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Chapter Fourteen
LOADS AND ANALYSIS

14.1 GENERAL

14.1.1 Introduction

Articles 1, 3 and 4 of the LRFD Bridge Design Specifications discuss various aspects of loads and analysis. Unless noted otherwise in Chapter Fourteen of the **Montana Structures Manual**, the LRFD Specifications apply to loads and analysis in Montana. Chapter Fourteen also presents additional information on MDT practices.

14.1.2 Types of Loads (Definitions)

1. Permanent loads. Loads which are always present in or on the bridge and do not change in magnitude during the life of the bridge. Specific permanent loads include:

   a. Gravitational Dead Loads:

      DC – dead load of all of the components of the superstructure and substructure, both structural and non-structural.

      DW – dead load of additional wearing surfaces and any utilities crossing the bridge.

      EL – accumulated lock-in, or residual, force effects resulting from the construction process, including the secondary forces from post-tensioning.

   b. Earth Pressures:

      EH – horizontal earth pressure.

      EV – vertical earth pressure from dead load of earth fill.

      ES – earth pressure from dead load of an earth surcharge.

      DD – downward skin friction on the sides of piles from settlement.

2. Transient loads. Loads which are not always present in or on the bridge or change in magnitude during the life of the bridge. Specific transient loads include:

   a. Live Loads:

      LL – vertical gravity loads due to vehicular traffic on the roadway.

      IM – dynamic load allowance due to moving vehicles, traditionally called impact.

      LS – earth pressure from vehicular traffic on the ground surface.

      BR – horizontal vehicular braking force.

      CE – horizontal centrifugal force from vehicles on a curved roadway.

   b. Water Loads:

      WA – pressure due to differential water levels, stream flow or buoyancy.

   c. Wind Loads:

      WS – horizontal and vertical pressure on superstructure or substructure due to wind.

      WL – horizontal pressure on vehicles due to wind.

d. Extreme Events:

EQ – horizontal loads due to earthquake ground motions.

CT – horizontal impact loads on piers or abutments due to vehicles or trains.

CV – horizontal impact loads due to aberrant ships or barges.

IC – horizontal static and dynamic forces due to ice action.

e. Superimposed Deformations:

TU – uniform temperature change due to seasonal variation.

TG – temperature gradient due to exposure of the bridge deck to solar radiation while the superstructure under the deck is shaded from the sun.

SH – differential shrinkage between different concretes or concrete and non-shrinking materials, such as metals and wood.

CR – creep of concrete or wood.

SE – the effects of settlement of substructure units on the superstructure.

FR – frictional forces on sliding surfaces from structure movements.

14.1.3 Limit States

Reference: LRFD Article 1.3.2.1

In the LRFD Specifications, the traditional design criteria have been grouped together with the groups termed limit states. The various limit states then have load combinations assigned to them. Components and connections of a bridge are designed to satisfy the basic LRFD equation for all limit states:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \qquad \text{(Equation 14.1.1)}$$

Where:

γ_i = load factor
Q_i = load or force effect
ϕ = resistance factor
R_n = nominal resistance
η_i = load modifier as defined in LRFD Equations 1.3.2.1-2 and 1.3.2.1-3

The left-hand side of Equation 14.1.1 is the sum of the factored load (force) effects acting on a component; the right-hand side is the factored nominal resistance of the component for the effects. The Equation must be satisfied for all the applicable limit state load combinations.

The load modifier η_i is either the product of, or the reciprocal of, the product of the factors η_D, η_R and η_i relate to ductility, redundancy and operational importance. Its location on the load side of the Equation may seem counter-intuitive because it seems more related to resistance than to load. It is on the load side for a logistical reason. When it modifies a maximum load factor, η_i is the product of the factors; when it modifies a minimum load factor, it is the reciprocal of the product. These factors are based on a 5% stepwise positive or negative adjustment, reflecting unfavorable or favorable conditions. They are somewhat arbitrary; their significance is in their presence in the LRFD Specifications and not necessarily in the accuracy of their magnitude.

MDT uses η_i values of 1.00 for all limit states.

14.1.4 Load Factors and Combinations

Reference: LRFD Article 3.4.1

LRFD Table 3.4.1-1 provides the load factors for all of the limit state load combinations of the Specifications. The significance of the strength limit state load combinations can be simplified as follows:

1. Strength I Load Combination. This load combination represents random traffic and the heaviest truck to cross the bridge in its 75-year design life. During this live-load

event, a significant wind is not considered probable.

2. Strength II Load Combination. This load combination represents an owner-specified permit load model. This live-load event will have less uncertainty than random traffic and thus a lower live-load load factor.

3. Strength III Load Combination. This load combination represents the most severe wind during the bridge's 75-year design life. During this extreme wind event, no significant live load would cross the bridge.

4. Strength IV Load Combination. This load combination represents an extra safeguard for long-span bridge superstructures (i.e., where the unfactored dead load exceeds seven times the unfactored live load). With long-span bridges, the live load becomes less significant. Thus, the only significant load factor would be the 1.25 dead-load maximum load factor. For additional safety, and based solely on engineering judgment, the load factor for DC has been arbitrarily increased to 1.5.

5. Strength V Load Combination. This load combination represents the simultaneous occurrence of an "average" live-load event and an "average" wind event with "average" load factors of 1.35 and 0.4, respectively.

Unlike the strength limit state load combinations, the service limit state load combinations are, for the most part, material-dependent. The Service I load combination is applied for the checking of cracking of reinforced concrete components and compressive stresses in prestressed concrete components. The Service II load combination is applied for checking permanent deformations of compact steel sections and slip of slip-critical (i.e., friction-type) bolted steel connections. The Service III load combination is applied for checking tensile stresses in prestressed concrete components. Finally, the Service I load combination is also used to calculate deformations and settlements of superstructure and substructure components.

The extreme-event limit state load combinations are applied for earthquakes (Extreme Event I), and various types of collisions (vessel, vehicular or ice) one at a time (Extreme Event II). The extreme-event limit states are different from the strength limit states because the event for which the bridge and its components are designed has a greater return period than the 75-year design life of the bridge.

