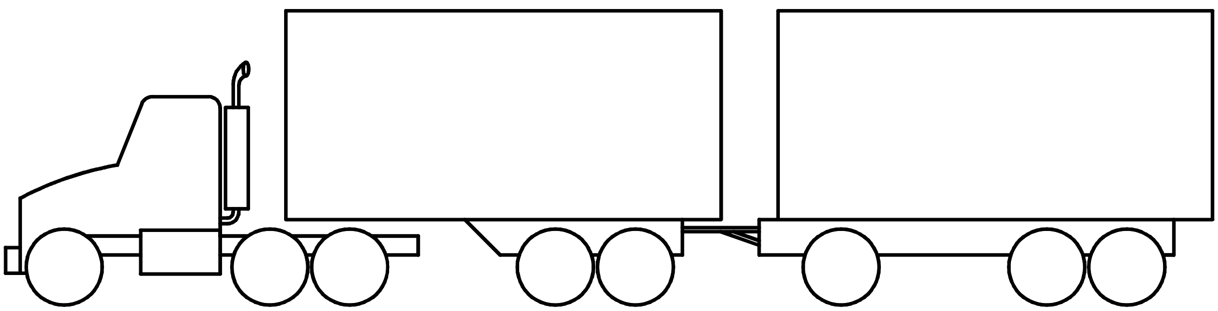
BRIDGE ANALYSIS GUIDE

2005 Edition

with

2009 Interim Update

Part 1



Michigan Department of Transportation

MICHIGAN DEPARTMENT OF TRANSPORTATION

CONSTRUCTION AND TECHNOLOGY SUPPORT AREA

MICHIGAN DEPARTMENT OF TRANSPORTATION
BRIDGE ANALYSIS GUIDE

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GENERAL ANALYSIS PROCEDURES

April 3, 2009

LOAD FACTOR RATING AND LOAD AND RESISTANCE FACTOR RATING

Load Rating Methods

There are three methods for performing load ratings. These methods are Allowable Stress Rating (ASR), Load Factor Rating (LFR) and Load and Resistance Factor Rating (LRFR). ASR is considered to be an obsolete code. While certain existing ratings are acceptable to remain in ASR, this method is only used for new Federal Ratings of policy exceptions such as timber and masonry bridges. LFR is being phased out as the preferred Federal Rating method. LRFR is the preferred Federal Rating method, and will be required on all bridges designed by Load and Resistance Factor Design (LRFD) after October 1st, 2010. Please refer to the Federal Highway Administration (FHWA) Bridge Technology website for further details on this policy (http://www.fhwa.dot.gov/bridge/nbis/103006.cfm)[1]. NBI Item 70, Bridge Posting, and the Michigan Operating Rating may be computed by LRFR, LFR or ASR. It is preferred that LFR is used for structures designed by Allowable Stress Design (ASD) or Load Factor Design (LFD) and LRFR is used for structures designed by LRFD. ASR may be used for timber and masonry.

Design Live Loads

Design live loads are used during the design of a new bridge, and reconstruction or rehabilitation designs. Design live loads are not legal loads. Generally speaking, design axle loads are more severe than legal axle loads and help to provide reasonable factors of safety for slab designs.

The American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges[3] specifies the HS-20 Live Load as the design live load for bridges designed under Allowable Stress Design (ASD) and Load Factor Design (LFD). Please refer to these specifications for details of this design load. HS-20 is used in Load Rating when calculating the Federal Inventory and Operating Rating for bridges analyzed by Allowable Stress Rating (ASR) or Load Factor Rating (LFR). In 1978, the HS-20 load in Michigan was increased by 25% and named HS-25. HS-25 was used for certain routes in Michigan to account for the stress caused by the heaviest legal loads. HS-25 is a design loading only, and is not used in Load Rating.

The HL-93 Live Load is the design live load for bridges designed under Load and Resistance Factor Design (LRFD). Please refer to the latest edition of the AASHTO LRFD Bridge Design Specifications[4] for details of this design load. HL-93 is used in Load Rating when calculating the Federal Inventory and Operating Rating for bridges analyzed by Load and Resistance Factor Rating (LRFR). In 2008, the HL-93 loading configuration was modified slightly, increased by 20% and renamed HL-93-mod. HL-93-mod is used for certain routes in Michigan to account for the stress caused by the heaviest legal and permit loads. HL-93-mod is a design loading only, and is not used in Load Rating.

State Regulations on Legal Loads

The extension of grandfather rights has allowed the states to continue operation of vehicles on state and interstate highways in excess of the limits mandated by federal regulations. These rights allowed individual states continued control of size and weight limits. As a result, each state has a different weight limit “package” consisting of different mixes of these combinations.

Michigan Regulations on Legal Loads

The three levels of Michigan Legal loads are called Normal, Designated and Special Designated, and are described in detail in Chapter 2 of the BAG. **It is the responsibility of the engineer to determine whether Normal, Designated or Special Designated loadings are appropriate for the specific agency/roadway under consideration**. As a majority of roadways in Michigan are Designated, only that loading is listed in this interim update in order to simplify the information contained and to avoid confusion. **Designated loading is not the most conservative loading and the assumption to use Designated loading should not be made on the presence of the loading in the Condensed Guide.**

Figure 2.1 in the Bridge Analysis Guide illustrates common legal vehicles used on Michigan roads (truck numbers 1-28). All of the legal vehicles are used to determine the Michigan Operating Rating and Load Posting Values.

Legal Loads in Other States and Provinces

The engineer should take into account the legal loads in neighboring states and provinces for border bridges. Chapter 3 of the BAG includes information of bordering states and a brief summary of the influence of North America Free Trade Agreement (NAFTA) requirements on bridges.

Load Factor Rating (LFR)

There are four categories of bridge rating for **Load Factor Rating (LFR)**. These four categories use three different groups of live loads.

<u>Federal Inventory Rating</u>

1. HS20 truck or lane load
2. In general, the truck load controls for shorter span lengths and lane load controls for longer lengths
3. For continuous structures, lane loadings may be continuous or discontinuous
4. As many lanes may be loaded as is required to produce the maximum desired effect
5. This rating is performed at the Inventory level

Federal Operating Rating

1. HS20 truck or lane load
2. In general, the truck load controls for shorter span lengths and lane load controls for longer lengths
3. For continuous structures, lane loadings may be continuous or discontinuous
4. As many lanes may be loaded as is required to produce the maximum desired effect
5. This rating is performed at the Operating level

Michigan Operating Rating (Legal or Posting Load Rating)

1. The controlling legal vehicle of the 28 different legal loads. Different vehicles may control different load effects (such as shear or moment). The truck that is recorded should be the truck that produces the lowest load factor for all limit states.
2. As many lanes may be loaded as is required to produce the maximum desired effect
3. Only one standard truck per lane is allowed on a span for spans <200-ft
4. A train of trucks must be applied for spans >200-ft (Chapter 5 of the BAG). A research project is currently in progress to find the appropriate loading configurations for spans between 200-ft and 400-ft and to develop site-specific analysis criteria for spans greater than 400-ft (10-3-2008).
5. The analyst must determine if Normal, Designated, or Special Designated loading applies
6. See Chapter 2 of the BAG for illustrations of the Legal Load vehicle configurations
7. See Chapter 10 of the BAG for tables for all maximum moments and shears for the Legal Load configurations, for simple span lengths between 5-ft and 300-ft
8. If any of the rating factors are below 1, then the lowest tonnage of all vehicles below 1 is the load limit for that Truck Type (1, 2 or 3 Unit)
9. If all vehicles in a particular category (1-unit, 2-unit, 3-unit) can be safely carried by a bridge, the Posting Load will be the largest legal load in that category
10. This rating is performed at the Operating Level

Permit Load Rating (see Chapter 8)

1. This capacity rating is used when a request has been made to transport a load that is not included in the Michigan legal loads
2. The exact load shall be analyzed and that one vehicle placed so as to produce the maximum effect
3. See Chapter 8 of the BAG for a chart illustrating the more common permit type vehicle configurations
4. See Chapter 10 of the BAG for tables for all maximum moments and shears for the more common permit type vehicle configurations, for simple span lengths between 5-ft and 300-ft
5. This rating is performed considering loading of only one lane for Load Factor and Allowable Stress Ratings
6. This rating is performed at the Operating Level

Load and Resistance Factor Rating (LRFR)

Similar to LFR, there are four categories of bridge rating for **Load and Resistance Factor Rating (LRFR)**. These four categories use three different groups of live loads.

Federal Inventory Rating (also called Design Load Rating at Inventory Level)

1. HL-93 loading
2. This load rating is sometimes referred to as a "screening" level for other states, however, **some Michigan Legal Loads exceed this design loading and therefore the Legal Load Rating should always be calculated**.
3. As many lanes may be loaded as is required to produce the maximum desired effect
4. This rating is performed at the Inventory level

Federal Operating Rating (also called Design Load Rating at Operating Level)

1. HL-93 loading
2. As many lanes may be loaded as is required to produce the maximum desired effect
3. This rating is performed at the Operating level

Michigan Operating Rating (Legal or Posting Load Rating)

1. The controlling legal vehicle of the 28 different legal loads. Different vehicles may control different load effects (such as shear or moment). The truck that is recorded should be the truck that produces the lowest load factor for all limit states.
2. The Live Load Factor, γ_L, to be used for the Strength I and II Limit States varies based on the Average Daily Truck Traffic (ADTT) of the structure and the weight of the truck being analyzed. See MDOT Research Report R-1511[5] for more information on the variable load factor. Tables 4a-1 through 4a-3 summarize the Live Load Factors for the Strength I and II Limit States. The Load Factor may be interpolated for a specific ADTT.
3. The Live Load Factor to be used for the Service II Limit State varies based on the weight of the truck being analyzed. Trucks with a Gross Vehicle Weight (GVW) less than 100-kip use a Load Factor of 1.3. Trucks with a GVW greater than or equal to 100-kip use a Load Factor of 1.0 for Service II.
4. As many lanes may be loaded as is required to produce the maximum desired effect.
5. The loading configuration of Legal Loads varies for moments and shear at interior supports as well as for span lengths greater than 200-ft. Table 4a-7 summarizes the loading configurations required to analyze Legal Loads. Spans greater than 400-ft require site-specific analysis. A research project is currently in progress to find the appropriate loading configurations for spans between 200-ft and 400-ft and to develop site-specific analysis criteria for spans greater than 400-ft (10-3-2008).
6. The analyst must determine if Normal, Designated, or Special Designated loading applies.
7. If posting is required, the lightest Posting Loads for each category (1 unit, 2

unit, and 3 unit) must be calculated

8. If all vehicles in a particular category (1-unit, 2-unit, 3-unit) can be safely carried by a bridge, the Posting Load will be the largest legal load in that category

