APPENDIX A
RECOMMENDED METHODOLOGY FOR ESTIMATING BRIDGE NETWORK COSTS DUE TO TRUCK WEIGHT LIMIT CHANGES

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This appendix presents the procedure of the recommended methodology for estimating bridge network costs resulting from changes in truck weight limits. The concept and background information for this methodology is presented in Chapter 3. Thus, reference to that chapter may be helpful in reviewing the methodology presented here as a procedure. The following four cost impact categories are included separately in the next sections.

1) Fatigue of existing steel bridges,
2) Fatigue of existing reinforced concrete (RC) decks,
3) Deficiency due to overstress for existing bridges, and
4) Deficiency due to overstress for new bridges.

A planning period PP needs to be determined before using the procedure. Interaction between the analyses for different cost impact categories is not explicitly identified in this procedure, such as avoiding analysis already done for another cost impact category or avoiding possible double counting costs for the same bridge but for different treatment actions. For example, if a bridge is to be replaced due to deficiency in Category 3, it should be excluded then from cost estimation for other categories. This should be exercised in application of the methodology.

This appendix also contains the default data needed for the Level I analysis. The default database includes the FHWA VMT data for Year 2000. Only a typical sample of this data set is shown here for a functional class of roads in a state. It is because the database includes this kind of information for 12 functional classes for all the states (including the District of Columbia) and it is too large to be printed here. In attachment 5 Software Module Carris, this database is provided electronically in each of the example files.

A-1 Methodology for Cost Impact Category 1: Fatigue of Existing Steel Bridges

A-1.I - Level I Procedure (with lower data requirements)

1. Identify all possibly vulnerable bridges (for example, those with steel primary members and on impacted routes). Partition them into N groups, each having similar features (for example, by age, type of structure, type of fatigue prone detail, functional class of the road, truck traffic volume, etc.) Randomly select one (or more) typical bridge(s) representative for each group, whose result will be used to estimate the entire group's cost by multiplication. (This screening may use the guidelines given in Section A-5.1.3.)

2. For Bridge Group n=1 (for the typical bridge or each of the typical bridges of this group):
   a) Generate the truck weight histogram (TWH) and truck volume for the Base Case, using the agency bridge inventory (or the NBI) and available WIM data (or the FHWA VMT data sampled in Data Set A-5.2.1). Then predict the TWH and truck volume under the Alternative Scenario, using the recommended method in Section A-5.1.1. (Some groups may have the same TWHs because they carry roads that belong to the same functional class.) The truck volumes can be estimated using AADT, truck traffic percentage, and traffic growth factor available or derivable from the agency's bridge inventory or the NBI.)
   b) Estimate the remaining mean and safe lives for both the Base Case and the Alternative Scenario, using the results of Step 2.a). (This step should follow the procedure given in the *AASHTO Manual for Condition Evaluation of Bridges* or the new *AASHTO Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges*. Note that the estimation of total number of stress cycles over the bridge life can be improved, as given in Eq. 3.3.2.4 and discussed in Section 3.3.)
   c) Select an action from the following options: i) do nothing, ii) repair, iii) monitoring, iv) replacement, v) combination of ii) and iii), or vi) combination of iii) and iv). The default action is repair.
   d) Estimate the cost of action for the typical bridge, according to the selection made in Step 2.c). (Data Set A-5.2.4 can be used as default costs for repair.)
   e) Compute the changed probability of failure according to Eq. 3.3.3.3 for the pre-selected planning period PP, using the remaining mean and safe lives obtained in Step 2.b)
   f) Compute the expected cost as the product of the cost of action from Step 2.d) and the changed probability of failure from Step 2.e).
   g) Estimate the costs for the group of bridges by multiplying the expected cost for the representative bridge obtained in Step 2.f) by the number of bridges in the group. If more that one typical bridge is used for the group, average the expected costs for these bridges. Then multiply this averaged cost by the number of bridges in the group.

3. Repeat Step 2 for Bridge Group n=n+1, until n=N

4. Sum the costs from all bridge groups.

The following parameters may need to be examined for their effects on the final result in the sensitivity analysis:

1) The window parameters defined in Fig. 3.1 and the percentage increase parameter for exogenous shift in Eq. 3.2.2.8 for the TWH prediction method for the Alternative Scenario.
2) Load distribution factor used to calculate the stress range.
3) Dynamic impact factor.
4) ADTT.
5) Unit cost data used.
6) Selection of responding action.
7) Sample bridges selected.

A-1.II - Level II Procedure (with higher data requirements)

1. Identify all possibly vulnerable bridges (for example, those with primary steel members and on impacted routes, or other details of significant impact cost), say the total number of such bridges is M. (This screening may use the guidelines given in Section A-5.1.3.)

2. For Bridge m=1:
   a) Generate the truck weight histogram (TWH) and truck volume for the Base Case, using the agency bridge inventory and WIM data. Then predict the TWH and truck volume under the Alternative Scenario, using the recommended method in Section A-5.1.1.
   b) Estimate the remaining mean and safe lives for both the Base Case and the Alternative Scenario, using the results of Step 2.a) and site-specific data. (This step should follow the procedure given in the *AASHTO Manual for Condition Evaluation of Bridges* or the new *AASHTO Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges*. Note that the estimation of the total number of stress cycles over the bridge life can be improved, as given in Eq. 3.3.2.4 and discussed in Section 3.3.)
   c) Select an action from the following options: I) do nothing, ii) repair, iii) monitoring, iv) replacement, v) combination of ii) and iii), or vi) combination of iii) and iv).
   d) Estimate the cost of action for the bridge, according to the selection made in Step 2.c), using jurisdiction specific unit cost data.
   e) Compute the changed probability of failure according to Eq. 3.3.3.3 for the pre-selected planning period PP, using the remaining mean and safe lives obtained in Step 2.b).
   f) Compute the expected cost as the product of the cost of action from Step 2.d) and the changed probability of failure from Step 2.e).

3. Repeat Step 2 for Bridge m=m+1, until m=M

4. Sum all costs.

The following parameters may need to be examined for their effects on the final result in the sensitivity analysis:

1) The window parameters defined in Fig. 3.1 and the percentage increase parameter for exogenous shift in Eq. 3.2.2.8 for the TWH prediction method for the Alternative Scenario.
2) Load distribution factor used to calculate the stress range.
3) Dynamic impact factor.
4) ADTT.
5) The unit cost data used.
6) Selection of responding action.

