



# Bridge Procedures and Design Guide

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# **CHAPTER ONE**

## **GENERAL INFORMATION**

The New Mexico Department of Transportation uses the Current Edition of the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications and the current interims as the primary standards for the design of bridges in the State of New Mexico. However, under certain circumstances it may be preferable to base design on the Seventeenth Edition of the AASHTO Standard Specification for Highway Bridges instead. One such circumstance would be when rehabilitating or widening an existing bridge which was originally designed using the AASHTO Standard Specifications. The State Bridge Engineer's approval needs to be obtained prior to using the Standard Specification. It is assumed that users of this manual are familiar with the AASHTO LRFD Bridge Design Specifications and with current bridge design practice, methods, and procedures.

### **1.1 PURPOSE OF MANUAL**

The purpose of this manual is to supplement the AASHTO LRFD Bridge Design Specifications and all interims. It is NOT intended to replace the AASHTO LRFD Bridge Design Specifications or any other design guide or regulatory code. The intent is to provide guidance as well as an interpretation of AASHTO LRFD Bridge Design Specifications specific to New Mexico bridge design practice so that uniformity in bridge design procedures will be attained. This manual is intended to provide general guidance, and not solutions that are specific to any project.

New bridge design information is continually developing. This information is usually the result of specific project experiences, ongoing research, and information published by other agencies. In order to incorporate this new information into future designs, NMDOT

will revise this document as necessary.

An effort has been made to incorporate project experience, design preferences and selected research into this edition.

### **1.1.1 Design Innovation**

Although much of this Guide is devoted to the presentation of standardized design techniques and construction practices, this in no way implies that the NMDOT Bridge Bureau will not use innovative approaches. Quite the opposite is true. The presentation and implementation of ideas that will save construction time, save money or will improve quality are encouraged. Innovative ideas do however need to be presented to and approved by the State Bridge Engineer prior to implementing them.

## **1.2 DESIGN METHOD**

The information presented on the various bridge elements and the design example calculations are based on LRFD methodology.

The main objective of LRFD is to proportion structures so that for all limit states, the summation of factored loads (Also multiplied by load modifiers) are less than the factored resistance of the structure. Limit states identified by the AASHTO LRFD Specifications include service, strength, fatigue and fracture, and extreme events.

At this writing, routing of trucks through New Mexico is still dependent on the Load Factor Rating (LFR) methodology. For this reason, all new bridges are required to have an inventory rating of at least HS-25 and an operating rating of at least HS-42 in accordance with the NMDOT Bridge Rating Guide. Note that in some cases this will produce a more conservative design than LRFD.

Imperial units are to be used in all bridge design drawings and calculations.

### 1.2.1 Standards and Design References

The design of highway bridges is to be in accordance with AASHTO LRFD Bridge Design Specifications. Other necessary or useful documents are:

1. NMDOT Standard Specifications for Highway and Bridge Construction, Current Edition (herein after referred to as the NMDOT Standard Specifications),
2. Any applicable NMDOT Special Provisions and Supplemental Specifications,
3. AASHTO Roadside Design Guide,
4. AASHTO Policy on Geometric Design of Highways and Streets,
5. NMDOT Materials Geotechnical Design Guide,
6. NMDOT Drainage Design Guide,
7. NMDOT Bridge Rating Guide,
8. NMDOT Design Manual, and
9. Design Directives issued by the State Bridge Engineer.

### 1.2.2 Software

A variety of computer programs and software are available for the design of bridge elements. While the design of modern highway bridges necessitates that such programs be used, the NMDOT does not endorse or advocate the use of any particular set of programs. The bridge designer is responsible for checking the reliability and accuracy of the data generated by any program or software.

## 1.3 GENERAL REQUIREMENTS

In the design of a bridge, the design team must contend with a number of design features and parameters. Some features (e.g.

vertical clearance) have standard minimum requirements and are addressed in subsequent sections. Others, such as design speed, typical section, clear zone requirements, sidewalks, screening fence, etc. must be coordinated with the Project Development Engineer.

### 1.3.1 Geometric Requirements at Grade Separation Structures

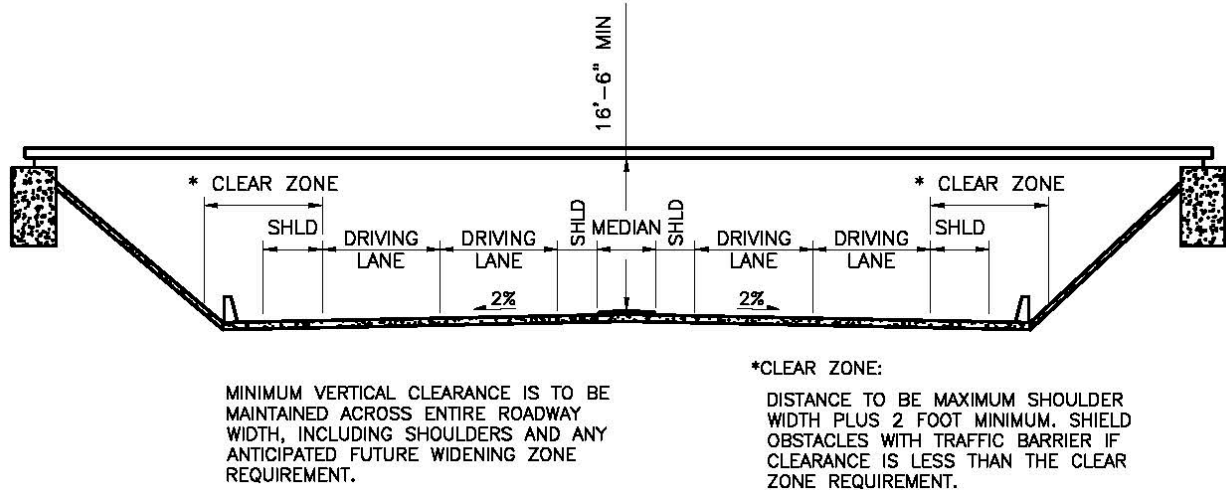
The bridge designer must satisfy both on bridge and under-bridge clearance requirements. The on-bridge horizontal clearance is discussed in 1.3.11 Bridge Widths. The following discussions on clearances pertain to under-bridge clearance requirements. Clearance requirements are defined in the AASHTO LRFD Bridge Design Specifications, the AASHTO Policy on Geometric Design of Highways and Streets, and the AASHTO Roadside Design Guide.

### 1.3.2 Horizontal Clearances

At highway overpass structures, it is generally desirable to provide an under-bridge horizontal clearance from the edge of a traffic lane to an obstruction (pier, abutment or embankment slope) equal to the clear zone distance specified in Chapter 3 of the AASHTO Roadside Design Guide. When providing this clear zone distance is impractical, clearance requirements should be established by the design team and obstructions shielded by a traffic barrier. The clearance should never be less than the full shoulder width plus 2 feet.

### 1.3.3 Vertical Clearances

Vertical clearances, specified in the AASHTO Policy on Geometric Design of Highways and Streets, are based on the Functional Classification of the Highway. For interstate highways, AASHTO specifies a minimum structure clearance of 16 feet plus a recommended 6 inches for future overlay, for a total clearance



**Figure 1.3A**  
**Typical Highway Clearances**

of 16'-6". In New Mexico this minimum clearance is required for all new grade separation structures where significant truck traffic is anticipated, and a clearance of 17'-0" should be provided wherever feasible.

In limited cases, the 16'-6" minimum clearance requirement may be relaxed. If significant truck traffic is not anticipated, the vertical clearance of structures crossing secondary roadways (local roads and streets) may be reduced to 15'-3" with approval from the State Bridge Engineer and the District office. This lower clearance allowance is based on the 14 foot legal limit for truck heights, 9 inches for some additional clearance, and 6 inches for future overlay. Typical highway clearances are shown in Figure 1.3A.

For pedestrian bridges, the established minimum clearance for vehicular bridges must be increased by 1'-0" to a minimum of 17'-6". A clearance of 20'-0" must be provided if the superstructure of the pedestrian bridge is fracture critical.

An additional circumstance where a deviation from the standard 16'-6" clearance might be warranted is the case of a grade separation structure without an interchange where no suitable detour route is available for over height loads. Many roundabouts do not provide an adequate turning radius for larger

or longer trucks, and may not be considered adequate for detours. Clearance requirements in such circumstances need to be determined site specifically, and should be based on the height of the permit vehicles that regularly use the route. Some important interchanges may require greater clearances. All deviations from the standard clearance should be coordinated with the State Bridge Engineer. See Table 1.1 for further guidance.

### 1.3.4 Posting Requirements

A vertical clearance sign denoting the minimum vertical clearance shall be posted for overhead structures with a minimum vertical clearance of less than 16 feet. Clearance signs are not required to be posted on overpass structures that have vertical clearances of 16 feet or greater.

**Table 1.1 Vertical Clearance Table**

Project Type	Vertical Clearance	Documentation Requirement (See Notes) <sup>[4]</sup>
<b>Interstate and Other Freeways</b>		
New Bridge	> 16.5 ft	[1]
Widening Over or Under Existing Bridge	> 16 ft	[1]
	≤ 16 ft	[2]
Resurfacing Under Existing Bridge	> 16 ft	[1]
	≤ 16 ft	[2]
Other With No Change to Vertical Clearance	> 14.5 ft	[2]
	≤ 14.5 ft	[2]
<b>Nonfreeway Routes</b>		
New Bridge	> 16.5 ft	[1]
Widening Over or Under Existing Bridge	> 16.0 ft	[1]
	≤ 16.0 ft	[2]
Resurfacing Under Existing Bridge	> 16.0 ft	[1]
	≤ 16.0 ft	[2]
Other With No Change to Vertical Clearance	> 14.5 ft	[2]
	≤ 14.5 ft	[2]
<b>Bridge Over Railroad Track</b>		
New Bridge	> 23.5 ft	[1]
	≤ 23.5 ft	[3]
Existing Bridge	> 22.5 ft	[3]
	≤ 22.5 ft	[3]
<b>Pedestrian Bridge Over Roadway</b>		
New Bridge	≥ 17.5 ft	[1]
Existing Bridge	< 17.5	[2]
<b>Notes:</b> <sup>[1]</sup> Approval not required <sup>[2]</sup> Requires both the District and Bridge Bureau approval <sup>[3]</sup> Requires written agreement between railroad company and the NMDOT <sup>[4]</sup> In locations where no suitable detour route is available for over height load, additional vertical clearance may be required. Contact the Bridge Bureau for required clearance. <b>Note:</b> For any existing bridge that has been hit by any over height vehicle, contact the Bridge Bureau for direction.		

### 1.3.5 Railroad Requirements and Clearances

The NMDOT Rail Bureau must be advised and involved at the earliest opportunity in the project planning phase. This is necessary so that agreements, insurance requirements, flagging protection, and other relationship documents can be prepared.

Early involvement is especially important for highway projects that require soil exploration activities within railroad property. For entry to be granted, a preliminary layout showing proposed borehole locations must be submitted to the railroad company. Subsequently, the appropriate entry documents must be secured by the NMDOT. Proper insurance protection must be provided and approved. The railroad company may also require flagging protection under some soil exploration conditions.

In the early development of plans, a preliminary layout showing clearances should be submitted to the railroad company for their concurrence. The following sections give clearance recommendations that will satisfy the requirements of most railroad companies, but note that each railroad sets its own clearance requirements. All layouts should be submitted through the NMDOT Rail Bureau.

While preliminary layouts and entry documents are developed at the beginning of the project, the primary highway/railroad agreements are initiated after the railroad is authorized by the Department to proceed with its engineering efforts. Normally, this occurs after any changes from the Department's Pre-Final Design Inspection have been incorporated into the construction plans and the railroad can be provided a set of near-final plans. The railroad's engineering effort typically consists of the relocation of utilities and other activities that will accommodate the

project. The railroad's engineering and the preparation of the agreements takes a minimum of 6 months after engineering authorization has been provided. Any changes made to the construction plans that affect railroad property, after engineering authorization has been provided, will likely require an additional 6 month review period.

In the design of railroad overpasses, consider each crossing unique and make every effort to keep the cost of the bridge project to a minimum.

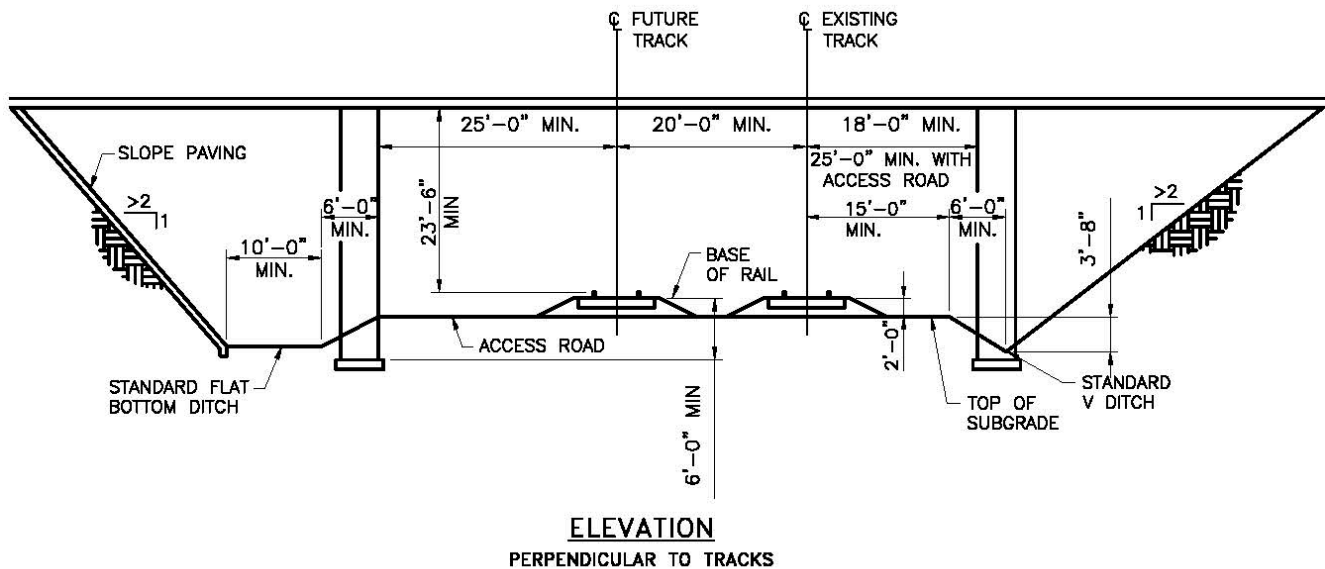
### 1.3.6 Railroad Horizontal Clearances

Typical railroad under-bridge horizontal clearances are shown graphically in Figure 1.3B. Actual clearances should be established on a project by project basis. If conditions at a particular site require clearances less than those shown, the railroad company should be contacted for absolute minimum requirements. Such an issue would generally be handled thru the NMDOT Rail Bureau.

### 1.3.7 Railroad Vertical Clearances

Except for those railroad routes that railway companies have plans to electrify, vertical clearances for new railroad overpass structures should be at a minimum of 23'-6". An additional allowance may be required, but should be determined on a project by project basis. When widening existing structures, most railroad companies require that the vertical clearance that existed prior to the widening not be reduced.

For railroad routes to be electrified, the Code of Federal-Regulations (23CFR) provides for 26'-0" of vertical clearance from top of rail to bottom of an overhead structure for a 50 KV line.



**Figure 1.3B**  
**Typical Railroad Clearances**

The 26' of vertical clearance will accommodate 12" of future ballasting. The NMDOT Rail Bureau should be contacted to determine whether or not the requirement applies to any particular site.

### 1.3.8 Crash Wall Requirements

Piers supporting bridges over railways should be protected from damage by the redirection and deflection of railroad equipment. The American Railway Engineering and Maintenance-of-Way Association (AREMA) recommends using either crash walls or piers of heavy construction for piers having a clear distance of less than 25 feet from the center-line of a track. The AREMA states that piers are of heavy construction if they have a cross-sectional area equal to or greater than that required for the crash wall and the larger of its dimensions is parallel to the track. Because some railroad companies have a specific interpretation of 'heavy construction,' the definition should be checked with the particular railroad company.

### 1.3.9 Waterway and Free Board Requirements

For bridges over waterways the NMDOT Drainage Bureau will prepare and issue a drainage report. This report will contain information on the 50-year, 100-year and 500-year flood events. Refer to the latest edition of the NMDOT Drainage Design Guide for further information.

Waterway data shown on the plans should be shown for the 50-year, 100-year and 500-year flood events. The data must be shown either on the bridge layout sheet or the general notes sheet, and include cross-references. Required waterway data is listed in Section 2.3.2.

Many rivers are controlled by the U.S. Army Corps of Engineers' Section 404 Permit Program. Contractors are advised of construction restraints imposed by the 404 permit in notes furnished by the Drainage Engineer.



These notes should be shown with the waterway data.

### **1.3.10 Free Board**

Free Board shall be 2' above the design flood high water elevation, whether this is for the 50 or 100 year event.

If the design flood is the 50 year event, Free Board for the 100-year event shall be equal or greater than zero (Water should not extend above the bottom of the girders).

There are no Free Board requirements for the 500-year event.

Regardless of the free board requirement, the minimum clearance under the bridge and the channel is to be four feet. This is to help in maintenance such as cleaning of the channel. Clearances less than four feet need to be approved by the District Office and State Bridge Engineer.

### **1.3.11 Bridge Widths**

The major dimensional features of a bridge are the barrier system, the roadway horizontal clearances, and the median treatment, where applicable.

The typical barrier system consists of concrete barrier railing or metal railing. The bridge railing should be able to safely redirect a vehicle under impact conditions. At certain locations, a pedestrian walkway or bicycle path can be incorporated into the barrier system. See Section 1.3.15 Bridge Railing Requirements, and Section 1.3.16 Pedestrian Screening for further discussion.

In determining roadway horizontal clearances, the full width of the approach roadway should be maintained across the entire structure. The bridge is a small part of the continuous roadway and should be designed with the same

cross section dimensions as the roadway. The minimum clear roadway widths for new and reconstructed bridges in the AASHTO Policy on Geometric Design of Highways and Streets shall be used. These widths are based on the functional classification. The minimum clear width for all new bridges on streets with curbed approaches should be the same as the curb-to-curb width of the approaches. For roadways with shoulders and no curbs, the clear roadway width should be the same as the approach roadway width and the faces of the bridge railing elements should be set at the outside edges of the shoulders. However, no bridge on a rural highway is to be designed with a shoulder less than 4 feet wide. Sidewalks on approaches should be carried across all new structures.

Median treatment is a design team issue. Median treatment depends on whether the approach roadway is divided or undivided. On a divided highway with a wide median, the overpass will likely be built as two separate parallel structures. Bridge cost and traffic safety should be key considerations in making this decision.

### **1.3.12 Protection of Embankments**

Embankment slopes should be protected from roadway runoff erosion. Rundowns or other catchment devices should be provided where necessary to convey the runoff to the bottom of embankment slopes. Standard details of rundowns for bridges are shown on NMDOT's website ([dot.state.nm.us](http://dot.state.nm.us)) under the Standard Drawing Section 515.

### **1.3.13 Waterways**

Embankments in stream beds should be protected from stream flow erosion by wire enclosed riprap or an approved equal. Requirements for riprap are generally contained in the project drainage report. Riprap should extend a minimum of 2 feet above design

flood elevation and should be buried in the stream bed to the contraction scour depth if possible.

#### 1.3.14 Grade Separation Structures

Embankment slopes steeper than 3:1 should be protected from erosion. The design team will decide on the best erosion measures, but some examples include: slope paving, rock plating and interlocking paving blocks. Standard slope paving details are shown on NMDOT Standard Drawing 511-05-1/1.

#### 1.3.15 Bridge Railing Requirements

AASHTO Bridge Specifications require that railing be provided along the edges of all bridge structures for the protection of vehicles and pedestrians. Most bridge railings differ from roadside barriers in that they are an integral part of the structure and are designed to have virtually no deflection when struck by an errant vehicle. An in-depth discussion of bridge railing may be found in the AASHTO Roadside Design Guide. See NMDOT's website for Bridge Railing Standard Drawings.

There are four types of railing the NMDOT uses and a description of each is given below. Where a pedestrian walkway or bicycle path is provided on a bridge, a barrier-type bridge railing of adequate height should be installed between the pedestrian walkway and the roadway. A pedestrian railing or screen should be provided on the outer edge of the walkway. In low speed traffic, a raised curb with a combination railing (a barrier rail having pedestrian screening) may be used.

1. **Crash Tested Traffic Railing** (or vehicular railing) is used as a bridge or structure-mounted railing, instead of a metal barrier or median barrier. Traffic railing is used only to protect traffic. The standard practice of the NMDOT is to use concrete bridge barrier railing on all new structures. The general rule is to

use the 42" barrier on Interstate highways, US highways, and other routes with a high percentage of heavy truck traffic and the 32" barrier elsewhere. However, the height of railing to be used needs to be coordinated with the Design Team. NMDOT Standard Drawings Section 514 contains standardized details for 32" and 42" concrete bridge barrier railing.

There are, however, situations where metal bridge railing is permitted. Again, the general rule is to use the 42" railing on Interstate highways, US highways, and other routes with a high percentage of heavy truck traffic and the 32" railing elsewhere. Crash tested side-mount metal bridge railing is also available. Side-mounted bridge railing is advantageous for replacement of decks that have curbs. The curbs can be eliminated and the bridge widened in some cases. NMDOT Standard Drawings Section 543 contains standardized details for 32" and 42" metal bridge railing.

**2. Low Speed Traffic Railing** Upon NMDOT approval, in an urban setting where the speed limit is below 45 mph, Type A or Type D metal railing shown in the NMDOT Standard Drawings Section 543 or other state's railing that has been crashed tested may be used for vehicular traffic.

3. **Pedestrian Railing** is used on pedestrian bridges and the outer edge of a sidewalk on bridges. NMDOT Standard Drawings Section 543 contains standardized details for pedestrian railing.
4. **Combination Railing** is designed to protect both vehicles and pedestrians or bicycles. A combination railing in conjunction with a raised curb and sidewalk is used only on low speed highways. NMDOT Standard Drawings Section



607 contains standardized details for combination railing.

### **1.3.16 Pedestrian Screening**

Pedestrian screening should be provided on new overpass structures that have sidewalks or are expected to carry significant amounts of pedestrian traffic. Pedestrian screening protects under-bridge traffic from trash or debris that might be thrown onto the roadway and keeps pedestrians and bicyclists from exiting the bridge prematurely. Pedestrian screening is also required for all new bridges over railroads. The commonly used types of pedestrian screening are those in NMDOT Standard Drawings Section 607.

### **1.3.17 Use of Approach Slabs and Sleepers**

Approach slabs should be used on most bridge structures, particularly on those over 100 feet in length. Because an expansion joint may be placed at the approach slab and sleeper interface rather than at the abutment, the Department prefers an approach slab with sleeper on all new bridges. This is especially true for bridges with integral and semi-integral abutments, and bridges with concrete approach pavements.

A minimum approach slab length of 14 ft should be used on all new bridges. For bridges built on a new alignment or in a high fill area, approach slab length should be 20 ft.

Approach slabs are to be supported by competent backfill consisting of either select A-1-a material conforming to base course specifications or flowable fill. The NMDOT Geotechnical Section should be consulted for approach slab backfill requirements.

### **1.3.18 Transition Slabs**

On skewed bridges, transition slabs consisting of concrete and mild reinforcing should be included on the ends of the approach slabs.

Use of the transition slabs are of great benefit in providing a smooth transition between the bridge and bituminous pavement at the approaches. An example design drawing of a transition slab is available from the NMDOT Bridge Bureau.

### **1.3.19 Deck Drainage**

Water must be drained from the bridge. Ponding of water on a bridge deck can be hazardous, particularly in cold weather when ice may form. The NMDOT Drainage Design Guide contains precise information on situations that require consideration of deck drainage. The drainage of the deck needs to be coordinated with the NMDOT Drainage Bureau. Drains that penetrate the bridge deck should be long enough to extend beyond and below the girders at least 9 inches. However, allowing runoff to drain into a waterway under a bridge may not be permitted due to environmental concerns. These issues should be discussed with the NMDOT Drainage Bureau and the NMDOT Environmental Bureau.

### **1.3.20 Corrosion Protection**

Reinforcing bars in bridge superstructures should be protected from the corrosive action of deicing compounds. For new structures, all reinforcing bars in the superstructure, with the exception of those in prestressed girders, and all reinforcing bars in the approach slabs and transition slabs, where applicable, are to be protected by epoxy coating or galvanizing, or otherwise made of corrosion resistant steel. For requirements for protecting reinforcing bars in prestressed concrete girders, contact the NMDOT Bridge Bureau.

For bridge concrete repairs, it is recommended that sacrificial anodes be included at all concrete patch locations and concrete replacement areas where reinforcing steel has been exposed. This recommendation does not

apply when non-cementitious patch materials are used.

### **1.3.21 Design Interior vs. Exterior Girder**

Section 4.6.2.2.2 of the LRFD Specifications presents live load distribution factors for both interior and exterior girders. This implies that a design for both interior and exterior girders needs to be completed. To avoid this time consuming process, or, alternately, the unnecessary expense of designing all girders for the larger distribution factor of an exterior girder, it is desirable to set the geometry of the bridge cross section so that the distribution factor for an exterior girder is preferably less than that for an interior girder. Design all girders for the most critical distribution factor.

The cross-sectional geometry for regular girder bridges with a concrete deck shall be set so that the distribution factor for the exterior girder does not control the design. In many situations, keeping the cantilever length equal to or less than 3'-6" will keep the distribution factors within these limits. Measure from the centerline of the exterior girder.

### **1.3.22 Aesthetics**

Aesthetics should be a consideration of any bridge design, particularly for overpass structures and structures exposed to public view. While there are many opinions of what a "beautiful bridge" looks like, there are a few general guidelines to follow. The use of clean lines can improve bridge appearance. Removing end blocks from prestressed concrete girders and removing stiffeners from the outside of steel girders can achieve this effect. Bents should have smooth round columns with neat cap lines. Other minor enhancements, such as shadow lines on parapet railing, may have pleasing effects.

Color on bridges should generally be limited to those elements that will be exposed to

public view. On most stream crossings this would limit coloring to the barrier railing only. Conversely on overpass structures all elements, including abutments, piers, wingwalls, exterior girders, edge of deck and barrier railing, would be colored.

While aesthetics is an important element it should not be the factor driving design decisions. Bridge type and form selections should be made on the basis of cost and sound engineering principles. Aesthetic treatments are then incorporated into the type of bridge selected using these principles.

### **1.3.23 Approved Products Lists**

Many products used in the construction of bridges are proprietary. Sometimes, there are several different proprietary products available that do essentially the same thing. Describing these products in generic terms for inclusion in the specifications is difficult and in some cases almost impossible. To be able to utilize these products while still maintaining control of quality, the Department makes use of the "Approved Products List" process.

Through this process, manufacturers submit evidence of adequate performance on their products. This evidence is reviewed by the Department and, if found acceptable, the product is placed on the appropriate list of qualified products. Products for which such lists are maintained include paint, patching materials, water repellent and others. Presented below is a listing of products for which approved products lists are maintained. Also included in the list is the section of the Standard Construction Specification Section which governs the use of the product and the Departmental organization that maintains the list.

<u>Product</u>	<u>Specifica- tion Section</u>	<u>Maintaining Organization</u>
MSE walls	506	Geotechnical Section
Water proofing	511	Bridge Con- struction Unit (BCU)
Non-shrink Mortar	521	BCU/Materials Lab
Chemical Adhesive Anchors	522	BCU/Materials Lab
Penetrating Water Repellent	532	BCU/Materials Lab
Concrete Repair material	533	BCU/Materials Lab
Crack Sealants	535	BCU/Materials Lab
Epoxy overlay	536	BCU/Materials Lab
Paint for new Structural Steel	544	BCU
Paint for misc. Steel Structures	545	BCU
Paint for Recoating steel Structures	546	BCU

Most of the products are also contained in a master listing of approved products. The Products Evaluation Committee maintains this list. Contact the NMDOT Maintenance Bureau for information.

#### **1.4 DESIGN TEMPLATES AND EXAMPLES**

Many structural elements have become essentially standardized over the years through frequent use but vary from bridge to bridge to

meet the requirements of the particular bridge being designed. For these types of elements the NMDOT Bridge Bureau has developed design template drawings. These design template drawings are completed plan sheets on which the information that varies from bridge to bridge is left blank. For a particular job the sheet is utilized by simply filling in the information that is missing and incorporating the drawing into the plan set. Structural elements for which plan templates are available are:

- Prestressed girders – All Standard AASHTO Shapes,
- Prestressed girders – All Bulb Tee Shapes, and
- Steel diaphragms for prestressed girders.

Current examples of other common bridge design items may be obtained from the NMDOT Bridge Bureau, including:

- General Notes,
- Estimated Quantities,
- Reinforcing Schedule Sheet,
- Approach slabs,
- Wingwalls,
- Transition slabs,
- Approach and abutment backfill compaction details,
- Deck Drains
- Elastomeric bearing pads, and
- Bat Boxes.

There are many other drawings, details and sample plan sets available from the Bridge Bureau.

The use of these design template drawings and details have many advantages including time savings, standardization and a reduction in the number of plan errors and plan omissions. Their use is highly encouraged.

### **1.5 DEFLECTION CRITERIA AND SPAN TO DEPTH RATIOS**

In New Mexico the deflection criteria contained in LRFD article 2.5.2.6.2 shall be applied. The criteria on span to depth ratios contained in article 2.5.2.6.3, however, need not be.

## **CHAPTER TWO**

### **DESIGN PROCESS**

This section discusses the design process from the perspective of the NMDOT's overall project development process and from the perspective of the Bridge Design Engineer.

#### **2.1 NMDOT PROJECT DEVELOPMENT PROCESS**

The NMDOT has formal reviews or inspections at different stages of project development. The following sections discuss the tasks to be completed and the reports to be submitted at each review.

A Preliminary Field Review is held at the beginning of each project. This is an informal meeting with all design team members. All feasible bridge alternatives and types will be discussed, the possible bridge types and layouts will be discussed, and preliminary cost estimates will be considered. The Preliminary Field Review will reduce the number of bridge alternatives to consider in preparation of the Preliminary Bridge Type Selection Report.

