscape is often more orderly and manicured, and involves incorporated areas.

A rural setting is more natural, may include agricultural or forested areas, and generally involves unincorporated areas.

1. **Aesthetic Treatment of Wall in Urban Area.** Aesthetic treatment shall be considered for a wall placed in an urban area. The large volume of users, as well as adjacent land owners, who view such a structure are increasingly demanding that it be aesthetically treated so as to reduce negative visual impacts that can result. See Figure 410-1A, General Aesthetic Guidelines for Retaining Wall in Urban Area.

2. **Aesthetic Treatment of Wall in Rural Area.** The extent of aesthetic treatment for a wall placed in a rural setting is dependent upon further classification. A rural highway can be classified as either a commercial or scenic route.
a. A commercial route carries high levels of commercial traffic, medium levels of commuter traffic and medium to low levels of tourist traffic. It can be either 2 lanes, or 4 lanes divided or undivided. It requires minimal aesthetic treatment.

b. A scenic route carries high levels of tourist traffic, and medium to low levels of commercial and commuter traffic. It is highly scenic, and passes through, links, or is adjacent to parks, tourist areas, recreational areas, or historic areas. It can be either 2 lanes, or 4 lanes divided or undivided. Priority will be given to the recreational driving experience and aesthetic treatment.

See Figure 410-1B, General Aesthetic Guidelines for Retaining Wall in Rural Area.

### 410-1.02(06) Miscellaneous Factors

Other factors to consider are as follows.

1. A wall shall not dominate the area of effective vision of the driver.
2. The wall shall be used to accommodate the mounting of necessary lighting fixtures.
3. The walls shall be extended to meet overpasses and bridge abutments.
4. The wall elevation shall follow the natural grade of the land.
5. The ends of the wall shall be tapered to meet adjacent slopes.
6. The wall shall be aligned to follow adjacent landforms or as required by roadway alignment.
7. Where possible, wall alignment shall be varied.
8. Backfill slopes shall not exceed 2:1 for revegetation.
9. Drainage shall be provided at the base of an upslope wall.
10. For consistency, fixtures, wall finishes, patterns, line, form, color, and texture shall be coordinated and repeated to emphasize continuity for the entire transportation system within a given locality.
410-1.03 Earth-Retaining System Classification

For a highway application, an earth-retaining system can be used for a grade separation, bridge abutment, slope stabilization, or excavation support. A system can be designed to provide adequate lateral support. However, the most important factors in design are cost and efficiency. Therefore, common wall systems are classified based on the factors that will govern their selection and use.

Earth-retaining systems are classified as shown in Figure 410-1C according to construction method such as fill or cut, and the mechanisms of lateral load support such as externally stabilized or internally stabilized. Fill wall refers to bottom-up construction. Cut wall refers to top-down construction. Using Figure 410-1C, each wall system is classified in two manners. For example, a soldier pile and lagging wall is classified as an externally-stabilized cut-wall system. A mechanically-stabilized-earth wall is classified as an internally-stabilized fill-wall system.

410-2.0 PLANS-PREPARATION PROCEDURES

410-2.01 Wall Types

The earth-retaining systems that have been approved by INDOT for inclusion into project plans are categorized into two groups. For a Group 1 system, the designer shall develop a complete design and set of plan details. For a Group 2 System, the designer will make only a conceptual application. The designer will review a contractor-chosen propriety design after the letting, through working drawings and computations. Figure 410-2A provides a listing of systems in a fill section by classification and group category. Figure 410-2B provides such a listing of systems in a cut section.

410-2.02 Applications

The cast-in-place reinforced-concrete retaining wall will be considered the basic system for each application. Another system type may be considered if it is more economical or provides unique solutions to site-specific problems.

A Group 1 system is non-proprietary, while a Group 2 system is proprietary. A Group 1 system may be solely included in a project. A Group 2 system must have competitive alternatives to be included in a project.
Many earth-retaining systems have proprietary features. Some companies provide services including design assistance, preparation of plans and specifications for the structure, supply of the manufactured wall components, and construction assistance.

410-2.03 Design Procedure

An appropriate earth-retaining system design shall be developed in accordance with this chapter. The project manager shall be provided with documentation of these decisions prior to the geotechnical investigation. A copy of all correspondence and computations for each suggested earth-retaining system shall be included. A copy of these documents shall be submitted with the structure type and size plans.

410-2.03(01) Responsibility

The designer will be responsible for all design and detailing where a cast-in-place rigid, semi-gravity wall or a non-gravity cantilever wall is specified.

The designer will be responsible for the conceptual application, external stability, and review of the proprietary design for another type of earth-retaining system.

At the Field Check Plans submission, the designer will provide the Office of Geotechnical Services with a set of plans including cross sections and the information as follows:

1. top and bottom elevations;
2. beginning and end stations; and
3. stations of step locations in the bottom of the wall.

410-2.03(02) Design Methods


410-2.03(03) Wall-Selection Criteria

Other considerations in determining the acceptability of a particular earth-retaining system shall include the following:
1. geotechnical constraints;
2. future uses of the site;
3. differential deflection or settlement of wall sections;
4. project-specific special features;
5. long- and short-term wall stability;
6. comparable degree of safety;
7. accessibility to construction site;
8. staged-construction limitations;
9. right-of-way limits;
10. site-imposed physical limitations;
11. seismic activity;
12. wall inundation;
13. aesthetics;
14. economics;
15. environment; and
16. construction-time constraints.

The decision to select an earth-retaining system shall consider technical feasibility and its economy compared with a cast-in-place retaining wall. With respect to economy, the factors to be considered are as follows:

1. earthwork situation, cut or fill;
2. wall area;
3. average wall height;
4. foundation conditions;
5. availability and cost of select backfill material;
6. availability and cost of required right of way;
7. complex horizontal and vertical alignment changes;
8. need for a temporary excavation support system;
9. traffic maintenance during construction; and
10. aesthetics.

Each earth-retaining system has different performance histories, and this can create difficulty in adequate technical evaluation. Some systems are more suitable as a permanent wall, others are more suitable as a low-height wall; some are more applicable for a rural area, while others are more suited for a suburban area. The selection of the most appropriate system will thus depend on the specific project requirements. See Figure 410-2D, Wall Types and Classification of Earth-Retaining Systems; Figure 410-2A, Fill-Section Wall-System Selection Chart; and Figure 410-2B, Cut-Section Wall System Selection Chart, for system-selection guidelines.
The Office of Geotechnical Services shall be informed of each potential system to be considered for a project, so that it can provide site-specific recommendations.

410-2.03(04) Contract-Documents Requirements

1. Final Plans and Design Requirements. Plans for a conventional cast-in-place reinforced-concrete retaining wall or a permanent sheet-pile wall shall be fully detailed to include, but not be limited to, plan view, elevation view, sections as required, reinforcement schedules, detail clarification, allowable bearing pressure, and bill of materials.

Plans for another earth-retaining system shall include the project-specific information as follows:

a. beginning and ending wall stations;

b. elevations of top of wall at beginning and end of wall at 50-ft intervals, all profile break points, and roadways profile data at wall line;

c. original and proposed ground profiles in front of and behind the retaining wall;

d. cross sections at retaining-wall locations showing limits of excavation and backfill;

e. horizontal wall alignment;

f. details of wall appurtenances such as traffic barriers, copings, and drainage outlets;

g. the locations and configurations of signs and lighting including conduit locations;

h. right-of-way limits;

i. construction-sequence requirements including traffic control, access, and staged-construction sequences;

j. elevation of highest permissible level for foundation construction. Location, depth and extent of all unsuitable material to be removed and replaced;

k. quantities table showing estimated wall area and quantities of appurtenances and traffic barriers;
l. elevations of bearing pads, locations of bridge seats, skew angle, and all horizontal and vertical survey control data at abutments including clearances and details of abutments;

m. extreme high water and normal water levels at stream locations;

n. allowable soil bearing pressure for a retaining wall with reinforced backfill;

o. magnitude, location, and direction of external loads due to bridges, overhead signs or lights, and traffic and slope surcharges.

p. limits and requirements for drainage features beneath, behind, or through the earth-retaining structure;

q. special facing-panels treatments and module finishes or colors; and

r. critical soil properties that do not satisfy the minimum requirements set out in the INDOT Standard Specifications.

The plans shall be sealed and signed by a professional engineer. Such engineer will be responsible for the complete design of a cast-in-place concrete retaining wall or a permanent sheet-pile wall, and for the conceptual application and location of another earth-retaining system.

The maximum factored applied bearing pressure shall be calculated and compared to the factored soil-bearing resistance recommended in the geotechnical report. If the recommended factored soil-bearing resistance is less than the maximum factored applied bearing pressure, the Office of Geotechnical Services shall be contacted for additional guidance.

The limits for establishing pay quantities for each wall-system group shall be as shown in Figure 410-2C.

2. Special Provisions. A unique special provision shall be provided for an earth-retaining system not included in the INDOT Standard Specifications or the recurring special provisions. See Section 19-3.0 for the unique-special-provision preparation and approval procedure.

The feasibility of using an earth-retention system depends on the existing topography subsurface conditions and soil and rock properties. A comprehensive subsurface
exploration program is required to evaluate site stability, settlement potential, need for drainage, etc., before repairing a slope or designing a new type of earth-retaining system.

410-3.0 LIMIT STATES, LOAD FACTORS, AND RESISTANCE FACTORS

410-3.01 Limit States

410-3.01(01) Service Limit State

An earth-retaining system shall be investigated for excessive vertical and lateral displacement, and overall stability, at the service-limit state. Tolerable vertical and lateral deformation criteria shall be developed based on the function and type of wall, anticipated service life, and consequences of unacceptable movements to the wall and potentially affected nearby structures, both structural and aesthetic. Overall stability shall be evaluated at the service-limit state using limiting equilibrium methods of analysis.

The requirements of LRFD 10.6.2.2, 10.7.2.2, and 10.8.2.2 shall apply to the investigation of wall movements. For an anchored wall, deflections shall be estimated in accordance with LRFD 11.9.3.1. For an MSE wall, deflections shall be estimated in accordance with LRFD 11.10.4. The effects of wall movements on adjacent facilities shall be considered in the development of the wall design.

410-3.01(02) Strength Limit State

An earth-retaining system shall be investigated at the strength-limit state for bearing resistance failure, lateral sliding, excessive loss of base contact, pullout failure of anchors or soil reinforcements, and structural failure.

410-3.01(03) Extreme Event Limit State

The Extreme Event Limit state shall be considered in the design. The applicable load combinations and load factors specified in LRFD Table 3.4.1-1 shall be investigated. Unless otherwise specified, all resistance factors shall be taken as 1.0 in investigating the Extreme Event Limit state.
410-3.02 Load Factors

Load factors and load combinations shall be as described in LRFD 11.5.5 and LRFD Tables 3.4.1-1 and 3.4.1-2. The maximum and minimum load factors specified in LRFD Table 3.4.1-2 shall be considered in performing the stability analysis on the earth-retaining system. All applicable load combinations shall be investigated. For maximum and minimum load-factors applications, see LRFD Figures C11.5.5-1 and C11.5.5-2.

410-3.03 Resistance Factors

In conducting wall-stability analysis, resistance factors for sliding and bearing resistance shall be as described in LRFD 10.5. Resistance factors for a permanent earth-retaining system shall be as specified in LRFD Table 11.5.6-1.

410-4.0 CAST-IN-PLACE REINFORCED-CONCRETE CANTILEVER FILL WALL

410-4.01 Foundation Information

A cantilever wall consists of a base slab or footing from which a vertical wall or stem extends upward. Reinforcement is provided in both members to supply resistance to bending. A cantilever wall can be founded on spread footings or on piles. Pertinent soils information on loading conditions, foundation considerations, consolidation potential, and external stability is included in the geotechnical report.

Installation of structure backfill material behind a cantilever wall shall be with 1:1 backfill slopes. If site restrictions do not allow for the use of 1:1 structure-backfill slopes, a memorandum shall be submitted to the Office of Geotechnical Services requesting soil properties at the site. The memorandum shall be submitted at the Preliminary Field Check stage if possible. If 1:1 backfill slopes are not being used, more vigorous design methods shall be used.

410-4.01(01) Overall Stability

The overall stability shall be evaluated using limiting methods of analysis as described in LRFD 11.6.2.3.
410-4.01(02) Bearing Resistance

Bearing resistance shall be investigated at the Strength Limit state using factored loads and resistances, assuming the soil-pressure distributions as follows:

1. **Wall Supported with a Soil Foundation.** The vertical stress shall be calculated assuming a uniformly-distributed pressure over an effective base area as shown in *LRFD* Figure 11.6.3.2-1. *LRFD* Equation 11.6.3.2-1 shall be used to calculate vertical stress.

2. **Wall Supported with a Rock Foundation.** The vertical stress shall be calculated assuming a linearly-distributed pressure over an effective base area as shown in *LRFD* Figure 11.6.3.2-2. If the resultant is within the middle one-third of the base, *LRFD* Equations 11.6.3.2-2 and 11.6.3.2-3 shall be used. If the resultant is outside the middle one-third of the base, *LRFD* Equations 11.6.3.2-4 and 11.6.3.2-5 shall be used.

410-4.01(03) Limiting Eccentricity, or Overturning

In investigating wall overturning, vertical moments shall be taken about the centerline of the spread footing. Horizontal moments shall be taken about the bottom of the spread footing. The vertical effect of surcharge acting above the footing shall not be included in considering overturning. For a foundation on soil, the location of the resultant of the reaction forces shall be within the middle one-half of the base width. For a foundation on rock, the location of the resultant of the reaction forces shall be within the middle three-fourths of the base width.

410-4.01(04) Sliding Resistance

The requirements of *LRFD* 10.6.3.4 will apply.

The factored resistance against failure by sliding shall be taken as

\[ RR = \phi R_n = \varphi_t R_t + \varphi_{ep} R_{ep} \]

If the soil beneath the footing is cohesionless, the nominal sliding resistance between soil and foundation shall be taken as

\[ R_t = V \tan \delta \]
Where

\[
\tan \delta = \tan \Phi_f \text{ for concrete cast against soil, or }
\]

\[
0.8 \tan \Phi_f \text{ for a precast-concrete footing.}
\]

For a footing on clay, the sliding resistance shall be taken as the lesser of the following:

1. the cohesion of the clay, or

2. where the footing is supported on at least 6 in. of compacted granular material, one-half the normal stress on the interface between the footing and the soil.

For a spread footing on rock, the footing shall be embedded into the rock a minimum of 6 in.

**410-4.01(05) Passive Resistance**

Passive resistance shall be neglected in wall-stability computations. Passive resistance may be considered only if the wall extends below the depth of maximum scour, freeze-thaw effect, or other disturbances. Only the embedment below the greater of these depths shall be considered.

**410-4.01(06) Structural Design**

The structural design of individual wall elements and the wall foundation shall be as described in LRFD Sections 5, 6, 7, and 8. Flexural and shear design is based on LRFD load factors of 1.35 for vertical earth pressure, 1.50 for lateral earth pressure, and 1.75 for lateral earth pressure from live-load surcharge. Concrete shall be class B for the footing and class A for the stem. Reinforcing-steel yield strength shall be 60,000 psi. Reinforcement required to resist the formation of temperature and shrinkage cracks shall be as described in LRFD 5.10.8.

**410-4.01(07) Seismic Design**

For a gravity or semi-gravity retaining wall, the pseudo-static approach developed by Mononobe and Okabe may be used to estimate the equivalent static forces of seismic loads. This method is described in LRFD Appendix A11.
410-4.01(08) Drainage

Backfill behind a retaining wall shall be properly drained. If drainage is not provided, the wall shall be designed for loads due to earth pressure, plus full hydrostatic pressure due to water in backfill.

If a wall is adjacent to a roadway or sidewalk, pipe drains shall be placed in back of the wall instead of weep holes. A pipe underdrain of 6 in. dia. shall be used, with the flow line at the bottom of a square course of fine aggregate, 2-ft by 2-ft. This system shall be discharged into a storm sewer or ditch. For rehabilitation of an existing retaining wall, plan details shall be developed to replace inadequate pipe underdrain systems. A minimum slope of 0.5% shall be used for pipe underdrains.

410-4.02 Stem Design

The criteria to be considered in designing the stem are as follows.

1. For stem height of at least 16 ft through 26 ft, the back face shall be battered 12V:1H. The rear face can be battered depending on the site requirements.

2. The minimum stem thickness is 12 in. for a stem with a constant thickness. The minimum stem thickness at the top is 10 in. for a battered stem. Stem thickness at the bottom is based on the load requirements or batter.

3. Stem height is determined from site conditions.

4. The stem shall be located so as to produce the most economical footing.

5. Shear stress in the wall shall be checked at the base of the stem.

6. No. 4 reinforcing bars spaced at 1’-6” shall be placed in the front of the stem as longitudinal and vertical temperature reinforcement.

7. Moment shall be determined at the base of the stem and where required for bar cutoffs.

8. Loads due to a railing or parapet on top of the wall shall not be applied simultaneously with loads from earth pressure. These are dynamic loads which are resisted by the mass of the wall and passive earth pressure.

9. For a wall within roadway clear zone, the stem shall be shielded with a TL-5 barrier, or designed to withstand the LRFD collision force.
410-4.03 Footing Design

For footing-design criteria, see Chapter 408. Additional criteria to be considered in designing the footing are as follows.

1. Minimum footing thickness is 1.5 ft for a spread footing, or 2 ft for a pile footing.

2. The bottom of the footing shall be placed at 3 ft below the finished ground line. If the finished ground is on a grade, the bottom of the footing shall be sloped to a maximum grade of 5%. If the grade is steeper than 5%, the footing shall be placed level and steps shall be used.

3. Maximum pile spacing in each row is 10 ft.

4. Maximum pile batter is 4V:1:H.

5. Piles shall be embedded 1 ft into the footing. Reinforcing steel shall be placed on the tops of the piles.

6. For a spread footing, reinforcing steel shall be placed with a 4 in. clearance from the bottom of the footing. The edge clear distance shall be 2 in.

7. The footing moment shall be determined at the face of the stem based on vertical loads and resultant soil pressure.

8. A design for heel moment without considering the upward soil or pile reaction is not required unless such a condition actually exists.

9. For the toe, shear shall be determined at a distance from the face of the stem equal to the effective distance, \(d\), of the footing. For the heel, shear shall be determined at the face of stem.
410-4.04 Shear-Key Design

The criteria to be considered in designing the shear key are as follows.

1. The key shall be placed in line with the stem except under severe loading conditions.

2. The key width shall be 1 ft. The minimum key depth is 1 ft.

3. The key shall be placed in unformed excavation against undisturbed material.

4. The key shall be analyzed for the forces shown in Figure 410-4A, Factor of Safety Against Sliding for Spread Footing – Example.

5. The shape of a shear key in rock is determined from the site conditions.

410-4.05 Miscellaneous Design Information

Optional transverse construction joints are permitted in the footing. Footing joints shall be offset a minimum of 1 ft from the wall joints. Reinforcing steel shall be placed through the footing joints.

A vertical expansion joint shall be provided at intervals not exceeding 90 ft for a conventional retaining wall, as indicated in LRFD 11.6.1.6.

410-5.0 RETAINING WALL WITH GROUND REINFORCEMENT, OR FILL WALL

410-5.01 Mechanically-Stabilized-Earth (MSE) Wall

An MSE wall is a cost-effective alternative where a reinforced-concrete or gravity-type wall has traditionally been used to retain earth. This includes a bridge abutment and wingwalls, or an area where the right of way is restricted, such that an embankment of cut backslope with stable side slopes cannot be constructed. It is suited to economical construction in steep-sided terrain, ground subject to slope instability, or an area where foundation soils are poor. An MSE wall is not suitable for some applications as listed in the AASHTO LRFD Bridge Design Specifications.

Some additional uses of an MSE wall include the following:

1. a temporary structure which has been cost-effective for a detour necessary for a highway reconstruction project; or
2. phased construction.

The relatively small quantities of manufactured materials required, rapid construction, and competition among the developers of different proprietary systems has resulted in a cost reduction relative to traditional types of retaining walls. An MSE wall is likely to be more economical than another wall system for a wall height of about 10 ft or where special foundations are required for a conventional wall.

One advantage of an MSE wall is its flexibility and capability to absorb deformations due to poor subsoil conditions in the foundations. Also, based on observations in seismically-active zones, this type of structure has demonstrated a higher resistance to seismic loading than a cast-in-place concrete structure.

Precast-concrete facing elements can be made with various shapes and textures, with little extra cost, for aesthetic considerations. Masonry units, timber, and gabions can also be used with advantage to blend into the environment.

410-5.01(01) Advantages and Disadvantages

1. Advantages.
   a. uses simple and rapid construction procedures and does not require large construction equipment;
   b. does not require experienced craftsmen with special skills for construction;
   c. requires less site preparation than another alternative;
   d. requires less space in front of the structure for construction operations;
   e. reduces right of way acquisition;
   f. does not require rigid, unyielding foundation support because an MSE structure is tolerant to deformations;
   g. it is cost-effective; and
   h. it is technically feasible to a height in excess of 80 ft.

2. Disadvantages.
a. requires a relatively large space behind the wall or outward face to obtain enough wall width for internal and external stability;

b. requires select granular fill. At a site where there is a lack of granular soils, the cost of importing suitable fill material can render the system uneconomical;

c. suitable design criteria are required to address corrosion of steel reinforcement elements and deterioration of certain types of exposed facing elements, such as geosynthetics, to ultraviolet rays and potential degradation of polymer reinforcement in the ground; and

d. the design can require a shared design responsibility between material suppliers and the owner, and greater input from geotechnical specialists in a domain often dominated by structural engineers.

410-5.01(02) Constraints and Conditions

The primary environmental condition affecting reinforcement type selection and potential performance of an earth-retaining structure with reinforced backfill is the aggressiveness of the backfill material that can cause deterioration to the reinforcement.

The lower limit to height is usually dictated by economy. Where used with a traffic barrier, a low wall on a foundation of less than 10 to 13 ft is often uneconomical, as the cost of the overturning moment leg of the traffic-barrier approaches one-third of the total cost of the MSE structure in place. For a cantilever retaining wall, the barrier is simply an extension of the stem with a smaller impact on overall cost.

The total size of structure, or area of facing elements, has little impact on economy compared with other retaining-wall types. However, the unit cost for an MSE wall of less than 3000 ft\(^2\) is likely to be 10 to 15% higher.

410-5.01(03) Relative Costs

Site-specific costs of an MSE wall are a function of factors including cut-fill requirements, wall size and type, in-situ soil type, available backfill materials, facing finish, or temporary or permanent application. An MSE wall with a precast-concrete facing is usually less expensive than a reinforced-concrete retaining wall for a height of greater than about 10 ft and average foundation
conditions. A modular-block wall is competitive with a concrete retaining wall at a height of less than 15 ft.

410-5.01(04) Description of MSE-Wall System

Figure 410-5(0)C shows the typical cross section of an MSE wall.

1. Systems Differentiation. Since the expiration of the fundamental process and concrete-facing-panels patents obtained by the system’s first proprietary manufacturing company, the engineering community has adopted the generic term mechanically-stabilized-earth to describe this type of retaining wall construction.

A system for an MSE-wall structure is defined as a complete supplied package that includes design, specifications, and all prefabricated materials of construction necessary for the complete construction of a soil-reinforced structure. Technical assistance during the planning and construction phase is also included.

2. Ground Reinforcement. An MSE-wall system can be described from the reinforcement geometry, stress transfer mechanism, reinforcement material, and the type of facing and connections.

a. Reinforcement Geometry. The types of reinforcement geometry that can be considered are as follows.

(1) Linear Unidirectional. These include strips, including smooth or ribbed steel strips.

(2) Composite Unidirectional. These include grids or bar mats characterized by grid spacing greater than 6 in.

(3) Planar Bidirectional. These consist of continuous sheets of welded wire reinforcement, or woven wire mesh. The reinforcement or mesh is characterized by means of element spacing of less than 6 in.

b. Reinforcement Material. Reinforcement material consists of metallic reinforcements, typically of mild steel. The steel is galvanized or epoxy coated. Where non-metallic reinforcement material must be used, it shall be inextensible, and it must be approved by INDOT.

c. Reinforcement Extensibility. The classes of extensibility are as follows.
(1) Inextensible. The deformation of the reinforcement at failure is much less than the deformability of the soil. INDOT permits only inextensible reinforcement in an MSE-wall system.

(2) Extensible. The deformation of the reinforcement at failure is comparable to or greater than the deformability of the soil.

3. Facing Systems. The types of facing elements used in MSE systems control their aesthetics because they are the only visible parts of the completed structure. A wide range of finishes and colors can be provided in the facing. The facing provides protection against backfill sloughing and erosion, and can provide a drainage path. The type of facing influences settlement tolerances. The facing types are as follows.

a. Segmental Precast-Concrete Panels. These have a minimum thickness of 5½ in., and are of cruciform, square, rectangular, diamond, or hexagonal geometry. Temperature and tensile reinforcement are required, but will vary with the size of the panel. Vertically-adjacent units are connected with shear pins.

b. Welded-Wire Grids. Wire grid can be bent up at the front of the wall to form the wall face. This type of facing is used mainly for a temporary structure.

c. Gabions. Gabions, or rock-filled wire baskets, can be used as facing with reinforcing elements consisting of welded wire reinforcement, welded bar-mats, geogrids, geotextiles, or the double-twisted woven mesh placed between or connected to the gabion baskets.

d. Post-Construction Facing. For a wrapped faced wall, the facing, whether geogrid, geotextiles, or wire reinforcement, can be attached after construction of the wall by means of shotcreting, or placing cast-in-place concrete or other materials. This approach adds cost but is advantageous where significant settlement is anticipated.