A-2 Methodology for Cost Impact Category 2: Fatigue of Reinforced Concrete Decks

A-2.I Level I Procedure (with lower data requirements)

1. Identify all possibly vulnerable bridges (on impacted roads and with a reinforced concrete deck supported by beams). Partition them into N groups, each having similar features (for example, by age, deck thickness, concrete strength, structure types, etc.) Randomly select one (or more) typical bridge(s) representative for each group, whose result will be used to estimate the entire group's cost by multiplication. The guidelines in Section A-5.1.4 can be used for this screening and grouping.

2. For Bridge Group n=1 (for the typical bridge or each of the typical bridges of this group):
   a) Generate the wheel weight histogram (WWH) in Section A-5.1.2 for the Base Case, using WIM data (or the FHWA VMT data sampled in Data Set A-5.2.1 and apply the wheel weight generating method in Section A-5.1.2 based on the TWH) and the agency bridge inventory (or the NBI). Then predict the TWH under the Alternative Scenario, using the recommended method in Section A-5.1.1. (This step may be omitted if this TWH is available as a result of the analysis for Cost Impact Category 1 in Section A-1). Then apply the wheel weight generating method in Section A-5.1.2 to generate the WWH under the Alternative Scenario. (Some groups may have the same WWH because they carry roads that belong to the same function class. The truck volumes can be estimated using AADT, truck traffic percentage, and traffic growth factor available or derivable from the agency's bridge inventory or the NBI.)
   b) Estimate the remaining mean and evaluation lives for both the Base Case and the Alternative Scenario, using Eq. 3.4.2.1 and the results of Step 2.a). (This step should follow the procedure presented in Section 3.4. Its concept is similar to that for steel fatigue in the *AASHTO Manual for Condition Evaluation of Bridges* or the new *AASHTO Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges*.
   c) Select an action from the following options: i) do nothing, ii) patching and then concrete overlay, iii) concrete overlay, iv) patching and then asphalt concrete overlay, v) asphalt concrete overlay, or vi) patching and then replacement. The default action is concrete overlay.
   d) Estimate the unit cost of action for the selection made in Step 2.c) (Data Set A-5.2.5 can be used for the default cost estimation for concrete overlay.) The unit cost is in dollars per deck area.
   e) Compute the changed probability of failure for the pre-selected planning period PP, using Eq. 3.4.2.7. The probability of failure is defined as the probability that the deck reaches the end of service life within the planning period.
   f) Then compute the expected unit cost for the typical bridge as the product of the changed probability of failure from Step 2.e) and the cost from Step 2.d), using Eq. 3.4.2.7.

3. Estimate the cost for the group of bridges by multiplying the expected unit cost per deck area of the representative bridge obtained in Step 2.f) by the total deck area in the group. If more that

one representative bridge is used for the group, average the expected unit costs per deck area first. Then multiply this averaged expected unit cost by the total bridge deck area in the group.

4. Go to Step 2 for Bridge Group n=n+1, until n=N

5. Sum the costs from all groups.

The following parameters may need to be examined for their effects on the final result in the sensitivity analysis:

1) The window parameters defined in Fig. 3.1 and the percentage increase parameter for exogenous shift in Eq. 3.2.2.8 for the TWH prediction method for the Alternative Scenario.
2) Dynamic impact factor.
3) Cost data.
4) Deck thickness.
5) Concrete compressive strength.
6) Sample bridges selected.
7) Responding action.

A-2.II - Level II Procedure (with higher data requirements)

1. Identify all possibly vulnerable bridges (on impacted roads and with a reinforced concrete deck supported by beams), say the total number is M. The guidelines in Section A-5.1.4 may be used for this screening.

2. For Bridge m=1:
   a) Generate the wheel weight histogram (WWH) in Section A-5.1.2 for the Base Case, using WIM data and apply the wheel weight generating method in Section A-5.1.2 and the agency bridge inventory. Then predict the TWH under the Alternative Scenario, using the recommended method in Section A-5.1.1. (This TWH may have been made available in the analysis for Cost Impact Category 1 in Section A-2.) Apply the wheel weight generating method in Section A-5.1.2 to generate the WWH under the Alternative Scenario. (The truck volumes can be estimated using AADT, truck traffic percentage, and traffic growth factor available or derivable from the agency's bridge inventory.)
   b) Estimate the remaining mean and evaluation lives for both the Base Case and the Alternative Scenario, using Eq.3.4.2.1 and the results of Step 2.a). (This step should follow the procedure presented in Section 3.4. Its concept is similar to that for steel fatigue in the *AASHTO Manual for Condition Evaluation of Bridges* or the new *AASHTO Guide Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges*.
   c) Select an action from the following options: i) do nothing, ii) patching and then concrete overlay, iii) concrete overlay, iv) patching and then asphalt concrete overlay, v) asphalt concrete overlay, or vi) patching and then replacement.
   d) Estimate the unit cost of action for the selection made in Step 2.c), using jurisdiction specific data. The unit cost is in dollars per deck area.
   e) Compute the changed probability of failure according to Eq. 3.3.3.3 for the pre-selected planning period PP, using Eq. 3.4.2.7. The probability of failure is defined as the probability that the deck reaches the end of service life within the planning period.
   f) Then compute the expected unit cost for the typical bridge as the product of the changed probability of failure from Step 2.e) and the cost from Step 2.d), using Eq. 3.4.2.7.

3. Repeat Step 2 for Bridge m=m+1, until m=M

4. Sum all costs.

The following parameters may need to be examined for their effects on the final result in the sensitivity analysis:

1) The window parameters defined in Fig. 3.1 and the percentage increase parameter for exogenous shift in Eq. 3.2.2.8 for the TWH prediction method for the Alternative Scenario.
2) Dynamic impact factor.
3) Cost data.
4) Deck thickness.
5) Concrete compressive strength.
6) Responding action.