##### **2.1.1 Preliminary Design Inspection**

The first review milestone is the Preliminary Design Inspection (PDI), which is considered the minimum 30% complete point. Prior to this inspection, most of the major decisions and studies needed to begin bridge design will have been completed. These include:

- Preparation of the Preliminary Bridge Type Selection Report,
- The location survey,
- P&P sheets,
- A Final Drainage Report, and
- Preliminary traffic geometrics.

Using this information, a preliminary bridge layout and a cost estimate are prepared for

inclusion in the Preliminary Design Inspection package.

The preliminary layout should include plan and elevation views of the bridge and the proposed bridge section. The existing bridge and other site conditions in the vicinity should also be indicated.

Due to the incomplete nature of the design at this point, a detailed quantity-based estimate is usually not warranted. For most structures, an estimate based on the deck area of the bridge will suffice. However, care should be taken in selecting the unit price to develop the estimate. If possible, the unit price should be based on the actual bid price for a structure similar to and in the same general vicinity as the one under consideration.

The preliminary layout and cost estimate should be submitted to the State Bridge Engineer for review and to the Project Development Engineer for inclusion in the PDI package. The preliminary layout should also be submitted to the NMDOT Geotechnical Section so they can schedule and prepare for field exploration.

##### **2.1.2 Pre-Final Design Inspection**

The second review milestone is the Pre-Final Design Inspection (PFDI), which is considered the minimum 60% completed point. During the time interval between this inspection and the previous one, the following items of work are either completed or are initiated:

- The preliminary layout is finalized by incorporating comments received at the Preliminary Design Inspection and from the State Bridge Engineer, and by adding additional information,

- Work on the design and detailing of the bridge superstructure is initiated, and
- The foundation exploration and preliminary Foundation Report are completed.

At PFDI, the minimum requirements for bridge plans are completion of the final bridge layout, transverse section sheets and an updated cost estimate.

The final layout should include the plan and profile views of the bridge, the section view, grading plan for the bridge (as part of the plan view), bank protection requirements, geometry of the horizontal and vertical alignments, waterway data, provisions for drainage of the bridge and roadway surfaces, and any applicable construction phasing requirements.

The cost estimate for the bridge should be updated as necessary to reflect any major changes in concept or size that have been made between Pre-final and the Final Design Inspections.

Final bridge layout and transverse section sheets and the updated estimate should be submitted to the State Bridge Engineer, the Project Development Engineer, and the foundation engineer.

### 2.1.3 Final Design Inspection

The third review milestone is the Final Design Inspection (FDI), which is considered the 90% completed point. In the time interval between this inspection and the Pre-final Inspection the following work is done:

- The design and detailing of the bridge superstructure is completed,
- The Final Foundation Report is completed and made available to the bridge designer,

- The substructure is designed and detailed,
- Quantity take-offs are made,
- The general notes sheet and estimated quantity tabulations are made, and
- LFD and LRFD bridge superstructure ratings have been performed and checked.

Bridge plans submitted for the Final design Inspection should be essentially complete. At this stage, design and detailing work for the bridge deck, superstructure, and substructure should be complete. All other bridge and structural related items should be laid out and detailed and the general notes sheet and estimated quantity tabulations prepared. Also, the plans need to be revised to incorporate comments from PFDI.

Prior to the FDI, the bridge plans should be submitted to the State Bridge Engineer, the NMDOT Geotechnical Section, and to the Project Development Engineer.

### 2.1.4 PS&E Review

The PS&E (Plans, Specifications and Estimate) review is the FINAL meeting and considered the 100% completed point. At this stage the plans should be complete and ready to let. All design and detail checks should be finished, all project specific specifications should be prepared, and plans should be revised to incorporate comments received at previous design inspections, from the foundation engineer and from the State Bridge Engineer.

Plans need to be submitted to the Project Development Engineer at least two weeks prior to the scheduled PS&E review date. This advance submittal allows time for printing, distribution of the plans, and for inspection participants to review plans prior to the reviews.



### 2.1.5 Finalizing Plans for Letting

After PS&E review, plans should be revised to update quantities and to incorporate comments received at the review. After review comments are incorporated, the plans need to be accepted and signed by the State Bridge Engineer or Bridge Bureau representative. The final plans are then ready to be submitted to the PS&E Bureau for inclusion into the bidding documents.

## 2.2 BRIDGE DESIGN PROCESS

The process of bridge design can usually be divided into the following phases:

1. Bridge Type Selection and Layout Preparation,
2. Foundation Investigation and Analysis (performed by the NMDOT Geotechnical Section),
3. Detailed Design and Plan Development, and
4. Checking, Reviews, and Approvals.

The following sections briefly discuss each of these phases.

### 2.2.1 Bridge Type Selection and Layout Preparation

Prior to beginning the Bridge Type Selection and Layout Preparation phase, the bridge designer will require the following information:

- Existing bridge as-builts and Inspection Report,
- Project Scoping Report,
- Project Survey Information,
- Project Roadway Typical Section Sheet,
- Project Roadway P&P Sheets,
- Preliminary Drainage Report (for stream crossing structures),
- Clearance requirements for crossings other than stream crossings,
- Preliminary Interchange Layout Sheets (for grade separation structures), and

- Phase I A/B Evaluation of Alternatives Report (if applicable).\*\*

\*\* If the project does not include a Phase I A/B report, briefly discuss in the Bridge Type Selection Report the scope of work, span length, clearances, and alignment. Include any constraints on the design of the bridge such as cultural resources, drainage requirements, etc.

A Bridge Type Selection report will be prepared for all bridge projects. The report should identify and discuss bridge types that fit the constraints identified in the Phase I A/B Evaluation of Alternatives. For bridges costing in excess of \$500,000, the report should include a formal comparison of at least the two most promising types.

When selecting the bridge type, give preliminary consideration to all feasible types using the format discussed in further detail later in this section.

Functional requirements for the bridge should be obtained from the sources listed above, discussions with the project development engineer and, when available, design inspection reports.

Each bridge project should be considered as a potential candidate for Accelerated Bridge Construction (ABC). ABC is bridge construction that uses innovative planning, design, materials, and construction methods in a safe and cost-effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges. Examples of ABC may include the use of prefabricated bridge elements and systems (PBES), geosynthetic reinforced soil-integrated bridge system (GRS-IBS), and slide-in bridge construction.

The Bridge Engineer will begin the type selection process by filling out an ABC Decision Matrix, such as the matrix shown in WisDOT Bridge Manual Figure 7.2-1, provided in Appendix C. While not every bridge project may be a candidate for ABC, the

Bridge Engineer will be responsible for weighing advantages/disadvantages and costs when completing the ABC decision matrix.

The Bridge Engineer will then fill out the two tables shown in Figures 2.1 and 2.2. The first table will help the Bridge Engineer perform a preliminary analysis of feasible bridge types—to list project needs and the demands these needs place on the bridge for design, construction, and service. When listed in the table, these needs can be used to narrow the number of available bridge types to those most feasible for the project. Only the most feasible bridge types (two to four) should be advanced for formal comparison in the Bridge Type Selection report. Next, the Bridge Engineer will assign weighting factors to all evaluation criteria and enter them into the Decision Matrix shown in the second table. Upon completion, the Bridge Engineer will forward the ABC Decision Matrix and these two tables to the PDE, who will circulate them for comment and agreement among the project team and stakeholders.

The following criteria must be discussed for each bridge type included in the formal comparison to support selection of the preferred bridge type. These criteria will also be included in the weighted decision matrix published in the body of the report. Other criteria identified in the preliminary analysis of feasible bridge types may be added.

- Functional Requirements  
*Vertical and horizontal clearance, waterway conveyance, freeboard requirements, scour potential (number and location of piers), impacts to environmentally sensitive areas, etc.*
- Economics  
*Traffic control cost, bridge type cost, user delay cost, etc.*
- Future Maintenance  
*Expected life and repair activities for bridge components, coatings, etc.*
- Construction Feasibility  
*Detour length, impact to roadway users, room to construct, distance to fabricators, etc.*
- Aesthetics

- Accelerated Bridge Construction (if warranted by ABC Decision Matrix)

Once the preferred bridge type is selected, the Bridge Engineer should prepare an executive summary and include it at the beginning of the report. This executive summary should include a brief statement of the preferred bridge type, what types were considered in the formal comparison, and how the final selection was made.

Preliminary bridge layouts—including structure location, profile, and transverse section sheets—should be prepared for the formally compared bridge types and appended to the report. (Before preparing final layout sheets and proceeding with design and plan development, designers should ensure that final information has been received for the typical sections, the alignment and/or interchange geometry. The drainage report should also be finalized.) Checklists of contents to be shown on these sheets are contained in Section 2.3.2 and Section 2.3.4.

Cost estimates should be appended to the report for each of the formally compared bridge types.

Other items to be appended include the ABC Decision Matrix and preliminary analysis of feasible bridge types. The most recent inspection report from the existing bridge, if applicable, may also be appended to the report.

After the Bridge Type Selection and Layout Preparation phase is completed, the layout sheets are submitted to the State Bridge Engineer for approval of the selected bridge type and to the bridge foundation engineer to initiate foundation exploration work. The preliminary bridge layout for each bridge shall be approved for serviceability and cost effectiveness by the State Bridge Engineer or his representative before final bridge design begins.



		Bridge Type								
		CIP Slab	P/S AASHTO Girder				Steel Girder	CIP Multi-cell Box	Precast Segmental	Cable Stay
			Type 54	BT-54	Type 63	BT-63				
Project Needs/Constraints	6 Spans- 110' per span	NG	NG	NG				NG		
	85" max. superstructure depth								NG	
	Falsework not permitted due to site conditions	NG						NG		
	ABC is justified	NG						NG		NG
	Project Budget: \$8 M								NG	NG

NG = NO GOOD

**Figure 2.1**  
**Preliminary Analysis of Feasible Bridge Types**

		Bridge Type					
		P/S Concrete Type 63		P/S Concrete BT-63		Steel Plate Girder	
Evaluation Criteria	Weighting Factor (1 to 10)	Raw Score (1.0 to 5.0)	Weighted Score	Raw Score (1.0 to 5.0)	Weighted Score	Raw Score (1.0 to 5.0)	Weighted Score
Functional Requirements							
Economics							
Future Maintenance							
Construction Feasibility							
Aesthetics							
ABC (if warranted)							
Other (please identify)							
Total Score				Total Score		Total Score	

**Figure 2.2**  
**Weighted Decision Matrix**

### 2.2.2 Foundation Investigation and Analysis

Foundation Investigation and Analysis is the responsibility of the NMDOT Geotechnical Section. A thorough discussion of this work is beyond the scope of this section and the reader should refer to the NMDOT Materials Geotechnical Manual for greater detail. The interactive nature of this process, however, is worthy of discussion and will be the focus of this section. To ensure the success of the project, the bridge design engineer and the project foundation engineer should actively communicate throughout the design of the bridge.

This communication should be initiated during the Bridge Type Selection and Layout Preparation phase of the project. At that time, discussions between the bridge design engineer and the project foundation engineer should be held. Design parameters and site conditions may be reviewed and any foundation conditions that could affect the selection of the structure type should be identified. For cost estimating and preliminary design, a probable foundation type can usually be identified.

After the Bridge Type Selection and Layout Preparation phase, the completed bridge layout and transverse section sheets are submitted to the NMDOT Geotechnical Section. The NMDOT Geotechnical Section will use these layout sheets to plan the subsurface investigation of the site and to request field borings from the NMDOT Geotechnical Exploration Unit.

Following completion of site and laboratory testing of subsurface samples, the NMDOT Geotechnical Section will begin foundation design and preparation of the Foundation Report. Interaction with the bridge design engineer is also required during this phase. To

complete the bridge design, the NMDOT Geotechnical Section will need loading information, both vertical and horizontal, from the bridge design engineer. For structures that are "fixed" against horizontal movement (i.e., integral abutment structures), the design process is iterative. Additionally, the project foundation engineer should be apprised of construction phasing or other design constraints, since such features should be "designed around."

The foundation investigation and analysis phase of the project culminates with completion and distribution of the Final Foundation Report. The bridge design engineer utilizes the information contained in the report for completing structural design and preparing plans for the bridge's substructure elements.

### 2.2.3 Detailed Design and Plan Development

Detailed design and plan development is the most time consuming phase of bridge design. During this phase, design of each structural element is completed and detailed construction plans are developed. Because this phase is so labor intensive, it is important not to begin work until consensus on the proposed structure type and layout is obtained. At the Preliminary Design Inspection, all design team members had an opportunity to review and comment on the proposed structure type and layout. By a separate submittal, the State Bridge Engineer also had an opportunity to review and comment on the plans. Therefore, after the Preliminary Design Inspection, detailed superstructure design may begin and can run concurrently with the Foundation Investigation and Analysis phase. Substructure design and detailing should be delayed until after the Foundation Report has been prepared.

#### 2.2.4 Checking, Reviews and Approvals

Quality control checking by the NMDOT Bridge Bureau and the formal review and approval process by the State Bridge Engineer is discussed in this section. A systematic quality control process should be an integral part of the design procedure. Checks should be completed at appropriate times throughout the design process. For example, design computations prepared for the bridge type selection should be checked before the bridge layout sheets are drawn. Likewise, the layout sheets should be checked before detailed design and plan preparation begins. In this way, each step is verified before the next step begins, and time lost in correcting errors is minimized.

A qualified person, other than the primary design engineer, checks the design and quantity computations and the plan sheets. When checking design computations and design plans, all entries are reviewed. The following guidelines may be used when checking calculations and plans:

1. When checking calculations, concurrence with the entry is indicated by a check mark.
2. When checking plans, concurrence with the entry is noted by highlighting the entry in yellow.
3. An exception with the entry is noted in red for both plans and calculations.
4. All omissions are also noted in red.
5. All exceptions to computations on plan sheets should be verified by the primary design engineer before they are used to make plan revisions.

The approval of the State Bridge Engineer is required at two stages of plan development. First, approval on bridge type and bridge layout should be obtained before detailed design and plan development work begins. Second, approval of completed plans should be se-

cured before the project is let. Approval of Bridge Type Selection and layout is usually solicited prior to or with the Pre-final Design Inspection submittal. Approval of final plans must be secured shortly after the PS&E review. Bridge plans are not final until the State Bridge Engineer or NMDOT Bridge Bureau Unit Supervisor has signed them.

A set of completed plans also needs to be submitted to the foundation engineer so that he can perform a final review of the foundation details and ensure that the recommendations contained in the foundation report have been correctly interpreted and incorporated into the plan set. A set of completed plans is usually submitted to the foundation engineer for review at the same time that a set is submitted to the State Bridge Engineer. After review, the foundation engineer will initial the bridge layout (or structure location) sheets, indicating concurrence with the foundation information contained in the plans.

### 2.3 CONTENTS OF FINAL BRIDGE PLANS

Bridge plans shall include the following sheets and details:

1. General Notes and Summary of Quantities Sheet(s),
2. Bridge Layout or Structure Location Sheet(s),
3. Substructure Details,
4. Superstructure Details,
5. Reinforcing Schedule, and
6. Specialized Drainage Details.

For each of the above listed items, a checklist of details and information that should be shown on the plans is given in the following sections. The checklists are only guidelines to assist design engineers in preparing plans. Information should be added to accurately depict the structure being designed. Addi-

tionally, information may be transferred from one sheet to another as the need arises.

The NMDOT Bridge Bureau has developed a standardized drafting template, which is available through the NMDOT Bridge Bureau. Plans should be prepared using this template.

The following information should be shown on each sheet as applicable:

- Control Number.
- Title.
- Sheet Number.
- Approval Signature Line for the State Bridge Engineer (including Date Line).
- References to other sheets, details and standards.
- North arrow(s).
- Bridge Number.

### 2.3.1 General Notes Sheet

1. Scope of Work – show for all bridge rehabilitation and preservation projects (not necessary for new and replacement bridges), and
2. Information relating to the entire bridge plan set, including:
  - General Notes,
  - Index of Sheets,
  - Design Data – List design specification, concrete design strength, yield strength of reinforcing bars, grade of structural steel, live load (including NMDOT P327-13 Permit Vehicle), allowance for future wearing surface, earth pressure and wind velocity,
  - Inventory and Operating Load Ratings (LFR and LRFR),

- Estimated Quantities (this may also be on a separate sheet),
- The Bridge Number of the previous bridge, if applicable.

Care should be taken to ensure that General Notes, or any other notes provided in the plans, do not repeat requirements that are already provided in the Standard Specifications or Special Provisions.

### 2.3.2 Bridge Layout or Structure Location Sheet(s)

1. Plan view of the intersecting roadways or waterway containing the following information:
  - Location, station and bearing of new and existing roadways.
  - Roadway lane, shoulder, and median widths.
  - Abutment and pier location, indicating stationing at centerline of bearing.
  - Stationing, grade lines with elevations, and earthwork slopes.
  - Ditches, rundowns, drains, slope protection.
  - Crashwall features (if overpass structure) for protecting the substructure from traffic under the structure, i.e. C.W.B.
  - Approach slabs.
  - Vertical and horizontal clearances.
  - Location of soil borings.
  - Reference to standard details.
  - Existing groundline contours, existing topography, location of utilities, location of existing bridge substructure units.
2. Profile view along the centerline of the bridge (or centerline of construction) containing the following information:

- Under-bridge roadway/railway section and horizontal clearances (if overpass structure).
  - Existing groundline.
  - High-water elevations for the 50 year, 100-year and the 500-year flood event (if waterway crossing).
  - Centerlines of bearing, show stationing and elevations.
  - Location of fixed end and expansion bearings.
  - Bridge length from back to back of backwalls and span lengths from centerlines of bearing.
  - Girder type, type of deck construction and deck width.
  - Finished grade elevation at centerlines of bearing.
  - Details of ditches, drains, slope protection, top and toe of slope, etc.
  - Reference to standard details.
  - Barrier railing on bridge.
3. Vertical and horizontal curve information.
  4. Detail of earthwork at abutments and wingwalls.
  5. Benchmarks and field book numbers.
  6. Geometry of staking plan (curved bridges only).
  7. Under the heading WATERWAY DATA show the following information for stream and river crossings for the 50 year, 100 year and the 500 year event:
    - Drainage Area and Type of Terrain (acres)
    - Design Flood (cfs)
    - High-water elevation (ft)
    - Design Velocity of Flow (ft/s)
    - Scour expected (ft)
    - Debris
- Note about 404 Permit for waters controlled by the US Army Corps of Engineers.

### 2.3.3 Substructure Details

The substructure consists of all bridge components that are below bearing seat elevation. The substructure details begin with the foundation layout and notes, showing the location and requirements for the foundation elements. Additional details necessary to adequately describe the substructure follow. Details, diagrams and tables describing each component follow. The details of each component may be incorporated into one drawing or may be shown on separate drawings, as appropriate. All details must be sufficiently dimensioned and annotated to construct the bridge.

Details that may be shown, depending on the structure type, are given below. Under each item is the information that should be included.

1. Foundation Layout Sheet including Driven Pile Table or Drilled Shaft Table containing the following information from the foundation report:
  - Location (abutment or pier no.),
  - Number required at each location,
  - Estimated length,
  - Design/maximum loads, and
  - Estimated and/or minimum penetration elevations.
2. Abutment Details:
  - Plan, elevation and sections,
  - End plate placement detail,
  - Foundation element layout,
  - Details of reinforcement,
  - Wingwall details, and

- Lateral and vertical anchorage details.
3. Pier Details:
    - Plan, elevation, sections,
    - Details of reinforcement,
    - Pile splice details for each size of piling used on the bridge (if applicable), and
    - Lateral and vertical anchorage.
  4. Earth Retaining Wall Details:
    - Plan, elevations and sections,
    - Construction notes, and
    - Details.

#### 2.3.4 Superstructure Details

The superstructure consists of all bridge components that are above bearing seat elevation. The superstructure details begin with the transverse section view, showing components relative to each other. Additional details necessary to adequately describe the superstructure follow. Details, diagrams and tables describing each component follow. The details of each component may be incorporated into one drawing or may be shown on separate drawings, as appropriate. All details must be sufficiently dimensioned and annotated to construct the bridge.

Details that may be shown, depending on the structure type, are given below. Under each item is the information that should be included.

1. Transverse Bridge Section:
  - Deck and deck cross slopes,
  - $\frac{3}{4}$  inch drip groove at edge of deck,
  - Rebar,
  - Girder type and girder spacing, and
  - Bridge railing.
- a. Prestressed concrete girder bridges typically have sections taken at in-

termediate diaphragms and at abutment backwalls. Supplementary details include:

- Section thru intermediate diaphragms, and
  - Details of pier and abutment diaphragms.
- b. Steel girder bridges typically have sections taken near mid-span and near the pier. Supplemental details include:
    - Detail of cross frame at pier,
    - Detail of cross frame at abutments, and
    - Diaphragm details, layout and connections.
  - c. Related but separate details include:
    - Enlarged slab details at girders (showing minimum design haunch), at bridge railing, and at median barrier, and
    - Construction phasing requirements.

2. Framing Plan:
  - Girder spacing,
  - Location of diaphragms, intermediate stiffeners, cross framing, etc., and
  - Location of centerlines of bearing.
3. Deck Details:
  - Reinforcing plan(s) showing longitudinal and transverse steel, top and bottom mats: layout, rebar size and spacing, rail post spacing,
  - Construction joint details,
  - Slab joint section and details,
  - Dead load deflection diagram,
  - Diagram for placing concrete deck,
  - Plan showing top of deck elevations, and

- Detail showing offset of one-half of the bar spacing between top transverse bars and bottom transverse bars.
4. Bearing Details:
- Elastomeric bearing pad details,
  - Bearing pad design loads, deflection,
  - Sole plate details, and
  - Bearing details at abutments and piers.
5. Steel girder details:
- Girder elevation showing girder size, flange and web plate sizes, size and location of intermediate and diaphragm connection plates,
  - Span lengths,
  - Location of intermediate stiffeners, shop splices, and field splices,
  - Girder camber diagrams and dead load deflection diagrams,
  - Shear stud details and spacing,
  - Notation on material that needs to meet Charpy V-Notch test requirements,
  - Web cutting details,
  - Appropriate details for variable depth girders, i.e. web haunch detail,
  - Field splice and cover plate details,
  - Shop splice details,
  - Shear connector detail, show size and spacing, and
  - Sketch showing tension flange.
6. Prestressed Concrete Girder Details:
- Standard drawings have been prepared by NMDOT for various types of prestressed girders. The following checklist is to be used to complete the drawings for project specific applications.
- a. End view:
    - Strand pattern (delete strands not required), and
    - Change bar sizes as required.
  - b. Section near centerline of girder:
    - Strand pattern (delete strands not required), and
    - #4S1 bar projection from top of girder.
  - c. Girder Data Table:
    - Weight of girder,
    - Camber at release,
    - Camber at erection (must be adjusted for camber growth to the time of erection), and
    - Dead load deflection @ 90 days (include all dead loads, i.e. allowance for stay-in-place deck forms, future wearing surface, barrier railing, etc.).
  - d. Elevation view:
    - S bar spacing,
    - H bar spacing (conform to AASHTO section 5.9.4.4.2, Confinement Reinforcement. Spacing shall not exceed 6 inches and shall extend 1.5d from the ends of the girders),
    - Total number of strands and number of straight and drape stands,
    - Dimension from mid-span to centerline of bearing, and the dimension from the centerline to centerline of bearing,
    - Spacing for the diaphragm holes,
    - Distance to hold down point,



- Designer needs to check the requirements for anchorage zone reinforcing, and
  - $f_{se}$  and prestressing losses.
- e. Reinforcing bar schedule:
  - Add missing information to the table.
- f. Title page block:
  - Add project name and any required identification information (span number, girder designation, etc.).
- g. Design data:
  - $f'_c$  and  $f'_{ci}$
- 7. Abutment/Pier Diaphragm Details:
  - Plan, section, and elevation views,
  - Reinforcing details,
  - Vertical restrainer details,
  - Blockout details at bearings and restraining devices, and
  - Details for separating superstructure and substructure elements as appropriate.
- 8. Joint Seal Details:
  - Plan and section details,
  - Reference to standard drawing,
  - Movement length, and
  - Seal length.
- 9. Table showing depth of superstructure ("P" dimensions):
  - Show depth of each component (deck, haunch at girder ends, girder, bearing, etc.).
- 10. Approach slab and details:
 

NMDOT has prepared worksheets for approach slabs with and without sleepers. These drawings can be obtained from the NMDOT Bridge Bureau. The

bridge designer can complete and submit the drawings with the plan set.

### 2.3.5 Reinforcing Bar Schedules

Reinforcing bar schedules are provided on bridge design plans to assist the Contractor in identifying the various reinforcing bars in a concrete structure. These schedules should contain the following information:

- Bar marks (each unique to a particular bar and including the bar size),
- Size of bars,
- Length of bars,
- Indication of plain or epoxy-coated bars,
- Type (bent shape) of bars, and
- Quantity of bars needed.

Reinforcing bar schedules may either be placed on individual sheets corresponding to the structural elements depicted on each sheet, or they may be combined into a single schedule following the approach slab sheets and list all of the reinforcing bars contained in the plans. The latter is usually preferred by Contractors as a combined schedule assists them in developing estimates more efficiently.

### 2.3.6 Specialized Drainage Drawings

Specialized drainage drawings associated with a bridge may need to be included to address such features as riprap and drainage structures that are not adequately addressed by standard drawings. These drawings are prepared based on recommendations and designs provided by the Drainage Engineer.

### 2.3.7 Environmental Requirements

Supplemental sheets to the bridge sheets may also be required to address specific environmental commitments associated with the project. The most common items addressed by



these drawings include bat boxes and wetland mitigation areas.

Other environmental requirements not addressed in the bridge plans may need to be addressed through Notices to the Contractor or Special Provisions. Coordination with the NMDOT Environmental Division is essential to ensuring that environmental requirements are adequately addressed in the Contract.



## CHAPTER THREE

### BRIDGE LOADS

This section will identify and define what the NMDOT fundamentally recognizes and uses as the loads and load combinations in its bridge design procedures.

In using the LRFD design method, factored loads are compared to factored resistances. The load factor applied to an individual load depends both upon the type of load and the limit state under consideration.

The Load Combinations and Factors presented in Table 3.4.1-1 of the AASHTO LRFD Specification are to be used in New Mexico.

#### 3.1 LOAD TYPE DEFINITIONS AND LOAD MODIFIERS

Definitions of what normal and/or project specific loads and loading combinations are generally included by NMDOT in the design process follow. In addition, practices currently in use are described.

The reader is referred to the AASHTO LRFD Bridge Design Specifications, Section 3 (Loads and Load Factors) for more thorough definitions of these terms. In addition, the LRFD Specifications provide direction and guidance concerning all other items or matters not specifically covered below.

##### 3.1.1 Load Modifiers

AASHTO LRFD article 1.3.2.1 required that loads be multiplied by both a load factor and load modifiers. There are three modifiers related to the ductility, redundancy and importance of the structure. The application of the modifiers related to ductility and redundancy are straightforward. Some direction in the application of the modifier related to the importance of the structure is however necessary.

In the application of that modifier, bridges crossing major rivers and all bridges carrying interstate traffic shall be considered important bridges. All others shall be considered typical bridges. Major rivers are defined as the Rio Grande, the San Juan, the Animas, the Pecos and the Canadian.

##### 3.1.2 Dead Loads

Dead loads consist of the weights of all permanent portions of an entire structure and include weights of any anticipated future additions. Provisions are to be made in the design calculations to add 30 psf for future wearing surface overlays to the deck. In addition, 15 psf for the use of metal stay-in-place bridge deck forms needs to be added. Also, future utility and planned future bridge expansion effects shall be accommodated in the bridge design, if known. The weight of the concrete wall barrier can be distributed equally among all girders if the conditions of AASHTO Section 4.6.2.2.1 are met.

In the absence of more specific material data, use AASHTO Table 3.5.1-1 for computing dead loads of various materials.

##### 3.1.3 Live Loads

Live loadings that bridges must carry consist of the moving dynamic weights of motor vehicles, pedestrians, equestrians, cyclists, and all others crossing the bridges. The discussion below is limited to highway motor vehicle loads.

The Design Live Load (HL93) consists of a design truck or design tandem and a design lane load. With a few exceptions the design truck or the design tandem is applied simultaneously with the lane load.