The fatigue-and-fracture limit state load combination, although strictly applicable to all types of superstructures, only affects the proportions of a limited number of steel superstructure components.

In LRFD Table 3.4.1-1, the load factors for all of the permanent loads, shown in the first column of load factors, are represented by the variable γ_P. This reflects the fact that the strength and extreme-event limit states load factors for the various permanent loads are not constants, but they can have two extreme values. These two extreme values of the various permanent load factors, maximum and minimum load factors, are given in LRFD Table 3.4.1-2. Permanent loads are always present on the bridge, but the nature of variability is that the actual loads may be more or less than the nominal specified design values. Therefore, maximum and minimum load factors reflect this variability. The application of these permanent load factors is discussed in Section 14.2.

The load factors for the superimposed deformations for the strength limit states also have two specified values: a load factor of 0.5 for the calculation of stress and a load factor of 1.2 for the calculation of deformation. The greater value of 1.2 is used to calculate unrestrained deformations, such as a simple span expanding freely with rising temperature. The lower value of 0.5 for the elastic calculation of stress reflects the inelastic response of the structure due to restrained deformations. For example, one-half of the temperature rise would be used to elastically calculate the stresses in a constrained structure. Using 1.2 times the temperature rise in an elastic calculation would overestimate the stresses in the structure which resists the temperature inelastically through redistribution of the elastic stresses. The application of these load factors for the superimposed deformation is discussed in Section 14.4.5.

14.2 PERMANENT LOADS

14.2.1 General

Reference: LRFD Article 3.5

The LRFD Specifications specify seven components of permanent loads, which are either direct gravity loads or caused by gravity loads. New in this group is downdrag, "DD," which is a negative load in driven piles or drilled shafts as a result of consolidation of soil through which they are driven or drilled. Prestressing is considered, in general, to be part of resistance of a component and has been omitted from the list of permanent loads in Section 3 of the Specifications. However, when designing anchorages for prestressing tendons, the prestressing force is the only load effect, and it should appear on the load side of the LRFD Equation.

As discussed previously in Section 14.1.4 and shown in Table 3.4.1-2 of the LRFD Specifications, there are maximum and minimum load factors for the permanent loads. The maximum or minimum permanent-load load factors should be selected to produce the more critical load effect. For example, in continuous superstructures with relatively short-end spans, transient live load in the end span causes the bearing to be more compressed while transient live load in the second span causes the bearing to be less compressed and perhaps lift up. To check the maximum compression force in the bearing, place the live load in the end span and use the maximum DC load factor of 1.25. To check possible uplift of the bearing, place the live load in the second span and use the minimum DC load factor of 0.90.

In superstructure design, maximum permanent-load load factors are used almost exclusively, with the most common exception being uplift of a bearing. Maximum and minimum permanent-load load factors are used routinely for substructure design. The application of these load factors for substructure design is discussed more completely in Section 14.3.

14.2.2 Uplift

Reference: LRFD Article 3.4.1

In the former AASHTO Standard Specifications, uplift was treated as a separate load combination. With the introduction of maximum and minimum load factors in the LRFD Specifications, load situations such as uplift where a permanent load (in this case a dead load) reduces the overall force effect (in this case a reaction) have been generalized. Permanent load factors, either maximum or minimum, must be chosen for each load combination to produce extreme force effects.

Secondary forces from pre- or post-tensioning are included in the permanent load, EL. As specified in LRFD Table 3.4.1-2, a constant load factor of 1.0 should be used for both maximum and minimum load factors.

14.2.3 Deck Slab

Reference: LRFD Article 9.7.3

MDT uses the Traditional Design methodology outlined in Article 9.7.3 of the LRFD Specifications, unless otherwise approved by the Bridge Design Engineer. For bridge deck and slab design requirements, see Chapters 15 and 16 of this **Manual**.

Bridge deck dead load (DL) for design consists of composite and non-composite components.

Non-composite loads include the weight of the plastic concrete, forms and other construction loads typically required to place the deck. Calculate the non-composite DL using the full slab volume including haunch volumes times the unit weight of concrete. Use 24 kN/m^3 to account for concrete weight and construction loads.

Composite loads are applied to combined beam and slab section properties and include the weight of any curb, rail or barrier placed after the deck concrete has hardened. In addition, include an allowance for a future overlay of $5x10^{-4}$ MPa over the entire deck area between the rail or curb faces. Future wear should not be included for slabs of buried structures.

14.2.4 Dead Load Distribution

Reference: LRFD Article 4.6.2.2.1

For distribution of the weight of plastic concrete including that of the integrated sacrificial wearing surface, the formwork should be assumed to be simply supported between interior beams and cantilevered over the exterior beams.

Superimposed dead loads (e.g., curbs, barriers, sidewalks, parapets, railings, future wearing surfaces), if placed after the deck slab has cured, may be distributed equally to all girders. In some cases, such as staged construction and heavier utilities, a more accurate distribution of superimposed dead loads is warranted.

14.2.5 Downdrag on Deep Foundations

Deep foundations (i.e., driven piles and drilled shafts) through unconsolidated soil layers may be subject to downdrag, DD. Downdrag is a negative load on the deep foundation as the soil surrounding it consolidates and settles. This additional load is calculated as a skin-friction effect. If a bridge is at a site where downdrag is anticipated, MDT practice is to mitigate instead of quantify. Section 20.3.4 discusses two potential downdrag mitigation methods.

14.3 TRANSIENT LOADS

14.3.1 General

The LRFD Specifications recognize 19 transient loads. Static water pressure, stream pressure, buoyancy and wave action have been integrated as water load, WA. Creep, settlement, shrinkage and temperature (CR, SE, SH, TU and TG) have been elevated in importance to "loads," being superimposed deformations causing force effects. Vehicular braking force, BR, has been increased considerably to reflect the improvements in the mechanical capability of modern trucks.