Facings using welded wire or gabions have the disadvantages of uneven surface, exposed backfill materials, more tendency for erosion of the retained soil, possible shorter life from corrosion of the wires, and more susceptibility to vandalism. These disadvantages can, be countered by providing shotcrete or by hanging facing panels on the exposed face and compensating for possible corrosion. The advantages of such facings are low cost, ease of installation, design flexibility, positive drainage depending on the type of backfill that provides increased stability, and possible treatment of the face for vegetative and other architectural effects. The facing can be adapted and blended with a natural country
environment. The facings, and geosynthetic-wrapped facings, are advantageous for construction of a temporary or other structure with a short-term design life.

4. **Reinforced-Backfill Material.** An MSE wall requires structure backfill for durability, positive drainage, constructability, and soil-reinforcement interaction which can be obtained from structure backfill.

5. **Miscellaneous Construction Materials.** A wall with precast-concrete panels requires bearing pads in the horizontal joints that provide some compressibility and movement between panels and precludes concrete-to-concrete contact. Such materials shall be in concordance with the INDOT *Standard Specifications*.

All joints shall be covered with a polypropylene geotextile strip to prevent the migration of fine aggregates from the backfill.

### 410-5.01(05) Selection Criteria

Each topic described below shall be considered at the preliminary design stage. The appropriate elements and performance criteria shall be determined. The process consists of the successive steps as follows.

1. Consider all possible alternatives, and choose an earth-retaining system. Cantilever, gravity, semigravity, or counterforted concrete wall, or reinforced-soil slopes are the usual alternatives to an MSE wall and abutments.

2. In a cut situation, an in-situ wall such as a tieback anchored wall, soil-nailed wall, or nongravity cantilevered wall is often more economical. Where limited right of way is available, a combination of a temporary in-situ wall at the back end of the reinforcement and a permanent MSE wall is often competitive.

3. Consider facing options. The development of project-specific aesthetic criteria is principally focused on the type, size, and texture of the facing, which is the only visible feature of an MSE structure.

4. For a permanent application, an MSE wall with precast-concrete panels shall be considered. It is constructed with a vertical face. The precast-concrete panels can be manufactured with a variety of surface textures, colors, and geometrics.

5. At a more remote location, a gabion, timber faced, or vegetated MSE wall may be considered.
6. For a temporary wall, significant economy can be achieved with wire facings, geosynthetic wrapped facings, or wood-board facing. It can be made permanent by applying shotcrete or cast-in-place concrete in a post-construction application, provided that the wall design satisfies the criteria for a permanent wall.

7. Develop performance criteria for loads, design height, embedment, settlement tolerances, foundation capacity, effect on adjoining structures, etc. Performance criteria for an MSE structure with respect to design requirements are governed by design practice or the *LRFD Bridge Design Specifications* and the INDOT *Standard Specifications*.

8. Consider site effects on corrosion or degradation of reinforcement.

9. Consider site effects with regard to river banks or a floodplain area.

   a. River Banks or Floodplain Area. The top of the leveling pad shall be at least 1 ft above the ordinary high-water elevation. No. 8 stone shall be placed behind the wall instead of structure backfill up to the $Q_{100}$ high-water elevation.

   b. Wall Embedment. The minimum embedment depth to the top of the leveling pad shall be 3 ft, except for a structure founded on rock at the surface, where no embedment is required. A minimum horizontal bench width of 4 ft shall be provided in front of a wall founded on slopes. For a wall constructed along a river or stream where the depth of scour has been reliably determined, a minimum embedment of 2 ft below the $Q_{500}$ scour depth is recommended.

410-5.01(06) Design Criteria [Rev. May 2012, May 2013, Mar. 2017]

The recommend minimum resistance strengths with respect to failure modes are as follows.

1. **External Stability.** Sliding eccentricity, $e$, at base, plus bearing capacity, deep-seated stability, and seismic stability shall be checked based on *LRFD* 11.10.5.

   The design height of the wall, $Z$, shall be measured from the theoretical top of the leveling pad to a point above the top of the wall as calculated from the formula as follows:

   $$Z = H + L \tan \beta$$
Where:

\[ H = \text{height of the wall from the theoretical top of the leveling pad to the top of the coping}, \]
\[ L = \text{width of the reinforced zone}, \]
\[ \beta = \text{surcharge slope angle as measured from the top of the coping}. \]

See Figure 410-5(0)A.

2. **Internal Stability.** Pullout resistance shall be checked based on LRFD 11.10.6.
   
a. **Design Limits and Wall Height.** The length and height required to satisfy the project’s geometric requirements shall be established to determine the type of structure and external loading configurations.
   
b. **Length of Ground Reinforcement.** The minimum reinforcement length for an MSE wall is the greater of 0.7\( H \) or 8 ft. A greater length may be required for a structure subject to surcharge loads, or if the factored MSE-wall loads are more than the factored bearing resistance.
   
c. **External Loads.** The external loads can be surcharges required by the geometry, adjoining footing loads, line loads from traffic, traffic impact loads, or sound-barrier loads. Traffic live loads and impact loads are applicable where the traffic lane is located horizontally from the face of the wall within a distance of less than one half the wall height.
   
d. **Wall Embedment at End Bent or Longitudinal-Edge Encroachment where Stream Parallels Roadway.** The minimum embedment depth to the top of the leveling pad shall be 3 ft. However, for a structure founded on rock at the surface, no embedment is required. A minimum horizontal bench width of 4 ft shall be provided in front of a wall founded on slopes. Typical abutment scour-protection countermeasures will be required in front of the wall.

3. **Seismic Activity.** Due to an MSE wall’s flexibility, it is resistant to dynamic forces developed during a seismic event. See the LRFD Bridge Design Specifications for seismic-design considerations.

4. **Protection of MSE Wall Against Collision.** An MSE-wall bridge abutment placed adjacent to a roadway shall be checked for vehicle-collision forces as described in Section 403-3.07.
5. **Acute Angles.** Acute angles should be avoided because of construction difficulties, e.g. compaction in corners and placement of reinforcement. Where two intersecting walls form an enclosed angle, the angle is to be greater than or equal to 70 degrees.

6. **Wall Curves.** Sharp curves should be avoided in the wall layout. The curvature of a wall will impact the size of panel that can be provided. Typically, a 10-ft wide panel can accommodate a radius of 100 ft. and a 5-ft wide panel can accommodate a radius of 50 ft.

7. **Utilities.** Utilities should not be placed through the reinforced zone. Where utility placement in the reinforced zone is unavoidable, future access must be provided to the utility without disrupting the reinforcement. The breakage or rupture of the utility must not have a detrimental effect on the stability of the MSE wall.

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**410-5.01(07) Information to be Shown on Plans [Rev. Dec. 2012, Mar. 2017]**

**Wall Envelope.** The wall envelope should be determined from the plans’ elevation view with three control lines. Control Line 1 defines the elevation of the top of coping, or wall, if no coping is used. Control Line 1 should be shown on the elevation view with stations and elevations in conjunction with cross-section locations. It should be located on the back face of the MSE wall or coping. Control Line 2 defines the elevation of the existing or proposed ground line in front of the wall. Control Line 3 defines the elevation of the top of the leveling pad. It is obtained by offsetting a minimum distance of 2 ft below the proposed ground line in front of the wall to the top of the leveling pad. All control lines should be shown and identified as such on the plans. Control Lines 1 and 3 should also be labeled as neat lines.

The minimum area required for the wall to be constructed should be defined by means of an envelope. The limits of the envelope are the beginning- and end-of-wall stations and the locations of Control Lines 1 and 3. From this information, a wall-elevation view along the front face of the wall showing leveling-pad and step locations, elevations, and dimensions should be prepared and shown on the plans as conceptual information for the contractor. The minimum area within the envelope described above should be the pay quantity for the wall. Figure 410-5(0)B shows the difference between the minimum area required and an estimated amount of additional surface area required to construct the wall based on the wall-panel sizes and leveling-pad step increments described below. The area below Control Line 3 is conceptual information for the contractor and should not be included in the panels’ pay quantity because it can vary depending on the wall system the contractor chooses. Pay quantities for each wall should be shown on the plans.

The plans should show the minimum height from the top of the leveling pad to the existing or proposed ground line, as required. The plans should also show all stations and offsets relative to the survey centerline on the back face of the wall for the beginning and ending points, and all such
offsets for turn-point locations where the wall forms an angle. Leveling-pad steps should be in 2.5-ft increments. The bottom of the pad should be level.

**Drainage.** Details for drainage of the surface-water infiltration and reinforced-soil backfill should be included for all MSE walls. Figure 410-5(0)C shows the standard drainage details. It is the designer’s responsibility to determine the elevation of the drainage pipe such that it will drain and outlet adequately.

**Wall Panels.** Panels of 10-ft length by 5-ft height should be assumed. The top of the wall or coping may be sloped. The standard panel thickness should be taken as 6 in. The decorative panel thickness should be taken as 9 in. Panel sizes and wall thickness should not be shown on the plans, as the wall-system manufacturer will show these values on the working drawings.

**Elevation View.** An elevation view should show and label all obstructions that project through an MSE wall panel by station and elevation. A section view may be considered to provide additional details as needed. The beginning and ending locations should be checked to determine where the final grading elevations are equal both in front of and behind the wall, whereby the wall is no longer required.

**Plan View.** A plan view on the MSE Wall Details sheet should show and label obstructions and their offset from the back of the wall panel. Obstructions include but are not limited to, piles, pile sleeves, catch basins, signal or sign foundation, guardrail posts, and culverts. Where obstructions cannot be avoided, the wall-system designer must modify the wall design using one of the methods in LRFD 11.10.10.4. Details to avoid obstructions must be shown in the MSE wall working drawings.

**MSE Wall at an End Bent.** When an end bent is placed behind an MSE wall, expanded polystyrene should be shown for gap between the front face of the end bend and the back of the MSE wall. Do not show Styrofoam or extruded polystyrene. See Figure 409-2G, End Bent Placed behind MSE Wall for additional details.

**410-5.02 Modular-Block Facing Units with Reinforced Backfill**

A concrete modular-block retaining wall is a Group 2 system. The maximum height shall be 15 ft, measured from the top of the leveling pad to the top of the wall. See Figure 410-5A for a modular-block-wall typical section.

A concrete modular-block retaining wall is constructed from blocks which are available in a variety of facial textures and colors, providing a variety of aesthetic appearances. See Figure 410-5B, Types of Modular Blocks. They range in facial area from 0.5 to 1 ft². An integral feature of the facing is a front batter ranging from nominal to 15 deg. The shape of the blocks permits the wall
to be built along a curve, either concave or convex. The blocks are dry-stacked, therefore mortar or grout is not used to bond the units together, except for the top two layers.

410-5.02(01) Design Procedure

1. **Earth-Pressure Considerations.** The backslope is either horizontal such that \( B = 0 \) deg, or with sloping backfill such that \( B > 0 \) deg. The modular-block wall can be vertical such that \( A = 0 \) deg, or with setback such that \( A > 0 \) deg.

The angles \( A \) and \( B \) are shown in Figure 410-5C or 410-5D.

For a wall with a setback of \( 0 \) deg, the active earth pressure coefficient for external stability, \( K_a \), can be determined from Equation 410-5.1, Rankine’s formula.

\[
K_a = \left( \cos B \right) \frac{\cos(B - X)}{\cos(B + X)} \tag{Equation 410.5-1}
\]

where \( X = \sqrt{\cos^2 B - \cos^2 \phi_r} \)

For a wall with a setback of \( 1 \) deg, \( K_a \) can be determined from Equation 410-5.2, Coulomb’s formula, with \( \delta = 0 \).

\[
K_a = \frac{\cos^2 (\phi_r + A)}{\cos^2 A [\cos(A - \delta)] [1 + \sqrt{Z/Y}]^2} \tag{Equation 410-5.2}
\]

where \( \phi_r = \) angle of internal friction of the retained soil (from geotechnical report)

\( A = \) wall setback angle from vertical

\( \delta = \) interface friction angle between reinforced soil zone and retained soil, which shall be taken as \( 0 \) deg

\( B = \) backslope angle (see Figure 410-5C or 410-5D)

\( Z = \sin(\phi_r + \delta) \sin(\phi_r - B) \)

\( Y = \cos(A - \delta) \cos(A + B) \)

2. **External Stability.**

   a. **Analysis of Overturning.** Eccentricity, \( e \), at the base shall be checked based on LRFD 11.10.5.
b. Analysis of Sliding. Sliding at the base shall be checked based on LRFD 11.10.5. Sliding shall also be checked at the level of the first geogrid from the bottom using the geogrid coefficient of direct sliding, but including the shear strength between modular-block units. If the geogrid coefficient of direct sliding is unknown, use 0.65 tan θ.

c. Analysis of Soil Bearing Pressure. The bearing pressure at the bottom of the reinforced-soil mass and blocks, BP, is determined by using the Meyerhof stress distribution.

\[
BP = \frac{R}{L_2 - 2e} \\
(Equation 410-5.3)
\]

where \( e \) is determined by taking moments about the center of the base length \( L_2 \).

\[
e = 0.5hH_1 \cos C + 0.33hH_2 \cos C - 0.5H_1 \sin C (L_2 + h \tan A) - H_2 \sin C - 0.5V_1 (h + H) \tan A - 0.5V_1 W_w - V_2 (H \tan A + 0.67L + W_w - 0.5L_2) - \frac{2R}{V_2 H \tan A}
\]

BP ≤ Factored bearing resistance

The factored bearing resistance is provided by the Office of Geotechnical Services.

If a break in the slope behind the wall is located horizontally within a distance of \( 2H \), broken-back backfill may be used. If the break is located at \( 2H \) or greater from the wall, a horizontal backslope shall be used.

The only difference between broken-back backfill and horizontal backslope is the magnitude of forces \( H_1 \) and \( H_2 \). The magnitude of these forces is a function of \( K_a \), which is shown at the beginning of the design procedure. Both procedures use this formula for \( K_a \). However, for broken-back backfill, angle \( I \) shall be substituted for angle \( B \). Where the break in the slope behind the wall is located \( 0.5H \) from the back face of the reinforced soil mass, live load surcharge, Sur, shall be considered in the design. If the break is located at \( 0.5H \) or greater from the back face of the reinforced soil mass, Sur shall not be considered in the design.

The failure plane for a modular-block wall with geogrid, or extensible, reinforcement is defined as a straight line passing through the heel on the retained-earth side of the lowermost bock at an angle \( \alpha \) from the horizontal. The angle \( \alpha \) is calculated from Equation 410-5.4.
\[
\tan(\alpha - \phi) = \frac{x(x + y)
\left[1 + y \tan(\delta - A)\right]}
{x + y\left[1 + \tan(\delta - A)\right]} \tag{Equation 410-5.4}
\]

where
\[X = \tan(\phi_i - B)\]
\[Y = \cot(\phi_i + A)\]

\(\phi_i\) = angle of internal friction of reinforced infill soil
\(\delta\) = angle of friction at back of wall, assume 2/3 \(\phi_i\)

See Figures 410-5E and 410-5F for definitions of \(A\), \(B\), and \(\dot{a}\).

The failure plane for broken-back backfill and horizontal backslope with extensible reinforcement is based on angle \(\dot{a}\) as shown above.

The horizontal stress, \(\sigma_h\), at each reinforcement level for extensible reinforcement can be computed by multiplying the vertical stress, \(\sigma_v\), at that level, by the active earth pressure coefficient \(K_a\).

\[K_a = \frac{\cos^2(\phi_i + A)}{\cos^2 A \cos(\delta - \delta) \left[1 + \sqrt{Z/Y}\right]^2} \tag{Equation 410-5.5}\]

where
\[Z = \sin(\phi_i + \delta) \sin(\phi_i - B)\]
\[Y = \cos(A - \delta) \cos(A + B)\]

\(\phi_i\) = peak angle of internal friction of the reinforced soil zone, or 34 deg
\(\delta\) = interface friction angle which is assumed to be two-thirds of the angle of internal friction of the reinforced infill soil, or 22.7 deg

The vertical stress, \(\sigma_v\), is based on the vertical loads being distributed over a length determined by the Meyerhoff formula. The same procedure shall be applied to calculate the maximum bearing pressure at the bottom of the reinforced-soil mass shown in the external-stability equations. The same equations can be used, except \(h\) and \(H\) shall be decreased by the distance from the top of the leveling pad to the level of the extensible reinforcement where vertical earth pressures are being calculated. If this procedure results in \(R\) appearing to the right of center of \(L_2\) (see Figure 410-5D), then calculate \(\sigma_v\) based on the height of overburden plus surcharge at the center of the contributing area, \(L_a\), for the geosynthetic reinforcement being considered. The values of \(\delta_h\) and \(L_a\) are shown in Figure 410-5F.
3. **Internal Stability.**

a. **Soil-Reinforcement Forces.** For both extensible and inextensible reinforcements, the surcharge shall be included for stress calculations. The force in the soil reinforcement is determined at the location of the failure plane as follows:

\[
R_e = 0.5ZH_1 \cos C + 0.33ZH_2 \cos C - H_1 \sin C \left[0.5L_2 + (H - 0.5Z)\right] \tan A - H_2 \sin C \left[0.5L_2 + (H - 0.67Z)\right] \tan A - V_1 (0.5L_2 + H \tan A - 0.5L) - V_2 (H - 0.5Z) \tan A
\]

(Equation 410-5.6)

where:

- \(V_1 = 0.5LW_iH\)
- \(H_1 = ZSurK_aW_r\)
- \(\delta = \) external friction angle, the lesser of \(\phi_i\) or \(\phi_r\)
- \(\phi_i = \) internal friction angle of reinforced infill soil
- \(\phi_r = \) internal friction angle of retained soil
- \(C = \delta - A [C \text{ cannot exceed } B.]\)
- \(H_2 = 0.5ZK_aW_r\)
- \(K_a\) (see Equation 410-5.5)
- \(W_r = \) unit weight of retained soil
- \(L = \) length of soil reinforcement
- \(V_1 = LSurW_i\)
- \(V_2 = ZLW_i\)
- \(W_i = \) Unit weight of reinforced infill soil

\[
\sigma_v = \frac{R}{L - 2e}
\]

(Equation 410-5.7)

If an alternate method is required to calculate \(\phi_v\),

\[
\sigma_v = W_i(d + Sur)
\]

(Equation 410-5.8)

where \(d\) and its location are shown in Figure 68-5F.

For extensible reinforcement, \(\sigma_h = \sigma_v K_a\), where \(K_a\) is based on Equation 68-5.4.

For extensible reinforcement,

\[
\sigma_h = \sigma_v K_a
\]

Multiplying \(\sigma_h\) by its contributing area will provide the force in the soil reinforcement, \(F_g\). This is the force in the reinforcement at the failure plane.
Because geogrid reinforcement is continuous, the contributing area is the vertical spacing, and the resulting force is on a per-foot basis.

The vertical forces $V_1$, $V_2$, and $V_3$, and horizontal forces $H_1$ and $H_2$ shall be determined using calculations accompanying the stability check. However, $V_3$, $H_1$, and $H_2$ are based on the soil plane above the reinforcement. The procedure outlined above shall be followed to find the force in the soil reinforcement.

The forces in the geogrid at the back face of the blocks and at the failure plane are assumed to be equal at the bottom of the wall. They vary linearly to a point at one-half the wall height where the force is equal to 85% of the force at the failure plane. For the upper half of the wall, the force at the back face of the blocks is assumed to equal 85% of the force at the failure plane.

The force $F$ in the geogrid is equal to $\sigma_h$ times the contributing area. Since geogrid reinforcement is continuous, the contributing height is used.

The connection strength between the geogrid and the blocks shall be determined from National Concrete Masonry Association Test Method SRWU-1. The service-state-condition strength shall be based on a deformation of the geogrid relative to the block, measured at the face of the blocks, or $\frac{1}{2}$ in. The connection strength used for design shall be the lesser of the peak-connection strength or the service-state-connection strength.

$$\frac{\text{Connection Strength}}{F} \geq 1.5$$

The allowable force, $F$, in the geogrid reinforcement shall be in accordance with the AASHTO LRFD Bridge Design Specifications. The values of Limit State tensile load, $T_l$, and serviceability state tensile load, $T_w$, as described in the AASHTO LRFD Bridge Design Specifications, shall be determined.

A factor of safety or reduction factor for extensible reinforcement with respect to environmental and aging losses, $FD$, and a factor of safety or reduction factor for extensible reinforcement with respect to construction damage, $FC$, are required. If project-specific test results are available, $FD$ shall be taken as 2.0. If tests are not available, $FD$ shall be taken as 1.1 minimum. If project-specific test results are available, $FC$ shall be taken as 3.0. If tests are not available, $FC$ shall be taken as 1.3 minimum. An overall factor of safety, $FS$, shall be taken as 1.78.
b. **Pullout Capacity of Extensible Reinforcement.** The pullout capacity is developed by extending the geogrid beyond the failure plane for a sufficient distance to develop a force \( F_U \), equal to 1.5\( F \). The minimum length of soil reinforcement is 0.7\( H \), 6 ft, or the distance to the failure plane plus 3 ft, whichever is greater. \( F_U \) shall be calculated as follows:

\[
F_U = 2\sigma_v L_A f_d \tan \phi_i
\]

(Equation 410-5.9)

where:
- \( \sigma_v \) = vertical stress, or 120\( \delta \) as shown in Figure 410-5E
- \( L_A \) = length of reinforcement beyond the failure plane
- \( f_d \) = equivalent coefficient of direct sliding derived from pullout tests
- \( \phi_i \) = angle of internal friction of the reinforced-soil zone, or 34 deg

Geogrid pullout can occur as a result of a combination of soil shearing on plane surfaces parallel to the direction of geogrid movement and soil bearing on transverse geogrid surfaces perpendicular to the direction of geogrid movement. Ultimate pullout capacity shall be based on a maximum elongation of the embedded geogrid of \( \frac{1}{2} \) in., measured at the leading edge of the compressive zone within the soil mass.

4. **Pay-Quantities Determination.** The surface area of modular-block units and wall erection to be shown on the plans is based on the neat-line limits of the wall-system envelope. The neat-line limits shall be considered as the vertical distance from the top of the leveling pad to the top of the modular-block units, and the horizontal distance from the beginning to the end of the leveling pad.

**410-5.02(02) Summary of Design Requirements**

1. **Blocks Data.**
   a. A block shall consist of one piece.
   b. Minimum thickness of front face = 4 in.
   c. Minimum thickness of internal cavity walls other than front face = 2 in.
   d. \( f_c' = 5000 \text{ psi} \)

2. **Traffic Surcharge.** Live load surcharge = 2 ft = 1.7 psi
3. **Retained Soil.**
   
a. Unit weight = 120 lb/ft³
b. Angle of internal friction, $\phi_i$, shall be determined from test information shown in the geotechnical report.

4. **Design Life.** Design life shall be 75 years minimum.

5. **Soil-Pressure Theory.** Either Coulomb’s or Rankine’s theory shall be used at the designer’s discretion.

6. **Soil Reinforcement.**
   
a. Shall be either inextensible or extensible.
b. Minimum length shall be 70% of the wall height, and not less than 6 ft.
c. Length shall be equal throughout the wall height.
d. Maximum vertical spacing between layers = 2 ft.
e. Shall extend a minimum of 3 ft beyond the failure plane.

### 410-5.03 Modular-Block Gravity Wall without Ground Reinforcement

The proprietary modular blocks used in combination with ground reinforcement can also be used as a pure gravity wall (see Section 410-4.02). The height to which it can be constructed is a function of the width of the blocks, the setback of the blocks, the backslope angle, and the angles of internal friction of the retained earth behind the wall. The base of the block wall shall be placed at least 3 ft below the finish-grade elevation. A wall of this type is limited to a height of 5 ft or less, and is limited to a maximum differential settlement of 1 in 200. However, a wall of this type shall not be used to support a highway or other structure.

Dry-cast modular-block wall units are relatively small, squat concrete units that have been designed and manufactured for retaining-wall application. The weight of a unit ranges from 30 to 100 lb, with units of 60 to 100 lb used for highway work. Unit height ranges from 4 to 8 in. Exposed-face length varies from 8 to 18 in. Nominal width, or dimension perpendicular to the wall face, of a unit ranges between 9 and 24 in. Units are manufactured solid or with cores. Full-height cores are filled with aggregate during erection. Units are dry-stacked without mortar in a running-bond configuration. Vertical adjacent units are interconnected to prevent sliding.

The material specifications for the blocks used for a gravity wall are identical to those for the blocks used for a modular-block wall with ground reinforcement.
The design of a modular-block gravity wall shall be in accordance with the project requirements and the procedures described herein.

The modular-block manufacturer shall check the wall for overturning and internal stability and make certain that the factored bearing-resistance requirements are satisfied. The Office of Geotechnical Services will check the wall for sliding, global stability, and settlement, and will provide the factored bearing resistance and the equivalent fluid pressure acting on the back of the wall.

The pay quantities shall be determined as described in Section 410-5.02(01) item 4.