### 3.1.3.1 Standard Trucks and Lane Loads

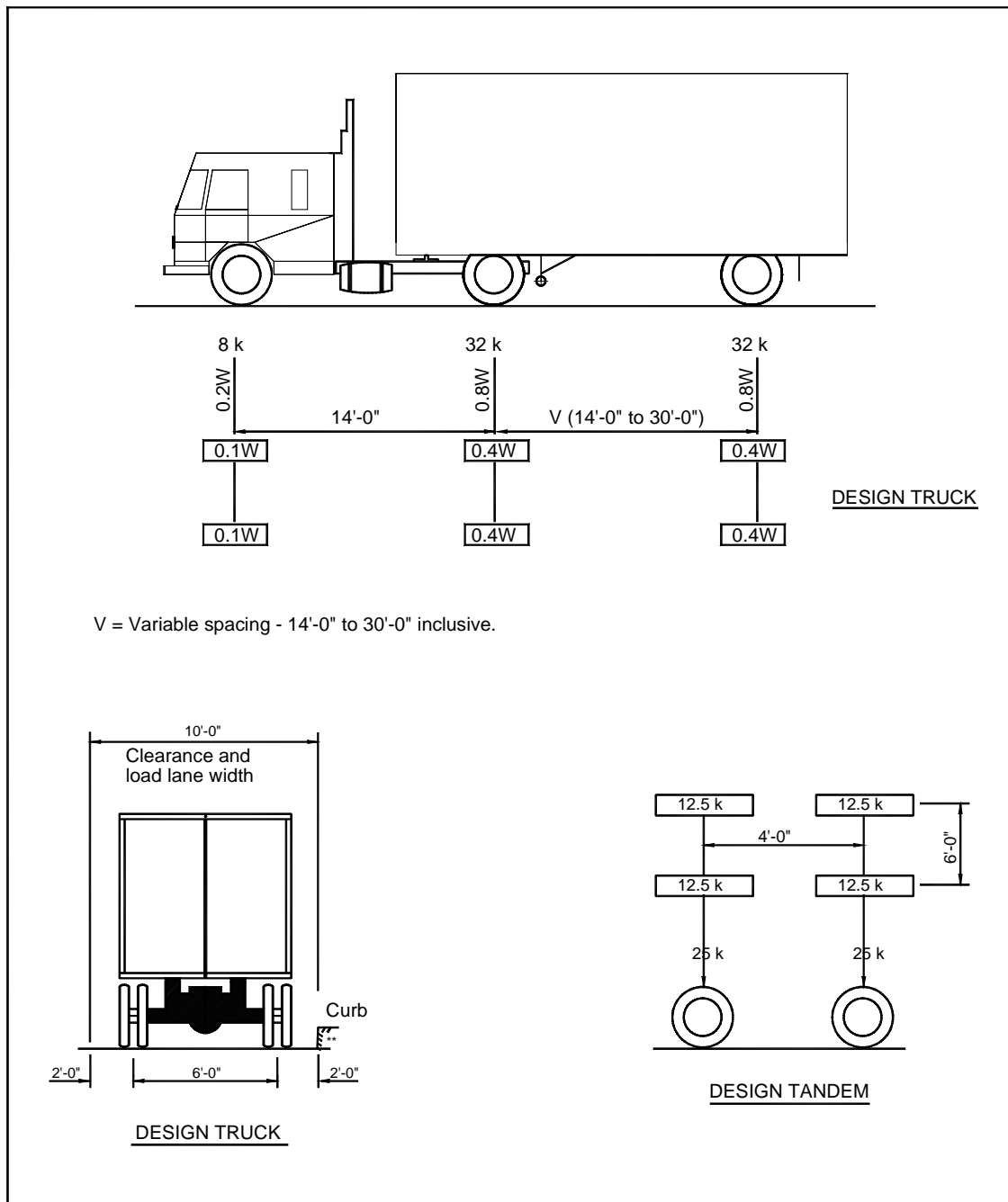
The design truck and design tandem loads are shown in Figure 3.1A. The design lane load consists of a load of 0.64 klf. The HL93 shall be applied as follows.

#### Maximum Force Effect

1. The design truck in combination with the design lane load.
2. The design tandem in combination with the design lane load.
3. For negative moments and pier reaction only, 90% of 2- design trucks in combination with 90% of a design lane. Rules for placing the trucks are in Article 3.6.1.3.

The combination producing the largest effect shall be used.

**Figure 3.1A**  
**Design Truck and Design Tandem**



### 3.1.3.2 Design Traffic Lanes

For the purposes of design, the bridge roadway is divided into longitudinal design traffic lanes 12 feet wide. Fractional parts of design lanes shall not be used. Roadway widths from 20 feet to 24 feet shall have two design lanes, each equal to one-half the roadway width.

Both the standard truck and lane load use a width of 10 feet. The 10 foot loads may move within the 12 foot wide lanes which, in turn, may move between curbs. Traffic lanes and loads within lanes shall be positioned to produce maximum stresses in the structural components being analyzed.

### 3.1.4 Dynamic Load Allowance

The highway live loads applied to structural bridge elements which are above the ground surface are increased by a specified allowance to account for various dynamic, vibratory, and impact forces. For components other than wood and structures other than culverts, the factor to be applied to the static live load (excluding lane load) to account for dynamic effects (IM) is

Component	Allowance
Deck Joints-	
- All Limit States	1.75
All other components	
- Fatigue and Fracture Limit States	1.15
- All other Limit States	1.33

For Culverts the allowance is

$$IM = 1 + 0.33 (1 - 0.125 De) \geq 1.0$$

De = Depth of cover in feet.

For wood components, a dynamic load allowance need not be applied (AASHTO 3.6.2.3).

### 3.1.5 Centrifugal Forces

Bridges on curves are designed for a horizontal radial force, taking into effect any superelevation. The centrifugal force is equal to the products of the truck or tandem axle

$$C = f \left( \frac{V^2}{gR} \right)$$

weights and the factor C:

Where C is a factor, f is 4/3 for load combinations other than fatigue and 1.0 for fatigue, V is the design speed in ft/sec., R is the radius of the curve in feet and g is the gravitational acceleration. Neither lane loads nor impact on live loads shall be used in the computation of centrifugal forces. The force shall be applied at 6 foot above the roadway surface measured along the centerline of the roadway. Multiple presence factors shall apply.

### 3.1.6 Braking Forces

The braking force per lane is to be taken as the larger of 25% of axle loads or 5% of axle loads plus the lane load. All lanes carrying same direction traffic are to be considered loaded. The load is applied at a center of gravity point 6 feet above the floor slab. The longitudinal force is transmitted to the substructure through the superstructure. Multiple presence factors shall apply.

### 3.1.7 Temperature, Shrinkage and Creep

#### 3.1.7.1 Temperature

Forces, stresses, and movements in bridge components and members due to temperature variations (both atmospheric and in construction materials) must be calculated and

provided for in such items as elastomeric bearing pads, expansion bearing devices, deck joint sealing systems, and deck joint openings. New Mexico temperature ranges used in movement and force calculations are listed in Table 3.1A.

**Table 3.1A**  
**Temperature Ranges**

<b>Elevation Less than 4500 Feet Above Sea Level</b>		
Structure Type	Temperature Range	Min/Max design temperature
Steel	120°F	(0° to 120°F)
Concrete	80°F	(10° to 90°F)

<b>Elevation Greater than 4500 Feet Above Sea Level</b>		
Structure Type	Temperature Range	Min/Max design temperature
Steel	130°F	(-20° to 110°F)
Concrete	80°F	(0° to 80°F)

Note: Each bridge location should be evaluated for average temperature to make sure the elevation of 4500 ft. is applicable.

The full temperature range is used in design of the superstructure and substructure because the structure is anticipated to have these full movements during its life.

The thermal movement used in the design of elastomeric bearing pads shall be not less than 75% of the total anticipated movement due to temperature. The assumption made is that the girders will not be placed on the pads at the upper or lower end of the temperature range. The designer shall specify the range of temperature in which the girders shall be placed.

Unless a more precise method of measuring the temperature of the girders is used, the setting temperature of the girders or bridge component shall be taken as the average of

the actual air temperature over the 24-hour period immediately before the setting of the girders or component.

The size of deck joint seals and required deck openings may be found in the appropriate standard drawings issued by the Department. These standards consider factors in addition to temperature, such as creep and shrinkage. If the movement length of a structure exceeds that given in the standard drawings, a special joint seal may be required. If situations arise which require special consideration, contact the State Bridge Engineer for assistance.

Thermal expansion coefficient conversions are given by:

$$\text{Steel: } \Delta L = L \times 0.000078 \times \Delta T$$

$$\text{Concrete: } \Delta L = L \times 0.000072 \times \Delta T$$

where  $\Delta L$  is expressed in inches and  $L$  is expressed in feet and  $\Delta T$ , the change in temperature, is in °F. Generally the effect of temperature gradient need not be considered.

### 3.1.7.2 Creep and Shrinkage

A prestressed girder design example is available on the NMDOT Bridge Bureau website (<http://dot.state.nm.us>). Example calculations for Shrinkage and Creep are available in this design example. The design approach in the example is derived from the NYSDOT Bridge Manual, 1<sup>st</sup> Edition with 2010 Addendum. This is the preferred method for calculating shrinkage and creep in the girders. The method outlined in AASHTO is also acceptable.

### 3.1.8 Wind Loads

Wind loads applied to bridges are to be calculated according to the AASHTO LRFD Bridge Design Specifications. The structural supports of signs, luminaires, and traffic signals are designed according to the

AASHTO Standard Specification for Structural Supports for Highway Signs, Luminaires and Traffic Signals. Sound walls are designed according to AASHTO's Guide Specifications for Structural Design of Sound Barriers. The wind load for structural supports and for sound walls shall be calculated using a design wind velocity of 80 mph.

The wind loads and forces applied to bridges are given in the AASHTO LRFD Bridge Design Specifications. These loads and forces are computed from a base wind velocity of 100 mph. If a change is made to the base wind velocity due to a higher known maximum wind velocity, the new base wind velocity is shown on the plans after approval by the State Bridge Engineer. Changes in the design wind velocity may be made if permanent terrain features make such changes safe and advisable.

### 3.1.9 Stream Pressure and Buoyancy Forces

The structure and its components are designed to withstand the maximum stresses induced by stream flow and buoyancy. The structural elements which may be affected by these forces include substructure piers, footings, abutments, pilings, and walls to the superstructure deck, girders, railings, connections, and others.

The NMDOT Bridge Bureau's policy is to design bridges over waterways for the design flood event using the AASHTO strength load combinations, and to check the bridges to ensure they can withstand the 500-year flood event using the AASHTO Extreme Event load combinations. Scour, flotation, and the connection between the superstructure and substructure are of particular concern for the 500-year event.

During a 500-year flood event, the high water elevation may overtop the bridge. Bridge

design should consider that in some cases this flooding could cause the roadway embankment to fail. If this occurs the water level may drop and result in lower stream velocities. This actually may produce lower stream forces than the design flood.

### 3.1.10 Earth Pressures

Lateral earth pressures should be calculated according to Section 3.11.5 of the AASHTO LRFD Bridge Design Specifications. Although the Rankine method was specified in earlier versions of the Specifications, the current code uses equations based on the Coulomb earth pressure theory. The difference between the two methods is shown in Figure C3.11.5.3-1 of the Specifications.

Equations to determine Equivalent Fluid Pressure are available in Section 3.11.5.5 of the Specifications. The equivalent-fluid method may be used where Rankine earth pressure is applicable. Applicability of the Rankine theory is discussed in Section C3.11.5.3. This method shall only be used where the backfill is free-draining. Therefore, adequate drainage for the backfill shall also be provided in the structure design to minimize any hydrostatic pressure buildup. The minimum active earth pressure to be designed for in structures that retain fill such as abutments and retaining walls shall be not less than an equivalent fluid pressure (for the soil) of 36 lb/ft<sup>2</sup> per foot of height.

The various Rankine formulas to be used for earth pressure calculations are given in Section 3.11.5 of the Specifications. The formulas provide values for active or passive pressures with or without cohesion on flat or inclined backfill slopes. In addition to the active and passive earth pressures values, the at-rest pressure  $K_o$  should be used in rigid structures that do not yield from earth pres-



sure. The formula for  $K_o$  can be found in Section 3.11.5.2 of the Specifications.

Live load surcharges are specified in AASHTO LRFD Article 3.11.6.4. At bridge abutments which have approach slabs, a live load surcharge is to be applied in both abutment and wing wall design. The surcharge is applied to account for the pressures generated by the high compaction effort used in the backfill behind the abutment.

### 3.1.11 Seismic Forces

NMDOT Bridge Bureau has set forth the following Policy Statements for determining seismic load factors for the LRFD Code. Bridges crossing the Rio Grande and interstate highways shall be considered essential. All others shall be considered “other” bridges.

Structures subjected to earthquake forces shall be designed to survive the strains resulting from the design earthquake motion. Factors that are considered when designing to resist earthquake motions are:

1. The proximity of the site to known active faults.
2. The seismic response of the soil at the site.
3. The dynamic response characteristics of the total structure.

The design to survive earthquake forces shall be in accordance with 3.10 of the AASHTO LRFD Bridge Design Specifications or the AASHTO Guide Specifications for LRFD Seismic Bridge Design.

Seismic Design Parameters should be determined based on location using software developed by the U.S. Geological Survey (USGS). A version of the USGS software specifically for bridges designed according to AASHTO LRFD is available on the USGS website. The method (procedure) of seismic

analysis is selected based on these Seismic Design Parameters, the complexity of design required for the bridge, and the number of bridge spans proposed.

Seismic load factors shall be in accordance with AASHTO Section 3.4.1, modified as follows:

AASHTO 3.4.1, next-to-the-last paragraph:  $\gamma_{EQ}$  shall be determined on a project-specific basis for operationally important structures as determined by the NMDOT Bridge Bureau. For all other bridges,  $\gamma_{EQ} = 0$ .

Typically, vehicular live loads have not been shown to be in phase with the bridge during seismic events. The inertial effect of live loads is therefore assumed to be negligible.

### 3.1.12 Vehicular Collision

The provisions of Article 3.6.5 of the LRFD specifications shall apply as applicable.

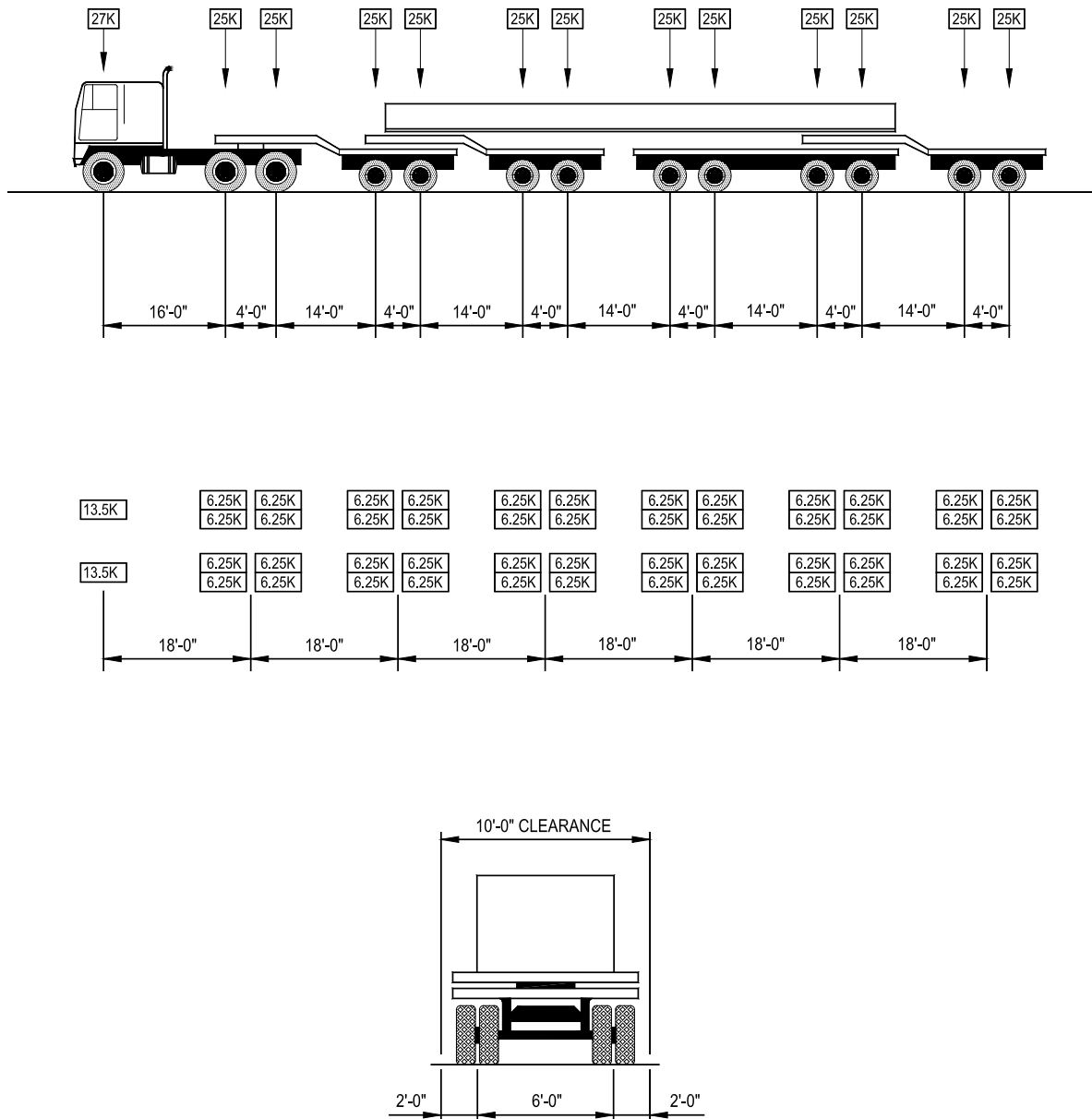
### 3.1.13 New Mexico Permit Load

The current NMDOT Permit Vehicle, P327-13, shall be used for the design of superstructure and substructure excluding the bridge deck slab. Refer to Figure 3.1B for a definition of the Permit Vehicle. This is applied in AASHTO LRFD 3.4.1 Strength II. The Permit Vehicle shall be used for all state-owned and operated bridges, unless otherwise approved by the State Bridge Engineer. The Permit Vehicle need not be applied with other live loads.

## 3.2 LOAD COMBINATIONS

Load combinations are presented in 3.4 of the LRFD Bridge Design Specifications. Each component of the structure shall be designed to safely withstand all applicable load combinations.

**Figure 3.1B**  
**Permit Design Live Load P327-13**



## CHAPTER FOUR

### REINFORCED CONCRETE DESIGN

Cast-in-place, reinforced concrete is used extensively for many bridge elements. These elements include abutments, wingwalls, piers, foundations, decking, and cast-in-place girders to name a few. Although reinforced concrete is widely used it does have its limitations for its use as the primary structural members.

Superstructures are rarely constructed of conventionally reinforced concrete. Prestressed concrete is much more common. The exception is the slab bridge which has a proven record of being a very durable long lasting structure.

The span lengths that can be achieved with a slab bridge are fairly short, between 20 feet and 40 feet; therefore, they tend to be used on small stream crossings. The superstructure depth for slab bridges generally ranges between 12 inches and 18 inches. Where site conditions require a shallow superstructure, the slab bridge is quite often the best choice. A skew greater than 45 degrees, however, may eliminate the feasibility of using a slab bridge.

#### 4.1 MATERIAL STRENGTHS AND PROPERTIES

The NMDOT Standard Specifications for Highway and Bridge Construction contains requirements for materials used in bridge construction. There are designated classes of concrete, each having a minimum compressive strength, to which concrete structures must conform. The following section gives the concrete class and the minimum material properties on which to base design.

##### 4.1.1 Concrete

Concrete compressive strengths usually used for design are listed below. The strength listed for substructure design may be increased to 4000 psi if the increase is advantageous for the design under consideration.

Superstructure elements:	4000 psi
Substructure elements:	3000 psi
Drilled shafts:	3000 psi

Each of these bridge components are discussed in the following paragraphs.

Superstructure concrete is Class HPD. All cast-in-place concrete placed in bridges above the bearings is superstructure concrete. The wingwalls, end diaphragms, and concrete above the bearings in bridges with integral and semi-integral abutments are constructed of superstructure concrete.

Box culverts and wingwalls are constructed with Class AA concrete.

Bridge substructure elements and retaining walls are constructed with Class A concrete. Substructure elements include abutments, bents, and piers.

Drilled shafts concrete bearing piles are constructed of Class G concrete.

##### 4.1.2 Reinforcing Bars

Reinforced concrete structures are designed and constructed with Grade 60 rebar having a yield strength of 60 ksi. All components should be designed with a yield strength of 60 ksi, even if Grade 75 rebar is detailed on a particular component.

When checking flexural reinforcement distribution per AASHTO LRFD Section 5.6.7, use the following exposure factor statewide in Equation 5.6.7-1:

$\gamma_e = 1.00$  for Class 1 exposure condition

Epoxy coated, galvanized or uncoated corrosion resistant bars are used for:

- a) All bars in cast-in-place elements of the deck, superstructure and approach slabs.
- b) The bars projecting from a prestressed girder's top flange, excluding epoxy bars.
- c) The prestressed girder's reinforcement that is within 4 feet of the end, excluding epoxy bars.

## 4.2 DETAILING REQUIREMENTS

Information on the proper practice for detailing of reinforcing bars is available in the Manual of Standard Practice by the CRSI (Concrete Reinforcing Steel Institute). Designers and detailers should be familiar with this publication. All dimensions on plans are to centerlines of bars unless otherwise noted on the details. Bend radii are also to the centerline of the bar unless otherwise noted. Calculated weights of bars are based on centerline lengths.

Minimum bend diameters for both uncoated and coated bars are listed in the NMDOT Standard Specifications.

### 4.2.1 Lengths and Splices

All sizes of bars are readily available in lengths up to 60 ft, including epoxy coated rebar. Shorter length bars may be used and spliced if preferred; however, the extra splices should not be detailed unless there are special reasons for detailing the splices.

Mechanical splices/ connectors may be used for AASHTO M31, Grade 60 deformed bar if conditions of the construction require them. The splice strength of the couplers (verified by tests) must be greater than or equal to 125% of the yield strength of the spliced reinforcing bars and be capable of developing the specified tensile strength of the bars. The State Bridge Engineer must approve the location and the type of coupler.

Splices at critical locations should be avoided whenever possible. If mechanical splices need to be placed at a critical location, stagger the splices one half of a class A splice length if possible.

### 4.2.2 Reinforcing for Bearing Piles

The use of spiral reinforcement in cast-in-place concrete bearing piles (drilled shaft and augercast piles) has created problems with constructing these foundations. Tie reinforcement should be used instead.

### 4.2.3 Concrete Cover

Concrete cover must be as specified in the NMDOT Standard Specifications for Highway and Bridge Construction Section 540 Steel Reinforcing. Standard practice at NMDOT is to detail to the centerline of bar unless otherwise noted; however, in cases where clear cover is measured in the field, it is desirable to provide a clear cover dimension in lieu of a centerline dimension. Common instances of this are:

- Top and bottom clear cover in bridge decks,
- Top and bottom clear cover in approach slabs, and
- Clear cover of drilled shaft reinforcing bars.

## 4.3 STANDARD DECK SLABS

The NMDOT Bridge Bureau first made deck slab design tables and standard details availa-

ble in the 1979 Bridge Design and Detailing Instructions. Slabs designed using these tables have proved to be more rigid and durable than those of conventional design. The NMDOT has noted less durability and a shorter deck life in decks thinner than those specified in the standard design tables. The use of the standard designs should therefore continue.

#### 4.3.1 Deck Slab Design

The standard deck slab detail shown in Figure 4.3A are basically the 1979 designs, but some minor modifications have been made to them. Because the slab has already been designed, the designer simply needs to determine the slab thickness and distribution reinforcement from the corresponding table, based on the girder spacing, and include them into the standard slab detail. The standard slab is based on the design presented in the 1979 Bridge Manual.

The standardized slab design can be used for most deck slabs; however caution should be exercised if the bridge has a large skew or some other unusual features. Additionally, if deck slab cantilevers are longer than about 4 feet, the adequacy of the tabulated amount of reinforcing to carry the loads imposed on the cantilever needs to be checked.

In bridges with a skew equal to or greater than 20 degrees, transverse reinforcing should be placed perpendicular to the girders. Bridges with a skew less than 20 degrees can have reinforcing placed parallel to the skew. When reinforcing is placed parallel to the skew, the Effective Span “S” in the design table should also be measured along the skew (See Figure 4.3B).

NMDOT has used thinner decks in a few instances over the years. Experience has shown that thinner decks do not have the long-term durability of the standard deck slab details of

Figure 4.3A. The standard deck should therefore always be used unless approval to use a thinner deck is obtained from the State Bridge Engineer. Approval has historically been granted in cases where rehabilitation budgets haven’t allowed for superstructure modifications to carry a heavier deck.

#### 4.3.2 Slab Details

To reduce transverse cracking in newly constructed bridge decks, the transverse bars in the top and bottom mats of deck slab reinforcement should be offset by 1/2 of the bar spacing. This does not apply to slab bridges.

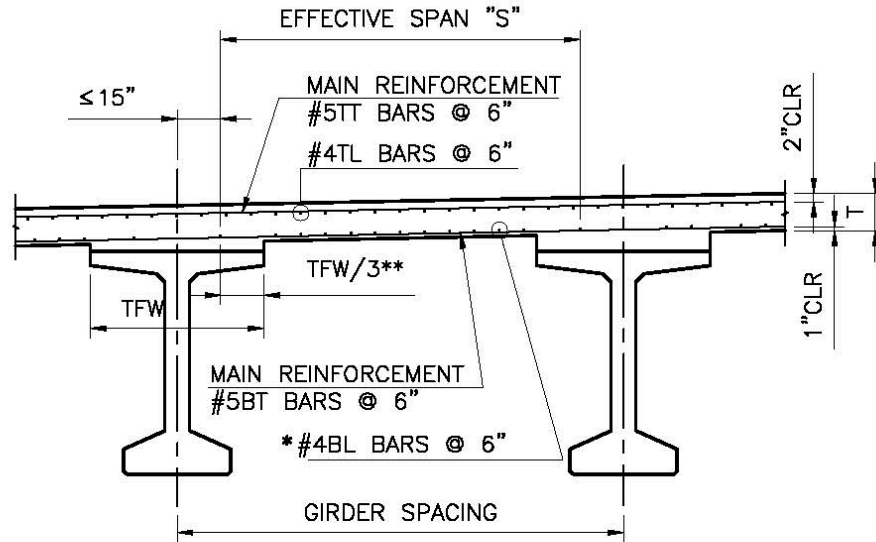
The use of staggered lap splices in adjacent lines of longitudinal bars may be used to reduce the required lap splice lengths. This practice is encouraged but not required, particularly if it results in an overly complex reinforcing bar layout.

For all new bridge decks, stay-in-place deck forms are customarily used; these forms must be ventilated to avoid trapping moisture.

When using corrugated steel stay-in-place deck forms, special attention should be given to the amount of cover on the bottom bars. If the bottom transverse bars are parallel to and located over the valleys of the corrugations, then one inch of cover is provided relative to the bottom longitudinal bar. Otherwise, one inch of cover shall be provided relative to the transverse bars. See Figure 4.3D for a comparison of bridge deck cover and thickness requirements for conventional wood forms and corrugated metal stay-in-place deck forms.

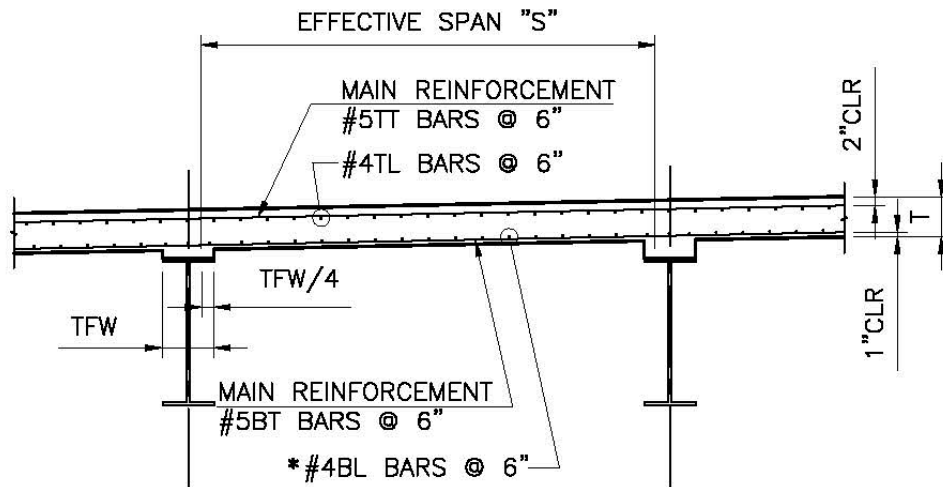
The theoretical strength of the stay-in-place deck forms and the concrete that fills the troughs in the forms shall be neglected in all final structural strength calculations.

**Figure 4.3A**  
**Standard Deck Slab Details for HS 20 Loading**



\*USE TABULATED SPACING FOR DISTRIBUTION BARS IN THE MIDDLE HALF OF SPAN AND DOUBLE THE SPACING IN THE OUTER QUARTERS OF SPAN.

\*\*DIMENSION SHALL BE  $TFW/3$ , BUT NOT EXCEEDING 15" FROM THE  $\bar{C}$



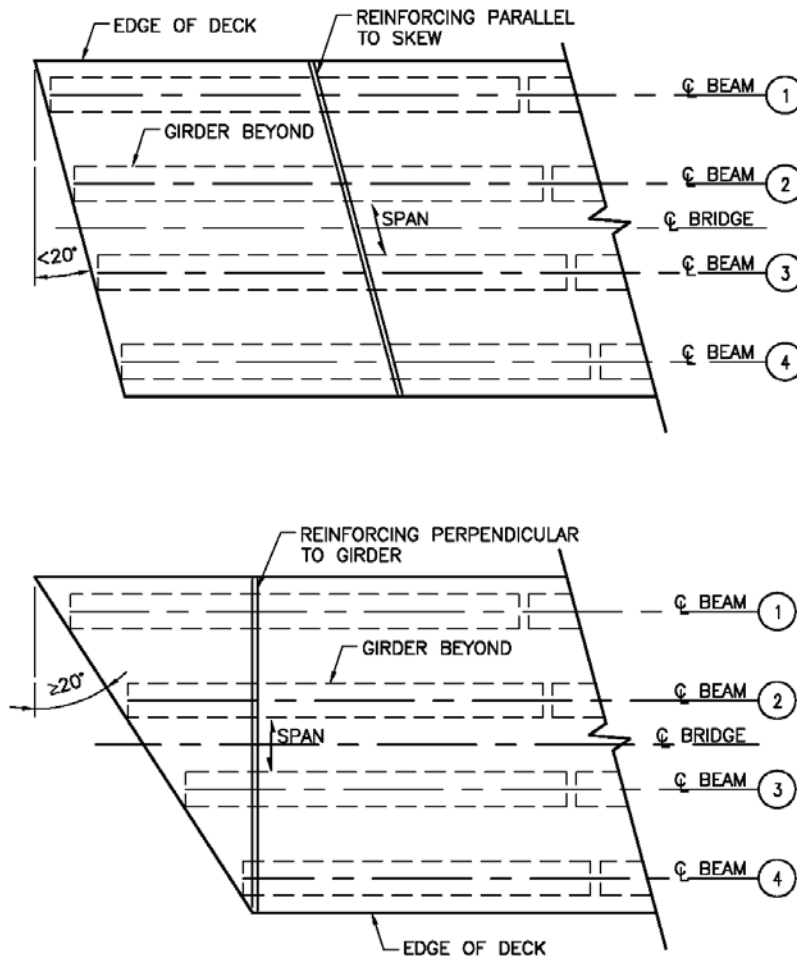
\*USE TABULATED SPACING FOR DISTRIBUTION BARS IN THE MIDDLE HALF OF SPAN AND DOUBLE THE SPACING IN THE OUTER QUARTERS OF SPAN.