<u>Permit Load Rating</u>

1. This capacity rating is used when a request has been made to transport a load that is not included in the Michigan legal loads
2. There are two levels of Permits identified in LRFR. See Table 6A.4.5.4.2a-1 of the AASHTO Manual for Bridge Evaluation[6] (MBE) for more information. Routine Permits are annual or unlimited permits that are allowed to mix with traffic. Special or Limited Crossings are limited to less than 100 crossings and may or may not be escorted to prevent other vehicles on the structure.
3. Routine Permits should use Strength Limit State Live Load factors, γ_L, as identified in MDOT Research Report R-1511 and as given in Tables 4a-4 through 4a-6, based upon ADTT and GVW. The load factor may be interpolated for a specific ADTT. These permits are based on as many lanes loaded as would produce the maximum effect.
4. Special or Limited Crossing Permits may use the Strength Limit State Live Load factors given in Table 6A.4.5.4.2a-1 of the MBE. These permits are based on single lane loading.
5. The Live Load Factor to be used for the Service II Limit State varies based on the weight of the truck being analyzed. Trucks with a Gross Vehicle Weight (GVW) less than 100-kip use a Load Factor of 1.3. Trucks with a GVW greater than or equal to 100-kip use a Load Factor of 1.0 for Service II Limit State.
6. See Chapter 8 of the BAG for a chart illustrating the more common permit type vehicle configurations
7. See Chapter 10 of the BAG for tables for all maximum moments and shears for the more common permit type vehicle configurations, for simple span lengths between 5-ft and 300-ft
8. The loading configuration of Legal Loads varies for moments and shear at interior supports as well as for span lengths greater than 200-ft. Table A-9 summarizes the loading configurations required to analyze Permit Loads. Spans greater than 400-ft require site-specific analysis. A research project is currently in progress to find the appropriate loading configurations for spans between 200-ft and 400-ft and to develop site-specific analysis criteria for spans greater than 400-ft (10-3-2008).

Michigan Legal Vehicle Load Factors for Strength Limit States, 5000 ADTT						
	Normal Loading		Designated Loading		Special Designated Loading	
Truck	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}
1	33.4	1.80	33.4	1.80	39.0	1.80
2	41.4	1.80	47.4	1.80	45.4	1.80
3	54.4	1.80	54.4	1.80	54.4	1.80
4	67.4	1.80	67.4	1.80	67.4	1.80
5	78.0	1.80	84.0	1.75	84.0	1.75
6	95.4	1.61	101.4	1.54	101.4	1.54
7	113.4	1.44	119.4	1.39	119.4	1.39
8	85.4	1.73	91.4	1.65	91.4	1.65
9	51.4	1.80	51.4	1.80	49.5	1.80
10	59.4	1.80	65.4	1.80	56.4	1.80
11	77.4	1.80	83.4	1.76	67.1	1.80
12	111.4	1.45	117.4	1.41	117.4	1.41
13	119.4	1.39	125.4	1.35	125.4	1.35
14	132.4	1.31	132.4	1.31	132.4	1.31
15	137.4	1.28	143.3	1.25	143.3	1.25
16	132.4	1.31	138.4	1.28	138.4	1.28
17	145.4	1.24	151.4	1.21	151.4	1.21
18	148.0	1.23	154.0	1.20	154.0	1.20
19	111.4	1.45	117.4	1.41	117.4	1.41
20	87.4	1.71	87.4	1.71	87.4	1.71
21	145.4	1.24	151.4	1.21	151.4	1.21
22	155.4	1.20	161.4	1.17	161.4	1.17
23	148.0	1.23	154.0	1.20	154.0	1.20
24	116.0	1.42	122.0	1.37	122.0	1.37
25	158.0	1.18	164.0	1.16	164.0	1.16
26	50.0	1.80	50.0	1.80	50.0	1.80
27	72.0	1.80	72.0	1.80	72.0	1.80
28	80.0	1.80	80.0	1.80	80.0	1.80

Table 4a-1

Michigan Legal Vehicle Load Factors for Strength Limit States, 1000 ADTT						
	Normal Loading		Designated Loading		Special Designated Loading	
Truck	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}
1	33.4	1.65	33.4	1.65	39.0	1.65
2	41.4	1.65	47.4	1.65	45.4	1.65
3	54.4	1.65	54.4	1.65	54.4	1.65
4	67.4	1.65	67.4	1.65	67.4	1.65
5	78.0	1.65	84.0	1.65	84.0	1.65
6	95.4	1.57	101.4	1.51	101.4	1.51
7	113.4	1.40	119.4	1.36	119.4	1.36
8	85.4	1.65	91.4	1.61	91.4	1.61
9	51.4	1.65	51.4	1.65	49.5	1.65
10	59.4	1.65	65.4	1.65	56.4	1.65
11	77.4	1.65	83.4	1.65	67.1	1.65
12	111.4	1.42	117.4	1.37	117.4	1.37
13	119.4	1.36	125.4	1.32	125.4	1.32
14	132.4	1.28	132.4	1.28	132.4	1.28
15	137.4	1.25	143.3	1.22	143.3	1.22
16	132.4	1.28	138.4	1.25	138.4	1.25
17	145.4	1.21	151.4	1.19	151.4	1.19
18	148.0	1.20	154.0	1.18	154.0	1.18
19	111.4	1.42	117.4	1.37	117.4	1.37
20	87.4	1.65	87.4	1.65	87.4	1.65
21	145.4	1.21	151.4	1.19	151.4	1.19
22	155.4	1.17	161.4	1.15	161.4	1.15
23	148.0	1.20	154.0	1.18	154.0	1.18
24	116.0	1.38	122.0	1.34	122.0	1.34
25	158.0	1.16	164.0	1.14	164.0	1.14
26	50.0	1.65	50.0	1.65	50.0	1.65
27	72.0	1.65	72.0	1.65	72.0	1.65
28	80.0	1.65	80.0	1.65	80.0	1.65

Table 4a-2

Michigan Legal Vehicle Load Factors for Strength Limit States, 100 ADTT						
	Normal Loading		Designated Loading		Special Designated Loading	
Truck	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}
1	33.4	1.40	33.4	1.40	39.0	1.40
2	41.4	1.40	47.4	1.40	45.4	1.40
3	54.4	1.40	54.4	1.40	54.4	1.40
4	67.4	1.40	67.4	1.40	67.4	1.40
5	78.0	1.40	84.0	1.40	84.0	1.40
6	95.4	1.40	101.4	1.40	101.4	1.40
7	113.4	1.35	119.4	1.31	119.4	1.31
8	85.4	1.40	91.4	1.40	91.4	1.40
9	51.4	1.40	51.4	1.40	49.5	1.40
10	59.4	1.40	65.4	1.40	56.4	1.40
11	77.4	1.40	83.4	1.40	67.1	1.40
12	111.4	1.36	117.4	1.32	117.4	1.32
13	119.4	1.31	125.4	1.27	125.4	1.27
14	132.4	1.23	132.4	1.23	132.4	1.23
15	137.4	1.21	143.3	1.18	143.3	1.18
16	132.4	1.23	138.4	1.20	138.4	1.20
17	145.4	1.17	151.4	1.14	151.4	1.14
18	148.0	1.16	154.0	1.13	154.0	1.13
19	111.4	1.36	117.4	1.32	117.4	1.32
20	87.4	1.40	87.4	1.40	87.4	1.40
21	145.4	1.17	151.4	1.14	151.4	1.14
22	155.4	1.13	161.4	1.11	161.4	1.11
23	148.0	1.16	154.0	1.13	154.0	1.13
24	116.0	1.33	122.0	1.29	122.0	1.29
25	158.0	1.12	164.0	1.10	164.0	1.10
26	50.0	1.40	50.0	1.40	50.0	1.40
27	72.0	1.40	72.0	1.40	72.0	1.40
28	80.0	1.40	80.0	1.40	80.0	1.40

Table 4a-3

Overload Class Vehicle Load Factors for Strength Limit States, Annual Permits, 5000 ADTT						
	Class A		Class B		Class C	
Truck	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}
1	120.0	1.39	120.0	1.39	120.0	1.39
2	120.0	1.39	120.0	1.39	120.0	1.39
3	120.0	1.39	118.0	1.40	114.0	1.43
4	120.0	1.39	108.0	1.48	98.0	1.58
5	120.0	1.39	104.0	1.52	88.0	1.70
6	126.0	1.35	108.0	1.48	90.0	1.67
7	138.0	1.28	114.0	1.43	93.0	1.64
8	149.6	1.22	127.6	1.34	105.6	1.50
9	158.4	1.18	129.6	1.33	105.6	1.50
10	177.0	1.12	146.4	1.24	122.0	1.37
11	180.0	1.11	159.0	1.18	138.0	1.28
12	190.6	1.10	160.2	1.18	134.4	1.30
13	195.0	1.10	168.8	1.14	147.4	1.23
14	211.2	1.10	179.2	1.11	153.6	1.20
15	238.0	1.10	204.0	1.10	170.0	1.14
16	244.4	1.10	203.6	1.10	173.0	1.13
17	272.6	1.10	232.4	1.10	182.8	1.10
18	283.4	1.10	241.6	1.10	200.0	1.10
19	277.2	1.10	234.4	1.10	200.8	1.10
20	264.0	1.10	225.8	1.10	191.4	1.10