A-3 Methodology for Cost Impact Category 3: Deficiency Due to Overstress for Existing Bridges

A-3.I - Level I Procedure (with lower data requirements)

1. Identify the criterion for deficiency in the load rating format. Select a rating vehicle model that can cover the most severe practical-maximum-truck-loads under the Alternative Scenario. This model may include several vehicles, depending on the Alternative Scenario considered. These vehicles should produce the moment envelope for new legal or permit vehicles.

2. For each bridge in the network, use available ratings in the bridge inventory (with NBI being the default database), estimate the new load rating factor under the Alternative Scenario RF_{AS} as follows

$$RF_{AS} = RF_{BC} \, (M_{BC\ rating\ vehicle} / M_{AS\ rating\ vehicle}) \, / \, AF_{rating} \qquad (3.5.1.1)$$

where $M_{BC\ rating\ vehicle} / M_{AS\ rating\ vehicle}$ is the ratio of the maximum moments due to the current (Base Case) and the future (Alternative Scenario) rating vehicle models, for the critical section. Generic spans may be used for estimation of these maximum moments. RF in Eq. 3.5.1.1 stands for rating factor, with subscripts AS and BC respectively indicating the Alternative Scenario and Base Case. The live load factor adjustment factor for rating AF_{rating} is defined as

$$AF_{rating} = [2W_{AS}^* + 1.41\ t(ADTT_{AS})\ \sigma_{AS}^*] \, / \, [2W_{BC}^* + 1.41\ t(ADTT_{BC})\ \sigma_{BC}^*] \qquad (3.5.1.2)$$

where W* and σ* are the mean and standard deviation of the top 20 percent of the TWH, and t is a function of annual daily truck traffic (ADTT) as given in Table A-3.1 below. The TWHs used are functional class dependent. The TWHs for the Base Case can be generated using WIM data or the FHWA VMT data, sampled in Data Set A-5.2.1. The TWHs for the Alternative Scenario are generated using the recommended prediction method in Section A-5.1.1. The ADTT data can be taken from the agency's bridge inventory or the NBI.

3. Identify all deficient bridges under the Alternative Scenario (excluding those already deficient under the Base Case), according to the results of Steps 1 and 2. Namely these bridges have $RF_{BC} \geq 1.0$ and $RF_{AS} < 1.0$. The total number of deficient bridges is N.

4. For Deficient Bridge n=1, then n=n+1, until n=N
   a) Select responding action from the following options (according to RF_{AS} and possibly considering other information, such as other needs for the bridge). i) Do nothing, ii) posting with weight limit enforcement, iii) strengthening, iv) replacement, or v) combination of ii) and iii) or ii) and iv). The FHWA sufficient rating may also be considered in this decision process. The default responding action is replacement.

b) Estimate the cost for the responding action selected. (Data Sets A-5.2.3 and A-5.2.6 may be used as the default data to estimate enforcement and replacement costs.)

5. Sum all costs. (Note: If the number of additional deficient bridges is large, the deficient bridges may be partitioned into N groups, according to age, span length, material type, structure type, etc. Randomly select one or more representative bridges. Perform Step 4 for each representative bridge. Multiply the cost result, or the average cost result if more than one bridge is used to represent the group, by the number of bridges in the group. Then sum the costs of the groups.)

The following parameters may need to be examined for their effects on the final result in the sensitivity analysis:

1) The window parameters defined in Fig. 3.1 and the percentage increase parameter for exogenous shift in Eq. 3.2.2.8 for the TWH prediction method for the Alternative Scenario.
2) Responding action to deficiency.
3) Sample bridges, if used. (See Step 5 above.)
4) The generic spans used for moment calculation.

A-3.II – Level II Procedure (with higher data requirements)

1. Identify the criterion for deficiency (in the load rating format). Select a rating vehicle model that can cover the most severe practical maximum truck loads. This model may include several vehicles depending on the Alternative Scenario considered. These vehicles should produce the moment envelope for new legal or permit vehicles.

2. For each bridge in the network, use detailed information in the agency's bridge inventory and bridge plans, find the new rating factor under the Alternative Scenario RF_{AS} as follows

$$RF_{AS} = RF_{BC,\text{ using AS rating vehicle}} / AF_{rating} \qquad (3.5.1.3)$$

where $RF_{BC,\text{ using AS rating vehicle}}$ is the rating factor using the Base Case's live load factor but the new vehicle model under the Alternative Scenario.

$$AF_{rating} = [2W_{AS}^* + 1.41\ t(ADTT_{AS})\ \sigma_{AS}^*] / [2W_{BC}^* + 1.41\ t(ADTT_{BC})\ \sigma_{BC}^*] \qquad (3.5.1.2)$$

where W* and σ* are the mean and standard deviation of the top 20 percent of the TWH, and t is a function of annual daily truck traffic (ADTT) as given in Table A-3.1 below. Subscripts $_{BS}$ and $_{AS}$ respectively refer to the Base Case and the Alternative Scenario. The ADTT data can be taken from the agency's bridge inventory. The TWHs for the Base Case are generated using site specific or jurisdiction specific WIM data. The TWHs for the Alternative Scenario are generated using the recommended prediction method in Section A-5.1.1.

3. Identify all deficient bridges under the Alternative Scenario (excluding those already deficient under the Base Case), according to the results of Steps 1 and 2. These bridges should have $RF_{BC} \geq 1.0$ and $RF_{AS} < 1.0$. The total number of deficient bridges is M.

4. For Bridge m=1, then m=m+1, until m=M:
   a) Select responding action from the following options (according to RF_{AS} and possibly considering other information, such as other needs for the bridge). i) Do nothing, ii) posting with weight limit enforcement, iii) strengthening, iv) replacement, or v) combination of ii) and iii) or ii) and iv). The FHWA sufficient rating can also be considered in this decision process.
   b) Estimate the cost for the responding action selected.

5. Sum all costs.

The following parameters may need to be examined for their effects on the final result in the sensitivity analysis:

1) The window parameters defined in Fig. 3.1 and the percentage increase parameter for exogenous shift in Eq. 3.2.2.8 for the TWH prediction method for the Alternative Scenario.
2) Selected responding action.