**Figure 4.3B**  
**Deck Slab Design Table**

T (in)	S <sub>max</sub> (ft)
8"	6'-7"
8 ½"	7'-7"
9"	8'-6"
9 ½"	9'-5"
10"	10'-3"
10 ½"	11'-1"
11	11'-10"

T=Slab Thickness      S<sub>max</sub>=Effective Span

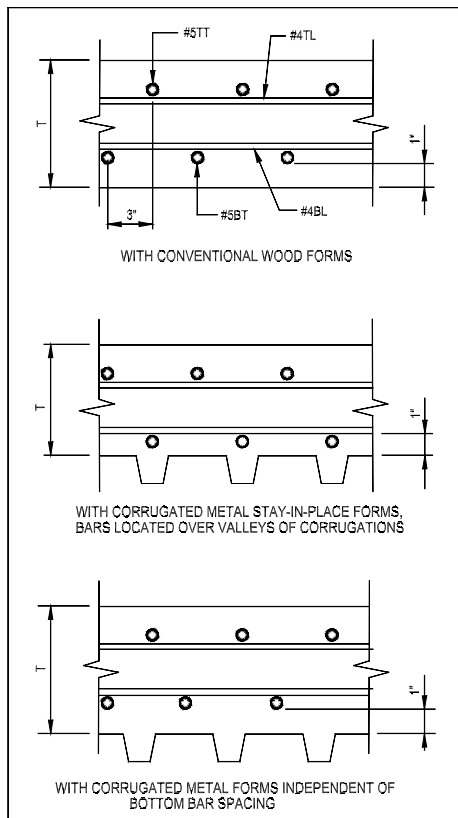
**Figure 4.3C**  
**Transverse Reinforcing in Skewed Decks**





When an existing bridge is being redecked, saving weight on the new deck is sometimes an important issue. In such cases, one weight saving option that can be used is to specify that the corrugations in the metal forms be filled with extruded polystyrene. This will eliminate the extra weight caused by the use of the forms and the 15 psf dead load allowance required for using them need not be considered in the design. The drawings need to clearly state the design loads used and if extruded polystyrene is required. The design load for metal deck plus extruded polystyrene is 3 psf for girder spacings of 10 feet or less. For spacing greater than 10 feet, the bridge design engineer will need to determine the design load for the metal deck plus extruded polystyrene.

**Figure 4.3D - Comparison of Deck Thickness and Cover Requirements**



### 4.3.3 Concrete Deck Placement

When placing the bridge deck, full width placement is the preferred manner to place concrete. However, if the total skewed width of the deck slab exceeds 60 ft, or if full width placements at the placement rate specified (typically 30 ft/hr) results in a volumetric rate of placement greater than 45 cu. yd./ hr, the State Bridge Engineer should be contacted for instructions regarding placement width.

Deck slabs on prestressed concrete girders are typically placed according to a predetermined placement sequence. However, the deck may be permitted to be placed continuously from one end of the bridge to the other by the State Bridge Engineer provided that the Contractor's proposed deck placement procedure adequately insures that the end to end placement will be successful.

An additional item of concern is placement of concrete diaphragms. Integral abutment diaphragms and pier diaphragms for continuous for live load bridges must be place monolithically with the deck slab concrete. If preplaced, they can severely spall when the deflections due to deck slab placement occurs.

### 4.3.4 Deck Overhang Design

It has been considered by some bridge engineers to be a good practice to design the deck slab overhang such that the railing system will fail before the deck does. The rationale behind this practice seems to be that deck slabs are difficult to repair. While in theory this seems to be reasonable, it does lead to an excessive amount of reinforcing steel in the overhang. Additionally, it is not true that deck slab overhangs are hard to repair. For a skilled bridge crew the process is quite straight forward and quickly accomplished. This practice should therefore not be used in New Mexico and deck slab overhangs need not be designed stronger than the deck to rail connection.



### 4.3.5 Precast Concrete Deck Panels

With the increased interest in Accelerated-Bridge Construction methods and reduce construction impacts on motorists, precast concrete deck panels may be preferred in certain situations.

There are two types of precast concrete deck panels that are typically used: partial-depth panels and full depth panels.

Partial Depth panels act in a manner similar to steel stay-in-place decks forms; they remain in place and support cast-in-place concrete. However, the partial depth panels are considered a structural part of the final bridge deck. These panels are typically about 4 inches thick. The panels typically span between the beams, leaving room between them for horizontal shear reinforcement from the girders. A mat of reinforcing steel is then placed on top of the panels, and the remainder of the concrete deck is poured on top and between the panels, creating a composite decking system. Example details are available by contacting the NMDOT Bridge Bureau.

Full depth panels act as the final bridge deck and are placed directly on the girders. Leveling devices are used to ensure that the correct top of slab elevations are achieved. They can be designed with normal reinforcing or with prestressing strands. Full depth panels are made composite with the girders through the use of shear keys. The AASHTO LRFD Bridge Specifications have included some requirements for the use of Full Depth Panels in Section 9.7.5.



## CHAPTER FIVE

### PRESTRESSED CONCRETE DESIGN

Prestressed concrete girder bridges have been quite economical for spans between 45 ft and 145 ft. An advantage to these bridges is that they are adaptable to most geometric conditions. Prestressed concrete girder bridges are by far the most common bridge type used by the NMDOT. Also, many software packages are available for the design of prestressed girders, and most girder details are standardized.

A disadvantage to prestressed girder bridges is that they require a relatively deep section. The superstructure depth (the girder depth plus the deck slab thickness) of these bridges ranges between 3.0 ft to 6.75 ft. Also, long girders tend to be sensitive to handling stresses.

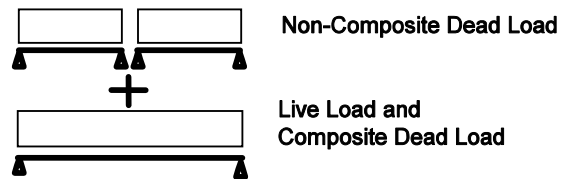
When developing the girder layout, consider the following NMDOT preferences. The NMDOT Bridge Bureau prefers girder spacing to be limited to 9 feet where possible. For ease and stability during deck construction, an odd number of girders is preferred. This also helps to accommodate future deck replacements where phased construction is required and traffic is to be maintained on one half of the structure.

#### 5.1 PRESTRESSED CONCRETE DESIGN PROCEDURE

When designing the prestressed concrete girders, the loading should be applied as discussed in this section.

All new bridges should be designed with the HL-93 truck load specified in the AASHTO LRFD Specifications and the NM P327-13 permit load. Exceptions for the NM P327-13 permit load must be approved by the State Bridge Engineer.

The girders will be subjected to non-composite dead load before the bridge is made continuous, so non-composite dead load shall be applied to simple spans. Composite dead load and live load should be applied to the composite superstructure section (continuous spans).



When calculating the composite cross-section, zero haunch should be assumed. However, additional non-composite dead load should be included equal to the weight of 2" nominal haunch.

For additional information, an example prestressed concrete girder design is available on the NMDOT Bridge Bureau website ([dot.state.nm.us](http://dot.state.nm.us)).

After the design of the superstructure is completed, the design must be load rated as per guidance in the NMDOT Bridge Rating Guide for new bridges.

#### 5.2 MATERIAL STRENGTHS AND PROPERTIES

The material strengths and properties discussed below are typical of NMDOT practice.

##### 5.2.1 Girder Concrete

Historically, NMDOT has designed prestressed concrete girders using 28 day compressive strengths in the range of 6,000 psi or less and release strengths of less than 5,000 psi. However, current practice is to use

high strength concrete with ultimate and release strengths much higher than these.

Interviews with local prestressing plants indicate that 28 day compressive strengths up to 9,500 psi are readily achievable. It has been observed that the design strengths achieved are regularly about 9,500 psi even when a lower strength is specified. This can affect camber and creep calculations. Therefore, it is preferred that all girder designs be completed using a final compressive strength of 9,500 psi, even when lower strengths could be used in design. The strength should be specified on the bridge plans.

For release strength ( $f'_{ci}$ ), also specify the strength needed. Round this value up to the next 100 psi.

### 5.2.2 Prestressing Steel

Uncoated seven-wire steel strands are by far the most common type of prestressing steel used. There are two types of prestressing strands: they can be either low-relaxation strands or stress-relieved strands. Similarly, there are two grades of strands that are commonly available: Grade 250 or Grade 270, with ultimate strengths of 250,000 psi and 270,000 psi respectively. Most current designs call for Grade 270 low-relaxation strands, and this is regarded as the current standard for design. The seven-wire strands must conform to the requirements of AASHTO M 203.

There are two strand sizes of prestressing strands used by NMDOT; the ½ in diameter strand and the 0.60 inch diameter strand. 0.60

inch diameter strands should be used for new prestressed girder bridges.

As per AASHTO LRFD 5.9.2.2, the allowable pretensioning stress immediately prior to transfer is  $0.75 \cdot f_{pu}$ , where  $f_{pu}$  is the ultimate stress of the prestressing steel. Slight overstressing up to  $0.8 \cdot f_{pu}$  is allowed to offset seating losses. Therefore, the initial prestressing force for the ½ inch strand is 30,975 lb/strand, with slight overstressing up to 33,000 lb/strand. For the 0.60 inch strand, the initial prestressing force is 43,950 lb/strand with a slight overstressing of up to 46,900 lb/strand allowed.

While many of the strand sizes are not normally used, Table 5.1A shows the design properties for various strand diameters.

### 5.2.3 Conventional Reinforcing Steel (Non-Prestressed)

Stirrups, shear reinforcement, tension reinforcement, and bars projecting from the top of the girder have traditionally been Grade 40 rebar. However, Grade 60 rebar is now preferred and Grade 40 rebar is reserved for bars that require bending in the field.

Reinforcing bars projecting above the tops of the girders and all reinforcing bars located within 4 feet of girder ends shall be uncoated corrosion resistant bars. Projecting bars must be long enough to extend into the bottom mat of deck reinforcement taking into account camber at erection and dead load deflections.

**Table 5.1A - Strand Data**

Nominal Diameter in	Nominal Area in <sup>2</sup>	Nominal Weight of Strand lb/1000 ft	Ultimate Strength, $f_{pu}$ (lb/strand)
Grade 250			
7/16	0.108	367	27,000
1/2	0.144	490	36,000
0.60	0.216	1737	54,000
Grade 270			
3/8	0.085	290	23,000
7/16	0.115	390	31,000
1/2	0.153	520	41,300
1/2 special	0.167	550	45,000
0.60	0.217	740	58,600

### 5.3 DETAILING REQUIREMENTS

The NMDOT Bridge Bureau has standard drawings for prestressed concrete girders. These are available in electronic format from the NMDOT Bridge Bureau website. A check list to be used in completing the drawings for project specific applications is contained in Section 2.3.4 Superstructure Details, Item No. 6. Standard template drawings have been prepared for the following girder types:

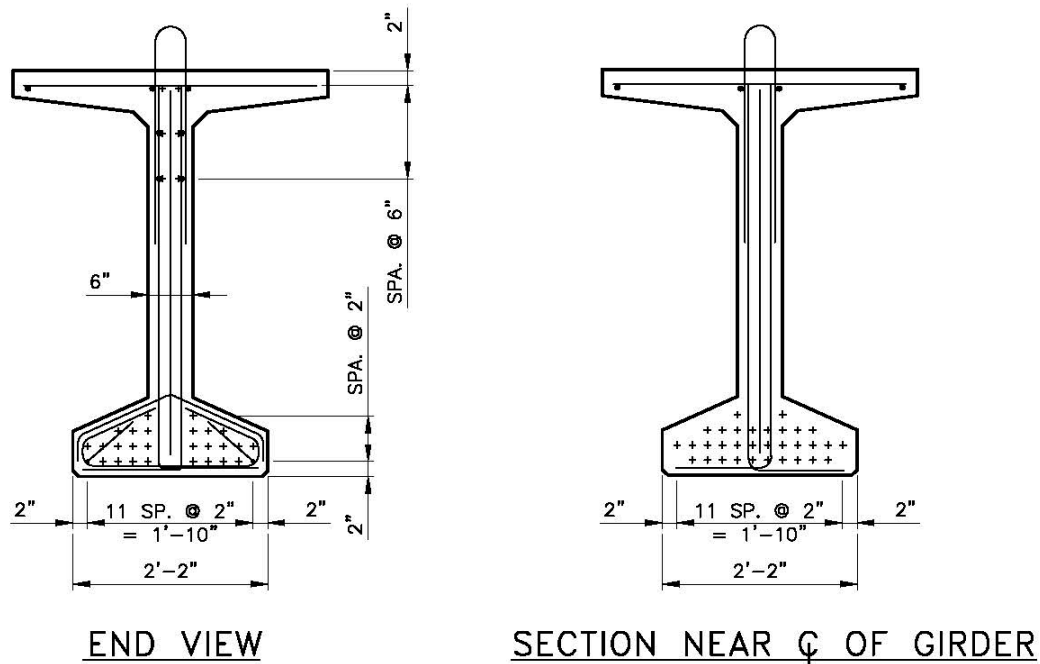
- Type 36
- Type 45
- Type 54
- Type 63
- Type 63 Modified
- Type 72 Modified
- BT-54
- BT-63
- BT-72

While standard drawings contain shear reinforcing details, the contractor may submit alternate details to the State Bridge Engineer for approval. Alternate details will be considered only if accompanied by complete shop drawings for the girders. No adjustment in payment will be made if alternate shear reinforcing details are approved.

#### 5.3.1 End Zone Cracking

It has been observed that some of the standard prestressed girders with thinner web sections have experienced unusual cracking patterns at the ends of the girders. To alleviate this issue, girders with webs 6" thick or less with draped strands should be detailed as shown in Figure 5.1. The draped strands shall be spaced at a minimum of 6" at the ends of the girders if possible.

Another problem that typically occurs at the ends of larger girders with thin top flanges is cracking and spalling of these flanges caused by haul chains. This type of damage can be prevented by using haul chain ports that pass through the top flanges near the girder web. Details for these ports may be found on the NMDOT website. Note that haul chain ports through the girder web are NOT permitted.

**Figure 5.1 – Draped Strands in Thin-Webbed Girders****5.3.2 Adjacent Prestressed Box Girders**

Adjacent Prestressed Box Girder Bridges are constructed by placing prestressed box girders next to each other and tying them together in such a way that they act as a unit and deflect equally under live loads. There is typically a shear key filled with grout between the girders and the girders are usually tied together using post-tensioned rods. Sample drawings for Adjacent Prestressed Box Girders are available from the NMDOT Bridge Bureau.

To prevent maintenance and safety problems with adjacent prestressed box girder and adjacent slab bridges, there are a few guidelines that should be followed. The Prestressed Concrete Institute (PCI) Bridge Design Manual is a good source of example details and calculations. As discussed in the PCI manual, the girders should be post-tensioned two at a time. (See Figure 5.2) The first two adjacent girders are placed and the tie rods are installed. As additional girders are placed, they are tied to the adjacent girder. All keyways and post-tensioning ducts should be grouted with an

approved high-strength grout after post-tensioning is completed. For girders greater than 36" in height, two post-tension rods should be used vertically at each tie location (See Figure 5.3). Additional prestressed concrete box details are available from the NMDOT Bridge Bureau.

It has been observed in New Mexico as well as several other states that non-composite adjacent box girder and adjacent slab bridges have not performed well. These bridges were topped by asphalt, had a thin concrete non-composite topping, or used the top of the girder flanges as the wearing surface. Failed shear keys can allow uneven distribution of wheel loads and the overstressing of girders. This can cause cracking in the girders, which allows water to migrate to the prestressing strands. In some states, girders have failed or collapsed due to broken strands caused by corrosion. To ensure composite action, precast adjacent box girder and precast slab bridges shall have a minimum deck thickness of 5" with at least one mat of reinforcing in the deck.

Figure 5.2 – Typical Post-Tensioning Layout (Skewed Bridge)

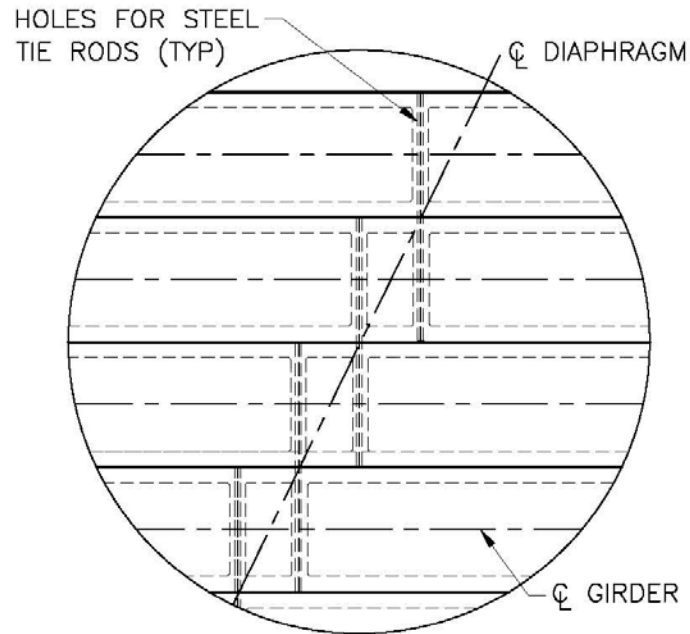
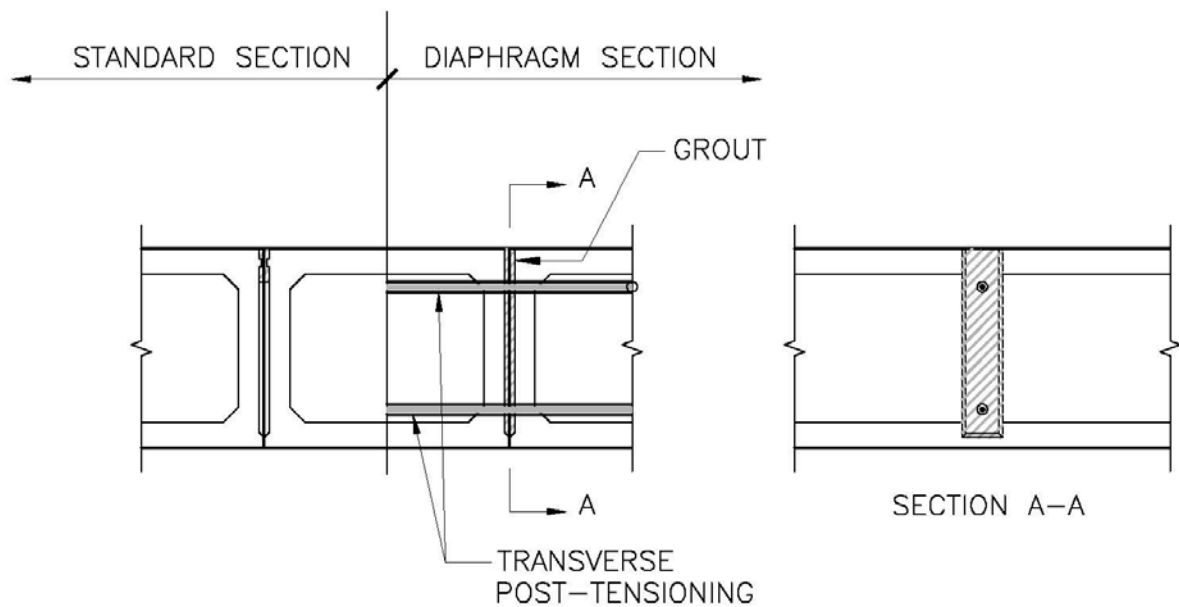


Figure 5.3 – Typical Post-Tensioning Layout (Girders with Height > 36")



#### 5.4 HOLD DOWNS, LOSSES, AND OTHER STRAND DATA

Designers should limit the vertical forces at the hold downs to a value of 5,000 lb per strand and 40,000 lb per hold down. This is a request from local prestressing plants since dangerous failures have occurred with higher vertical forces. Therefore limiting the force at the hold down to the listed value is important. The vertical force at the hold down can be reduced by lowering the location of the draped strands at the ends of the girders, provided that initial stresses at the girder ends remain within allowable limits. If multiple hold downs are required, they should be spaced at 8'-0" O.C. minimum spacing. Also, the Department has allowed debonding of prestressing strands, but this should be minimized as much as practical.

In calculating prestressing strand losses, use the method prescribed in the AASHTO LRFD Bridge Design Specifications. The hours to release is generally 15 to 18 hours. For design purposes, use 15 hours. For calculating shrinkage losses in new girders use a value for relative humidity of 25%. A value of 50% may however be used when evaluating existing girders for possible reuse.

#### 5.5 CAMBER GROWTH AND HAUNCH

The haunch is a small layer of concrete between the girder and the concrete slab. Its purpose is to allow for variations in the profile of the girder and for differences in girder camber. Differences between the actual girder camber and the calculated camber are taken up by the deck haunch. It is important to note that the haunch varies along the length of the span. In most cases the minimum design haunch is maintained as a minimum at the center of the span. The haunch dimension at the centerline of bearing should be used in the determination of girder seat elevation.

Figures 5.4, 5.5, and 5.6 illustrate how the haunch at the girder ends is computed.

On several projects, the actual camber in prestressed concrete girders substantially exceeded the calculated value shown on the plans. This excess camber occurred where BT-54 girders and larger 63 inch and 72 inch girders were used. The excess camber can, particularly on phase constructed bridges and on widening projects, create problems with maintaining an adequate deck slab thickness over the girders.

To try to alleviate this type of problem, the Department has begun implementing the following procedures.

The camber value to be used in design and to be reported on the plans is the camber at the time of erection. This is the calculated value due to the effect of prestressing and the self-weight of the girder, with an allowance for camber growth up to 90 days after release of prestressing. It is customary to apply multipliers to the initial girder camber to estimate the camber of a member at erection.

The dead load deflection values to be reported on the girder drawings and upon which the screed setting diagrams are to be based, is the sum of the deflections caused by all composite and non-composite dead loads, exclusive of deflections caused by the girder self-weight, at the time of erection (90 days after release of prestressing).



Figure 5.4  
Haunch for Bridge on Tangent

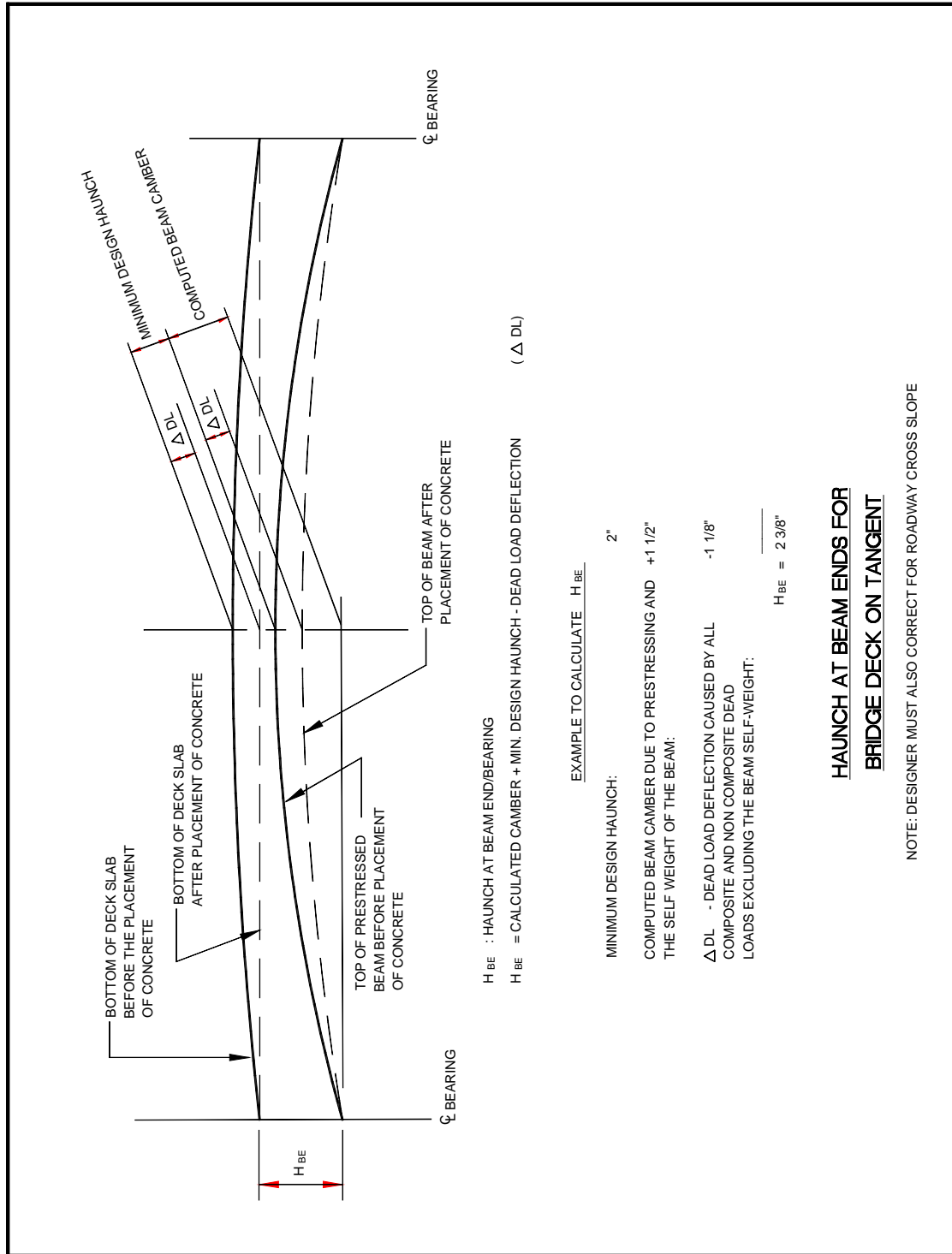


Figure 5.5  
Haunch for Bridge on Vertical Crest Curve

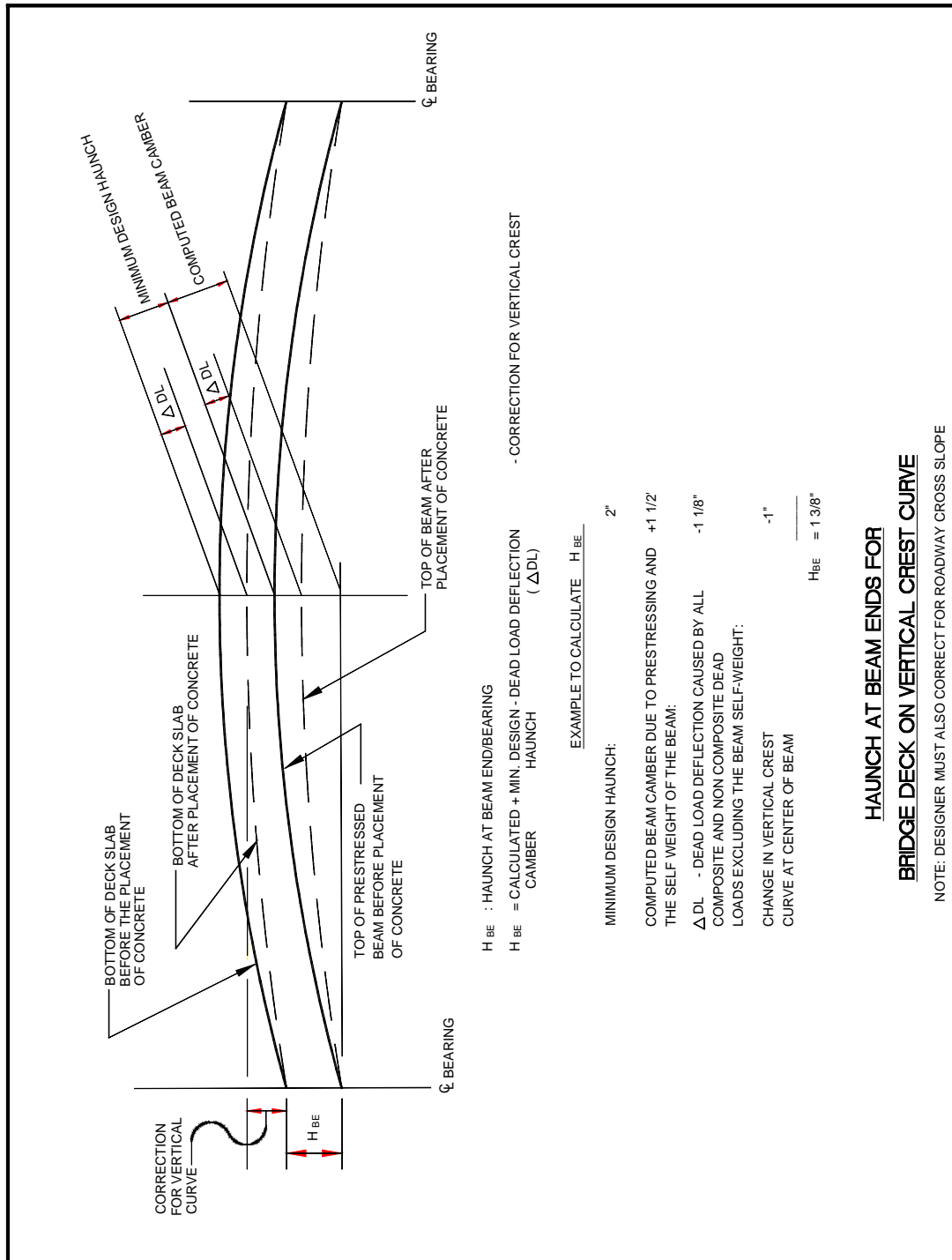
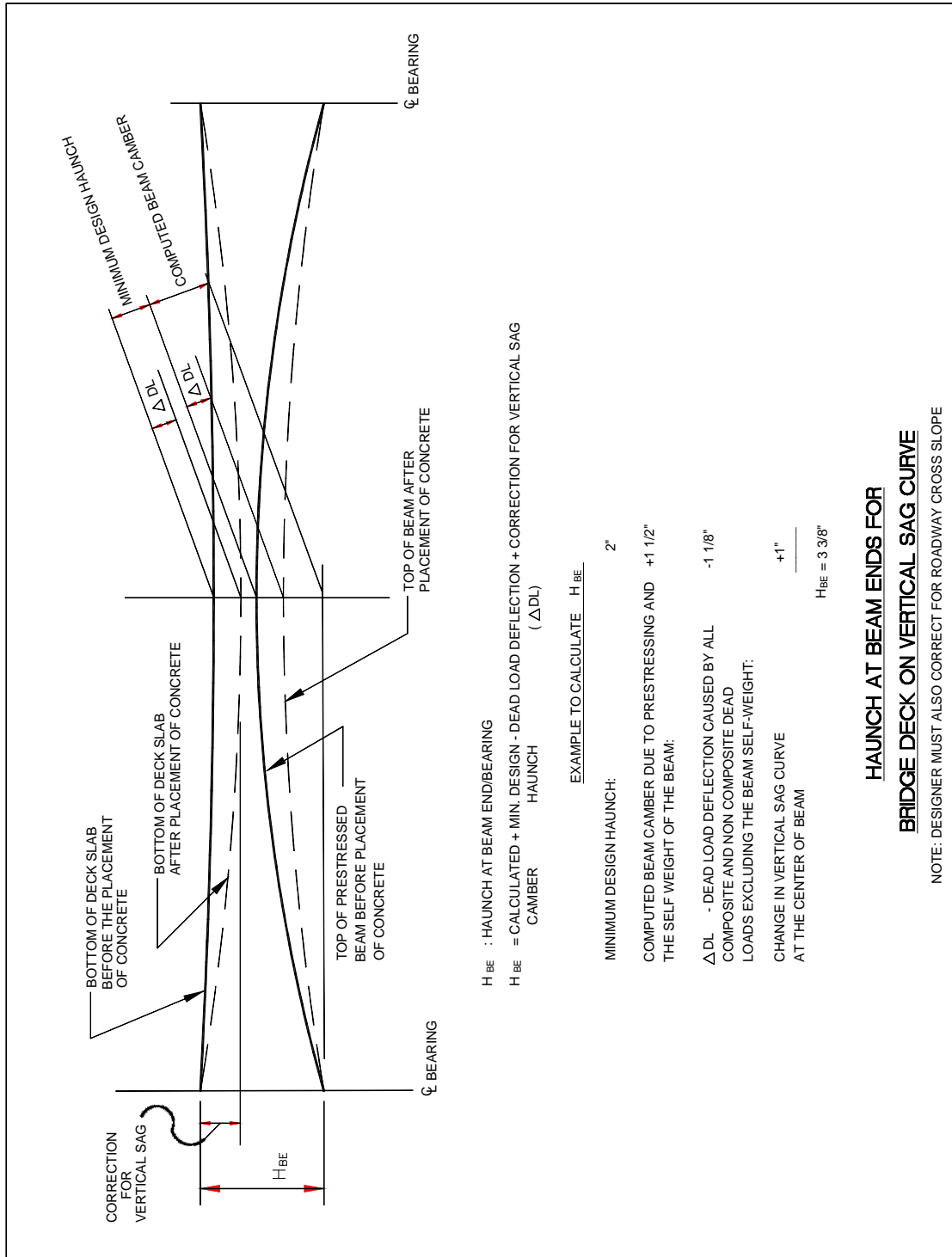