14.3.2 Vehicular Live Load (LL)

14.3.2.1 General

Reference: LRFD Articles 3.6.1.1, 3.6.1.2 and 3.6.1.3

For short and medium span bridges, which predominate in Montana, vehicular live load is the most important load. The HL93 live-load model is a notional load in that it no longer qualifies as a true representation of actual truck weights. Instead, the force effects (i.e., the moments and shears) due to the superposition of vehicular and lane load are a true representation of the force effects due to actual trucks.

The components for each design lane are:

1. either the familiar MS18 truck, now called the design truck, or a 220-kN tandem, similar to the Alternate Loading, both of the former Standard Specifications; and

2. a 9.5-kN/m uniformly distributed lane load, similar to the lane load of the former Standard Specifications but without any of the associated concentrated loads.

Note that the dynamic load allowance, IM, of 0.33 is applicable only to the constituent axle and wheel loads of the design truck and the design tandems, but not to the uniformly distributed lane load.

The multiple presence factor of 1.0 for two loaded lanes, as given in LRFD Table 3.6.1.1.2-1, is the result of the Specifications' calibration process, which has been normalized relative to the occurrence of two side-by-side, fully correlated, or identical, vehicles. The multiple presence factor of 1.2 for one loaded lane should be used wherever a single design tandem or single design truck or their constituent axle or wheel loads govern, such as in overhangs, decks, etc. The factor for one loaded lane should never be used for fatigue loads.

The LRFD Specifications retain the traditional design lane width of 3.6 m and the traditional spacing of the axles and wheels of the MS18 truck. Both vehicles (the design truck and design tandem) and the lane load occupy a 3.0-m width placed transversely within the design lane for maximum effect. The lane load is no longer an alternative to the truck, but one applied simultaneously to the truck.

The Specifications require that two closely spaced design trucks superimposed on the lane load be applied on adjacent spans of continuous structures for negative moments and reactions. The reduced probability of such an occurrence of fully correlated, or identical, vehicles is accommodated by multiplying the resulting force effects by 0.9. This sequence of highway loading is specified for negative moment and reaction due to the shape of the influence lines for such force effects. It is not extended to other structures or portions of structures because it is not expected to govern for other influence-line shapes.

14.3.2.2 Load Applications

14.3.2.2.1 Use of Two Design Trucks

Reference: LRFD Article 3.6.1.3.1

The combination of the lane load and a single vehicle (either a design truck or a design tandem) does not always adequately represent the real-life loading by two closely spaced heavy vehicles, interspersed with other lighter traffic. Two design trucks, with a clear distance not less than 15 000 mm between them and with an adjustment factor of 0.90 will approximate a statistically valid representation.

In positioning the two trucks to calculate the negative moment over an internal support of a continuous girder, spans should be at least approximately 2810 mm in length to position a truck in each span's governing position. If the spans are larger than 2810 mm in length, the trucks remain in governing positions but, if they are smaller than 2810 mm, the maximum force effect can only be attained by trial-and-error with either one or both trucks in off-positions (i.e., non-governing positions for each individual span).

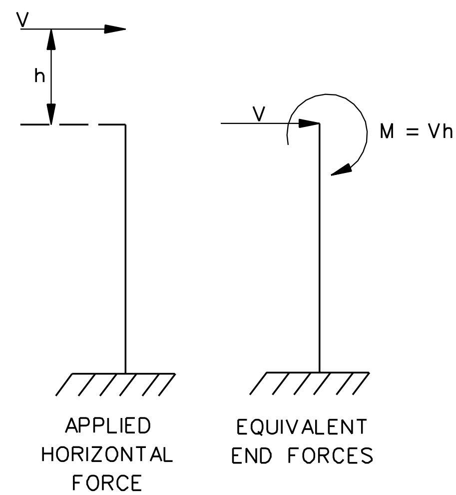
In any case, the moment can be calculated using the influence ordinates directly under each truck's axles.

14.3.2.2.2 Application of Horizontal Superstructure Forces to the Substructure

The transfer of horizontal superstructure forces to the substructure is dependent on the type of superstructure-to-substructure connection. Connections can be fixed, pinned or free for both moment and shear. Although expansion shoes are assumed to be free connections, a horizontal force transfer due to friction should be considered in a conservative nature. Include friction forces where design loads would increase, but neglect friction forces where design loads would decrease.

If the horizontal superstructure force is being applied to the substructure through a pinned connection, there is no moment transfer. Apply the superstructure force to the substructure at the connection.

For a fixed or moment connection, apply the superstructure horizontal force with an additional moment to the substructure as shown in Figure 14.3A. The additional moment is equal to the horizontal force times the distance between the force's line of action and the point of application.



TRANSFER OF HORIZONTAL SUPERSTRUCTURE FORCE TO SUB-STRUCTURE THROUGH MOMENT CONNECTION

Figure 14.3A

14.3.2.3 Fatigue Load

Reference: LRFD Article 3.6.1.4.1

The LRFD Specifications define the fatigue load for a particular bridge component by both magnitude and frequency. The magnitude of the fatigue load consists of a single design truck per bridge with a load factor of 0.75; i.e., an MS13.5 truck. This produces a considerable reduction on the stress range in comparison with the stress ranges of the former Standard Specifications. However, fatigue designs using the LRFD Specifications are virtually identical to those of the Standard Specifications. This is accomplished through an increase in the frequency from values on the order of two million cycles in the Standard Specifications to frequencies on the order of tens and hundreds of millions of cycles. This change to more realistic stress ranges and cycles was made to increase the designer's understanding of the extreme fatigue lives of steel bridges.

14.3.2.4 Vehicular Centrifugal Force (CE), Vehicular Braking Force (BR) and Wind on Live Load (WL)

Reference: LRFD Articles 3.6.3, 3.6.4 and 3.8.1.3

Vehicular centrifugal forces, vehicular braking forces and wind on live load shall be applied longitudinally or transversely, as appropriate, at a distance of 1800 mm above the roadway's profile grade.