Table A-3.1 Function t for Eq. 3.5.1.2 (AGLichtenstein & Associates 1999)

ADTT	t(ADTT)	
	Two or more lanes	One lane
5000	4.3	4.9
1000	3.3	4.5
100	1.5	3.9

A-4 Methodology for Cost Impact Category 4: Deficiency Due to Overstress for New Bridges

A-4.I – Level I Procedure (with lower data requirements)

1. Generate the TWH under the Base Case and predict the TWH under the Alternative Scenario for the network. (The FHWA VMT data can be used to generate the TWH for the Base Case, whose sample is given in Data Set A-5.2.1. The method of prediction is presented in Section A-5.1.1) Note that there is only one such TWH for the entire network respectively under the Base Case and Alternative Scenario. Namely, all roadways of different functional classes will use the same TWH. This is different from Cost Impact Category 3 where rating requirements with respect to truck load are site dependent or functional class dependent.

2. Determine an adjustment factor for design load as the ratio of the design live load factors for the Base Case and the Alternative Case, as follows:

$$AF_{design} = (2W_{AS}^* + 6.9\ \sigma_{AS}^*) / (2W_{BC}^* + 6.9\ \sigma_{BC}^*)$$

$$AF_{design} \geq 1 \qquad (3.6.1.3)$$

   W* and σ* are the mean and standard deviation of the top 20 percent of the TWH. Subscripts $_{BS}$ and $_{AS}$ respectively refer to the Base Case and Alternative Scenario.

3. Identify a new design vehicle load model that can cover the most severe truck loads under the Alternative Scenario. This model can be the practical maximum truck loads under the Alternative Scenario, and it may include multiple vehicles to envelope maximum moment effects due to new legal and permit vehicles.

4. Identify all bridges to be impacted (to be constructed). They may be approximated using recently constructed and replaced bridges. The number of years Q to look back in identifying these bridges may need iteration to have an appropriate number of bridges. The total number of bridges identified is N.

5. For Bridge n=1, use the following procedure to find the cost for the bridge.

   a) Find design load change factor DLCF as follows:

$$DLCF = (M_{AS,\ design\ vehicle} / M_{BC,\ design\ vehicle})\ AF_{design} \qquad (3.6.1.1)$$

$$M_{AS,\ design\ vehicle} / M_{BC,\ design\ vehicle} \geq 1 \qquad (3.6.1.2)$$

   where $M_{AS,\ design\ vehicle} / M_{BC,\ design\ vehicle}$ is the ratio of the maximum moments due to the design vehicle under the Base Case and the new design vehicle under the Alternative Scenario, for the critical section. Generic spans can be used for estimation of these maximum moments. Practically, the ratio should not be lower than 1. AF_{design} is the ratio between the live load factors under the Base Case and the Alternative Scenario. It should not be less than 1, either.

b) Based on DLCF, estimate the incremental new bridge cost. The default Data Set A-5.2.7 can be used for this purpose.

6. Repeat Step 5 for Bridge n=n+1, until n=N

7. Sum all costs and divide the sum by Q, to find an averaged annual incremental new bridge cost. Then multiply it by PP.

The following parameters may need to be examined for their effects on the final result in the sensitivity analysis:

1) The window parameters defined in Fig. 3.1 and the percentage increase parameter for exogenous shift in Eq. 3.2.2.8 for the TWH prediction method for the Alternative Scenario.
2) The bridges identified as expected new bridges. More years of history of new bridges constructed in the network may be included and averaged to an annual cost for this category of cost impact.
3) Possible increase of available resources that will result in more new bridges to be built. This may be covered at the network level by a growth factor to the total costs obtained.
4) The generic spans used for maximum moment estimation.

A-4.II – Level II Procedure (with higher data requirements)

1. Generate the TWH under the Base Case and predict the TWH under the Alternative Scenario for the network, using jurisdiction specific WIM data. Note that there is only one such TWH for the entire network respectively under the Base Case or Alternative Scenario. Namely, all roadways of different functional classes will use the same TWH. This is different from Cost Impact Category 3 where rating requirements with respect to truck load are site dependent or functional class dependent.

2. Determine an adjustment factor for design load as the ratio of the design live load factors for the Base Case and the Alternative Case, as follows:

$$AF_{design} = (2W_{AS}^* + 6.9\ \sigma_{AS}^*) / (2W_{BC}^* + 6.9\ \sigma_{BC}^*)$$

$$AF_{design} \geq 1 \qquad (3.6.1.3)$$

W^* and σ^* are the mean and standard deviation of the top 20 percent of the TWH. Subscripts $_{BS}$ and $_{AS}$ respectively refer to the Base Case and Alternative Scenario.

3. Identify a new design vehicle load model that can cover the most severe truck loads under the Alternative Scenario. This model can be the practical maximum truck loads under the Alternative Scenario, and it may include multiple vehicles to envelope maximum moment effects due to new legal and permit vehicles.

4. Identify all bridges to be impacted (to be constructed), for the next PP years. The total number of bridges is M. If identifying all future bridges is not possible, find the new bridges in the immediately past Q years.

5. For Bridge m=1, use the following procedure to find the cost for the bridge.

   a) Find design load change factor DLCF as follows:

$$DLCF = (M_{AS,\ design\ vehicle} / M_{BC,\ design\ vehicle})\ AF_{design} \qquad (3.6.1.1)$$

$$M_{AS,\ design\ vehicle} / M_{BC,\ design\ vehicle} \geq 1 \qquad AF_{design} \geq 1 \qquad (3.6.1.2)$$

   where $M_{AS,\ design\ vehicle} / M_{BC,\ design\ vehicle}$ is the ratio of the maximum moments due to the design vehicle under the Base Case and the same under the Alternative Scenario, for the critical section. Detailed span information should be used to calculate these maximum moments for the critical section. Practically, the ratio should not be lower than 1. AF_{design} is the ratio between the live load factors under the Base Case and the Alternative Scenario. W^* and σ^* are the mean and standard deviation of the top 20 percent of the TWH.

   b)Based on DLCF, estimate the incremental cost from the Base Case to the Alternative Scenario. Use jurisdiction specific cost data.

6. Repeat Step 5 for Bridge m=m+1, until m=M

7. Sum all costs. If the new bridges in the past Q years are used, find the average annual new bridge cost and then multiply it by PP years.