410-5.03(01) Design Procedure

In designing a modular-block gravity wall without setback, the active-earth-pressure coefficient, $K_a$, can be determined from the Rankine formula.

In designing a modular-block gravity wall with setback, the active-earth-pressure coefficient, $K_a$, can be determined from the Coulomb formula. The interface friction angle between the blocks and soil behind the blocks shall be assumed to be zero.

The forces acting on a modular-block gravity wall are shown in Figure 410-5G, Modular-Block Gravity-Wall Analysis. The unit weight of the block shall be taken as 140 lb/ft$^3$. The unit weight of the drainage aggregate inside or between the blocks shall be taken as 120 lb/ft$^3$. Passive soil pressure is not permitted to resist sliding. Sheer between the blocks shall be resisted by friction, keys, or pins.

410-5.03(02) Design Considerations

1. **Overturning.** For overturning, moments are taken about the outside corner of the block. The vertical components of the soil pressure forces can be conservatively ignored.

2. **Sliding.** Sliding resistance is similar to that for a modular-block gravity wall with ground reinforcement, except that ground reinforcement is not provided for this type of wall.

3. **Bearing Pressure.** The maximum factored bearing pressure at the bottom of the lower block shall be less than or equal to the factored bearing resistance which is provided in the geotechnical report.
410-6.0 PREFABRICATED MODULAR GRAVITY WALL, OR CUT WALL

410-6.01 General

The design of a prefabricated modular gravity wall shall be in accordance with the AASHTO LRFD Bridge Design Specifications. The requirements of LRFD 11.11 will apply.

The use of this type of wall shall be coordinated with the Office of Materials Management. This type of wall consists of proprietary modular structural elements and the fill material within these elements.

The approved prefabricated modular gravity wall types are as follows:

1. prefabricated concrete modular gravity wall, bin type, without ground reinforcement;
2. metal binwall; and
3. gabion wall.

410-6.02 Advantages and Limitations

A modular gravity wall may be used where a conventional cast-in-place gravity, cantilever, or counterfort concrete retaining wall will be considered. In addition to the cost comparison, the advantages and limitations in selecting a prefabricated modular gravity wall shall be investigated as an earth-retaining system.

410-6.02(01) Advantages

The advantages are as follows:

1. low construction cost;
2. fast and easy construction;
3. no temperature effect on the erection;
4. flexibility and tolerance to differential settlement; and
5. units can be economically disassembled and re-used.
410-6.02(02) Limitations

The limitations are as follows.

1. It shall not be used with a radius of less than 800 ft, unless the curve can be substituted with a series of chords.
2. A steel modular system shall not be used where the groundwater or surface runoff is acid-contaminated, or where deicing spray is anticipated.
3. Available systems are proprietary.
4. Limited to a bottom-up construction method.
5. Corrosion of exposed metallic bins, and wires used in gabion walls, is possible.
6. Storage space is required at the site for relatively large prefabricated units.
7. Possible unavailability of required fill material, especially for a gabion wall.
8. Some systems are susceptible to vandalism.

410-6.03 Drainage

Measures shall be taken to drain the material within and behind the modules or bins using underdrain pipe. The pipe shall be wrapped in a geotextile fabric.

410-6.04 Design Considerations

The design shall be in accordance with the *LRFD Bridge Design Specifications*. Similar to a conventional retaining wall, a prefabricated modular gravity wall shall be evaluated for sliding, overturning due to limiting eccentricity, and bearing resistance at the Strength Limit state. Wall settlement shall be evaluated at the Service Limit state.

410-6.04(01) Load Factors and Load Combinations

Load factors and load combinations shall be as specified in *LRFD* Tables 3.4.1-1 and 3.4.1-2. Maximum and minimum load factors as specified in *LRFD* Table 3.4.1-2 shall be considered in performing a stability analysis. All applicable load combinations shall be investigated.
410-6.04(02) Resistance Factors

In conducting a wall-stability analysis, resistance factors for sliding and bearing resistance shall be as specified in *LRFD* 10.5. In conducting the structural design for the modular-structural-wall elements, resistance factors shall be as described in *LRFD* Section 5 for concrete members, or Section 6 for steel members.

410-6.04(03) Lateral Earth Pressures

The magnitude of active earth pressure and the location of resultant loads shall be as shown in *LRFD* Figures 3.11.5.9.1 and 3.11.5.9-2. Where the back face of the modules forms an irregular shape, or stepped surface, the earth pressure shall be computed on a plane surface drawn from the upper back corner to the lower back corner of the wall. For stability analysis of sliding and overturning, the system shall be assumed to act as a rigid body. Passive earth pressure shall be neglected in stability calculations, unless the base of the wall extends below the depth of maximum scour, freeze-thaw, or other disturbance. Lateral earth pressures due to live-load surcharge and earth-loading surcharge shall be investigated. Appropriate live-load surcharge as specified *LRFD* 3.11.6.4 shall be considered where applicable. Rankine’s theory or Coulomb’s theory shall be used at the discretion of the designer.

The angle of friction, $\delta$, between the back of the modules and backfill is stated in *LRFD* for three possible situations. For friction-angle selection, see *LRFD* Table C3.11.5.9-1. This angle affects the magnitude and direction of the resulting lateral earth-pressure force. In performing an external analysis of the system, only the forces acting on or inside the pressure plane may be utilized.

410-6.04(04) Sliding Resistance

Resistance to sliding is provided by infill to the foundation-material interface friction at the base of the module. In performing sliding analysis, the following shall be considered.

1. Sliding shall be evaluated at the Strength Limit state.

2. The requirements of *LRFD* 10.6.3 and 11.11.4.2 will apply.

3. Calculation methods are similar to those for a cast-in-place concrete wall.

4. Load factors shall be as shown in *LRFD* Figure C11.5.5-2.

5. Passive resistance on the front of the wall due to wall embedment is most-often neglected.
6. The coefficient of sliding friction between the infill and foundation material at the wall base shall be the lesser of the friction angle of the infill and the friction angle of the foundation soil.

7. For granular soil, a friction angle of 30 deg maximum shall be used.

For a precast-concrete modular binwall with footings, 80% of the weight of the soil in the modules is transferred to the footing supports, with the remaining soil weight being transferred to the area of the wall between the footings.

**410-6.04(05) Limiting Eccentricity Due to Overturning [Rev. Oct. 2012]**

Resistance to limiting eccentricity due to overturning is provided by the infill within the module. In performing a sliding analysis, the following shall be considered.

1. Eccentricity shall be evaluated at the Strength Limit state.

2. The requirements of *LRFD* 10.6.3 and 11.11.4.4 will apply.

3. Calculation methods are similar to those for a cast-in-place concrete wall.

4. Load factors shall be as shown in *LRFD* Figure C11.5.6-2.

5. The location of the resultant of the reaction forces shall be within the middle two-thirds of the base width.

6. Passive resistance on front of the wall due to wall embedment is most-often neglected.

7. Unless a structural bottom is provided to retain the soil within the module, a maximum of 80% of the soil fill for a precast-concrete modular binwall, or a metal binwall within the modules, is effective in resisting overturning moments.
410-6.04(06) **Bearing Resistance**

In performing bearing-pressure analysis, the following shall be considered.

1. Bearing-resistance analysis shall be evaluated at the Strength Limit state.

2. The requirements of *LRFD* 10.6.3 and 11.11.4.3 will apply.

3. Calculation methods are similar to those for a cast-in-place concrete wall.

4. Load factors shall be as shown on *LRFD* Figure C11.5.5-1.

5. If a footing is provided at the rear and at the front of a precast-concrete modular binwall, the dead loads and earth-pressure loads are transferred to these point supports per unit length of the wall. A minimum of 80% of the soil weight inside the modules shall be considered to be transferred to the front and rear support points. If a foundation is provided under the total area of the module, all soil weight shall be considered.

410-6.04(07) **Structural Design**

In performing structural design or selecting a structural wall system, the following shall be considered.

1. Structural design shall be performed at the Strength Limit state.

2. The requirements of *LRFD* 11.11.5.1 will apply.

3. The structural demand, including compression, shear, bending stresses, etc., of the wall elements shall be compared to the manufacturer’s recommended structural capacity.

4. Prefabricated modular units shall be designed for the difference between the factored earth pressure behind the wall and the factored pressures developed inside the modules.

5. During construction, the rear face of the wall shall be designed for the earth pressure inside the module.

6. For concrete modules, strength and reinforcement requirements shall be in accordance with *LRFD* Section 5.

7. For steel modules, strength requirements shall be in accordance with *LRFD* Section 6.
410-6.04(08)  Overall Stability

The overall stability shall be evaluated using limiting-equilibrium methods of slope stability analysis. The following shall be considered.

1. Overall stability shall be investigated at the Service Limit state.
2. The requirements of LRFD 11.6.2.3 will apply.
3. Calculation methods are similar to those for a cast-in-place wall.
4. The resistance factors shall be as described in LRFD 11.6.2.3.

410-6.04(09)  Lateral and Vertical Displacement

The lateral and vertical displacement shall be evaluated at the Service Limit state, and compared with tolerable wall-movement criteria. The tolerable differential movement in terms of a horizontal differential settlement is most often 1/300 along the alignment of the wall. However, for a gabion wall, this is most often 1/50.

410-6.05 Prefabricated Concrete Bin Wall

410-6.05(01)  General

A prefabricated concrete modular gravity wall consists of interlocking reinforced precast concrete cells or modules, a cast-in-place concrete floor or dense-graded aggregate base, an optional reinforced-concrete parapet placed on top of the wall or concrete cap, and structural infill inside the modules. The parapet can be placed and held rigidly to a cast-in-place concrete slab.

The height and width shall be as determined based on the site-specific conditions, site constraints, and as required by the design. The proposed length and width of the modules shall be submitted to the Office of Geotechnical Services for review and approval during the design stage.

Foundation preparation shall include removing unsuitable material and vegetation, stabilizing weak or compressible material and replacing it with B borrow, then proof-rolling the foundation area. The wall base shall be constructed using dense-graded crushed aggregate or concrete. The concrete base shall have a 28-days compressive strength of 3,000 psi.
The fill material within and behind the modules shall be structure backfill. A geotextile filter layer shall be placed behind the front vertical joints and behind the modules to prevent migration of fines and allow passage of water. Drainage details for the wall system shall be shown on the plans.

Polyethylene foam rod and a rubber pads shall be placed in the front horizontal joint.

410-6.05(02) Design Procedure

The design shall be as described in Section 410-6.04. The LRFD Bridge Design Specifications permit 80% of the weight of the soil to be effective in resisting limiting eccentricity due to overturning moments. The basis of this practice is empirical and recognizes the fact that some of the soil in the modules is in direct contact with the foundation soil. A value of greater than 80% is permitted if the actual value can be verified from full-scale field tests or if the bins are constructed with structural floors.

Longitudinal differential settlements tolerance along the face of the wall shall result in a slope of less than 1/300. A system can be relatively rigid, and is therefore subject to structural damage due to differential settlements, especially in the longitudinal direction. Therefore, the ultimate bearing capacity for the footing design can be comparable to that for a cast-in-place wall because both are relatively sensitive to differential settlements.

LRFD Equation 11.11.5.1-1 is provided for determining the factored pressure inside the bin module, in addition to other design provisions. A value of 2 T/yd³ may be used for the soil density.

Sliding and limiting eccentricity due to overturning stability computations shall be made by assuming that the system acts as a rigid body. The lateral earth-pressure force per unit width behind a prefabricated modular binwall shall be taken as specified in LRFD 3.11.5.9, and applied at the height and direction specified therein.

Structural design shall be as described in Section 410-6.04(06). For material properties and structural design, LRFD Section 5 will apply. The concrete shall have a minimum 28-days compressive strength of 4,000 psi. Reinforcement shall be grade 60 uncoated bars or welded wire fabric.
410-6.06  Metal Binwall

410-6.06(01) General

A prefabricated metal modular binwall system functions as a gravity wall utilizing its own weight and the weight of the soil inside the modules to resist overturning and sliding. It is a proprietary wall system whose design is provided by the wall supplier. A steel modular wall system shall not be used where the groundwater or surface runoff is contaminated with acid or where deicing spray is anticipated.

Foundation preparation shall include removing unsuitable material and vegetation, stabilizing weak or compressible material and replacing it with B borrow, then proofrolling the foundation area. The wall base shall be constructed of dense-graded crushed aggregate.

Bins consist of adjoining closed-face cells filled with structure backfill to form a gravity-type wall. The cells are constructed of lightweight steel members that are bolted together at the site. The steel structure is flexible to allow for minor ground movements. The steel structure consists of S-shaped steel stringers and spacers, vertical connectors, grade plates, and stringer stiffeners.

The wall height ranges from 5 to 35 ft. The base width of a binwall ranges from approximately 40 to 60% of the wall’s height, depending on surcharges, backslopes, batter, etc. The base of the binwall shall be placed at least 3 ft below the finish-grade elevation.

The fill for the interior of the bin and behind the wall shall be structure backfill. Drainage details for the wall system shall be shown on the plans.

410-6.06(02) Design Procedure

The design shall be as described in Section 410-6.04. The LRFD Bridge Design Specifications permit 80% of the weight of the earth to be effective in resisting overturning moments. The basis of this practice is empirical and recognizes the fact that some of the earth in the modules is in direct contact with the foundation soil. A value of greater than 80% is permitted if the actual value can be verified from full-scale field tests, or if the bins are constructed with floors.

Longitudinal differential settlements along the face of the wall shall result in a slope of less than 1 to 200. A system can be relatively rigid, and is therefore subject to structural damage due to differential settlements, especially in the longitudinal direction. Therefore, the ultimate bearing capacity for footing design can be comparable to that for a cast-in-place wall because both are relatively sensitive to differential settlements.
LRFD Equation 11.11.5.1-1 is provided for determining the factored pressure inside the bin module, in addition to other design provisions. A value of 2 T/yd³ can be used for the soil density.

Sliding and overturning stability computations shall be made by assuming that the system acts as a rigid body. The lateral earth pressure force per unit width behind the wall shall be taken as specified in LRFD 3.11.5.9, and applied at a height and in a direction as specified therein.

Structural design for the metal bins wall shall be as described in Section 410-6.04(07). For material properties and structural design, LRFD Section 6 will apply.

The concrete for a precast-concrete modular binwall shall have a minimum 28-days compressive strength of 4000 psi. Reinforcement shall be grade 60 uncoated bars or welded wire reinforcement. Infill soil shall be structure backfill. Drainage details for the wall system shall be shown on the plans.

410-6.07 Gabion Wall

410-6.07(01) Background

A gravity retaining wall can be constructed from rock-filled wire baskets commonly called gabions or gabion baskets. The gabions are manufactured from a heavy wire mesh formed into rectangular baskets. Common basket sizes include a standard depth of 3 ft; heights of 1, 1.5, or 3 ft; and lengths of 3 to 12 ft. Individual baskets are placed on the prepared earth surface, reinforced with internal tie wires, and filled with riprap stone. After the baskets are filled, the lids are closed and wired shut to form a relatively rigid block. Succeeding rows of gabions are laced to the filled underlying gabions and are filled in the same manner until the wall reaches the design height. A proprietary gabion-basket manufacturer will supply details for the wires, lacing, and lid closure. However, the manufacturer does not provide internal or external wall design. External stability considerations are determined by the Office of Geotechnical Services.

Foundation preparation shall include removing unsuitable material and vegetation, stabilizing weak or compressible material and replacing it with B borrow, then proof rolling the foundation area. The wall base shall be constructed of dense-graded crushed aggregate.

A gabion wall can be used for a variety of applications. A wall on a grade can be accommodated by either putting steps in the wall or by sloping the base of the wall. A gabion wall on a grade of 5% or more has a more pleasing appearance if steps are utilized. A gabion wall can be constructed adjacent to a stream or lake so that at least a portion of the wall is below the water line. For this application, it is necessary to dewater the wall site during construction. For an in-water installation, the wall shall be protected against erosion or scour by the use of riprap or other suitable
A gabion wall can also be constructed along a curved alignment. However, a sharp curve with a radius of less than 25 ft can be difficult to construct and shall be avoided. A layer of geotextile fabric shall be placed on the back side of the wall prior to backfilling to prevent soil migration and loss. The minimum embedment for a gabion wall is 1’-6”.

The durability of a gabion wall is dependent upon maintaining the integrity of the gabion baskets. Galvanized steel wire is required for all each gabion installation. In an area of high corrosion potential due to soil, water, salt spray, or abrasion conditions, a polyvinyl chloride coating is required in addition to galvanizing. Conditions at each individual site shall be assessed to determine corrosion potential. Although gabions are manufactured from a heavy gage wire, there is a potential for damage due to vandalism. The potential for such vandalism shall be considered at each specific site.

In gabion-wall design, the mass of a wall will increase disproportionately with increases in height; therefore, doubling the height of a wall will more than double the mass of the wall. The ratio of the base width to height will vary, but this value shall not be below 0.5. In practice, this value will range from 0.5 to 0.75 depending on backslope, surcharge, and angle of internal friction of retained soil. A gabion wall has shown to be economical for a low to moderate height, but is less economical as height increases. A height of about 18 ft shall be considered as a practical limit for a gabion wall.

A gabion wall is tilted back into the slope for design stability. A declination of 6 deg is used, but another angle is acceptable. A geotechnical investigation and analysis is conducted by the Office of Geotechnical Services to determine soil-design parameters for retained and foundation soils. Consolidation potential due to wall loads is considered in determining foundation design parameters.

The rough texture of the gabion baskets provides an attractive surface for climbing vines and plants. Plantings of this type at the base of the wall can provide a more natural appearance within a few growing seasons.

**410-6.07(02) Design Procedure**

The design of a gabion wall shall be as described in Section 410-6.04. It is not specifically addressed in the LRFD Bridge Design Specifications. It shall, however, be designed in accordance with the applicable portions of LRFD Article 11.11.

Design of a gabion wall shall consider lateral earth pressure and all surcharges. Resistance to such loads is developed by proportioning the cross-sectional area of the wall to achieve a sufficient mass to ensure stability. The analysis is similar to that for another type of modular gravity wall.
Sliding and rotation shall be considered for the full height of the wall and at each gabion layer in the wall.

The factored base pressure of the wall cannot exceed the foundation-soil bearing resistance. Wall-base pressure can be determined by using the Meyerhoff method in which vertical loads are distributed over a base area reduced for eccentricity. This method is shown in Figure 410-6A, Broken-Back Slope, Simplified Example, and Figure 410-6B, Sloping Backfill, Simplified Example. More-precise base pressures can be determined from a static analysis of all forces acting about location of the resultant. Global stability can be determined from conventional soil mechanic methods or programs.

Lateral earth pressures are determined by multiplying vertical loads by the coefficient of active earth pressure, $K_a$. This value can be determined by either the Rankine method or the Coulomb method at the discretion of the designer.

In addition to the actual weight of the gabions, all earth backfill bearing directly on the gabions shall be included as part of the wall system. Lateral earth pressure shall be assumed to act on a vertical plane rising from the back of the wall base. These conditions are illustrated in Figures 410-6A and 410-6B.

Gabion-wall analysis is simplified by separating the wall into individual sections based on gabion placement. Surcharge loads shall be added in determining driving loads but shall not be included in computing resisting values.

410-6.07(03) Summary of Design Requirements

1. **Foundation-Design Parameters.** Use values provided by the Office of Geotechnical Services.

2. **Traffic Surcharge.** Traffic live load surcharge = 2 ft = 240 lb/ft$^3$

3. **Retained Soil.**
   a. Unit weight = 120 lb/ft$^3$
   b. Angle of internal friction as determined from tests made by the Office of Geotechnical Services.

4. **Soil-Pressure Theory.** Rankine’s Theory or Coulomb’s Theory shall be used at the discretion of the designer.
410-7.0 SPECIAL EARTH-RETAINING SYSTEMS

410-7.01 Steel-Sheet-Piling Nongravity Cantilever Wall

A steel-sheet-piling wall is used as a temporary wall, but it can also be used in a permanent location. See Figure 410-2C, Cut-Wall System Selection Chart, for characteristics, including advantages and disadvantages, of a sheet-piling wall.

410-7.01(01) Design Procedure

A description of the design of a sheet-piling wall along with some simplified earth-pressure distributions is shown in the LRFD Bridge Design Specifications. This type of wall is also referred to as a flexible cantilevered wall. A steel-sheet-pile wall can be designed as a cantilevered wall up to approximately 15 ft height. A steel-sheet-pile wall of greater height can require tiebacks with either prestressed soil anchors or deadman-type anchors. Anchored-wall design and details are discussed in Section 410-7.02. The preferred method of designing cantilever sheet piling is shown in the United States Steel Sheet Piling Design Manual, Conventional Method. The Office of Geotechnical Services will provide the soil-design parameters including cohesion values, angle of internal friction, angle of wall friction, soil densities, and water-table elevations.

Areas of permanent steel sheet piling above the ground line shall either be coated or painted prior to driving, or made from weathering steel. Corrosion potential shall be considered in steel-sheet-piling design.

The appearance of a permanent steel-sheet-piling wall can be enhanced by applying either precast-concrete panels or cast-in-place concrete surfacing. Welded stud-shear connectors can be used to attach cast-in-place concrete to a sheet of piling. See Figure 410-7A, Sheet-Piling Wall, Concrete-Facing Detail. Surface finishes obtained by using form liners or other means, and concrete stain or a combination of stain and paint are recommended for the concrete facing.

For information on steel sheet piling required for railroad protection, see Section 17-5.04.
410-7.01(02) Summary of Design Requirements

1. Foundation-Design Parameters. Use values provided by the Office of Geotechnical Services.

2. Traffic Surcharge. Traffic live-load surcharge = 2 ft = 240 lb/ft³

3. Retained Soil.
   a. Unit weight = 120 lb/ft³
   b. Angle of international friction as determined from tests from the Office of Geotechnical Services.


410-7.02 Soldier-Pile and Lagging Wall

410-7.02(01) General

Wall elements shall be designed to resist all horizontal and vertical loads in accordance with the LRFD Bridge Design Specifications. If anchor inclination is required, the structural analysis of the soldier piles shall consider the interaction effects of combined axial load and flexure in accordance with the Specifications.

410-7.02(02) Embedment Depth

For cantilever piles without anchors, the embedment shall be determined to satisfy horizontal force equilibrium and moment equilibrium about the bottoms of the piles.

For piles with anchors, the depth of the embedment is determined by means of moment equilibrium of lateral forces about the lowest anchor level.

The moment resistance of the soldier-pile member shall be neglected at the level of the lowest anchor.

Depth of embedment, D, shall also be sufficient to provide necessary vertical capacity or adequate kick-out resistance through development of passive pressure.
410-7.02(03) Design of Timber Lagging

The lagging thickness is determined from past construction experience as related to depth of excavation, soil condition, and soldier-pile spacing. Otherwise, soil-pressure distribution recommended by the Office of Geotechnical Services shall be used to determine the lagging thickness.

410-7.02(04) Design of Fascia Wall

The fascia wall shall be reinforced concrete. It shall be designed in accordance with the *LRFD Bridge Design Specifications*. The minimum structural thickness of fascia wall shall be 9 in. Architectural treatment of facing shall be addressed in the special provisions.

Concrete strength shall not be less than 3000 psi at 28 days. The wall shall extend a minimum of 2 ft below the ground line adjacent to the wall.

Permanent-drainage systems shall be provided to prevent hydrostatic pressures from developing behind the wall. A cut which slopes toward the proposed wall will invariably encounter natural subsurface drainage.

Vertical chimney drains, prefabricated drains, or porous engineering fabrics can be used for normal situations to collect and transport drainage due to a weep hole or pipe located at the base of the wall. Concentrated areas of subsurface drainage can be controlled by means of installing horizontal drains to intercept the flow at a distance well behind the wall.

410-7.02(05) Stage-Construction Check

The earth-pressure distribution for an anchored wall changes during wall installation. The procedure for checking the stability of the wall system for temporary construction loadings shall be the responsibility of the anchored-wall specialty contractor subject to Department approval.

410-7.02(06) Design of Bond Length

The bond length shall not be shown on the plans.

For design purposes, the required bond length shall be approximated with sufficient accuracy so that cost estimates and right of way acquisition can be confidently made.
The bond transfer values for soil grout length, or bond length, shall be verified by means of testing to determine the required bond length.

Other considerations are as follows.

1. A minimum bond length shall be specified. The recommended value is 15 ft in soil, or 10 ft in rock.

2. A bond length exceeding 40 ft in soil or 25 ft in rock does not efficiently increase the anchor capacity.

3. At a site with restricted right of way, the maximum bond length is the distance from the end of unbonded length to within 2 ft of the right-of-way line.

4. To permit high-pressure grouting without damage to existing facilities and to ensure adequate overburden pressure to mobilize the full friction between soil and grout, a 15-ft minimum overburden cover over the bond zone is recommended for anchors of average capacity, i.e., 150 kip or less.