Table 4a-4

Overload Class Vehicle Load Factors for Strength Limit States, Annual Permits, 1000 ADTT						
	Class A		Class B		Class C	
Truck	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}
1	120.0	1.36	120.0	1.36	120.0	1.36
2	120.0	1.36	120.0	1.36	120.0	1.36
3	120.0	1.36	118.0	1.37	114.0	1.40
4	120.0	1.36	108.0	1.45	98.0	1.54
5	120.0	1.36	104.0	1.48	88.0	1.65
6	126.0	1.32	108.0	1.45	90.0	1.63
7	138.0	1.25	114.0	1.40	93.0	1.59
8	149.6	1.19	127.6	1.31	105.6	1.47
9	158.4	1.16	129.6	1.30	105.6	1.47
10	177.0	1.10	146.4	1.21	122.0	1.34
11	180.0	1.10	159.0	1.16	138.0	1.25
12	190.6	1.10	160.2	1.15	134.4	1.27
13	195.0	1.10	168.8	1.12	147.4	1.20
14	211.2	1.10	179.2	1.10	153.6	1.18
15	238.0	1.10	204.0	1.10	170.0	1.12
16	244.4	1.10	203.6	1.10	173.0	1.11
17	272.6	1.10	232.4	1.10	182.8	1.10
18	283.4	1.10	241.6	1.10	200.0	1.10
19	277.2	1.10	234.4	1.10	200.8	1.10
20	264.0	1.10	225.8	1.10	191.4	1.10

Table 4a-5

Overload Class Vehicle Load Factors for Strength Limit States, Annual Permits, 100 ADTT						
	Class A		Class B		Class C	
Truck	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}	GVW (kips)	Load Factor, γ_{LL}
1	120.0	1.30	120.0	1.30	120.0	1.30
2	120.0	1.30	120.0	1.30	120.0	1.30
3	120.0	1.30	118.0	1.32	114.0	1.34
4	120.0	1.30	108.0	1.39	98.0	1.40
5	120.0	1.30	104.0	1.40	88.0	1.40
6	126.0	1.27	108.0	1.39	90.0	1.40
7	138.0	1.20	114.0	1.34	93.0	1.40
8	149.6	1.15	127.6	1.26	105.6	1.40
9	158.4	1.12	129.6	1.25	105.6	1.40
10	177.0	1.10	146.4	1.16	122.0	1.29
11	180.0	1.10	159.0	1.12	138.0	1.20
12	190.6	1.10	160.2	1.11	134.4	1.22
13	195.0	1.10	168.8	1.10	147.4	1.16
14	211.2	1.10	179.2	1.10	153.6	1.14
15	238.0	1.10	204.0	1.10	170.0	1.10
16	244.4	1.10	203.6	1.10	173.0	1.10
17	272.6	1.10	232.4	1.10	182.8	1.10
18	283.4	1.10	241.6	1.10	200.0	1.10
19	277.2	1.10	234.4	1.10	200.8	1.10
20	264.0	1.10	225.8	1.10	191.4	1.10

Table 4a-6

Span Length	Load Effect	Legal Trucks GVW ≤ 100-kips	Legal-Heavy Trucks GVW > 100-kips and Permit Trucks
L≤200-ft	Positive Moment and Reactions at Exterior Supports	Truck + Impact	Truck + Impact
	Negative Moment and Reactions at Interior Supports	0.75*(Two Trucks Spaced 30-ft Apart + Impact) + 0.2-klf	(Truck + Impact) + 0.2-klf
200-ft<L≤400-ft	Positive Moment and Reactions at Exterior Supports	0.75*(Truck + Impact) + 0.2-klf	(Truck + Impact) + 0.2-klf
	Negative Moment and Reactions at Interior Supports	0.75*(Two Trucks Spaced 30-ft Apart + Impact) + 0.2-klf	(Truck + Impact) + 0.2-klf

Table 4a-7
LRFR Loading Configurations for Legal, Legal-Heavy and Permit Loads

References

1. Lwin, M. (2006). *INFORMATION: Bridge Load Ratings for the National Bridge Inventory*. Federal Highway Administration. October 30, 2006. http://www.fhwa.dot.gov/bridge/nbis/103006.cfm.

2. AASHTO (2002). *Standard Specifications for Highway Bridge Design, 17th Edition*. Washington, DC.

3. AASHTO (2008). *LRFD Bridge Design Specifications, 4th Edition*. Washington, DC.

4. AASHTO (1994). *Manual for Condition Evaluation of Bridges, 4th Edition*. Washington, DC.

5. Curtis, R., and Till, R. (2008). *Recommendations for MI Specific Load and Resistance Factor Design Loads and Load and Resistance Factor Rating Procedures*. R-1511. Michigan Department of Transportation. http://www.michigan.gov/documents/mdot/MDOT_Research_Report_R1511_233374_7.pdf.

6. AASHTO (2008). *The Manual for Bridge Evaluation*. American Association of State Highway and Transportation Officials. Washington, D.C.

Chapter 6

LIVE LOAD DISTRIBUTION

April 3, 2009

LIVE LOAD DISTRIBUTION

General

The intent of this chapter is to provide the user with guidance in the selection and application of live load distribution factors for the purpose of determining design shears and moments in stringers and beams providing support for bridge deck systems. This chapter has been developed on the premise that the user has an understanding of the AASHTO bridge code(s) and that these references are readily available.

These recommendations utilize current AASHTO guidelines supplemented by existing MDOT- approved practice for structure types not specifically covered in the bridge code. Distribution factors provided within this document are in accordance with the provisions of Chapter 6 of the AASHTO *Manual for Condition Evaluation of Bridges, 1994 Second Edition* with current interim specifications and the AASHTO *Standard Specifications for Highway Bridges, Sixteenth Edition, 1996* with current interim specifications, References 1 and 2, respectively.

Load Factor rating procedures are implemented to obtain ratings consistent with established MDOT practice and to comply with FHWA National Bridge Inventory (NBI) requirements. Load and Resistance Factor Rating procedures are still being studied and developed by AASHTO at this time, and should not be implemented beyond the scope as prescribed below. For other information on this topic, see Chapters 4 and 5 of Reference 3, References 4, and Reference 10.

The analyst may be confronted with situations where live load distribution factors derived in accordance with Article 3.23 of the AASHTO LF Code will lead to an analysis that shows that the supporting members can not safely carry all legal loads at the operating level (rating factor < 1.0). In that case, the analyst, and/or agency, may choose to implement one or a combination of other more refined methods to obtain load distribution factors that more accurately reflect the true behavior of the structure. These alternate methods are listed and described below.

Although the primary method illustrated in this Guide is Load Factor, for the circumstances mentioned here, live load distribution by LRFD is also discussed in this chapter. Recommendations to utilize LRFD derived distribution factors in conjunction with LF rating procedures are supported by research conducted by the University of Michigan Department of Civil Engineering for the Michigan Department of Transportation.

Applicability

Application of the information and methods in this chapter is limited to structures for which load distribution takes place mainly through flexure and torsion in the longitudinal and transverse directions, with deflections due to shear being negligible. Bridge types that satisfy this criteria are defined as *shallow superstructure* bridges, and include the solid slab, voided slab, and slab-on-girder bridges. In contrast, multicell box girder bridges exhibit significant shear deformation, which is accompanied by bending of the top and bottom flanges about their own centerlines. For this reason, if similar orthotropic plate theory is to be implemented in determining structural behavior, a provision must be included to account for shear deformation.

The simplified method of applying a factor to determine the transverse distribution of live load, known as the D-Type Method, was developed by idealizing bridges as orthotropic plates. To satisfactorily idealize a bridge as an orthotropic plate, it must reasonably satisfy the following conditions:

1. The width is constant.
2. Line support conditions exist.
3. Skew angle does not exceed 20 degrees.
4. Curvature is negligible; $L^2/bR < 1.0$

   where: L = Bridge Length

   R = radius of curvature measured to the bridge centerline, and
   b = (½)(Deck Width).
5. A solid or voided slab bridge is of uniform depth across the section.
6. Slab-on-girder bridges are made up of at least four parallel prismatic beams of similar stiffness.
7. The deck overhang for slab-on-girder bridges does not exceed 60 percent of the spacing between the girders.

For more information on this topic see Reference 24.

Limitations/Exclusions

The AASHTO LF code prescribed methods used to determine the transverse distribution of wheel loads are empirically derived and have been developed to *conservatively* encompass a broad range of basic superstructure types and geometry. Analysis of structurally and/or geometrically complex bridges is beyond the scope of this report. For examples of complex bridge types see Ref 1, section 6.1.6. These structures must be evaluated on a case-by-case basis using advanced modeling techniques or other owner approved methods. As noted above, should it become necessary to predict structural capacity with greater precision to evaluate marginal structures, those with a rating factor slightly less than 1.0, the AASHTO LRFD Bridge Design Specifications provide a more refined approach to accomplish this objective. Guidelines and limitations for the implementation of the LRFD specifications are presented in the final section of this chapter. Other, still more highly refined methods, which include three-dimensional modeling or field testing may be utilized to more accurately determine capacity for structures deemed marginal.

Alternate Methods for Determining Live Load Distribution

- **LRFD Live Load Distribution Factors**. MDOT has sponsored load testing of several bridges on the trunkline system, with one objective being to obtain live load distribution factors that more accurately reflect how loads are distributed in the transverse direction. Full reports of these research efforts are contained in References 27, 28 and 29. The structures examined were all composite slab-on-steel beam bridges with skew angles not exceeding 30 degrees. Beam spacing for these bridges ranged from 4'-4" to 9'-4", and span lengths ranged from 32 feet to 140 feet.

  One of the conclusions and recommendations provided by this series of reports is that AASHTO LRFD live load distribution methods may be used in conjunction with LFD analytic techniques for rating those bridges which are similar in structure type and fall within the skew, span length, and beam spacing limits considered in the studies. MDOT supports, and has adopted the above live load distribution recommendation.

- **Refined Analytical Methods**. Other analytical methods which may be implemented to obtain results that more accurately reflect the true bridge capacity include Finite Element Analysis and Grillage Analysis. Great care must be exercised when creating these models to ensure that the boundary conditions and model geometry are correct and that loads are place at positions that produce the maximum response in the components being investigated. Guidelines for detailed analysis of bridge decks and sample problems to illustrate their application are given in Appendix H of NCHRP Project 12-26 final report. Reference 17.

- **Load Testing**. In some cases, it may prove to be more economical to load test a particular bridge (or group of bridges) rather than to post the bridge for restricted loads, or to reconstruct the bridge in question. The analyst should confer with the owning agency to determine if load testing is economically appropriate for marginal structures. References 26 and 31 may prove useful in understanding the load testing process.