The following parameters may need to be examined for their effects on the final result in the sensitivity analysis:

1) The window parameters defined in Fig. 3.1 and the percentage increase parameter for exogenous shift in Eq. 3.2.2.8 for the TWH prediction method for the Alternative Scenario.
2) The bridges identified as expected new bridges. More years of new bridges constructed in the network may be included and averaged to an annual cost for this category of cost impact.
3) Possible increase of available resources that will result in more new bridges to be built. This may be covered at the network level by a growth factor to the total costs obtained.

A-5 Guidelines and Default Data

A-5.1 Supplementary Guidelines and Methods

A-5.1.1 A Method for Predicting Changes in Truck Weight Histograms (TWHs)

1. Acquire WIM data or VMT data appropriate for the site of interest. A sample VMT data from the default set is given in Data Set A-5.2.1 in this appendix. For WIM data, organize the data into the format of Data Set A-5.2.1, using the vehicle type definition given there. Then normalize the data by making the sum of all frequencies equal to one. For FHWA VMT data, normalize the traffic amounts to frequencies by dividing each value by the sum of all the values. This process produces a TWH with the truck types identified. (The comprehensive TWH for the Base Case can be generated by adding all traffic amounts at the same weight over all truck types. Then normalize the result , making the sum of the frequencies equal to unity. This TWH is to be used for steel fatigue assessment.)

2. According to the considered Alternative Scenario, identify the truck type(s) that will be expected to change operation behavior, i.e., those that will be subject to shifting. Identify whether exogenous shifting is expected. If yes, estimate the percentage increase parameter r_{GVWk} in Eq.3.2.2.8 to quantify the change.

3. For each shift, identify the type(s) of trucks for which pay-load will increase (shift to), and the type(s) of trucks pay-load will decrease (shift away). Perform the shifting according to Eq. 3.2.2.2. Calculate the traffic amount change as a result of this shifting. Perform also exogenous shifting if needed, according to Eq. 3.2.2.8.

4. Generate the comprehensive TWH for the Alternative Scenario using the result of shifting, including all truck types. (The comprehensive TWH is generated by summing the frequencies at the same truck weight over all truck types, with their relative traffic amounts taken into account.)

A-5.1.2
A Method for Developing Truck Wheel Weight Histograms (WWHs) Based on TWHs

1. Identify the TWH to be used, which can be a result from the procedure in Section A-5.1.1 or from another data source. The TWH needs to have the truck type identified, as shown in Data Set A-5.2.1 of this appendix.

2. For each wheel of the truck type, for each GVW interval in the TWH, compute the wheel's mean weight according to the regression relation as follows.

   Mean Wheel Weight = 0.5 Mean Axle Weight = 0.5(e + f GVW)

   where e and f are given in Data Set A-5.2.2 of this appendix if no more site specific or jurisdiction specific data are available.

3. For a GVW interval of the truck type, distribute the traffic at that GVW interval (i.e., the frequency of that interval) among a number of (10 to 20) wheel weight intervals, according to the following truncated skewed double exponential probability density function $f'_X(x,\lambda)$.

$$f_X^{'}(x,\lambda) = f_X(x,\lambda)/A \qquad where \quad \lambda > 0 \tag{3.2.5.2}$$

X is the residual wheel weight to be added to the mean wheel weight. λ is its skew factor, set at 0.1. A is the area of the skewed double exponential probability density function $f_X(x,\lambda)$ after truncation eliminating the area for $x > x_0$. x_0 represents the maximum wheel weight on bridges, set at 18 kips. The skewed double exponential probability density function $f_X(x,\lambda)$ is defined as follows

$$f_X(x,\lambda) = 2F_X(\lambda x)\, f_X(x) \qquad where \quad \lambda > 0 \tag{3.2.5.3}$$

and

$$f_X(x) = \frac{1}{2\beta}\exp\left[-|x-\mu| \big/ \beta\right] \tag{3.2.5.4}$$

$$F_X(x) = \begin{cases} \dfrac{1}{2}\exp\left[(x-\mu) \big/ \beta\right] & (x-\mu) < 0 \\[2ex] 1-\dfrac{1}{2}\exp\left[-(x-\mu) \big/ \beta\right] & (x-\mu) \geq 0 \end{cases} \tag{3.2.5.5}$$

where μ is the mean value equal to zero and 0.707 β is the standard deviation. β is set at 1.25 kips.

4. Go back to Step 3, until all GVW intervals have been treated as described.

5. Go back to Step 2, until all wheels of the truck type have been treated.

6. Sum all WWHs from Step 5 to one WWH for a truck type, then divide it by the number of axles for that truck type.

7. Go back to Step 1 to repeat for another truck type, until all interested truck types are treated. Sum all WWHs for all truck types to one grand WWH, with a weight coefficient for each type's WWH according to its relative traffic amount compared with the total traffic amount.

A-5.1.3 Guidelines for Identifying Possibly Vulnerable Steel Bridges for Fatigue Assessment

The following guidelines utilize the data commonly available in an agency's inventory database to identify bridges possibly having fatigue-prone details of Category E or E'. The quantitative discriminators, such as span length, web depth and year of construction, are recommended and selected by experienced bridge engineers, including those on AASHTO Committee T14, and should identify the majority of possibly vulnerable bridges with the critical details.

A-5.1.3.1 Common Critical Fatigue-Prone Details

END WELDS OF PARTIAL-LENGTH COVERPLATES

This critical fatigue-prone detail is most commonly found on rolled-beam bridges constructed prior to about 1975. In the mid-1970s, the poor fatigue resistance of the end welds of coverplates was well documented. As such, from about 1975 forward, welded coverplates have fallen out of favor. Coverplates are not cost-effective on built-up plate girders as the designer can merely specify a thicker or wider flange with no additional welding required. Such is not the case for rolled beams with fixed dimensions.

TERMINATION OF LONGITUDINAL WEB STIFFENERS

This critical fatigue-prone detail can be found on plate-girder bridges with web depths greater than 60 in., thus spans perhaps longer than 130 ft. Longitudinal stiffeners are used to increase the shear resistance of deeper plate-girder webs. Girder webs under about 60 in. in depth would not be longitudinally stiffened. (Note that 60 in. was arbitrarily chosen as a minimum for longitudinally stiffened webs. Good practice would suggest an even greater minimum depth, but perhaps designs exist of 60 in. in depth with longitudinal stiffeners.)

