Chapter 11

PRELIMINARY DESIGN

NDOT STRUCTURES MANUAL

September 2008

Table of Contents

Table of Contents
(Continued)

Table of Contents
(Continued)

Table of Contents
(Continued)

Table of Contents
(Continued)

Section **Page**

Chapter 11
PRELIMINARY DESIGN

This Chapter addresses the preliminary phase of bridge design. It provides guidance to bridge designers in determining the most appropriate overall structure type to meet the structural, geometric, hydraulic, environmental, economical and right-of-way characteristics of the site.

11.1 INTRODUCTION

11.1.1 Objectives of Preliminary Design

Bridge design is accomplished in two equally important phases — preliminary design and final design. Decisions made during preliminary design may significantly impact structure performance, functionality and long-term maintenance.

The preliminary design phase concludes with the selection of a structure type and development of the Structure Front Sheet. The bridge designer must compile the back-up information to support the proposed construction materials, span configuration, superstructure type, abutment type, pier type and location, foundation type, structure dimensions, roadway features, pedestrian features, etc.

The final design process is generally well understood. The preliminary design phase varies from project-to-project. This Chapter assists engineers in preparing preliminary designs for common highway bridges.

Preliminary design includes evaluating many bridge features, which includes the elements of the bridge (foundations, abutments, piers, girders, bearings, expansion joints), materials (concrete, steel) and geometrics (clearances, structure depth, structure width, span lengths). The designer must evaluate each feature to identify the most appropriate selection. High-cost features or those with a “fatal flaw” should be eliminated early in the evaluation process. However, some features may have more than one acceptable solution. This results in the need for an alternatives analysis; see Section 11.8. The superstructure type selection generally drives the need for the alternatives analysis.

11.1.2 Chapter Presentation

In general, this Chapter has been organized to present the decision-making process in preliminary design from the location of the bridge to the structure-type selection for the site.

11.2 BRIDGE LOCATION

Establishing the location of a bridge is an interactive process among NDOT units responsible for roadway, bridge, hydraulics, geotechnical, right-of-way and environment. In addition, District Offices, local governments and the public are involved in determining bridge location. Bridges are an integral part of the transportation system and should be located considering economics, ease of construction, and the minimization of environmental impacts to optimize service to the traveling public. This Section summarizes the significant factors that determine the location of a bridge.

11.2.1 <u>Roadway Design</u>

11.2.1.1 General

Often the considerations for bridge design differ from those for roadway design because the design life of the bridge is 3 to 4 times the design life of the roadway. Roadway design factors which impact bridge location and structure type selection include:

- horizontal alignment (e.g., tangent, curve, superelevation, skew);
- vertical clearances and alignment (e.g., longitudinal gradient, vertical curves);
- traffic volumes;
- roadway width;
- presence of medians, sidewalks and bike lanes; and
- clear zones through underpasses.

The roadway designer establishes the roadway alignment. Ideally, bridges would be located where they are on tangent alignment with no skew, width changes or superelevation transitions. However, project constraints seldom allow this. Bridges are usually located where they fit into the transportation system irrespective of the effect on bridge design and construction. Although bridges can be designed to accommodate almost any given geometry, the bridge designer must work closely with the roadway designer to minimize the adverse effect of some of the following roadway design issues to minimize costs.

Section 11.9 discusses roadway design elements and criteria specifically as they pertain to the roadway design portion of a bridge. Section 11.2.1 discusses roadway design issues specifically as they pertain to bridge location considerations.

11.2.1.2 Horizontal Alignment

Many bridges are constructed on horizontal curves. This complicates the design, geometry and construction of bridges and reduces the number of bridge types that are considered. The analysis of horizontally curved bridges with small radii of curvature requires a refined analysis. Hand-calculation methods are available but are accurate only for horizontally curved bridges of large radii of curvature, and they should be used only as a check of the refined analysis. See Chapter 13 for guidance on acceptable methods of analysis. In general, cast-in-place concrete and structural steel are best suited for horizontally curved bridges.

11.2.1.3 Skew

Skews of less than approximately 30° are acceptable for most bridge types and result in moderate detailing challenges. Some structure types with skews more than 30° may require a refined analysis. All structures with skews of more than 60° should be analyzed by refined methods. Bridges having a high skew may also have long-term functionality problems such as uplifting of girders in the acute corners and/or the bridge bearings translating sideways. Alternatives to these highly skewed bridges should be considered. See Chapter 13 for guidance on acceptable methods of analysis for bridges of varying skew.

11.2.1.4 Vertical Curvature

Vertical curvature is not typically considered in the structural analysis of bridges. The geometry, however, is more difficult, and vertical curvature is reflected in design because the calculated camber for steel and precast concrete girders must be included. All bridges with significant vertical curvature require perpendicular placement of the deck-finishing machine. See Section 11.2.1.7.

11.2.1.5 Variable Width

Most bridges will have a constant width along their entire length. However, ramps and roadway approaches sometimes will extend onto or through a bridge. This can create complex detailing and design challenges. The transitions in bridge width can be either linear or curved. The easiest transitions to detail and design are those that have a linear change in width across the entire length of bridge where the bridge is on a tangent alignment. Detailing and design can become very complex when the bridge is on a horizontal curve with a linear or curved width transition.

11.2.1.6 Superelevation Transition

Superelevation transitions do not create additional structural analysis; however, the geometry is more difficult. Most bridges can be easily constructed with the transitions on a bridge if the transition is constant over the bridge's entire length. Superelevation transitions on only one side of a crown section should be avoided. The deck-finishing machine requires constant adjustment to match the changing roadway crown. All bridges with superelevation transitions require perpendicular placement of the deck-finishing machine.

11.2.1.7 Deck-Finishing Equipment

Deck-finishing machines are used to place, consolidate and finish concrete for bridge decks. Limitations on their use need to be considered in the preliminary design. A deck-finishing machine can finish a deck up to 120 ft in width if the machine is placed perpendicular to the girders. Skews will reduce this width. The roadway geometry, width, width variation, crown, and crown breaks can dictate the deck finishing. Closure pours can be used when multiple longitudinal deck pours are needed. Closure pours must be shown on the contract plans. Locate closure pours where the anticipated traffic wheel lines are away from the longitudinal joints.

11.2.2 Hydraulics

11.2.2.1 General

The Hydraulics Section will prepare a Hydraulics Report or provide preliminary hydraulic recommendations in coordination with the Structures Division's structure-type selection. The critical hydraulic factors may include:

- channel bottom elevation and width;
- water surface elevation for the design-year flood;
- skew angle and side slopes of channel;
- required low-chord elevation;
- bridge scour potential; and
- freeboard required for the passage of debris.

Bridges crossing streams and rivers should be located such that the effects of scour and river meander are minimized. Most river systems in Nevada have the potential for significant scour and meander. Scour is a function of the stream flow, size of bridge opening, pier and abutment locations and widths and soil type. The Geotechnical and Hydraulics Sections will provide preliminary information to determine the potential for scour at each proposed site. Meanders can cause a significant cost increase to the project. Spur dikes and other heavy riverbank armoring are sometimes needed to control a river's meander. The Hydraulics Section should provide preliminary information on the potential for river meander and the cost for its mitigation. Costs for scour and meander mitigations must be included in the evaluation of alternatives if there are differences between the alternatives.

11.2.2.2 Division of Responsibilities

The Hydraulics Section is responsible for hydrologic and hydraulic analyses for both roadway drainage appurtenances and bridge waterway openings. The Hydraulics Section will perform the following for the design of bridge waterway openings for new bridges:

- the hydrologic analysis to calculate the design flow rates based on the drainage basin characteristics;

- the hydraulic analysis to determine the necessary dimensions of the bridge waterway opening to pass the design flood, to meet the backwater allowances and to satisfy any regulatory floodplain requirements; and

- the hydraulic scour analysis to assist in determining the recommended foundation design for the new bridge.

Based on the hydraulic analysis, the Hydraulics Section will provide the following to the Structures Division for new bridges:

- the water surface elevation for the design-year flood and 100-year flood;

- a suggested low-chord elevation;

- the necessary bridge waterway opening dimensions, skew angle, bottom of channel elevation and channel centerline station;

- the results of its hydraulic scour analysis;
- any necessary channel and abutment protection measures; and
- the recommended deck drainage design.

The Hydraulics Section is also responsible for determining that the bridge design is consistent with regulations promulgated by the Federal Emergency Management Agency (e.g., development within regulatory floodplains).

11.2.2.3 Hydraulic Definitions

The following presents selected hydraulic definitions which have an application to bridge design:

1. Auxiliary Waterway Openings. Relief openings provided for streams in floodplains through the roadway embankment in addition to the primary bridge waterway opening.
2. Bridge Backwater Effect. The incremental increase in water surface elevation upstream of a highway facility.
3. Base Flood. The flood having a 1% chance of being exceeded in any given year (i.e., the 100-year event).
4. Base Floodplain. The area subject to flooding by the base flood.
5. Bridge Waterway Opening. The opening provided in the roadway embankment intended to pass the stream flow under the design conditions.
6. Design Flood Frequency. The flood frequency selected for determining the necessary size of the bridge waterway opening.
7. Flood Frequency. The number of times a flood of a given magnitude can be expected to occur on average over a long period of time.
8. Freeboard. The clearance between the water surface elevation based on the design flood and the low chord of the superstructure.
9. Maximum Allowable Backwater. The maximum amount of backwater that is acceptable to NDOT for a proposed facility based on State and Federal laws and on NDOT policies.
10. 100-Year Flood Frequency. A flood volume (or discharge) level that has a 1% chance of being equaled or exceeded in any given year.
11. Overtopping Flood. That flood event that will overtop the elevation of the bridge or roadway approaches.
12. Peak Discharge (or Peak Flow). The maximum rate of water flow passing a given point during or after a rainfall event or snow melt. The peak discharge for a 100-year flood is expressed as Q_{100}.
13. Recurrence Interval (Return Period). For a given discharge, the number of years between occurrences of that discharge. For example, the recurrence interval for a 100-year flood discharge is 100 years.

14. Regulated Floodway. The floodplain area that is reserved in an open manner by Federal, State or local requirements (i.e., unconfined or unobstructed either horizontally or vertically) to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program (NFIP).

15. River Stage. The water surface elevation above some elevation datum.

16. Scour. The action at a bridge foundation in which the movement of the water erodes the channel soil that surrounds the foundation. There are several types of scour:

    a. Contraction. A constriction of the channel (i.e., the flow area) that may be caused, for example, by bridge piers.

    b. Local. Removal of material from around piers, abutments, embankments, etc., due to high local velocities or flow disturbances such as eddies and vortices.

    c. Long-Term. Aggradation and degradation of the stream bed.

17. Thalweg. The path of deepest flow.

18. Standard Flood. The 500-year flood event.

11.2.2.4 Hydraulic Design Criteria

The following summarizes NDOT's basic hydraulic criteria used for the design of bridge waterway openings:

1. Design Flood Frequency. The minimum design flood frequency is based on the roadway classification and ranges from the 10-year event to the 100-year event. The design flood frequency is increased to the 100-year event if necessary to mitigate adverse flood impacts. A 50-year event is typically used for bridges on roadways classified as interstate or principal arterial.

2. Maximum Allowable Backwater. On FEMA-delineated floodways, no backwater may be introduced by the structure. On FEMA-delineated floodplains, 1 ft of maximum backwater may be introduced. For all sites, the maximum allowable backwater shall be limited to an amount that will not result in unreasonable damage to upstream property or to the highway. The Hydraulics Section will determine the allowable backwater for each site.

3. Freeboard. Where practical, a minimum clearance of 2 ft should be provided between the design water surface elevation and the low chord of the bridge to allow for passage of debris. Where this is not practical, the clearance should be established by the bridge and hydraulic designers based on the type of stream and level of protection desired. For example, 6 in may be adequate on small streams that normally do not transport debris. Urban bridges with grade limitations may not provide any freeboard. On bridge replacement projects, efforts should be made to at least match pre-existing low-chord elevations.

4. Scour. The bridge foundation must not fail or be damaged for the scour design event of the 100-year flood. The overtopping flood is used as the design event if less than the

100-year flood. Lesser flood events should be checked if there are indications that less frequent events may produce deeper scour than the 100-year or overtopping flood. The bridge foundation must also be checked using estimated total scour for the lesser of the 500-year or overtopping event. The foundation must not fail while maintaining a minimum geotechnical factor of safety of 1.0 under the appropriate flood scour conditions.

11.2.2.5 Environmental Considerations

See Section 11.2.5 for more discussion.

Many aspects of the environmental assessment with respect to the site are also related to the hydraulic design of a stream crossing. These include the effects on the aquatic life in the stream; effects on other developments, such as a domestic or irrigation water supply intake; and the effects on floodplains.

The evaluation of the typical hydraulic engineering aspects of bridge design are interrelated with environmental impacts. These include the effects of the crossing on velocities, water surface profiles, flow distribution, scour, bank stability, sediment transport, aggradation and degradation of the channel, and the supply of sediment to the stream or water body.

The environmental process for stream crossing projects may also precipitate the need for several State and Federal permits and approvals that are water related. The bridge designer should consider the future requirements for these permits and approvals in the preliminary bridge design stage.

11.2.2.6 Stream Types

The three basic types of streams are braided, straight and meandering.

Hydraulic analysis of braided streams is extremely difficult because of the inherent instability and unpredictable behavior of such streams. Constricting a braided channel into one channel or placing roadway fill between subchannels may change sediment transport capacity at some locations. Where practical, an alternative crossing site at a reach of stream that is not braided should be selected.

A straight reach of stream channel in an otherwise meandering stream may be viewed as a transient condition. Aerial photographs and topographic maps should be examined for evidence of past locations of the channel and of tendencies for meanders to form in the straight reach.

For meandering streams, the outside bank of a bend (i.e., the bank with the longer radius of curvature) presents the greatest hazard to highway facilities because the stream attacks that bank. The design of crossings at bends is complex because it is difficult to predict flood flow distribution. The stream is usually deeper at that bank, velocities are higher, and the water surface is superelevated. The location of a structure in the overbank area may encourage a cutoff and, if the bend system is moving, approach fills and abutments will be subjected to attack as the bend moves downstream.

11.2.2.7 Bridge vs Culvert

In some cases, the waterway opening for a highway-stream crossing can be accommodated by either a culvert or a bridge. Estimates of costs and risks associated with each will indicate which structural alternative should be selected. Figure 11.2-A lists many of the advantages and disadvantages of bridges and culverts. The selection will be a collaborative effort by the bridge designer, Hydraulics Section and the Environmental Services Division. Construction and Maintenance may also become involved.

11.2.2.8 Abutments

The principal hydraulic concerns for abutments are orientation and protection from scour-related failure. Concerns for scour are usually resolved by protective and preventive measures that are identified by the Hydraulics Section. Orientation is usually the same as for adjacent piers.

11.2.2.9 Piers

11.2.2.9.1 *Coordination*

The location of piers in waterways is an interactive process among the Structures Division, Geotechnical Section and the Hydraulics Section. Initially, the Hydraulics Section will determine the required channel geometry to meet the hydraulic criteria (e.g., maximum backwater for 100-year flood). The bridge designer will determine the number and length of spans, types of piers and low-chord elevation. The Hydraulics Section will evaluate the bridge design proposal to determine if it meets the hydraulic requirements of the waterway opening. For example, meeting the hydraulic criteria may require that span lengths be increased. Next, the bridge designer and Geotechnical Section will evaluate potential foundation designs for the pier and provide preliminary design information to the Hydraulics Section for scour analysis. If the resulting foundation design is judged to be too costly, the bridge designer will evaluate reducing the number of piers or eliminating piers altogether based on overall structure costs, environmental impacts, constructibility, etc.

The highway profile (i.e., vertical alignment and bridge end elevations) is an additional highway design element in the iterative process to identify the number and location of piers in waterways. The profile can have a significant impact on the overall bridge opening and floodplain flow conditions. The Roadway Design Division may prefer, for example, to lower the highway profile due to significant right-of-way impacts which, all other factors being equal, reduces the hydraulic capacity of the waterway opening and increases the frequency of overtopping.

Ultimately, all of these factors (i.e., structural, hydraulic, geotechnical, roadway, environmental, costs) must be evaluated to identify the optimum number and location of piers.

11.2.2.9.2 *Costs*

Economy of construction is usually a significant consideration in the determination of spans, pier locations and orientation, and substructure and superstructure design. Construction costs are always a factor in the structural design of a bridge to ensure the use of economically available structural materials, but the cost of construction is only one element of the total economic cost of a stream crossing system. There are hydraulic considerations, maintenance costs and risks of future costs to repair flood damages that should also be factored into the decision on the number of piers and their location, orientation and type.

Bridges	
Advantages	*Disadvantages*
Less susceptible to clogging with drift, ice and debris. Waterway increases with rising water surface until water begins to submerge superstructure. Scour increases waterway opening. Flowline is flexible. Minimal impact on aquatic environment and wetlands. Widening does not usually affect hydraulic capacity. Capacity increases with stage.	Requires more structural maintenance than culverts. Abutment fill slopes susceptible to erosion and scour damage. Piers and abutments susceptible to failure from scour. Susceptible to ice and frost formation on deck. Bridge railing and parapets hazardous as compared to recovery areas. Deck drainage may require frequent maintenance cleanout. Buoyancy, drag and impact forces are hazards to bridges. Susceptible to damage from stream meander migration.
Culverts	
Advantages	*Disadvantages*
More roadside recovery area can be provided. Grade raises and widening projects sometimes can be accommodated by extending culvert ends. Requires less structural maintenance than bridges. Frost and ice usually do not form before other areas experience the same problems. Capacity increases with stage. Capacity can sometimes be increased by installing improved inlets. Usually easier and quicker to build than bridges. Scour is localized, more predictable and easier to control. Storage can be utilized to reduce peak discharge. Avoids deep bridge foundations.	Multiple barrel culverts, whose width is considerably wider than the natural approach channel, may silt in and may require periodic cleanout. No increase in waterway as stage rises above soffit. May clog with drift, debris or ice. Possible barrier to fish passage. Susceptible to erosion of fill slopes and scour at outlets. Susceptible to abrasion and corrosion damage. Extension may reduce hydraulic capacity. Inlets of flexible culverts susceptible to failure by buoyancy. Rigid culverts susceptible to separation at joints. Susceptible to failure by piping and/or infiltration.