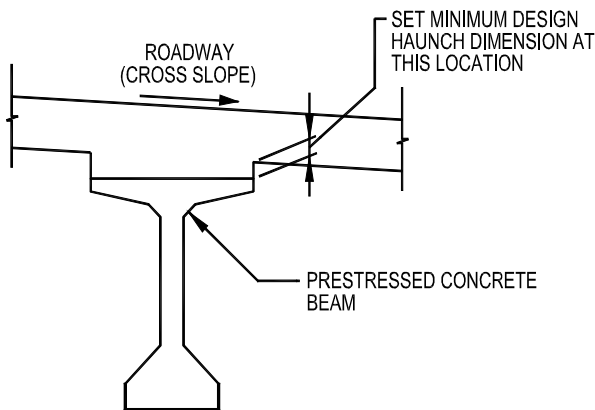
Figure 5.6  
Haunch for Bridge on Vertical Sag Curve



For girders less than 54 inches the minimum design haunch is 1 inch. The minimum design haunch for 54 inch, 63 inch and 72 inch girders is 2 inches. This minimum haunch dimension is set at the edge of the top flange where the deck slab grade is the lowest (as illustrated in Figure 5.7). The design haunches can be increased in special cases such as girder length exceeding their typical design length.

The contractor limits the camber growth to a value not to exceed the predicted camber by 1 inch at the time of deck slab placement. Camber growth is limited by weighting, fabrication scheduling or other approved means.

No adjustments in girder camber are to be made for predicted long-term deflections.



**Figure 5.7**  
**Slab Detail at Girder Camber**

## 5.6 REUSE OF EXISTING PRESTRESSED CONCRETE GIRDERS

Prestressed concrete girders have been used in bridge construction in New Mexico since the early 1960's. In many of the older bridges of this type, the deck slab and concrete substructure elements have become deteriorated because of the corrosive action of deicing salt on the reinforcing steel. The prestressed concrete girders in them are, however, in many cases still sound. During

the rehabilitation of such bridges the prestressed girders are usually salvaged and reused but the deck slab is replaced. Some common design issues that are encountered when designing for this situation are:

1. Having been designed for thinner slabs than are in use today, the girders in some cases have insufficient strength to carry the additional dead load of the thicker slab.
2. The older bridges also were almost always designed as simple spans with joints in the deck slab over each bent. These joints need to be eliminated.
3. Concrete at the girder ends may be deteriorated.

Suggested methods of addressing these issues are presented below.

### 5.6.1 Insufficient Strength

Increasing the strength of an individual prestressed girder is for the most part neither practical nor cost effective. There are however some simple methods of getting around the problem. These include:

- Not designing the rehabilitated bridge to carry the weight of a future overlay.
- Specifying that the corrugations in the bottom of the stay-in-place forms be filled with extruded polystyrene. The 15 psf allowing for the use of the SIP forms can then be reduced to 3 psf. For girder spacing greater than 10 feet, the designer will need to calculate the design load for the stay-in-place forms plus extruded polystyrene. Also see Section 4.3.2 regarding placing extruded polystyrene in form corrugations.
- Adding one or more additional girder lines.
- Using a value for relative humidity of 50% rather than 25% when calculating losses due to shrinkage.

### 5.6.2 Eliminating Deck Joints

Experience has proven over and over again that deck joints in bridges cause problems. Attempts to seal them are usually not completely effective

and salt contaminated water leaking through them causes deterioration in abutments, piers, bearings and girder ends. It has been evident to the NMDOT Bridge Bureau for many years that completely eliminating the joints is the most desirable solution.

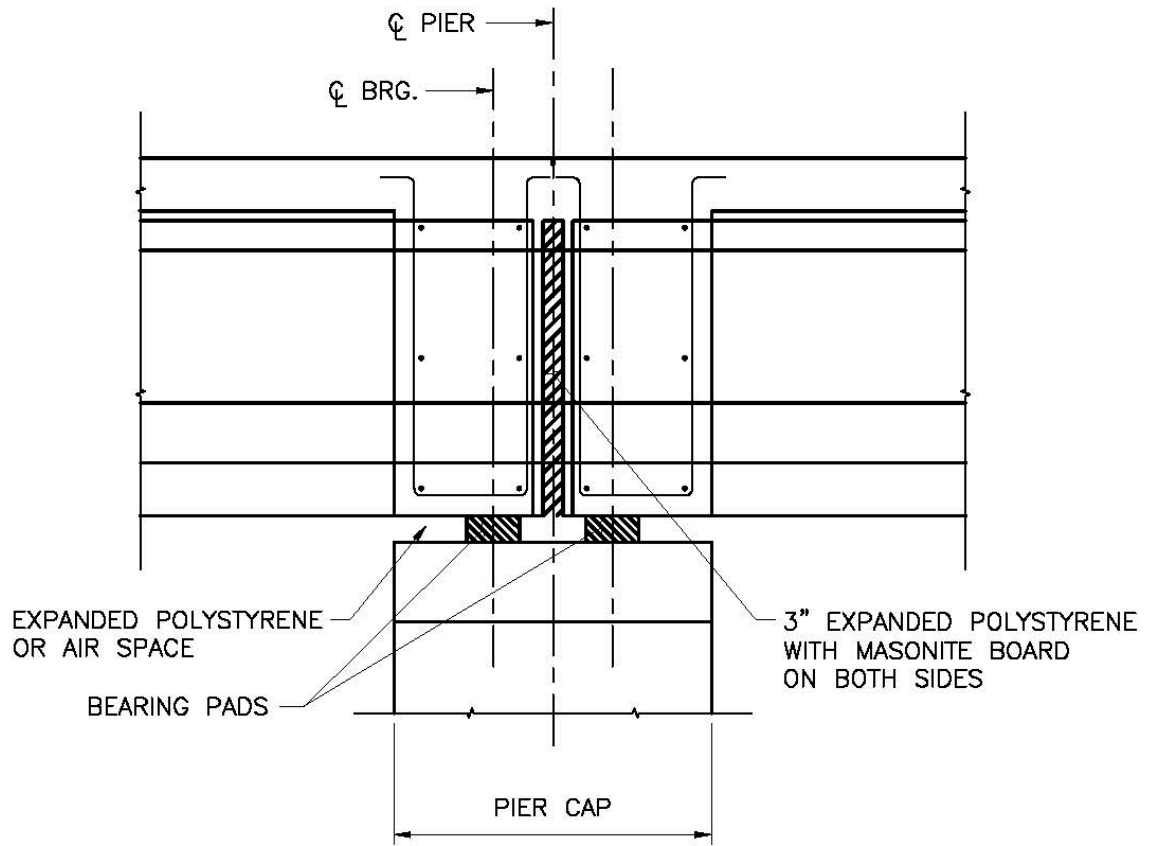
Early attempts at joint elimination in rehabilitated prestressed concrete bridges consisted of converting abutments to integral designs and, at piers, converting to a continuous for live load design. The integral abutment approach has been successful and should continue. There have however been some problems with the continuous for live load conversion. Making the conversion significantly increases shear forces at the piers and shear cracking in some bridges treated in this manner has been reported.

A preferable solution to the full continuous for live load conversion is to make the deck slab continuous but allow the girders to continue to function as simple spans. A detail that will permit this has been successfully used is shown in Figure 5.8.

### **5.6.3 Deteriorated Girder Ends**

Leaking joints cause deterioration. The treatments discussed in 5.6.2 will both stop the corrosion and serve as a repair. Converting to an integral abutment design and making the deck slab continuous over the piers have eliminated the source of the problem. Also, the process of casting the concrete diaphragms encapsulates the girder ends and stabilizes them. The only additional process that is needed is to remove delaminated concrete and clean concrete and any exposed rebar prior to casting the diaphragms.

**Figure 5.8**  
**Preferred Method of Eliminating Pier Deck Joints**  
**In Existing Prestressed Girder Bridges**



**SECTION THRU PIER DIAPHRAGM**

## CHAPTER SIX

### STRUCTURAL STEEL

In steel girder bridges, girders are usually rolled beams, continuous welded plate girders with a constant depth, or continuous welded plate girders with a variable depth. Steel girders are normally used for spans over 145 feet. The superstructure depth (including the deck slab) ranges from 3 feet to 10 feet. Steel bridges tend to be lighter than concrete girder bridges.

#### 6.1 STRUCTURAL STEEL DESIGN

Design of the structural steel girders should be according to this section. All new bridges should be designed with HL-93 truck load specified in the AASHTO LRFD Specifications and the NM P327-13 permit load as outlined in Chapter 3 of this design guide.

A comprehensive handbook released by the FHWA is available to be used as a guide during the design of steel bridges. The Steel Bridge Design Handbook, FHWA Publication No. FHWA-IF-16-002, is available at no cost on the AISC website ([www.aisc.org](http://www.aisc.org)).

The handbook leaves certain design decisions up to the engineer's judgment and the bridge owner's preference. Contact the NMDOT Bridge Bureau if clarification on particular design elements is required.

When developing the framing plan, consider the following NMDOT preferences. The NMDOT Bridge Bureau prefers steel girder spacing to be limited to 9 feet where possible. For ease and stability during deck construction, an odd number of girders is preferred. This also helps to accommodate future deck replacement where phased construction is required and traffic is to be maintained on one half of the structure.

For negative moment regions, the effective moment of inertia should be calculated neglecting the concrete in the deck. Only the steel girder and the deck reinforcing should be used for the composite section in those regions.

In the design of a steel bridge, field bolting is preferred. Field welding should be minimized. If field welding is required, Field Welding Notes should be included on the plans.

After the design of the superstructure is completed, the design must be load rated as per guidance in the NMDOT Bridge Rating Guide for new bridges.

#### 6.1.1 MATERIAL STRENGTHS AND PROPERTIES

AASHTO M 270 Grade 50 steel is typically used for all main load carrying girder components including the girder web, the top and bottom flanges and splice connection plates. Additionally, Grade 70 steel may be considered for use in these components if it seems advantageous. AASHTO M 270 Grade 36 is still sometimes used for stiffeners, diaphragms, and cross frame girder connector plates. Sole plates and other minor components of steel or concrete bridges must conform to the requirements of ASTM A36.

Weathering steel is a material that should be considered for steel structures in New Mexico. Before shipping, weathering steel should be sand blasted to SSPC SP 6 requirements with all shop markings removed, wet down, and then dried to provide a clean steel with a protective surface. When painted steel is used, it is acceptable to use paint in conjunction with weathering steel as a back-up in case the coating system fails if it does not significantly impact cost, but it is not required.

Diaphragm bolts, splice connection bolts, cross frame bolts, tension plate bolts, and bolts for all other major components of steel bridges shall be high strength bolts conforming to the requirements of AASHTO M 164 (equivalent to ASTM A 325). Anchor bolts / anchor rods must conform to AASHTO M 183 (ASTM A36) or ASTM F1554 grade 36, 55, or 105. For ASTM F1554 the weldability Supplement S1 is recommended as inexpensive insurance for a more flexible solution should the anchor bolt / anchor rod is placed incorrectly in the field. Shear connector studs must conform to AASHTO M 169.

## 6.2 DETAILING REQUIREMENTS

The steel girder detailing requirements are listed in Section 2.3.4. Many of the problems encountered with steel bridges are fatigue related. The most common problem is fatigue related cracking that occurs along the welds of secondary members (stiffeners, diaphragms, and lateral bracing) to the main girder. This is also known as “out of plane” bending. Cracks in the welds begin at corners and propagate along the weld. To minimize these issues, *Guidelines for Design Details*, an AASHTO/NSBA Steel Bridge Collaboration document, should be utilized during the detailing of the girders. This document is available for free on the AISC website ([www.aisc.org](http://www.aisc.org)).

The use of stiffeners should be carefully evaluated. It is often desirable to accept higher material costs to avoid additional fabrication costs and reduce the risk of future fatigue issues. In general, the use of longitudinal stiffeners should be avoided.

## 6.3 RECOATING STEEL ELEMENTS ON EXISTING BRIDGES

Prior to 1986 structural steel elements on NMDOT bridges were painted with a system which included a lead based primer. If

adequate precautions are not observed, removal of this paint can cause severe health problems for workers. Additionally, the paint residue is harmful to most organisms if introduced into the environment. It must therefore be collected and disposed of in a safe manner. This is both problematic and expensive.

For these reasons, NMDOT recoating specifications (Section 546 of The Standard Specifications For Highway And Bridge Construction) utilizes a procedure whereby only non-adherent paint is removed. Adherent paint is left in place and encapsulated by the new coating system. Proper application of this system requires that certain items be included in the plans for bridges that are to be recoated. These items are outlined in the following section.

### 6.3.1 Plan Requirements for Bridges to be Recoated

For bridges that are to be recoated the following items need to be included in the plans:

1. Include the lump sum pay item “Recoating Structures” in the listing of Estimated Quantities
2. Include pay items and quantity estimates for areas to be cleaned to SSPC-SP3 and SSPC-SP11 requirements.
3. Include the lump sum pay item “Safety and Environmental Requirements” in the listing of estimated quantities.

Automatically assume the presence of red lead paint for all bridges built prior to 1986. For bridges built between 1986 and 1990 the designer will have to ascertain whether or not the paint contains lead by researching as-built plans, or if this fails, arranging to have the paint analyzed in a laboratory.



## **CHAPTER SEVEN**

### **BEARING DEVICES AND GIRDER ANCHORAGES**

#### **7.1 GENERAL**

Conventional Bridges supported by abutments and piers require bearing devices that will transfer the girder reactions to the substructure elements without over-stressing them. Bridge bearings generally are more critical and more elaborate than those needed for buildings. The forces created by temperature changes, elastic deformation, shrinkage and creep are usually more acute than for buildings and need additional attention if they are to be properly addressed.

Because of total exposure to weather, bridges will often experience more frequent and greater expansion and contraction movements than most other structures. Therefore, bridge bearings should be designed as maintenance free as possible. In addition to bridge dead loads, bearing devices must be capable of withstanding and transferring dynamic live load forces and resulting vibrations that may be transmitted to and through them. Lack of attention to bearing devices and details may result in premature wear and/or eventual substructure failure.

Bridge bearings are of two general types, expansion and fixed. Expansion bearings provide for rotational movements of the girders as well as longitudinal movement for expansion and contraction of bridge spans. Corrosion in an expansion bearing may cause friction, which may interfere with expansion and contraction of the span. This corrosion and the resulting friction forces may lead to future maintenance problems.

The function of the fixed bearing is to prevent the superstructure from moving longitudinally on the substructure elements. The fixed bearing acts as a hinge by permitting rotational movement while at the same time preventing

longitudinal movement. Both expansion and fixed bearings transfer lateral forces such as wind and centrifugal loading from the superstructure to the substructure. Both bearing types are set parallel to the direction of structural movement.

#### **7.2 ELASTOMERIC BEARINGS**

Elastomeric bearing pads are by far the most commonly used bridge bearing device in New Mexico. Bearing pad material and testing shall conform to the requirements of the current AASHTO LRFD Bridge Construction Specification Section 18.2.

Elastomeric bearings are fabricated as plain bearing pads or as laminated (reinforced) bearing pads. Plain bearing pads consist of the elastomer only. Laminated (reinforced) bearing pads consist of alternating bonding layers of elastomer and steel or fabric reinforcement. These bearings are designed to transmit loads and accommodate movements between a bridge and its supporting structure. Performance information indicates that elastomeric bearings are functional and reliable when designed within the structural limits of the material.

Plain bearing pads must be 3/4 inch or less in thickness. Pads thicker than 3/4 inch must be reinforced with laminates. Each laminate layer should have a spacing of 3/4 inch or less throughout its entire thickness. AASHTO Bridge Specifications do not permit tapered elastomer layers in reinforced bearings. The Department requires that the thickness of the laminate steel reinforcement layers (sheet metal shims) be specified as 1/8 inch and conform to ASTM A1008 or A1011 Grade 36. The Department's preferred bearing pad details are shown in Figure 7.1.

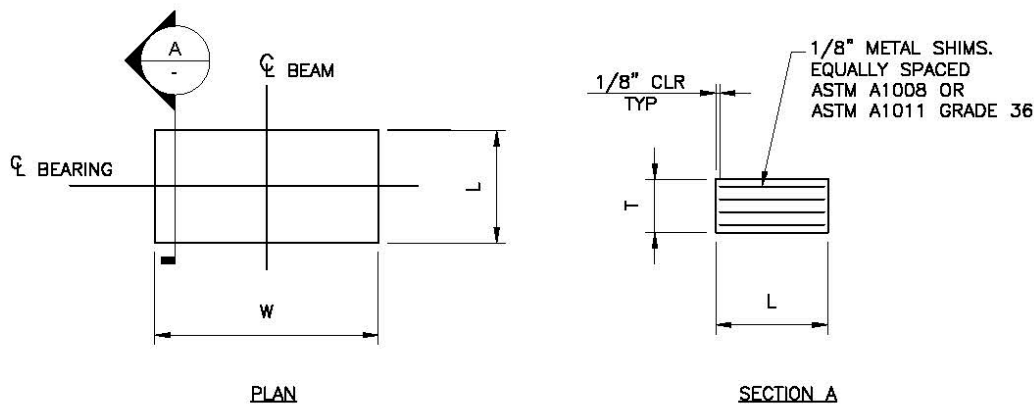
The range of thermal movement used in elastomeric bearing pad design shall be not less than 75 percent of the total anticipated movement due to temperature. Refer to Section 3.1.7.1 for specifics. Pads should also be designed for all other anticipated movement including creep, shrinkage and elastic shortening.

Laminated (reinforced) bearings can be designed according to the AASHTO LRFD

Specifications, Article 14.7.5, and 14.7.6 Method A or B. The Department prefers to use Method A as it is a conservative design and requires less testing. If Method B is used, the designer will need to specify the additional required testing of the bearing pads.

Laminated bearings must be placed on level bearing surfaces, or gravity loads will produce shear strain in the bearing due to inclined

Figure 7.1 - Bearing Pad Details



ELASTOMERIC BEARING PAD

DUROMETER NO. \_\_\_\_\_  
SHEAR MODULUS \_\_\_\_\_

ELASTOMERIC BEARING PAD SCHEDULE								
LOCATION	DIMENSIONS			NO. SHIMS	DESIGN LOAD DL	DESIGN LOAD LL	NO. REQ'D	MOVEMENT LENGTH
	W	L	T					
ABUT. 1								
PIER NO.								
PIER NO.								
ABUT. 2								

forces. The surface between the pad bottom and the concrete must be level and full contact between the pad and concrete must occur over the entire area.

The maximum stress associated with the angle between the alignment of the underside of the girder (due to the slope of the gradeline, the rotation of the girder due to loading, and camber) and the alignment of the bottom of the bearing must not exceed the allowable stress defined in the AASHTO LRFD Specifications Article 14.7.6.3.5a. The top sole plate may need to be tapered so that the rotation angle is not exceeded. The tapered sole plate must have a minimum thickness of  $\frac{3}{4}$  inch.

It is recommended that keeper bars be used to keep pads from “walking” out from under the girders at expansion bearings.

When anchor bolts / anchor rods are used to hold the sole plate and bearing pad in place, the anchor bolts / anchor rods are to be threaded along their full length. The nuts, anchor bolts, and washers are to be galvanized according to NMDOT Standard Specifications. Generally the use of anchor bolts / anchor rods should be avoided.

During the process of welding sole plates to the beams heat from the welding may damage the bearing pads. To avoid such damage, sole plates need to be thick enough so that there will be a distance of at least  $1\frac{1}{2}$  inches between the weld and the surface of the bearing pad closest to it. The distance is measured along the diagonal between the weld and bearing pad.

### 7.3 TFE BEARING SURFACES

TFE (polytetrafluoroethylene) is also known as PTFE or its trademark name, Teflon. Bearings with TFE surfaces are designed to translate or rotate by sliding a self-lubricating

polytetrafluoroethylene (TFE) surface across a smooth, hard, mating surface preferably of stainless steel or other equally corrosive-resistant material. Teflon expansion bearings are not used without providing for rotation. A layer of elastomer is provided to facilitate rotation due to live load deflection or change of camber. The Teflon sliding surface must be bonded to a rigid backing material capable of resisting horizontal shear and bending stresses to which the sliding surfaces may be subjected. Design and construction requirements for TFE bearing surfaces are given in the AASHTO LRFD Specifications.

An advantage of TFE is that it can be made into different shapes and forms. Its use as a bearing material is suited to many different types of expansion bearings. Many combinations of Teflon bearings and backing materials are commercially available. The AASHTO LRFD Specifications contain minimum coefficient of friction values to be used for the design of TFE bearing devices. These values vary with contact pressure and the type of PTFE surface being specified. See Table 14.7.2.4-1 and 14.7.2.5-1 in the LRFD Specification.

### 7.4 GIRDER ANCHORAGES

A design feature generally associated with bearings is the girder anchorage system. The intent of such a system is to prevent longitudinal, transverse, vertical or a combination of any of these movements at the girder ends. The traditional method of providing beam anchorages in New Mexico has been through the anchor bolt / anchor rod and sole plate method. Experience has proven that this design is prone to problems. There are several reasons for this. First, sole plates have, in many instances, not been correctly positioned during construction so that the slotted holes are not able to accommodate the total range of expected movement. Secondly, the gap T

between the sole plate and the beam seat causes the horizontal loads to be applied to the bolt at an eccentricity of  $T$  plus half of the sole plate thickness above the beam seat. This in turn creates a bending movement in the anchor bolt / anchor rod. Since the anchor bolts / anchor rods lack flexural capacity, in many cases they simply bend over, pull out of the concrete, or crack the concrete. More effective methods of anchorage therefore need to be used.

When possible, the preferred method of providing beam anchorage is through the use of fully integral or semi-integral pinned attachments to substructure units. Contact the NMDOT Bridge Bureau for preferred details.

In any case, it is important to provide blockouts to provide access for bearing inspection and potential future bearing repairs. Typical blockout details are shown in Figure 7.2.

## 7.5 DESIGNING FOR SUPERIMPOSED DEFORMATIONS

As was discussed in 7.1, one function of expansion bearings is to permit movements caused by temperature change, shrinkage and creep. The movements caused by these factors

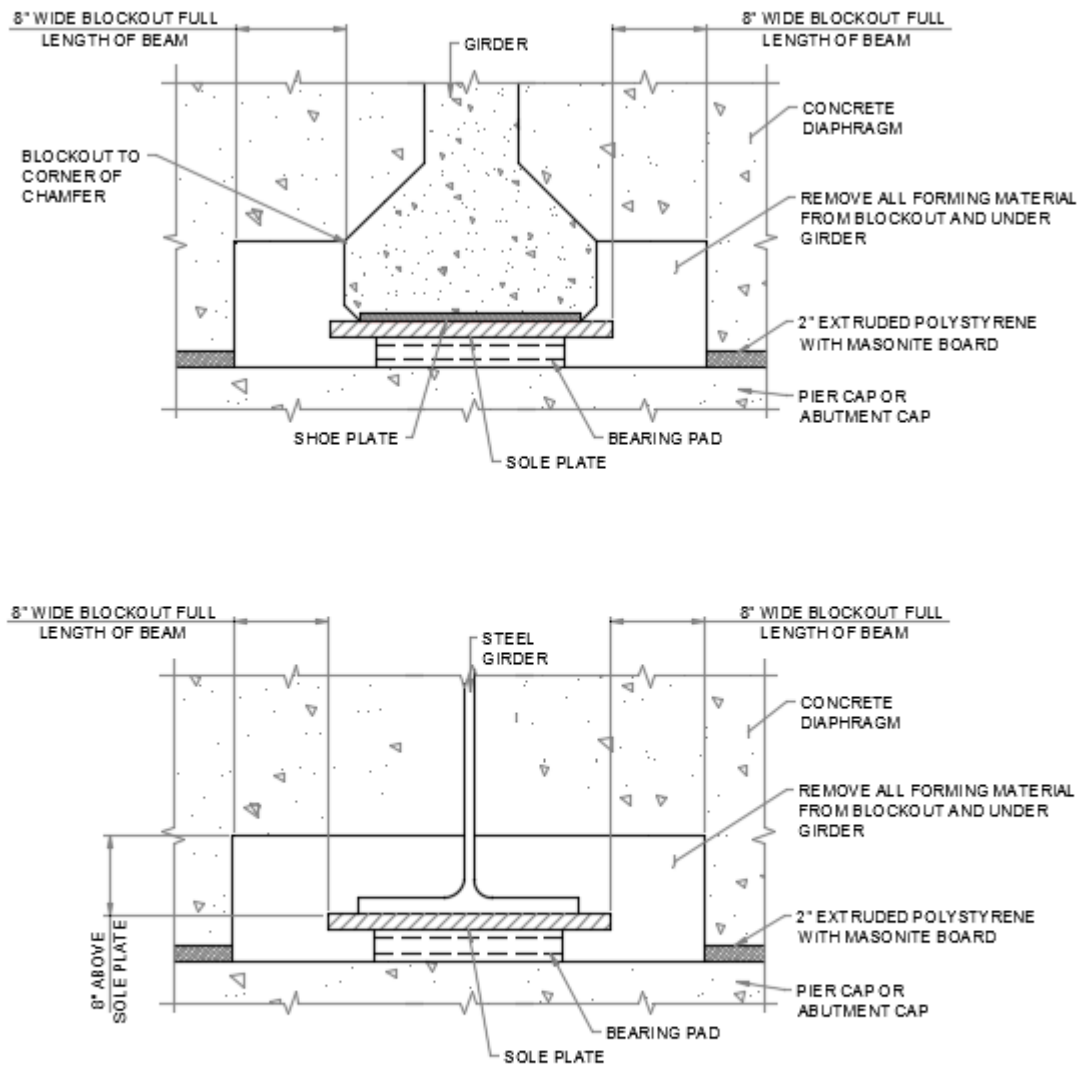
deform the pad and thus develop stresses in them. These stresses are transferred as forces into the substructure. If care is not taken in the selection and design of the bearings, the movement induced forces can have a profound effect on the design and cost of the substructure in new bridges - and can seriously overstress existing substructures in bridges that are being rehabilitated.

Some methods that can be use in reducing the movement induced stresses that are transferred into the substructure are:

- To increase the thickness of the pad to the extent that stability considerations will permit, or
- To use Teflon coated bearings (This method should be used only if a conventional pad that will meet code requirements cannot be designed).

There are without a doubt other solutions. The thing to remember is that forces due to the superimposed deformations can be huge. Careful attention needs to be paid to the design so that the forces are limited in magnitude.

### Figure 7.2 - Blockout Details





## CHAPTER EIGHT

### DECK JOINTS

#### 8.1 GENERAL

Bridge deck joints are designed to allow for expansion and contraction movements resulting from thermal, rotational, and other external forces. Bridge joints must be properly designed and installed to insure their integrity and serviceability. Bridge deck joints are designed to have long service lives with little or no maintenance.