14.3.2.5 Application of Live Load to Piers

Reference: LRFD Article 3.6.1.3.1

To promote uniformity in application of live load to pier bents, hammerhead piers and similar substructures, the following procedure is suggested unless a more exact distribution of loads is used:

1. The live load distribution factor for each girder shall be determined assuming the deck is acting as a simple beam between interior girders and as a cantilever spanning from the first interior girder over the exterior girder.

2. Design lanes shall be placed on the bridge to produce maximum force effect for the component under investigation. The HL93 live load shall be placed within its individual design lane to likewise produce the maximum effect. One, two or more design lanes shall be considered in conjunction with the multiple presence factors of LRFD Table 3.6.1.1.2.-1, as can be accommodated on the roadway width.

3. Two closely spaced design trucks superimposed over the lane load as specified in LRFD Article 3.6.1.3 for negative moment in continuous girders and interior reactions shall be used with a distribution factor derived as discussed above in a line-girder analysis to determine the reaction on interior piers.

14.3.3 Friction Forces (FR)

Reference: LRFD Article 3.13

Section 19.3 discusses friction forces within the context of bearings.

14.3.4 Earthquake Effects

Reference: LRFD Article 3.10

Montana is one of several states in which a high seismic risk exists. The risk is not uniform across the state, with much of the eastern part of the state relatively non-seismic. The probability of high seismic acceleration is predicted only for the Missoula and Butte Districts.

Article 3.10.3 of the LRFD Specifications requires that each bridge be classified according to its Importance Category. In Montana, all bridges are designed as "Other Bridges."

MDT has developed a program to evaluate proposed new bridges in the State highway system to ensure that they meet the AASHTO criteria for seismic design. The Seismic Unit supports the Bridge Design Section in the seismic design and analysis of new bridges. In this capacity, the Unit performs a significant amount of preliminary design work for bridges within the context of addressing seismic vulnerability.

14.3.5 Ice Forces on Piers

Reference: LRFD Article 3.9

During the Field Review or Survey Phase of the project, the Bridge Area Engineer will contact the local landowners, the county and/or local maintenance personnel to determine if ice problems exist at the bridge site. Discussions should include whether or not ice jams are common, if localized flooding or roadway overtopping occurs, and whether or not the ice at the bridge site is in large floes or broken chunks. If there is an existing bridge, it should be inspected for ice damage. Tree bark may also show scarring from ice damage. This information should help in determining an appropriate ice elevation. The ice force is usually applied at the design high water, unless these investigations or specific site circumstances indicate otherwise. Do not use the reduction in ice forces as suggested by LRFD C3.9.2.3.

For uniformity of State practice, use the following values for thickness of ice "t":

1. "t" = 460 mm for the Missouri River below Loma, the Yellowstone River below Laurel and the Milk River below Malta.

2. "t" = 300 mm for anything else.

Use the following values for pressure:

1. 1.15 MPa for the Missouri River below Loma and the Yellowstone River below Laurel.

2. 0.77 MPa generally statewide, except 0.38 MPa for small streams or rivers that usually do not freeze over in winter or where ice movement would consist mostly of broken fragments and disintegrated ice. This includes larger rivers in the western part of the state such as the Flathead, Kootenai and Clark Fork below Missoula.

These values represent engineering data for design. For the information to be presented in the construction plans, refer to Figure 14.3B. Use the "small stream" reduction factor of 0.5 as allowed by Article 3.9.2.3 for all streams with a width less than 90 m at the mean water level. In the absence of better information, using the width of the channel at Q_2 flows instead of the mean water elevation will result in a conservative design. Consider separate channel widths if there are islands, other bridges or channel features in the upstream vicinity of the bridge that would break up or otherwise prevent large unbroken ice floes from impacting the bridge.

14.3.6 <u>Live Load Surcharge (LS)</u>

Reference: LRFD Article 3.11.6.2

When reinforced concrete approach slabs are provided at bridge ends, live load surcharge need not be considered on the end bent; however, the reactions on the end bent due to the axle loads on the approach slabs shall be considered.

It is MDT policy to design the end bents to allow the eventual use of approach slabs but not to initially use them. This allows for an approach slab to be added later. Thus, the end bents must be able to resist the reactions due to axle loads on an approach slab and the lateral pressure due to the live load surcharge but not both in combination.

Design Parameters Used			Information to show in Plans		
Pressure (MPa)	t = 300 (mm)	t = 460 (mm)	Light	Moderate	Severe
0.38	✓		✓		
0.77	✓			✓	
1.15		✓			✓

ICE LOADING PARAMETERS
Figure 14.3B

14.4 ELASTIC STRUCTURAL ANALYSIS

14.4.1 General

Reference: LRFD Article 4.5.2

The LRFD Specifications are a hybrid design code in that the force effects on the load side of the LRFD equation are determined through elastic analysis procedures in most cases, yet the components' resistances are based on inelastic responses. The hybrid nature of structural design is acceptable based on the assumption that the inelastic component of structural performance will always remain relatively small because of redistribution of force effects. This redistribution is assured by providing adequate redundancy and ductility of the structures, which is MDT's general policy for the design of bridges.

Section 14.4 discusses the approximate analysis of girder-slab superstructures and their longitudinal and transverse components. The Section also provides methods of analysis for distribution of lateral loads such as wind and centrifugal force by frame action and/or diaphragms and for axial and flexural effects of imposed deformations such as elastic shortening, creep, shrinkage, temperature and settlement.

14.4.2 Influence Lines

Influence lines can serve as a tool to determine load positions for maximum effect and for evaluating the magnitude of that effect. Constructing an influence line can consist of dividing the structure into eight or ten equal intervals and calculating the force effects due to a unit load at each resulting node.

Recognizing that the influence line is essentially a deflection diagram drawn for a unit relative displacement introduced into the structure at the point of interest can simplify the process. For flexure, consider the relative displacement to represent a unit rotation.

14.4.3 Distribution of Live Load in Multi-Girder Superstructures

Reference: LRFD Article 4.6.2.2.1

14.4.3.1 General

Distribution, for the purpose of this Section in the **MDT Structures Manual**, means the determination of the maximum portion of the total applied live load that may be carried by an individual girder of the superstructure.