BRIDGE vs CULVERT
(Hydraulics)

Figure 11.2-A

11.2.2.9.3 *Design Factors*

The number of piers in any channel should be limited to a practical minimum, and piers in the channel of small streams should be avoided, if practical. The cost of construction of a pier in the water increases with increasing water depth. Piers properly oriented with the flow do not contribute significantly to bridge backwater, but they do contribute to general scour. In some cases, severe scour can develop immediately downstream of bridges because of eddy currents and because piers occupy a significant area in the channel. Lateral and vertical scour also occur at some locations.

Piers should be aligned with flow direction at flood stage to minimize the opportunity for drift to be caught in piling or columns, to reduce the contraction effect of piers in the waterway, to minimize debris forces and the possibility of debris dams forming at the bridge, and to minimize backwater and local scour. Pier orientation is difficult where flow direction changes with stage or time. Circular piers, or some variation thereof, may be the best alternative if orientation at other than flood stage is critical.

Piers located on a bank or in the stream channel near the bank are likely to cause lateral scouring of the bank. Piers located near the stream bank in the floodplain are vulnerable because they can cause bank scour. They are also vulnerable to failure from undermining by meander migration.

Pier shape is also a factor in local scour. A solid pier (e.g., a pier wall) will not collect as much debris as a pier bent or a multi-column pier. Rounding or tapering the leading edges of piers helps to decrease the accumulation of debris and reduces local scour at the pier.

11.2.2.10 Foundations

The foundation is usually the element of a bridge that is most vulnerable to attack by floods. Examination of individual boring logs and plots of the profiles of various subsurface materials are important to the prediction of potential scour depths and to the estimation of the bearing capacity of the soils.

Driven piles or drilled shafts usually depend upon the surrounding material for skin friction and lateral stability. In some cases, they can be extended to rock or other dense material for load-carrying capacity through tip resistance. Tip elevations for piling or drilled shafts should be based on estimates of potential scour depths and bearing to avoid losing lateral support and load-carrying capacity during floods.

The bridge designer must consider the potential scour and the possibility of channel shifts in designing foundations for bridges on floodplains and spans approaching the stream channel. The thalweg (i.e., the line or path connecting the lowest flow points along the channel bed) should not be considered to be in a fixed location when establishing founding elevations. The history of a stream and a study of how active it has been can be useful in making decisions on pile and drilled shaft tip elevations.

11.2.2.11 Bridge Deck Drainage

See Section 16.4.

11.2.3 Geotechnical

Foundations can be either shallow (spread footing) or deep (driven piles and drilled shafts). Shallow foundations are significantly less expensive than deep foundations. The Geotechnical Section provides preliminary foundation information for a proposed site. Bridges located at different sites have different foundation requirements. These differences must be included in an alternative's cost. In addition, the structure type can be influenced by the supporting soils. Heavy bridges such as cast-in-place concrete box girders can require a substantially larger foundation compared to a structural steel bridge. See Section 11.7.

11.2.4 Right-of-Way

Right-of-way and utilities have a significant influence on most projects. Their cost can be as high as the cost of construction; therefore, the location of a bridge must be carefully considered. In addition, right-of-way acquisition and utility relocations can require a significant timeframe to complete. The Right-of-Way Division can provide preliminary estimates on the number of properties and utilities for each bridge location. The Division will provide estimates on cost, number of properties and utilities encountered, possible difficult acquisitions, and approximate time frames. In addition to property acquisition, most projects require temporary and permanent easements for construction staging areas, access, future maintenance and actual construction. These must also be considered when evaluating alternatives.

The designer should consider the following right-of-way factors when selecting the structure type:

1. Expensive Right-of-Way. If right-of-way will be expensive, this may lead to the use of retaining walls and other measures to reduce right-of-way impacts.

2. Structure Depth. The available right-of-way at the bridge site may affect the vertical alignment of the structure which may, in turn, affect the acceptable structure depth to meet the vertical clearance requirements. The depth of the superstructure is a significant issue in urban areas. Right-of-way acquisition costs are high, and roadway profiles cannot usually be raised due to access rights on approaches. All costs including approach costs, right-of-way acquisition, easements, etc., for each alternative must be included in an alternatives analysis.

3. Detours. For bridge widening projects, if right-of-way is not available for detours, it may be necessary to maintain traffic across the existing bridge during widening.

Any bridge design must be consistent with NDOT utility accommodation policies. Section 16.5.4 discusses utility attachments to bridges.

11.2.5 Environmental

11.2.5.1 General

The evaluation of potential environmental impacts can have a significant impact on bridge location, structure-type selection and configuration, especially for highway bridges over streams. In general, any bridge project should, within reason, attempt to minimize the environmental impacts, especially in sensitive areas (e.g., wetlands, endangered species habitat). The Environmental Services Division is responsible for identifying all environmental resources within the proposed project limits and for evaluating the potential project impacts on these resources.

In addition, the Environmental Services Division is responsible for ensuring that the State and Federal requirements for public involvement are met.

Cultural resources, endangered species (including plants and habitat), and other environmental concerns must be identified at each bridge location. Almost all current NDOT right-of-way has been cleared for cultural resources and endangered species. Although projects within NDOT right-of-way may not need extensive evaluation, these projects could still have environmental issues. The proposed location of the bridge can be of no consequence, require some form of mitigation, or be so significant that the bridge must be moved to another location. See the *NDOT Environmental Services Manual* for a detailed discussion on environmental considerations and permits.

The following Sections discuss specific environmental impacts and actions that may be required for a bridge project.

11.2.5.2 Environmental Class of Action

For every NDOT project, the Environmental Services Division will determine the Environmental Class of Action. Based on the results of the evaluation of project impacts and the nature and scope of the proposed project, the Division will determine the level of NEPA compliance processing for the project. This will be one of the following:

1. Categorical Exclusion. A Categorical Exclusion (CE) is issued for categories of projects that do not individually or cumulatively have a significant effect on the environment and, therefore, do not require the preparation of an EA or EIS.

2. Environmental Assessment. An Environmental Assessment (EA) is prepared for projects for which the significance of the environmental impact is not clearly established.

3. Environmental Impact Statement. An Environmental Impact Statement (EIS) is prepared for projects where it is known that the action will have a significant effect on the environment. Mitigation measures are typically required to be incorporated into the proposed improvements if environmental impacts are deemed significant.

11.2.5.3 Permits/Approvals

A proposed bridge project may precipitate the need for one or more environmental permits or approvals. Except for floodplains and permits with the Tahoe Regional Planning Agency (TRPA) (which are the responsibility of the Hydraulics Section), the Environmental Services Division is responsible for coordinating with the applicable Federal or State agency and acquiring the permit or approval. This will require considerable coordination with the Structures Division. The following sections briefly discuss these permits/approvals.

11.2.5.3.1 *US Army Corps of Engineers Section 404 Permit*

The Section 404 Permit is required for the discharge of dredge or fill material into any waters of the United States, including wetlands. The purpose of Section 404 is to restore and maintain the chemical, physical and biological integrity of the Nation's waters through the prevention, reduction and elimination of pollution.

11.2.5.3.2 *Section 401 Water Quality Certification*

Pursuant to Section 401 of the Clean Water Act, the Section 401 Water Quality Certification is issued by the Nevada Division of Environmental Protection (NDEP) based on regulations issued by the US Environmental Protection Agency. The purpose of the Section 401 Certification is to restore and maintain the chemical, physical and biological integrity of the Nation's waters through prevention, reduction and elimination of pollution. A Section 401 Certification (or waiver of Certification) is required in conjunction with any Federal permit (e.g., a Section 404 Permit) to conduct any activity that may result in any discharge into waters of the United States.

11.2.5.3.3 *Section 402 NPDES Permit*

Pursuant to Section 402 of the Clean Water Act, the Section 402 National Pollutant Discharge Elimination System (NPDES) Permit is issued by NDEP based on regulations issued by the US Environmental Protection Agency. The purpose of the Section 402 Permit is to restore and maintain the chemical, physical and biological integrity of the Nation's waters through prevention, reduction and elimination of pollution.

11.2.5.3.4 *US Coast Guard Section 9 Permit*

Pursuant to Section 9 of the Rivers and Harbors Act of 1899, the Section 9 Permit is issued by the US Coast Guard. The purpose of the Section 9 Permit is to protect and preserve the navigable waterways of the United States against any degradation in water quality. The Permit is required for structures or work (other than bridges or causeways) affecting a navigable waterway (tidal or non-tidal). Examples of work include dredging, channelization and filling. The Colorado River in Southern Nevada is the only river in the state that requires a Section 9 Permit.

11.2.5.3.5 *Floodplains Encroachment*

Pursuant to Executive Order 11988 "Floodplain Management," NDOT must seek approval from FEMA for any Federally funded/regulated project that produces a significant floodplain encroachment. If a project will have a significant floodplain encroachment, the project will require either an EA or EIS. A proposed action that includes a significant floodplain encroachment will not be approved unless FHWA finds (pursuant to 23 CFR650A) that the proposed action is the only practical alternative.

In addition, NDOT must secure a TRPA Permit when working in the Tahoe basin.

11.2.5.4 Historic Bridges

Based on Section 106 of the National Historic Preservation Act of 1966 (as amended), NDOT must consider the effects of the project on properties included in or eligible for inclusion in the National Register of Historic Places (NRHP). Where such properties will be affected, the Advisory Council on Historic Preservation (ACHP) must be afforded a reasonable opportunity to comment on the undertaking. NDOT must implement special efforts to minimize harm to any property on or eligible for the NRHP that may be adversely affected by the proposed project. The mitigation is accomplished through written agreements among NDOT, the ACHP and the Nevada State Historic Preservation Officer (SHPO). This applies not only to historic bridges, but to other historic properties within and adjacent to NDOT right-of-way.

11.2.5.5 Hazardous Waste

The Hazardous Waste Section within the Environmental Services Division is responsible for identifying and evaluating hazardous waste sites and for determining the needed mitigation measures. Three specific types of hazardous waste that may require treatment for a bridge project include:

1. Paint Removal. Removal of paint from steel bridges that may contain heavy metals or from concrete bridges that may contain asbestos.

2. Fine Surface Finish. This type of concrete finish may contain asbestos or heavy metals.

3. Timber Removal. Salvaging or disposing of timber, from an existing bridge, that may contain creosote or other wood preservative.

4. Plates. Asbestos blast plates on railroad overpasses.

11.2.5.6 Construction

For information on construction-related environmental impacts, reference the following NDOT publications:

- *Planning and Design Guide*; and

- *Construction Site Best Management Practices*, which discusses temporary stream crossings and clear water diversions.

11.2.5.7 Other Environmental Impacts

Occasionally, a proposed bridge project may precipitate other environmental impacts. These include Section 4(f), Section 6(f), Section 106 (other than historic bridges) and threatened and endangered species. Contact the Environmental Services Division for more information.

11.2.6 District

The District Office must be consulted when selecting a bridge location. District input is typically via a review of the Type and Size Report (TSR) and/or Front Sheet. District maintenance and construction personnel can assist with local knowledge of the area including potential political issues, usage of the roadway, possible detours, effect of falsework on the transportation system and other issues. The District Office must also approve non-standard temporary vertical clearances on State Highways.

11.2.7 Local Governments

Local agencies are involved in most projects. Coordination is usually with the local agency's engineering staff, planning staff and/or consultant. Presentations to the local government's elected officials are necessary on some projects. The local government input should be considered but, in most cases for projects on the State Highway System, the final decision is made by NDOT. However, some projects include local funding or replacement of a locally owned bridge. In these cases, local input and approval is necessary. Most local governments have limited funding, and enhancements beyond the minimum requirements allowed in the Nevada Bridge Program cannot usually be included. Local agencies must also approve non-standard temporary vertical clearances on their roads.

11.2.8 General Public

Input from the general public will be sought at informational meetings and public hearings. The information collected at these meetings should be considered, but the final decision is made by NDOT. Formal public hearings completed under NEPA have greater weight than an informational meeting. The Environmental Services Division should be contacted concerning the comments received at public hearings to discuss the relevance of the comments and the required action to be taken.

11.3 SPAN LENGTH AND CONFIGURATION

11.3.1 General

The total required length of a bridge is, in most cases, fairly easy to determine. Determining the optimum number of spans is more difficult. This depends upon the:

- roadway profiles;
- vertical clearances;
- construction requirements (e.g., river diversions, falsework openings);
- environmental factors;
- depth of structure;
- allowable locations of piers;
- foundation conditions;
- waterway opening requirements;
- safety of underpassing traffic;
- navigational requirements; and
- flood debris considerations.

Initially, the bridge designer should consider using a single span. This is usually ideal for most moderate length bridges. Spans up to 225 ft are achievable using cast-in-place, post-tensioned box girders or structural steel plate girders. However, for these span lengths, a fairly deep structure is required, which increases approach roadway costs. The additional approach roadway costs must be included in an alternatives analysis. See Section 11.8.

11.3.2 Waterway Crossings

Abutments for bridges crossing streams and rivers are usually placed at the banks of the river so that the bridge does not affect flow. In addition, abutments can also be placed a sufficient distance back from the edge of the banks (outside the "ordinary high-water elevation") to keep excavations, backfill and riprap out of the river eliminating the need for a US Army Corps of Engineers Section 404 permit. Generally, Section 404 permits are easy to obtain, but some river systems have construction windows, endangered species and water quality requirements that greatly restrict construction activities below the "ordinary high-water elevation."

Piers are expensive, time consuming to construct and reduce the hydraulic opening. Their use should be minimized. Each pier is assumed to collect debris during a flood, which further reduces the hydraulic opening and increases scour. However, more supports allow for a shallower superstructure depth. Streams and rivers almost always require deep foundations. A bridge with foundations that remain out of the water greatly reduces foundation costs and can in many cases be the least cost alternative. Access into a stream or river (usually through adjacent property), pile drilling and driving equipment logistics, river diversions, settling basin requirements, environmental restrictions and risk of flooding greatly increase the cost of placing a support or multiple supports in a stream or riverbed.

See Section 11.2.2 for a further discussion on hydraulic considerations for bridge design.

11.3.3 Highway Crossings

Highway bridges over other highways should have their abutments set based on the anticipated future width requirements of the highway beneath the bridge. The number of lanes and shoulder widths are based on 20-year traffic projections. However, the *LRFD Specifications* provides a 75-year design life for bridges. Traffic projections to 75 years are highly speculative; however, some provision for future widening beyond 20 years should be considered. Open abutments can accommodate the future construction of a retaining wall to increase the width of roadway under the bridge. Locate the abutment a sufficient distance back to allow for the placement of a conventional retaining wall. The excavation for a future retaining wall should not influence the active pressure of the abutment spread footing. If the retaining wall influences the abutment spread footing, a tieback (ground anchor) retaining wall will be required. Section 11.9.6.1 discusses this further.

Bridges in rural areas generally do not need consideration for widening beyond 20 years, and additional span length should not be included. However, in urban areas, the roadway under a bridge may be widened several times over the life of the bridge. The Planning Division and Roadway Design Division may provide an estimate of the potential maximum build out in a community. The number of piers should be minimized with consideration given to clear spanning. No pier should be placed in the area of potential widening between the abutment and roadway. Most highway-over-highway bridges can accommodate a pier in the median because medians are also used for barrier rail, lighting and sign supports. Consistency of structure type along a corridor should also be considered.

11.3.4 Railroad Crossings

The rationale for the location of abutments for highway bridges over railroads is similar to highway-over-highway bridges. Pier locations, however, are different. Generally, the railroad requires that its tracks and maintenance roads be clear spanned. Typically, three-span and single-span bridges are built over railroad facilities. See Section 21.1 for more information.

11.3.5 Urban Bridges and Structure Depth

The depth of the superstructure is a significant issue in urban areas. Right-of-way acquisition costs are high, and roadway profiles cannot usually be raised due to access rights on approaches. All costs, including approach roadway costs, right-of-way acquisition, easements, etc., for each alternative must be included in an alternatives analysis.

11.3.6 Cantilever End Spans

Bridges with cantilever end spans are structurally efficient, have shallow superstructure depths, and require only a small retaining wall to support the approach fill. However, they have demonstrated poor performance and should be avoided. The end spans move up and down with seasonal temperature changes and may have a permanent movement upward due to long-term superstructure creep and shrinkage. This creates a “bump” at certain times of the year that can be a hazard. Long-term maintenance problems often develop due to the need for overlays and expansion joint reconstruction.

11.4 GENERAL DESIGN CONSIDERATIONS

As discussed in this Section, the bridge designer must evaluate certain general design factors in the selection of the structure type and size.

11.4.1 Definition of Terms

11.4.1.1 Substructure vs Foundation

Foundations include the supporting rock or soil and those bridge elements that are in direct contact with, and transmit loads to, the supporting rock or soil. In the *NDOT Structures Manual*, this definition will be used. Typically, foundations include piles, drilled shafts, spread footings and pile caps.

11.4.1.2 Substructure vs Superstructure

The *NDOT Structures Manual* will refer to the substructure as any component or element (not including the foundation) supporting the bearings. The superstructure then consists of the bearings and all of the components and elements resting upon them. For those supports without bearings (e.g., integral abutments, fixed columns to integral caps), the distinction between superstructure and substructure can be blurred.