Water on bridges creates problems. It causes deterioration in concrete and steel elements in all bridges. In addition, concrete in some New Mexico bridges is alkali-silica reactive. This reaction, which over time will cause complete disintegration of the concrete, is accelerated when water is present. Despite this need to keep water off structural elements, joint seals have for the most part proven to be problematic and leak over time. The number of joints used in a bridge should therefore be kept to an absolute minimum or, preferably, eliminated altogether if that is possible.

Where possible, expansion joints should be placed at the end of the approach slabs thus eliminating joints over the abutments and piers. On long bridges, expansion joints may be placed over piers or abutments but should be kept to a minimum. If only one joint is placed on the bridge, the joint is placed on a high point if practical. This is done to prevent the bridge from creeping downhill and to minimize the amount of water passing over the joint.

It is important that the designer consider drainage problems with seals at concrete barrier or metal railing ends. A pass-through design may be acceptable where there is little runoff or where minor erosion problems exist below the structure. If a pass through design is

used rundowns are generally recommended to take the water from the joint to the toe of the slope. Where water would create problems, turn-ups may be required.

#### 8.2 DECK JOINT TYPES

There are four separate approved systems for bridge joint seals. A brief description of each is provided in the following sections along with the Standard Drawing number. Note that each system is subject to Special Provisions and to provisions of NMDOT Standard Specifications.

Designers must detail joints on the plans and specify which joint systems are acceptable at each joint. Designers should also review end treatment requirements for the joints and provide scuppers and splash plates where necessary. All applicable NMDOT Standard Drawings and Special Provisions must be referenced or included in the contract plans.

Deck joint opening sizes (Dimension "A") are contained in Standard Drawings issued by NMDOT. Allowable movement lengths for spans on bridge structures are also given in tables on the same Standard Drawings. Structural movement lengths exceeding those given in the tables may require a special joint seal. When these unusual situations arise the State Bridge Engineer should be contacted for assistance.

A Joint Use Matrix is given in Table 8.1. This matrix lists applications where the various bridge joint seal systems may be used.

The most durable seal and the one most commonly used is the Type A Bridge Joint Strip Seal.

### 8.2.1 Bridge Joint Strip Seals

Bridge joint strip seal devices are molded neoprene or EPDM glands inserted and mechanically locked between armored interfaces of extruded steel sections. The name "strip seal" is derived from the strip profile of the neoprene or EPDM seal. During structure movements, a preformed central hinge enables the strip seal profile to fold between the seal extrusions. Strip seal design details are given in NMDOT Standard Drawings.

Of the five bridge joint seal systems described, strip seals (Type A and Type B) are considered to be the most durable; therefore, they are the preferred joint seal systems for major seals. Strip seals are used to keep water and debris out of the joint. Turn-ups, scuppers, or splash guards should be provided to divert water from reinforced earth walls or other erosion prone areas and structures.

**Table 8.1**  
**Joint Use Matrix**

SYSTEM	MOVEMENT	NEW CONSTRUCTION			EXISTING AND REHABILITATION		
		DECK JOINTS	APPROACH SLAB TO SLEEPER	SEALING FIXED (CLOSED) JOINTS	RECONSTRUCTION OF JOINTS (2)	REPLACE COMPRESSION JOINT SEALS	REPLACE BOLT-DOWN JOINTS
STRIP SEAL TYPE A	0-4 IN.	√	√		√		
STRIP SEAL TYPE B	0-4 IN.				√		
POLYMER SEAL	0	(3)	(3)	√	(3)	(3)	(3)
ASPHALTIC PLUG SEAL	± 1"						(1)
PREFORMED FOAM	(4)		√	√		√	√

**NOTES:**

- (1) Use only for bridge rehabilitation where there is a bituminous overlay.
- (2) Reconstruction of joints means, for example, removal of a deteriorated backwall and part of a deck, and rebuilding the backwall and deck with a new seal.
- (3) The use of this joint is discouraged.
- (4) Reference manufacturer's data



The strip seal system has Type "A" and Type "B" installations. The Type "A" should be used on a new structure or on an older structure where backwalls need replacement. Type "B" installations are primarily intended for rehabilitation projects where bolt-down expansion joints are being replaced.

Strip seals are the most common joints used by the NMDOT. This system has proven to require little or no maintenance and they generally remain watertight.

### 8.2.2 Polymer Bridge Joint Seals

In most installations, the NMDOT has had poor results using Polymer Bridge joint seals, and the use of the joint seal is discouraged.

This system inserts a backer rod into the joint. The backer rod should be at least 25% larger than the joint, or of a size recommended by the manufacturer for that particular joint size. The joint is then sealed with a silicone polymer or a polyurethane sealing element, the top surface of which is recessed ½ inch below the new bridge deck surface.

Polymer Bridge Joint Seals may be used on either new or rehabilitation projects. On rehabilitation projects, they have a wide range of use, but are particularly suitable for replacement of compression joint seals. On new projects, the recommended use is only for closed (fixed) joint applications. However, polymer joint seals should not be used for major seal installations, except with approval of the District or State Bridge Engineer. Their use should be restricted to minor seals. Examples of these applications would be joints between bridge decks and approach slabs, or wingwalls and approach slabs.

### 8.2.3 Asphalt Plug Bridge Joint Seals

The NMDOT has had mixed results with this type of seal. They are therefore not often called for.

This joint seal system consists of a backer rod in the joint gap, a closure plate of steel or aluminum over the gap, and a polymer-modified asphaltic binder to use as a primer. When mixed with aggregate, the asphaltic binder also serves as an elastomeric material for filling the joint cavity. The System 4 joint seal can only accommodate relatively small movements of plus (tension) or minus (compression) 1 inch. This type of joint seal system may be used only in bridge rehabilitations where there are bituminous overlays. The joint should primarily be used for replacement of bolt-down joints.

The materials for this joint system must be made of components from the same manufacturer. This system needs correct fabrication for the asphalt plug joint to have good rideability.

### 8.2.4 Preformed Foam Bridge Joint Seal Systems

Preformed foam bridge joint seal systems are used in situations where a small movement is expected. These seal systems were adopted to replace the polymer bridge joint seal. These seals shall be installed in accordance with the manufacturer's recommendations.

The first preformed bridge joint seal adopted by NMDOT was the preformed closed-cell foam seal (often referred to by the trade name "evazote"). A related system, the preformed silicone-coated foam joint system, has been recently adopted as an improvement over the preformed closed-cell foam seals. Details for the installation of preformed foam bridge joint systems are found in NMDOT Standard Drawings Section 564.



## **CHAPTER NINE**

### **SUBSTRUCTURE DESIGN**

#### **9.1 SUBSTRUCTURE TYPES**

The foundation type is usually based on the recommendation from the NMDOT Foundation Engineer. The type of foundation generally depends on the soil loading or waterway conditions. Early in the development of a bridge, geotechnical information may not be available and the project foundation engineer may have to rely on existing soils information from a nearby structure to determine the most practical foundation type.

Typical Foundation types are:

1. Drilled Shafts (wet or dry conditions)
2. Steel H Piling
3. Pipe Piling (closed or open end)
4. Spread Footing

Field splices for steel H piles, plate dimensions, and weld symbols are shown in the attached Figure 9.1. Permissible field splices for pipe piles are shown in Figure 9.2. Details of the splices, plates, and sizes of welds should be shown on the plans.

The type of bridge bearing selected can have large force impacts on substructure when stiff substructures are used. It is very important to select a bearing/substructure system that is functional and economical. Sometimes the process can become quite iterative.

#### **9.2 PIERS**

Piers can be multiple or single column bents. If the structure is not too wide, a single column bent may be used to reduce the clutter underneath the bridge. Single columns usually rest on a pile cap footing or drilled shaft.

The bents of many structures have multiple round columns with a rectangular pier cap. The columns usually rest on drilled shafts or a pile cap footing. For very short bents on stream crossings, a line of piling may be extended into the pier cap and encased in concrete to form a curtain wall.

#### **9.3 ABUTMENTS**

There are two basic types of abutments, open end and closed end (or retaining). Each has several subtypes. Open end abutments are located near the top of the approaching roadway embankment. Closed end abutments retain the soil so that an embankment does not exist under the bridge. The type of abutment used is based on economic considerations.

Open end abutments may be backwall, integral and semi-integral, or spill-through type as shown in Figure 9.3. The backwall type is generally supported on piles or drilled shafts which extend through the embankment. The spill-through abutment is a common type used in New Mexico. The fill extends from the bottom of the cap beam and is allowed to spill through the open spaces between the columns.

Closed end abutments can be mechanically stabilized earth (MSE), double T, or conventionally reinforced retaining walls. The MSE wall is not a true closed type abutment because it is not load bearing and is used with an open type abutment. The recommended minimum offsets required for an MSE abutment are shown in Figure 9.4. The earthwork requirements for both open and closed type abutments are shown in Figure 9.5.

The NMDOT has had success with the use of MSE walls supporting spread footing type abutments. Since the abutments and the bridge settle with the MSE wall and the approach embankment, smooth riding bridges can be obtained. This same settlement of the abutments can however be problematic if it is large or if the abutment is part of a continuous multi-span bridge. Use of this type of abutment should therefore be carefully considered and discussed with both the State Bridge Engineer and the Foundation Engineer before design is begun.

#### 9.4 JOINT BETWEEN SUPERSTRUCTURE AND SUBSTRUCTURE

In the past when a horizontal joint was required between the superstructure and substructure, bituminous joint material was shown. The bituminous joint material has turned out to be too stiff. The bituminous joint material is covered under AASHTO M 213 (ASTM D 1751) which allows the material to have a compressive stress range between 100 to 1,250 psi. The high compressive stress of the bituminous joint material has sometimes resulted in cracking and spalling at the joints. To try to eliminate this type of cracking, extruded polystyrene insulation should be used in lieu of the bituminous joint material at locations such as the horizontal joint between the superstructure and substructure at the piers and abutments, the joint between the approach slab and the abutment, the joint between wingwalls and approach slab, and the joints between wingwalls and abutment elements.

#### 9.5 DETAILING OF DRILLED SHAFTS AND PIER COLUMNS

In New Mexico, a 5" horizontal clear cover for drilled shaft reinforcing bars and a 6" clear cover between the bars and the bottom of the shaft have been implemented to ensure the long-term integrity of the reinforcing. Also, it is important for the designer to remember that when permanent casing is installed in a drilled shaft, the diameter of that casing is 6" larger than the nominal drilled shaft diameter, and that a column extending from the top of a drilled shaft is 6" less than the nominal drilled shaft diameter, as shown in Figure 9.6.

#### 9.6 REUSE OF EXISTING FOUNDATIONS

Rehabilitation of bridges often necessitates changes or replacement of the superstructure while leaving the substructure in place. Generally, if the layout of the superstructure does not change the loading distribution on the substructure and the additional dead load on the bridge is within the allowable load for the future wearing surface, the Department does not require the substructure to be analyzed.

To reuse an existing foundation system, there can be no significant reconfiguration of loads from the existing system (i.e. changes in bearing locations or pier fixities).

In addition to loading, there are several issues that the Designer needs to investigate to determine if the existing substructure can be reused:

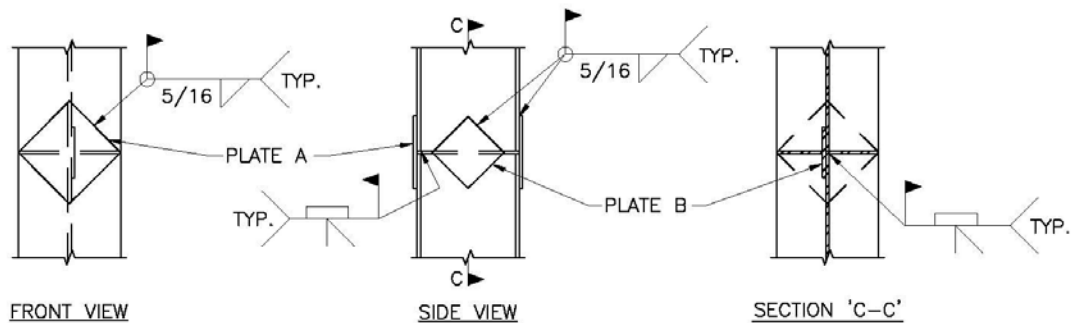
1. Scour: the Designer needs to evaluate both channel degradation and local scour.
2. Reinforcing: some older bridges are severely under reinforced; the Designer needs to determine if strengthening of the substructure is feasible.
3. Condition of the Substructure Elements:
  - a. The condition rating of the substructure elements should be a 6 (satisfactory) or greater. Additionally, the substructure should show no distress under existing live load.
  - b. Existing timber piles – the Department recommends that a core be taken where feasible. The location and fluctuation of the water table also needs to be evaluated in determining if an additional 40 year service life can be achieved.
  - c. Existing steel piles – existing steel piles need to be evaluated for loss of steel cross-section due to corrosion, including verifying the wall thickness of pipe piles. The location and fluctuation of the water table also needs to be evaluated in determining if an additional 50 year service life can be achieved. Additionally, the existing soil conditions need to be analyzed, including resistivity of the soil, pH of the soil, and evaluation of the soluble salts contained within the soil.

If the existing substructure is not deemed to be serviceable for reuse as part of a new bridge structure, reusing the existing abutments and wingwalls as earth retaining structures or scour protection independent of the new bridge structure should be considered.

**Figure 9.1**  
**Field Splice for Steel Bearing Piles**

TABLE OF SPLICE PLATE DIMENSIONS		
HP BEARING PILES	PLATE A	PLATE B
HP 10x57	$7\frac{1}{4} \times \frac{1}{2} \times 7\frac{1}{4}$	$5\frac{3}{8} \times \frac{1}{2} \times 5\frac{3}{8}$
HP 12x74	$8\frac{5}{8} \times \frac{1}{2} \times 8\frac{5}{8}$	$6\frac{3}{4} \times \frac{1}{2} \times 6\frac{3}{4}$
HP 12x84	$8\frac{3}{4} \times \frac{1}{2} \times 8\frac{3}{4}$	$6\frac{3}{4} \times \frac{1}{2} \times 6\frac{3}{4}$
HP 14x73	$10\frac{3}{8} \times \frac{1}{2} \times 10\frac{3}{8}$	$7\frac{7}{8} \times \frac{1}{2} \times 7\frac{7}{8}$
HP 14x89	$10\frac{3}{8} \times \frac{1}{2} \times 10\frac{3}{8}$	$7\frac{7}{8} \times \frac{1}{2} \times 7\frac{7}{8}$

SPLICE PLATES SHALL BE AASHTO M-183 STEEL  
(SPLICE PLATES SHALL BE AASHTO M270 GRADE 50 STEEL)

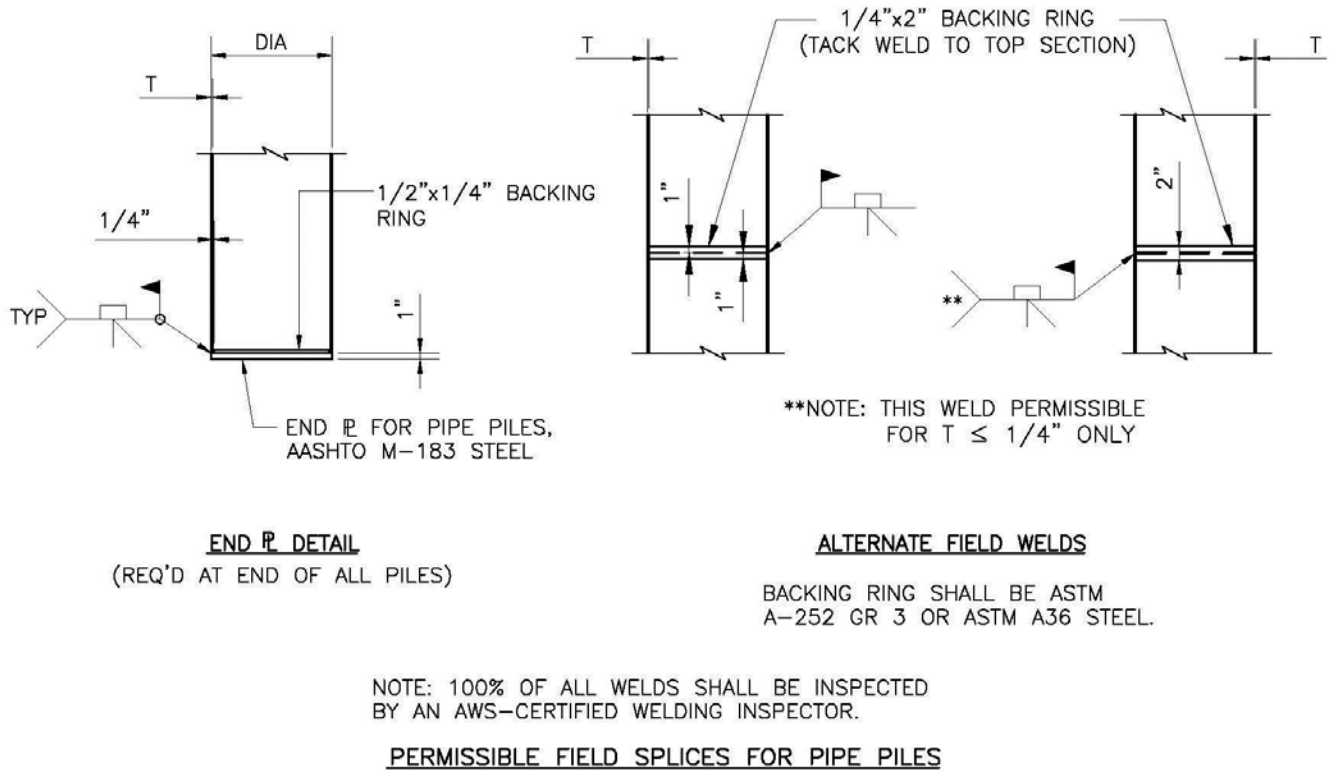


### FIELD SPLICE DETAILS FOR STEEL BEARING PILES

1. THE CONTRACTOR SHALL SUBMIT FOR APPROVAL THE WELDING PROCEDURE SPECIFICATION FOR REQUIRED WELDS BEING USED. THE SUBMITTAL SHALL BE FORWARDED TO THE BRIDGE DESIGN BUREAU FOR APPROVAL.
2. THE CONTRACTOR SHALL SUBMIT THE WELDER'S CERTIFICATION FOR THE WELDERS PERFORMING THE WELDING. THE CERTIFICATION WILL NEED TO MATCH THE WELDING PROCEDURE SPECIFICATION AND THE WELDING POSITION REQUIRED TO PERFORM THE WORK. THE SUBMITTAL SHALL BE FORWARDED TO THE BRIDGE DESIGN BUREAU FOR APPROVAL.
3. THE CONTRACTOR SHALL PURCHASE A WELD FILLET GAGE CAPABLE OF MEASURING THE REQUIRED 5/16" FILLET WELD. THE WELD FILLET GAGE SHALL BECOME THE PROPERTY OF THE NMDOT. THE COST OF THE FILLET GAGE SHALL BE INCLUDED IN THE PRICE OF THE DRIVEN PILES.

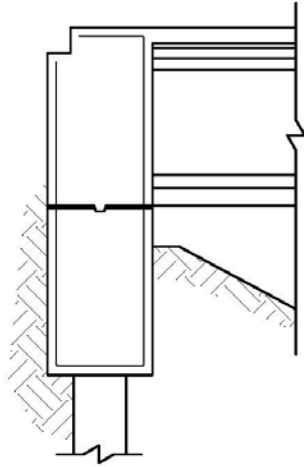
**Note:** This detail is available from the NMDOT Bridge Bureau

**Figure 9.2**  
**Field Splice for Pipe Piles**



**Note: This detail is available from the NMDOT Bridge Bureau**

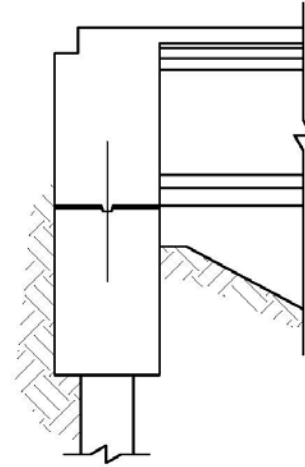
**Figure 9.3**  
**Types of Abutments**



FULLY INTEGRAL TYPE

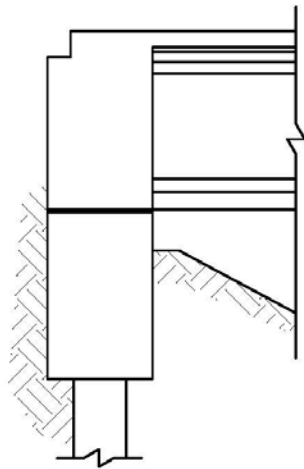
(OPEN TYPE)

NOTE: VERTICAL RESTRAINT  
MAY BE REQUIRED



SEMI-INTEGRAL PINNED

(OPEN TYPE)



SEMI-INTEGRAL FLOATING

(OPEN TYPE)

NOTE: LATERAL AND  
VERTICAL RESTRAINT MAY  
BE REQUIRED

**Figure 9.4**  
**Types of Abutments**

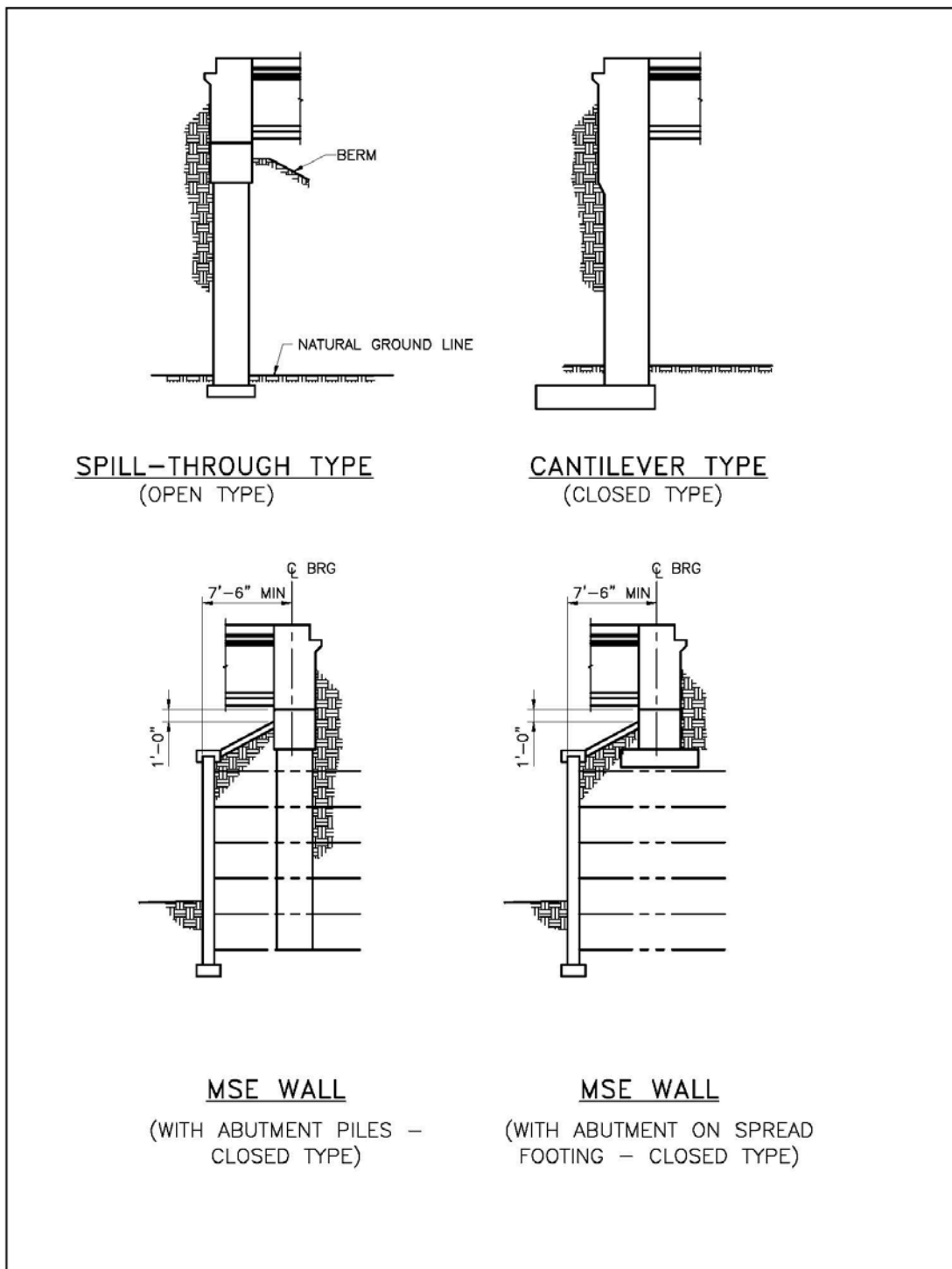




Figure 9.5  
Earthwork at Abutments and Wingwalls

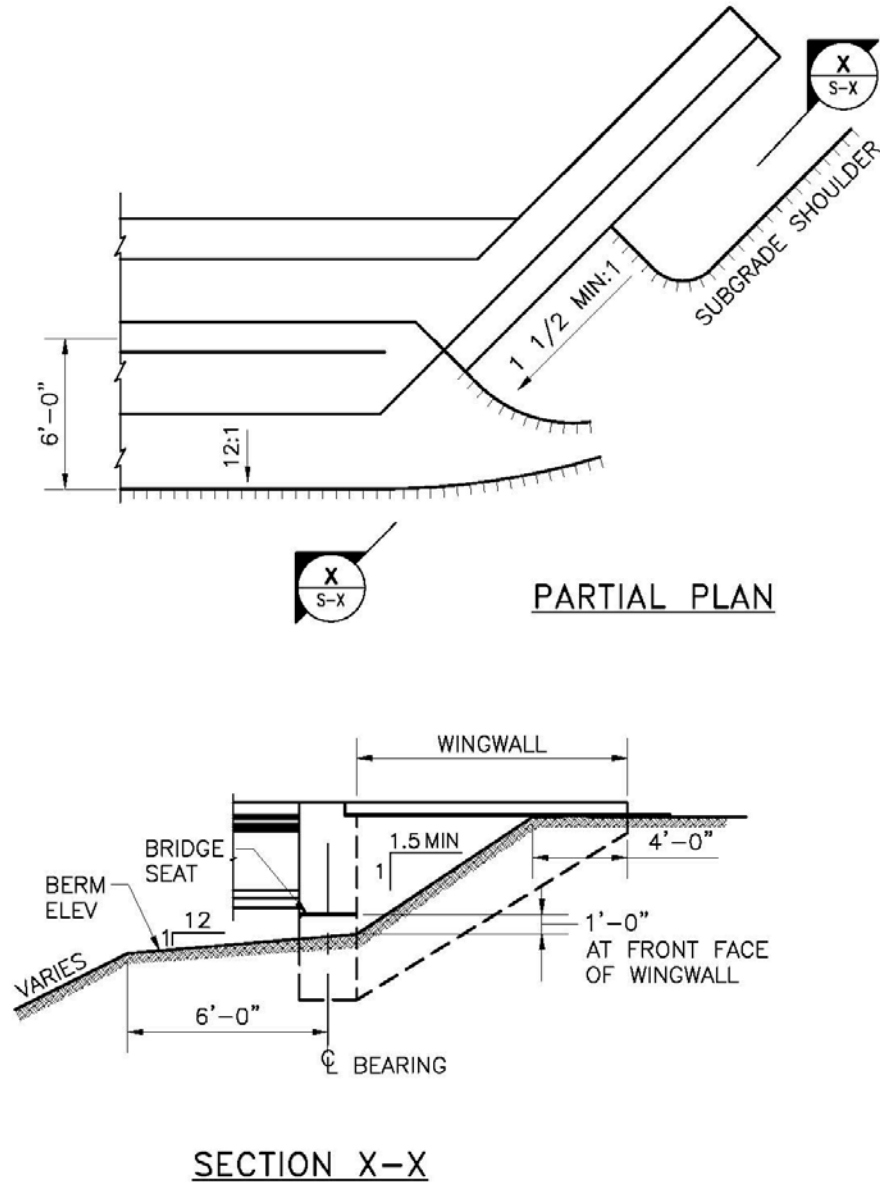
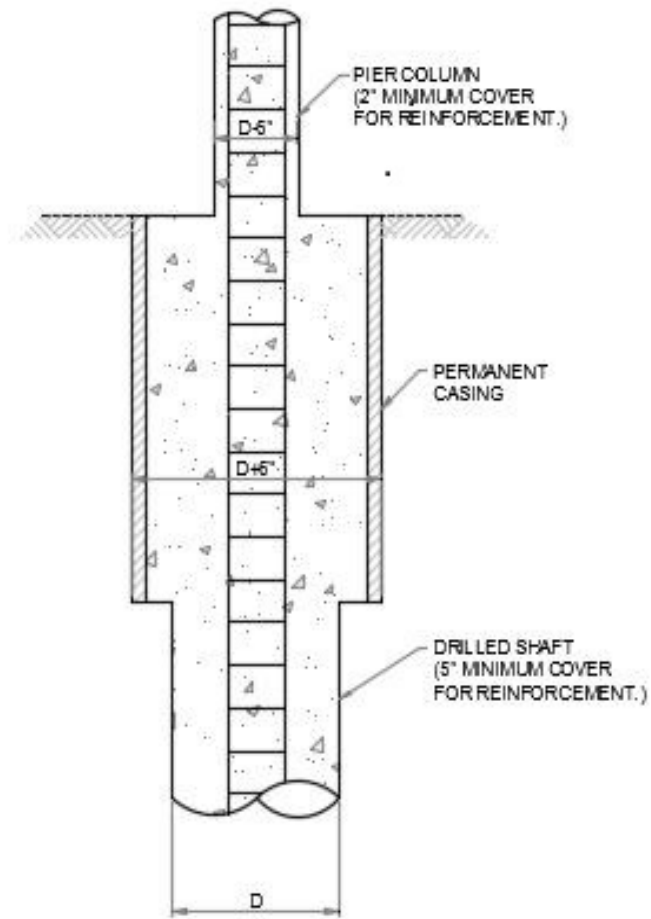


Figure 9.6  
Drilled Shaft Transition to Pier Column



## **CHAPTER TEN**

### **CONSTRUCTION ACTIVITIES**

#### **10.1 FORMAL MEETINGS DURING CONSTRUCTION**

A Preconstruction Conference is held at the beginning of construction on virtually every NMDOT Project. Its main purpose is to go over all major aspects of construction, lines of communication, and EEOC requirements for the job.