LONGITUDINAL CONNECTION PLATES

This critical fatigue-prone detail is most commonly found on girder bridges of greater than about 150-ft span length, constructed prior to 1980. The most common longitudinal connection plate is that used to connect lateral bracing systems to girders. Such bracing systems were only used on longer girder bridges. During the 1970s, the costly practice of providing lateral-bracing systems on girder bridges was slowly discontinued.

A-5.1.3.2 Application of the Above Guidelines

The above guidelines are recommended to be used to screen bridges in a network for a Level I analysis. This level of detail- and data-requirement allows the bridges to be grouped according to their characteristics. On the other hand, if a Level II analysis is performed, every bridge in the network should be examined individually to identify fatigue prone details for further detailed analysis. Accordingly the above set of guidelines will not be needed since every bridge will be subject to an examination using the design drawings. Agencies having an inventory of E and E' details can advantageously use the available data for estimating fatigue accumulation due to heavy trucks.

A-5.1.3.3 Other Details of Low Fatigue Strength

Besides the above critical fatigue-prone details, several common details of low fatigue strength are identified below, for agencies that would like to include them in their application of the recommended methodology. Including these types of details here is to provide a wide coverage of this issue of steel fatigue. It does not indicate that these types of details would necessarily contribute to the total cost impact significantly.

TRANSVERSE CONNECTION PLATES WELDED TO THE TENSION FLANGE

This critical fatigue-prone detail is most commonly found on girder bridges with transverse diaphragms or floor beams, constructed prior to about 1985. In the mid-1980s, due to a preponderance of distortion-induced fatigue cracking of web gaps between cut-short transverse connection plates and the flanges, the AASHTO specifications first required that transverse connection plates be rigidly attached to both flanges, most easily through welding. Transverse connection plates are used on girder bridges to connect transverse members such as diaphragms and floor beams to the main longitudinal girders.

TRANSVERSE STIFFENERS

This critical fatigue-prone detail is found on plate-girder bridges. Transverse stiffeners are used to increase the shear resistance of plate-girder webs. Whether a plate girder is unstiffened (with no transverse stiffeners), fully stiffened (with transverse stiffeners along its entire length), or partially stiffened (with transverse stiffeners only along apart of its length where shear is more critical) is a designer prerogative. As such, more conclusions about their presence in a particular plate girder cannot be drawn.

RIVETED TRUSS MEMBERS

This critical fatigue-prone detail is mostly commonly found on truss bridges constructed prior to about *1965*. Riveted construction of truss members was replaced by welded construction (and, perhaps, in the case of fracture-critical truss members, by bolted construction) during the 1960s. At the same time, riveted connections of trusses were replaced with field-bolted connections. As such truss bridges constructed after about *1965*, are mostly likely not of riveted construction.

RIVETED BUILT-UP GIRDERS

This critical fatigue-prone detail is mostly commonly found on built-up girder bridges constructed prior to about *1965*. The reasoning for this conclusion is similar to that discussed above for riveted truss bridges.

SECONDARY BENDING

Secondary bending results from partial fixity at beam or truss joints that are assumed pinned, or distortions of various members of the bridge, especially bracing members (Moses et al. 1987). Developing general screening guidelines for this type of fatigue prone details is much more difficult, mainly because the stress range at these details is very much dependent on the local arrangement and possibly construction quality.

It is understood that a significant percentage of fatigue failure observed in the field belongs to this category. Thus, it is recommended that the agency identify the characteristics of possibly vulnerable details based on its past experience (e.g., year built, framing type, span length, geographical location in the jurisdiction, etc.). Validation of these vulnerability characteristics can be performed by randomly selecting a sample of bridges satisfying these discriminators and confirming the existence of the focused detail type(s). The confirmed discriminators can then be used for the entire network, with detailed analysis methods developed respectively for the vulnerable details.

A-5.1.4 Guidelines for Identifying Possibly Vulnerable RC Decks for Fatigue Assessment

The recommended methodology targets at RC decks on beams or girders, not thick slabs without beams. Typically the targeted decks have a thickness from 0.114 m (4.5 in.) to 0.241 m (9.5 in.). The spacing of the supporting beams ranges from 1.03 m (6 ft) to 3.66 m (12 ft). These may be used as discriminators to identify the vulnerable bridges using the agency's bridge inventory. If the NBI is used, beam bridges with steel, reinforced concrete, or prestressed concrete superstructure should be included in the vulnerable bridge population.