11.4.1.3 Concrete Slab vs Bridge Deck

A concrete slab (whether cast-in-place/precast or conventionally reinforced/post-tensioned) refers to a superstructure consisting solely of a concrete slab constructed without any supporting girders. The *LRFD Specifications* refers to this as a slab superstructure. A bridge deck refers to a concrete slab supported on longitudinal or transverse supporting components (e.g., girders, beams).

11.4.2 Live-Load Deflection Criteria

Reference: LRFD Articles 2.5.2.6.2 and 2.5.2.6.3

11.4.2.1 General

The *LRFD Specifications* states that the traditional live-load deflection criteria is optional for bridges both with and without sidewalks because static live-load deflection is not a good measure of dynamic excitation. Nonetheless, in the absence of a better criterion and because of durability concerns, NDOT has determined that it is appropriate to limit live-load deflections. The live-load deflection criteria of the *LRFD Specifications* are calibrated to yield comparable results for the HL-93 notional live-load model as the provisions of the *Standard Specifications for Highway Bridges* with the HS20-44 live-load model. Therefore, NDOT mandates the optional live-load deflection check.

11.4.2.2 Criteria

The bridge designer shall limit the live-load deflections to the span-length-based criteria of Section 11.5.1.4 and considering the presence or the absence of pedestrian traffic. The minimum superstructure depth limits of Section 11.5.1.4 shall also be met. Section 11.5.1.4 provides limits based upon provisions of the *LRFD Specifications* for NDOT-specific superstructures types.

11.4.3 Continuous vs Simple Spans

In general, continuous structures provide superior structural performance when compared to bridges with simple spans and joints, and their use is strongly recommended for multi-span bridges. However, in rare cases, it may be appropriate to use simple spans (e.g., widenings of existing simple-span bridges), where high differential settlements are anticipated, and for longer spans or other geometric constraints. Back-to-back multiple simple spans should be avoided, if possible.

11.4.4 Jointless Bridges

NDOT prefers to use, where possible, integral or dozer abutments. When such abutments are used and when estimated total abutment movements are less than ½ in, the approach slab may be tied directly to the abutment (no joint). In this case, provision for bridge movement shall be made at the roadway end of approach slabs. For greater movements with integral or dozer abutments and for all other abutment types, an expansion joint shall be used at the bridge end of approach slabs.

11.4.5 Slab-on-Girder Bridges

The following discussion on slab-on-girder bridges does not apply to cast-in-place, post-tensioned concrete box girders.

11.4.5.1 Composite Action

Reference: LRFD Articles 4.5.2.2 and 9.4.1

Composite action enhances the stiffness and economy of girder bridges by using the bridge deck as an integral part of the girder cross section. NDOT policy is that all bridge decks and their supporting members shall be made fully composite throughout the entire span of the bridge, in both positive and negative moment regions. Thus, the shear connectors and other connections between decks and their supporting members shall be designed to develop full composite action. In other words, the shear connections must be able to resist the horizontal interface shear at the nominal resistance of the section.

The stiffness characteristics of composite girders shall be based upon full participation of the effective width of the concrete deck in the positive moment regions. Composite concrete bridge decks shall be considered uncracked throughout the span for the determination of moments and shears for Service and Strength limit states in structural analysis.

11.4.5.2 Number of Girders

Because of concerns for redundancy, new bridges shall have a minimum of four girders per span. An exception is for narrow bridges on low-volume roads where a minimum of three girders may be used.

The cost of a girder bridge increases with the number of girders in the cross section. Conversely, structure redundancy increases with the number of girders. The basic objective is to identify a girder spacing and corresponding number of girders that optimizes the design of the superstructure by providing sufficient redundancy with minimal cost. In addition, the designer should consider the structural implications on maintaining traffic across the bridge during future operations to redeck or widen the bridge.

11.4.5.3 Interior vs Exterior Girders

Reference: LRFD Article 4.6.2.2.1

To simplify future bridge widenings and for economy of fabrication, all girders within a span should be designed identically to the governing condition, either interior or exterior girder. This also eliminates the possibility of misplacement during construction.

11.4.6 <u>Seismic Requirements</u>

Reference: LRFD Articles 3.10, 4.7.4, 5.10.11, A10.1-3 and 11.6.5

The bridge designer shall incorporate the seismic requirements of the *LRFD Specifications* with the selection of a superstructure, substructure or foundation type. The seismic demand of the bridge and the flexibility/stiffness of the bridge are coexistent. Therefore, the structure-type selection must satisfy the seismic performance, ductility requirements, plastic hinge location, etc., as specified in the *LRFD Specifications*.

Ideally, bridges should have a regular configuration so that seismic behavior is predictable and so that plastic hinging is promoted in multiple, readily identifiable and repairable yielding components. Selecting a structural form based solely on gravity-type loading considerations and then adding seismic-resistive elements and details is unlikely to provide the best solution.

Although the *LRFD Specifications* seismic provisions do not discuss preliminary structure-type selection, certain guidelines should be followed. In general, structure type should be selected with the following considerations:

1. <u>Alignment</u>. Straight bridges are preferred because curved bridges can lead to unpredictable seismic response.

2. <u>Substructure Skew</u>. Substructure units should have little or no skew. Skewed supports cause rotational response with increased displacements.

3. <u>Superstructure Weight</u>. Superstructure weight should be minimized.

4. <u>Joints</u>. The deck should have as few expansion joints as practical.

5. <u>Foundations</u>. Shallow foundations should be avoided if the foundation material is susceptible to liquefaction.

6. Substructure Stiffness. Large differences in the stiffness of the substructure units should be avoided. Seismic forces should be uniformly distributed to all substructure units. This can be accomplished by varying the cross section, providing isolation casings, or strategically locating pinned vs fixed column ends.

7. Plastic Hinges. The formation of plastic hinges should be avoided if at all possible. Where their formation is unavoidable, plastic hinges should be forced to develop only in the columns rather than the cap girders, superstructure or foundations, and they should be accessible for inspection and repair after an earthquake.

11.4.7 Approach Slabs

Approach slabs are required on all bridges.

11.4.8 Foundation Considerations

The following applies, in general, to foundation considerations:

1. Grade Adjustment. When considering structure-type selection, the ability to adjust the structure through jacking is an important issue, which is required by LRFD Article 2.5.2.3. Jacking stiffeners or diaphragms may be required. The subgrade may settle differently from the calculated estimates. It is understood that, where superstructures and substructures are integral with each other, this facilitation for adjustment cannot exist.

   The nature of the subgrade should be considered prior to the final selection and design of the superstructure, substructure and foundation to ensure adjustability if needed.

2. Settlement Limits. Experience demonstrates that bridges can accommodate more settlement than traditionally allowed in design due to creep, relaxation and redistribution of force effects. LRFD Article 10.6.2.2.1 mandates that settlement criteria be developed consistent with the function and type of structure, anticipated service life and consequences of unanticipated movements on service performance. Further, in the commentary it suggests that longitudinal angular distortions between adjacent spread footings greater than 0.008 radians in simple spans and 0.004 radians in continuous spans should not be ordinarily permitted.

11.4.9 Aesthetics

Reference: LRFD Article 2.5.5

11.4.9.1 General

Structures should be aesthetically pleasing to the traveling public. The *LRFD Specifications* emphasizes and NDOT encourages the objective of improving the appearance of highway bridges in the State. The Landscape/Aesthetics Section has developed the *NDOT Landscape and Aesthetics Master Plan*, which presents NDOT's policies, procedures and practices for incorporating aesthetic features into NDOT projects. The following discussion presents a brief overview on aesthetic practices for bridges. See the *Master Plan* for more information.

11.4.9.2 Aesthetic Classifications for Structures

The following classifications present a common language for the aesthetics of highway structures, in order from the least sophisticated application to the most sophisticated:

- Standard Structures,
- Accentuated Structures,
- Focal Structures, and
- Landmark Structures.

11.4.9.3 Design Guidelines for Bridges

The bridge designer should adhere to the following design guidelines for aesthetic treatments of bridges:

- Use a consistent bridge design.
- Use simple substructure and support features.
- Use visually light bridge barrier railings.
- Consider fill embankments and approach barriers as part of the bridge design.
- Use landscape or rock mulch to stabilize embankments.
- Select vandalism-resistant finishes.
- Create a visual design unity among all existing and new structures.
- Integrate landscape and aesthetics at the onset of project planning.
- Use a uniform, consistent color palette for all highway structures, and ensure that accent colors highlight structural aspects.

11.4.10 <u>Construction</u>

11.4.10.1 General

The *LRFD Specifications* requires that, unless there is a single obvious method, at least one sequence of construction should be indicated in the contract documents. If an alternative sequence is allowed, the contractor should prove that stresses that accumulate in the structure during construction will remain within acceptable limits.

11.4.10.2 Access and Time Restrictions

Water-crossing bridges will typically have construction restrictions associated with their construction. These must be considered during structure-type evaluation.

The time period that the contractor will be allowed to work within the waterway may be restricted by regulations administered by various agencies. Depending on the time limitations, a bridge with fewer piers or faster pier construction may be more advantageous even if more expensive.

11.4.10.3 Staged Construction

Occasionally, due to the proximity of existing structures or a congested work area, it may be necessary to build a structure in multiple stages. The arrangement and sequencing of each stage of construction is unique to each project, and the bridge designer must consider the requirements for adequate construction clearances and the requirements of the traveling public. If staged construction is required, then a staging sequence and controlling lane/construction dimensions must be shown in the contract documents.

11.4.10.4 Construction Costs

Initial construction cost is one factor in the selection of the structure type, but not the only factor. Future expenditures during the service life of the bridge should also be considered. The initial costs depend on a variety of factors including:

- type of structure,
- economy of design,
- market conditions,
- experience of local contractors,
- vicinity of fabrication shops, and
- local availability of structural materials and labor.

These factors may change rapidly, and the designer may have no control over them. It may be advisable to prepare competitive plans (i.e., for both concrete and steel superstructures) occasionally even for small-span structures. A review of Post-Construction Reports on completed bridges may avoid future errors.

11.4.10.5 Falsework

Temporary falsework is an expensive construction item. If the bridge is over a waterway and/or will have a high finished elevation, the cost of the falsework may become prohibitive, and the designer should consider other structural systems.

The following will apply to the use of falsework:

1. Railroads. Each railroad company has its own requirements for falsework over its facilities. Depending on the railroad company and the type and amount of railroad traffic, the railroad company may prohibit the use of falsework. The railroad company should be contacted early in project development to determine if falsework may be used and its minimum clearance requirements. See Chapter 21 for more information.

2. Environmental. Some sites may be very environmentally sensitive, and the use of falsework may be prohibited.

3. Hydraulics. For falsework over a waterway, the Hydraulics Section will provide the minimum falsework opening dimensions.

4. Traffic Impacts. Constructing falsework over traffic poses a number of risks. Installing and removing falsework requires extended lane closures or expensive traffic cross-overs. Vehicular impacts to falsework can pose a hazard to the traveling public and construction works. Impact girders that protect the falsework can be constructed on low-volume roads with low posted speed limits. However, the impact girder itself may also

be a hazard. Increasing the vertical clearance to the falsework and using an over-height detection system are more positive methods to reduce risk. See Section 11.9.6.3.

5. Traffic Capacity. The volume and composition of traffic will impact the falsework opening, which must provide sufficient capacity to accommodate the traffic flow.

Figure 11.4-A presents a schematic of a typical falsework opening.

11.4.10.6 Drainage

Refer to Section 3.3.10 of the *NDOT Drainage Manual* for drainage considerations during construction.

11.4.11 Maintenance and Durability

The structure-type selection will, over the life of the structure, have a major impact on maintenance costs. Based on type of material, the following is the approximate order of desirability from a maintenance perspective:

- prestressed concrete,
- reinforced concrete,
- unpainted weathering steel, and
- painted structural steel.

The following maintenance considerations apply:

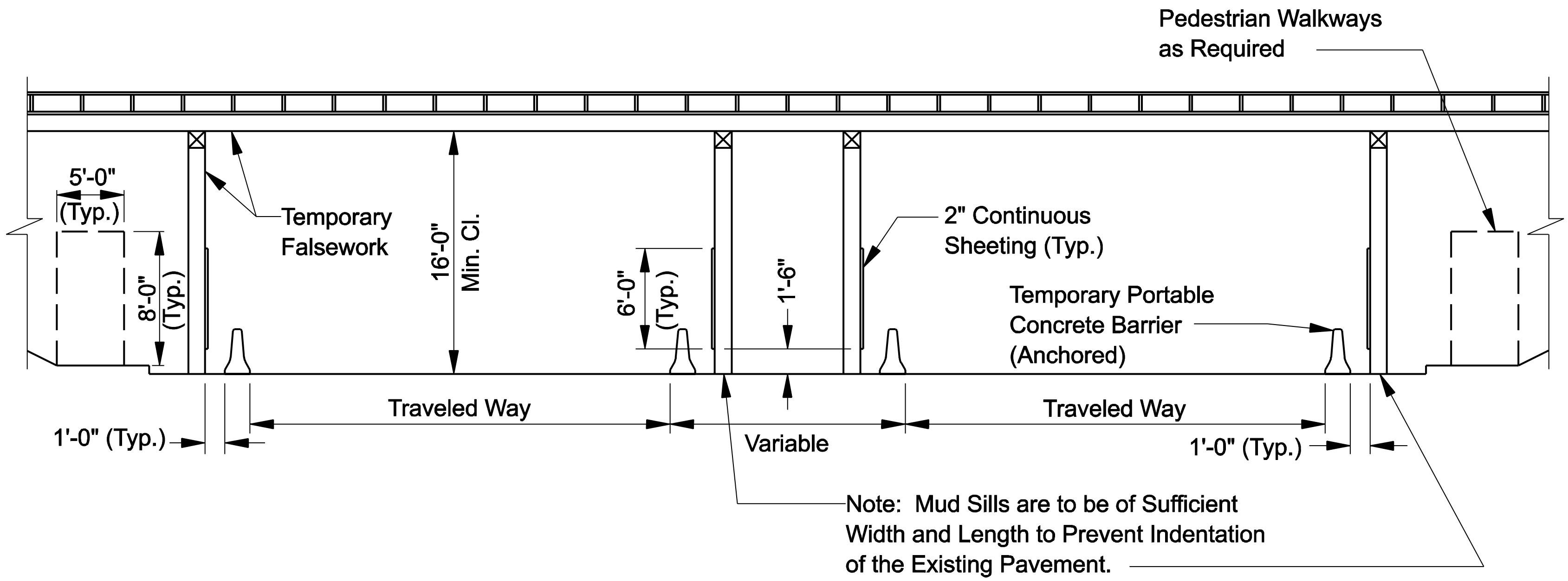
1. Deck Joints. Open, or inadequately sealed, deck joints have been identified as the foremost reason for corrosion of structural elements by permitting the percolation of salt-laden water through the deck. To address this, the *LRFD Specifications* promotes jointless bridges with integral abutments, continuous decks and improvements in drainage. If an in-span hinge must be used, consider using a second joint seal below the one at deck level consisting of a neoprene sheet trough.

2. Paint. The potential environmental issues associated with removing paint from steel structures makes the use of weathering steel preferable to painted steel from a maintenance perspective. However, in general, NDOT discourages the use of unpainted weathering steel because of aesthetic considerations. If weathering steel is used, the bridge designer must address the staining problem that can result from the use of weathering steel.

3. Drainage. Avoid elaborate plumbing systems where a closed system is used for bridge deck drainage. See Section 16.4.

4. Bridge Inspection. In addition to the maintenance needs of the structure, the bridge designer should consider the bridge inspection logistics including access.

5. Structural Details. As another maintenance/inspection consideration, the bridge designer should, as practical, limit the number of structural details (e.g., bearings, expansion joints).



TYPICAL FALSEWORK OPENING

Figure 11.4-A

11.4.12 Future Widening

In general, the bridge designer should consider the possibility of future structure widening. For example, structures supported by single columns or cantilevered piers cannot practically be widened; a separate adjacent structure will be required.

Almost every superstructure type can be widened, but not with the same level of ease. Slabs, slab on girders, and systems consisting of prefabricated elements lend themselves best to widening.

11.5 SUPERSTRUCTURES

This Section discusses those factors that should be considered in the selection of the superstructure type in preliminary design.

11.5.1 <u>General Considerations</u>

11.5.1.1 Introduction

Throughout the nation, many types of superstructures have been developed for the myriad applications and constraints that prevail at bridge sites. However, NDOT, like most other State DOTs, has narrowed its typical selection of superstructure types to a relatively small number based on NDOT's experience, geography, terrain, environmental factors, local costs, local fabricators, the experience of the contracting industry, availability of materials and NDOT preference. This promotes uniformity throughout the State and simplifies the bridge type selection and design process.

11.5.1.2 Span Length Ranges

Figure 11.5-A presents the typical span length ranges for the common and special application NDOT superstructure types. The upper limit of the span ranges relates to the structural capacity of the superstructure type. The lower limit of the range suggests a boundary below which other superstructure types are usually more cost effective; the lower limit does not suggest that the superstructure type cannot be used for shorter spans.

11.5.1.3 Typical Girder Spacings

Figure 11.5-B presents the typical girder spacings for the common and special application NDOT superstructure types. Generally, wider girder spacing results in a lower cost superstructure. However, wider girder spacings may require thicker decks, deeper girders, larger cross frames and higher concrete strengths for concrete girders, and they reduce redundancy.

11.5.1.4 Typical and Minimum Depths

Reference: LRFD Articles 2.5.2.6.2 and 2.5.2.6.3

Figure 11.5-C presents the typical and minimum depths for the common and special application NDOT superstructure types.