While the focus of the Preconstruction Conference is not structures, the Bridge Engineer may still wish to attend if bridges and structures are a significant part of the project, if there are significant structure-related issues that merit discussion, or if communication between the Project Manager and the Bridge Engineer is a potential concern.

A Predeck Conference is held prior to the placement of a bridge deck. Unlike the Preconstruction Conference, the Predeck Conference is directly related to and focused on the bridges on a project, and the Bridge Engineer should attend if possible. It is also an excellent opportunity for the Bridge Engineer to visit the Project if that hasn't happened yet.

In addition to the Predeck and Preconstruction Meetings, there may be other preactivity meetings (for example, Prefabrication and Pre-Erection meetings for structural steel) that may require the Bridge Engineer's attention and attendance. Also, Infrastructure Design Directive IDD-2017-01 calls for a 50% meeting between the Project Development Engineer and the Project Manager that may require the Bridge Engineer's input and/or attendance.

#### **10.2 CONSTRUCTION SUBMITTAL REVIEW**

Construction submittal reviews are an integral part of the construction support services (otherwise known as Phase III services) that are performed on behalf of the Department during the construction of a project. Timely attention to these services reduces construction delays and the likelihood of associated claims.

Here are a few points of procedure that apply to construction submittal reviews:

- All construction submittals, including shop drawings, submittals required by the Standard Specification to be reviewed and accepted by the NMDOT Bridge Bureau, and Requests for Information (RFIs) are to go through the NMDOT Project Manager.
- The Bridge Design Engineer needs to coordinate with the Project Manager on schedule for reviewing shop drawings.
- For shop drawings involving steel fabrication, send a copy of reviewed and accepted shop drawings to the NMDOT Bridge Bureau Construction Unit.
- For shop drawings involving precast concrete, send a copy of reviewed and accepted shop drawings to the District 3 Precast Inspection Unit.

The NMDOT Bridge Bureau has developed flowcharts for Phase III services by consultants and NMDOT personnel. These flowcharts are provided in Appendix B.

### **10.3 FIELD REVIEWS AND PROJECT FOLLOW THROUGH**

It is recommended that the Project Manager be contacted at least once a month while a bridge is under construction to discuss the construction process and project issues.

Whenever possible, the Project Manager should be contacted prior to field visits. This eliminates the impression that the visit is some sort of a surprise inspection, and it gives the Project Manager adequate time to ensure access to key project elements.

## CHAPTER ELEVEN

### LOAD RATING

*Note: The guidance in this chapter was originally developed in 2010. The NMDOT is currently developing a Bridge Rating Guide, and this Chapter of the Bridge Procedures and Design Guide shall be rendered obsolete upon its release.*

#### 11.1 GENERAL INFORMATION

The New Mexico Department of Transportation (NMDOT) uses the current edition of *The Manual for Bridge Evaluation* (Washington DC: American Association of State Highway and Transportation Officials, AASHTO) as the engineering standards for bridge load rating. AASHTO Bridge Rating (available through AASHTOWare) is NMDOT's chosen tool for permanently recording bridge structural parameters in electronic form.

All new bridges and all major bridge rehabilitation projects, and any changes to an existing bridge's loading condition require a complete bridge load rating—including, when possible, a AASHTO Bridge Rating model. Some examples of the changes to an existing bridge's loading condition may include the addition of concrete barrier railing, addition of utility lines or water lines, addition of a topping/overlay, etc. Additionally, when any bridge becomes "Structurally Deficient," a qualified bridge engineer must re-evaluate the bridge capacity. This re-evaluation requires a special-emphasis site inspection to support the updated bridge load rating. Typically, a bridge becomes "Structurally Deficient" when the deck, superstructure, or substructure (Items 58, 59, or 60 in the National Bridge Inventory Condition Rating) drop to Condition Rating 4 (Poor Condition) or less.

##### 11.1.1 Purpose

Bridge load-ratings provide several useful results:

- Confirm a bridge has adequate design for normal operations capacity
- Identify those bridges that do not have adequate capacity for normal operations and consider such bridges for posting
- Provide bridge capacity information and models for routing overload permit vehicles
- Identify unused capacity in existing bridges
- Support examination of Structurally Deficient bridges
- Provide a review on new bridge design

##### 11.1.2 Records Required for Rating

The engineer in charge of a specific bridge load rating is responsible for insuring that available documents are adequate to rate the bridge in the current condition and design. The minimum records required are:

- Bridge drawings of superstructure elements
  - Original and significant rehabilitations
- Bridge Inspection Reports

Some circumstances require a special-emphasis site inspection to complete the load rating. The most notable circumstances are bridges classified by routine bridge inspections to be "Structurally Deficient."

##### 11.1.3 Deliverables

A typical bridge load rating provides two deliverables: 1) An AASHTO Bridge Rating model for NMDOT's bridge model database and 2) a completed "NMDOT Bridge Load Rating Form." Load ratings for a "Structurally Deficient" bridge also require explanation of how the structural deficiency affects the bridge load capacity.

## 11.2 RATING METHODS

*The Manual for Bridge Evaluation* includes three rating methods: Load and Resistance Factored Rating (LRFR, based on Load and Resistance Factored Design, LRFD), Load Factored Rating (LFR, based on Load Factored Design), and Allowable Stress Rating (ASR, based on Allowable Stress Design). NMDOT uses BRASS as the primary underlying engine for AASHTO Bridge Rating analyses.

Use ASR for timber bridges and decks. Use LFR and LRFR for all pre-stress concrete, reinforced concrete (including slab bridges), and steel bridges.

NMDOT plans to move exclusively to LRFR ratings in the future; however, the procedure NMDOT currently uses to route overweight vehicles still requires LFR ratings. Therefore, NMDOT is requiring both LFR and LRFR ratings for new bridges.

## 11.3 RATING NOTES AND VALUES

Virtis inputs require some values that are not readily apparent from drawings. Instead, these values typically rely on “policy values” established through standard assumptions in the absence of actual data. NMDOT also notes some issues that may be misleading, or otherwise unclear, during drawing interpretation for AASHTO Bridge Rating models. This section covers policies and notes applicable to completing New Mexico bridge models.

In bridges that consist of beams supporting a deck, the concrete deck slabs and metal decks that satisfactorily carry normal traffic need not be routinely evaluated for load capacity. Routine inspections will identify those metal and concrete bridge decks with unsatisfactory performance—which will likely result in a “Structurally Deficient” bridge classification and require a special-emphasis inspection. In

contrast, timber decks should be routinely evaluated for load capacity.

Members of substructures need not be routinely checked for load capacity. Substructure elements such as pier caps and columns should be checked in situations where the engineer has reason to believe that their capacity may govern the load capacity of the entire bridge. Examples of distress that could trigger a substructure load-rating include: a high degree of corrosion and section loss, caps cracked and distorted under torsion with inadequate shear stirrups, changes in column end conditions due to deterioration, changes in column unbraced length due to scour, or columns with impact damage. Such cases will generally render a bridge “Structurally Deficient” and require a special-emphasis inspection.

When a bridge exhibits load capacity rating less than LFR HS20/HS33 Inventory/Operating (see Section 11.4 for HS20 definition) or LRFR  $R_f < 1.0$ , the bridge should be re-examined for rating improvement opportunities. These opportunities routinely include:

- Review the assumptions and model simplifications that affect bridge rating.
- Bridge drawings may include allowance for future wearing surface, but the bridge might have no wearing surface. If the bridge has no wearing surface or a lighter wearing surface, a rater may reflect this in the bridge model. In no case should the bridge model contain a wearing surface that is less than the actual existing wearing surface.
- AASHTO adopted greater shear reinforcement requirements in 1994. Pre-stress concrete bridges designed prior to that relied on a 1979 Interim standard and prior provisions. AASHTO Bridge Rating currently has no way to alter its shear analysis to recognize the 1979 Interim standard. If the controlling failure mode in a pre-stress

concrete girder is shear, use the shear policy outlined under Section 11.3.5 “Pre-stress Concrete” to modify the rating.

Non-routine methods to improve load-rating results include:

- AASHTO Bridge Rating has deck finite element modeling capability. In a few rare cases, it is possible that using a finite element determined distribution factor will be more accurate and allow rating improvements.
- Use refined Methods of Analysis described in *AASHTO LRFD Bridge Design Specifications* Article 4.4.
- In extraordinary cases, consider “Nondestructive Load Testing” as described in Section 8 of *The Manual for Bridge Evaluation*.

Insure that deviations used to achieve sufficient load capacity are documented in “Notes, additional loads, comments or deviation from general rating practice” in the Bridge Load Rating Form. For instance, if shear controls the bridge rating and produces ratings less than HS20/HS33 and the shear stirrups were designed based on 1979 Interim Standard or prior provisions, the Engineer should load rate the bridge with shear ignored. If the bridge ratings are still lower than HS20/HS33, these values should be recorded on the Bridge Load Rating Form (see Section 11.5). If the bridge ratings are higher than HS20/HS33, per NMDOT policy the ratings will be reported as HS20/HS33. A note should then be used listing the ratings when shear controls and when shear is ignored, see Figure 11.5 for an example of this note.

### 11.3.1 Materials\Concrete

This section covers specific entries in the AASHTO Bridge Rating file tree, Materials\Concrete. The reader can launch AASHTO Bridge Rating and open the

specific file folders “Material” and “Concrete.” Many other sections that follow also cover specific entries as identified in **bold**.

**Density.** Typically, use 0.150 kcf density for dead loads and 0.145 kcf for modulus of elasticity.

**Copy from Library.** Many older NM bridge drawings will call for “Class A Concrete.” This is not the same concrete as found in the AASHTO Bridge Rating Standard Library called “Class A” or “Class A (US).” NMDOT Class A Concrete is 3,000 psi while that listed in the AASHTO Bridge Rating Standard library is 4,000 psi concrete. The AASHTO Bridge Rating Standard library has no concrete matching NMDOT Class A. Virtis Standard Library Class A matches NMDOT Class AA. NMDOT Agency library files are available from the Bridge Bureau, and they include all NMDOT concrete classes.

### 11.3.2 Impact/Dynamic Load Allowance and Factors.

For general ratings without deviations from standard practice, make no changes to “Impact/Dynamic Load Allowance” or “Factors” in AASHTO Bridge Rating.

### 11.3.3 Superstructure Definitions

**New Superstructure Definition.** Generally, when selecting “New Superstructure Definition” in AASHTO Bridge Rating, select “Girder System Superstructure.” However, circumstances may prevent using a “Girder System.” For example, the most common circumstance requiring selecting a “Girder Line Superstructure” over a “Girder System” is modeling a slab bridge. AASHTO Bridge Rating has no method to model a complete slab bridge. Instead, it is modeled as a 12-inch strip analysis in a Girder Line. See Section 11.3.6.



**Structure Typical Section\Parapet or Railing.** Notice that typical bridge barrier rails in New Mexico are placed 18-inches from the bridge edge, but are not a full 18-inches wide. Therefore, modeling a bridge barrier rail as 0 from the bridge edge to the back of the rail is technically not accurate. A better approach is to model the barrier rail (parapet) as: “Measure To” *Front*; “Distance” *1.50 feet*.

**Structure Typical Section\Wearing Surface.** NMDOT current policy is to design for a 30 psf future wearing surface. Previous design policy included a 15 psf wearing surface. If a bridge does not currently have a wearing surface, *The Manual for Bridge Evaluation* allows rating without the design future wearing surface included. One should always rate the bridge with the actual wearing surface if it is equal to, or greater than, the designed wearing surface. However, if a bridge has a wearing surface less than design wearing surface, first rate the bridge with the assumed design surface (typically 30 psf or 15 psf). If the bridge rates at least an HS20 Inventory and HS33 Operating, retain the model with the future wearing surface included. If the bridge rates less than HS20/33, then remove or reduce the wearing surface in the model to match actual bridge condition. Model and rate the bridge with this reduced wearing surface.

#### 11.3.4 Member Alternatives

**New Member Alternative Description\Girder property input method.** When provided a choice between “Schedule based” and “Cross-section based” in “Member Alternative Description,” always select “Schedule based.” “Cross-section based” does not provide the same valuable schematics.

**New Member Alternative Description\Crack Control Parameter (Z) and Exposure factor.** Use  $Z = 170$  kip/in for the

concrete crack control parameter. Use Exposure factor = 1.0.

**New Member Alternative Description\Default rating method.** Each “Member Alternative” in AASHTO Bridge Rating has an input for “Default rating method.” Timber can only be rated using ASD. AASHTO Bridge Rating defaults to ASD on timber girders and decks. For all other “Member Alternatives” set the “Default rating method” to LRFR so that each model is set to rating bridges with LRFR when “Rating Method” selected is “Member Alternative.” This sets the AASHTO Bridge Rating bridge database for compliance with future LRFR ratings.

**Deck Profile\Structural Thickness.** Past NMDOT policy provided a ¼” sacrificial thickness in concrete decks. This is no longer NMDOT policy. A rating for older bridges with drawings that specify this sacrificial surface may, instead, include the full deck thickness without section reduction for sacrificial surface. Typically enter the full deck thickness for “Structural Thickness.”

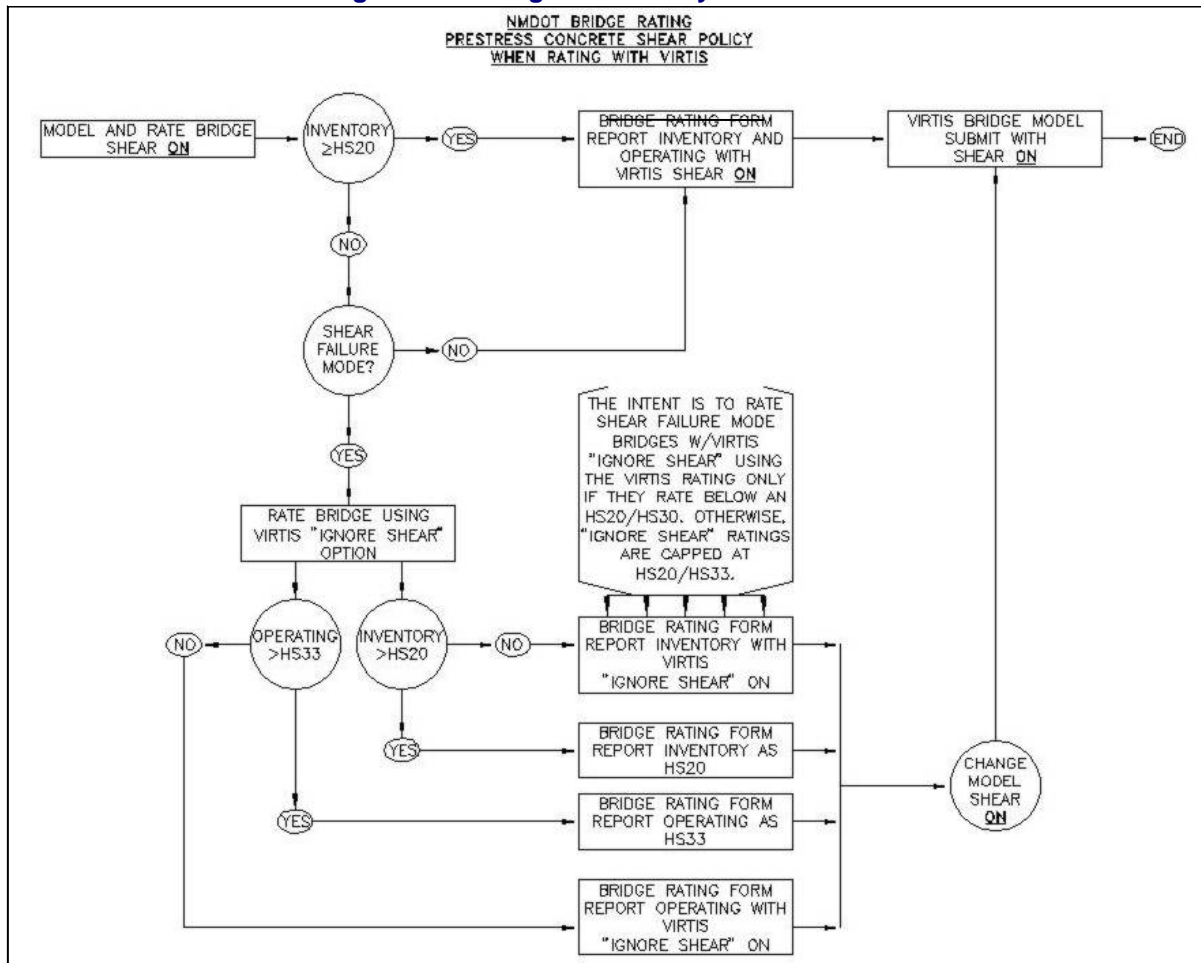
#### 11.3.5 Pre-stress Concrete

Note that pre-stress concrete bridges that are simple spans for both dead and live load but have *jointless decks* are not uncommon—both in bridge conversions to jointless decks and in new bridges. (See Section 5.6.2 “Eliminating Deck Joints” in this NMDOT *Bridge Procedures and Design Guide*). Caution: it is easy to misinterpret such bridges as being continuous for live loads. If it is possible that a continuous deck bridge is *not* continuous for live load, check details for breaks in continuity over the piers. One should model such jointless-deck simple-span bridge girders as simple span, not continuous.

Many pre-stress concrete bridges designed before 1994 have served normal operations



**Figure 11.3A**  
**NMDOT Bridge Load Rating Shear Policy for Pre-stress Concrete**



and overload permit loads well. These same bridges may theoretically fail in shear when analyzed with current shear design models. AASHTO Bridge Rating does not have an option to use the 1979 Interim standard. Therefore, if an existing pre-stress concrete bridge is in service without visible signs of shear distress and was designed under the 1979 Interim standard, use the procedure shown in Figure 11.3A.

**Beam Shapes\Pre-stress Beam Shapes\I Beams.** Pre-stress I-beam strand patterns in AASHTO Bridge Rating default to vertical distances from bottom on 2-inch spacing. NMDOT typically specifies strand patterns placed as 2-inch spacing from the bottom, but also as 2-inch spacing from the top for draped stands. The result is that strand patterns in New Mexico may not match the AASHTO Bridge Rating defaults for AASHTO beams

with an odd number of inches for overall height. These are the Type 45 (AASHTO III), Type 63 (AASHTO V), and Type BT-63. Modify strand patterns as needed in these beams to match NMDOT pre-stress beam strand patterns accurately.

**Pre-stress Properties.** LRFR ratings require inputs that are not required by LFR. Since NMDOT will convert to LRFR rating, these inputs must be included in all AASHTO Bridge Rating models. Under “Superstructure Definitions” and “Pre-stress Properties,” they are:

- Transfer time, use 15.0 hours
- Age at deck placement, use 270 days
- Final age, use 3650 days

**Deck Profile\Reinforcement.** Pre-stress concrete bridge superstructures have steel reinforcing bars in the deck. However, the

composite decks in *simple spans* are always modeled in compression for longitudinal capacity. Therefore, the deck rebar for simple spans (simple for dead *and* for live loads) is immaterial for load rating capacity and can be ignored. In contrast, the deck rebar is critical in pre-stress continuous bridges (simple for dead but continuous for live load).

### 11.3.6 Concrete Slab

**Superstructure Definitions.** Unlike most bridges, a concrete slab bridge cannot be modeled in AASHTO Bridge Rating as a “Girder System.” Instead, one must rate a concrete slab bridge using a “Girder Line.” AASHTO Bridge Rating “Girder Line” modeling does not allow AASHTO Bridge Rating to calculate several bridge aspects. Instead, the user must calculate these by hand. Calculated values available in a “Girder System” but not available for a “Girder Line” include:

- Live load distribution factors
- Dead load distribution
  - Wearing surface
  - Bridge barrier rail (parapet, railing)
  - Medians and curbs
  - Utilities

**Member\Member Location.** Users may rate concrete slab bridges using only “Interior” for “Member Location.” This assumes that the original designer correctly designed the exterior (or edge) of the slab bridge. Alternatively, it recognizes that, although not modeled in design, a bridge barrier rail provides edge stiffness. Modeling only an interior equivalent slab assumes that the interior controls the slab bridge rating.

**Live Load Distribution.** The Live Load Distribution Factors for LFR using a 1-ft equivalent interior strip are:

Moment—

$$DF = \frac{1 \cdot ft}{4 \cdot ft + 0.06 \cdot S} \geq \frac{1}{7}$$

Deflection—

$$DF = \frac{1}{6}$$

For LRFR ratings, use the following distribution factors for moment, shear, and deflection:

One-lane loaded—

$$DF = \frac{12 \cdot in}{10 \cdot in + 5 \cdot \frac{in}{ft} \cdot \sqrt{L \cdot W_1}}$$

Two or more lanes loaded—

$$DF = \frac{12 \cdot in}{84 \cdot in + 1.44 \cdot \frac{in}{ft} \cdot \sqrt{L \cdot W_2}} \geq \frac{1 \cdot ft \cdot N_L}{W}$$

DF = Live Load Distribution Factor

N<sub>L</sub> = Number of design lanes  
 Not applicable if w < 20 ft (only one lane loaded)  
 2 if 20 ft ≤ w < 24 ft  
 Integer part of (w/12ft) otherwise

L = Modified span length; the lesser of S or 60 ft

S = Longitudinal Span length

W = Edge-to-edge bridge width

W<sub>1</sub> = Modified edge-to-edge width, one-lane loaded; lesser of W or 30 ft.

W<sub>2</sub> = Modified edge-to-edge width, two or more lanes loaded; lesser of W or 60 ft.

w = Clear roadway width between curbs or barriers

**Girder Profile\Section.** Use a 12-inch Girder Line Equivalent Slab strip to model slab bridges. This is the selected NMDOT policy width calculation for equivalent slab width. Using a different width complicates model comparisons for quality control.

### 11.3.7 Steel

**Deck Profile\Reinforcement.** BRASS LFR load rating does not consider minimum negative flexure concrete deck reinforcement. However, BRASS LRFR rating does. LRFD 6.10.1.7 covers “Minimum Negative Flexure Concrete Deck Reinforcement.” If the deck reinforcing steel is not included in a continuous steel bridge model, AASHTO Bridge Rating/BRASS LRFR will fail the bridge under Service II over the piers. Therefore, all AASHTO Bridge Rating continuous steel bridges must include the deck reinforcement in the model for future use in LRFR rating.

**Lateral Support.** A steel girder with a concrete deck poured in direct contact develops an affinity for concrete. Therefore, a continuous hardened concrete deck provides continuous lateral support to the top flange of a girder. Note that the lateral support applied in the “Lateral Support” window applies only to the top flange. AASHTO Bridge Rating picks up the discrete locations of lateral support on the bottom flange from the “Framing Plan Detail, Diaphragms.”

### 11.3.8 Timber

Use Allowable Stress Rating (ASR) for timber girders and timber decks. Timber decks often control the rating in more primitively designed bridges found in New Mexico’s inventory. Timber decks must be routinely evaluated for a bridge load rating.

**Materials\Timber.** *Girder unknown species and grade.* In selecting timber materials, AASHTO Bridge Rating has a “Copy from Library” option. For completely unknown species and grade, but reasonably typical New Mexico timber bridge, choose Douglas Fir-Larch, No. 1 for Beams and Stringers or for Posts and Timbers. If the rater knows that the girders in a particular bridge are free of notable weathering and without checks or

splits, the rater may choose Douglas Fir-Larch, Dense No. 1.

“Beams and Stringers” are rectangular timber members whose nominal dimensions are greater than 5” in both directions, but one dimension is more than 2” greater than the other is. E.g., 9”x14”, 8”x12”.

“Posts and Timbers” are rectangular timber members whose nominal dimensions are greater than 5” in both directions and the dimensions are the same or no more than 2” difference. E.g., 12”x12”, 8”x10”.

*Southern Yellow Pine Dense.* In some old timber bridge drawings, the species and grade specified for the bridge is Southern Yellow Pine Dense Longleaf and Shortleaf. This is a dated specification. If a timber bridge uses this specification for girders, select “Southern Pine (Dry or Wet), Dense Select Structural, 5”x5” & larger.” For the deck, the drawings will call for Southern Yellow Pine Dense. However, unless otherwise known, assume the deck has been replaced. Since the replacement deck species is probably unknown, see below.

*Deck unknown species and grade.* Decks will typically be lumber. Lumber reads as “2”– 4” thick 2” and wider,” or similar description in the AASHTO Bridge Rating library. Choose Hem-Fir No. 2 unless more is known from records or reports.

**Timber Beam Shapes\Rectangular\ Properties.** The “Compute” button will fill in all areas except “Nominal load,” “Nominal width,” and “Nominal depth.” Use 50 lb/cf and actual dimensions to calculate “Nominal load.” “Nominal width” or “Nominal depth” is the next highest whole number for the actual dimension. 9.25” → 10”

**Superstructure Definitions.** A rater should be sure to select “Deck type” as “timber” the

first time “Girder System Superstructure Definitions” comes up in the bridge model inputs. Failure to note the correct “Deck type” will require restarting the “Superstructure Definition.” AASHTO Bridge Rating does not allow returning and correcting the “Deck type” after initial entry.

**Deck.** The “Deck LL distribution width” is 15-inches plus the “Total deck thickness.”

**Deck\Factors.** Always exercise the Deck\Factors input page. For all three moisture conditions, leave the default, “Wet,” selected. Use the “Compute” button to supply the various modification factors for the deck timber. Unless one is familiar with judging “Shear factor” by visual inspection or if the rater has not had an opportunity to inspect the deck lumber, enter 1.0 for the “Shear factor.”

**Structure Typical Section\Wearing Surface.** Many timber bridges have excessive asphalt wearing surfaces. A timber bridge’s load rating is sensitive to this dead load. The rater must examine the bridge documents, or examine the bridge itself, to ascertain an estimate of the existing overlay thickness.

**Beam Details\Adjustment Factors.** As with the Deck Factors, always exercise the Beam Details\Adjustment Factors input page. Moisture conditions should all remain “Wet.” The “Compute” button will supply all factors except the “Shear factor.” Enter 1.0 for “Shear factor” unless otherwise familiar with a girder visual inspection and what to look for to allow an improved “Shear factor.”

### 11.3.9 Library Explorer

NMDOT Bridge Bureau maintains an “Agency File” for addition to the AASHTO Bridge Rating Library. This Agency File contains materials, vehicles, and appurtenances encountered repeatedly in New Mexico.

Consultants should insure they have the latest NMDOT Agency Files when appropriate.

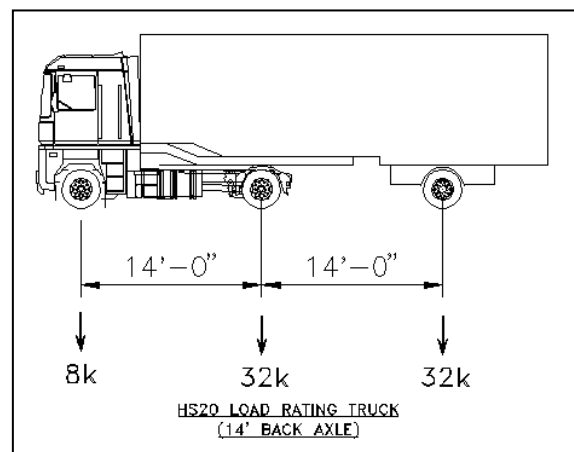
## 11.4 RATING VEHICLES

Figure 11.4A through 11.4F illustrate rating vehicles used in New Mexico.

### 11.4.1 NMDOT Rating Truck

Figure 11.4A shows a modified HS20 Truck. The modification from the standard AASHTO HS20 is that the rear axle is fixed at 14-feet rather than varying from 14 to 30-feet. This modified HS20 Truck is the basis for NMDOT’s Permit Load bridge evaluation software, OVLOAD, and its “Method of Equivalent Loading.” NMDOT requires this fixed-rear-axle truck be the basis for Inventory and Operating rating. One should note that the fixed axle yields slightly better ratings in some bridge geometries than would the unmodified HS20. Use this modified HS20 Truck. This truck is available in NMDOT’s AASHTO Bridge Rating Agency File, titled “HS20 Rating Truck.” A  $R_f = 1$  using this NMDOT Rating Truck results in a bridge capacity rating of HS20.

NMDOT is aware that the HS20 Rating Truck without the variable axle from 14-ft to 30 ft does not conform to national standards. The use of this HS20 Rating (Fixed 14-ft axles) in



**Figure 11.4A**  
**NMDOT HS20 Modified Truck for Rating**

New Mexico dates back to 1982 when New Mexico State University first authored OVLOAD. Subsequently this became NMDOT's standard practice. New Mexico does report load ratings to the Federal National Bridge Inventory (NBI) using this fixed-axle truck. However, NMDOT will ignore this minor non-conformity and, instead, concentrate load-rating policy changes on bringing future NMDOT Bridge rating into conformance with LRFR rating approach—both for NBI reporting and for OVLOAD input.

#### 11.4.2 HL-93

AASHTO LRFD Bridge Design Specifications defines the “Design Vehicular Live Load” known as HL-93 in Article 3.6.1.2. HL-93 will become integral to NMDOT's bridge load rating when NMDOT transitions to LRFR.