Distribution of Loads - Load Factor

<u>General</u>

The fraction of vehicle load effect transferred to a single member should be selected in accordance with the current AASHTO LFD Code (Ref 2, Article 3.23). These values represent a possible combination of diverse circumstances. The option exists to substitute field measured values, analytically calculated values or those determined from advanced structural analysis methods utilizing the properties of the existing span(s). During the implementation of any one of these methods, the position of loading shall be investigated to provide the condition causing the maximum response in the components being evaluated.

Impact, I, shall be added to the live load used for rating in accordance with the current AASHTO LFD Code. Specification impact may be reduced when conditions of alignment, enforced speed posting, and similar situations require a vehicle to substantially reduce speed in crossing the structure.

<u>Distribution of Live Load - Inventory Loads</u>

Inventory loads shall be distributed in accordance with Tables 6.1 and 6.2 below.

<u>Distribution of Live Load - Operating Loads</u>

In general, operating loads shall be distributed in accordance with Tables 6.1 and 6.2 below. In that circumstance where spans greater than 200' are to be rated, and an equivalent distributed load occupies one lane and a vehicle load occupies one or more adjacent lanes, standard live load distribution factors for interior beams are not applicable. For this case live load moment must be calculated by either distributing the equivalent distributed load and the adjacent vehicle loads using the lever rule or by more detailed analytical methods.

<u>Distribution of Live Load - Permit Loads</u>

Permit loads shall be distributed in accordance with Tables 6.1 and 6.2 below. The use of the live load distribution factor for one loaded lane (clear deck width less than 18') is appropriate.

<u>Live Load Distribution Factors - General</u>

The information provided in Tables 6.1 and 6.2 that follow has been derived from Article 3.23 of the AASHTO LF code and supplemented by the 1983 MDOT Bridge Analysis Guide for structure types common to the state of Michigan that are not specifically addressed by AASHTO. *The live load distribution factors provided herein are in terms of number of lanes per girder as opposed to the wheel lines per girder convention used in the AASHTO LF code; i.e. (S/5.5 wheel lines per girder) x (1 lane/2 wheel lines) = S/11.0 lanes per girder.*

The user is cautioned to refer to AASHTO Articles 3.11 and 3.12 for guidelines defining the application of live load and reductions in load intensity. Regarding the latter, Article 3.12.2 of the 1999 interim specifications states that reductions in load intensity shall not be applied when the distribution factors of Table 6.1 are used to determine moments in longitudinal beams.

For the purpose of this document, and in accordance with the AASHTO code, the following definitions apply:

Concrete Box Girder:

a. Precast solid, voided or cellular adjacent concrete boxes with shear keys and a cast-in-place concrete overlay.

b. Precast solid, voided or cellular adjacent concrete boxes with shear keys, no concrete overlay, and with or without transverse post-tensioning.

Concrete Spread Box Beams: Closed precast concrete boxes positioned with a space between interior webs supporting a cast-in-place concrete slab.

Floor Beam: Transverse beam spanning between main longitudinal members.

Longitudinal: In the direction of traffic flow.

Longitudinal Beam: Primary load carrying member supporting the floor system.

Stringer: Longitudinal beam spanning between floor beams.

Distribution of Live Load to Interior Stringers and Beams

In calculating bending moments in longitudinal stringers or beams, no longitudinal distribution of axle or truck loads shall be assumed, i.e. axle loads are considered to be “point” loads. The lateral distribution of load used to determine live load bending moment shall be determined by applying to the stringer or beam the fraction of truck or lane loads determined in Table 6.1 that follows.

The distribution values contained in Table 6.1 pertain to inventory, operating and permit ratings.

DECK TYPE	BEAM TYPE	CLEAR DECK WIDTH LESS THAN 18'	CLEAR DECK WIDTH 18' AND GREATER
Timber[a] plank[b]	All beam types	S/8	S/7.5
Timber[a] nail laminated[c] 4" thick or multiple layer[d] floors over 5" thick		S/9	S/8
Timber[a] nail laminated[c] 6" or more thick		S/10 If S>5' use footnote f	S/8.5 If S>6.5' use footnote f
Glued laminated[e] panel 4" thick	Glued laminated stringer	S/9	S/8
Glued laminated[e] panel 6" or more thick		S/12 If S>6' use footnote f	S/10 If S.7.5' use footnote f
Glued laminated[e] panel 4" thick	Steel stringer	S/9	S/8
Glued laminated[e] panel 6" or more thick		S/10.5 If S>5.5' use footnote f	S/9 If S>7' use footnote f
Concrete	Steel I-Beam stringers[g] and prestressed concrete girders	S/14 If S>10' use footnote f	S/11 if S>14' use footnote f
	Concrete T-Beams	S/13 If S>6' use footnote f	S/12 If S>10' use footnote f
	Timber stringers	S/12 If S>6' use footnote f	S/10 If S>10' use footnote f
	Concrete box girders[h]	S/16 If S>12' use footnote f	S/14 If S>16' use footnote f
	Steel box girder	See AASHTO Std. Spec. Section 10.39.2	
	Prestressed concrete spread box beams	See AASHTO Std. Spec. Section 3.28	
Steel grid less than 4" thick	All beam types	S/9	S/8
Steel grid 4" or more thick		S/12 If S>6' use footnote f	S/10 if S>10.5' use footnote f
Steel bridge corrugated plank[i] (2" minimum depth)	All beam types	S/11	S/9
Concrete Jack Arch or Encased Beams[j] with or without tie rods	Steel Stringer	S/10	S/8

TABLE 6.1
Distribution of Lane Loads in Interior Longitudinal Beams

<u>Table 6.1 Notes:</u>

S = Average Girder Spacing in Feet

a. Timber dimensions shown are nominal thickness.

b. Plank floors consist of pieces of lumber laid edge to edge with the wide faces bearing on the supports.

c. Nail laminated floors consist of pieces of lumber laid face to face with the narrow edges bearing on the supports, each piece being nailed top the preceding piece.

d. Multiple layer floors consist of two or more layers of planks, each layer being laid at an edge angle to the other.

e. Glued laminated panel floors consist of vertically glued laminated members with the narrow edges of the laminations bearing on the supports.

f. In this case the load on each stringer shall be the reaction of the live load assuming the flooring between the stringers to act as a simple beam.

g. “Design of I-Beam Bridges” by N. M. Newmark–Proceedings, ASCE, March 1948.

h. The sidewalk live load shall be omitted for interior and exterior box girders designed in accordance with the wheel load distribution indicated herein.

i. Distribution factors for Steel Bridge Corrugated Plank set forth above are based substantially on the following reference:Journal of Washington Academy of Sciences, Vol. 67, No. 2, 1977 “Wheel Load Distribution of Steel Bridge Plank,” by Conrad P. Heins, Professor of Civil Engineering, University of Maryland. These distribution factors were developed based on studies using 6"x2" steel corrugated plank. The factors should yield safe results for other corrugated configurations provided primary bending stiffness is the same or greater than the 6"x2" corrugated plank used in the studies.

j. Table 1 of the 1983 MDOT Bridge Analysis Guide. (Ref 8)

Distribution of Live Load to Exterior Stringers and Beams (Ref. 2, Article 3.23.2.3)

Steel - Timber - Concrete T-Beams

> The live load bending moment for outside stringers or beams shall be determined by applying to the stringer or beam the reaction of the wheel load obtained by applying the Lever Rule. Note that computations should be carried out in terms of number of wheels per girder and multiplied by a factor of 1/2 (1 lane/2 wheels) to obtain number of lanes per girder. The requirement of an exterior stringer having at least the carrying capacity equal to that of an interior stringer is an important design consideration, but is optional for load rating calculations.

Concrete Box Girders

The factor for the lane load distribution to the exterior girder shall be $W_e/14$, where W_e is the width of exterior girder that shall be taken as top slab width, measured from the midpoint between girders to the outside edge of the slab.

Distribution of Live Load to Transverse Floor Beams

In calculating bending moments in transverse floor beams, no transverse distribution of wheel load shall be assumed in a manner similar to the assumption that no longitudinal distribution of wheel load is considered in computing bending moments in longitudinal beams (Ref 2, Article 3.23.2.1).

If longitudinal stringers are omitted and the floor is supported directly on the floor beams, the beams shall be designed for loads determined in accordance with Table 6.2. The longitudinal distribution of wheel load used to determine live load bending moment shall be determined by applying to the beam the fraction of wheel load determined in Table 6.2. Refer to Chapter 9 of this guide for an example calculation. *Note the departure from the lane load convention used in this guide;* ***wheel, not lane, loads*** *are used in calculating moments in transverse floor beams.*

DECK TYPE	FRACTION OF WHEEL LOAD TO EACH BEAM
Plank[a,b]	S/4
Nail laminated[c] or glue laminated[e], 4" in thickness, or multiple layer[d] floors more than 5" thick.	S/4.5
Nail laminated[c] or glue laminated[e], 6" or more in thickness	S[f]/5
Concrete	S[f]/6
Steel grid(less than 4" thick)	S/4.5
Steel grid (4" or more thick)	S[f]/6
Steel bridge corrugated plank (2" minimum depth)	S/5.5

TABLE 6.2
Distribution of Wheel Loads in Transverse Floor Beams

Table 6.2 Notes:

S = Longitudinal Spacing of Floor Beams in Feet

a-e For footnotes a through e, see Table 6.1

f If S exceeds the denominator, the load on the beam shall be the reaction of the wheel loads assuming the flooring between beams to act as a simple beam.

Distribution of Loads - Load and Resistance Factor Design

Application

As noted above, application of the LRFD live load distribution methodology shall be limited to slab-on-steel beam bridges of the type and geometry considered in the University of Michigan studies. The analytical requirements for other structure types is beyond the scope of this document and must be addressed on a case-by-case basis using a more rigorous owner-approved approach.

Beam spacing shall be checked to ensure compliance with the ranges specified in Tables 4.6.2.2.2b-1 and 4.6.2.2.3a-1. Note that the range of beam spacings for structures considered in the University of Michigan studies falls within the range of applicability as specified in these articles.

The multiple presence factors, defined in Table 3.6.1.1.2-1, have been included in the approximate equations for distribution factors in Articles 4.6.2.2 and 4.6.2.3 for both single and multiple lanes loaded; these factors, m, do not need to be applied to distribution factors determined in accordance with the provisions of these articles. Where use of the lever rule is specified, the engineer must determine the number of vehicles and lanes and, therefore, must include the multiple presence (see Ref 3, Commentary Article C3.6.1.1.2).