11.5.1.5 Substructure/Foundation Type Considerations

The selection of the foundation type typically occurs after the superstructure type selection. See Section 11.7. Therefore, the designer must anticipate the nature of the foundation characteristics in selecting the type of superstructure. The bridge designer should consider the following:

Structure Type		Span Length Ranges	
		Simple	**Continuous**
Common Superstructure Types			
Cast-in-Place, Post-Tensioned Concrete Box Girder		100′ - 225′	100′ - 250′
Composite Steel	Plate I-Girders	90′ - 250′	100′ - 400′
	Rolled Beams	30′ - 90′	
Composite Steel Tub Girders		120′ - 250′	120′ - 400′
Precast, Prestressed Concrete I-Girders		30′ - 80′	Up to 180′ spliced
Special Application Superstructure Types			
Cast-in-Place, Conventionally Reinforced Concrete Slab		Up to 40′; up to 60′ for voided slabs	
Cast-in-Place, Post-Tensioned Concrete Slab		40′-65′	
Precast, Post-Tensioned Concrete U-Girders		Spliced Lengths: 100′-230′	Spliced Lengths: 100′-250′

SPAN LENGTH RANGES

Figure 11.5-A

Structure Type		Typical Girder Spacing
Common Superstructure Types		
Cast-in-Place, Post-Tensioned Concrete Box Girder		Two times girder depth; 7′-10′ preferred
Composite Steel	Plate I-Girders	8′-14′
	Rolled Beams	6′- 10′
Composite Steel Tub Girders		Web-to-web spacing: 8′-12′
Precast, Prestressed Concrete I-Girders		6′-10′
Special Application Superstructure Types		
Cast-in-Place, Conventionally Reinforced Concrete Slab		N/A
Cast-in-Place, Post-Tensioned Concrete Slab		N/A
Precast, Post-Tensioned Concrete U-Girders		Two times girder depth; 8′-10′ preferred

TYPICAL GIRDER SPACING

Figure 11.5-B

Structure Type		Typical Depths (Relative to Span Length, L)	Minimum Superstructure Depth
Common Superstructure Types			
Cast-in-Place, Post-Tensioned Concrete Box Girder		Simple: 0.045L Continuous: 0.040L	3′-6″
Composite Steel	Plate I-Girders	Simple: 0.045L Continuous: 0.040L	3′
	Rolled Beams		2′
Composite Steel Tub Girders		Simple: 0.045L Continuous: 0.040L	4′
Precast, Prestressed Concrete I-Girders		Simple: 0.050L Continuous: 0.045L	3′-4′
Special Application Superstructure Types			
Cast-in-Place, Conventionally Reinforced Concrete Slab		Simple: 0.045L Continuous: 0.040L	1′-6″
Cast-in-Place, Post-Tensioned Concrete Slab		Simple: 0.030L Continuous: 0.027L	2′
Precast, Post-Tensioned Concrete U-Girders		Simple: 0.045L Continuous: 0.045L	4′

Note: For variable depth members, values may be adjusted to account for the change in the relative stiffness of positive and negative moment sections.

TYPICAL AND MINIMUM DEPTHS

Figure 11.5-C

1. Number of Supports. The expected foundation conditions will partially determine the number of and spacing of the necessary substructure supports. This will have a significant impact on the acceptable span lengths.

2. Dead Load. When foundation conditions are generally poor, the bridge designer should consider the economics of using structural steel over concrete to reduce dead load.

3. Scour. The geologic or historic scour may have a significant impact on the foundation design which may, in turn, have a significant impact on the superstructure type selection.

11.5.2 Cast-in-Place, Post-Tensioned Concrete Box Girder

11.5.2.1 Description

The cast-in-place, post-tensioned concrete box girder has been the most common structure type used in Nevada over the last 25 years. Practical limitations on span length are approximately 225 ft for simple spans and 250 ft for continuous spans. These bridges are torsionally stiff and can be used where there are high skews and tight-radius curves. A variable depth and width can also be easily accommodated. The depth variation can be linear or parabolic and extend either over a partial length or the entire span length.

In general, prestressed bridges do not have the long-term camber creep problems of conventionally reinforced concrete structures. This is due to the prestressing counteracting the dead load. In addition, the girder sections do not crack at service loads. The full cross section is effective in resisting loads and deflections resulting in a reduced structure depth.

The CIP, PT concrete box girder has the lowest construction and maintenance cost of all bridge types commonly used by NDOT. However, it involves the use of falsework and requires the longest time to construct. NDOT policy is to require a minimum of 16 ft of vertical clearance to falsework over traffic. If the temporary vertical clearance is less than 18 ft, protective systems are required. These temporary clearance requirements for falsework result in permanent vertical clearances that are considerably higher than required by NDOT minimum vertical clearances; see Section 11.9.6.2. These higher permanent clearances increase the approach roadway costs that must be included in the alternatives analysis. There is also a risk to the public and construction personnel when falsework spans over traffic. Use of falsework over rivers is risky and should be avoided.

All structures outside of Clark County must be protected from road salts. Epoxy-coated reinforcing steel and low water-cement ratio concrete have historically been the minimum requirements for deck protection. The use of high-performance concrete (low permeability and other factors) will help increase the life of deck slabs. However, decks of post-tensioned concrete box girders cannot be replaced without an adverse redistribution of the post-tensioning force. As such, additional deck protection should be considered for this structure type. The thickness of the top slab should be a minimum of 8 in.

Simple-span bridges typically require higher concrete strengths. Values up to 6500 psi have been used. The inventory and operating ratings of long simple-span bridges are very sensitive to small changes in the top slab concrete strength. A concrete strength of only 90% of the specified 28-day strength may have an adverse impact on the live-load capacity of a post-tensioned concrete box girder. Therefore, consider increasing the required 28-day strength by 5% to 10% over that required by design to account for possible production strength variations.

11.5.2.2 Typical Usage

The cast-in-place, post-tensioned box girder is normally used for simple spans over 100 ft and continuous spans over 130 ft. It may be considered for span lengths less than 100 ft if a shallower structure depth is needed.

11.5.2.3 Advantages/Disadvantages

The advantages of this structure type include low construction cost, low maintenance costs, good aesthetics, low depth-to-span ratio, and easy adaptability to complex geometry. High torsional resistance makes it desirable on horizontal curved alignment.

The disadvantages include longer construction time, need for falsework and complicated formwork, and the inability to replace the top slab.

11.5.2.4 Appearance

The appearance is good from all directions. The system conceals utilities, pipes and conduits.

11.5.2.5 Typical Girder Spacing

Girder spacings are normally from 7 ft to 10 ft for this structure type. Wider girder spacings can be used; however, the amount of post-tensioning for each web becomes crowded and inefficient. Reducing the number of webs in this structure type does not demonstrably reduce costs. The amount of post-tensioning can generally be reduced if more webs are added. Fewer ducts in a web results in an increased eccentricity between the post-tensioning center of gravity and the neutral axis, which increases efficiency.

11.5.3 <u>Composite Steel I-Girders</u>

11.5.3.1 Description

Composite steel I-girders are fairly common in Nevada. The steel girders can be either rolled shapes with spans up to approximately 90 ft or plate girders with spans up to approximately 250 ft. Plate girders can have a constant or variable depth. Abrupt depth changes should be avoided for aesthetic reasons. Changes in structure depth can be accomplished over the length of a span by linearly varying the web depth. Continuous girders can also be deepened at the supports and reduced at the center of the span where vertical clearance is tight. These haunched girders usually have a parabolic variation in depth.

Most structural steel is fabricated out of State, and shipping increases the cost of this structure type. Girder field sections can be easily transported in lengths up to approximately 125 ft. Splices are used to construct single girder lines up to approximately 1000 ft in length. The designer must consider how this structure type will be erected, where the erection crane(s) will be located and how the girders will be delivered to the site.

This is a girder-and-deck type of structure; therefore, the deck can be removed if needed without adversely affecting the steel I-girders. This structure type can also be used with large skews and on horizontal curves. Detailing of steel I-girders is critically important. Poor detailing will greatly increase the cost of the bridge. In addition, structural steel is susceptible to fatigue cracking and brittle fracture, but good detailing practices greatly reduce this potential.

NDOT's standard paint system for steel bridges is an inorganic zinc (zinc in a silicate media) base coat, an epoxy middle coat and urethane top coat. The inorganic zinc is applied to all steel surfaces except the top of top flange. Inorganic zinc is considered a Class B coating for slip resistance and can be included on the faying surfaces of all bolted connections.

See Chapter 15 for a detailed discussion on NDOT's design practices for structural steel superstructures.

11.5.3.2 Typical Usage

NDOT typically limits the use of structural steel superstructures to sites where a cast-in-place, post-tensioned concrete box girder superstructure is not the best choice. Structural steel is used where falsework necessary for cast-in-place, post-tensioned concrete boxes is not allowed, where construction duration is an issue, or where the falsework may cause problems (e.g., low clearances, safety).

Rolled beams are used for spans up to approximately 90 ft. Welded plate girders are used for spans from approximately 90 ft to 400 ft. Typically, simple spans may be used up to approximately a 250-ft span length; continuous spans may be used for spans from 120 ft to 400 ft. If a rolled-beam design is proposed for a new bridge, the contract documents should allow the substitution of a welded plate-girder with equivalent plate dimensions at the contractor's discretion.

Transportation of girders must be considered when identifying field splice locations.

11.5.3.3 Advantages/Disadvantages

When compared to other superstructure types, advantages of composite steel I-girders include fast on-site construction, no falsework, relatively simple details and formwork, good aesthetics, adaptable to complex geometrics, low dead weight, deck can be replaced and long-span capability. The structural characteristics for composite steel I-girders provide low dead load and, therefore, may be suitable when foundation conditions are poor.

The disadvantages of composite steel I-girders include moderate to high construction costs, high maintenance costs and attention to detailing practices. Detailing of steel girders is important. Poor detailing will greatly increase the cost of the bridge and can decrease durability through fatigue cracking. Composite steel I-girder bridges have a higher maintenance cost than concrete bridges.

11.5.3.4 Typical Girder Spacing

Girder spacings for steel I-girders are normally from 8 ft to 14 ft. Deep plate girder sections benefit the most from wide girder spacings. Shallow plate girders and rolled beams do not accommodate wider girder spacings and may require spacings less than 8 ft when at the limit of the depth-to-span ratios. The design of the shallow girder sections can be controlled by deflection requirements.

11.5.4 <u>Composite Steel Tub Girders</u>

11.5.4.1 Description

Composite steel tub girders are typically considered in urban areas where a steel-I girder could be used but enhanced aesthetics are desired. Steel tub girders are plate girders with two webs with a common bottom flange. The webs are usually inclined to improve aesthetics and reduce the width of the bottom flange. Spans are economical up to approximately 250 ft. Steel tub girders can have a variable depth, but this significantly increases the cost of the bridge. They can also be used on very tight-radius curves due to their high torsional stiffness. They do not, however, adapt well to skews or variable widths. Fabrication, transportation and erection of this structure type must be carefully considered. Steel tub girders are difficult to handle in the shop due to their size and weight. They require significant bracing during fabrication and erection. In addition, they are susceptible to thermal movements once erected and require temporary external bracing between boxes.

The paint system for steel tub girders is the same as for steel I-girders.

11.5.4.2 Typical Usage

Composite steel tub girders can be used for simple spans up to 250 ft and for continuous spans from 120 ft to 400 ft. This structure type is used mainly in urban areas.

11.5.4.3 Advantages/Disadvantages

Advantages of composite steel tub girder bridges include fast on-site construction, no falsework requirements, low dead weight, adaptability to tight-radius curves, deck can be replaced, good aesthetics and longer span capability. Higher torsional resistance makes them desirable on a horizontally curved alignment.

Few fabricators are available to construct composite steel tub girders. Disadvantages include the highest construction cost of all common NDOT superstructure types, high maintenance costs, and not readily adaptable to skewed or variable-width bridges. Composite steel tub girders require complicated fabrication, welding and erection. This structure type has higher maintenance cost than concrete bridges.

11.5.4.4 Appearance

This structure type is generally pleasing; it is more attractive than steel or precast concrete I-girders.

11.5.4.5 Typical Girder Spacing

Tub spacing is normally 8 ft to 12 ft. Deep sections benefit the most from wider spacings. Shallow sections do not accommodate wider girder spacings.

11.5.5 Precast, Prestressed Concrete I-Girders

11.5.5.1 Description

Precast, prestressed concrete I-girders have historically been used infrequently by NDOT. Relatively small bridges, common in Nevada, and precast plants located outside of Nevada have contributed to a higher cost for this structure type. Practical limitations on span length are approximately 100 ft for simple spans and 125 ft for continuous spans. Some States have spliced the girders to extend the spans even longer. This is a girder-and-deck type of structure; the deck can be removed if needed without adversely affecting the girders. This structure type is best used on multiple spans with a large number of girders. It does not adapt well to large skews and cannot be used on tight horizontal curves or bridges with a variable width.

See Chapter 14 for a detailed discussion on NDOT's design practices for prestressed, precast concrete girders.

11.5.5.2 Typical Usage

Precast concrete girders are generally only used where falsework is not allowed (e.g., over railroads), where construction duration is an issue, or where falsework may cause considerable problems (e.g., low clearances, safety). Precast, prestressed concrete I-girders are used for spans from 30 ft to 150 ft. The transportation and erection of precast I-girders must be considered in selecting girder lengths.

11.5.5.3 Advantages/Disadvantages

Advantages of this structure type include moderate construction cost on small bridges to fairly low construction cost on large bridges, low maintenance cost, no falsework, deck can be replaced, and moderately fast on-site construction. Its disadvantages include poor aesthetics due to thick bottom flanges with relatively narrow girder spacings, cannot be adapted to complex geometrics, limited span lengths and slightly higher depth-to-span ratios. Precast, prestressed concrete I-girders require careful handling during transportation and erection.

11.5.5.4 Appearance

Straight girders on curved alignment are discouraged, effectively precluding the use of precast I-girders except for relatively large radius curves.

11.5.5.5 Typical Girder Spacing

Girder spacings are normally from 7 ft to 10 ft. Concrete strength and the number of prestressing strands usually control the girder spacing. Concrete strengths of 6000 psi to 7000 psi can be produced in most precast plants. These higher strengths allow longer spans and/or increased girder spacing.

11.5.6 <u>Special Application Superstructure Types</u>

For special applications, it may be appropriate to select a superstructure type other than the common NDOT types. This Section briefly identifies those types (and their potential application) that may be used in special applications.

11.5.6.1 Cast-in-Place, Conventionally Reinforced Concrete Slab

11.5.6.1.1 *Description*

NDOT occasionally uses cast-in-place, conventionally reinforced concrete slabs (CIP concrete slabs) because of their suitability to short spans and low clearances and their adaptability to skewed and curved alignments. It is the simplest among all superstructure systems, it is easy to construct, and structural continuity can be achieved without difficulty.

NDOT practice is to use a constant-depth slab or a slab with haunches in the negative-moment regions.

11.5.6.1.2 *Typical Usage*

The CIP concrete slab is used for bridge spans up to approximately 40 ft; voided slabs are used up to approximately 60 ft.

11.5.6.1.3 *Advantages/Disadvantages*

The advantages of this structure type are low construction costs and low maintenance costs. The details and formwork are the simplest of any superstructure type. Construction time is also fairly short. The disadvantages are that the CIP concrete slab requires falsework and has a high depth/span ratio.

11.5.6.1.4 *Appearance*

Neat and simple, especially for low, short spans.

11.5.6.2 Cast-in-Place, Post-Tensioned Concrete Slab

11.5.6.2.1 *Description*

The basic distinction between the CIP concrete slab and the cast-in-place, post-tensioned concrete slab is the difference in the manner of reinforcement; therefore, most of the information in Section 11.5.6.1 is applicable. This structural system can be used for greater span lengths than the CIP concrete slab at comparable structural depth.

11.5.6.2.2 *Typical Usage*

Consider using this structure type for spans up to 65 ft. It may be the best selection where a very low depth/span ratio is required.

11.5.6.2.3 *Advantages/Disadvantages*

The advantages are low construction costs, low maintenance costs and thin superstructure depth. The disadvantages are long construction time and the requirement for falsework. The CIP, PT concrete slab is more difficult to construct than conventionally reinforced concrete slabs.

11.5.6.2.4 *Appearance*

This is the same as conventionally reinforced concrete slabs.

11.5.6.3 Precast, Post-Tensioned Concrete U-Girders

11.5.6.3.1 *Description*

The precast, post-tensioned U-girder bridge has recently gained in popularity. It provides the look and behavior of cast-in-place, post-tensioned concrete box girder construction but uses precast elements. It is higher in cost than both cast-in-place, post-tensioned concrete and precast, prestressed concrete I-girders. The spans are comprised of segments that are erected on temporary towers, connected together by cast-in-place concrete, and then post-tensioned full length.

Many concrete U-girders have a combination of pretensioning and post-tensioning. The pretensioning carries the girder self weight and construction loads, and the post-tensioning is designed to carry loads in the permanent configuration.

11.5.6.3.2 *Typical Usage*

This structure type can also be used to widen an existing cast-in-place concrete box girder to provide compatible aesthetics and structural performance. In addition, it can be used where clearances restrict the use of falsework.

11.5.6.3.3 *Advantages/Disadvantages*

Advantages include no falsework over traffic, low maintenance costs, good aesthetics and low depth-to-span ratios. Disadvantages include non-standard U-girder sections, moderate to long construction time, higher construction cost, complex construction, cannot be adapted to complex geometrics and decks are difficult to replace.

11.5.6.3.4 *Appearance*

The appearance is good from all directions. Concrete U-girders provide the look of cast-in-place, post-tensioned concrete box girder construction.

11.5.6.3.5 *Typical Girder Spacing*

The typical girder spacing for both cast-in-place, post-tensioned box girders and precast I-girders should be used for U-girders.