5. Anchors founded in mixed ground condition shall be designed assuming the entire embedment is the weakest deposit.

6. The minimum unbonded length is 15 ft.

410-7.03 Drilled-Shaft Wall

Drilled shafts can be used to stabilize soil slopes. The shafts must be able to withstand the bending stress exerted on them from the unstable mass, and extended a sufficient depth below the critical failure surface in order to develop enough passive resistance to withstand the driving force. Drilled shafts can also be used as an earth-retention wall in different forms. The drilled shafts can touch each other as a tangent-piles wall, they can overlap each other as a secant-piles wall, or they can be spaced with a gap between shafts as an intermittent-pile wall. Drilled shafts increase the stability of slopes and embankments by increasing the shear resistance across the potential failure surface. If the drilled-shaft wall is an intermittent wall, the spacing of the shafts shall be designed so that soil from the unstable mass does not squeeze through the gap between shafts. A design procedure is provided in NHI 132036 Earth Retaining Structures, and FHWA IF-99-025 Drilled Shafts: Construction Procedures and Design Method.
An unstable slope can be stabilized as shown in Figure 410-7B. Some of the forces from the downward-moving mass are transferred to the upper portion of the drilled shafts, which serves to increase the resisting forces in the soil, with a resulting increase in the factor of safety.

The portion of the drilled shaft below the sliding surface shall be designed to resist the applied forces without excessive deflection or bending moment. Technology is being developed to allow a rational solution for the design of the drilled shafts.

Drilled shafts have some advantages in stabilizing a slope. The drilling of an excavation and the placing of concrete will cause fewer disturbances than driving a pile. Also, a crane-mounted drilling machine can be rigged so that it can sit above the slope and reach 50 ft or more to make the boring.

410-7.04 Anchored Wall

An anchored wall uses vertical members as main load carrying members, such as soldier piles, i.e., rolled steel sections, cylinder piles, sheet piles, or slurry walls to resist forces. The main members are connected to high-strength steel bars or strand anchors, which are fixed into soil or rock with high-strength grout and stressed to counteract the horizontal earth-pressure loads. Figure 410-7C provides an anchored-wall typical section. This type of wall is most practical in a cut section and is best suited for a situation where excavation for a retaining wall with a footing is impractical because of traffic, utilities, existing structures, or right of way restrictions. An advantage in using an anchored wall is that it causes minimal disturbance to the soil behind the wall and structures resting on this soil. Non-stressed anchors, called deadman anchors, rely on passive pressure of the soil in front of the deadman panel to resist horizontal forces. An anchored wall shall be designed by an anchored-wall specialty contractor subject to Department approval.

410-7.04(01) Principles of Anchored-Wall Design

Anchored-wall design includes the following:

1. evaluation of the feasibility of anchors;
2. selection of an anchor system;
3. estimation of anchor capacity;
4. determination of unbonded length, bonded length; and
5. selection of corrosion protection.
The economical use of anchors shall be determined for a particular site based on installation ability and development of anchor capacity. The presence of utilities or other underground facilities can affect whether anchors can be installed.

Anchors can consist of bars, wires, or strands. The choice of appropriate type is usually left to the contractor but may be specified by the designer if site conditions exist which preclude the use of certain anchor types. Strands and wires have advantages with respect to tensile strength, limited work areas, ease of transportation, and storage. Bars are more easily protected against corrosion, and are easier to stress and transfer load.

A reliable estimate of the safe anchor capacity shall be provided by a geotechnical engineer to determine the feasibility of anchoring. The capacity of each anchor shall be verified through testing. Requirements for test methods and frequency are provided in the AASHTO LRFD Bridge Construction Specifications. Typical system design values are as follows.

1. Design loads shall range from 60 to 240 kip.

2. The anchor-wall system shall be analyzed to ensure long-term stability. The minimum unbonded length of 15 ft for soil or rock anchors shall be shown on the plans. A longer free length may be required in plastic soil. The designer shall contact the Office of Geotechnical Services.

3. The angle of inclination shall be between 10 deg and 45 deg. A 15-deg angle is preferred to simplify grouting and to minimize vertical forces imposed on the wall by the anchors. A steeper angle of up to 45 deg is recommended only to reach deep bearing strata or avoid existing substructures.

The ultimate anchor transfer load per unit length can be preliminarily estimated using the guidelines shown in the LRFD Bridge Design Specifications for soil or rock. The final anchored-wall design will be the responsibility of the anchored-wall specialty contractor selected for wall construction.

The maximum allowable anchor design load in soil can be determined by multiplying the bonded length by the ultimate transfer load and dividing by a Factor of Safety of 2.5.

The maximum allowable anchor design load in rock can be determined by multiplying the bonded length by the ultimate transfer load and by dividing by a Factor of Safety of 3.0.
410-7.04(02) Earth-Pressure Distribution

For an anchored wall with two or more anchors constructed from the top down, the earth-pressure force resultant per unit width of wall, pounds per inch, can be determined from the LRFD Bridge Design Specifications.

The design shall first consider the active earth-pressure coefficient, $K_a$, unless structures exist within a lateral distance equal to twice the wall height. For this situation, the average earth-pressure coefficient, $K$, shall be computed as follows:

$$K = K_0 - \left( \frac{x}{2H} \right) (K_0 - K_a)$$

Where:
- $x =$ Distance from structure to wall, ft
- $H =$ Height of wall, ft
- $K_0 =$ Coefficient of at-rest earth pressure

$K_a$ permits lower wall-design pressure if small wall displacements can be tolerated, i.e., ground subsidence occurs.

$K_0$ increases wall-design pressure but limits wall displacement, i.e., ground subsidence is limited.

410-7.04(03) Corrosion Protection

See the AASHTO LRFD Bridge Design Specifications for corrosion-protection guidelines. Corrosion-protection requirements for the anchor head, the unbonded length, and the anchor length shall be included in the specifications for the anchored wall.

410-7.04(04) Determination of Anchor Spacing

Suggested temporary test loads are between 75 and 80 percent of Guaranteed Ultimate Tensile Strength (GUTS). Suggested limits for design loads are between 0.5 and 0.6 of GUTS, or typically 53%.

Typical horizontal-pile spacings of 6 to 10 ft and vertical-anchor spacings of 8 to 12 ft are commonly used. The minimum spacing of 4 ft in both directions is not recommended for considering the effectiveness and disturbance of anchors due to installation.
410-7.04(05) Drainage

An anchored wall shall have weepholes of 4 in. diameter, located a minimum of 1 ft above the final ground line and spaced about 10 ft apart.

Drainage panels shall be installed at each weephole and shall extend from the base of the wall to a level that is 1 ft below the top of the wall as described in the LRFD Bridge Design Specifications. A drainage panel consists of a strip of prefabricated geocomposite drain material of 2 ft width. Drainage features shall be shown on the plans.

410-7.05 In-Situ-Reinforced Wall

410-7.05(01) Soil-Nailed Wall

A soil-nailed wall is an earth-retaining system consisting of reinforced in-situ material which can be either original ground or an existing embankment. Construction is accomplished by means of excavating from the top of wall elevation down in stages of typical height of 4 to 6 ft. After each stage of excavation, soil-reinforcing elements, or soil nails, generally consisting of reinforcing bars, are placed and grouted into drilled holes which have been drilled at a slight downward inclination from level into the in-situ material. The face of each stage of excavation is protected by a layer of reinforced shotcrete. After the full height of the wall has been excavated and reinforced, a finish layer of concrete facing is placed for the full head of the wall.

Soil nailing is most applicable for retaining excavations and for increasing the stability of slopes.

The designer is responsible for the structural design and preparation of the contract documents. The Office of Geotechnical Services is responsible for the geotechnical design. The geotechnical aspect of the design establishes the soil-nail size, length, spacing, and minimum drilled-hole diameter.

This type of wall shall be considered experimental where the conditions exist as follows:

1. the wall height is greater than 30 ft;
2. the wall is to be built in clay or soils with sufficient clay content such that the soil mass will behave as a clay, based on engineering considerations; and
3. the wall has an unusual surcharge load.

A permanent facing system is required. The permanent face of the wall shall be vertical, although the shotcrete facing of the soil nailed wall may be battered.
Soil nailing has technical and economic advantages over an MSE retaining wall as follows.

1. A soil-nailed wall is constructed incrementally from the top down, which will eliminate the cost of temporary sheeting or shoring systems required for MSE wall excavation.

2. The volume of excavation is significantly reduced as compared to that for an MSE wall.

3. Borrow is not required for a soil-nailed wall.

4. Soil-nailed wall construction and excavation shall proceed significantly faster than MSE-wall construction due to less excavation volume and elimination of shoring.

5. Only light construction equipment and simple grouting equipment are required to install the nails. Grouting of the boreholes is generally accomplished by gravity. This feature is significant for a project site in a traffic congested area.

However, the specific details of the nail length and location shall be developed by the contractor and submitted for review and approval by the Department. The soil parameters for soil-nailed-wall design are listed in Figure 410-7D.

A soil-nailed wall in clay soil requires nail lengths between 0.7 and 1.0 times the wall height, \( H \), with 0.85\( H \) as a typical ratio. Permanent wall easements may be necessary to accommodate the soil nails.

Ultimate pullout resistance, or friction limit, of each nail is a function of the size and shape of the drill hole, strength characteristics and density of the soil in which it is placed, drilling method, length tested, method to clean the drill hole, and grouting method or pressure used.

The construction of a soil-nailed mass results in a composite coherent mass similar to that of an MSE wall. The locus of maximum tensile forces separates the nailed soil mass into the zones as follows:

1. an active zone, or potential sliding soil wedge, where lateral shear stresses are mobilized and result in an increase of the tension force in the nail; and

2. a resistance, or stable, zone where the generated nail forces are transferred into the ground.
The design of a soil-nailed retaining structure is based on evaluation of the following:

1. global stability of the structure and the surrounding ground with respect to a rotational or translational failure along potential sliding surfaces; and
2. local stability at each level of nails.

A computer program shall be used for soil-nailed-wall analysis. Global-stability analysis consists of evaluating the global stability of the soil-nailed retaining structure and the surrounding ground with respect to a rotational or translational failure along potential sliding surfaces. It requires determination of the critical sliding surface which can be dictated by the satisfaction of the subsurface soil and intensity of surcharge loads, as well as the specific design of the reinforcing elements’ spacing, length, and location. Because global stability is a function of the nail length and spacing, it is evaluated as part of the design of the wall, and cannot be evaluated independently of reinforcement spacing, as is typical for an MSE wall.

Requirements for the installation of a prefabricated vertical wall drain shall be included in the contract documents.

A soil-nailed wall shall be designed by a specialty contractor based on its knowledge and experience in the practice of soil-nailed-wall construction.

**410-7.05(02) Micropile Wall**

1. **General.** A micropile wall, including a root-pile wall or an insert wall, consists of an array of drilled and grouted micropiles that penetrate below a potential surface of sliding. For this wall system, the micropiles are connected at the ground surface to a reinforced-concrete cap beam. The design of a root-pile wall uses small-diameter piles spaced closely together in a complex three-dimensional network. The purpose of the micropile system is to knit the soil into a coherent mass that behaves as a gravity-retaining structure. The vertical and battered piles of an insert wall are larger in diameter and are spaced farther apart in comparison to a root-pile wall. This wall system provides sliding resistance through tensile and flexural resistance developed in the piles.

Micropiles are drilled piles of less than 12 in. dia., constructed with steel reinforcement, and bonded to the ground with grout using gravity or pressure-grouting techniques. Micropiles can be used for structural support, slope stabilization, or a retaining system. Information on the design and construction of micropiles for structural support and slope stabilization is provided in Sabatini and Tanyu, 2006.
A micropile wall can be used for temporary shoring or as a permanent earth-retaining system. Micropiles are relatively expensive compared to other forms of deep foundation elements such as driven piles or drilled shafts. Inasmuch as drilled shafts and driven-pile elements are used as vertical wall elements, e.g., a secant-pile wall consists of driven steel soldier piles, the use of micropiles for a wall system will likely be a viable and cost-effective system only where driven piles or drilled shafts cannot be installed.

The principal components of a micropile wall consist of vertical micropile elements installed from the ground surface at or near the final excavated wall-face line, and sub-horizontal elements installed from the ground surface which resembles a ground anchor. Figure 410-7E shows a cross section of a micropile retaining wall. The A-frame system formed by the vertical and sub-horizontal micropiles is structurally connected with a reinforced-concrete grade beam.

The advantages of a micropile wall are as follows.

a. It can be constructed in a remote area or where there is restricted access.

b. It can be installed in difficult and variable ground conditions, e.g., karstic area, uncontrolled fill, cobbles, boulders, etc.

c. Unlike another drilled-shaft system, the construction of micropiles is less affected by soft clays, running sands, or a high groundwater table.

d. Vibration and noise is minimal.

e. No significant spoil is generated during construction of the micropiles.

f. Due to high tension and compression capacities of micropiles, a relatively tight frame configuration can be used allowing for construction under limited right-of-way constraints.

The limitations of a micropile wall are as follows.

a. An underground easement is required for installation.

b. Vertical micropiles have limited lateral capacity.

c. Micropiles may not be suitable where liquefaction can occur due to concerns of buckling resulting from loss of lateral support, though this effect can be evaluated.
d. Design methods are not well developed primarily due to limited performance data for wall applications.

2. **Construction Materials and Methods.** A micropile retaining wall is constructed from the top-down and follows this sequence.

a. At the ground surface, an area is excavated that is wide and deep enough to accommodate the cap beam.

b. The formwork is installed for the cap beam and the cap-beam steel reinforcement is placed.

c. The corrugated plastic sleeves are placed for installation of the micropiles through the cap beam.

d. The concrete cap is poured.

e. Micropiles are installed through the plastic sleeves.

f. Excavation, application of temporary shotcrete facing, and installation of geocomposite drains and other drainage systems are made until final excavation grade is reached.

g. The cast-in-place wall facing is installed, if required.

Excavation in front of the wall is performed in lifts of typically not more than 6 ft. During excavation, shotcrete is applied to the excavation face to temporarily prevent raveling of the soil face. Connection to the micropiles is performed via head studs that are welded to the front-line micropiles. Following completion of the excavation, a leveling pad is poured to allow erection of one-sided forms. Once the leveling pad is completed, the wall face is constructed from CIP concrete. Headed studs welded to the micropiles are embedded in the CIP grade beam and wall face to provide connection of the micropile structure to the CIP wall face.

The typical construction sequence for simple gravity-grouted and pressure-grouted micropiles includes drilling the pile shaft to the required tip elevation, placing the steel reinforcement, placing the initial grout with a tremie, and placing additional grout under pressure as applicable. The drilling and grouting equipment and techniques used for the micropile construction are similar to those used for the installation of soil nails or ground anchors.
3. **Typical Micropile Construction Sequence Using Casing.** The amount of steel reinforcement placed in a micropile is determined based on the loading it supports and stiffness required to limit elastic displacements. Reinforcement can consist of a single reinforcing bar, a group of reinforcing bars, a steel pipe casing, or rolled structural steel. Reinforcement can be placed either prior to grouting, or placed into the grout-filled borehole before the temporary casing, if used, is withdrawn.

4. **Micropile-Wall Design.** There is no generally-accepted procedure available for the design of this system. It can be analyzed using soil-structure interaction analyses in which the axial stiffness and bending stiffness of the vertical and battered micropiles are explicitly modeled. All stages of excavation in front of the wall can be modeled. With this approach, other potential failure mechanisms shall be considered separately, including the potential for soil to squeeze in between the small-diameter micropiles and the potential for structural failure of the vertical micropiles due to buckling. Buckling is checked since the relatively small-diameter vertical micropiles will experience compressive loads as they are close to the exposed ground surface.

The design approach is described as follows.

a. The micropiles section is assumed, and the ultimate bending capacity of the micropile sections shall be calculated. The flexural rigidity, $EI$, of the micropiles shall be calculated. These values can later be used in numerical analyses. Micropiles that consist of a centralized reinforcing bar in a drilled and grouted hole can be analyzed using LPILE’s ultimate bending-analysis module. The tensile and compressive capacity of each section shall be calculated.

b. The system can be analyzed as a rigid gravity wall. The wall geometry is defined as ground-enclosed by the micropile-system envelope. Earth pressures are calculated using classical earth pressure theories, assuming that the wall deforms sufficiently to allow the soil to reach the active state. Sliding of the system shall be analyzed to include the shear capacity of the front micropile. The embedment of the micropile shall be checked to evaluate whether sufficient passive resistance can be developed in front of the micropile to mobilize the required micropile shear strength. Overturning shall be checked by means of summing overturning moments about toe of micropile wall. The bond length of the rear row of battered micropiles required to resist the overturning moment with respect to tensile rupture and pullout failure shall be computed.

c. The micropile-wall system shall be analyzed using the free-earth-support method, as used for anchored-bulkhead design. The front row of closely-spaced micropiles is considered to be analogous to a sheet-pile wall. The battered rows of micropiles
are analogous to deadman anchors. The analysis shall be modified such that it is assumed that one-half of the calculated active earth load is applied to the vertical micropile row as a triangular pressure distribution. One-half of the calculated active earth load is applied to the rear-battered micropile row. A lateral pile analysis shall be completed for both the vertical and battered micropiles using an appropriate computer program, such as LPILE. Global stability of the micropile system shall be evaluated.

d. Finite-element analyses can be performed to predict deformations in the structure. Worst-case geometries and construction stages of the temporary excavation support for the wall shall be analyzed.

e. The potential for soil flow in between the relatively closely-spaced micropiles shall be evaluated.

5. Load Testing of Micropiles. Load testing is performed in the field to verify the following:

a. the design loads can be carried by the micropiles without excessive movements;
b. the contractor’s equipment and installation procedures are adequate; and
c. the long-term behavior of the micropiles is as anticipated.

Micropiles are tested individually using the same conventional static-load testing procedures as are used for driven piles and drilled shafts. These tests include incremental loading which can be applied in compression, tension, or laterally, and which may be cycled, i.e., load/unload, until the micropile reaches the selected maximum test load; structural displacement limit; or ground creep, i.e., movement under constant load, threshold.

Well-defined testing programs, consistent with a well-developed design approach, are available. Load testing shall be performed to verify the displacement response and capacity of micropiles used for a wall system. Such a testing program shall be developed on a project-specific basis.

Performance data, including micropile load transfer, axial loads, bending moments, and displacements shall be collected for a micropile wall system to enable design methods to be updated.
410-8.0 REINFORCED SOIL SLOPES

Reinforced soil slopes (RSS) are a cost-effective alternative for new construction where right of way or other considerations can make a steeper slope desirable. As shown in Figure 410-8A, Slope Reinforcement Using Geosynthetics to Provide Slope Stability, multiple layers of reinforcement are placed in the slope during construction or reconstruction to reinforce the soil and provide increased slope stability. Reinforced soil slopes are a form of mechanically-stabilized earth that incorporates planar reinforcing elements in constructed earth sloped structures with face inclinations of usually 45 deg or less. Geosynthetics are used for reinforcement.

410-8.01 Purpose of Reinforcement

The principal purpose for using reinforcement is to construct an RSS embankment at an angle steeper than can otherwise be safely constructed with the same soil as shown in Figure 410-8A. The stability allows for construction of steepened slopes on a firm foundation for a new highway and as a replacement for a flatter unreinforced slope or a firm foundation for a retaining wall. A roadway can also be widened over existing flatter slopes without encroaching on existing right-of-way. In repairing a slope failure, the new slope will be safer, and reusing the slide debris rather than importing higher quality backfill can result in substantial cost savings. The minimum Factor of Safety for internal stability is 1.3.

Another purpose for using reinforcement is at the edges of a compacted fill slope to provide lateral resistance during compaction as shown in Figure 410-8B, Slope Reinforcement Providing Lateral Resistance During Compaction. The increased lateral resistance allows for an increase in compacted soil density over that normally achieved and provides increased lateral confinement for the soil at the face. A modest amount of reinforcement in compacted slopes can prevent sloughing and reduce slope erosion. Edge reinforcement also allows compaction equipment to more safely operate near the edge of the slope.

Right-of-way savings can be a substantial benefit, especially for a road-widening project in an urban area where acquiring new right of way is always expensive and maybe impossible. RSS also provide an economical alternative to a retaining wall. RSS can be constructed at about one-half the cost of an MSE wall structure.

Further compaction improvements have been made in cohesive soils through the use of geosynthetics with in-plane drainage capabilities, e.g., nonwoven geotextiles, which allow for rapid pore pressure dissipation in the compacted soil.
Compaction aids placed as intermediate layers between reinforcement in steepened slopes can also be used to provide improved face stability and to reduce layers of more expensive, primary reinforcement as shown in Figure 410-8A.

The use of vegetated-faced RSS that can be landscaped to blend with a natural environment can also provide an aesthetic advantage over a retaining-wall-type structure. However, there are maintenance issues to be addressed such as mowing grass-faced, steep slopes.

For an RSS structure, the choice of slope facing can be controlled with climatic and regional factors. For a structure of less than 30 ft height with slopes of 1:1 or flatter, a vegetative green slope can be constructed using an erosion-control mat or mesh and local grasses. Where vegetation cannot be successfully established or where significant runoff may occur, armored slopes using natural or manufactured materials shall be used to reduce future maintenance.

In terms of performance, due to inherent conservation in the design, RSS are actually safer than flatter slopes designed at the same Factor of Safety. As a result, there is a lower risk of long-term stability problems developing in the slopes. Such problems can occur in compacted fill slopes that have been constructed to low Factors of Safety or with marginal materials, e.g., deleterious soils such as shale, fine grained low cohesive silts, plastic soils, etc. The reinforcement can also facilitate strength gains in the soil over time from soil aging through improved drainage, further improving long-term performance.

410-8.02 Economics

RSS are not normally constructed with rigid facing elements. Slopes constructed with a flexible face can thus readily tolerate minor distortions that can result from settlement, freezing and thawing, or wet-drying of the backfill. As a result, soil which satisfies the requirements for embankment construction can be used in a RSS system. However, a higher-quality material reduces concerns for the durability of the reinforcement, and is easier to handle, place, and compact, which speeds construction.

The performance of RSS is generally not affected by differential longitudinal settlements.

RSS construction with an organic vegetative cover shall be chosen to be consistent with native perennial cover that establishes itself quickly and thrives with available site rainfall, and is maintenance free.

RSS can be cost effective in a rural environment, where right-of-way restrictions exist, or on a widening project where long sliver fills are necessary. In an urban environment, they shall be
considered where existing right of way is sufficient for construction, as they are always more economical than a vertically-faced MSE-wall structure.

410-8.03 References

Reference publications regarding RSS include the following.