In addition to the requirements defined by the University of Michigan study for structure type, skew, span length, and beam spacing, the distribution of live load, specified in Articles 4.6.2.2.2 and 4.6.2.2.3, may be used for beams that meet, at least, the following conditions:

- Deck width is constant;
- Unless otherwise specified, the number of beams is not less than four;
- Beams are parallel and have approximately the same stiffness;
- Unless otherwise specified, the roadway part of the overhang, d_e, does not exceed 3.0 ft;
- Curvature in plan is less than the limit specified in Article 4.6.1.2; and
- Cross-Section is consistent with one of the cross sections shown in Table 4.6.2.2.1-1

Procedure

Distribution factors determined by LRFD methodology are calculated as a function of superstructure section properties, material properties, and bridge longitudinal and transverse geometry. A general approach for the computation of these factors is outlined below. A numerical example, complete with code references, for a composite slab on steel I-beam bridge is provided in Chapter 9 of this guide.

1. Compile beam and superstructure data for both interior and exterior beams:
   a. Deck width.
   b. Deck thickness.
   c. Number of beams.
   d. Beam spacing.
   e. Cantilever length.
   f. Beam non-composite section properties.
   g. Beam modulus of elasticity.
   h. Deck modulus of elasticity.

2. Using the above data compute the following:
   a. Distance from exterior web of exterior beam to curbline, d_e.
   b. Longitudinal Stiffness Parameter, K_g.
   c. Distance between CG non-composite girder and CG deck, e_g.
   d. Beam/Deck Modular Ratio, $n = E_B/E_{D.}$

3. Using the parameters calculated in Item 2, compute:
   a. Interior beam shear and moment distribution factor for one lane loaded.
   b. Interior beam shear and moment distribution factor for multiple lanes loaded.
   c. Exterior beam shear and moment distribution factor for one lane loaded.
   d. Exterior beam shear and moment distribution factor for multiple lanes loaded.

4. Evaluate distribution factors for each of the cases investigated in Item 3 above to determine the governing (greatest) shear and moment distribution factors to apply to the interior and exterior beams respectively.

Supplemental Research

For more information about recent research related to live load distribution, the following references are suggested:

1. *Distribution of Wheel Loads on Highway Bridges, Phase III* (Ref 17)

   Development of simplified wheel load distribution criteria for five common bridge types namely, slab on girder, slab, box girder, spread box beam and multibox beam.

2. *Load Testing of Bridges. 1996* (Ref 26)

   Load tests of five bridges, three reinforced concrete T-beam and two slab-on-steel beam bridges, to develop an efficient proof load testing procedure. Proof load testing data are valuable in that they can be used to verify load carrying capacity.

3. *Development of a Guide for Evaluation of Existing Bridges. 1998* (Ref 27)

   Load tests of five steel beam bridges to determine safe load limits, response to dynamic loads and actual live load distribution factors.

4. *Development of a Guide for Evaluation of Existing Bridges Phase 2. 2000*
   (Ref 28) .

   A continuation of the 1998 test program, load tests were performed on six steel beam bridges to determine safe load limits, response to dynamic loads and actual live load distribution factors.

5. *Verification of Girder Distribution Factors for Steel Girder Bridges. 2001*
   (Ref 28)

   A continuation of the above testing programs, load tests were performed on six steel beam bridges with span lengths ranging up to 139' to determine actual live load distribution factors.

Chapter 7

POSTING PROCEDURES

April 3, 2009

POSTING PROCEDURES

When to Post a Bridge

As noted in Chapter 5, Analysis Vehicle Selection, all bridges will be examined for their ability to safely carry all legal load configurations at the Operating Rating level. If the load carrying capacity of a bridge is insufficient for all legal loads, the bridge must be posted to restrict vehicles that are too heavy from crossing the bridge. When the load carrying capacity is less than 3 tons, the bridge must be closed to traffic. If the Operating Rating indicates that the bridge can carry all legal loads, then posting is not required.

A bridge owner may also elect to post a bridge for lower bridge weights than those determined by calculations. Lower postings can extend the life of a bridge.

Load Posting Process

The load posting process includes the following steps:

- Analysis discovers that posting is necessary.
- The coding for NBI Item 41 is changed to “B” using the Michigan Bridge Inspection System (MBIS).
- A load posting order is signed by the responsible individual within the agency.
- Bridge is posted correctly for reduced loads. The posted capacity must be no more than the calculated capacity.
- The coding for NBI Item 41 is changed to “P” using the Michigan Bridge Inspection System (MBIS).
- Submit photo of posted bridge to MDOT.

The process required by the Michigan Department of Transportation for posting a bridge includes properly coding the Structure Inventory and Appraisal forms. When a load rating is performed and the bridge capacity is deficient for legal loads, item 41 of the SI&A form must be coded properly. Three of the nine possible codes for item 41 that are shown below are taken from the Michigan Structure Inventory and Appraisal Coding Guide (Ref 10) :

Code	Description
A	Open, no restriction
B	Open, posting recommended but not legally implemented (All signs not in place or do not show the correct information or are not in the correct location)
P	Posted for load (may include other restriction such as temporary bridges which are load posted).
K	Bridge closed.

If a particular bridge is currently not posted and a load rating shows the capacity to be insufficient for legal loads, item 41 should have its coding changed from an “A” to a “B.” Only after the bridge is posted, can the coding be changed from a “B” to a “P.”

It is imperative that corrective action be taken when the requirement for load posting is known. When a bridge requires posting (item 41 coded as “B”), a person or persons having authority over the bridge in question must give signed approval for the sign installation. After the approval, the posting sign can be ordered and then installed. See example #6 (Simple Span Composite Prestressed Concrete Box Beam) in Chapter 9 for a load rating that results in load posting.

In addition to load posting the bridge and updating NBI Item 41 with the appropriate coding via MBIS, a photo showing both the bridge and the load posting sign is to be submitted to the MDOT Bridge Operations Unit of the Construction and Technology Division.

Sign Configurations

The Michigan Manual of Uniform Traffic Control Devices-1994 Edition (MMUTCD), Part 2B Regulatory Signs, gives examples of typical signs used for posting bridges. As with any road sign, the information shown on the signs must be clear and concise so that the operator of a vehicle can understand the meaning quickly.

Figure 7.1 illustrates examples of various posting signs. Sign type R12-5 is the most common bridge load posting sign. Signs R12-1 and R12-2 may be useful in situations where severe load restrictions apply to a bridge or to a bridge component. Sign R12-4 can be used to combine the information contained on R12-1 and R12-2. In any case, careful analysis of the structure will determine the types of loading that control and will, therefore, dictate the information required on the posting signs.

After a load rating has been completed and it is determined that the bridge can not support legal axle loads, the bridge owner must order the fabrication of posting signs. The signs should be installed in advance of each end of the bridge. Advance sign locations and locations of signs near the bridge are both described in the MMUTCD.

Posting Routes With Multiple Posted Bridges

When multiple bridges on the same route require posting, it may be appropriate to post the entire section of road based on the bridge with the lowest capacity. Some typical scenarios are as follows:

- Two or more posted bridges are in immediate succession. If it is not physically possible to cross one bridge and not cross the other, then both bridges should be posted with the most restrictive posting. Advance warning signs should be placed so that overweight vehicles can take a detour route.
- Two posted bridges are separated by non-commercial driveways and/or intersections with roads with little commercial traffic. In this case, both bridges should be posted with the most restrictive posting. Advance warning signs should be placed so that overweight vehicles can take a detour route.
- Two posted bridges are separated by commercial/industrial facilities and/or intersections with roads with significant commercial traffic. In this case, the bridges should be posted individually to allow trucks to use the bridge with the higher posting. Advance warning signs should be placed before each bridge so that overweight vehicles can take a detour route.

When posting two or more bridges for the most restrictive posting, it is possible that the result will be more restrictive than either bridge posted individually. Example, the one unit – two unit – three unit posting for bridge #1 is 12-40-45 and the posting for bridge # 2 is 10-42-47, Posting the bridges the same would require a posting of 10-40-45. It is important that no bridge shall be posted to allow any load which exceeds its computed capacity.