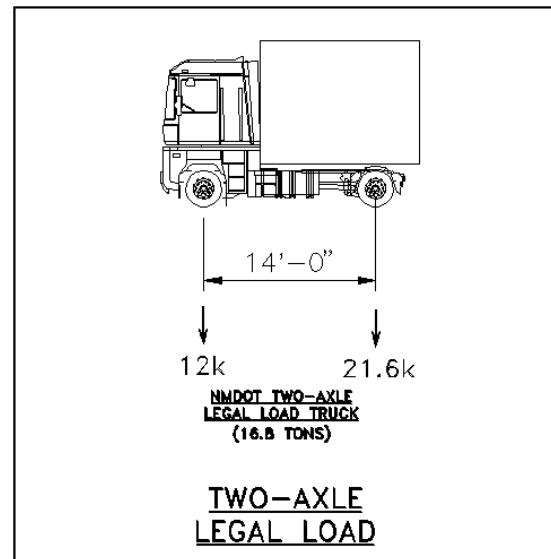
#### 11.4.3 Legal Loads

New Mexico uses a family of nine trucks as legal load model trucks. Three of these trucks are AASHTO Legal Loads. Six are trucks derived from New Mexico Law to capture a range of likely load-effects from trucks meeting the legislative definition for legal trucks in New Mexico. Figure 11.4B illustrates the two-axle legal load truck. Figure 11.4C illustrates the three trucks with different axle spacing and weights intended to capture three-axle truck effects. Likewise, 11.4D shows a four-axle, and 11.4E shows three five-axle trucks. Figure 11.4F illustrates the AASHTO six-axle vehicle.

#### 11.4.4 Bridge Posting Analysis

NMDOT uses legal load trucks from Section 11.4.3 to establish bridge load posting. Bridges with an Operating Rating less than HS17 (Rating Factor < 0.85 for HS 20 Rating Truck) require rating the bridge with this family of nine trucks.

Engineering load rating is only one factor of many in the basis for decisions related to



**Figure 11.4B**  
**NM Legal 2-Axle**

bridge posting. Posting is a policy decision made by the bridge owner. In New Mexico, the bridge owner is one of the NMDOT Districts. Posting is not a purely engineering activity. Other issues involved in a District's decision regarding bridge posting are:

- Bridge structure redundancy
- Bridge condition or visible distress
- Character of traffic
- Likelihood of overweight vehicles
- History of abuse
- Posting enforceability

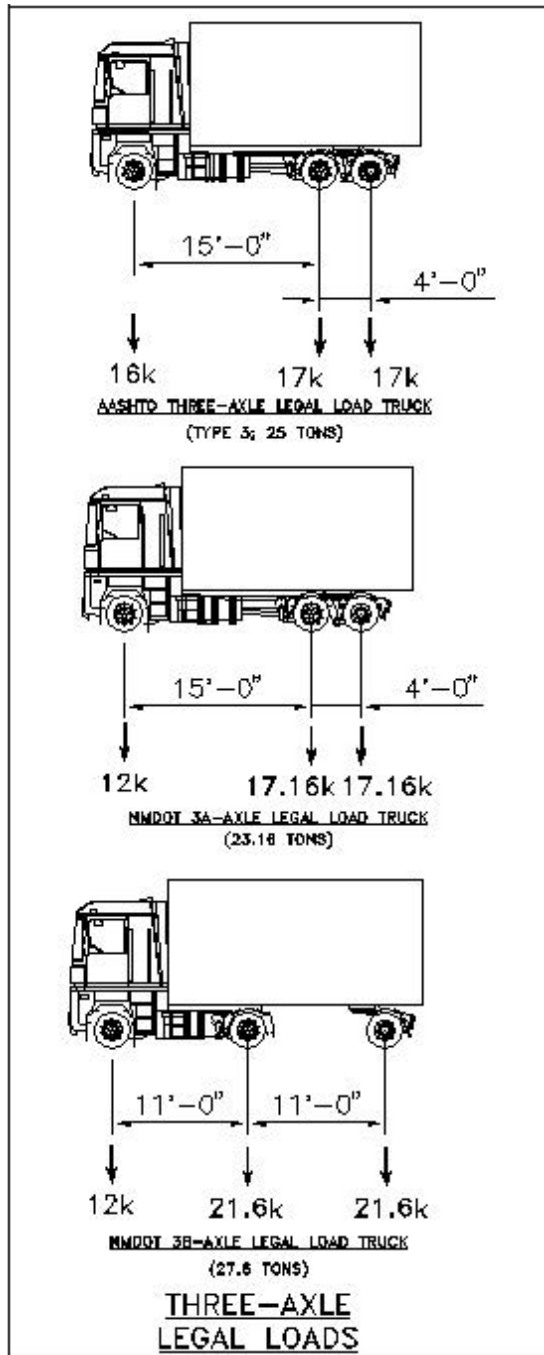
For the engineering side of posting, as a bridge's capacity becomes further below standard design standards, there is increased uncertainty in the specific effects and in public compliance with load postings.

This increased uncertainty calls for an increased safety factor. Use the following equation from *The Manual for Bridge Evaluation* to produce such increased safety factor:

$$\text{Safe\_Posting\_Load} = \frac{W}{0.7} \cdot (\text{RF} - 0.3)$$

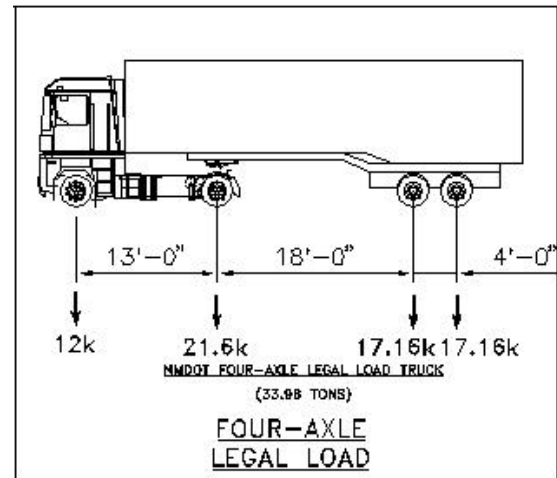


RF = Legal load rating factor  
W = Weight of rating vehicle  
For each axle class, two-axle through six-axle, the resulting lowest “Safe Posting Load” becomes the engineering posting load analysis for that bridge and axle number—with the exception that in no case will fewer axle



**Figure 11.4C**  
**NM Legal 3-Axle**

posted load be greater. For example, a three-axle posting cannot be greater than a four-axle posting.



**Figure 11.4D**  
**NM Legal 4-Axle**

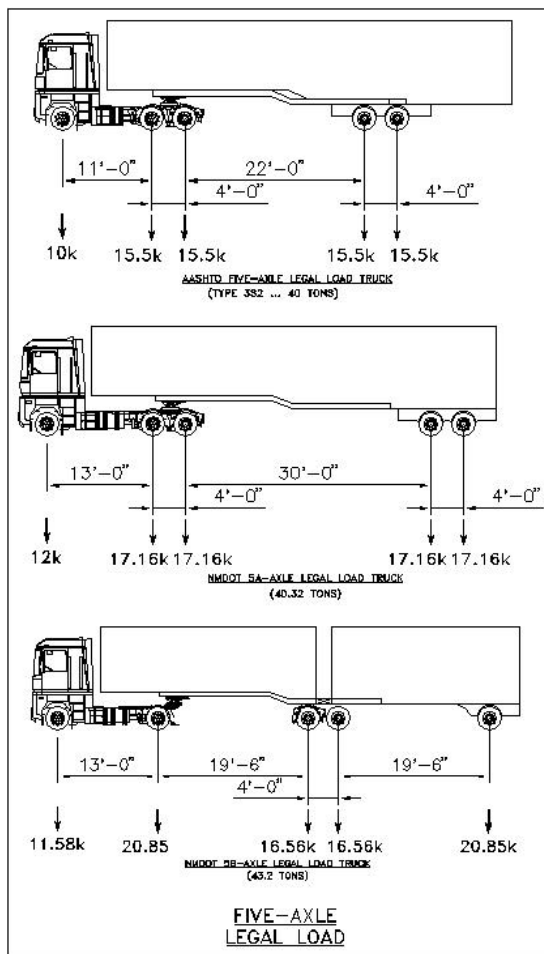
## 11.5 BRIDGE LOAD RATING FORM

A current version of the bridge load rating form is available from the State Bridge Rating Engineer. Figure 11.5 shows an example NMDOT Bridge Rating Form. This is the version current at time of this writing; however, form details may be subject to change. Bridge Inspection Reports contain the information needed for the “Bridge Information” section. These Bridge Inspection Reports also provide the information for “Structure Conditions” under the “Rating Approach Summary.”

*Notes, additional loads, comments or deviation from general rating practice:* This section under “Rating Approach Summary” should never be ignored. Many bridges require some commentary not covered in other “Rating Approach Summary” items. The standard items and this “Notes ...” section should provide enough detail so that another engineer, using the same drawings and same tools, would be readily able to replicate the rating.

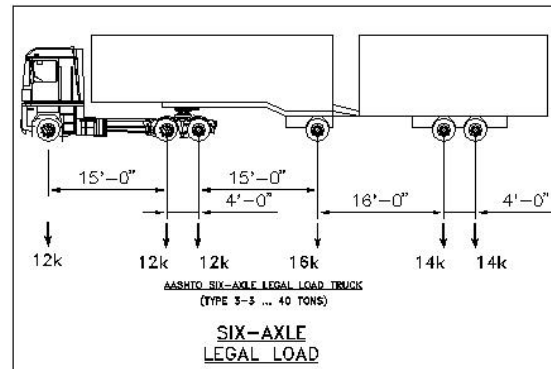
## 11.6 QUALITY ASSURANCE

AASHTO Bridge Rating software demands are high. It always requires accuracy and detail. Sometimes it requires elevated engineering judgment. To improve the assurance NMDOT receives a quality product, NMDOT requires a Quality Control process.



**Figure 11.4E**  
**NM Legal 5-Axle**

NMDOT's Quality Control process has two engineers create independent AASHTO Bridge Rating Models. The senior of the two engineers, or a third Engineer, uses both models to rate the bridge girders and compares the outcome. Significant differences require reviewing the two models, identifying where they differ, and making changes to



**Figure 11.4F**  
**NM Legal 6-Axle**

models to correct the mismatch. The process continues until both bridges rate within 2% for each girder in both bridge girder systems.

Consultants may adopt NMDOT's Quality Control process or they may propose one of their own. NMDOT's State Bridge Engineer or State Bridge Load Rating Engineer must approve a proposed consultant Quality Control process that differs from the NMDOT Quality Control approach.

NMDOT will also sample consultant's ratings to check results quality. NMDOT will report any errors, even minor ones, back to the consultant. A pattern of errors will lead to a request for discussion regarding rating procedures that might address and correct inaccurate modeling.

NMDOT's quality review will not be comprehensive and will not reliably catch all errors. NMDOT reviews are samplings and spot checks only. Consultant Quality Assurance remains primarily the consultant's responsibility.

**Figure 11.5**  
**NMDOT Bridge Rating Form**

New Mexico Department of Transportation <b>BRIDGE RATING FORM</b>									
Today's date: <i>January 20, 2010</i>		Name of Person Completing Form: <i>Frank Form</i>							
BRIDGE INFORMATION									
Structure Number: <i>07734</i>	Facility/Route Carried: <i>NM-88</i>	Feature Intersected: <i>Gabbit Gulch</i>	Location MP/miles from intersection: <i>MP 5.1/4 mi W of Jct NM-2</i>			District: <i>Four</i>	Location Latitude/Longitude: <i>36.133d N - 108.34d W</i>		
RATING APPROACH SUMMARY									
Rating Method: <input checked="" type="checkbox"/> LFR <input type="checkbox"/> ASR <input type="checkbox"/> LRFR LFR rating using HS-20 req'd on all bridges.					Live Loads Checked: <input checked="" type="checkbox"/> HS20 <input type="checkbox"/> HL-93 <input type="checkbox"/> Legal Loads <input type="checkbox"/> Other—Identify: _____				
<input type="checkbox"/> Original Rating <input checked="" type="checkbox"/> Re-Rating ... Reason: <i>Document load rating and generate Virtis Model</i>									
Criteria Used in Rating: _____									
Current Wear Surface		<input checked="" type="checkbox"/> No <input type="checkbox"/> Yes	Magnitude (lb/ft <sup>2</sup> ): _____						
Future Wear Surface		<input type="checkbox"/> No <input checked="" type="checkbox"/> Yes	Magnitude (lb/ft <sup>2</sup> ): <i>15</i>						
SIP Forms		<input checked="" type="checkbox"/> No <input type="checkbox"/> Yes	Magnitude (lb/ft <sup>2</sup> ): _____		Concr Sacrificial Wear		<input checked="" type="checkbox"/> No <input type="checkbox"/> Yes	Thk(in): _____	
Utilities		<input checked="" type="checkbox"/> No <input type="checkbox"/> Yes	Magnitude/Description: _____						
Structure Conditions		Deck (58) = <i>5</i>		Super (59) = <i>6</i>		Sub (60) = <i>6</i>			
Notes, additional loads, comments or deviation from general rating practice: <i>Original design allowed for 15 psf future wearing surface. However, bridge rates low and has no current wearing surface; therefore, allowance for future wearing surface was ignored in rating. Rating with "shear on" results in a rating of HS 15.72/26.76. Rating with "shear off" the ratings were HS 19.24/41.98. Per NMDOT policy, the ratings were reported as HS 19.24/33.</i>									
CALCULATION TOOLS OR METHODS USED: <i>Virtis/BRASS</i>									
Quality Control Method, if any—Describe: <i>Two engineers, (Ed Lead and J Ingles) modeled the bridge independently. Then the senior engineer (Lead) compared and converted the bridge models to rate within 2% of each other.</i>									
RATING									
LFR or ASR Rating	Inventory Rating Factor (HS20)		$R_r = 0.962$		<b>LFR INVENTORY RATING</b>		HS <i>19.24</i>		Required
	Operating Rating Factor (HS20)		$R_r = 1.65$		<b>LFR OPERATING RATING</b>		HS <i>33.0</i>		Required
	<b>LEGAL LOADS</b> (OP RATINGS)	Type 3	$R_r =$ _____	NM 2-Axle	$R_r =$ _____	NM 4-Axle	$R_r =$ _____	Required when	
		Type 3-3	$R_r =$ _____	NM 3A-Axle	$R_r =$ _____	NM 5A-Axle	$R_r =$ _____	LFR Operating	
		Type 3S2	$R_r =$ _____	NM 3B-Axle	$R_r =$ _____	NM 5B-Axle	$R_r =$ _____	Rating < HS17	
<b>LRFR HL-93</b>				Inventory $R_r =$ _____		Operating $R_r =$ _____		Optional	
Print Name		Date							
Rated By (P.E. in substantial charge)		<i>Edward Lead</i>		P.E.		<i>01/20/10</i>			
Signature		<i>Edward Lead</i>		Engineering Seal:					
Rating Organization		<i>NMDOT Bridge Load Rating Unit</i>							
NMDOT Accepted		<i>LaVerne Andrews</i>		P.E.		<i>01/20/10</i>			
Signature		<i>La Verne Andrews</i>							
		NMDOT Bridge Load Rating Unit							
PLANS AND FILES									
Electronic Files Used to Model and Rate Bridge Available at, or delivered to: <i>Saved to BRIDGEWARE PRODUCTION</i>									
OR Paper Documentation for Bridge Rating Available at, or delivered to:									
Plans Available at, or delivered to:									
Structure Number: <i>07734</i>									





# APPENDIX A

## Bid Items For Bridges And Structures



**APPENDIX A**  
**BID ITEMS FOR BRIDGES AND STRUCTURES**

<b>SPECIFICATION SECTION</b>	<b>DESCRIPTION</b>	<b>UNIT OF MEASURE</b>	<b>ROUND UP QUANTITY TO</b>
210	Major Structure Excavation	CU.YD.	1
210	Major Structure Backfill	CU.YD.	1
501	Driven Piles	LIN.FT.	1
501	Prebored Holes for Bearing Piles	LIN.FT.	1
502	Drilled Shaft Foundation	LIN.FT.	1
502	Permanent Casing	LIN.FT.	1
502	Steel Shape Reinforcement	LBS.	10
502	Soil Borings	LIN.FT.	1
502	Rock Cores	LIN.FT.	1
502	Obstruction Removal	LIN.FT.	1
503	Micropiles	LIN.FT.	1
506	MSE Wall	SQ.FT.	5
511	Structural Concrete Class ____	CU.YD.	1
511	Structural Concrete Class ____, ____ in	SQ.YD.	1
511	Special Surface Finish	SQ.FT.	1
511	Waterproofing	SQ.FT.	1
512	Superstructure Concrete	CU.YD.	1
512	Ultra High Performance Concrete	CU.YD.	0.1
514	Concrete Barrier Railings for Bridges	LIN.FT.	1
515	Reinforced Concrete for Minor Structures	CU.YD.	1
516	Flowable Fill	CU.YD.	1
518	Prestressed Concrete Bridge Members	LIN.FT.	1
530	Deck Scarification	SQ.YD.	1
530	Deck Preparation	SQ.YD.	1
530	Replacement Reinforcing Bars Grade 60	LBS.	10
531	Layered Bridge Deck Sealing	SQ.YD.	1
532	Penetrating Water Repellent Treatment	SQ.YD.	1

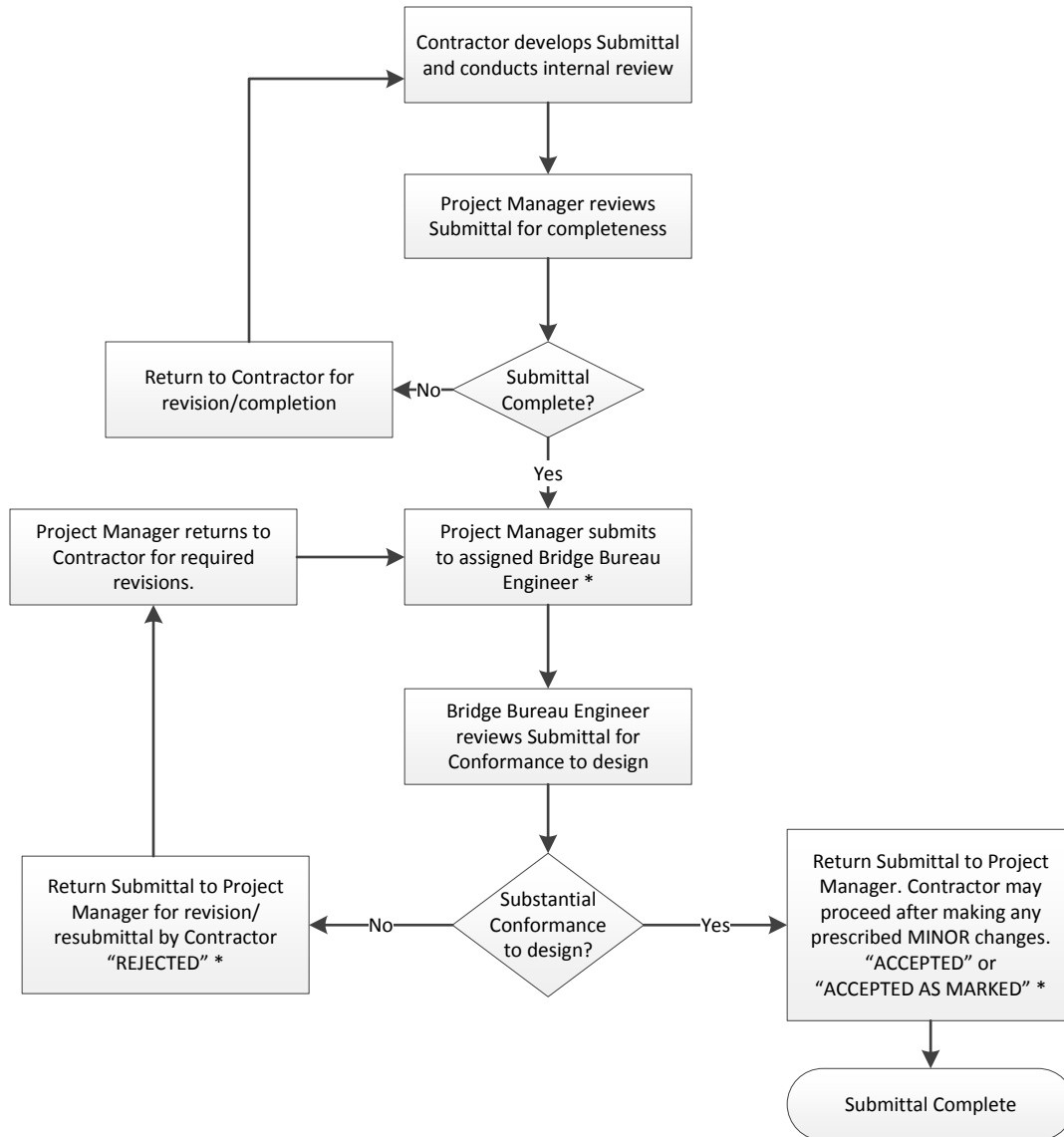
<b>SPECIFICATION SECTION</b>	<b>DESCRIPTION</b>	<b>UNIT OF MEASURE</b>	<b>ROUND UP QUANTITY TO</b>
533	Repair of Concrete Structures	SQ.YD.	1
534	Epoxy Injection	Gallon	1
535	Crack Sealing	LIN.FT.	1
535	Concrete Surface Treatment	SQ.YD.	1
536	Polymer Concrete Bridge Deck Overlay	SQ.YD.	1
540	Reinforcing Bars Grade 60	LBS.	10
541	Structural Steel for Concrete Bridges	LBS.	10
541	Structural Steel for Steel Structures	LBS.	10
543	Metal Railing	LIN.FT.	1
546	SP 3 Power Tool Cleaning and Priming	SQ.FT.	1
546	SP 11 Power Tool Cleaning and Priming	SQ.FT.	1
550	Treated Timber	MBF	1
561	Elastomeric Compression Joint Seals	LIN.FT.	1
562	Bridge Joint Strip Seal	LIN.FT.	1
564	Preformed Closed Cell Foam Bridge Joint Seals	LIN.FT.	1
565	Preformed Silicone-Coated Foam Joint System	LIN.FT.	1
601	Removal of Structures and Obstructions	L.S.	

# APPENDIX B

## Flow Charts for Phase III Services



# Structural / Bridge Bureau Submittal Flow Chart Phase III Service by Bridge Bureau



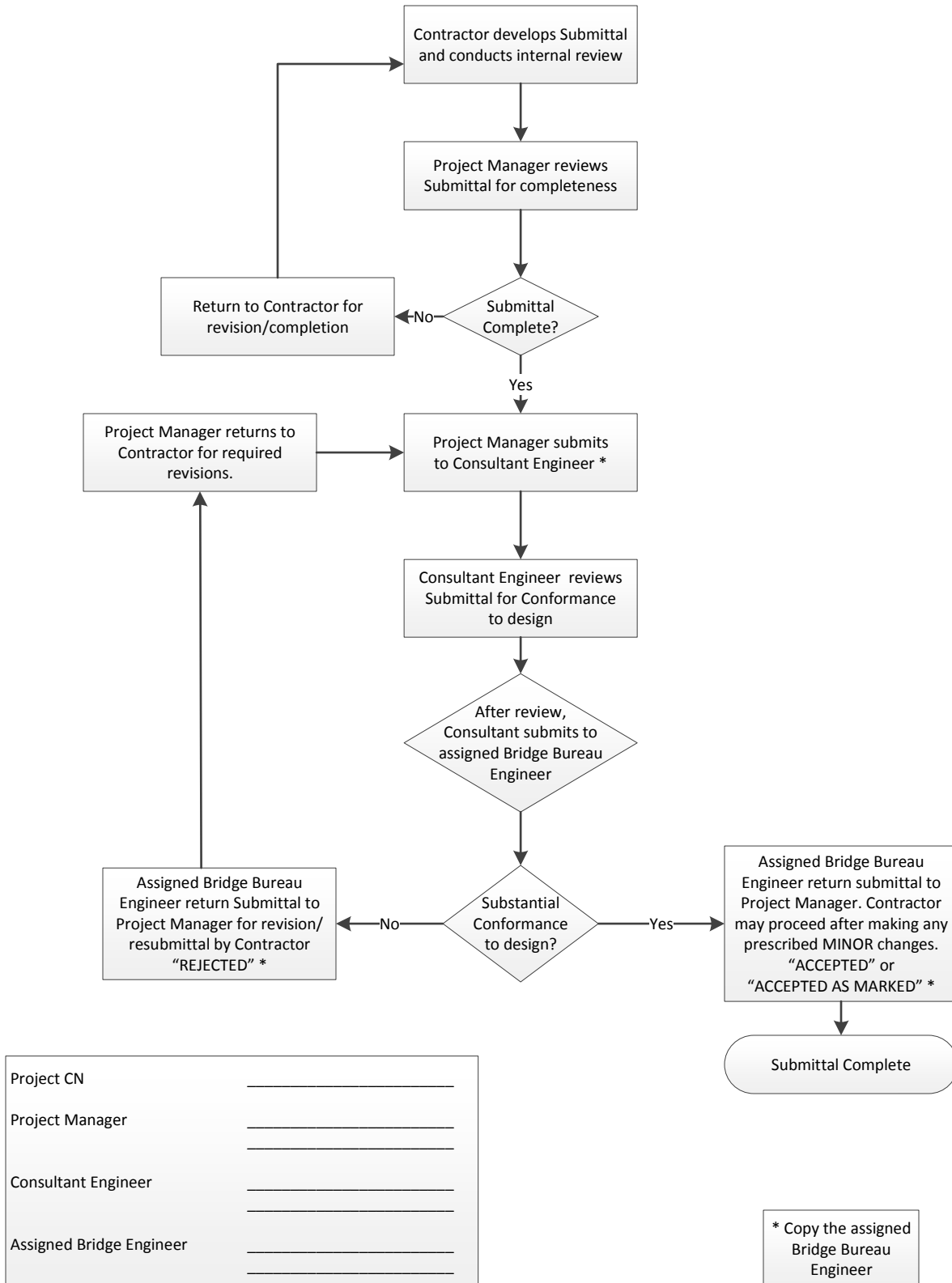
Project CN	_____
Project Manager	_____
Assigned Bridge Engineer	_____
Bridge Design Section Manager	<u>kathy.crowell@state.nm.us</u> <u>505-470-5663</u>

\* Copy the Bridge Design Section Manager





# Structural / Bridge Bureau Submittal Flow Chart Phase III Service by Consultant





# APPENDIX C

## Wisconsin DOT ABC Decision Matrix





% Weight	Category	Decision-Making Item	Possible Points	Points Allocated	Scoring Guidance
17%	Disruptions (on/under Bridge)	Railroad on Bridge?	8	<input type="text"/>	0 No railroad track on bridge 4 Minor railroad track on bridge 8 Major railroad track on bridge
		Railroad under Bridge?	3	<input type="text"/>	0 No railroad track under bridge 1 Minor railroad track under bridge 3 Major railroad track(s) under Bridge
		Over Navigation Channel that needs to remain open?	6	<input type="text"/>	0 No navigation channel that needs to remain open 3 Minor navigation channel that needs to remain open 6 Major navigation channel that needs to remain open
8%	Urgency	Emergency Replacement?	8	<input type="text"/>	0 Not emergency replacement 4 Emergency replacement on minor roadway 8 Emergency replacement on major roadway
23%	User Costs and Delays	ADT and/or ADTT (Combined Construction Year ADT on and under bridge)	6	<input type="text"/>	0 No traffic impacts 1 ADT under 10,000 2 ADT 10,000 to 25,000 3 ADT 25,000 to 50,000 4 ADT 50,000 to 75,000 5 ADT 75,000 to 100,000 6 ADT 100,000+
		Required Lane Closures/Detours? (Length of Delay to Traveling Public)	6	<input type="text"/>	0 Delay 0-5 minutes 1 Delay 5-15 minutes 2 Delay 15-25 minutes 3 Delay 25-35 minutes 4 Delay 35-45 minutes 5 Delay 45-55 minutes 6 Delay 55+ minutes
		Are only Short Term Closures Allowable?	5	<input type="text"/>	0 Alternatives available for staged construction 3 Alternatives available for staged construction, but undesirable 5 No alternatives available for staged construction
		Impact to Economy (Local business access, impact to manufacturing etc.)	6	<input type="text"/>	0 Minor or no impact to economy 3 Moderate impact to economy 6 Major impact to economy
14%	Construction Time	Impacts Critical Path of the Total Project?	6	<input type="text"/>	0 Minor or no impact to critical path of the total project 3 Moderate impact to critical path of the total project 6 Major impact to critical path of the total project
		Restricted Construction Time (Environmental schedules, Economic Impact – e.g. local business access, Holiday schedules, special events, etc.)	8	<input type="text"/>	0 No construction time restrictions 3 Minor construction time restrictions 6 Moderate construction time restrictions 8 Major construction time restrictions
5%	Environment	Does ABC mitigate a critical environmental impact or sensitive environmental issue?	5	<input type="text"/>	0 ABC does not mitigate an environmental issue 2 ABC mitigates a minor environmental issue 3 ABC mitigates several minor environmental issues 4 ABC mitigates a major environmental issue 5 ABC mitigates several major environmental issues
3%	Cost	Compare <i>Comprehensive</i> Construction Costs (Compare conventional vs. prefabrication)	3	<input type="text"/>	0 ABC costs are 25%+ higher than conventional costs 1 ABC costs are 1% to 25% higher than conventional costs 2 ABC costs are equal to conventional costs 3 ABC costs are lower than conventional costs
18%	Risk Management	Does ABC allow management of a particular risk?	6	<input type="text"/>	0-6 Use judgment to determine if risks can be managed through ABC that aren't covered in other topics
		Safety (Worker Concerns)	6	<input type="text"/>	0 Short duration impact with TMP Type 1 3 Normal duration impact with TMP Type 2 6 Extended duration impact with TMP Type 3-4
		Safety (Traveling Public Concerns)	6	<input type="text"/>	0 Short duration impact with TMP Type 1 3 Normal duration impact with TMP Type 2 6 Extended duration impact with TMP Type 3-4
12%	Other	Economy of Scale (repetition of components in a bridge or bridges in a project) (Total spans = sum of all spans on all bridges on the project)	5	<input type="text"/>	0 1 total span 1 2 total spans 2 3 total spans 3 4 total spans 4 5 total spans 5 6+ total spans
		Weather Limitations for conventional construction?	2	<input type="text"/>	0 No weather limitations for conventional construction 1 Moderate limitations for conventional construction 2 Severe limitations for conventional construction
		Use of Typical Standard Details (Complexity)	5	<input type="text"/>	0 No typical standard details will be used 3 Some typical standard details will be used 5 All typical standard details will be used
Sum of Points:			0	(100 Possible Points)	

**Figure 7.2-1**  
ABC Decision-Making Matrix