11.5.6.4 Long-Span Culverts

Reference: LRFD Section 12

Long-span culverts may be an attractive alternative for small stream and ditch crossings (where they can protect the stream bed), minor highway and street crossings, and pedestrian or wildlife crossings. As discussed in Section 11.2.2.7, hydraulics is one of the significant issues in selecting a culvert or a bridge. Long-span culverts are commonly made of steel or concrete. The most common configurations used are the three-sided concrete or steel culvert, four-sided monolithic precast concrete box culvert, structural plate pipe arch and circular pipe. Spans of 50 ft or less are reasonable; pipe arch spans up to 80 ft are possible.

If a single or multiple specialty installation is being considered, the designer should consult with the manufacturer(s) of the specialty structure for design information (e.g., cost, availability, design).

11.5.7 Superstructure Types Used With Approval

Superstructure types other than the “common” and “special application” types may be used. The bridge designer should investigate the experience of other owners, and the acceptability of these superstructure types may be based upon their successful experiences. The Chief Structures Engineer must provide written approval for the selection of structure types not considered “common” or “special application.”

11.5.7.1 Cast-in-Place, Conventionally Reinforced Concrete Box Girders

The basic distinction between cast-in-place (CIP), conventionally reinforced concrete (RC) box girders and cast-in-place, post-tensioned concrete box girders is the difference in the manner of reinforcement; therefore, some of the information in Section 11.5.2 is applicable. However, the disadvantages of CIP RC boxes result in very limited application, if at all in Nevada. This structural system can only be used for shorter span lengths than the CIP, PT concrete boxes at comparable structural depth. CIP RC boxes have low construction and maintenance costs with good aesthetic qualities. However, this superstructure type requires falsework, requires a long construction time and demonstrates long-term creep deflection problems not associated with comparable prestressed superstructure types. CIP RC box girders are limited to shorter span lengths due to inherently high depth-to-span ratios.

11.5.7.2 Timber Bridges

Reference: LRFD Section 8

Timber structures shall be avoided, unless the bridge site is a remote, off State highway system location where a conventional bridge is impractical. The maximum span length is 40 ft, and the typical girder spacing is 4 ft to 6 ft. Depth/span ratios are as follows:

- simple span (timber girder): 1/10
- simple span (glulam girder): 1/12
- continuous span: 1/14

Timber bridge design details and construction are simple, and they can be aesthetically pleasing in the proper setting. Timber bridges are lightweight and require no falsework. Disadvantages include high maintenance costs, limited useful life and barrier rail connection problems.

Glued-laminated timber shall be used (rather than sawn timber) for main load-carrying elements (e.g., girders, caps). Glued-laminated timber deck panels are also available.

11.5.7.3 Segmental Concrete Box Girders

Reference: LRFD Article 5.14.2

Segmental construction is only considered on large projects where the total deck area is approximately 250,000 sq ft or more. However, it can be used in other special applications that dictate the use of this structure type. A significant investment in the casting facilities and erection equipment is required for precast segmental construction. This structure type usually consists of a single-cell box girder built in segments by either the precast or cast-in-place method.

Advantages include no falsework, least effect on traffic of any structure type, low maintenance costs, good aesthetics, fast on-site construction for precast method and low depth-to-span ratios. Disadvantages include the complexity of time-dependent analysis and design, variable construction costs, limited number of qualified contractors, complex construction and construction engineering, lengthy construction time for the cast-in-place method, economical only on large projects, and decks cannot be replaced.

Most segmental concrete bridges have a single-cell superstructure with only two girder webs. The spacing of the web is based on the efficiency of the deck design. They are either cast-in-place or precast construction with longitudinal post-tensioning and transverse post-tensioning in the deck. The longitudinal post-tensioning can be placed internal to the webs as with conventional cast-in-place, post-tensioned construction; fully external to the web; or a combination of internal and external. Internal post-tensioning has the same considerations as cast-in-place, post-tensioned concrete box girders. Externally post-tensioned bridges usually have thinner webs because a wider web is not needed for the post-tensioning. Research at the University of San Diego indicates a combination of internal and external tendons has the lowest seismic resistance of the three and should not be used in moderate-to-high seismic areas outside Clark County.

Precast box segments differ from project to project, but the American Segmental Bridge Institute has established standard sections. Spans are used typically up to approximately 250 ft.

Concrete segmental construction requires that the bridge be designed for a specified method of construction and included in the contract documents with assumed erection loads. The design of the substructure is, in many cases, controlled by the construction of the bridge and not the permanent loads.

The precast method is usually less expensive than the cast-in-place method and is used mainly for shorter spans. Precast segments are match cast in a casting yard, transported to the site, drawn together in the erected position by temporary post-tensioning, the span closed by cast-in-place concrete, and subsequently post-tensioned the full length of span. Erection methods can be by balanced cantilever, temporarily supported by under-slung trusses, or by an overhead gantry.

A traveling form supports cast-in-place segmental construction and is advanced when the poured concrete has reached sufficient strength. Erection methods include balanced cantilever, a gantry crane or a system of stays.

These projects are complex. An experienced contractor and contractor's engineer are necessary. Erection methods and equipment used to erect the segments vary from project to project. The contractor is required to verify the design based on their means and methods of construction.

11.5.7.4 Other Superstructure Types

Superstructures types other than those specified elsewhere in Section 11.5 may be used. These include:

- steel trusses,
- steel tied arches,
- concrete arches, and
- cable-supported concrete or steel bridges.

11.6 SUBSTRUCTURES

11.6.1 Objective

This Section discusses the types of substructure systems used by NDOT and their general characteristics. The bridge designer should use this guidance to select the substructure type that is suitable at the site to economically satisfy the geometric and structural requirements of the bridge and to safely use the strength of the soil or rock to accommodate the anticipated loads. Chapter 18 discusses the detailed design of substructure elements, including detailed figures for each abutment type.

11.6.2 Abutments

Reference: LRFD Article 11.6

11.6.2.1 General

Abutments can be classified as flexible or rigid. Flexible (also known as integral or dozer) abutments transmit earth pressures on the abutments through the superstructure eliminating expansion joints at the end of the superstructure (for total movements of less than ½ in). Rigid (also known as seat) abutments incorporate expansion joints at the end of the bridge to accommodate thermal movements. Flexible abutments must be able to accommodate the movements through elastic behavior of the bridge and the surrounding soil because the deck and girders are integral with the abutment. Flexible abutments are considered pin-ended, expansion bearings in the superstructure analysis. Rigid abutments can be fixed or expansion based upon the choice of bearings.

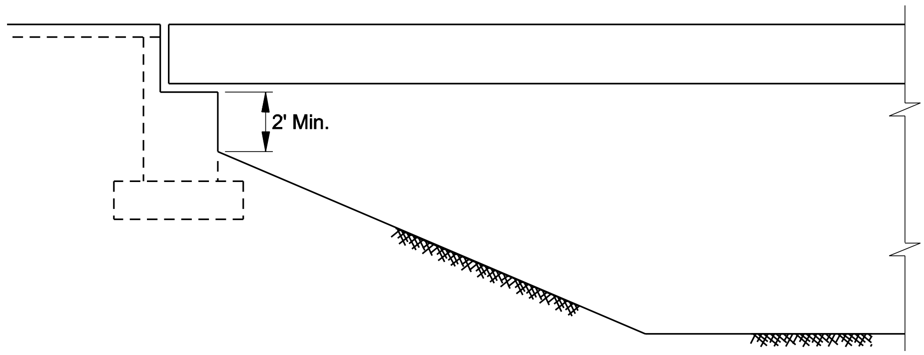
Abutments may be further classified as either open or closed. See Figure 11.6-A for schematics. Open abutments are used for most bridges and are placed at the top of the slope. Slopes are typically 2H:1V. See Figure 11.6-B. This slope is based on stability requirements and erosion control. However, landscaping and aesthetic considerations may result in 3H:1V slopes. Contact the Landscaping/Aesthetics Section for slope recommendations when constructing bridges along corridors that have landscape requirements. In addition, the 2 ft of exposed abutment face may be increased for landscape aesthetics.

Open abutments result in longer spans compared to closed abutments, but the total overall cost is usually less compared to closed abutments because open abutments are typically shorter. In general, open abutments are considered more aesthetic than closed abutments.

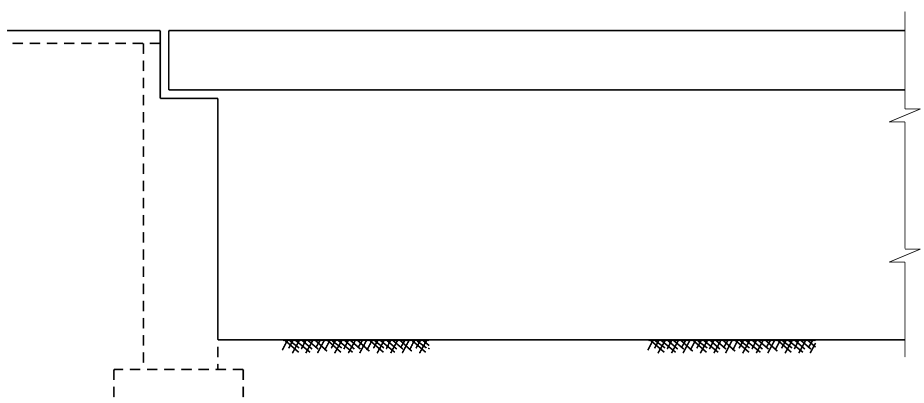
Closed abutments are used when span lengths need to be minimized. There are no fill slopes under the bridge but extensive retaining walls must be used. These retaining walls run either along the approaches to the bridge or parallel to the abutment. Retaining walls along the approaches are preferred from a visual perspective. Closed abutment footings must be placed below the level of the highway running beneath the bridge resulting in tall exposed abutment faces.

11.6.2.2 Basic Types

An abutment may be designed as one of the following basic types:



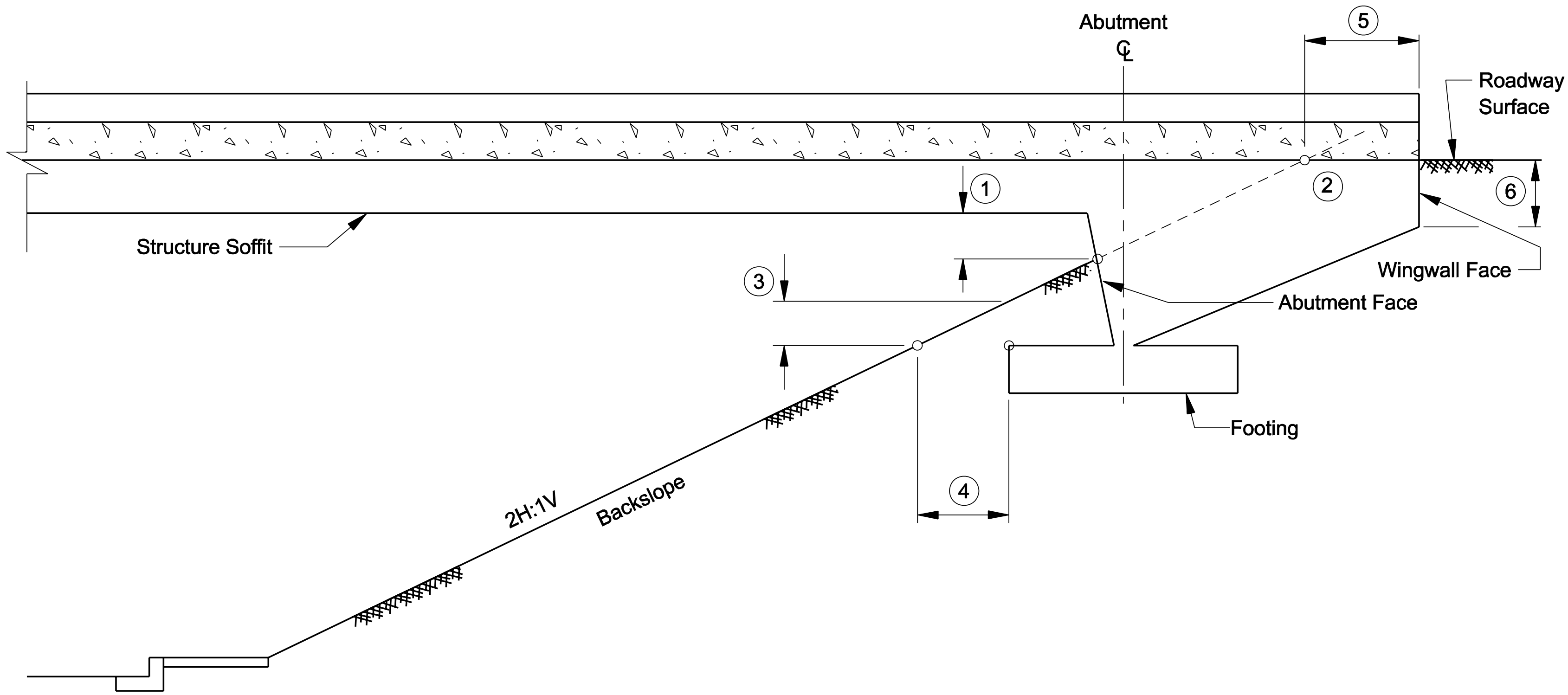
OPEN ABUTMENT



CLOSED ABUTMENT

OPEN vs CLOSED ABUTMENTS

Figure 11.6-A



1. The intersection of the backslope and abutment face should occur approximately 2 ft below the structure soffit.
2. Intersection of backslope and roadway surface.
3. Earth cover on footing. Minimum is 1.5 ft.
4. Distance from top edge of footing to face of slope. Minimum is 4.0 ft.
5. Distance from backslope/roadway surface intersection to end of wingwall. 5.0 ft preferable.
6. Distance from roadway surface to bottom of end of wing. 3.0 ft preferable.

OPEN ABUTMENT SLOPE

Figure 11.6-B

- integral abutments,
- seat abutments, and
- dozer abutments.

Each of these is discussed in the following Sections.

Flexible abutments, either integral or dozer abutments, are generally preferred for bridges of length up to 400 ft where the soil pressure on the two abutments is opposing and approximately balanced. Seat abutments are used where these conditions and other geometric limitations are not met.

11.6.2.3 Integral Abutments

Integral abutments are segregated into the following sub-types:

1. Diaphragm-with-Footing Abutment. This abutment is formed by extending an end diaphragm down to a footing. The end diaphragm is rigidly attached to the superstructure and free to slide on the footing. Stops (shear keys) are placed on the footing to limit the movement of the diaphragm during seismic events. The extension of the end diaphragm down to the footing forms the abutment stemwall. The footing may be either a spread footing or pile supported. If piles are used, two rows of driven piles or small-diameter drilled shafts spaced a minimum of 6 in to each side of the centerline of bearing, or a large-diameter (36 in minimum) drilled shaft is required.

   The diaphragm-with-footing abutment is the most widely used and is generally preferred. It can be used with the restriction that the diaphragm height measured from the top of footing to the bottom of the superstructure does not exceed 9 ft or twice the superstructure depth. A footing pedestal of up to 3 ft in height may be used to comply with this restriction.

2. Diaphragm-with-Pile Abutment. This abutment type is restricted to shorter bridge lengths. The end diaphragm is extended down to the foundation as for diaphragm-with-footing abutments. However, the diaphragm is supported on a single row of piles and is connected to them. The connection to the piles may be rigid where the pile is embedded into the diaphragm or pinned where the diaphragm is connected to a pile cap by a pin connection. In either case, the diaphragm is rigidly connected to the superstructure.

   Diaphragm-with-pile abutments may be used where piles are required and the bridge can be supported on a single row of piles. The piles must be installed accurately without drift and must be capable of accommodating expected thermal movements. Stiff, large-diameter drilled shafts are not suitable for use with this type of abutment. Drilled shafts of any size should not be used on post-tensioned structures due to the movement resulting from prestress shortening.

11.6.2.4 Seat Abutments

Seat abutments consist of a footing, stemwall, seat and backwall. The superstructure is supported by bearings on the abutment seat. The backwall retains the backfill above the abutment seat so that the backfill is not in contact with the superstructure. The approach slab extends over the top of the backwall, and there is an expansion joint between the approach slab and the superstructure deck.

11.6.2.5 Dozer Abutments

A dozer abutment is a hybrid of the conventional seat abutment and an integral abutment. The superstructure is supported by bearings on top of the abutment stem wall. The superstructure end diaphragm extends beyond and below the stem wall. The height of the diaphragm extension may be adjusted as required by design.

Dozer abutments are used in lieu of the diaphragm-with-footing abutment where the stem wall height exceeds the height limit for a diaphragm or where skews exceed 30°. However, dozer abutments are complex to design and detail for high skews.

11.6.2.6 Applicability and Limitations

Integral and dozer abutments are advantageous because they mobilize earth passive pressure to resist and dampen seismic forces and eliminate damage to abutment backwalls from seismic movements. Additionally, both integral and dozer abutments provide the capability to move the expansion joint at the end of the bridge to a point where joint leakage does not promote deterioration of bearings or abutment seats.

Integral and dozer abutments may be used on bridges with flares, skewed support or horizontal curvature only where the effect of these design features is limited. The determination of acceptable limits is based more on experience than theory. Flares and curvature are limited because the soil pressures must be reasonably balanced such that the imbalanced force can be readily accommodated. Structure skew also results in unbalanced soil pressures because the lines of action of the soil pressures on the two abutments does not coincide. Additionally, for integral abutments, the horizontal axis of rotation of a skewed abutment is not parallel to the bending axis of the superstructure girders, which produces torsion in the superstructure. Dozer abutments reduce this effect through the use of bearings similar to a seat abutment.

The same abutment type should be used at both ends of the bridge. Integral and dozer abutments shall not be used where there are expansion hinges in the superstructure. Figure 11.6-C presents the more detailed limits that should be used in selecting an abutment type.