14.4.3.2 Load Distribution Factors in the LRFD Specifications

Reference: LRFD Articles 4.6.2.2.1, 4.6.2.2.2 and 4.6.2.2.3

LRFD Article 4.6.2.2.2 presents several common bridge superstructure types, with empirically derived equations for live load distribution factors for each one. Each distribution factor gives a fraction of a lane live load to apply to a girder to evaluate it for moment or for shear. The factors account for interaction among loads from multiple lanes.

The empirical formulas result from regression analyses performed on results of finite element analyses of a large sample of typical superstructures. The equations are intended to produce results within 5% of the finite element analyses on which they rely. See *Distribution of Wheel Loads on Highway Bridges* by T. Zokaie, T. A. Osterkamp and R. A. Imbsen, Final Report, NCHRP Project No. 12-26, for details on the development of the distribution factors.

Distribution factors simplify the design process and minimize potential modeling errors. They reduce the problem of modeling the entire bridge from a two- or three-dimensional analysis down to a one-dimensional analysis of a girder.

Some assumptions that allow this model simplification are:

1. Relative stiffness among different parts of a girder determines longitudinal distribution of live load moment. Deck stiffness determines the transverse distribution. Live load moment distribution factors must include properties of the girder and deck. The calculation of live load moment distribution then becomes an iterative process, in which the designer assumes properties, tests the moment distribution and resulting stresses, then modifies section properties and repeats the process.

2. Force effects, moments and shears that control design consist of the extreme case for each one. Because extreme effects can occur anywhere along the girder, the extreme moment and the extreme shear rarely occur together in the same location or due to the same loading.

3. Distribution factors assume the same vehicle or load in all lanes. This assumption makes analyzing special permit vehicles difficult.

4. The distribution factors represent placing loads in design lanes to generate the extreme effect in a specific girder. The location of design lanes is not related to the location of striped lanes on the bridge. Summing all the distribution factors for all the girders produces a number of design lanes greater than the bridge can carry. This occurs because each girder must be designed for the maximum load it could be subjected to.

14.4.3.3 Refined Analysis in the LRFD Specifications

Reference: LRFD Articles 4.6.2.2 and 4.6.3

The more sophisticated distribution-factor equations are analytically superior to the old "S over" factors which have been used for bridges with spans and girder spacings far beyond those for which they were originally developed.

The tables of distribution factors given in LRFD Article 4.6.2.2 include a column entitled "Range of Applicability." The LRFD Specifications suggests that bridges with parameters falling outside the indicated ranges be designed using the refined analysis requirements of LRFD Article 4.6.3. In fact, these ranges of applicability do not necessarily represent limits of usefulness of the distribution-factor equations, but they represent the range over which bridges were examined to develop the equations. Other states have conducted parametric studies to extend these ranges for typical bridges in their states which have demonstrated that the factors can be used far outside of the range of certain parameters which were specifically studied. Therefore, MDT policy is to use refined analysis only with the approval of the Bridge Design Engineer and only where the simple distribution factors are clearly inadequate. One example may be where the overhang limitations are exceeded.

Refined analysis includes both two- and three-dimensional models. The study that developed the simple distribution factors also investigated refined analysis methods. The study showed that the extra complication of three-dimensional analysis provides no additional value when compared to a more simple two-dimensional grid analysis. Typically, in a grid analysis, longitudinal elements represent the girders including any composite deck, and the transverse elements represent the deck. LRFD Article 4.6.3.3 gives general requirements for grid analysis in terms of numbers of elements and aspect ratios.

14.4.3.4 Continuous Frames

Reference: LRFD Articles C5.7.3.2, 5.8.3.2 and 5.11.1.1

Centerline distances shall be used in the analysis of continuous frame members.

For concrete frames, the value of the moment of inertia for the computation of flexural stiffness of slabs, girders, columns, etc., shall be based on the gross concrete section; the effect of reinforcement may be neglected.

Moments used for designing a section at the support shall be based on the moment value at the face of rectangular columns and at the face of an equivalent square for round columns.

The critical section for bond shall be taken at the same place as for negative bending, and the shear used for computing bond shall be based on the same loading and section as for negative bending. Bond should also be investigated at planes where changes of section or of reinforcement occur and at the point of inflection. The flexural bond stress need not be considered in compression nor in those cases of tension where anchorage bond is less than 0.8 of the permissible.

The critical section for shear shall be the "d" distance from the support, as stated in the LRFD Specifications, except when concentrated loads fall within the "d" distance. In this case, the shear shall be checked at the point load and adequate stirrups provided to that point.

14.4.3.5 Overhang

Reference: LRFD Articles 4.6.2.2.1, 4.6.2.2.2d and 4.6.2.2.3b

For the purpose of live-load distribution, large overhangs (i.e., those requiring refined analysis) are defined as those where the roadway portion of the overhang exceeds 1675 mm for I-shaped steel or concrete girders. Because overhang dimensions are limited in Section 15.4.1.2 of this **Manual**, large overhang distribution considerations do not need to be considered for MDT practice.

For economy of construction, allow for the reuse of girders if the bridge is widened in the future, to reduce the probability of misplacing seemingly identical but actually different girders on the construction site; all prestressed girders are fabricated to the governing condition, interior or exterior. For economy in fabrication, steel girders are also typically fabricated to the governing condition in terms of web-plate and flange-plate sizes and transitions.

14.4.3.6 Number of Girders

Reference: LRFD Article 9.7.2.4

Studies indicate that the cost of a bridge is directly proportional to the number of girders in the cross section. Two-girder arrangements are discouraged because of concerns for the level of redundancy. Article 9.7.2.4 of the LRFD Specifications implies that, with a 200-mm thick non-prestressed concrete deck slab, the girder spacing can safely be extended to approximately 4000 mm. Although not commonly used, a three-girder layout is possible for a bridge site that carries a narrow roadway where span lengths are not at the maximum for the girder type used.