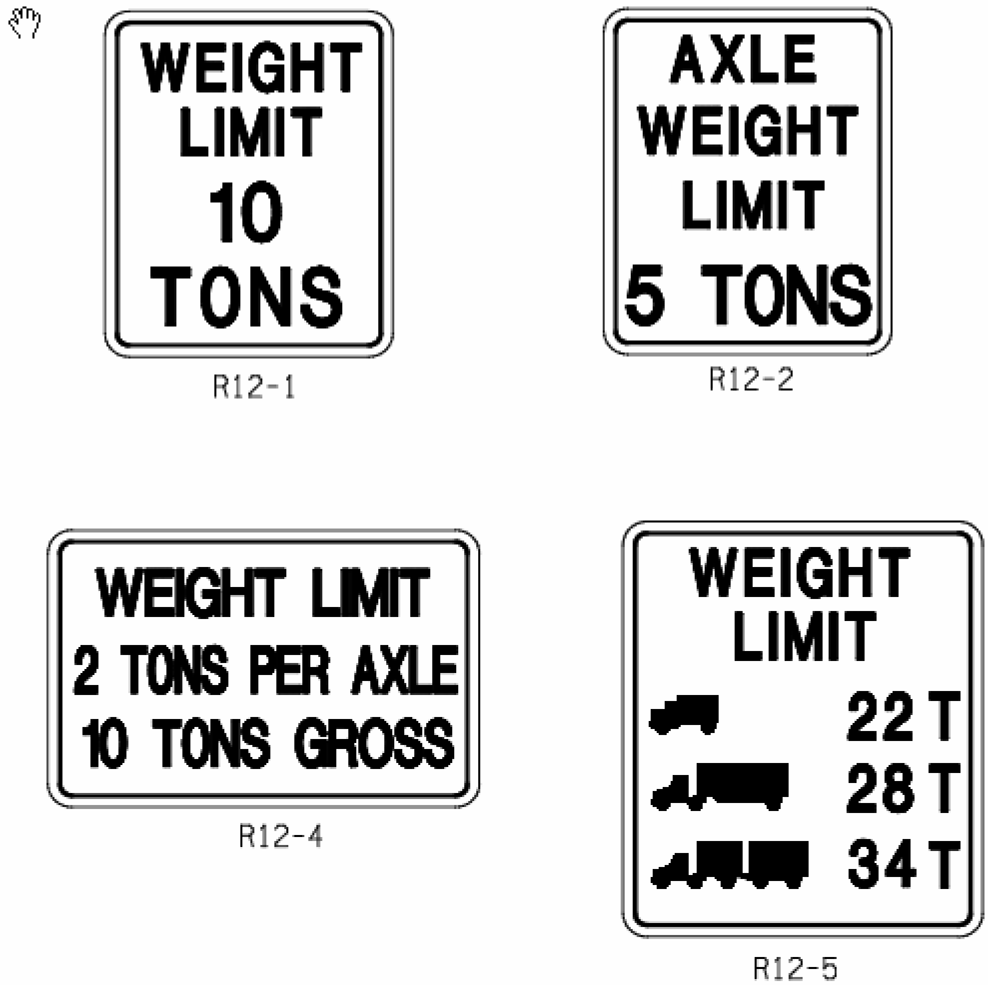


FIGURE 7.1
Sign Posting Examples

Chapter 9

CALCULATION EXAMPLES

April 03, 2009

Example 9-1 LFR General

A simply supported bridge with a span length of 40-ft requires load rating. The structure is located in an agency that allows designated loading. The design loading is HS-20. Structural analysis finds the total moment capacity of the controlling beam is 1000 k*ft. The dead load moment is 100 k*ft. The live load distribution factors are 0.5 for multiple lanes loaded, and 0.39 for a single lane loaded. The shear and service capacities do not control. The analysis was done using the LFR method.

The design load is greater than H15 and therefore the slab does not require analysis (See BAG, Chapter 4). The substructure is in good condition, and therefore the substructure does not require analysis.

Federal Inventory Rating, Item 66

The Impact for a span of 40-ft is 0.3. The weight of an HS-20 vehicle is 36-tons.
From Table 10.9 of the BAG, the live load moment for a 40-ft span is 450 k*ft

$$RF = \frac{C - A_1 * D}{A_2 * L * GDF * (1 + I)}$$

Eq 6B.5.1-1 The Manual for Bridge Evaluation

where:

RF	Rating Factor
C	Capacity
D	Dead Load Effect
L	Live Load Effect
GDF	Girder Distribution Factor
I	Impact
A_1	Dead Load Effect Factor
A_2	Live Load Effect Factor

$$RF = \frac{1000 - 1.3*100}{2.17*450*0.5*(1+0.3)} = 1.37$$

It is acceptable to enter the Federal Inventory Rating as a Rating Factor. However, the Rating Factor (RF) can be converted to Metric Tons, as shown in the following equations.

Multiply the RF by the weight of the HS20 truck

$$1.37 * 36 tons = 49.32 tons$$

Convert to metric tons

$$49.32 tons * 0.907 \frac{metrictons}{tons} = 44.7 metrictons$$

<u>Federal Operating Rating, Item 64F</u>

The Impact for a span of 40-ft is 0.3
From Table 10.9 of the BAG, the live load moment for a 40-ft span is 450 k*ft

$$RF = \frac{C - A_1 * D}{A_2 * L * GDF * (1 + I)}$$

$$RF = \frac{1000 - 1.3*100}{1.3*450*0.5*(1+0.3)} = 2.29$$

Once again, the RF could be submitted or the Rating in Metric Tons can be found.

Multiply the RF by the weight of the HS20 truck

$$2.29 * 36tons = 82.44tons$$

Convert to Metric Tons

$$82.44tons * 0.907 \frac{metrictons}{tons} = 74.8metrictons$$

Note: Metric tons greater than 99.9 need to be coded as 99.9. Entering a rating of 125.1 metric tons, for example, would be recorded as 12.5 metric tons.

<u>Michigan Operating Rating (Legal or Posting Load Rating), Item 64M</u>

The Impact for a span of 40-ft is 0.3
From Table 10.5 of the BAG, the maximum live load moment for a Designated Loading, 40-ft span, is 680-kip*ft for Truck #17:

$$RF = \frac{C - A_1 * D}{A_2 * L * GDF * (1 + I)}$$

$$RF = \frac{1000 - 1.3*100}{1.3*680*0.5*(1+.3)} = 1.51$$

As the rating factor is greater than 1, no posting is required.

For most structures, the #5, #18, and #23 vehicles will produce load effects close to the maximum ***when analyzed by LFR***. For structures where these three vehicles produce ratings significantly greater than 1, engineering judgement may be used to determine if the other twenty-five vehicles need to be analyzed.

MDOT Overload Class, Item 193

From Table 10.10 of the BAG, the maximum live load moment for a Class A loading, 40-ft span is 964 k*ft. The GDF for this rating is taken as 0.39 as Overloads are analyzed as single lane loading in ASR and LFR.

$$RF = \frac{C - A_1 * D}{A_2 * L * GDF * (1 + I)}$$

$$RF = \frac{1000 - 1.3 * 100}{1.3 * 964 * 0.39 * (1 + 0.3)} = 1.37$$

The RF>1 for all Overload trucks and therefore the Overload Class is A.

A sample summary sheet is shown in Figure 9-1.

BRIDGE ANALYSIS SUMMARY

Bridge ID B01-00001

The above structure was analyzed using: Hand Calcs

Version or Other:

The analysis is based on field inspection dated: 1/1/1998

The controlling component and failure mode are:

Moment at Midspan

NEW INVENTORY CODING

NBI Item 63- Operating Rating Method	6-LF Rating Factor	
NBI Item 64F- Federal Operating Rating	2.29	Rating Factor
MDOT Item 64MA- Michigan Operating Method	6-LF Rating	
MDOT Item 64MB- Michigan Operating Rating	1.51	Rating Factor
MDOT Item 64MC and D- Michigan Operating Truck	17	D - Designated
NBI Item 65- Inventory Rating Method	6-LF Rating Factor	
NBI Item 66- Federal Inventory Rating	1.37	Rating Factor
NBI Item 41- Open Posted Closed	A-Open	
NBI Item 70- Bridge Posting	5 - 100% or more	
NBI Item 141- Posted Loading		US Tons
MDOT Item 193A- Michigan Overload Class	A	
MDOT Item 193C- Overload Status	- No Restriction	

Analyzed By- Signature and Date	ABC	1-2-98
Checked By- Signature and Date	DEF	1-3-98
Database Updated By- Initials and Date	GHI	1-4-98

Figure 9-1

Example 9-2 LFR with Deterioration

10 years later, the same structure is inspected and significant deterioration of the beam is found. The maximum moment capacity is reduced to 400 k*ft based upon the inspection. The deck and substructure are both in good condition. All other information remains the same.

The design load is greater than H15 and therefore the slab does not require analysis (See Chapter 4 of the BAG). The substructure is in good condition, and therefore the substructure does not require analysis.

Federal Inventory Rating, Item 66

$$RF = \frac{400 - 1.3 * 100}{2.17 * 450 * 0.5 * (1 + 0.3)} = 0.43$$

Federal Operating Rating, Item 64F

$$RF = \frac{400 - 1.3 * 100}{1.3 * 450 * 0.5 * (1 + 0.3)} = 0.71$$

Michigan Operating Rating (Legal or Posting Load Rating), Item 64M

$$RF = \frac{400 - 1.3 * 100}{1.3 * 680 * 0.5 * (1 + 0.3)} = 0.47$$

The RF is less than 1.00, and so posting must be considered. It is helpful to calculate the Capacity Available for Live Load (L_A). This is found by rearranging the load rating equation.

Calculate the Live Load Capacity assuming an Operating Rating of 1.0

$$L_A = \frac{C - 1.3 * D}{1.3 * RF * GDF * (1 + I)}$$

$$L_A = \frac{400 - 1.3 * 100}{1.3 * 1 * .5 * (1 + 0.3)} = 319.5kft$$

As this is designated loading, Tables 10.4, 10.5, and 10.6 of the BAG will be checked.

For 1-Unit, Designated Load vehicles, check Table 10.4 for a 40-ft span. Trucks 2, 3, 4 and 5 have live load moments greater than 319.5k*ft. Truck #2 has the highest Moment to Weight ratio (7.96), and therefore controls. The Live Load Moment for Truck #2 is 377k*ft and it weighs 23.7tons.

$$\frac{319.5}{7.96} = 40.1kips \approx 20tons$$

For continuous span configurations that are not provided in the BAG, the following steps may be taken to identify the posting load.

Calculate the rating factor for all trucks of that Unit Type. For all vehicles with Rating Factors less than 1.00, multiply the Rating Factor by the Truck Weight to find the Posting Load. The lowest Posting Load should be selected.

Truck Number	Moment (k*ft)	RF	Truck Weight (Ton)	Posting Load (Ton)
1	268	1.19	16.7	NA (RF>1)
2	377	0.85	23.7	**20.1**
3	406	0.79	27.2	21.5
4	488	0.65	33.7	21.9
5	463	0.69	42.0	29.0

Table 9-1

For 2-Unit, Designated Load vehicles, check Table 10.5 of the BAG for a 40-ft span. Trucks 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18 and 27 have live load moments greater than 319.5k*ft. Truck #9 has the highest Moment to Weight ratio (7.08), and therefore controls. The Live Load Moment for Truck #9 is 364k*ft and it weighs 25.7tons.

$$\frac{319.5}{7.08} = 45.1 kips \approx 22 tons \text{ (rounding down)}$$

For 3-Unit, Designated Load vehicles, check Table 10.6 of the BAG for a 40-ft span. All of the 3-unit trucks have live load moments greater than 319.5k*ft. Truck #20 has the highest Moment to Weight ratio (4.71), and therefore controls. The Live Load Moment for Truck #20 is 411k*ft and it weighs 43.7tons.

$$\frac{319.5}{4.71} = 67.8 kips \approx 33 tons \text{ (rounding down)}$$

The posting for this structure would be:

1-Unit	20*tons*
2-Unit	22*tons*
3-Unit	33*tons*

While the Posting Load gives us the lowest tonnage for signing purposes, this does not always correspond to the lowest Rating Factor for that truck type. Looking at Table A-1, we find that the lowest 1-Unit Truck Rating Factor is 0.65, Truck #4, although the Posting Tonnage was controlled by Truck #2 with a Rating Factor of 0.85. From the Michigan Operating Calculation above, we know that the controlling 2-Unit Rating Factor is 0.47. Looking in Table 10.6 of the BAG, we find the maximum moment of the 3-Unit Trucks to be 601-k*ft for the #23 Truck with a corresponding Rating Factor of 0.53. These values will be recorded on the Summary Sheet.