The thermal movement of an abutment should be limited to approximately 1 in of expansion or contraction for integral and dozer abutments. For structures with integral abutments, the wingwalls are cantilevered from the diaphragm wall and are assumed to move with the structure. They are usually oriented parallel to the roadway to avoid resisting passive earth pressures due to the thermal movement. Wingwalls on structures with dozer or seat abutments are supported by a footing or piles. They are usually oriented parallel to the roadway, but may be set at angles if desired, such as at stream crossings. Approach slabs are required on paved roads and preferred on unpaved roads. The approach slab is stationary with a watertight joint to accommodate movement at the end of the structure where seat abutments are used. Where the wingwalls are oriented parallel to the roadway, the approach slab should extend over the wingwalls and support the barrier rail.

11.6.3 <u>Piers</u>

Reference: LRFD Article 11.7

Geometric Parameters	**Abutment Type**			
	Integral Abutment		**Dozer Abutment**	**Seat Abutment**
	Diaphragm with Footing	**Diaphragm with Piles**		
Maximum Length [(1)] Concrete Steel	 400 ft 250 ft	 250 ft 150 ft	 400 ft 250 ft	 Unlimited Unlimited
Maximum Flare [(2)]	20%	10%	40%	Unlimited
Maximum Skew	30°	20°	30°	Unlimited
Maximum Curvature [(3)]	10°	10°	10°	Unlimited

(1) Values are for cold climate per LRFD Article 3.12.2.1. Values may be increased by 20% in areas of moderate climate.

(2) Adjust diaphragm height to approximately balance soil pressures. Limitation does not apply to simple span bridges with diaphragm abutments.

(3) Central angle of a horizontal curve within the bridge limits or the difference in survey bearings of abutment centerlines.

LIMITS FOR ABUTMENT TYPES

Figure 11.6-C

11.6.3.1 General

Piers consist of a pier cap supported on columns or a pier wall. Although rarely used by NDOT, under certain conditions, the economy of substructures can be enhanced by extending a deep foundation above ground level to the superstructure forming a pile bent.

11.6.3.2 Pier Caps

Pier caps are usually reinforced concrete members that transfer girder loads into columns or pier walls. In all cases, a pier cap shall be used. These can be integral, drop or outrigger caps.

Integral caps are mainly used with cast-in-place concrete girders but also can be used with precast and steel girders. Integral caps used with steel girders can be either steel cross girders or post-tensioned concrete. They should be used only when necessary. Integral steel caps are non-redundant, expensive and require precise fabrication. Integral concrete caps with steel girders are difficult to construct, usually require temporary falsework and do not allow inspection of the top tension flanges after the bridge goes into service.

Outrigger caps are used where a column support must extend beyond the edge of the superstructure. Outrigger caps should not be used unless necessary. They should be simple spans with pin connections at the top of the columns. Pin connections reduce the torsional shear forces in the outrigger cap. Most outrigger caps are integral concrete and post-tensioned to reduce their depth, control cracking and enhance torsional resistance.

11.6.3.3 Columns and Pier Walls

Columns and pier walls are substructure components that support the cap. Either single or multiple columns can be used depending upon the width and skew of the bridge. Columns are used in highway-over-highway construction. NDOT's standard columns have an octagonal or rounded shape with 4 ft outside dimensions. This allows the use of a continuous transverse spiral or welded hoops. Rectangular columns up to 8 ft in width by 4 ft in depth have been used with interlocking transverse spiral reinforcement.

A single-column pier should be considered for narrow bridges over rivers. River meander can change the direction of flow, which has an adverse effect on pier walls. Water hitting a pier at an angle will greatly increase scour. This does not adversely affect a single round column. However, single columns are usually at least 6 ft in diameter.

Pier caps supported on columns for highway-over-highway applications may consist of a minimum of two columns. Where redundancy considerations are an issue, the bridge designer should consider a minimum of three columns.

A pier wall is a continuous wall extending to almost the outside face of bridge. They are typically 2′-6″ wide with tied reinforcement.

Pier walls are also used for bridges over railroads to satisfy AREMA crash wall requirements. Columns with crash walls can be used, but a pier wall eliminates the need for a massive cap.

Pile bents shall not be used where large lateral forces may develop due to collision by vehicles, scour or stream flow intensified by accumulated debris Where used, the piles may be either steel H-piles or pipe piles.

11.6.3.4 General Usage

The following will apply to the selection of a pier type:

1. Water Crossings. If the foundation conditions allow, a drilled-shaft, single-column pier is preferred. Multiple columns are usually preferred to a pier wall. The Hydraulics Section will provide a recommendation to assist in this determination.

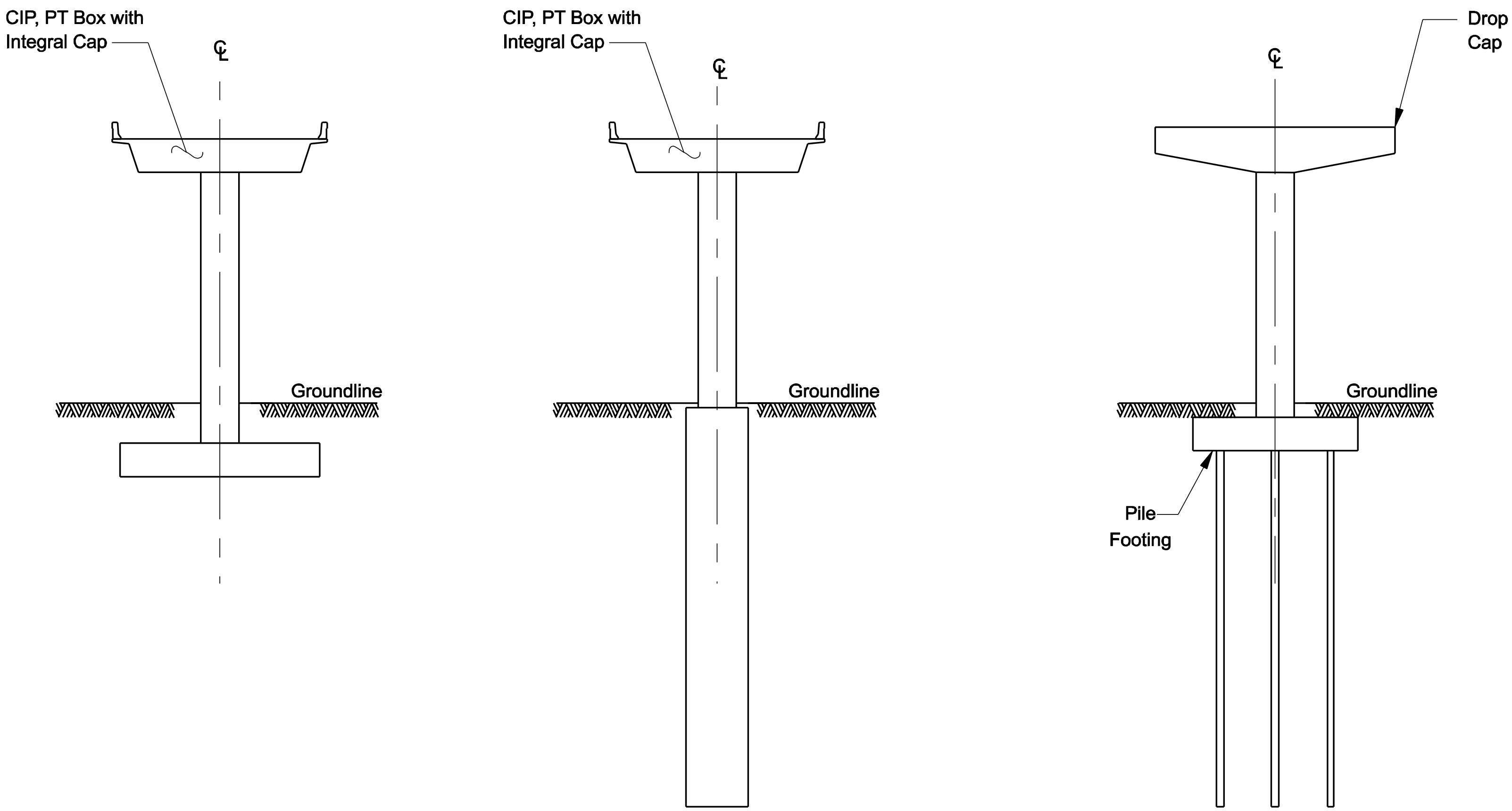
2. Meandering Rivers. For meandering rivers, the most desirable pier type is normally a single pier column. This type should be used, if practical.

3. Railroad Crossings. Use a solid pier wall that satisfies AREMA requirements if the pier is within 25 ft of the track centerline or future track centerline. See Section 21.1.3.4 for more information.

4. Highway Grade Separation. Preferably, use multiple column piers.

11.6.3.5 Schematics

Figure 11.6-D presents schematics of typical pier types used by NDOT in combination with typical NDOT foundations. Note that other pier/foundation combinations may be appropriate that are not shown in Figure 11.6-D.



SINGLE-COLUMN PIER
WITH SPREAD FOOTING

(a)

SINGLE-COLUMN PIER
WITH DRILLED SHAFT

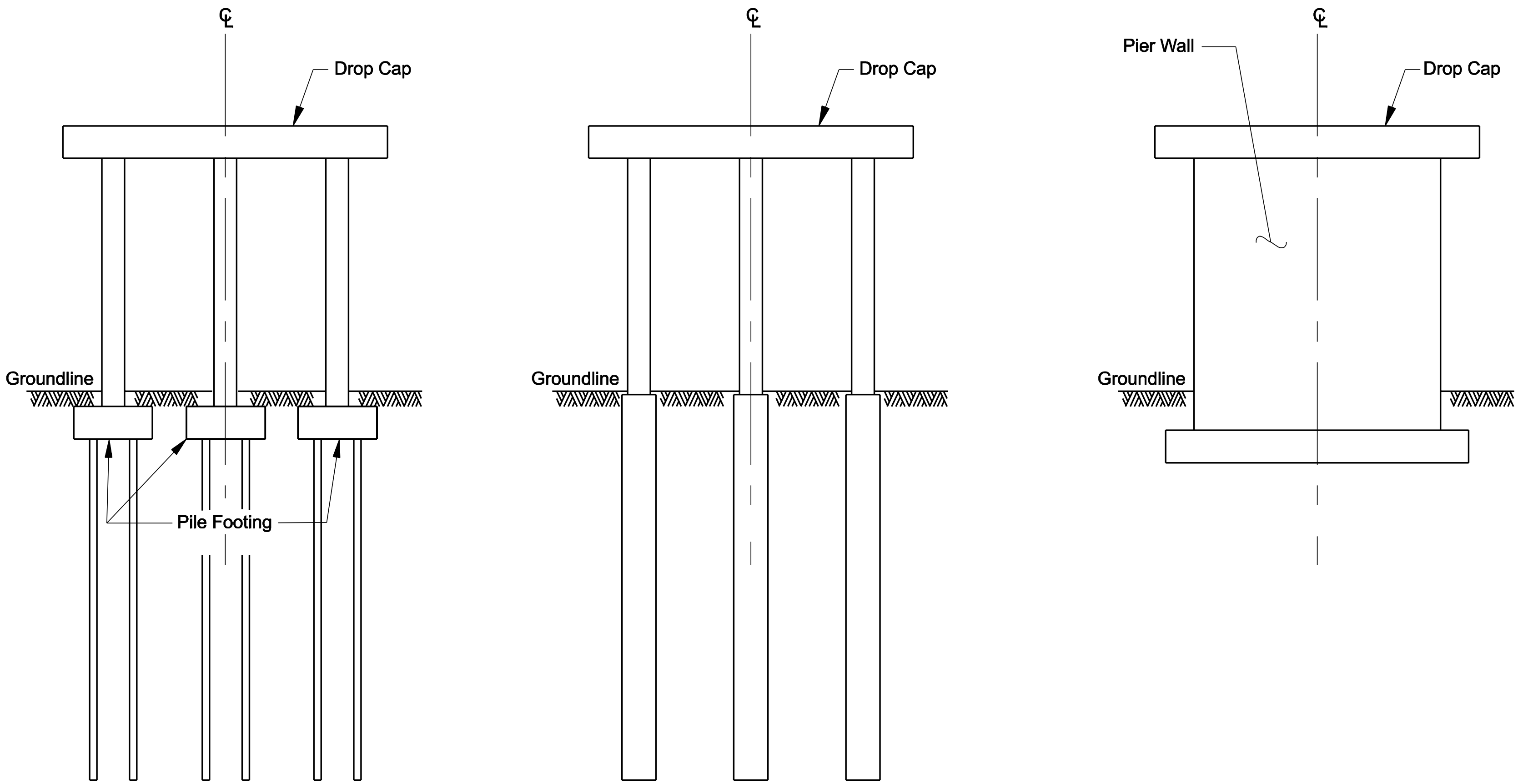
(b)

SINGLE-COLUMN PIER
WITH DRIVEN PILES

(c)

TYPICAL PIER/FOUNDATION COMBINATIONS

Figure 11.6-D



MULTI-COLUMN PIER
WITH DRIVEN PILES
(d)

MULTI-COLUMN PIER
WITH DRILLED SHAFTS
(e)

PIER WALL
WITH SPREAD FOOTING
(f)

TYPICAL PIER/FOUNDATION COMBINATIONS
(Continued)

Figure 11.6-D

11.7 FOUNDATIONS

11.7.1 Coordination

11.7.1.1 General

Coordination between the bridge designer and Geotechnical Section for foundation-type selection and design is performed in two phases. During preliminary design, the bridge designer provides the Geotechnical Section with a structure layout (Front Sheet) and preliminary foundation vertical loads. The Geotechnical Section performs the drilling, sampling and testing and then provides the preliminary foundation recommendations. These recommendations will include either spread footings or deep foundations (with recommended pile type).

For waterway crossings, bridge foundations must be designed by an interdisciplinary team of hydraulic, geotechnical and bridge engineers to withstand the effects of estimated total scour, including:

- local scour at piers and abutments,
- contraction scour, and
- long-term aggradation or degradation.

During final design, vertical loads are refined, lateral loads determined and provided to the Geotechnical Section along with scour depths (if a stream crossing). The Geotechnical Section provides final foundation recommendations. This can also include special requirements such as p-y curves for lateral pile design, downdrag potential, preloading requirements and ground modification. The pile depths are based on vertical loads with scour considerations. The bridge designer evaluates the structural requirements of piles and will extend the depth due to lateral loading if needed.

Chapter 17 discusses the detailed design of foundations, including the coordination among the Structures Division, Geotechnical Section and Hydraulics Section.

11.7.1.2 Seismic Analysis

For drilled-shaft/driven-pile-supported bridges that require a rigorous seismic analysis, the Structures Division performs lateral soil-structure interaction analyses using Extreme Event I loadings. If soil liquefaction is anticipated, the Geotechnical Section will provide the Structures Division with foundation downdrag loads due to liquefaction for use in developing the Extreme Event I load combination. The Geotechnical Section will also provide any lateral soil forces that act on the foundation as a result of seismically induced stability movements of earth retaining structures (e.g., embankments, retaining walls) or lateral soil movements attributable to lateral spread. These additional lateral loads should be included in the Extreme Event I load combinations when performing lateral soil-structure interaction. The Geotechnical Section will provide the soil parameters necessary to generate a p-y soil model of the subsurface that accounts for cyclic loadings and any liquefied soil conditions. The Structures Division then performs the lateral soil-structure interaction analysis with computer programs such as STRAINWEDGE or LPILE. The Structures Division uses this information to calibrate the seismic model. The Structures Division performs the seismic analysis in accordance with the *LRFD Specifications.*

If structural members are overstressed or if deflections exceed acceptable limits from any loading combination, then a redesign of the foundation is required. Redesign may include the adjustment of support member spacing or modification of member sizes. When a redesign of the foundation is required, the Structures Division must resubmit the redesign information (new foundation layout, sizes, foundation load combinations, etc.) to the Geotechnical Section. The Geotechnical Section will analyze the new foundation and resubmit the necessary information to the Structures Division.

11.7.2 <u>Types/Usage</u>

The following summarizes NDOT's typical practices for the selection of the type of foundation.

11.7.2.1 Spread Footings

Reference: LRFD Article 10.6

Spread footings are NDOT's preferred foundation type if soils and settlement allow their use. They may also be used beneath retaining walls and sound walls. The use of spread footings requires firm bearing conditions; competent material must be near the ground surface (i.e., a maximum of 15 ft below the ground line). They are not allowed at stream crossings where they may be susceptible to scour.

A spread footing is a shallow foundation consisting of a reinforced concrete slab bearing directly on the founding stratum. A spread footing's geometry is determined by structural requirements and the characteristics of supporting components, such as soil or rock. Their primary role is to distribute the loads transmitted by piers or abutments to suitable soil strata or rock at relatively shallow depths.

Settlement criteria need to be consistent with the function and type of structure, anticipated service life and consequences of unanticipated movements on service performance. Longitudinal angular distortions between adjacent spread footings greater than 0.008 radians in simple spans and 0.004 radians in continuous spans should not ordinarily be permitted.

Ground modification techniques may be used to improve the soil allowing the use of spread footings where they would not otherwise be appropriate as determined by the Geotechnical Section. These techniques are typically used to address differential settlement concerns or to avoid potential liquefaction problems. These techniques include the construction of columns of gravel in the ground called stone columns or compaction grouting through the pressure injection of a slow-flowing water/sand/cement mix into a granular soil.

11.7.2.2 Driven Piles

A driven pile is a long, slender deep foundation element driven into the ground with power hammers.

If underlying soils cannot provide adequate bearing capacity or tolerable settlements for spread footings, driven piles may be used to transfer loads to deeper suitable strata through skin friction and/or point bearing. The selected type of pile is determined by the required bearing capacity, length, soil conditions and economic considerations. NDOT primarily uses steel pipe piles, but will occasionally consider steel H-piles or prestressed concrete piles. See Section17.3 for more information.