14.4.4 <u>Wind Load Distribution by Frame Action</u>

Reference: LRFD Articles 3.4.1, 3.8.1 and 4.6.2.7.1

Wind load in the LRFD Specifications is addressed in LRFD Article 3.8.1. The basic wind load, acting normal to a surface for a 160-km/h wind velocity at a height not exceeding 10 m above ground level, is 2.4 kN/m^2. Surfaces exposed to the wind should normally include that of the girders, deck, curbs and/or barriers.

In accordance with Article 3.4.1 of the LRFD Specifications, the effects of wind load should be investigated for strength Limit States III and V. For Limit State III, wind load at full value should be considered with a load factor of 1.4. The absence of live load at this limit state reflects that, above approximately 90-km/h wind velocity, vehicles become dynamically unstable and tend to stay off the bridges. For Limit State V, wind load should be assumed to be acting on both structural and vehicular surfaces, with a load factor of 0.40. This factor is actually the product of the 1.40 load factor and the square of the 90/160 wind velocity ratio.

As specified in LRFD Article 4.6.2.7, wind load may be assumed to be distributed in either of the following three ways:

1. Method 1. For typical MDT practice, the web is laterally supported at the respective centers of the deck and the lower flange. In this case, the lower flange is acting as a lateral beam transmitting wind load on the lower half of the outside girder either to intermediate diaphragms or to the bearings.

   Note: Special circumstances may require Method 2 or 3.

2. Method 2. Horizontal wind bracing in the plane of the flange can distribute the wind load among adjacent flanges directly to the bearings. Except for the largest of bridges, horizontal wind bracing is not necessary nor economical.

3. Method 3. The web is acting as a vertical cantilever, framing into the deck and fixed at its centerline. Maximum vertical flexural stresses occur due to this action at the point where the web joins the top flange, and these stresses must be investigated. In concrete girders, these stresses are normally considerably less than the cracking strength of concrete.

   Where diaphragms are used, investigation of vertical flexural stresses is not required.

   In rolled beams, these stresses are also usually small. This type of action, however, should be investigated in welded plate girders only where the transverse (vertical) stiffeners are welded only to the top flange. If the vertical stresses without intermediate diaphragms are within specified limits, none are required; however, vertical stiffeners may be necessary for stability or other reasons during and/or after construction.

   For composite deck-girder construction, the shear connectors or extended stirrups normally have sufficient reserve to resist the small vertical wind moment and, thus, no investigation is required. In case of non-composite construction, this action should not be used.

The following example illustrates one method of calculation of force effects in an intermediate diaphragm of truss construction. As shown in Figure 14.4A, the 96.0-kN lateral force is the reaction of wind load as carried to the diaphragm by the lower flange. The resultant moment with respect to the top chord of the diaphragm is 96 x 1.5 = 144 kN-m. Vertical forces acting on the four girders are computed on the basis that the diaphragm is infinitely stiff. An inertia-like stiffness of the girder is calculated as:

$$I = \sum_{1}^{n} (1)(x^2) \qquad \text{(Equation 14.4.1)}$$

Where:

1 = represents each girder as a unit

x = distance of the girder from the center of the superstructure

n = number of girders

The resistance for any girder is then:

$$S = I \div x \text{ and } V = M \div S \qquad \text{(Equation 14.4.2)}$$

For the given four-girder superstructure:

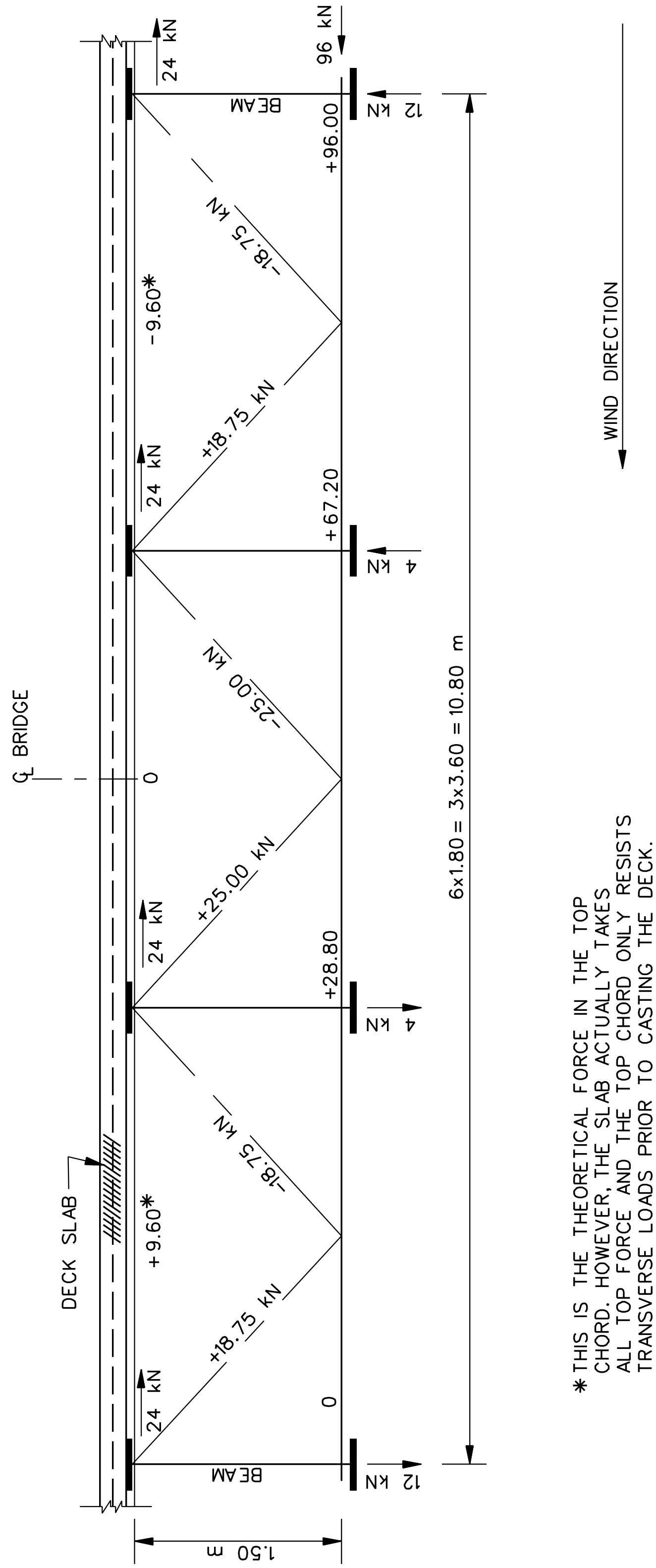
$I = 2[1.8^2 + 5.4^2] = 64.8\ m^2$

For the outside girders:

$S_o = 64.8 \div 5.4 = 12.0$ m
$V_o = 144 \div 12.0 = \pm 12.0$ kN

For the inside girders:

$S_i = 64.8 \div 1.8 = 36.0$ m
$V_i = 144 \div 36.0 = \pm 4.0$ kN



INTERMEDIATE DIAPHRAGM

Figure 14.4A

Lateral distribution of the wind force among the girders is indeterminate with the outside and inside girders being identical, or close to identical; however, a uniform distribution is acceptable. Accordingly:

$H_o = H_i = 96.0 \div 4 = 24.0$ kN.

Figure 14.4A indicates the calculated truss member forces for the basic wind load in addition to the cross-sectional dimensions of the superstructure. It should be noted for design that wind load is reversible in the diaphragm. Positive member force means compression, negative tension. The top chord is sized to resist construction loads prior to the hardening of the concrete deck, not design wind loads.

Figure 14.4B illustrates the configuration of a truss-like diaphragm required at bearing points. This truss is the reverse version of that selected in Figure 14.4A. The selections are arbitrary in that both systems are acceptable for either application; they are provided for completeness. Furthermore, the system provided in Figure 14.4B can be used for a strut-and-tie analysis of solid concrete diaphragms.

The total wind force is taken as 480 kN. Vertical forces on the girders are calculated the same way as for the intermediate diaphragm. The wind force is uniformly distributed (120 kN) at both the top and bottom of girders. Force effects in truss members are shown in Figure 14.4B.

14.4.5 Superimposed Deformations

Reference: LRFD Article 3.12

14.4.5.1 General

Superimposed deformations have little effect on common typical bridges such as slab-on-girder bridges. On these bridges, only the effects of temperature are typically considered in sizing expansion devices or in considering stresses and deformation in integral-type bridges. Superimposed deformations have special significance for some less common bridges, such as segmentally constructed concrete bridges.

Superimposed deformations include:

1. elastic shortening (ES),
2. creep (CR),
3. shrinkage (SH),
4. temperature (TU and TG), and
5. settlement (SE).

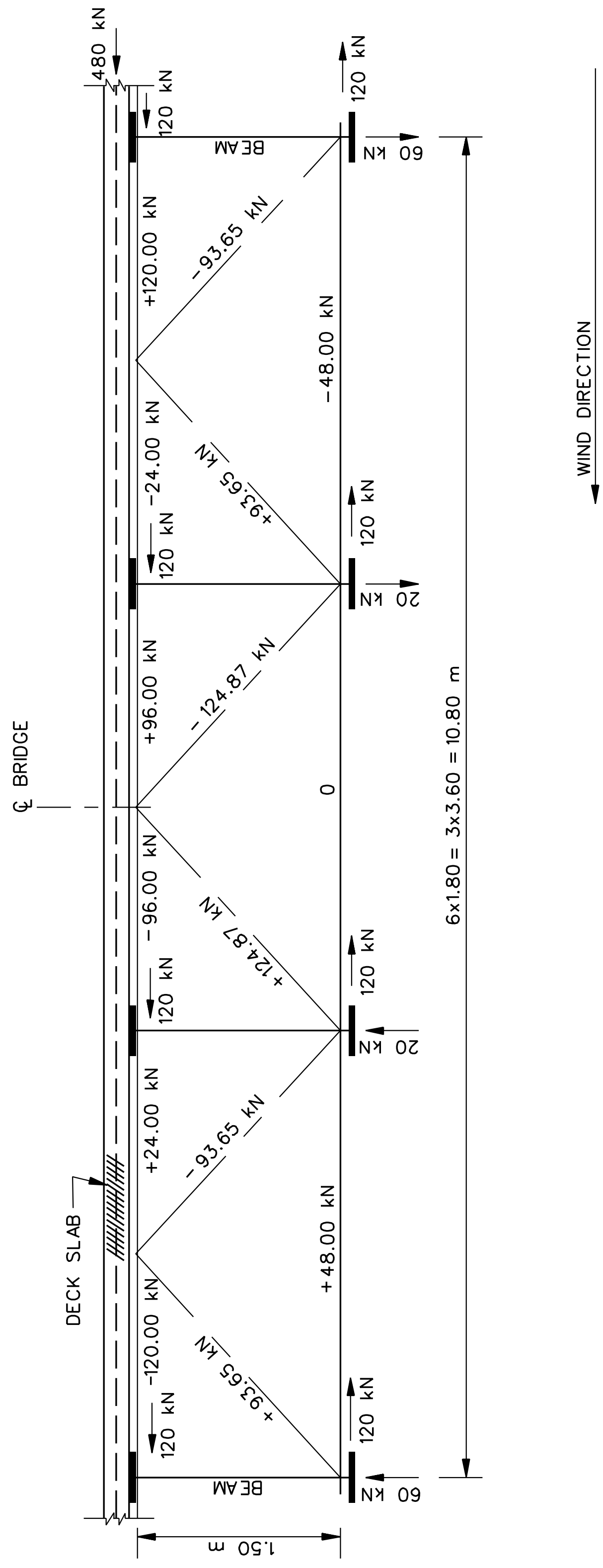
The analysis of force effects due to settlement is provided in Section 14.4.5.2.