<table>
<thead>
<tr>
<th>Line and Form</th>
<th>Curvilinear, cords of a wall that reflect adjacent landforms</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Color</strong></td>
<td>Respond to local setting using the following:</td>
</tr>
<tr>
<td></td>
<td>Colored concrete blocks</td>
</tr>
<tr>
<td></td>
<td>Concrete stain or paint</td>
</tr>
<tr>
<td></td>
<td>Natural or stained wood</td>
</tr>
<tr>
<td></td>
<td>Painted or weathering steel</td>
</tr>
<tr>
<td></td>
<td>Dip treatment to turn steel dull or gray</td>
</tr>
<tr>
<td><strong>Texture</strong></td>
<td>Respond to local setting using the following:</td>
</tr>
<tr>
<td></td>
<td>Stone</td>
</tr>
<tr>
<td></td>
<td>Barnboard concrete</td>
</tr>
<tr>
<td></td>
<td>Bush-hammered concrete</td>
</tr>
<tr>
<td></td>
<td>Ribbed rustication</td>
</tr>
<tr>
<td></td>
<td>Form liner</td>
</tr>
<tr>
<td></td>
<td>Cribbing</td>
</tr>
<tr>
<td></td>
<td>Binwall with textured concrete veneer</td>
</tr>
<tr>
<td><strong>Level of Detailing</strong></td>
<td>Can be highly detailed</td>
</tr>
<tr>
<td><strong>Accents</strong></td>
<td>Pilasters</td>
</tr>
<tr>
<td></td>
<td>Arches and caps</td>
</tr>
<tr>
<td></td>
<td>Contrasting lines, colors, or textures</td>
</tr>
<tr>
<td><strong>Fencing</strong></td>
<td>Can include custom design</td>
</tr>
</tbody>
</table>

**GENERAL AESTHETIC GUIDELINES FOR RETAINING WALL IN URBAN AREA**

*Figure 410-1A*
<table>
<thead>
<tr>
<th></th>
<th>Scenic Route</th>
<th>Commercial Route</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Line and Form</strong></td>
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<td>Angular</td>
</tr>
<tr>
<td><strong>Color</strong></td>
<td>Respond to local setting using the following:</td>
<td>Economically respond to local setting using the following:</td>
</tr>
<tr>
<td></td>
<td>Colored concrete blocks</td>
<td>Colored concrete blocks</td>
</tr>
<tr>
<td></td>
<td>Concrete stain or paint</td>
<td>Natural, stained, or</td>
</tr>
<tr>
<td></td>
<td>Natural or stained wood</td>
<td>painted concrete</td>
</tr>
<tr>
<td></td>
<td>Painted or weathering steel</td>
<td>Natural, stained, or</td>
</tr>
<tr>
<td></td>
<td>Dip treatment to turn steel</td>
<td>painted wood</td>
</tr>
<tr>
<td></td>
<td>dull or gray</td>
<td>Weathering steel</td>
</tr>
<tr>
<td><strong>Texture</strong></td>
<td>Respond to local setting using the following:</td>
<td>Economically match local setting using the following:</td>
</tr>
<tr>
<td></td>
<td>Stone</td>
<td>Rubbed-finish concrete</td>
</tr>
<tr>
<td></td>
<td>Barnboard concrete</td>
<td>Blasted concrete</td>
</tr>
<tr>
<td></td>
<td>Bush-hammered concrete</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ribbing</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Form liner</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cribbing</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rustication</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Binwall with textured concrete veneer</td>
<td></td>
</tr>
<tr>
<td><strong>Level of Detailing</strong></td>
<td>Can be highly detailed</td>
<td>Simple</td>
</tr>
<tr>
<td><strong>Accents</strong></td>
<td>Pilasters</td>
<td>Minimal</td>
</tr>
<tr>
<td></td>
<td>Arches and caps</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Contrasting lines, colors, or textures</td>
<td></td>
</tr>
<tr>
<td><strong>Fencing</strong></td>
<td>Can include custom design</td>
<td>Simple design</td>
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**GENERAL AESTHETIC GUIDELINES**
**FOR RETAINING WALL IN RURAL AREA**

*Figure 410-1B*
<table>
<thead>
<tr>
<th>FILL-SECTION WALL</th>
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<tr>
<td><strong>EXTERNALLY STABILIZED</strong></td>
<td><strong>INTERNALLY STABILIZED</strong></td>
</tr>
<tr>
<td>Rigid Gravity or Semi-Gravity Wall: Cast-in-Place Concrete Gravity Wall</td>
<td>Retaining Wall with Reinforced Backfill: Segmental, Precast Facing Mechanically-Stabilized-Earth Wall</td>
</tr>
<tr>
<td>Cast-in-Place Concrete Cantilever / Counterfort Wall</td>
<td>Prefabricated Modular-Block Facing With Soil Reinforcement Geotextile / Geogrid / Welded-Wire-Facing MSE Wall</td>
</tr>
<tr>
<td>Prefabricated Modular-Gravity Wall: Crib Wall</td>
<td>Reinforced Soil Slopes</td>
</tr>
<tr>
<td>Bin Wall</td>
<td></td>
</tr>
<tr>
<td>Gabion Wall</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CUT-SECTION WALL</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EXTERNALLY STABILIZED</strong></td>
<td><strong>INTERNALLY STABILIZED</strong></td>
</tr>
<tr>
<td>Nongravity Cantilevered Wall: Sheet-Pile Wall</td>
<td>In-Situ Reinforced Wall: Soil-Nailed Wall</td>
</tr>
<tr>
<td>Soldier-Pile and Lagging Wall Drilled-Shaft Wall</td>
<td></td>
</tr>
<tr>
<td>Anchored Wall: Ground Anchor, or Tieback</td>
<td></td>
</tr>
<tr>
<td>Deadman Anchor</td>
<td></td>
</tr>
</tbody>
</table>

CLASSIFICATION OF EARTH-RETAINING SYSTEMS

Figure 410-1C
<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Perm.</th>
<th>Temp.</th>
<th>Cost Effective Height Range (ft)</th>
<th>Wall Face Cost ($/ft^2) (1)</th>
<th>Required R / W (2)</th>
<th>Differential Settlement Tolerance (3)</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Gravity</td>
<td>X</td>
<td></td>
<td>3 - 10</td>
<td>25.00 – 35.00</td>
<td>0.5 – 0.7H</td>
<td>1 / 500</td>
<td>● Durable</td>
<td>● Deep foundation support can be necessary</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Requires small quantity of select backfill than MSE wall</td>
<td>● Relatively long construction time</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Concrete satisfies aesthetic rqmts.</td>
<td></td>
</tr>
<tr>
<td>Concrete Cantilevered</td>
<td>X</td>
<td></td>
<td>5 – 30</td>
<td>25.00 – 60.00</td>
<td>0.4 – 0.7H</td>
<td>1 / 500</td>
<td>● Durable</td>
<td>● Deep foundation support can be necessary</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Requires small quantity of select backfill than MSE wall</td>
<td>● Relatively long construction time</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Concrete satisfies aesthetic rqmts.</td>
<td></td>
</tr>
<tr>
<td>Concrete Counterfort</td>
<td>X</td>
<td></td>
<td>30 – 60</td>
<td>25.00 – 60.00</td>
<td>0.4 – 0.7H</td>
<td>1 / 500</td>
<td>● Durable</td>
<td>● Deep foundation support can be necessary</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Requires small quantity of select backfill than MSE wall</td>
<td>● Relatively long construction time</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Concrete satisfies aesthetic rqmts.</td>
<td></td>
</tr>
<tr>
<td>Concrete Crib</td>
<td>X</td>
<td></td>
<td>5 – 35</td>
<td>25.00 – 35.00</td>
<td>0.5 – 0.7H</td>
<td>1 / 300</td>
<td>● Does not require skilled labor or specialized equipment</td>
<td>● Difficult to make height adjustments in field</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Rapid construction</td>
<td></td>
</tr>
<tr>
<td>Metal Bin</td>
<td>X</td>
<td></td>
<td>5 – 35</td>
<td>25.00 – 35.00</td>
<td>0.5 – 0.7H</td>
<td>1 / 300</td>
<td>● Does not require skilled labor or specialized equipment</td>
<td>● Difficult to make height adjustments in field</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Rapid construction</td>
<td>● Subject to corrosion in aggressive environment</td>
</tr>
<tr>
<td>Gabion</td>
<td>X</td>
<td></td>
<td>5 – 25</td>
<td>25.00 – 50.00</td>
<td>0.5 – 0.7H</td>
<td>1 / 50</td>
<td>● Does not require skilled labor or specialized equipment</td>
<td>● Needs adequate stone source</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Construction requires significant labor</td>
</tr>
<tr>
<td>MSE (Precast Facing)</td>
<td>X</td>
<td></td>
<td>10 – 65</td>
<td>20.00 – 35.00</td>
<td>1.7 – 2.0H</td>
<td>1 / 100</td>
<td>● Does not require skilled labor or specialized equipment</td>
<td>● Requires select backfill</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Flexibility in facing choice</td>
<td>● Subject to corrosion in aggressive environment</td>
</tr>
<tr>
<td>MSE (Geotextile / Geogrid / Welded Wire Facing)</td>
<td>X X</td>
<td>5 – 50</td>
<td>15.00 – 35.00</td>
<td>0.7 – 1.0H</td>
<td>1 / 60</td>
<td>● Does not require skilled labor or specialized equipment</td>
<td>● Requires select backfill</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Flexibility in facing choice</td>
<td>● Geosynthetic reinf. subject to degradation in some environments</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Facing may not be aesthetically pleasing</td>
<td></td>
</tr>
<tr>
<td>Modular Block with Soil Reinforcement</td>
<td>X</td>
<td>5 – 25</td>
<td>15.00 – 25.00</td>
<td>0.7 – 1.0H</td>
<td>1 / 200</td>
<td>● Does not require skilled labor or specialized equipment</td>
<td>● Requires select backfill</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Flexibility in facing choice</td>
<td>● Metal reinf. subject to corrosion in aggressive environment</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Blocks are easily handled</td>
<td>● Difficult to achieve positive reinf. connection to blocks</td>
</tr>
<tr>
<td>Reinforced Soil Slopes</td>
<td>X X</td>
<td>10 – 100</td>
<td>10.00 – 25.00</td>
<td>0.5 – 1.0H</td>
<td>1 / 60</td>
<td>● Does not require skilled labor or specialized equipment</td>
<td>● Facing may not be aesthetically pleasing</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Flexibility in facing choice</td>
<td>● Geosynthetic reinf. subject to degradation in some environments</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Vegetation provides ultraviolet light protection to geosynthetic reinforc</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>● Vegetated soil face requires significant maintenance</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
(1) Total installed cost in 1998 U.S. dollars.
(2) R/W requirements expressed as distance, as fraction of wall height $H$, behind the wall face where fill placement is generally required for flat backfill conditions.
(3) Ratio of the difference in vertical settlement between two points along the wall to the horizontal distance between the points.
<table>
<thead>
<tr>
<th>Wall Type</th>
<th>Perm.</th>
<th>Temp.</th>
<th>Cost Effective Height Range (ft)</th>
<th>Wall Face Cost ($/ft²) (1)</th>
<th>Required R / W (2)</th>
<th>Lateral Movements</th>
<th>Water Tightness</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sheet Piles</td>
<td>X</td>
<td>X</td>
<td>0 – 15</td>
<td>15 – 130</td>
<td>None</td>
<td>Large</td>
<td>Fair</td>
<td>● Rapid Construction</td>
<td>● Difficult to construct in hard ground or through obstructions</td>
</tr>
<tr>
<td>Soldier Piles / Lagging</td>
<td>X</td>
<td>X</td>
<td>0 – 15</td>
<td>10 – 35</td>
<td>None</td>
<td>Medium</td>
<td>Poor</td>
<td>● Rapid Construction</td>
<td>● Difficult to maintain vertical tolerances in hard ground ● Potential for ground loss at excavated face</td>
</tr>
<tr>
<td>Anchored</td>
<td>X</td>
<td>X</td>
<td>15 – 65 (3)</td>
<td>15 – 75</td>
<td>0.6H + anchor bond lgth.</td>
<td>Small to medium</td>
<td>N / A</td>
<td>● Can resist large horizontal pressures</td>
<td>● Requires skilled labor and specialized equipment ● Anchors can require permanent easements</td>
</tr>
<tr>
<td>Soil Nailed</td>
<td>X</td>
<td>X</td>
<td>10 – 65</td>
<td>15 – 55</td>
<td>0.6 – 1.0H</td>
<td>Small to medium</td>
<td>N / A</td>
<td>● Rapid construction</td>
<td>● Nails can require permanent easements ● Difficult to design and construct below water table</td>
</tr>
</tbody>
</table>

Notes:  
(1) Total installed cost in 1998 U.S. dollars.  
(2) R/W requirements expressed as distance, as fraction of wall height \( H \), behind the wall face where wall anchorage components, i.e., ground anchors and soil nails, are installed.  
(3) For soldier-pile and lagging wall only.

CUT-SECTION-WALL-SYSTEM SELECTION CHART  
Figure 410-2B
Note: The limits for establishing pay quantities for an alternate wall design should be identical. The outermost limit of construction for an individual wall should be used as the limit for computing pay quantities for all other alternate wall designs.

PAY QUANTITY LIMITS FOR WALL-SYSTEM GROUPS

Figure 410-2C
<table>
<thead>
<tr>
<th>CLASSIFICATION</th>
<th>WALL TYPE</th>
<th>GROUP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Externally-stabilized fill</td>
<td>Bin, metal or concrete</td>
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<td>Cantilever, cast-in-place-concrete</td>
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<td>Reinforced-soil slope</td>
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<td>Sheet-pile</td>
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<td>Slurry, or diaphragm</td>
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<td>Soil-mixed</td>
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<td>Soil-nailed</td>
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</table>

WALL TYPES AND CLASSIFICATION OF EARTH-RETAINING SYSTEMS

Figure 410-2D
\[ \mu_1 = \text{Coefficient of Friction for Soil on Soil} = 0.70 \]

\[ \mu_2 = \text{Coefficient of Friction for Concrete on Soil} = 0.45 \]

Total Frictional Force \( F_f = 0.70 \left( \frac{1}{2} \left( 33 + 18 \right) (5.09) \right) + 0.45 \left( \frac{1}{2} \right) (18)(6.07) \]

\[ = 12.9 + 3.6 \]

\[ = 16.5 \text{ k per running foot} \]

Passive Pressure in front of Wall = \( P_p \)

\[ P_p = \frac{1}{2} K_p Wh^2 ; \ W = 0.12 \text{ k/ft}^3 \]

\[ P_p = \frac{1}{2}(0.75)(0.12)(5.5)^2 = 1.36 \text{ k} \]

**FACTOR OF SAFETY AGAINST SLIDING FOR SPREAD FOOTING - EXAMPLE**

**Figure 410-4A**
MSE RETAINING WALL SECTION SHOWING EXTERNAL-STABILITY VALUES AND BACKFILL TYPES AND LIMITS

Figure 410-5(0)A
TYPICAL MSE WALL ELEVATION VIEW WITH WALL ENVELOPE

Figure 410-5(0)B
MSE SECTION VIEW

Proposed Groundline at Face of Wall
Ground Reinforcement
Precast Facing Panel
Reinforced Backfill Zone
Retained Backfill Zone
In-situ or Embankment Soil
Concrete Leveling Pad
Drainage System See Detail

MSE WALL DRAINAGE DETAIL

Overlap Geotextile by 12" Min.
6" Ø Type 4 Pipe
Aggregate for Underdrains
Geotextiles for Underdrains

MSE WALL DRAINAGE DETAIL

TYPICAL MSE WALL CROSS SECTION

Figure 410-5(0)C
① Soil below leveling pad which is subject to frost heave should be removed to an elevation 3 ft below finished grade and replaced with granular backfill.

MODULAR-BLOCK-WALL TYPICAL SECTION

Figure 410-5A
TYPES OF MODULAR BLOCKS

Figure 410-5B
\[ L = 0.7H \]

\[ \theta = \text{angle of internal friction of retained soil.} \]

\[ \varphi_i = \text{angle of internal friction of reinforced infill soil.} \]

\[ \delta = \text{the lesser of } \varphi, \text{ or } \varphi_i, \text{ where} \]

\[ \varphi = \text{angle of internal friction of retained soil.} \]

**EXTERNAL STABILITY CALCULATIONS**

**SLOPING OR HORIZONTAL BACKFILL, } B \geq 0^\circ\)**

**Figure 410-5C**
IF C > B : LET C = B

EXTERNAL STABILITY CALCULATIONS
BROKEN-BACK BACKFILL, B > 0°

Figure 410-5D

\[
\delta = \text{the smaller of } \phi_i \text{ or } \phi
\]

\[
\phi_i = \text{angle of internal friction of reinforced infill soils.}
\]

\[
\phi = \text{angle of internal friction of retained soils.}
\]
\( \delta = \text{the lesser of } \phi, \text{ or } \phi', \text{ where} \)

\( \phi = \text{angle of internal friction of reinforced infill soils.} \)

\( \phi' = \text{angle of internal friction of retained soils.} \)

**EXAMPLE A: HORIZONTAL BACKSLOPE**

*Figure 410-5E*
EXAMPLE B: BROKEN-BACK BACKSLOPE

Figure 410-5F
MODULAR BLOCK GRAVITY WALL ANALYSIS

Figure 410-5G
\[ P_i = \frac{1}{2} \alpha H^2 K_s \]
\[ W_{gi} = (X_i)(Y_i)(1/)(\alpha g) \]

- \( P_1 \) = Lateral Earth Pressure
- \( P_2 \) = Surcharge Load
- \( W_{gi} \) = Weight of individual unit
- \( W_{si} \) = Weight of soil above units

**Figure 410-6A**

**BROKEN-BACK SLOPE - SIMPLIFIED EXAMPLE**
\( W_{si} = \text{Weight of soil above units} \)

\( W_{gi} = \text{Weight of individual unit} \)

\( P_1 = \text{Lateral Earth Pressure} \)

\( P_2 = \text{Surcharge Load} \)

\( W_{gi} = (X_i)(Y_i)(1)(\alpha g) \)

\( P_i = (1/2)\alpha H^2 K_s \)

SLOPING BACKFILL - SIMPLIFIED EXAMPLE

Figure 410-6B
Figure 410-7A

SHEET-PILING WALL - CONCRETE FACING DETAIL
TYPICAL SECTION

WF BEAMS

DEFORMED BAR CAGES
TYPICAL TYPES OF
LONGITUDINAL REINFORCING

LONGITUDINAL SHAFT SPACING
UNSTABLE SLOPE WARRANTING DRILLED-SHAFT SYSTEM

Figure 410-7B
ANCHORED WALL TYPICAL SECTION

Figure 410-7C
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<th>Soil Type</th>
<th>Depth Ranges</th>
<th>Total Unit Weight (lb / ft³)</th>
<th>Undrained (³)</th>
<th>Drained (⁴)</th>
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<td></td>
<td>Top of Stratum (ft)</td>
<td>Bottom of Stratum (ft)</td>
<td>Cohesion, C (lb / ft²)</td>
<td>Friction Angle, φ (deg)</td>
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<td>Fill – Sandy Loam with Slag and Cinders</td>
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<td>2 to 4</td>
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<td>Soft to Medium Stiff Clay Loam</td>
<td>2 to 4</td>
<td>6 to 8 (¹)</td>
<td>115</td>
<td>1500</td>
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<tr>
<td>Stiff to Hard Clay Loam (²)</td>
<td>6 to 8 (¹)</td>
<td>13 to 24</td>
<td>120</td>
<td>3000</td>
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<tr>
<td>Stiff to Hard Clay (²)</td>
<td>8 to 13</td>
<td>12 to 24</td>
<td>120</td>
<td>2500</td>
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<td>Stiff to Hard Loam</td>
<td>14 to 24</td>
<td>30 to 40</td>
<td>130</td>
<td>4000</td>
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Notes:  (1) Medium stiff clay loam extends to a depth of 15 ft below grade.  
(2) Clay loam and clay strata are interbedded. See specific soil boring logs for details of stratifications at specific locations.  
(3) Undrained strength parameters estimated from unconfined compression tests and calibrated penetrometer tests.  
(4) Drained strength parameters estimated from approximate correlations with Plasticity Index.

**SOIL PARAMETERS FOR SOIL-NAILED-WALL DESIGN**

Figure 410-7D
MIRCOPILE RETAINING WALL DETAILS

ROADWAY ABOVE SLOPE

SLOPE ABOVE ROADWAY

TYPICAL ROADWAY SLOPE STABILIZATION

TYPICAL PILE SECTIONS

TYPICAL EARTH RETENTION SECTION

MIRCOPILE RETAINING WALL DETAILS

Figure 410-7E
TO PROVIDE SLOPE STABILITY USING GEOSYNTHETICS

SLOPE REINFORCEMENT USING GEOSYNTHETICS TO PROVIDE SLOPE STABILITY

Figure 410-8A
SLOPE REINFORCEMENT PROVIDING LATERAL RESISTANCE DURING COMPACTION

Figure 410-8B
CHAPTER 412

Bridge Preservation

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There are approximately 19,000 bridges on public roads and streets in Indiana. Over 5,500 of these are on the State highway system. These bridges are designed and constructed to provide an adequate margin of safety and service life for the traveling public through the application of stringent design criteria and construction specifications. Nevertheless, all structural elements deteriorate over time, sometimes prematurely, and they will eventually present a hazard to the bridge users if not remedied. The Department’s efforts to maintain its bridge inventory in a state of good repair requires a balanced approach between bridge preservation and bridge replacement. This chapter focuses on bridge preservation.

412-1.0 INTRODUCTION TO BRIDGE PRESERVATION

Bridge preservation as defined by FHWA is actions or strategies that prevent, delay or reduce deterioration of bridges or bridge elements, restore the function of existing bridge, keep bridges in good condition and extend their life. Bridge preservation encompasses two work types: Preventative Maintenance and Rehabilitation.

Preventative maintenance treatments preserve the system, slow future deterioration, and maintain the functional condition. Preventative maintenance may be condition-driven or on a fixed cycle (scheduled) but does not include treatments that substantially increase structural capacity. Eligibility criteria contained in the Bridge and Culvert Preservation Initiative (BCPI) for bridge preventative maintenance treatments have been incorporated into this chapter. For a Local Public Agency (LPA) to participate in preventative maintenance activities, the LPA must have an INDOT-approved Bridge Asset Management Plan.

Rehabilitation treatments include major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects. Functional improvements such as adding a travel lane or increasing the vertical clearance are considered rehabilitation by the Department, but are not necessarily preservation.

The following sections describe project development procedures, design criteria, and type of treatments for Preventative Maintenance and Rehabilitation projects.
412-1.01 Bridge Preservation Program

Each district identifies bridge preservation projects through the Bridge Asset Management process. The process includes developing a preliminary scope of work and budget based on the need for preventative maintenance or rehabilitation treatments.

The need and extent of a bridge preservation project should consider the following:

1. Condition of structural elements;
2. Load carrying capacity;
3. Adequacy of seismic resistance;
4. Adequacy of traffic carrying capacity;
5. Need to address safety hazards;
6. Need to address geometric deficiencies; and
7. Economics of rehabilitation versus replacement.

Bridge Inspection reports created for the INDOT Bridge Inspection Program are a valuable resource when scoping a Bridge Preservation project. All routine, fracture-critical, and underwater reports can be found in the Bridge Inspection Application System (BIAS). For access to BIAS, contact the system administrator at INbridgeshelp@indot.in.gov.

412-1.02 Bridge Preservation Project Types and Treatments

The scope of work will determine whether a project is classified as either Preventative Maintenance or Rehabilitation. The scope of work should consider the following:

1. Age of the existing bridge deck;
2. Number of previous bridge deck overlays;
3. Quantity of estimated deck patching;
4. Remaining service life of the structure;
5. Cycle and cost of future repairs; and
6. Level One design criteria.

See Section 412-2.0 for design criteria and submittal process.

412-1.02(01) Preventative Maintenance Project
A Preventative Maintenance project aims to prevent, delay or mitigate deterioration. Treatments may be condition driven or on a fixed cycle (scheduled) but do not include structural or operational improvements of an existing bridge beyond its originally designed strength or capacity. For a Local Public Agency (LPA) to participate in preventative maintenance activities, the LPA must have an INDOT-approved Bridge Asset Management Plan.

Condition-Driven Preventative Maintenance

The following treatments are considered condition-driven preventative maintenance:

1. Bridge-Length Culvert Lining;
2. Bridge Deck Patching, partial or full depth;
3. Approach Slab Repair and Replacement;
4. Joint Repair, Replacement, and Elimination;
5. Mudwall Patching;
6. Bridge Deck Overlays - Polymeric (thin) or Concrete (rigid);
7. Spot Coating and Bridge Painting;
8. Substructure Patching or Sealing;
9. Superstructure Crack Mitigation;
10. Erosion Mitigation;
11. Debris Removal and Channel Cleaning;
12. Slopewall Repair and Replacement;
13. Bearing Repair and Replacement;
14. Scour Mitigation;
15. Bridge Deck Creak Sealing;
16. Brush Cutting¹ and Herbicide Application¹;
17. Railing Repairs¹;
18. Relief Joint Repairs¹; and
19. Upgrading End Treatments, Guardrail, Railing, Attenuators¹ ².

¹Items may only be included in a project which incorporates other preservation treatments.
²When found to be cost effective.

Eligibility criteria for condition-driven preventative maintenance treatments is shown in Figure 412-1A. Exceptions to the eligibility criteria may be considered when the bridge conditions or the proposed treatment do not meet the criteria but the recommended treatment is justified and proven to be effective. Exceptions require the approval of both the Bridge Asset Manager and the Bridges Division Director.
Scheduled Preventative Maintenance

Scheduled preventative maintenance treatments are more appropriately completed on a fixed cycle to maintain a structure at its current level and prevent or reduce deterioration. These treatments are typically accomplished by district maintenance forces. The Bridge Rehabilitation Department should be contacted prior to including a scheduled preventative maintenance treatment in a contract.

The following is a list of treatments considered as scheduled preventative maintenance:

1. Cleaning and Flushing Bridge Decks;
2. Substructure and Superstructure Washing;
3. Cleaning Deck Drains;
4. Cleaning and Lubricating Bearings;
5. Cleaning Joints; and

Eligibility criteria for scheduled preventative maintenance treatments is shown in Figure 412-1B.

412-1.02(02) Rehabilitation

Rehabilitation consists of treatments in which portions of the existing bridge remain and may include treatments which substantially increase structural capacity. In addition to the following, treatments which do not meet the eligibility criteria for preventative maintenance are considered rehabilitation.

1. Bridge deck replacement;
2. Bridge deck patching, exceeding maximum 10% deck area - partial or full depth;
3. Superstructure replacement;
4. Superstructure repair to restore structural capacity;
5. Coping reconstruction;
6. Bridge widening;
7. Increasing vertical clearance.

For questions regarding activities not listed above, the Bridges Division, Office of Bridge Design should be contacted.

412-1.02(03) Historic Bridge Rehabilitation
Historic bridge rehabilitation is a special project type which includes a Select or Non-Select historic bridge. Rehabilitation of a historic bridge must be undertaken in accordance with the programmatic agreement to manage and preserve Indiana’s Historic Bridges. See Section 412-5.0 for additional information on historic bridge rehabilitation.

412-2.0 PROJECT DEVELOPMENT

A Bridge Preservation project should be developed as either a Preventative Maintenance or a Rehabilitation project. The following activities may not be applicable to all Bridge Preservation projects but should be evaluated for each project. The design criteria and submittal process are distinct for each project type and discussed separately.

Scour Analysis

Each Bridge Rehabilitation project crossing a waterway requires a scour analysis. For an INDOT-maintained bridge, a Preventative Maintenance project does not require a scour analysis, except for Latex Modified Concrete (LMC) or other rigid overlay. For an LPA-maintained bridge, a scour analysis is required for each Preventative Maintenance project. When a scour analysis is required, the designer should contact the Bridges Division, Office of Hydraulics manager to determine if a scour analysis has been completed previously or should be completed as part of the current project. The determination should be documented in the Bridge Scoping Report or Bridge Preventative Maintenance Meeting Minutes.