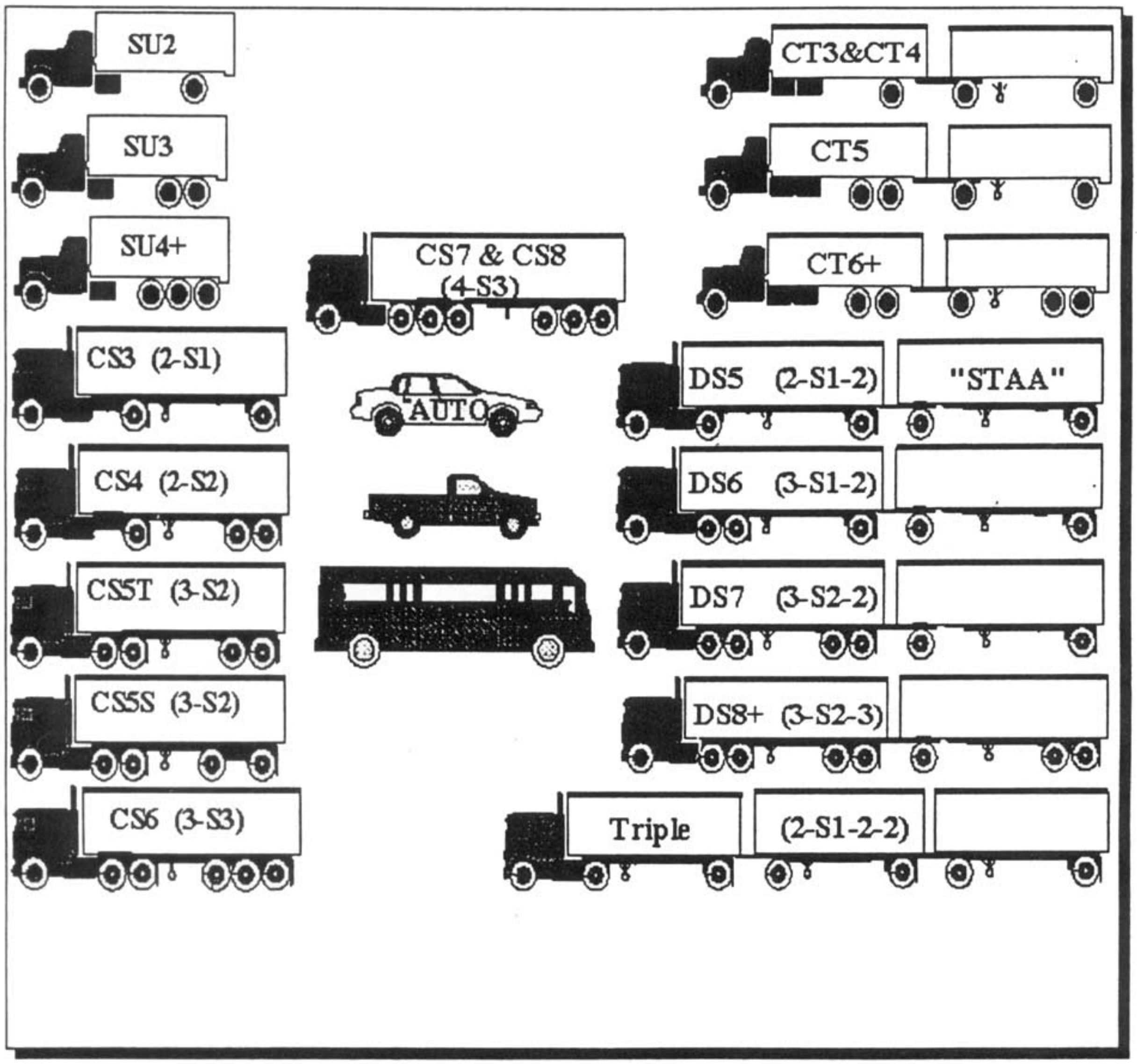
A-5.2 Default Data

Data Set A-5.2.1 Sample VMT Data for Year 2000 for the Base Case TWH - FHWA

Sample FHWA VMT Data for Year 2000

state	fc	wg	auto	LT4	SU2	SU3	SU4	CS3	CS4	3S2T	3S2S	CS6	CS7	CT4	CT5	CT6	DS5	DS6	DS7	DS8	TRP	BUS
24	UI	5	4886.795	481.5231	2.881357	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	UI	10	437.1038	736.6746	59.54172	0	0	0.011368	0	0	0	0	0	0.009431	0	0	0	0	0	0	0	0
24	UI	15	0	16.97135	27.92123	0.858155	0	0.060627	0.324145	0	0	0	0	0.126096	0.014536	0	0	0	0	0	0	0.549433
24	UI	20	0	0	29.13882	3.939409	0.083398	0.70858	0.53664	0.360085	0.107585	0	0	0.171155	0.174428	0.010113	0.002004	0.001245	0	0	0	1.630919
24	UI	25	0	0	13.1055	3.905623	0.382844	0.769207	1.062476	1.564827	0.131493	0.075862	0	0.230885	0.145356	0.016062	0.010017	0.004981	0	0	0	1.772622
24	UI	30	0	0	4.182605	3.993467	0.379561	0.557012	2.377066	4.401096	0.203216	1.289651	0.023984	0.231932	0.272543	0.02558	0.066113	0.01619	0	0	0	2.010576
24	UI	35	0	0	0.929471	2.985551	0.388098	0.299347	3.716866	20.42734	0.76505	1.441375	0.03769	0.185825	0.359757	0.0464	0.096164	0.039851	0	0	0	2.106827
24	UI	40	0	0	0.130125	2.013349	0.27795	0.181881	2.755235	29.02948	0.669419	1.820683	0.058248	0.125048	0.385195	0.051754	0.179306	0.039228	0	0	0	1.319441
24	UI	45	0	0	0.018583	1.337637	0.204375	0.162935	1.923262	19.42264	0.418387	1.972407	0.092511	0.063223	0.410632	0.064842	0.203347	0.077834	0	0	0	0.743271
24	UI	50	0	0	0.009292	0.703293	0.188025	0.04926	1.202939	16.2574	0.27494	1.744822	0.078806	0.034231	0.287079	0.048185	0.225384	0.068494	0	0	0	0.287416
24	UI	55	0	0	0	0.358541	0.188025	0.026524	0.605072	14.80371	0.203216	1.441375	0.116496	0.017465	0.31615	0.04997	0.321549	0.070362	0	0	0	0.128335
24	UI	60	0	0	0	0.15169	0.2289	0.003789	0.273722	13.38558	0.262986	1.517237	0.119922	0.014321	0.305248	0.046995	0.455777	0.070362	0	0	0	0
24	UI	65	0	0	0	0.089636	0.188025	0.003789	0.100845	14.12798	0.310801	1.137928	0.123348	0.011527	0.323418	0.049375	0.442755	0.056041	0	0	0	0
24	UI	70	0	0	0	0	0.171675	0	0.039618	19.15146	0.729188	2.351715	0.181596	0.002445	0.359757	0.021416	0.383654	0.066626	0	0	0	0
24	UI	75	0	0	0	0.013789	0.11445	0	0.010805	20.67183	0.848728	1.820683	0.215859	0.001397	0.294347	0.02439	0.24041	0.075966	0	0	0	0
24	UI	80	0	0	0	0	0.057225	0	0	12.63428	0.824819	1.137928	0.219286	0	0.141722	0.017251	0.114195	0.050437	0	0	0	0
24	UI	85	0	0	0	0	0.024525	0	0	3.992105	0.21517	0.606894	0.137054	0	0.156258	0.008328	0.036062	0.031134	0	0	0	0
24	UI	90	0	0	0	0	0.01635	0	0	0.898003	0.095631	0.531032	0.123348	0	0.145356	0.010708	0.00601	0.015567	0	0	0	0
24	UI	95	0	0	0	0	0.008175	0	0	0.177813	0.035862	0.606894	0.137054	0	0.14899	0.002974	0.005009	0.006227	0	0	0	0
24	UI	100	0	0	0	0	0	0	0	0.026673	0.023908	0.606894	0.14048	0	0.058143	0.004164	0.003005	0.003736	0	0	0	0
24	UI	105	0	0	0	0	0	0	0	0.008897	0	0.303447	0.161038	0	0.029071	0.001785	0	0.001245	0	0	0	0
24	UI	110	0	0	0	0	0	0	0	0	0	0.075862	0.130201	0	0.003634	0.001785	0	0.002491	0	0	0	0
24	UI	115	0	0	0	0	0	0	0	0	0	0.151723	0.157612	0	0	0.00119	0	0	0	0	0	0
24	UI	120	0	0	0	0	0	0	0	0	0	0	0.061674	0	0	0.00238	0	0.000623	0	0	0	0
24	UI	125	0	0	0	0	0	0	0	0	0	0	0.047969	0	0	0.000595	0	0	0	0	0	0
24	UI	130	0	0	0	0	0	0	0	0	0	0.075862	0.051395	0	0	0	0	0	0	0	0	0
24	UI	135	0	0	0	0	0	0	0	0	0	0	0.027411	0	0	0	0	0	0	0	0	0
24	UI	140	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	UI	145	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24	UI	150	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