MDOT Overload Class, Item 193

From Table 10.10 of the BAG, the maximum live load moment for a Class A loading, 40-ft span is 964 k*ft.

$$RF = \frac{C - A_1 * D}{A_2 * L * GDF * (1 + I)}$$

$$RF = \frac{400 - 1.3 * 100}{1.3 * 964 * 0.39 * (1 + 0.3)} = 0.42$$

The RF<1 for maximum Class A Overload truck. The engineer could next check Class B loading from Table 10.11 followed by Class C and Table 10.12 using the method outlined for Class A above. Alternatively, the Permit Live Load Capacity assuming single lane distribution and an Operating Rating of 1.0 can be found and compared to the Class B and C loading.

$$L_{APermit} = \frac{C - 1.3 * D}{1.3 * RF * GDF_{Permit} * (1 + I)}$$

$$L_{APermit} = \frac{400 - 1.3 * 100}{1.3 * 1 * 0.39 * (1 + 0.3)} = 409.6kft$$

From Table 10.11, the maximum live load moment for a Class B loading, 40-ft span is 822 k*ft and from Table 10.12, the maximum live load moment for a Class C loading, 40-ft span is 666 k*ft. The Permit Live Load Capacity is 409.6 k*ft, which is less than the Class B and Class C loadings and therefore the bridge is Class D. The Permits section may require the allowable axle load for each Overload Vehicle. This may be found by comparing the axle weights and maximum moment for each truck to the Permit Live Load Capacity, as shown in Table 9-2, and sample steps are outlined below.

Find the axle load from Figure 8.1 in the BAG.

$$Axle_{Truck1} = 60kips$$

Find the live load moment from Table 10.10 in the BAG.

$$Moment_{Truck1} = 600kft$$

Compare the Permit Live Load Capacity to the axle load and live load moment for the truck.

$$AllowableAxle_{Truck} = \frac{L_{APermit}}{Moment_{Truck}} * Axle_{Truck}$$

$$AllowableAxle_{Truck1} = \frac{409.6kft}{600kft} * 60kips = 41.0kips$$

Class C			Allowable Axle (kips)
Truck	Axle (kips)	Moment (k*ft)	
1	60	600	41.0
2	60	600	41.0
3	57	641	36.4
4	49	666	30.1
5	44	655	27.5
6	30	664	18.5
7	31	645	19.7
8	24	656	15.0
9	22	625	14.4
10	20	645	12.7
11	46	665	28.3
12	31	615	20.6
13	34	657	21.2
14	24	645	15.2
15	20	660	12.4
16	17	653	10.7
17	17.3	647	11.0
18	24	561	17.5
19	21.5	589	15.0
20	20.5	655	12.8

Table 9-2

A sample Summary Sheet is shown in Figure 9-2.

BRIDGE ANALYSIS SUMMARY

Bridge ID B01-00001

The above structure was analyzed using: Hand Calcs

Version or Other:

The analysis is based on field inspection dated: 1/1/2008

The controlling component and failure mode are:

Moment at Midspan including deterioration

NEW INVENTORY CODING

NBI Item 63- Operating Rating Method	6-LF Rating Factor	
NBI Item 64F- Federal Operating Rating	0.71	Rating Factor
MDOT Item 64MA- Michigan Operating Method	6-LF Rating	
MDOT Item 64MB- Michigan Operating Rating	0.47	Rating Factor
MDOT Item 64MC and D- Michigan Operating Truck	17	D - Designated
NBI Item 65- Inventory Rating Method	6-LF Rating Factor	
NBI Item 66- Federal Inventory Rating	0.43	Rating Factor
NBI Item 41- Open Posted Closed	B-Requires Posting	
NBI Item 70- Bridge Posting	0 - 59% or less	
NBI Item 141- Posted Loading	20-22-34	US Tons
MDOT Item 193A- Michigan Overload Class	D	
MDOT Item 193C- Overload Status	- No Restriction	

Analyzed By- Signature and Date ABC 1-2-2008

Checked By- Signature and Date DEF 1-3-2008

Database Updated By- Initials and Date GHI 1-4-2008

Figure 9-2

Example 9-3 LRFR General

A simply supported bridge with a span length of 40-ft requires load rating. The structure is located in an agency that allows designated loading. The design loading is HL-93. Structural analysis finds the total moment capacity (Strength I and II) of the controlling beam is 1000 k*ft. The capacity for Service II is 950 k*ft. The dead load moment is 100 k*ft. The live load distribution factors are 0.56 for multiple lanes loaded, and 0.44 for a single lane loaded. The shear and service capacities do not control. The ADTT of this structure is unknown, and so 5000 will be assumed. The analysis was done using the LRFR method.

The design load is according to HL-93 and therefore the slab does not require analysis (See Chapter 4 of the BAG). The substructure is in good condition, and therefore the substructure does not require analysis.

Federal Inventory Rating, Item 66

From Table E6A-1 of The Manual for Bridge Evaluation (MBE), the HL-93 Design Load (including Impact) is 722.0-k*ft per lane.

$$RF = \frac{C - \gamma_{DC} * DC - \gamma_{DW} * DW \pm \gamma_p * P}{\gamma_L * (LL + IM)}$$ Eq 6A.4.2.1-1 MBE

where:

RF	Rating Factor
C	Capacity
DC	Dead Load Effect due to structural components
DW	Dead Load Effect due to wearing surface and utilities
P	Permanent Loads other than dead loads
LL	Live Load Effect (including girder distribution)
IM	Dynamic Load Allowance (33% of tandem or truck allowance)
γ_{DC}	LRFD load factor for structural components
γ_{DW}	LRFD load factor for wearing surface and utilities
γ_P	LRFD load factor for permanent loads other than dead loads (γ_P=1)
γ_L	LRFD load factor for live load

In load rating, wearing surfaces that have been field verified may be treated as structural components (DC). In this structure, there are no terms for DW or P.

$$RF = \frac{1000 - 1.25 * 100}{1.75 * 722 * 0.56} = 1.24$$ Strength I

$$RF = \frac{950 - 1.00 * 100}{1.30 * 722 * 0.56} = 1.62$$ Service II

It is acceptable to enter the Federal Inventory Rating as a Rating Factor. However, the Rating Factor (RF) can be converted to Metric Tons, as shown in the following equations.

Multiply the RF by the weight of the HS20 truck

$$1.24 * 36 tons = 44.64 tons$$

Convert to metric tons

$$44.64 tons * 0.907 \frac{metrictons}{tons} = 40.5 metrictons$$

<u>Federal Operating Rating, Item 64F</u>

For Operating Rating, the γ_L changes to 1.35 for the Strength Limit State.

$$RF = \frac{1000 - 1.25 * 100}{1.35 * 722 * 0.56} = 1.60 \qquad \text{Strength I}$$

$$RF = \frac{950 - 1.00 * 100}{1.00 * 722 * 0.56} = 2.10 \qquad \text{Service II}$$

Once again, the RF could be submitted or the Rating in Metric Tons can be found.

Multiply the RF by the weight of the HS20 truck

$$1.60 * 36 tons = 57.60 tons$$

Convert to Metric Tons

$$57.60 tons * 0.907 \frac{metrictons}{tons} = 52.2 metrictons$$

Note: Metric tons greater than 99.9 need to be coded as 99.9. Entering a rating of 125.1 metric tons, for example, would be recorded as 12.5 metric tons.

<u>Michigan Operating Rating (Legal or Posting Load Rating), Item 64M</u>

The maximum ***factored with impact*** live load moment for the Strength Limit State, 5000 ADTT, Designated Loading, 40-ft span, is 1.80*1.33*488k*ft = 1168-kip*ft for Truck #4. This is found by combining Tables 10.4, 10.5 and 10.6 from the BAG and Table 4a-1 to find the controlling effect.

$$RF = \frac{C - \gamma_{DC} * DC - \gamma_{DW} * DW \pm \gamma_p * P}{\gamma_L * (LL + IM)} \qquad \text{Eq 6A.4.2.1-1 MBE}$$

$$RF = \frac{1000 - 1.25 * 100}{1168 * 0.56} = 1.34 \qquad \text{Strength I}$$

The maximum ***factored with impact*** live load moment for the Service Limit State, 5000 ADTT, Designated Loading, 40-ft span, is 1.00*1.33*680k*ft = 904-kip*ft for Truck #17:

$$RF = \frac{950 - 1.00*100}{904*0.56} = 1.68 \qquad \text{Service II}$$

As the rating factor is greater than 1, no posting is required.

<u>MDOT Overload Class, Item 193</u>

The maximum ***factored with impact*** live load moment for the Strength Limit State, Class A loading, 5000 ADTT, 40-ft span is 1666 k*ft. This is found from combining Table 10.10 from the BAG with Table 4a-4 to find the controlling effect. As the MDOT Overload Class refers to routine, annual permits, the multi-lane distribution will be applied.

$$RF = \frac{C - \gamma_{DC} * DC - \gamma_{DW} * DW \pm \gamma_p * P}{\gamma_L * (LL + IM)} \qquad \text{Eq 6A.4.2.1-1 MBE}$$

$$RF = \frac{1000 - 1.25*100}{1666*0.56} = 0.94$$

The RF<1 for maximum Class A Overload truck and therefore try Class B.

The maximum ***factored with impact*** live load moment for the Strength Limit State, Class B loading, 5000 ADTT, 40-ft span is 1571 k*ft.

$$RF = \frac{1000 - 1.25*100}{1571*0.56} = 1.00$$

Service Checks are optional for Permit Trucks and will not be checked for this structure.

The RF=1 for maximum Class B Overload truck and therefore the bridge is Class B.

A sample summary sheet is shown in Figure 9-3.