For INDOT-maintained bridges, scour countermeasures deemed necessary by analysis or a scour-critical bridge inspection rating will be included in the scope of work for each Rehabilitation project. For Preventative Maintenance projects, the necessary scour countermeasures will be installed as part systematic scour projects. For LPA-maintained bridges, scour countermeasures deemed necessary by analysis or a scour-critical bridge inspection rating must be included in the scope of work for both Rehabilitation and Preventative Maintenance projects. A decision matrix is illustrated in Figure 412-2C, Scour Analysis and Countermeasures.

When a scour analysis is completed as part of the project, it must be signed, sealed, and dated by a professional engineer licensed in Indiana and submitted for review at least 30 days prior to the Preliminary Plans Submission. A template for documenting scour calculations is available from the Department’s Editable Documents webpage, under Hydraulics.

Load Rating
For a Preventative Maintenance project, the need for the existing load rating should be determined at the field inspection. Utilizing an LMC or other rigid overlay requires a load rating, but a polymeric or other flexible overlay does not. Other treatments that add significant deadload, e.g. replacing an aluminum railing with a concrete railing also require a load rating. For a Rehabilitation project a load rating is required regardless of the preservation treatment proposed. The load rating request should be submitted via email to Coordinator 8 and the Bridge Load Rating Engineer copied. Relevant plans sheets that are too large to email should be uploaded to ERMS. The Load Rating Request Form and Load Rating Summary are available from the Department’s Editable Documents webpage, under Bridges.

Asbestos Report

An Asbestos Report is required for all Bridge Preservation projects. The designer should contact the project manager early in the development of the project to determine if the report is on file or needs to be completed. It is the responsibility of the District Bridge Inspection Engineer to complete the Asbestos Report for each of the INDOT-maintained bridges within their district. For LPA projects the designer is responsible for coordinating the obtaining of the report with the LPA.

Environmental, Utilities & Railroads, and Right of Way

Each Bridge Preservation project is subject to NEPA and permitting requirements, utility and railroad coordination, and right-of-way acquisition requirements.

412-2.01 Preventative Maintenance Project

The goal of a Preventative Maintenance project is to maintain the existing infrastructure in good condition. Preventative maintenance projects are discouraged on structures with load ratings less than 1. Preventative maintenance treatments proposed on a structure with a load rating less than 1 require the approval of the Bridges Division Director.

412-2.01(01) Design Criteria

A Level One Design Criteria checklist is not required for permanent conditions of a Preventative Maintenance project. As such, a design exception is not required for the retention of an existing feature which does not satisfy INDOT criteria. However, a new design feature which does not satisfy INDOT criteria created by the project, or existing ones made worse, require a design exception, because such action in effect changes the structure as built. See Section 40-8.02 for design exceptions.
A Level One Design Criteria checklist is required for maintenance of traffic, except for detours.

Bridge Railing and Americans with Disabilities Act (ADA) requirements for a Preventative Maintenance project are discussed in Section 412-2.01(02) Submittal Process

The milestone submissions for a Preventative Maintenance project include the following. See Chapter 14 for specific documents to be submitted for review.

1. Initial Field Check;
2. Initial Field Check Minutes;
3. Preliminary Plans (Optional);
4. Final Plans; and
5. Final Tracings.

The approval of the Initial Field Check Minutes will serve as scope approval for a Preventative Maintenance project. A sample of Initial Field Check Minutes is available on the Department’s website at www.in.gov/dot/div/contracts/design/dmforms/, under Bridges.

The need for a Preliminary Plans submission is at the discretion of the Bridge Rehabilitation reviewer.

412-2.02 Rehabilitation Project

A Rehabilitation project encompasses preservation treatments which do not meet eligibility criteria for preventative maintenance in Section 412-1.02(01). Rehabilitation treatments also include bridge deck or superstructure replacement, widening for added travel lanes or increased clear roadway and other rehabilitation altering the out to out coping width.

412-2.02(01) Design Criteria

A Level One Design Criteria checklist is required. Typically the 3R criteria are appropriate for bridge rehabilitation; however, the use of 3R versus 4R criteria depends on the scope of work. For example, a structure to be widened as part of an added travel lanes 4R project should utilize the 4R design criteria. See Chapters 53, 54, and 55 for 4R, 3R Freeway, and 3R Non-Freeway criteria, respectively.
Existing features made worse require a design exception, regardless of the proposed feature meeting the criteria. Reductions in vertical clearance are of particular importance. See Section 40-8.02 for design exceptions.

Exceptions to Level Two design criteria should be documented in accordance with Section 40-8.02

**412-2.02(02) Submittal Process**

A Rehabilitation project is subject to NEPA requirements, permits, right-of-way or utility and railroad coordination. A scour analysis will be required for stream crossings. A topographic survey may be required depending on the project scope or as determined during project development. A geotechnical investigation will be required if substructure widening or new foundations are part of the project scope. The milestone submissions for a Rehabilitation project include the following.

1. Initial Field Check;
2. Bridge Scoping Report;
3. Preliminary Plans;
4. Final Plans; and
5. Final Tracings.

See Chapter 14 for specific documents to be submitted for review. Approval of the Bridge Scoping Report is required prior to proceeding to the Preliminary Plans submission. A sample of Bridge Scoping Report is available on the Department’s website at [www.in.gov/dot/div/contracts/design/dmforms/](http://www.in.gov/dot/div/contracts/design/dmforms/), under Bridges.

**412-2.03 Historic Bridge Rehabilitation Project [Rev. Feb. 2018]**

The *Programmatic Agreement among the Federal Highway Administration, the Indiana Department of Transportation, the Indiana State Historic Preservation Officer, and the Advisory Council on Historic Preservation Regarding the Management and Preservation of Indiana’s Historic Bridges* (Historic Bridges PA) governs the project development process for historic bridges in Indiana. The PA was executed on August 22, 2006, and is available on the Indiana Historic Bridges Inventory website at [http://www.in.gov/indot/2530.htm](http://www.in.gov/indot/2530.htm).
Historic bridge project development process documents are available on the Historic Bridge Inventory Summary & Results webpage at http://www.in.gov/indot/2531.htm.

Where a project involves a historic bridge, the bridge owner must prepare a Historic Bridge Alternatives Analysis for review and concurrence by the Department, after which it will be submitted to consulting parties for review and approval as part of the Section 106 consultation process.

See Section 412-5.0 for additional information on historic bridges and alternatives analysis.

412-2.03(01) Design Criteria [Rev. Feb. 2018]

See Section 412-5.0 for design criteria associated with evaluating alternatives.

412-2.03(02) Submittal Process [Rev. Feb. 2018]

A Historic Bridge Rehabilitation project should follow the submittal process for a standard Rehabilitation project, except that a Historic Bridge Alternatives Analysis will serve as the Bridge Scoping Report.

412-3.0 PROJECT CONSIDERATIONS

This section discusses project considerations and issues that relate specifically to bridge preservation projects. Designs should also follow guidance in other sections of Part 4 as economically feasible and applicable to bridge preservation projects.

412-3.01 General Preservation Considerations

412-3.01(01) AASHTO Bridge Design Specifications

INDOT practice is to design new structures using the current edition of the AASHTO LRFD Bridge Design Specifications. Rehabilitation often involves structures designed under a previous set of design specifications. The matrix in Figure 412-3A describes which design methodology is to be used based on work type.
412-3.01(02) Minimizing Bridge Joints

Whenever possible, bridge deck joints should be eliminated. Consideration to leave an expansion joint in place should include the joint type and expansion length, structure type, and remaining life of the bridge. End bent construction may be either semi-integral or fully integral to achieve this goal. Semi-integral design should be used where existing battered piling is to be reused. Battered piling may be removed by excavation and removal to a depth of one foot below the bottom of the new end bent. INDOT practice is to build fully integral end bents where possible.

The use of a link slab may be considered for both end bent and interior joint elimination. Standard details have not be created for link slabs. Project-specific details should be developed in coordination with the Bridge Rehabilitation Department.

412-3.01(03) Bridge Railing

1. Preventative Maintenance Project. Existing bridge railing may remain in place as part of a preventative maintenance project if the railing is in good condition and is functioning as originally intended with the following exception. All existing aluminum bridge railing on the NHS should be replaced when the treatments include a rigid overlay. The current standards apply to new bridge railing. The intent to leave substandard railing in place should be clearly stated in the Initial Field Check Minutes and agreed upon by the Bridge Rehabilitation reviewer.

2. Rehabilitation Project. The current standards apply to bridge railing (retained or new) as part of a Bridge Rehabilitation project. A design exception is required for bridge railing which do not meet the current standards or required test level.

412-3.01(04) Americans with Disabilities Act (ADA)

1. Preventative Maintenance Project. Flexible and rigid overlays are considered alterations in accordance with the Department of Justice/Department of Transportation Joint Technical Assistance on the Title II of the ADA. When an overlay is included in a Preventative Maintenance project, ADA-compliant curb ramps must be included in the scope of work. A Determination of Technical Infeasibility is required for curb ramps which cannot be constructed compliantly due to an existing constraint. See Section 40-8.04.

2. Rehabilitation Project. For a Rehabilitation project, ADA requirements are evaluated with other Level One criteria.
412-3.01(05) **Roadside Safety**

1. Preventive Maintenance Project. Roadside safety features should be upgraded to current standards when proved to be cost effective at part of a preventative maintenance project.

2. Rehabilitation Project. All existing roadside safety items including but not limited to guardrail, transitions and end treatments will be upgraded to current operational standards.

412-3.01(06) **Field Survey**

When a rehabilitation project involves bridge deck replacement, superstructure replacement, or widening of the substructure, a field survey may be warranted. A typical survey will involve a structure profile and a check of features such as cap and bridge seat elevations.

The purpose of the survey is to verify elevations so that datum corrections can be made in the plans and do not have to be determined in the field during construction. The Bridge Rehabilitation Department will approve the extent of survey before field work begins.

Existing vertical and horizontal railroad clearances should be measured and included in the bridge inspection report if the project involves a railroad.

412-3.01(07) **Bearings**

Due to low cost, low maintenance and superior seismic performance, INDOT prefers the use of elastomeric bearings when possible. The decision to replace other bearing types will be made on a project by project basis. The replacement of steel bearings with elastomeric bearings should only be considered when deck or bent cap removal is included in the rehabilitation.

Replacement of steel bearings which are in good condition may not be warranted even during a deck removal. Existing bearings may be left in place when bent cap modifications would add substantial cost to the project, unless replacement or retrofit is required for seismic considerations.

Bearing types should not be mixed on the same pier or bent due to differences in movement.

412-3.01(08) **Anchor Systems for Reinforcement**
Preference is to lap new reinforcement to existing reinforcement. However, it may not always be possible or cost effective to expose sufficient existing reinforcement to lap with new reinforcement. An anchor system may be used to develop the strength of new reinforcement by attaching it to existing concrete. This type of connection develops the strength of the reinforcing within the new construction but does nothing to develop continuity with reinforcing that remains in the existing concrete. Thus, no attempt to transfer moment across the interface of old and new concrete should be attempted. When moment transfer is desired, existing reinforcing should be exposed and adequately spliced. This can be achieved by lapping new reinforcing with existing reinforcing or through the use of an approved mechanical splice.

The allowable materials used to anchor reinforcement are found on the Approved Materials List of Chemical Anchor Systems. The pay item for this work is Field Drilled Holes in Concrete, which includes creating the hole and applying the grout. Grout material for field drilled holes in concrete should be either a high-strength, non-shrink, non-metallic, cementitious grout or an approved 100% solids chemical anchor system in accordance with the Standard Specifications.

The embedment requirements to obtain a given tensile pullout value will vary between products. To maintain consistency, the plans should show the minimum required pullout value with the reinforcement sized to project 6 inches into the hole. Necessary adjustments to the hole depth or diameter or reinforcement length is the responsibility of the contractor.

Where vertical holes are to be drilled into the top of a concrete bridge deck, a minimum clearance of 2 in. should be maintained between the bottoms of the holes and the bottom of the slab. Where vertical holes are to be drilled over a concrete- or steel-beam flange, the holes may be extended to the top of the flange. A reduction in capacity may occur depending on the edge distance and spacing of field drilled holes in concrete. Further information on spacing between anchors and minimum edge distances can be found on the Approved Materials List for Chemical Anchor Systems.

If an anchorage system is used, place a note on the plans identifying the connection as follows:

*Field drilled hole in concrete. Embed bar 6 inches with an approved anchor system. Minimum pullout = _______ kips.*

The value should be obtained from Figure 412-3B, Design Data for Anchor Systems. The values shown are for ultimate loads. If the full strength of the reinforcement is required, the values for 125% $f_y$ should be used. For all other connections, 100% $f_y$ may be used. These values are general guidelines for the embedment of reinforcement. Threaded or smooth dowels and headed studs can obtain significantly different values. The designer should review manufacturers’ literature before specifying anchor systems.
412-3.01(09)  Vehicular Vibration During Construction

All structures deflect when subjected to live loading, and many bridge widening projects are constructed with traffic on the existing structure. Fresh concrete in the deck is subjected to deflections and vibrations caused by traffic. Studies such as NCHRP 86 *Effects of Traffic-Induced Vibrations on Bridge-Deck Repairs* have shown the following.

1. Good-quality reinforced concrete is not adversely affected by jarring and vibrations of low frequency and amplitude during the period of setting and early strength development.

2. Traffic-induced vibrations do not cause relative movement between fresh concrete and embedded reinforcement.

3. Condition evaluation of widened bridges has shown the performance of widened section, with and without the use of a closure pour, to be satisfactory.

Additional measures need not be taken to prevent movement and vibration during concrete pouring or curing.

412-3.02  Bridge Deck Preservation Techniques and Considerations

412-3.02(01)  Hydrodemolition

INDOT practice is to use hydrodemolition on bridge rehabilitation projects requiring deck patching when cost effective. This method should be specified to remove unsound or delaminated concrete. Hydrodemolition uses high pressure water jets to remove unsound concrete material. This minimizes residual cracking around patched areas caused by manual methods such as jackhammers. Hydrodemolition may not be cost effective for small bridge decks or in isolated locations.

When hydrodemolition is utilized, partial depth patching is included in the cost of hydrodemolition and should not be included in the contract.

412-3.02(02)  Circular Crown
The normal cross slope on a new bridge deck is 2%. When an existing deck is to remain in place a minimum 1.5% cross slope may be retained. Maintaining the minimum cross slope is a requirement when an existing bridge deck to remain in place was built using a circular crown or other irregular form. An overlay depth exceeding 3 inches requires the approval of the Bridges Division, Office of Bridge Design manager.

412-3.02(03) Multiple Overlays

It is acceptable to remove an existing overlay and replace it with a new one. A new overlay should not be placed over an existing bridge deck overlay. The second overlay is counterproductive and adds to the dead load of the structure.

412-3.02(04) Rigid Overlay

Rigid overlay is a general term, used describe multiple types of concrete overlays for bridge decks.

The latex modified concrete (LMC) overlay has been successfully used by INDOT since the 1970s. This overlay can be expected to protect the bridge deck for approximately 15 years depending on traffic and site conditions. The use of a rigid overlay requires proper preparation of the bridge deck by milling and removal of unsound concrete. The minimum LMC overlay thickness is 1.5 inches.

The use of a microsilica-based overlay on a State-owned bridge requires the approval of the Bridges Division, Office of Bridge Design manager.

Typically, a bridge deck should be replaced rather than overlaid a third time. A bridge deck should also be replaced if the amount of patching exceeds 35% of the deck area. See Figure 412-1A for the preventative maintenance eligibility requirements for rigid overlays.

412-3.02(05) Asphalt Overlay

This method was used in the past with limited success. However, new research, performance and materials may indicate that asphalt overlays over sheet membranes are an effective method of deck repair. Designers should contact the Bridge Rehabilitation Department prior to use.
412-3.02(06)  Flexible Overlay

A flexible overlay, also referred to as polymeric or thin overlay, has been utilized more in recent years. A flexible overlay is not appropriate for bridge decks which have previously received a concrete overlay and better suited for bridge decks with small amounts of delamination. See Figure 412-1A for the preventative maintenance eligibility requirements for flexible overlays.

412-3.02(07)  Deck Patching

Deck patching is often required for bridge deck overlay projects. Full-depth patching should be estimated during the field check as the area to be replaced. If full depth patching is required, a minimum 50 ft² should be estimated. Partial-depth patching is not included as a pay item and estimated quantities are not required for projects with hydrodemolition.

If a large contiguous area of the deck is to be replaced, details for replacement of reinforcement should be provided. Actual deck patching areas will be determined by the amount of unsound concrete removed during the hydrodemolition process or by deck chaining for projects without hydrodemolition.

412-3.02(08)  Additional Bridge Deck Overlay

Additional bridge deck overlay material is required to account for unsound concrete areas and surface irregularities. For bridge deck overlay projects, include an estimate of the pay item for additional bridge deck overlay. The estimate for this quantity should be calculated as follows:

\[
\text{Additional Bridge Deck Overlay (yd}^3\text{)} = (0.00617) \times (\text{Estimated Unsound Concrete Area, ft}^2) + (0.00486) \times (\text{Overlay Area, yd}^2)
\]

The estimated unsound concrete area should be determined at the field check and is equivalent to the area of the bridge deck that requires partial-depth patching. The coefficient assumes the patching is an average of 2 in. deep.

The estimated overlay area is the total exposed bridge deck surface. The coefficient assumes an additional 10% of the bridge deck area to compensate for surface irregularities.

The coefficients are derived as follows:
\[
0.00617 = (2\text{in.})\left(\frac{1\text{ft}}{12\text{in.}}\right)\left(\frac{1\text{yd}^3}{27\text{ft}^3}\right)
\]
\[
0.00486 = (1.75\text{ in.})\left(\frac{1\text{yd}}{36\text{in.}}\right)(.10)
\]

The material required for full-depth patching is included with the pay item for Bridge Deck Patching, Full Depth. It should not be included in the pay item for Bridge Deck Overlay, Additional.

412-3.02(09) Profile Grade Adjustments

Frequently, the profile of the proposed deck surface will be higher than the existing. An asphalt wedge should be constructed to transition between the new deck surface and the existing approach pavement. A pavement design is required to determine the appropriate pavement section.

An asphalt wedge consists of a continuation of the bridge deck profile which extends 30 ft beyond the end of the approach slab. Beyond the grade continuation, the grade tapers back to the existing at a rate of 1 inch in 60 ft. The minimum thickness of asphalt wedge material is 1.5 inches. This requires a saw cut and transition milling at the end of the wedge. Considerations include existing pavement type and condition, grade change dimension and approach roadway geometry.

If the adjacent roadway pavement is concrete, pavement replacement is often the best method for tying into existing profile grades. The same taper rates apply for concrete pavement as for asphalt wedges.

See the INDOT Standard Drawings series 722-HMAW and 306-TMPT for more details.

412-3.02(10) Overlay Dams

Overlay Dam. An overlay dam is thickened section of overlay material at the bridge end. The existing reinforcement is exposed and then encased with overlay material providing better protection from damage caused by snowplow blades and other hazards. Hydrodemolition or chipping hammers are used to remove an additional depth of existing deck at the bridge end. Overlay dams should be placed at expansion joints. Placement of overlays dams adjacent to 1A joints is optional. See Figure 412-3C for typical overlay dam details.
412-3.03 **Superstructure Preservation Techniques and Considerations**

**412-3.03(01) Dead Load Deflections**

Unless the widened structure is completely prefabricated, deflection of the beams or girders will occur due to superimposed dead loads, such as the deck slab, diaphragms, or railings. To prevent the undesirable effects of this deflection, the widening should initially be built above the grade of the existing structure to allow for dead-load deflection. The deflected widening should approximate the grade of the existing structure. If proper provisions are not made to accommodate the dead-load deflection, construction and maintenance problems will ensue. Where the dead-load deflection exceeds 2 in., a closure pour should be considered to complete the attachment to the existing structure. A closure pour serves two useful purposes: it defers final connection to the existing structure until after the deflection from the deck-slab weight has occurred, and it provides the width needed to make a smooth transition between differences in final grades that result from design or construction imperfections.

In terms of the effects of dead-load deflection, two groups of superstructure types can be distinguished: precast-concrete-beam or steel-beam construction. The largest percentage of deflection occurs when the deck concrete is placed. For cast-in-place construction, e.g., a reinforced-concrete slab bridge, the deflection occurs after the falsework is released.

In precast-concrete-beam construction, dead-load deflection after placement of the deck is usually insignificant. In a cast-in-place structure, the dead-load deflection continues for a lengthy time after the falsework is released. In a conventionally-reinforced concrete structure, approximately one half to three quarters of the total deflection occurs over a four-year period after the falsework is released due to shrinkage and creep. The theoretical differential deflection that occurs between a new and existing structure is difficult to account for during design. Past performance indicates, however, that theoretical overstress in the connection reinforcing has not resulted in maintenance problems. It is assumed that some of the additional load is distributed to the original structure with no difficulty, or its effects are dissipated by inelastic relaxation. When a closure pour is utilized, the width should account for the amount of dead load deflection that is expected to occur after the closure is placed. A minimum closure width of 20 in. is recommended.

INDOT is satisfied with the performance of its bridge decks that are widened without the use of closure pours. This satisfactory performance also applies to a deck replacement that is poured in two phases while maintaining traffic and without the use of closure pours. Consequently, deck widening and phased deck replacement do not require closure pours unless the designer or district representative recommends otherwise. An example of where a closure pour may be warranted is for a steel beam or girder structure where uplift could occur.
412-3.03(02) Longitudinal Joints

Past performance indicates longitudinal expansion joints in a bridge deck between a widened portion and the existing portion have been a continuous source of bridge maintenance problems. Therefore, longitudinal expansion joints should not be used to separate existing and widened bridge decks.

When widening between adjacent structures, provide a minimum 1” open joint between the copings. An example would be the addition of travel lanes to twin structures separated by concrete barriers.

Experience has shown a positive attachment of the widened and original decks provides a better riding surface, usually presents a better appearance, and reduces maintenance problems. A positive attachment of old and new decks should be made for the entire length of the structure. The preferred method for attachment is to lap reinforcement. The following recommendations should be considered when widening existing decks.

1. A structure should be widened by removing concrete for a distance sufficient to allow adequate length for lapping the original transverse deck reinforcing to that of the widening.

2. Where removal will not provide sufficient lap length, reinforcing should either be doweled to the widened section or have transverse reinforcing exposed and extended by means of a mechanical lap splice. The design should ensure the development of existing reinforcement.

3. A structure with no overhangs, such as a longitudinally reinforced concrete slab, may be attached by doweling the existing structure to the widening. Double-row patterns for the dowels are preferred over a single row. Benching into existing concrete as a means of support has proven to be unsatisfactory and should be avoided.

4. Removal of the deck past the outside beam line will result in a cantilever slab condition. The design should ensure that the deck can resist the loadings anticipated during construction.

5. A longitudinal construction joint located over a beam flange should be avoided when possible. Longitudinal construction joints should preferably be aligned with permanent lane lines. These joints tend to be more visible than the pavement markings during adverse weather conditions.
412-3.03(03) Concrete Slab Structures

The designer should assess the condition of the coping and the portion of the deck from the coping to the first construction joint. This area is often in poor condition due to the presence of water and ice melting chemicals and consideration should be given to removal of this portion.

The overall condition of the superstructure should be evaluated. Large amounts of patching are rare in this type structure, so excessive patching may indicate the need for replacement of the structure.

412-3.03(04) Reinforced Concrete Girder Structures

RC Girder structures should be evaluated for replacement. Their age and condition do not usually make them a candidate for widening. A cost study should be performed to validate the need for replacement versus widening.

Deck replacement or widening of RC Girders should be considered only where a cost comparison indicates the replacement is not cost effective. Due to tee beam action, the designer should analyze the effect of removing the deck over girders to remain in place. Temporary girder supports may be required and will likely add significant cost and time to the project.

For structures with RC Girder end spans and steel or concrete beam main spans, the concrete girders should be removed and replaced in order to eliminate the deck joint. The new or widened superstructure should be made continuous if possible.

412-3.03(05) Prestressed Concrete Girder Structures

The following should be considered when rehabilitating a bridge with prestressed concrete girders:

1. **Continuity.** Simple span structures should be made continuous when the bridge deck is being replaced. Factors to consider include span ratio, deck reinforcement, diaphragm reconstruction and expansion lengths.

2. **Girder Repair.** Deterioration at the ends of prestressed concrete girders due to joint leakage or shear cracking can be repaired by encasing the girder ends in concrete. An example is constructing an integral or semi-integral end bent at the support.
An alternate to repair using concrete encasement is to apply a fiber wrap system at the girder end. Cracks and spalls in the girder are repaired by epoxy injection or other methods, and then the fiber wrap system is applied to the girder surface. Fiber wrap may only be used for encasing deterioration and may not be utilized for adding capacity.

For prestressed girders that have damaged or exposed prestressing strands, the designer should evaluate if girder replacement is required. Prestressing strands can be repaired or replaced in lieu of deck and beam replacement. Repair methods include mechanically coupling strands or encasing strands in high strength grout or concrete.

3. **Field Drill Holes.** Field drilled holes can be used to retrofit existing prestressed girders, but are not desirable. Drilled holes should only be placed in the web and should be placed to avoid draped prestressing strands and mild reinforcement. Where the girder is to be encased in concrete, such as an integral end bent situation, drilling near reinforcement is acceptable. The designer should review bridge plans and shop drawings for strand and reinforcement locations.

4. **Diaphragms.** Replace existing pier and end diaphragms when the bridge deck is to be replaced. The designer should evaluate if interior (intermediate) diaphragms should be replaced based on existing conditions.