14.4.5.2 Force Effects Due to Settlement

Reference: None

Settlement is a downward (positive) movement of a pier or abutment caused by slip, consolidation or failure of the supporting soil. On rare occasions, an upward (negative) displacement may occur. This analysis may be necessary when problems occur on bridges in-service or under construction. They are not needed for routine design. The following discussion assumes uniform settlement trans-versely for each foundation. Non-uniform settlement or rotation of a foundation adds an extra layer of complexity to the analysis but can be treated in much the same manner by quantifying individual girder seat settlements.

The method of analysis calculates force effects in the superstructure due to settlement of the substructure. Pier displacements "Δ_i" are relative values, normalized with respect to the movements of the outside substructure units, as illustrated in Figure 14.4C. The normalization process in this instance consists of constructing a mathematical "string-line" between the extreme ends of the bridge and calculating the deviation of the interior supports from this "string-line." The actual and normalized settlements are presented in the following example:



DIAPHRAGM AT BEARINGS

Figure 14.4B

	Actual	Normalized
Pier 1	0	0
Pier 2	+7.0	+6.2
Pier 3	0	-2.6
Pier 4	+4.0	0

The following equation is a useful tool for determining the moments caused in a continuous structure due to settlement of one or more supports. It is based on the equation for deflection of a simply supported beam of constant inertia, and the equation is found (or its variation) in many engineering textbooks:

$$d_{ij} = \frac{Pc^3}{6EI}\left[1-\alpha^2-\beta^2\right]$$

where:

d_{ij} = deflection at point ‘i’ due to a load at point ‘j’

P = concentrated load

c = total length of beam

EI = rigidity of beam

α = x/c, where ‘x’ is the distance of point ‘i’ from the end support

β = b/c where ‘b’ is the distance of point ‘j’ from the end support

For further discussion, simplify the above to:

$$k_{ij} = \alpha\beta\left[1-\alpha^2-\beta^2\right]$$

k_{ij} values are calculated as follows:

1. α = 0.20 β = 0.80
   k_{22} = $(0.20)(0.80)\,[1 - 0.20^2 - 0.80^2]$
   = 0.0512

2. α = 0.20 β = 0.35
   k_{23} = $(0.20)(0.35)\,[1 - 0.20^2 - 0.35^2]$
   = 0.0586

3. α = 0.35 β = 0.20
   k_{32} = $(0.20)(0.35)\,[1 - 0.20^2 - 0.35^2]$
   = 0.0586

4. α = 0.35 β = 0.65

   k_{33} = $(0.35)(0.65)\,[1 - 0.35^2 - 0.65^2]$
   = 0.1035

Because:

$$k_{22}P_2 + k_{23}P_3 = \frac{6EI}{C^3}\Delta_2$$

and

$$k_{32}P_2 + k_{33}P_3 = \frac{6EI}{C^3}\Delta_3$$

one can insert the variables and solve for reactions, then moments:

$$+0.0512\,P_2 + 0.0586\,P_3 = +6.2\,\frac{6EI}{C^3}$$

$$+0.0586\,P_2 + 0.1035\,P_3 = -2.6\,\frac{6EI}{C^3}$$

from which: $P_2 = +426.2\,\dfrac{6EI}{C^3}$

$P_3 = -266.4\,\dfrac{6EI}{C^3}$

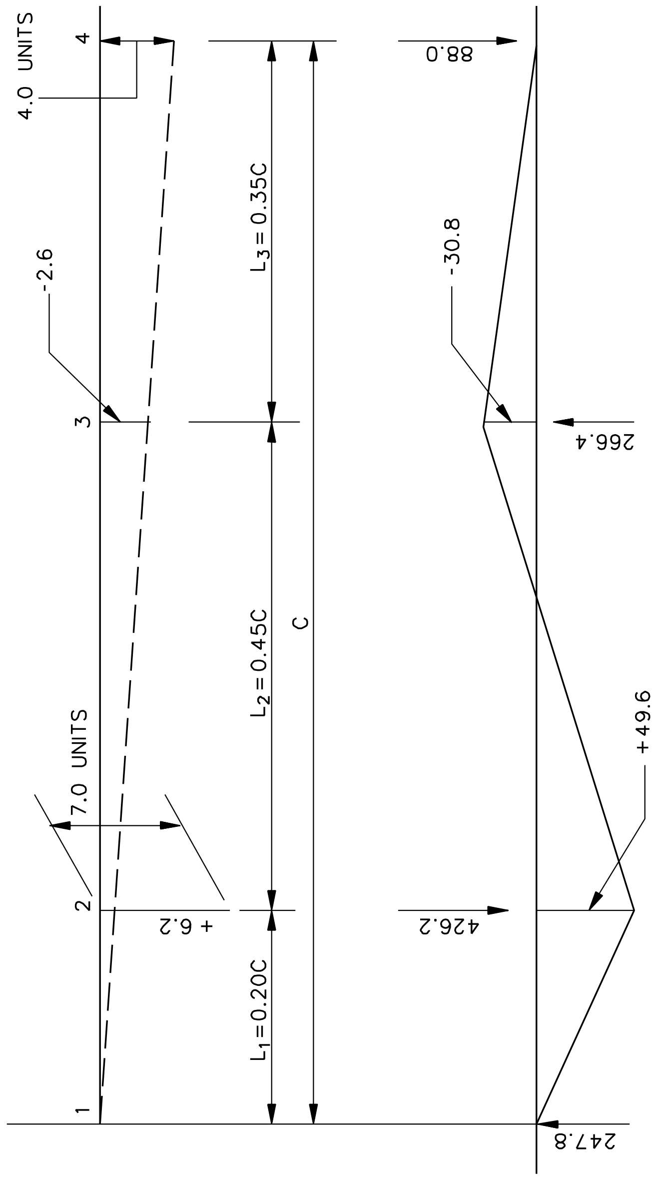
then: $P_1 = -247.8\,\dfrac{6EI}{C^3}$

$P_4 = +88.0\,\dfrac{6EI}{C^3}$

$M_2 = +49.6\,\dfrac{6EI}{C^2}$

$M_3 = -30.8\,\dfrac{6EI}{C^2}$

Forces and moments are indicated in Figure 14.4C.



FORCE EFFECTS DUE TO SETTLEMENT

Figure 14.4C