11.7.2.3 Drilled Shafts

Reference: LRFD Article 10.8

A drilled shaft (also called a caisson or cast-in-drilled-hole pile) is a long, slender deep-foundation element constructed by excavating a hole with auger equipment and placing concrete, with reinforcing steel, in the excavation. Casing and/or drilling slurry may be necessary to keep the excavation stable.

The bridge designer should use drilled shafts where significant scour is expected, where there are limits on in-stream work or tight construction zones, or where driven piles are not economically viable due to high loads or obstructions to driving. Limitations on vibration or construction noise may also dictate the selection of this foundation type. Drilled shafts can be a more costly foundation alternative.

11.8 ALTERNATIVES ANALYSIS

Many factors enter into the selection of the most suitable bridge type and size. Initial cost is important, but it should not be the only consideration. Durability and long-term maintenance requirements, aesthetics, constructibility, effect on the public and environment, use of falsework, geometric adaptability, quantity of embankment required for the approach roadway, permanent clearances, structural requirements, redundancy and other factors should be included as appropriate in the alternatives analysis.

The documentation for structure-type selection may be as little as several paragraphs of explanation or as detailed as a multi-page report. Cost of the bridge, project controversy, complexity of the site, public involvement and other issues will dictate the effort needed.

Every bridge could in theory have numerous alternatives. However, many alternatives can be eliminated due to their high cost or because they have a fatal flaw such as an incompatibility with a horizontally curved alignment. The analysis should be performed only on viable alternatives. Features such as location, span length, superstructure type, girder material and substructure type tend to dictate the need for an alternatives analysis. At least two viable superstructure types can usually be identified for most proposed bridges. This usually dictates the need for the alternatives analysis.

There are many available strategies to compare bridges in an alternatives analysis. There is no established method that is best for all projects. Use a rating method that supports the features of the project. The weighting of each evaluation factor can be used to provide more emphasis to certain factors if these factors warrant more consideration. Initial cost and effect on the public are evaluation factors that can have a major influence on the selection of a bridge type, but this will vary from project to project.

Foundations can be either shallow or deep. Shallow foundations are significantly less expensive than deep foundations. Bridges located at different sites can have different foundation requirements. These differences must be included in an alternative’s cost.

11.9 ROADWAY DESIGN ELEMENTS

11.9.1 Coordination

In general, the roadway design criteria will determine the geometric design of the roadway, and the bridge design will accommodate the roadway design across any structures within the project limits. This will provide full continuity of the roadway section for the entire project. This process will, of course, require communication between the bridge designer and roadway designer to identify and resolve any inconsistencies. This Section provides roadway design information that is directly relevant to determining the structural dimensions for the preliminary bridge design and to provide the bridge designer with some background in roadway design elements.

The Roadway Design Division is involved with all bridge projects, and the Structures Division and Roadway Design Division collaborate on the roadway design features crossing the bridge. Initially, Roadway sets the geometrics, which is based on Section 2.2 of the *NDOT Project Design Development Manual*. The bridge designer will check the proposed geometric design (e.g., clearances, horizontal curves, vertical curves, roadway approach, cross slopes, widths) to identify any modifications that may be warranted to better accommodate structural design considerations. Any proposed modifications are communicated to the Roadway Design Division.

11.9.2 Highway Systems

11.9.2.1 Functional Classification System

The functional classification concept is one of the most important determining factors in highway design. The functional classification system recognizes that the public highway network serves two basic and often conflicting functions — travel mobility and access to property. In the functional classification scheme, the overall objective is that the highway system, when viewed in its entirety, will yield an optimum balance between its access and mobility purposes.

The functional classification system provides the guidelines for determining the geometric design of individual highways and streets. Based on the function of the facility, the roadway designer selects an appropriate design speed, roadway width, roadside safety elements, amenities and other design values.

The following briefly describes the characteristics of the various functional classifications.

11.9.2.1.1 *Arterials*

Arterial highways are characterized by a capacity to quickly move relatively large volumes of traffic and by a restricted function to serve abutting properties. The arterial system typically provides for high travel speeds and the longest trip movements. The arterial functional class is subdivided into principal and minor categories for both rural and urban areas.

Principal arterials provide the highest traffic volumes and the greatest trip lengths. The freeway, which includes Interstate highways, is the highest level of arterial. In rural areas, minor arterials will provide a mix of interstate and interregional travel service. In urban areas, minor arterials may carry local bus routes and provide intra-community connections.

11.9.2.1.2 *Collectors*

Collector routes are characterized by a roughly even distribution between access and mobility functions. Traffic volumes will typically be somewhat lower than those of arterials. In rural areas, collectors serve intra-regional needs and provide connections to the arterial system. In urban areas, collectors act as intermediate links between the arterial system and points of origin and destination.

11.9.2.1.3 *Local Roads and Streets*

All public roads and streets not classified as arterials or collectors are classified as local roads and streets. These facilities are characterized by their many points of direct access to adjacent properties and their relatively minor value in accommodating mobility.

11.9.2.2 Federal-Aid System

11.9.2.2.1 *Background*

The Federal-aid system consists of those routes within Nevada that are eligible for the categorical Federal highway funds. NDOT, working with the local governments and in cooperation with FHWA, has designated the eligible routes. The following briefly describes the components of the Federal-aid system.

11.9.2.2.2 *National Highway System*

The National Highway System (NHS) is a network of principal arterial routes identified as essential for international, interstate, and regional commerce and travel and for national defense. It consists of the Interstate highway system, logical additions to the Interstate system, selected other principal arterials and other facilities that meet the requirements of one of the subsystems within the NHS.

11.9.2.2.3 *Surface Transportation Program*

The Surface Transportation Program (STP) is a flexible funding program that provides Federal-aid funds for:

- highway projects on all functional classes (except facilities functionally classified as “local”),
- bridge projects on any public road (including “local” functional classes),
- transit capital projects, and
- public bus terminals and facilities.

The basic objective of STP is to provide Federal-aid for improvements to facilities not considered to have significant national importance (i.e., facilities not on the NHS) and to minimize the Federal requirements for funding eligibility. The Federal funds allocated to STP are comparable to those funds previously designated for use on the former Federal-aid primary, Federal-aid urban and Federal-aid secondary systems. STP funds are distributed to each State based on its lane-miles of Federal-aid highways, total vehicle-miles traveled on those highways, and estimated contributions to the Highway Trust Fund.

11.9.2.2.4 *Highway Bridge Program*

The Highway Bridge Program (HBP), formerly known as the Highway Bridge Rehabilitation and Replacement Program, provides funds for eligible bridges located on any public road. The HBP is the cornerstone of FHWA's efforts to correct, on a priority basis, deficient bridges throughout the nation. The number of structurally deficient and/or functionally obsolete bridges in Nevada compared to the number nationwide basically determines Nevada's share of HBP funds.

HBP funds available to non-State maintained facilities are based on the provision that no less than 15% of the funds must be used on public roads that are functionally classified as local roads (urban and rural) or rural minor collectors.

HBP funds can be used for total replacement or for rehabilitation. HBP funds can also be used for a nominal amount of roadway approach work to tie the new bridge in with the existing alignment or to tie in with a new gradeline. HBP funds cannot be used for long approach fills, causeways, connecting roadways, interchanges, ramps and other extensive earth structures.

Eligibility for HBP funding is based on a Sufficiency Rating (SR) (0-100). The SR is based on an equation that considers many aspects of a bridge (e.g., structural adequacy, safety, serviceability, functionality, detour length). The following applies:

1. <u>Replacement</u>. Bridges scheduled for replacement require an SR less than 50 and must be classified as structurally deficient or functionally obsolete.

2. <u>Rehabilitation</u>. Bridges scheduled for rehabilitation require an SR less than 80 and must be classified as structurally deficient or functionally obsolete.

3. <u>Exception</u>. If the cost of rehabilitation is greater than replacement, then coordination with FHWA is required to determine if the bridge can be replaced.

4. <u>10-Year Rule</u>. If a bridge has been rehabilitated or replaced with HBP funds, it is not eligible for additional HBP funds for 10 years.

5. <u>SR $\geq$ 80</u>. If a bridge has an SR greater than or equal to 80, it is not eligible for HBP funds.

Section 22.1.3 and Section 28.2.12.3 discuss the Sufficiency Rating in more detail.

11.9.2.3 Jurisdictional Responsibilities

This Section briefly discusses the jurisdictional responsibility for the public highway system in Nevada.

11.9.2.3.1 *State-Maintained System*

The Nevada State-Maintained System represents those public highways, roads and streets for which NDOT has direct jurisdictional responsibility for all planning, design, construction and maintenance. The State-Maintained System may be identified by the route shield used on the facility, which may be:

- an Interstate Route,
- a US Route, or
- a Nevada State Route.

Frontage roads are also on the State-maintained system. Note that the State-Maintained System is not equivalent to the Federal-aid System, which is based on the functional classification system. The Federal-aid System includes most State-maintained routes and selected higher functional classification facilities not on the State-Maintained System.

11.9.2.3.2 *County/Municipal System*

For all public roads and streets not on the State-Maintained System, either a county or local municipality has jurisdictional responsibility for the facility.

11.9.3 Roadway Definitions

The following defines selected roadway elements that often have an application to the roadway design portion of a bridge:

1. Average Annual Daily Traffic (AADT). The total volume of traffic passing a point or segment of a highway facility, in both directions, for one year, divided by the number of days in the year.

2. Average Daily Traffic (ADT). The total volume of traffic during a given time period, greater than one day and less than one year, divided by the number of days in that time period.

3. Average Daily Truck Traffic (ADTT). The total number of trucks passing a point or segment of a highway facility, in both directions, during a given time period divided by the number of days in that time period.

4. Cross Slope. The slope in the cross section view of the travel lanes, expressed as a percent or ratio, based on the change in horizontal compared to the change in vertical.

5. Design Hourly Volume (DHV). Typically, the 30th highest hourly volume for the future year used for design, expressed in vehicles per hour.

6. Design Speed. The maximum safe speed that can be maintained over a specified section of highway.

7. K-Values for Vertical Curves. The horizontal distance needed to produce a 1% change in longitudinal gradient.

8. Longitudinal Grade. The rate of roadway slope expressed as a percent between two adjacent Vertical Points of Intersection (VPI). Upgrades in the direction of stationing are identified as positive (+). Downgrades are identified as negative (–).

9. Median. On a multilane facility, the area (or distance) between the inside edges of the two traveled ways. Note that the median width includes the two inside (or left) shoulders.

10. Normal Crown (NC). The typical cross section on a tangent section of roadway (i.e., no superelevation).

11. Overpass. A grade separation where a highway passes over an intersecting highway or railroad.

12. Profile Grade Point (Finished Grade). The line at which the profile grade is measured on the pavement.

13. Roadway. The portion of a highway, including shoulders, for vehicular use. A divided highway includes two roadways.

14. Superelevation. The amount of cross slope provided on a horizontal curve to counterbalance, in combination with the side friction, the centrifugal force of a vehicle traversing the curve.

15. Superelevation Transition Length. The distance needed to transition the roadway from a normal crown section to the design superelevation rate. Superelevation transition length is the sum of the tangent runout (TR) and superelevation runoff (L) distances.

16. Traveled Way. The portion of the roadway for the movement of vehicles, exclusive of shoulders and auxiliary lanes.

17. Truck. A heavy vehicle engaged primarily in the transport of goods and materials, or in the delivery of services other than public transportation. For geometric design and capacity analyses, trucks are defined as vehicles with six or more tires.

18. Truck Percentage (T). The percentage of trucks in the total traffic volume on a facility.

19. Twenty-Year ADT. For new construction and reconstruction projects, the projected future traffic volume most often used in project design.

20. Underpass. A grade separation where a highway passes under an intersecting highway or railroad.

11.9.4 Roadway Cross Section (Bridges)

11.9.4.1 Profile Grade Point/Profile Grade Line

The location of the profile grade point (which is in the cross section view) and the profile grade line (which is in the elevation view) on the bridge must match those on the approaching roadway. The profile grade point location varies according to the type of highway and type of median.

The profile grade is located between the dense-grade and open-grade on bituminous pavements and the top of the concrete on concrete pavements. Adjustments to bridge elevations are required to match the riding surface of bituminous pavements. See Section 16.3 for more discussion.

11.9.4.2 Cross Slope

Bridges on tangent sections typically provide a uniform cross slope of 2.0% from the crown line to the concrete barrier rail. On rare occasions, the cross slope of the shoulder (and sometimes one or more of the travel lanes) on the approaching roadway is steeper than 2.0%; therefore, the roadway must be transitioned to a uniform 2.0% slope before it reaches the bridge; this is the responsibility of the roadway designer when designing the roadway approaches.

11.9.4.3 Bridge Roadway Widths

In general, bridge widths should match the approach roadway widths (traveled way plus shoulders plus auxiliary lanes), which are determined by the Roadway Design Division. However, in determining the width for major water crossings, consider the cost of the structure, traffic volumes and potential for future width requirements.

11.9.4.4 Sidewalks

11.9.4.4.1 *Warrants*

The Roadway Design Division determines the warrants for sidewalks on the approach roadway and, if provided, the sidewalks are carried across the bridge. Sidewalk requirements for each side of the bridge will be evaluated individually; i.e., placing a sidewalk on each side will be based on the specific characteristics of that side. However, typical NDOT practice is to place a sidewalk on both sides of the bridge.

11.9.4.4.2 *Cross Section*

The typical sidewalk width is 5′-6″ as measured from the gutter line to the back of the sidewalk; i.e., this width includes the width of curb. The cross slope on the sidewalk is 2% sloped towards the roadway.

11.9.4.4.3 *ADA Requirements*

In general, the Roadway Design Division is responsible for establishing NDOT criteria to comply with the requirements of the *Americans with Disabilities Act*. ADA requirements pertain to sidewalk features such as width, cross slope and longitudinal grade, surface type and texture, curb ramps, etc. The bridge designer should coordinate with the Roadway Design Division to ensure that any sidewalk across a bridge meets the ADA requirements.

11.9.4.5 Bicycle Accommodation

The bicycle is classified as a vehicle according to Nevada law, and bicyclists are granted all of the rights and are subject to all of the duties applicable to the driver of any other vehicle.

A bridge may need to be configured to accommodate bicycle traffic. This must be coordinated with the Statewide Bicycle/Pedestrian Coordinator in the Transportation Planning Division, who will refer to the State Transportation Improvement Program (STIP) to determine the necessary bicycle accommodation for a specific roadway segment. In general, the bicycle accommodation on the approaching roadway will be carried across the bridge. One method of accommodation is to provide a shoulder wide enough to accommodate bicycles. Although a 4-ft wide shoulder may be considered adequate for bicycle traffic on the roadway, the shoulder should be increased by 2 ft to provide a shy distance where barriers are present. Therefore, a 6-ft wide shoulder is considered the minimum shoulder width for bridges that are designed to carry bicycle traffic. In addition, on bridges, a minimum of 4 ft from the edge of the traveled way should be clear of drainage inlets.

If the approaching roadway includes a separate bicycle lane, then the width of the lane will be carried across the bridge. Requests for and accommodation for anticipated future bicycle lanes are only warranted when they are part of NDOT long-range plans (i.e., the STIP).

11.9.5 <u>Alignment at Bridges</u>

11.9.5.1 Horizontal Alignment

The roadway designer will determine the horizontal alignment at the bridge based on NDOT criteria adopted by the Roadway Design Division (e.g., curve radius, superelevation transition). From the perspective of the roadway user, a bridge is an integral part of the roadway system and, ideally, horizontal curves and their transitions will be located irrespective of their impact on bridges. However, practical factors in bridge design and bridge construction warrant consideration in the location of horizontal curves at bridges. The following presents, in order from the most desirable to the least desirable, the application of horizontal curves to bridges:

1. Considering both the complexity of design and construction difficulty, the most desirable treatment is to locate the bridge and its approach slabs on a tangent section; i.e., no portion of a horizontal curve or its superelevation development will be on the bridge or bridge approach slabs.

2. If a horizontal curve is located on a bridge, the superelevation transition should not be located on the bridge or its approach slabs. This will result in a uniform cross slope (i.e., the design superelevation rate) throughout the length of the bridge and bridge approach slabs.

3. If the superelevation transition is located on the bridge or its approach slabs, the road designer should place on the roadway approach that portion of the superelevation development that transitions the roadway cross section from its normal crown to a point where the roadway slopes uniformly; i.e., to a point where the crown has been removed. This will avoid the need to warp the crown on the bridge or the bridge approach slabs.

11.9.5.2 Vertical Alignment

The bridge designer and road designer will coordinate on the vertical alignment of the roadway across a bridge based on the criteria adopted by the Roadway Design Division. The following applies specifically to the vertical alignment at bridges:

1. <u>Minimum Gradient</u>. The minimum longitudinal gradient will be preferably 1% with an absolute minimum of 0.5%.

2. <u>Maximum Grades</u>. The Roadway Design Division has adopted NDOT's maximum grade criteria based on the highway type, design speed and rural/urban location.

3. <u>Vertical Curves</u>. Crest and sag vertical curves will be designed according to the criteria adopted by the Roadway Design Division. If practical, no portion of a bridge should be located in a sag vertical curve. If the bridge is located in a sag vertical curve, the low point of the sag should not be located on the bridge or the bridge approach slab.

11.9.5.3 Skew

Skew is defined as the angle between the end line of the deck and the normal drawn to the longitudinal centerline of the bridge at that point. Typically, the bridge skew is determined by the roadway alignment, and the bridge is designed to accommodate the skew. The impacts of skew on structural design are discussed at their respective locations throughout the *NDOT Structures Manual*. In general, skew angles of more than 30° will affect the design of structural elements.