Steel diaphragms should be considered when widening existing prestressed girder structures.

### 412-3.03(06) Steel Beam and Plate Girder Structures

The following should be considered when rehabilitating a bridge with structural steel beams and plate girders:

1. **Shear Connectors.** Introducing composite action between the deck and the supporting beams is a cost-effective way to increase the strength of the superstructure. Beams should be made composite whenever the deck is being removed.

   Composite action considerably improves the strength of the upper flange in a positive-moment area, but its beneficial effect on the beam as a whole is only marginal. The combination of composite action in conjunction with selective cover plating of the lower flange is the most effective way of beam strengthening.
Introducing composite action near joints prevents the deck from separating from the beams, thus increasing the service life of the deck.

2. **Grinding and Peening.** If the penetration of surface cracks is small, the cracked material can be removed by means of selective grinding without substantial loss in structural material. Grinding should be performed parallel to the principal tensile stresses when possible. Surface striations should be removed because they may initiate future cracking.

   The most common application of grinding is to the toe of the fillet weld at the end of the cover plate so that it is in accordance with the fatigue requirements. Grinding can also be used where beams have been nicked due to the sawing off of an old deck.

   Peening is an inelastic reshaping of the steel at the surface locations of cracks, or of potential cracks, by use of ultrasonic methods. This procedure not only smoothes and shapes the transition between weld and parent metal, it also introduces compressive residual stresses that inhibit the cracking. Peening is most commonly used where fatigue is an issue such as at the ends of cover plates or welded diaphragm connections.

3. **Drilled Holes.** At the tip of a crack, tensile stress exceeds the ultimate strength of the metal causing rapid progression when the crack size attains a critical level. Drilled holes placed at the tips of the crack are commonly used to alleviate this problem. The tips of the crack should be located by means of an established crack-detection method.

   Sections should be checked to ensure that the reduced-member capacity due to the crack and the drilled hole is still adequate. The mitigation of the stress concentration at the tip is more critical, however, than the loss of net section. Hole size should be optimized to eliminate stress concentrations while considering loss of cross sectional area.

4. **Welded Diaphragms.** Current practice is to use only bolted connections for all new or replacement diaphragms. Welded diaphragms are a fatigue concern, however, retrofitting of existing welded diaphragm connections should be undertaken only when cracks are noticed upon inspection.

5. **Welded Cover Plates.** Termination of certain cover plates may be a fatigue concern. Typical mitigations should include peening or bolted retrofits. However, mitigations should be performed only when cracks are noticed upon inspection.

6. **Other Fracture-Critical and Fatigue-Prone Details.** An E or E’ fatigue category detail, and Hoan detail should be identified. The need for retrofit will be determined by analysis.
7. **Beam Straightening.** This technique is restricted to hot-rolled steels. Steel deriving its strength from cold drawing or rolling tend to weaken when heated. The premise of heat-straightening is that the steel, when heated to an appropriate temperature, usually to cherry color, loses some of its elasticity and deforms in a plastic (inelastic) manner. This enables the steel to rid itself of built-up stresses or permits forcing the steel into a desirable shape or straightness. The steel should not be overheated. Accordingly, this technique should be implemented only by those individuals having experience with it. The heating temporarily reduces the resistance of the structure. Measures such as vehicular restriction, temporary support, or temporary post-tensioning, may be applied as appropriate. Replacement should be considered when beams have been heat straightened more than once.

**412-3.03(07) Construction Loading**

For a Rehabilitation project with a beam or girder superstructure, construction loadings should be evaluated in accordance with AASHTO LRFD Article 3.4.2, described in Chapter 403-4.0. Engineering judgment should be used when determining the need to check construction loadings for deck replacements.

**412-3.03(08) Local Public Agency Adjacent Box Beam Bridges**

An LPA may place a 6 in. reinforced concrete deck over an adjacent box beam bridge that does not have a deck as preventative maintenance. The superstructure and substructure must meet the component rating criteria as shown in Figure 412-1A, Condition Driven Preventative Maintenance Eligibility Criteria.

**412-3.04 Substructure Preservation Techniques and Considerations**

**412-3.04(01) Geotechnical Assistance**

The Pavement Division, Office of Geotechnical Services should be notified early in the design process under each of the following circumstances:

1. new piling or foundations are to be constructed;
2. existing embankments are to be widened or require retaining walls;
3. new retaining walls are to be constructed; and
4. other known soils issues exist at the site.

412-3.04(02) Foundations

Foundation recommendations are the responsibility of the Pavement Division, Office of Geotechnical Services. The Office should be contacted early in the design of the project. Designers may provide suggested foundation types based on existing foundations.

Designers should consider differential settlement when designing foundations. Widened footings should be placed on piling unless original footings are founded on rock or some similarly hard stratum. This applies even when the existing structure makes use of spread footings.

For a stream crossing where the original structure is founded on spread footings and non cohesive soil, widening should not be accomplished using driven piling. Other types of piling to consider are micro piles, caissons, drilled shafts and auger cast piles.

When existing footings are on piling, pile driving records should be reviewed for guidance in estimating quantities.

412-3.04(03) Frame Bents

Frame bents may be widened by constructing additional columns and extending the cap. Connection of the new and existing cap may be made by dowelling into the existing cap unless loads are to be placed between the end of the existing cap and first new column. In this type of widening, a portion of the existing cap should be removed so that existing reinforcement can extend into the new construction and lap with new reinforcement. Existing crashwalls may be connected to new crashwalls by dowelling. The height of both existing and new crashwalls should meet current standards.

412-3.04(04) Solid Stem and Cantilever Piers

Solid wall type piers should generally be widened in kind. New construction may be connected to existing through the use of dowelling when the end of the existing wall is a flush surface. When the end of the existing wall is rounded, concrete should be removed back to the radius point. Where
the existing wall contains a cantilever or hammerhead, the cantilever can either be removed or the area beneath the cantilever can be filled with concrete.

Alternatively, walls may be widened with columns and a cap beam. A solid wall should be provided to a level above the high water at a stream crossing. The Bridges Division, Office of Hydraulics should be contacted for guidance.

412-3.04(05) End Bents

Every effort should be made to eliminate joints which exist above end bents. When converting an existing expansion end bent to an integral situation, all existing battered piles should be cut off below the bottom of the new end bent and may not be reused.

If the existing end bent is in good condition, consideration should be given to converting the end bent to a semi-integral situation. This will allow reuse of existing battered piles. When new concrete is placed adjacent to existing concrete, consideration should be given to the following:

1. When only one new pile is added past the end of the existing cap, a portion of the existing cap should be removed so that existing reinforcement can extend into the new construction and lap with new reinforcement.

2. When multiple piles are being added past the end of the existing cap, connection of the new and existing cap may be made by dowelling into the existing cap unless loads are to be placed between the end of the existing cap and first new pile. A portion of the existing cap should be removed so that existing reinforcement can extend into the new construction and lap with new reinforcement.

412-3.05 Special Project Considerations and Techniques

The following topics should be considered on a project-specific basis.

412-3.05(01) Cathodic Protection

A cathodic protection system should be considered for a location where traffic-maintenance costs are very high and where additional service life between repairs would be advantageous.
The advantage of cathodic protection is that it can halt the progress of corrosion without the removal of chloride-contaminated concrete. Corrosion requires an anode – a point on the reinforcement where ions are released. Cathodic protection is the application of direct current such that the steel becomes cathodic to artificial anodes located on the deck. These anodes usually consist of sheets of thin wire mesh. A relatively small DC rectifier operating on AC line voltage and a control panel are located beneath the bridge.

A cathodic protection system need not operate 24 hours per day to be beneficial. Therefore, it can be powered by means of solar panels or in line with the highway-lighting system.

412-3.05(02) Beam Straightening

This technique is restricted to hot-rolled steels. Steels deriving their strength from cold drawing or rolling tend to weaken when heated. The premise of heat-straightening is that the steel, when heated to an appropriate temperature, loses some of its elasticity and deforms in a plastic (inelastic) manner. This enables the steel to rid itself of built-up stresses or permits forcing the steel into a desirable shape or straightness. The steel should not be overheated. Accordingly, this technique should be implemented only by those individuals having experience with it. The heating temporarily reduces the resistance of the structure. Measures such as vehicular restriction, temporary support, or temporary post-tensioning, may be applied as appropriate.

412-3.05(03) Post Tensioning

External post-tensioning can be applied to either steel or concrete structural members to reduce tensile stresses, to strengthen beams, or to make simply-supported beams continuous. There is a variety of successful methods of post-tensioning. The designer should research the most updated literature prior to use.

412-3.05(04) Deadman Anchor

The lateral force exerted by retained earth or stone, and superimposed gravitational loads thereon, tends to push forward and rotate an abutment or retaining wall. One solution for this problem is the application of a deadman.
A deadman is a heavy solid mass, usually concrete blocks that are connected to the retaining structure by long steel rods. A deadman is located in a stable earth mass well behind the structure. For wingwalls, or walls located on both sides of the roadway, they can be connected together by steel rods.

The rods should be protected against corrosion, and the effects of differential settlement should be considered.

Since this stabilization technique modifies the wall support from a cantilever to simple span pinned, the wall reinforcement should be checked for the revised moments.

**412-3.05(05) Fiber Reinforced Polymer Composite Materials [Rev. Sep. 2016]**

Externally bonded fiber reinforced polymer (FRP) composite materials or “fiber wrap” may be considered for bridge preservation and retrofitting. However, the Department currently does not allow the use of FRP systems to restore the structural capacity of bridge components.

Preservation may include the repair of deteriorated concrete components, such as concrete piers near leaking joints, or the repair of prestressed concrete girders due to vehicle impact. Seismic retrofitting of columns may include fiber wrap in conjunction with adding confinement reinforcement.

Considerations for the use of FRP composite materials should include the cost compared to conventional construction materials, along with the benefit of shorter construction duration and less impacts to the travelling public.

**412-3.05(06) Scour Countermeasures**

When signs of scour at existing foundations are apparent, measures should be taken to correct the problem. The stability of a streambed and banks is largely a function of water velocity and the size of the material constituting such bed and banks. If the size exceeds critical dimensions, scour will not likely occur.

Artificially placed protective material can consist of natural stone, specially-made concrete, or recycled (crushed) concrete. The weight of the riprap material should be considered in the design of footings and foundations. For steeper embankments, the riprap may be enclosed in galvanized, wire mesh envelopes called gabions.
For more information, see Part 2 of the Manual.

412-3.05(07) Seismic Evaluation and Retrofit

For a Preventative Maintenance project, seismic evaluation is not required.

For a Rehabilitation project, seismic evaluation is not required if any of the following conditions are present:

1. The bridge is a single span;
2. the shelf length is greater than the calculated length of need; or
3. the existing bents are integral or semi-integral.

Where none of the above conditions are present, the decision to incorporate a seismic retrofit into the project scope should take into consideration the location and importance of the structure; the type and magnitude of rehabilitation; and cost. The discussion should be documented in the scoping field check minutes.

Guidance on specific seismic retrofit measures can be obtained from the current edition of the *Seismic Retrofitting Manual for Highway Structures, Part 1 – Bridges* published by the Federal Highway Administration.

412-4.0 CONDITION SURVEYS AND TESTS

The type and appropriate extent of bridge rehabilitation should be based on information acquired from condition surveys and tests. The selection of the condition surveys and tests for a proposed project is determined by a site-specific assessment for each bridge. The structural factors to be considered are as follows:

1. age;
2. estimated remaining life (i.e., before bridge replacement is necessary);
3. size;
4. historic significance; and
5. potential investment in bridge rehabilitation.

The information normally available that may be requested if deemed pertinent is as follows:

1. original design plans and previous rehabilitation plans;
2. as-built plans;
3. shop drawings;
4. pile-driving records;
5. geotechnical report;
6. previous surveys;
7. accident records;
8. flood and scour data, if applicable;
9. traffic data;
10. roadway functional classification;
11. current load rating;
12. bridge inspection reports;
13. structural ratings (sufficiency, operating, inventory); and
14. maintenance work performed to date.

The condition surveys and tests to be selected should be those appropriate for the bridge site conditions based on an assessment of the structural factors and the available information. The most effective method for determining the condition of a structure is visual inspection. Material tests have proven to be less effective, therefore contact the Bridges Division, Office of Bridge Design manager for further guidance and testing coordination.

### 412-4.01 Bridge Deck

For the purpose of this section, the bridge deck includes the structural continuum directly supporting the riding surface, deck joints and their immediate supports, curbs, barriers, reinforced-concrete bridge approaches, and utility hardware. The bridge deck and its appurtenances provide the services as follows:

1. support and transmittal of wheel loads to the primary structural components;
2. protection for the structural components beneath the deck;
3. lateral bracing for girders;
4. a smooth riding surface;
5. drainage of surface runoff; and
6. safe passageway for vehicular and bicycle or pedestrian traffic, e.g., skid-resistant surface, bridge railings, guardrail-to-bridge-railing transitions.

Deterioration in these services warrants investigation and possible remedial action. The most common cause of bridge deck deterioration is the intrusion of chloride ions from roadway deicing salts into the concrete. The chloride causes formation of corrosive cells on the steel reinforcement, and the corrosion product (rust) induces stresses in the concrete resulting in cracking, delamination...
and spalling. Chloride-ion (salt) penetration is a time-dependent phenomenon. There is no known way to prevent penetration, but it can be decelerated such that the service life of the deck is not less than that of the structure. Salt penetration is, however, not the only cause of bridge-deck deterioration. Other significant problems include the following.

1. **Freeze-Thaw.** This results from inadequate air content of the concrete. Freezing of the free water in the concrete causes random, alligator-type cracking of the concrete and then complete disintegration. There is no known remedy other than replacement.

2. **Impact Loading.** This results from vehicular kinetic energy released by vertical discontinuities in the riding surface, such as surface roughness, delamination, and inadequately-set or damaged deck joints. Remedial actions include surface grinding, overlaying or replacement of deck concrete, or rebuilding deck joints.

3. **Abrasion.** This results from contact with metallic objects, such as chains or studs attached to tires. Remedial actions include surface grinding or overlaying.

The objective of the condition surveys and tests is to quantify the extent of deterioration based on INDOT criteria to determine the appropriate remedial action.

### 412-4.01(01) Visual Inspection

Visual inspection and sounding are the primary methods for determining existing condition. Material tests should only be requested in special circumstances.

1. **Description.** A visual inspection of the bridge deck should identify the following:

   a. the approximate extent of cracking, delamination, spalling and joint opening;
   b. evidence of corrosion;
   c. evidence of efflorescence, discoloration, or wetness at the bottom of bridge deck;
   d. deformation in the riding surface or ponding of water;
   e. operation of bridge deck joints;
   f. functionality of bridge deck drainage system;
   g. bridge railing and guardrail-to-bridge-railing transitions accordance with current Department standards.
   h. deterioration and loss in a wooden deck;
   i. compatibility with geometric design criteria; and
   j. condition of bridge railing and bridge railing transitions and compliance with current standards.
2. **Purpose.** The purpose of visual inspection is to approximate the extent of cracking, corrosion, delamination, spalling and other deterioration identified during visual inspection and determine if a more extensive inspection is warranted.

3. **Consideration.** Visual inspection should be used for each potential deck rehabilitation project.

4. **Analysis of Data.** Based on the extent of bridge-deck delamination, the following will apply.

   a. Delamination of 5% of surface area is a rule of thumb for considering remedial action; and
   b. Delamination of 30 to 40% of surface area is a rule of thumb for considering bridge deck replacement.

Remedial work or deck replacement outside the above guidelines may be appropriate and should consider the following:

- traffic control;
- timing of repair;
- age of structure;
- AADT;
- slab depth;
- structure type; and
- hazard potential to other traffic (e.g. vehicular, cyclist, pedestrian).

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**412-4.01(02) Sounding**

1. **Description.** Sounding establishes the presence of delamination, based on audible observation by chain drag, hammer, or electromechanical sounding device. It is based on the observation that delaminated concrete responds with a hollow sound when struck by a metal object. For more information refer the design should refer to ASTM D 4580.

2. **Purpose.** The purpose of sounding is to determine the location and extent of delamination.

3. **Consideration.** Sounding should be used on each deck rehabilitation project, except where proper traffic control cannot be provided during the test.
4. **Analysis of Data.** Quantities are approximate for bid purposes only and should be rounded off to the nearest 5%.

### 412-4.01(03) Half-Cell Method

1. **Description.** Copper-copper sulphate half-cell method for the measurement of electrical potential is used as an indicator of corrosive chemical activity in the concrete. For more information, the designer should refer to ASTM C 876.

2. **Purpose.** The purpose of the half-cell method is to determine the level of activity of corrosive cells in the bridge deck with non-epoxy coated steel.

3. **Consideration.** The method should be used if non-destructive testing is warranted.

4. **Analysis of Data.** A potential difference of -0.35 V or more negative indicates active corrosion. A potential difference of -0.20 V or less indicates a state of no corrosion. The range between -0.20 and -0.35 V is considered questionably active.

### 412-4.01(04) Coring

1. **Description.** Cores of 1½ in. to 3¼ in. diameter are taken by means of a water-cooled, diamond-edged rotating shell.

2. **Purpose.** The purpose of coring is to establish strength, composition of concrete, crack depth, position of reinforcement, delamination, and profile of chloride content and gradient.

3. **Consideration.** Coring should be completed when questions exist relating to the compressive strength or soundness of the concrete, or if the visual condition of the reinforcement is desired. Coring should also be completed when compression or chloride analysis tests are requested.

4. **Analysis of Data.** Concrete cover of less than 2 in. is considered inadequate for corrosion protection. A concrete compressive strength of less than 3000 psi is considered inadequate. Core locations can have a significant impact on the findings.
412-4.01(05) Chloride Analysis

1. **Description.** Chloride analysis consists of a chemical analysis of pulverized samples of bridge-deck concrete extracted from slices of 1½ in. diameter cores.

2. **Purpose.** The purpose of chloride analysis is to determine the chloride content profile from the deck surface to a depth of about 3 in. or greater.

3. **Consideration.** The results from a chloride analysis test have proven difficult to analyze. The Office of Materials Management (OMM) does not promote the use of this testing method and the validity of test results should be discussed with the OMM before use.

4. **Analysis of Data.** In the past a chloride content of 0.06 lb/cft of concrete at the level of top reinforcement was used as a rule of thumb for indicating a potential for corrosion to occur in the uncoated reinforcement of the deck. However, new bridge deck concrete has been known to have chloride content in this range. Therefore, the results of this test should not be used solely in determining the degree of rehabilitation. The locations of cores can also have a significant impact on the findings.

412-4.02 Superstructure

For the purpose of this section, the superstructure includes all structural components located above the bearings, except the deck. For a bridge without bearings, such as a rigid frame or fixed arch, this includes every visible structural component, except the deck. The following describes condition surveys and tests which may be performed on the superstructure elements to determine the appropriate level of rehabilitation.

412-4.02(01) Visual Inspection

Visual inspection and sounding are the primary methods for determining existing condition. Material tests should only be requested in special circumstances.

1. **Description.** A visual inspection of the superstructure should include an investigation of the following:

   a. surface deterioration, cracking, and spalling of concrete;
b. major loss in concrete components;
c. evidence of efflorescence;
d. corrosion of reinforcement or prestressing strands;
e. loss in exposed reinforcement or prestressing strands;
f. corrosion of structural-metal components;
g. loss in metal components due to corrosion;
h. cracking in metal components;
i. excessive deformation in metal components;
j. loosening and loss of rivets or bolts;
k. deterioration and loss in wood components;
l. damage due to collision by vehicles, vessels, or debris;
m. damage due to leakage through deck joints;
n. ponding of water on abutment seats;
o. state and functionality of bearings; and
p. presence of low-fatigue-life components.

3. **Purpose.** The purpose of visual inspection is to record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted.

4. **Consideration.** It should be used on each bridge rehabilitation project.

5. **Analysis of Data.** Analysis should be performed as required.

**412-4.02(02) Fracture-Critical Member**

A fracture-critical member is defined as a structural metal component in tension whose failure would render the bridge dysfunctional or cause its collapse. A major portion of this determination relates to redundancy. For example, loss of a girder in a multi-girder or continuous girder structure may not be critical, while inadequate welding of a stiffener in other situations may be critical.

If the issue arises, criticality should be investigated by an experienced structural engineer.

**412-4.02(03) Tests for Cracking in Metals**

Such tests are used to determine the appropriate remedial action if the visual inspection revealed the existence of cracking in a steel structure. The extent and size of cracks should be established.
The following are the most common test methods used in locating cracks in a steel structure, and for measuring their extent and size:

1. **Dye-Penetration Test.** The surface of the steel is cleaned, then painted with a red dye. The dye is wiped off. If a crack is present, the dye penetrates the crack. A white developer is painted on the cleaned steel and cracks are indicated where the red dye bleeds from the crack.

2. **Magnetic-Particle Test.** The surface of the steel is cleaned and sprinkled with fine iron filings while a strong magnetic field is induced in the steel. Magnetism is not resisted by the void in the cracks; therefore, the particles form a footprint thereof.

3. **Acoustic Test.** Although the above two methods require no special equipment, the acoustic method needs both a transmitter and a receiver. The method works on the principle that cracks reflect acoustic waves. It can only establish the presence of cracks.

4. **Radiogram.** This is a highly reliable but cumbersome and expensive test because it requires a medium for producing x-rays which penetrate the cracks and mark the film located at the other side.

5. **Ultrasonic Test.** This consists of the use of testing devices that use high-frequency sound waves to detect cracks, discontinuities, and flaws in materials. The testing depends a great deal on the expertise of the one conducting the test to interpret the results.

All tests should be conducted by, at a minimum, a Level II ANSI approved technician. For more information, see *FHWA-RD-89-167, Fatigue Cracking of Steel Bridge Structures*; *AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges*; or *FHWA-NH1-90-043, Economic Design of Fracture Critical Members*.

### 412-4.02(04) Fatigue Analysis

1. **Description.** Fatigue is defined as a progression in the crack size caused by cyclical loading to a critical dimension at which cracking is no longer effectively resisted, thus, leading to fracture of the component. The progression is a function of the following:

   a. crack size;
   b. location of crack (i.e., structural detail);
   c. energy absorbing characteristics of metal;
   d. temperature; and
2. **Purpose.** The purpose of fatigue analysis is to establish type and urgency of remedial action.

3. **Consideration.** This analysis should be performed where cracks found by visual inspection are believed to be either caused by fatigue or are progression-prone under transient loading as well as where components have an E or E’ category detail. For welded cover plates, a fatigue analysis need be performed only when cracks are identified during visual inspection.

4. **Analysis of Data.** For the analysis, fatigue characteristics of the metal should be established by the applicable design specification. See Chapter 407 for fatigue considerations for structural steel.

### 412-4.03 Substructure or Foundation

The substructure transfers loads to the foundation such as rock or earth. Substructures include unframed piers, bents or abutments, footings, piles, and drilled shafts. A substructure including piles and drilled shafts is referred to as a deep foundation. They also include fenders and dolphins used in navigable waterways. The following briefly describes those condition surveys and tests which may be performed on these elements to determine the appropriate level of rehabilitation.

### 412-4.03(01) Visual Inspection

Visual inspection and sounding are the primary methods for determining existing condition. Material tests should only be requested in special circumstances.

1. **Description.** A visual inspection of the substructure components should address the following:

   a. surface deterioration, cracking, and spalling of concrete;
   b. major loss in concrete components;
   c. distress in pedestals and bearing seats;
   d. evidence of corrosion of reinforcement;
   e. loss in exposed reinforcement;
   f. deterioration or loss in wood components;
   g. leakage through joints and cracks;
h. dysfunctional drainage facilities;
i. collision damage;
j. changes in geometry such as settlement, rotation of wingwalls, or tilt of retaining walls;
k. compliance with current INDOT seismic-design standards;
l. accumulation of debris;
m. erosion of protective covers;
n. changes in embankment and water channel;
o. evidence of significant scour; and
p. underwater inspection (as needed).

2. **Purpose.** The purpose of visual inspection is to record all deterioration and signs of potential distress for comparison with earlier records and for initiating rehabilitation procedures if warranted.

3. **Consideration.** It should be used on each bridge rehabilitation project.

4. **Analysis of Data.** Analysis should be made as required.

### 412-4.03(02) Other Test Methods

Other test methods described in Section 412-4.02 for a bridge deck may be used to determine the level and extent of deterioration of the substructure components.

### 412-4.03(03) Scour Analysis

A scour analysis should be included for each bridge that crosses over water. INDOT’s methodology for performing this analysis is discussed in Chapter 203-5.0. The specific performance criteria for this analysis are discussed in Chapter 203-3.02(06).

Scour analysis for a consultant-designed bridge-rehabilitation project should be completed by the consultant and submitted to the Bridges Division, Office of Hydraulics for review. Scour analysis for an INDOT-designed project is completed by the Office of Hydraulics. Scour countermeasures may be necessary based on the results of the scour analysis and require the approval from the Office of Hydraulics.
412-5.0  HISTORIC BRIDGES [REV. FEB. 2018]

A historic bridge is one which was built prior to 1966, and is in, or is eligible for inclusion in, the National Register of Historic Places. The Department has developed a listing of all publicly-owned historic bridges that are National Register-eligible or -listed.