BRIDGE ANALYSIS SUMMARY

Bridge ID B01-00001

The above structure was analyzed using: Hand Calcs

Version or Other:

The analysis is based on field inspection dated: 10/1/2010

The controlling component and failure mode are:

Moment at Midspan

NEW INVENTORY CODING

NBI Item 63- Operating Rating Method	8-LRF Rating Factor (HL-93)	
NBI Item 64F- Federal Operating Rating	1.60	Rating Factor
MDOT Item 64MA- Michigan Operating Method	8-LRF Rating	
MDOT Item 64MB- Michigan Operating Rating	1.34	Rating Factor
MDOT Item 64MC and D- Michigan Operating Truck	4	D - Designated
NBI Item 65- Inventory Rating Method	8-LRF Rating Factor (HL-93)	
NBI Item 66- Federal Inventory Rating	1.24	Rating Factor
NBI Item 41- Open Posted Closed	A-Open	
NBI Item 70- Bridge Posting	5 - 100% or more	
NBI Item 141- Posted Loading		US Tons
MDOT Item 193A- Michigan Overload Class	B	
MDOT Item 193C- Overload Status	- No Restriction	

Analyzed By- Signature and Date	ABC	10/2/2010
Checked By- Signature and Date	DEF	10/3/2010
Database Updated By- Initials and Date	GHI	10/4/2010

Figure 9-3

Example 9-4 LRFR with Deterioration

10 years later, the same structure is inspected and significant deterioration of the beam is found. The maximum moment capacity is reduced to 400 k*ft based upon the inspection. The deck and substructure are both in good condition. All other information remains the same.

The design load is according to HL-93 and therefore the slab does not require analysis (See Chapter 4 of the BAG). The substructure is in good condition, and therefore the substructure does not require analysis. From the previous rating, it is known that Service does not control.

Federal Inventory Rating, Item 66

$$RF = \frac{400 - 1.25 * 100}{1.75 * 722 * 0.56} = 0.39$$ Strength I

Federal Operating Rating, Item 64F

$$RF = \frac{400 - 1.25 * 100}{1.35 * 722 * 0.56} = 0.50$$ Strength I

Michigan Operating Rating (Legal or Posting Load Rating), Item 64M

$$RF = \frac{400 - 1.25 * 100}{1168 * 0.56} = 0.42$$ Strength I

The RF is less than 1.00, and so posting must be considered. It is helpful to calculate the LRFR Capacity Available for Live Load (LL_A). The LL_A for LRFR is different than LFR. In LFR, the L_A should be compared to the unfactored load effect without impact. For LRFR, LL_A is compared to the factored load effect including impact. This is found by rearranging the load rating equation.

Calculate the Live Load Capacity assuming an Operating Rating of 1.0

$$LL_A = \frac{C - \gamma_{DC} * DC - \gamma_{DW} * DW \pm \gamma_p * P}{RF * GDF}$$

$$LL_A = \frac{400 - 1.25 * 100}{1 * 0.56} = 491 kft$$

As this is Designated Loading and 5000 ADTT, Tables 10.4, 10.5 and 10.6 from the BAG are combined with Table 4a-1 to find the controlling effect.

For 1-Unit, 5000 ADTT, Designated Load vehicles, check Tables 10.4 and 4a-1 for a 40-ft span. All trucks have factored with impact live load moments greater than 491-k*ft.

Calculate the rating factor for all trucks of that Unit Type. For all vehicles with Rating Factors less than 1.00, calculate the Posting Load as:

$$PostingLoad = \frac{W*(RF-0.3)}{0.7}$$ Eq 6A.8.3-1 MBE

Truck Number	Moment (k*ft)	RF	W=Truck Weight (Ton)	Posting Load (Ton)
1	642	0.76	16.7	11.0
2	903	0.54	23.7	8.1
3	972	0.51	27.2	8.2
4	1168	**0.42**	33.7	5.8
5	1079	0.46	42.0	9.6

Table 9-3

For 2-Unit, 5000 ADTT, Designated Load vehicles, check Tables 10.5 and 4a-1 for a 40-ft span. All trucks have factored with impact live load moments greater than 491-k*ft. A spreadsheet was created to select the correct posted load similar to the method described for the 1-Unit Truck.

Minimum RF: 0.44 (Truck #16)
Minimum Posting Load: 8.7 (Truck #11)

For 3-Unit, 5000 ADTT, Designated Load vehicles, check Table 10.6 and 4a-1 for a 40-ft span. All trucks have factored with impact live load moments greater than 491-k*ft. A spreadsheet was created to select the correct posted load similar to the method described for the 1-Unit Truck.

Minimum RF: 0.51 (Truck #23)
Minimum Posting Load: 14.1 (Truck #20)

The posting for this structure would be:

1-Unit	5*tons*
2-Unit	8*tons*
3-Unit	14*tons*

While the Posting Load gives us the lowest tonnage for signing purposes, this does not always correspond to the lowest Rating Factor for that truck type. Looking at the 2-Unit trucks, we find that the lowest Rating Factor is 0.44, Truck #16, although the Posting Tonnage was controlled by Truck #11 with a Rating Factor of 0.76. The lowest Rating Factors will be recorded on the Summary Sheet. Please note that this bridge was analyzed using artificial numbers similar to the LFR process. Posting limits as low as this may require engineering judgement regarding further action such as reducing the bridge to a single lane or closing it until repairs are made.

MDOT Overload Class, Item 193

From Tables 10.10 and 4a-6, the maximum ***factored with impact*** live load moment for the Strength Limit State, Class C loading, 5000 ADTT, 40-ft span is 1478 k*ft. This is much greater than the Capacity Available for Live Load, 491-k*ft, as calculated above.

The RF<1 for maximum Class C Overload trucks and therefore the bridge is Class D. The Permits section may require the allowable axle load for each Overload Vehicle. This may be found by comparing the axle weights and maximum moment for each truck to the Permit Live Load Capacity, as shown in Table 9-4, and sample steps are outlined below.

Find the axle load from Figure 8.1 in the BAG.

$$Axle_{Truck1} = 60kips$$

Find the live load moment from Table 10.12 in the BAG.

$$Moment_{Truck1} = 600kft$$

Find the live load factor from Table 4a-4.

$$\gamma_{LL\_Truck1} = 1.39$$

Compare the Permit Live Load Capacity to the axle load and live load moment for the truck.

$$AllowableAxle_{Truck} = \frac{LL_{APermit}}{Moment_{Truck} * \gamma_{LL} * 1.33} * Axle_{Truck}$$

$$AllowableAxle_{Truck1} = \frac{491kft}{600kft * 1.39 * 1.33} * 60kips = 26.6kips$$

Class C, 5000 ADTT				Allowable Axle (kips)
Truck	Axle (kips)	LL Factor, γ_L	Unfactored Moment (k*ft)	
1	60	1.39	600	26.6
2	60	1.39	600	26.6
3	57	1.43	641	23.0
4	49	1.58	666	17.2
5	44	1.7	655	14.6
6	30	1.67	664	10.0
7	31	1.64	645	10.8
8	24	1.5	656	9.0
9	22	1.5	625	8.7
10	20	1.37	645	8.4
11	46	1.28	665	20.0
12	31	1.3	615	14.3
13	34	1.23	657	15.5
14	24	1.2	645	11.4
15	20	1.14	660	9.8
16	17	1.13	653	8.5
17	17.3	1.1	647	9.0
18	24	1.1	561	14.4
19	21.5	1.1	589	12.3
20	20.5	1.1	655	10.5

Table 9-4

A sample Summary Sheet is shown in Figure 9-4.

BRIDGE ANALYSIS SUMMARY

Bridge ID B01-00001

The above structure was analyzed using: Hand Calcs

Version or Other:

The analysis is based on field inspection dated: 10/1/2020

The controlling component and failure mode are:

Moment at Midspan including deterioration

NEW INVENTORY CODING

NBI Item 63- Operating Rating Method	8-LRF Rating Factor (HL-93)	
NBI Item 64F- Federal Operating Rating	0.50	Rating Factor
MDOT Item 64MA- Michigan Operating Method	8-LRF Rating	
MDOT Item 64MB- Michigan Operating Rating	0.42	Rating Factor
MDOT Item 64MC and D- Michigan Operating Truck	4	D - Designated
NBI Item 65- Inventory Rating Method	8-LRF Rating Factor (HL-93)	
NBI Item 66- Federal Inventory Rating	0.39	Rating Factor
NBI Item 41- Open Posted Closed	B-Requires Posting	
NBI Item 70- Bridge Posting	0 - 59% or less	
NBI Item 141- Posted Loading	5-8-14	US Tons
MDOT Item 193A- Michigan Overload Class	D	
MDOT Item 193C- Overload Status	- No Restriction	

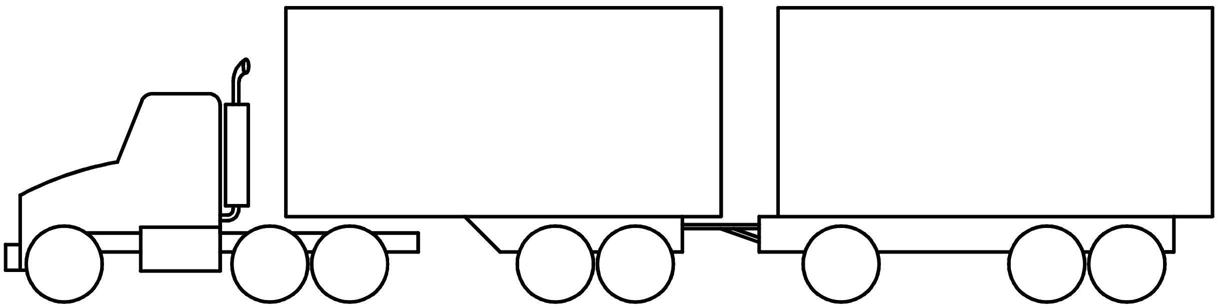
Analyzed By- Signature and Date ABC 10/2/2020

Checked By- Signature and Date DEF 10/3/2020

Database Updated By- Initials and Date GHI 10/4/2020

Figure 9-4

BRIDGE ANALYSIS GUIDE
2005 Edition
with
2009 Interim Update
Part 2



MDOT
Michigan Department of Transportation

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CONSTRUCTION AND TECHNOLOGY SUPPORT AREA