11.9.6 <u>Highway Grade Separations</u>

For bridges over highways, the geometry of the underpassing roadway will determine the length of the overpassing bridge. See Chapter 21 for railroads underpassing a highway bridge.

11.9.6.1 Roadway Cross Section

The approaching roadway cross section, including any auxiliary lanes, bicycle lanes, sidewalks, etc., should be carried through the underpass. Desirably, also include the clear zone width for each side through the underpass, although this could prove to be prohibitively expensive. In addition, it is important to consider the potential for further development or traffic increases in the vicinity of the underpass that may significantly increase traffic or pedestrian volumes. If appropriate, an allowance for future widening may be provided to allow for sufficient lateral clearance for additional lanes. The need for accommodating future travel lanes will be made on a case-by-case basis.

11.9.6.2 Vertical Clearances

The vertical clearance for underpassing roadways will significantly impact the vertical alignment of the overpassing structure and may dictate the selection of the superstructure type. Figure 11.9-A summarizes NDOT's minimum vertical clearance criteria. Provide these clearances over the entire roadway beneath the bridge from edge of roadway to edge of roadway. Where barriers are present, provide the minimum vertical clearance from face of barrier to face of barrier. If a bridge is likely to be widened in the future, the minimum vertical clearance should also extend over the potential width of the widening.

11.9.6.3 Falsework

Falsework may unduly interfere with traffic passing beneath the structure or may create an unacceptable safety hazard. The bridge designer shall contact the District Office to judge the impact of using falsework over traffic. The minimum vertical clearance for falsework on all facilities is 16′-0″. Vertical clearances for collector and local roads may be reduced to 14′-6″ if a readily available detour for over-height vehicles is available and approved by the District Engineer and owner of the local road. All falsework shall have protection from high-load hits unless it has a vertical clearance of more than 18′-0″. Falsework can be protected by one of the following methods:

Facility Type	**Minimum Clearance**		
	New/Replaced Bridges	**Rehabilitated/Existing Bridges to Remain**	**Temporary Structures** (1)
Freeway Under	16′-6″	16′-0″ (5)	16′-0″
Arterial Under	16′-6″	16′-0″ (5)	16′-0″
Collector Under	16′-6″ (4)	16′-0″ (5)	16′-0″
Local Under	16′-6″ (4)	16′-0″ (5)	16′-0″
Highway Under Overhead Sign or Pedestrian Bridge (2)	18′-0″	18′-0″	N/A
Railroad Under Highway:			
(Non-Electrified)	23′-4″	23′-4″	21′-0″
Electrified (25-kv line) (3)	24′-3″	24′-3″	(6)
Electrified (50-kv line) (3)	26′-3″	26′-3″	(6)

Notes:

(1) *See Section 11.9.6.3 "Falsework." Contact the District Office and/or Railroad Company for concurrence on a case-by case basis.*

(2) *AASHTO* A Policy on Geometric Design of Highways and Streets *recommends a minimum vertical clearance of 17′-0″. NDOT has adopted a vertical clearance of 18′-0″ for both new structures and existing structures.*

(3) *The additional vertical clearance for electrification is acceptable only after the Railroad Company has submitted justification that it will provide electrification on the track line. See Section 21.1.3.3.*

(4) *AASHTO* A Policy on Geometric Design of Highways and Streets *recommends a minimum vertical clearance of 14′-6″. NDOT has adopted a vertical clearance of 16′-6″.*

(5) *AASHTO* A Policy on Geometric Design of Highways and Streets *recommends a minimum vertical clearance of 14′-6″. NDOT has adopted a vertical clearance of 16′-0″.*

(6) *Contact the Railroad Company for acceptable temporary vertical clearances.*

MINIMUM VERTICAL CLEARANCES

Figure 11.9-A

1. Overhead Barrier Girder. A stout girder is placed ahead of the falsework. The overhead barrier girder shall not be part of the falsework but its own structure. Overhead barrier girders should not be used over roadways with high traffic volumes or posted speed limits of more than 35 mph. Criteria for the design of the barrier girder shall be established on a case-by-case basis.

2. Over-height Detection System. An electronic eye detects an over-height vehicle and warns the vehicle driver and construction workers. Adequate stopping distances and turnaround areas must be provided. Evaluate the cost of the detection system and the cost and time to supply power when considering this detection system.

3. Adjacent Robust Structures. Falsework protection is not required if robust structures are located on the approaches to the falsework and if these structures have vertical clearances less than the falsework vertical clearance.

The designer shall coordinate with the Traffic Division to determine if lane closures, truck re-routing, detours or a complete road closure is an option where falsework is proposed. Additional traffic control and road user costs associated with the use of falsework shall be considered when selecting a structure type.

11.10 STRUCTURE LENGTH CALCULATIONS

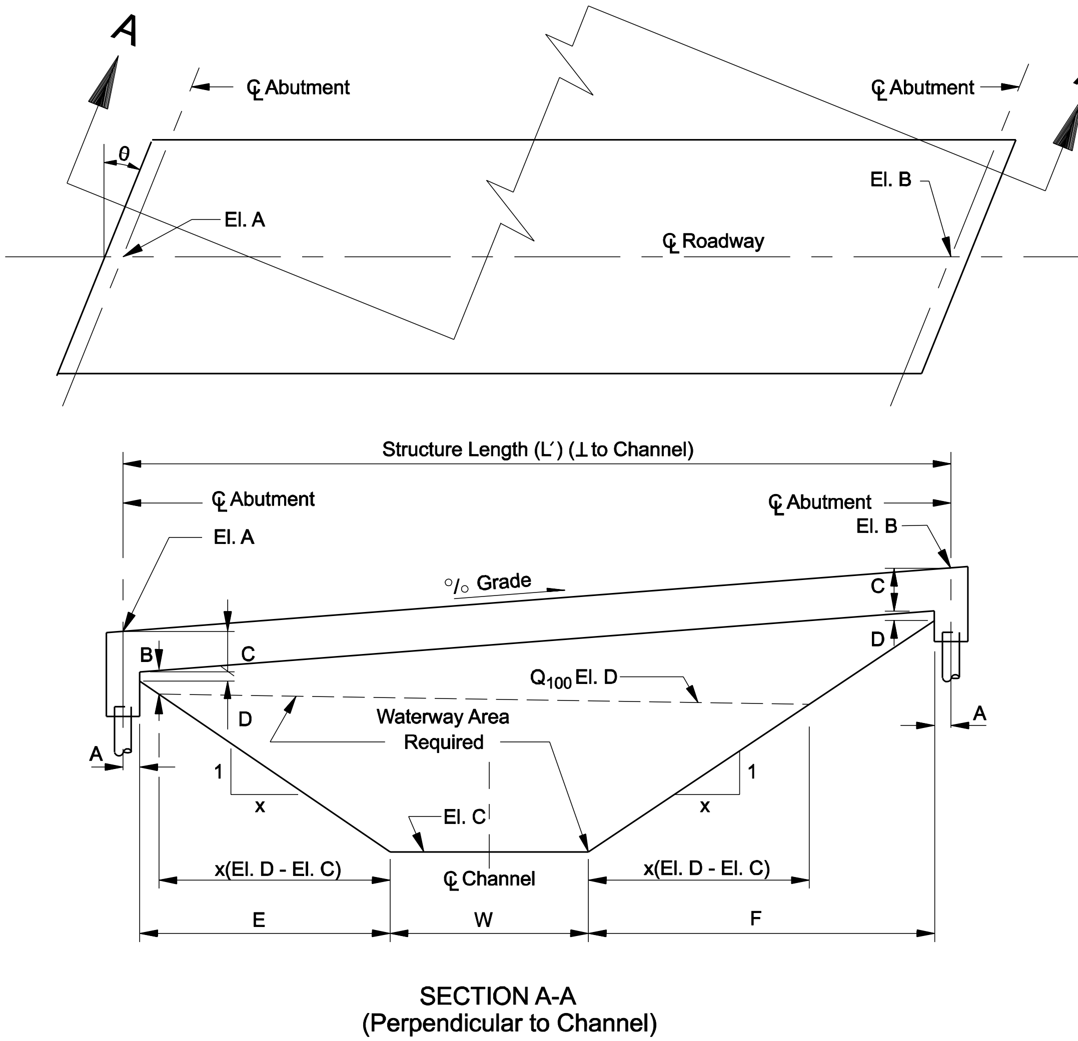
The overall structure length is measured from the centerline of abutment to the centerline of abutment. The following figures present criteria for determining structure length:

- Figure 11.10-A "Structure Length for Stream Crossings (Open Abutment)"
- Figure 11.10-B "Structure Length for Highway Crossings (Open Abutment)"
- Figure 11.10-C "Structure Length for Highway Crossings (Closed Abutment)"

The major variables that determine the structure length are:

- the use of an open abutment or closed abutment;
- seat width;
- for open abutments, the backslope;
- for waterway crossings, the waterway opening dimensions;
- for highway crossings, the width of the underpassing roadway cross section and clear zones;
- the longitudinal gradient along the roadway centerline; and
- the skew angle of the bridge.

The following figures assume that the bridge is on tangent and on a constant longitudinal gradient. The presence of a horizontal curve and/or a vertical curve will increase the length of the structure.



θ = Angle of skew
A = One half of the abutment width
B = Freeboard
C = Anticipated depth of superstructure
D = Distance from bottom of superstructure to top of abutment backslope (2′ minimum)
E = (x) (El. A – C – D – El. C)
F = (x) (El. B – C – D – El. C)
W = Width of channel (perpendicular to channel)
El. A = Elevation of top of deck
El. B = Elevation of top of deck
El. C = Bottom of channel elevation
El. D = Elevation of water surface at Q_{100}

L′ = Structure length perpendicular to channel from ℄ abutment to ℄ abutment

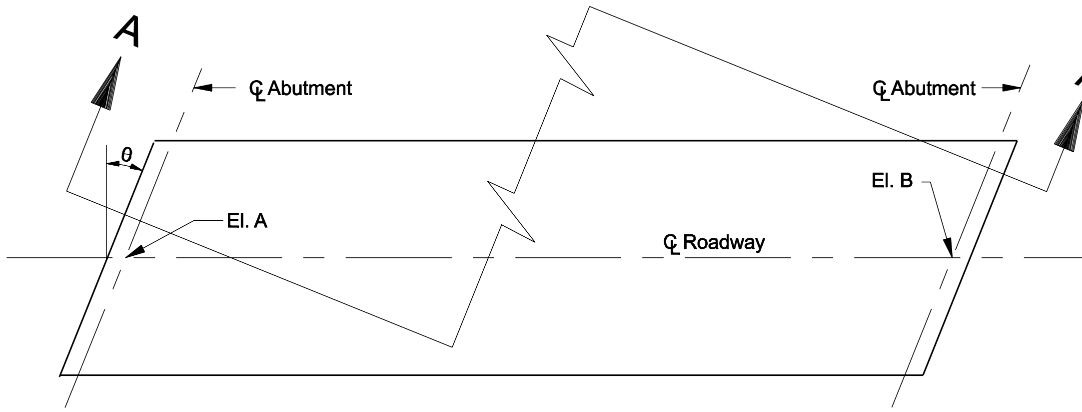
L = Structure length along ℄ roadway from ℄ abutment to ℄ abutment

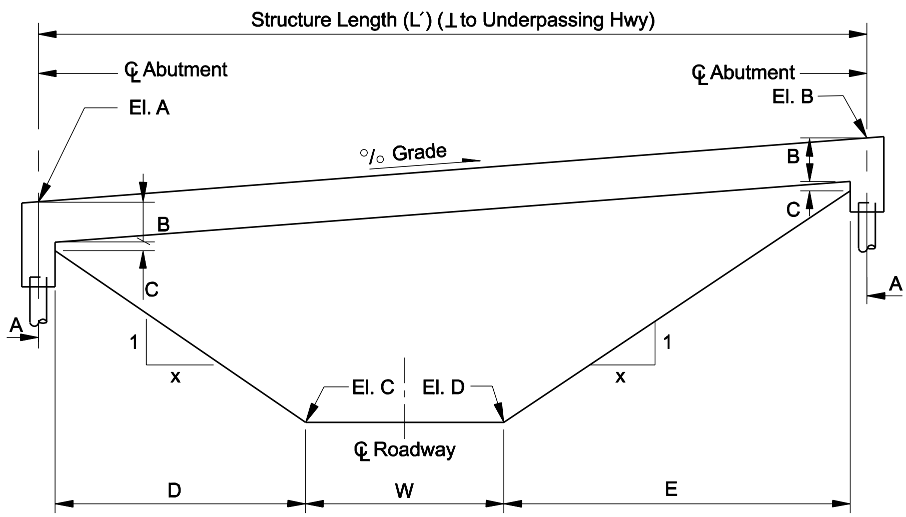
L′ = A + E + W + F + A

L = L′/cos θ

STRUCTURE LENGTH FOR STREAM CROSSINGS
(Open Abutment)

Figure 11.10-A





SECTION A-A
(Perpendicular to Underpassing Hwy)

θ = Angle of skew
A = One half of the abutment width
B = Anticipated depth of superstructure
C = Distance from bottom of superstructure to top of abutment slope (2′ minimum)
D = (x) (El. A – B – C – El. C)
E = (x) (El. B – B – C – El. C)
W = Width of underpassing roadway section
El. A = Elevation of top of deck
El. B = Elevation of top of deck
El. C = Elevation of toe of slope
El. D = Elevation of toe of slope

L′ = Structure length perpendicular to underpassing highway from ℄ abutment to ℄ abutment

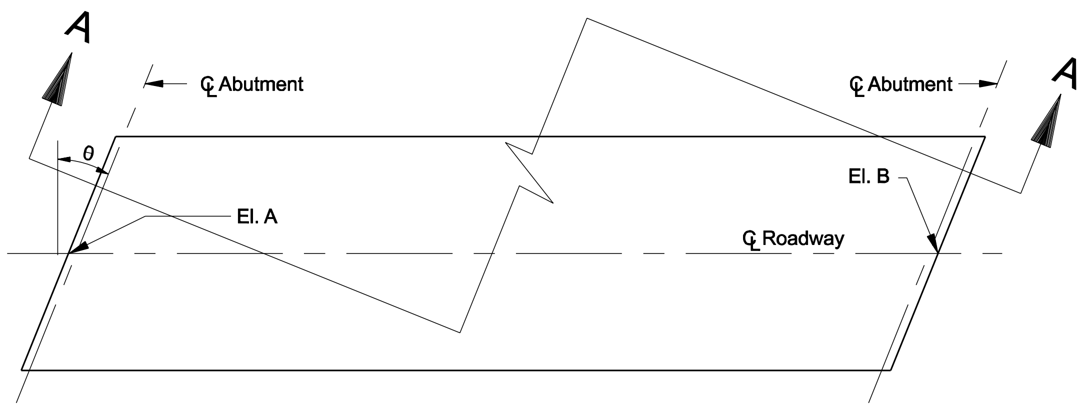
L = Structure length along ℄ roadway from ℄ abutment to ℄ abutment

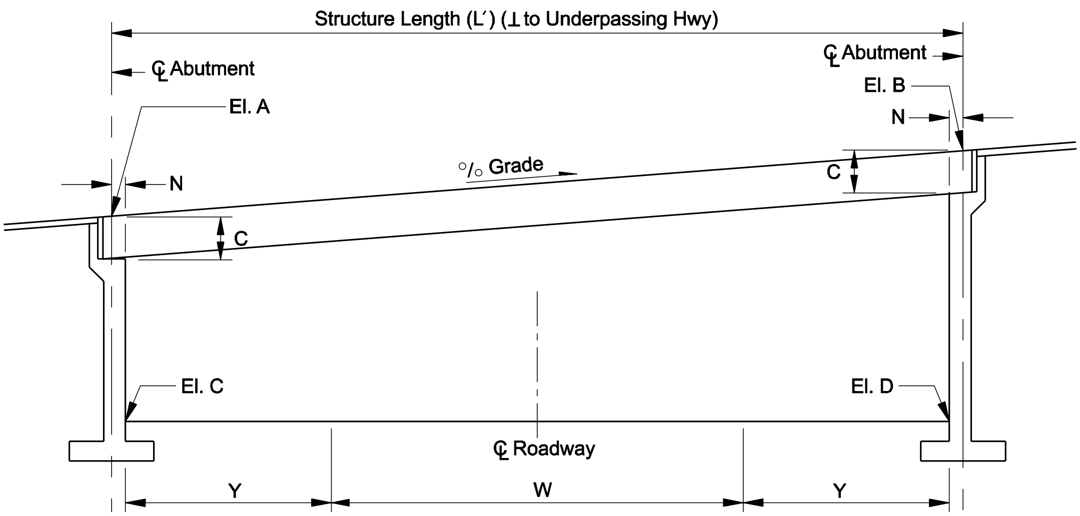
L′ = A + D + W + E + A

L = L′/cos θ

STRUCTURE LENGTH FOR HIGHWAY CROSSINGS
(Open Abutment)

Figure 11.10-B





SECTION A-A
(Perpendicular to Underpassing Hwy)

θ = Angle of skew
C = Anticipated depth of superstructure
N = One-half of seat width
W = Width of underpassing roadway section
Y = Width of clear zone, sidewalk, future expansion, bike lane
El. A = Elevation of top of deck at centerline of abutment
El. B = Elevation of top of deck at centerline of abutment
El. C = Elevation of toe of closed abutment
El. D = Elevation of toe of closed abutment

L′ = Structure length perpendicular to underpassing highway from ℄ abutment to ℄ abutment

L = Structure length along ℄ roadway from ℄ abutment to ℄ abutment

$L' = N + Y + W + Y + N$

$L = L'/\cos\theta$

STRUCTURE LENGTH FOR HIGHWAY CROSSINGS
(Closed Abutment)

Figure 11.10-C