Where a project involves a historic bridge, the bridge owner must prepare a Historic Bridge Alternatives Analysis for review and concurrence by the Environmental Services Division Office of Cultural Resources and Bridges Division Office of Bridge Design, after which it will be submitted to consulting parties for review and approval as part of the Section 106 consultation process.

412-5.01  Types of Historic Bridges

A historic bridge will be classified as either Select or Non-Select. The Department has determined each bridge’s classification in accordance with the Programmatic Agreement Among the Federal Highway Administration, the Indiana Department of Transportation, the Indiana State Historic Preservation Officer, and the Advisory Council on Historic Preservation Regarding the Management and Preservation of Indiana’s Historic Bridges (PA). The PA, a listing of Select and Non-Select bridges (inventory summary & results), and historic bridge marketing information are available on the Indiana Historic Bridges Inventory website at http://www.in.gov/indot/2530.htm.

412-5.01(01)  Select Bridge

A Select bridge has been identified as a historic bridge that is an excellent example of its structure type to be a suitable candidate for preservation. The intent of the PA is to preserve Select bridges in place for continued vehicular use. If rehabilitation alternatives are not in accordance with Section 412-5.02, and the owner is not granted a design exception or does not request one, the Select bridge must be bypassed or relocated for another use. See the PA for further guidance on bypassing or relocating the bridge.

412-5.01(02)  Non-Select Bridge

A Non-Select bridge has been identified as a historic bridge that is not an excellent example of its structure type, nor is a suitable candidate for preservation. If the rehabilitation alternatives are not
in accordance with Section 412-5.02, and the owner is not granted a design exception or does not request one, the Non-Select bridge must be marketed for re-use. In accordance with the PA, if no party steps forward to assume ownership of the bridge, the bridge may be demolished. See the PA for further guidance on marketing or demolishing the bridge.

### 412-5.02 Historic Bridge Alternatives Analysis [Rev. Feb. 2018]

Where a project involves a historic bridge, the bridge owner must complete a Historic Bridge Alternatives Analysis and receive concurrence from the Environmental Services Division Office of Cultural Resources and the Bridges Division Office of Bridge Design prior to proceeding to the Preliminary Plans milestone. The required contents of the analysis, including explanations and tips for discussion of alternatives, is available from the Department’s Historic Bridge Inventory Summary & Results webpage at [http://www.in.gov/indot/2531.htm](http://www.in.gov/indot/2531.htm), under Historic Bridge Project Development Process Documents.

The evaluation of alternatives must address the following alternatives for both Select and Non-Select Bridges. The list is a hierarchy, meaning that the analysis must prove an alternative is either not feasible or prudent prior to proceeding to the next alternative. Note that Select bridges must be preserved as part of the project.

1. No Build/Do Nothing
2. Rehabilitation for continued vehicular use (two-lane or one-lane option), meeting the Secretary of Interiors Standards for Rehabilitation.
3. Rehabilitation for continued vehicular use (two-lane or one-lane option), not meeting the Secretary of Interiors Standards for Rehabilitation.
4. Rehabilitation for continued vehicular use (one-way pair option), meeting the Secretary of Interiors Standards for Rehabilitation.
5. Rehabilitation for continued vehicular use (one-way pair option), not meeting the Secretary of Interiors Standards for Rehabilitation.
6. Bypass (non-vehicular use)/Build New Structure
7. Relocation of Historic Bridge and New Bridge Construction
8. Replacement – Demolition of Historic Bridge and New Bridge Construction

### 412-5.03 Design Criteria [Rev. Feb. 2018]
The following design criteria should be used to evaluate if a historic bridge can be rehabilitated for continued vehicular use. The criteria should not be viewed as an absolute, that is, a design exception may be appropriate and should be considered where appropriate.

A historic bridge on low-volume road should be evaluated using the design criteria described herein. A low-volume road is defined as having a design-year ADT of less than or equal to 400 vpd. A historic bridge on roadway with a design-year ADT greater than 400 vpd should be evaluated using the 3R design criteria for the applicable functional classification. See Chapter 55 for 3R criteria. The criteria for Existing Structure to Remain in Place should be used for bridge clear roadway width and structural capacity. These criteria should be used to determine if a historic bridge can be rehabilitated for continued vehicular use.

**Design Speed**

The existing posted speed should be used as the design speed. If the road is not posted, an engineering speed study should be performed and the road should be posted between logical termini.

**Approach Roadways Horizontal and Vertical Alignment**

These should be analyzed within 300 ft of either side of the bridge in accordance with AASHTO’s *Guidelines for Geometric Design of Very Low-Volume Local Roads (ADT < 400)* or the 3R design criteria, as applicable.

**Structural Capacity**

The structural capacity should be in accordance with Figure 412-2A, Historic Bridge Structural Capacity. The required capacity designations are those described in AASHTO *Standard Specifications for Highway Bridges*.

**Bridge Width**

The minimum bridge width should be in accordance with Figure 412-2B, Historic Bridge Minimum Clear Roadway Width.

**Bridge Railing**
Bridge railing may be left in place if there is no documented crash history or other evidence of crash history within the past 5 years such as damaged railing or concerns by local police agencies. If only slightly damaged, railing should be replaced in kind. If there is evidence of crash history within the past 5 years, the possible causes should be corrected, or new bridge railing provided as described in Section 404-4.0.

Approach Guardrail

Approach guardrail, if in place, should remain. If not in place, it may be omitted if there is no documented crash history or other evidence of crash history within the last 5 years, such as vehicles hitting the ends of the bridge railing or vehicles leaving the roadway. Crash history, such as that regarding damaged ends of bridge railings, may be an indicator of the need for approach guardrail.

In addition to those guardrails which the Department has standardized, there are others which have passed NCHRP 350 or MASH crash tests for specified test levels. If one of these devices is desired to be used for a specific project, the documentation to be provided is as follows:

1. an FHWA eligibility letter; and
2. complete details for the device as successfully crash tested.

Hydraulic Capacity

Improvements may consist of removal of sand bars or debris, channel clearing, or adding a supplemental structure. If a bridge is to remain in place and its approaches are realigned, the removal of existing roadway fill is an option toward improving the hydraulic capacity.

412-5.04 Economic and Other Criteria [Rev. Feb. 2018]

412-5.04(01) Select Bridge [Rev. Feb. 2018]

The appropriateness of rehabilitating a Select historic bridge should be determined based on the cultural significance of the bridge. The appropriateness of rehabilitating a Select bridge on a low volume road, as defined above, should further be assessed based on the cost effectiveness as follows:

1. if the initial rehabilitation cost is less than 80% of the replacement cost, rehabilitation is warranted; or
2. if the initial rehabilitation cost is equal to or greater than 80% of the replacement cost, the owner may request further consultation with FHWA to determine rehabilitation eligibility.
The above thresholds should not be viewed as absolute, i.e., if the initial rehabilitation cost is above 80% of the replacement cost, rehabilitation may still be considered a viable alternative. A rehabilitation project should result in a 20-year design life for the rehabilitated bridge.

A Select bridge may be rehabilitated and left in place, and a new bridge and new approaches may be built adjacent to it. This effectively creates one bridge and approaches for each direction of travel. For this situation, the new bridge must meet all design standards for a new bridge or obtain a design exception. Where appropriate, the new one-way bridge must be able to accommodate future widening to provide for two-way travel.

**412-5.04(02) Non-Select Bridge [Rev. Feb. 2018]**

The appropriateness of rehabilitating a Non-Select historic bridge should be determined based on the cultural significance of the bridge. A Non-Select bridge on a low-volume road, as defined above, should further be assessed based on the cost-effectiveness of the project and other criteria as follows.

If the initial rehabilitation cost is greater than or equal to 40% of the replacement cost, or the bridge meets two or more of the following criteria that cannot be economically corrected as part of a rehabilitation project, then replacement is warranted.

1. The bridge waterway opening is inadequate (i.e., National Bridge Inventory Item 71 is rated 2 or 3).
2. The bridge has a documented history of catching debris due to inadequate freeboard or due to piers in the stream.
3. The bridge requires special inspection procedures (i.e., the first character of National Bridge Inventory Item 92A or 92C is Y).
4. The bridge is classified as scour-critical (i.e., National Bridge Inventory Item 113 is rated 0, 1, 2, or 3).
5. The bridge has fatigue-prone welded components that are expected to reach the end of their service lives within the next 20 years. See Section 412-4.03(04) for information on conducting a fatigue analysis.
6. The bridge has a Sufficiency Rating of lower than 35.
The above cost thresholds should not be viewed as absolute. If the initial rehabilitation cost is above 40% of the replacement cost, rehabilitation may still be considered a viable alternative. A rehabilitation project should result in a 20-year design life for the rehabilitated bridge.
FIGURES

412-1A Condition-Driven Preventative Maintenance Eligibility Criteria
412-1B  Scheduled Preventative Maintenance Eligibility Criteria
412-2A Historic Bridge Structural Capacity
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CHAPTER 413

Wood/Other Structures
CHAPTER 65

Wood Superstructures

NOTE: This chapter is currently being re-written and its content will be included in Chapter 413 in the future.
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CHAPTER 65

WOOD SUPERSTRUCTURES

The AASHTO LRFD Bridge Design Specifications, Section 8, describes criteria for the design of a wood superstructure. Requirements for the design of wood decks and deck systems are described in the LRFD Specifications, Article 9.9. A useful reference book, Timber Bridges, Design, Construction, Inspection and Maintenance, is listed at the end of Sections 8 and 9. This Chapter describes general guidance in the design of a wood superstructure. The Chapter is structured as follows:

1. Section 65-1.0 provides general information for which there is not a direct reference in LRFD Specifications Sections 8 or 9.

2. Sections 65-2.0 and 65-3.0 provide information which augments and clarifies LRFD Specifications Sections 8 and 9. To assist in using these Sections, references to the Specifications are provided herein immediately following section titles, where applicable.

See Section 59-3.0 for additional information on wood superstructures.

65-1.0 GENERAL

65-1.01 Background

Most of the first highway bridges constructed were made from native untreated wood and were subject to insect and decay damage. Decay fungi have the basic requirements for growth and production of decay in wood, as follows:

1. air (they are aerobic organisms);
2. water;
3. a favorable temperature; and
4. a food source.

Wood can be protected by the elimination of just one of these favorable conditions. The food, of course, is the wood itself. This food may be made unavailable to the fungus by impregnating substances into the wood, making it unpalatable to attack organisms. Pressure treatment with approved wood preservatives is the only acceptable and effective method of wood preservation.
The fire potential of wood superstructures can be substantially reduced by better design choices, such as by using bridge members and components that have low surface-to-volume ratio. This can be done by using large solid sawn members and by laminating individual wood members into large components, such as beams and panels.

There are a number of different species of both softwoods and hardwoods that can be used for bridge construction. The choice of species is influenced by several considerations. For example, the availability of the cross-sectional sizes and lengths necessary for the actual structure is a consideration. The sizes used for highway bridges available in hardwoods are severely limited.

The availability of large, long members is considerably greater in western softwoods, especially Douglas fir-larch. A more important issue, from an engineering standpoint, is not necessarily availability, but rather certified grading and material certification. Most softwood production comes from mills that have a certified grading system in-place, whereas much of the hardwood production comes from mills that normally grade production based on appearance and not on strength.

Also, with respect to the choice of species, most of the commercially-available softwood stress grades can be readily treated with wood preservative to current specifications. The one softwood species that cannot be adequately treated is spruce. Hardwoods, in contrast, do not have a long historical record of treatment and performance for many of the species and wood preservatives.

Wood as a bridge construction material offers some advantages over steel and concrete. It responds well to impact loading and, unlike crystalline material, it fatigues at a very low rate, so low that fatigue considerations are not included in the design process. Treated wood is immune to the destructive actions of deicing chemicals. Treated wood is unaffected by freeze-thaw cycles.

Some disadvantages to the use of a wood superstructure are that it can burn and is generally not suitable for long spans. Many wood preservatives may be harmful to the environment; and the preservatives may not prevent long-term decay of the wood. A wood deck is generally not suitable for a high-traffic volume road due to spalling, cracking, or delamination of the asphalt wearing surface.

65-1.02 Usage

A wood superstructure should be limited to a low-volume, local road. Its use is subject to the approval of the Production Management Division’s Office of Structural Services manager. A
request to utilize a wood structure should address the items as follows:

1. cost;
2. AADT;
3. ADTT;
4. bridge-railing requirements;
5. local experience with wood structures; and
6. local maintenance capabilities.

A wood bridge, because of its rustic appearance, is very appropriate for use in a park, environmentally-sensitive location, or recreational-area project. A treated wood bridge may be used for a short-span locally-funded project, or for a trail or temporary structure. Structural components made of wood may be used on a rehabilitation project, wood-covered bridge, or a deck system for use on a steel truss. Figure 59-3B shows the approximate span-length range for the different types of treated wood structures.

There are practical considerations that are unique to wood structures that should be followed in the ultimate location and configuration of a wood structure. In addition to the transverse crown, it is advisable to have a profile grade of at least 0.3% to ensure complete deck drainage. The profile grade in the deck can be provided by means of a vertical curve. Longitudinal timber deck panels cannot be cambered to offset dead-load deflection. One of the recognized hazards of a timber bridge is fire. The potential for fire damage can be reduced by the use of large members and components with a low surface-to-volume ratio. A design features that will reduce fire potential is the proper placement of riprap. Properly placed riprap, in addition to providing protection from erosion, prevents the growth and accumulation of combustibles around the wingwalls and abutments.

Each wood superstructure should have a minimum of 6 in. freeboard above the design high water elevation based on the Q_{100} discharge.

**65-1.03 Wood Bridge Railings**

Bridge railings for a wood structure should be in accordance with the Test Level selection requirements provided in Section 61-6.01(01). There are no INDOT Standard Drawings for bridge railings that can be used on a wood structure. However, a useful reference regarding crash-tested wood bridge railings is a CD-ROM entitled *The National Wood in Transportation Program, Information on Modern Timber Bridges in the United States, 1988-2001.*
Where roadside barriers are installed at bridge railing ends, the barriers should blend in naturally with the surrounding environment, e.g., wood rails on wood posts.

65-2.0 BASIC CRITERIA

65-2.01 Materials

Reference: Article 8.4.1

The AASHTO *LRFD Specifications* provide design information on most of the commonly-used species and stress grades of wood-based products for a treated wood structure. A more complete listing may be found in the American Wood Council of the American Forest and Paper Association’s *LRFD Load and Resistance Factor Design Manual for Engineered Wood Construction*. That publication includes references to *AF&PA / ASCE 16-95, Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction*.

The extensive listing of a specific stress grade in either of the above-referenced sources does not imply that all of the listed stress grades are commercially available in the sizes and lengths required in bridge construction. The designer should check with regular suppliers of wood components for availability and cost in the final selection of size and stress grade of major bridge components.

65-2.02 Preservative Treatment

Reference: Article 8.4.3

All wood components used at a site conducive to decay and insect damage, such as a highway bridge, should be preservative-treated. Surfaces which are expected to be touched often by humans, e.g., pedestrian railings, should be treated with waterborne preservatives. All other components should be treated with oil-borne preservatives.

Details should be developed to show where all of the possible cutting and drilling will be done prior to pressure treatment. A spike or nail can provide access to the untreated interior portion of the wood component.
65-2.03 Metal Components

Reference: Article 8.4.2

The LRFD Specifications describe the design requirements for metal parts and attachments to a wood structure and their respective source specifications. Metal components employing forms of corrosion protection, such as weathering steel, epoxy coating, or cadmium plating, can be used where determined by the designer to be appropriate for the intended exposure condition.

Light-gage toothed metal connector plates are permitted by LRFD Specifications Article 8.4.2.2.8, but they should not be used in the superstructure as they tend to pull out under repetitive loading.

The design of structural-steel components should be in accordance with LRFD Specifications, Section 6.

65-3.0 DESIGN

65-3.01 General

Reference: Article 8.4.4

Considering strength versus weight, wood is a very efficient structural material. For example, an ultimate tensile strength of 13 ksi is obtained in testing straight-grained British Columbia fir. The same wood provides an ultimate compressive strength of approximately 11 ksi, indicating a slight compressive cellular instability of the material under compression parallel to grain. Because of the presence of knots, slope of grain, splits and checks, and other discontinuities, only a fraction of straight-grain specimen strength can be used in actual design.

Lumber grading is the process of separating lumber at the mill into categories that have the same strength-reducing characteristics or, groups that have the same strength properties. The size, extent, and combination of strength-reducing characteristics permitted within a specific stress grade are formalized and are then published in the form of Grading Rules. Grading Rules unique to each species or combination of species are approved by the Board of Review of the American Lumber Standards Committee and certified for conformance with U.S. Department of Commerce Voluntary Product Standard PS 20-94 (American Softwood Lumber Standard).

The primary purpose of lumber grading is to ensure that populations of wood products specified as being a specific stress grade will all exhibit material properties that are consistent with the
published values for that specific stress grade. The process of grading takes place at the point of production of the product. The only method presently used to improve the strength characteristics of wood is by means of laminating. In this process, the discontinuities become randomly distributed. If the number of laminations in a cross section is sufficiently large, the component strength can increase as it approaches the average strength of the species.

The designer must optimize the use of wood components in a wood superstructure. The cost of wood components is a function of many factors. Unlike steel and concrete, the unit cost of wood products is not closely related to volume or strength, but it includes factors related to the volume of various stress grades and sizes recovered in the milling process. Therefore, the judgment and knowledge of the designer, when addressing the classic question of a smaller-sized, higher-strength component or a larger cross section with lower strength, is central to economic wood bridge design.

The design of wood components includes a number of modification factors not normally associated with steel or concrete. Some of these factors address the variability inherent in wood. Others concern the response of the wood member to all of the environmental factors under which it is to perform. Most of these factors are applied to the base resistance side of the design equation.

One of the modification factors unique to wood-bridge design is the deck factor, $C_D$. This factor recognizes the load sharing between individual members under certain circumstances. It is applied only to solid sawn members, 2 in. to 4 in. thick, that are used in a structural system that creates load sharing between individual members. LRFD Specifications Article 8.4.4.4 recognizes only two applications for this factor, a stressed-wood deck, or a nail-laminated or spike-laminated wood deck.

Another modification factor is the moisture content factor, $C_M$, as used and specified in LRFD Specifications Article 8.4.4.3. For glued-laminated wood, it is considered to be wet if the in-service moisture content is greater than 16%. For such conditions, $C_M = 1.0$. If the in-service moisture content is less than 16%, as indicated in LRFD Specifications Table 8.4.4.3-1, the values for $C_M$ are greater than 1.0.

The dynamic load allowance values specified in LRFD Specifications Table 3.6.2.1-1 may be reduced 50% for a wood structure.
65-3.02 Solid Sawn Stringers and Glued-Laminated Beams

Reference: Section 8, Various Articles

Analysis of stringers for a stringer-type bridge is specified in Article 4.6.2.2. The distribution of wheel loads for moment in interior beams is shown in Table 4.6.2.2.2b-1. The distribution of wheel loads for moment for exterior beams is shown in Table 4.6.2.2.2d-1. Requirements for the analysis of a wood deck for a stringer-type bridge are included in Article 9.9.

Bracing requirements for wood stringers and glued-laminated beams are provided in Article 8.11.

65-3.03 Spike-Laminated Deck

Reference: Article 9.9.6

A spike- or dowel-laminated deck system consists of longitudinal panels that extend from support to support and produced in a manufacturing process where full-length individually treated planks are mechanically laminated into panels using metal spikes or dowels. The panels are generally 72 in. to 92 in. in width and range in thickness from 8 in. to 16 in. The effective span for design should be taken as the clear distance between supports, plus one-half of the bearing length at each support, but the effective design span should not exceed the clear span plus the deck thickness. For a multi-span bridge, all spans should be designed as simple spans.

The deck is made using a category of solid sawn members classified as dimension lumber. These are planks that are 2 in. to 4 in. in thickness and range in width from 8 in. to 16 in. The base-resistance value for this material is shown in LRFD Table 8.4.1.1.4-1. The correct size classification should be used for the material in question. The table includes base-resistance values for size classifications including beams and stringers (B&S), post and timbers (P&T), and dimension lumber. The individual members are cut to length and drilled for the connection hardware prior to treatment with an approved wood preservative.

Calculation of the equivalent strip width for analysis of a spike or dowel-laminated longitudinal deck, for spans greater than 15 ft, is described in Article 4.6.2.3. Determination of equivalent strip width for spans of 15 ft or less is shown in Article 4.6.2.1.3.

The connection between adjacent longitudinal panels should be accomplished using a longitudinal ship-lap joint. The configuration of this type of joint consists of attaching one-half of a laminate to the top half of the facia edge of one panel and the other half of the splice plank
to the lower half of the adjacent panel. The primary design consideration for this connection is
to provide sufficient shear resistance on the horizontal interface between the two portions of the
splice plank to recreate single-member bending resistance of the splice plank. The horizontal
shear resistance is developed by driving vertical spikes through the longitudinal ship-lap joint.
The spikes are spaced closer together near the supports.

This type of deck design system offers some advantages over other designs. All of the individual
laminates are treated prior to assembly into panels so that the resulting bridge component has a
large percentage of its volume impregnated with wood preservative over glued-laminated panels.
The basic material for the panels is rough sawn planks. By definition, rough sawn material has
some dimensional variability. The variability in member thickness is eliminated by surfaced on
one side (S1S).

The variability in the depth of the individual members is used to create a surface which is
conducive to the adhesion of the asphalt mixture. Once the panels are fabricated, the bottoms of
the panels are made smooth, thus forcing all of the variability to the top surface of the completed
panel. This provides many gripping surfaces for the asphalt mixture to adhere.

**65-3.04 Glued-Laminated Longitudinal Deck**

Reference: Article 9.9.4

A glued-laminated deck system consists of vertically-laminated panels which are prefabricated
by gluing adjacent laminations together with water-resistant adhesives. The effective span for
design should be taken as the clear distance between supports plus one-half of the bearing length
at each support. The effective design span should not exceed the clear span plus the deck
thickness. For a multi-span bridge, all spans should be designed as simple spans.

This type of longitudinal deck system employs longitudinal glued-laminated panels that extend
from support to support and are interconnected with transverse stiffener beams if the span
exceeds 8 ft. The panels are about 4 ft in width and vary in depth from 5 in to 14 in. The deck
panels are treated prior to shipping to the bridge site. *LRFD Specifications* Article 4.6.2.1.2
provides that, for a slab-type bridge spanning more than 15 ft and that the span is primarily in the
direction parallel to traffic, Article 4.6.2.3 should apply for determining equivalent strip widths.
For a slab-type bridge spanning 15 ft or less, Article 4.6.2.1.3 should apply. Article 9.9.4.3.1
applies to the design and location of transverse stiffener beams for this type of longitudinal deck
system.
65-3.05 Transversely Prestressed Deck

Reference: Article 9.9.5

This type of construction uses solid sawn members that are made to function as an orthotropic plate. One of the advantages of this construction technique is that it allows the use of non-continuous wood laminations, providing that the butt joints are staggered in accordance with LRFD Article 9.9.5.3. The controlling attribute in this system is deflection. Consequently, stress grades of material with relatively low strength design values can be used. The additional cost of the rather elaborate post-tensioning system is offset by the use of less costly wood components as described above. The primary load transfer mechanism is the friction between the individual laminates created by the normal force imparted by the post-tensioning system.

There are some concerns to be overcome in the design process. The most serious of these is the loss of post-tension force in the stressing bars due to the non-recoverable creep in the wood components. The level of long-term stress remaining in the bars is a function of many factors, including the number of times the bars are restressed, the time interval between restressing, the stressing sequence, relationship between the stiffness of the prestressing system and the transverse stiffness of the wood, and the type of wood preservative used. The difference between the moisture content of the wood at the time of fabrication, and the resulting equilibrium moisture content (EMC) of the structure, impacts the long-term stress in the bars in the prestressing system. LRFD Specifications Commentary C9.9.5.6.3 includes suggestions for restressing to offset long-term relaxation effects and creep losses.

Deck tie-down requirements for ensuring proper contact of the deck along each support are provided in LRFD Article 9.9.5.5.

65-3.06 Wearing Surface for Wood Deck

Reference: Article 9.9.8

An asphalt wearing surface should be used on a wood deck. The asphalt should have a minimum depth of 2 in. and should provide for the cross slope across the bridge. See LRFD Specifications Article 9.9.8 for methods of improving the adhesion of the asphalt wearing course and methods to provide a positive connection between the wood deck and the wearing course.

A spike-laminated or stress-laminated wood deck provides an irregular surface with many gripping surfaces for the asphalt mixture to adhere to. The geotextile or tack coat on a spike-laminated or stress-laminated deck should be used if recommended by the manufacturer.
Tension cracks will develop at each bridge support. Traffic tends to create a camelback-type hump at these cracks. This type of crack problem can be prevented by sawing a joint in the wearing surface over each pier and at the ends of the bridge. The joints are then filled and routinely maintained with a rubberized joint material.

A paving strip should be placed along the full length of the bridge at each curbline. The paving strip is to be of treated wood, of width equal to the depth of the asphalt wearing surface at the curbline. This strip has two functions. First, it ensures a uniform thickness at the curbline. Second, it provides a dam in front of the scupper opening that prevents the asphalt mat from yielding and deforming into the scupper opening during the compaction of the asphalt wearing surface on the bridge deck.