State - Numerical code for State: 24 = Minnesata
fc - Highway functional class: UI = Urban Interstate
wg - Weight group (in kips)
VMT - in million vehicle miles
Vehicle class - See definitions on next page.



Vehicle Class Definitions and Configurations

Data Set A-5.2.2 Regression Relations of Mean Axle Weights and Truck Weight – NCHRP 1251

Coefficients for Estimating Mean Axle Weight (Mean Axle Weight in kips = e + f * GVW in kips)

Vehicle Class	Vehicle	Number of observations	Axle Number	Coefficient e	Standard Error	Coefficient f	Standard Error	R-Square
3	2 Axle Buses	4,460	1	1.319	0.083	0.335	0.003	0.718
			2	-1.319	0.083	0.665	0.003	0.909
	3 Axle Bus	14,898	1	2.059	0.039	0.272	0.001	0.796
			2	6.400	0.076	0.314	0.002	0.580
			3	-8.459	0.082	0.413	0.002	0.670
4	2 Axle Single Unit	23,504	1	0.994	0.020	0.335	0.001	0.737
			2	-0.994	0.020	0.665	0.001	0.917
5	3 Axle Single Unit	20,172	1	5.480	0.035	0.160	0.001	0.441
			2	-2.703	0.026	0.431	0.001	0.914
			3	-2.778	0.025	0.409	0.001	0.914
	4 Axle Single Unit	627	1	3.699	0.310	0.219	0.005	0.725
			2	-1.758	0.279	0.305	0.005	0.865
			3	-1.392	0.029	0.291	0.005	0.839
			4	-0.549	0.359	0.185	0.006	0.585
6	3 Axle Tractor Semitrailer	26,717	1	3.545	0.028	0.147	0.001	0.396
			2	1.073	0.037	0.388	0.001	0.729
			3	-4.618	0.036	0.464	0.001	0.801
7	4 Axle Tractor Semitrailer	26,579	1	3.678	0.025	0.126	0.001	0.506
			2	2.782	0.042	0.261	0.001	0.598
			3	-2.838	0.026	0.297	0.001	0.838
			4	-3.621	0.030	0.315	0.001	0.812
8	Spread Tandem 5 Axle Tractor Semitrailer	18,021	1	8.093	0.032	0.029	0.001	0.134
			2	0.879	0.032	0.196	0.001	0.869
			3	0.069	0.028	0.202	0.000	0.901
			4	-4.431	0.033	0.286	0.001	0.930
			5	-4.610	0.033	0.287	0.001	0.933
	6 Axle Tractor Semitrailer	4,033	1	7.684	0.066	0.032	0.001	0.166
			2	-0.357	0.071	0.201	0.001	0.869
			3	-0.829	0.070	0.202	0.001	0.873
			4	-2.977	0.102	0.201	0.002	0.762
			5	-2.423	0.069	0.200	0.001	0.872
			6	-1.099	0.123	0.164	0.002	0.594
9	Conventional 5 Axle Tractor Semitrailer	25,335	1	7.603	0.024	0.041	0.000	0.246
			2	-0.132	0.025	0.222	0.000	0.905
			3	-0.534	0.024	0.219	0.000	0.907
			4	-3.603	0.028	0.259	0.001	0.911
			5	-3.334	0.029	0.258	0.001	0.901
10	5 Axle Truck-Tractor	24,657	1	7.093	0.018	0.047	0.000	0.455
			2	-0.122	0.019	0.214	0.000	0.944
			3	-0.547	0.019	0.207	0.000	0.938
			4	-2.897	0.022	0.262	0.000	0.949
			5	-3.527	0.022	0.269	0.000	0.952

Vehicle Class	Vehicle	Number of observations	Axle Number	Coefficient e	Standard Error	Coefficient f	Standard Error	R-Square
11	5 Axle Double	24,884	1	7.283	0.019	0.020	0.000	0.122
			2	1.408	0.025	0.223	0.000	0.909
			3	-2.579	0.024	0.255	0.000	0.934
			4	-2.734	0.024	0.247	0.000	0.925
			5	-3.377	0.027	0.254	0.000	0.916
12	6 Axle Double	19,859	1	7.962	0.036	0.015	0.001	0.027
			2	1.038	0.030	0.136	0.001	0.751
			3	0.323	0.031	0.141	0.001	0.753
			4	-3.013	0.038	0.244	0.001	0.859
			5	-3.175	0.040	0.239	0.001	0.843
			6	-3.134	0.041	0.224	0.001	0.812
	7 Axle Double	991	1	7.329	0.128	0.025	0.002	0.140
			2	1.478	0.235	0.149	0.004	0.631
			3	-0.051	0.250	0.158	0.004	0.629
			4	-2.241	0.304	0.173	0.005	0.579
			5	-0.964	0.256	0.167	0.004	0.647
			6	-2.239	0.283	0.166	0.004	0.596
			7	-3.312	0.298	0.161	0.005	0.555
	8 Axle Double	269	1	7.372	0.196	0.021	0.002	0.245
			2	1.554	0.377	0.113	0.004	0.708
			3	0.786	0.397	0.116	0.005	0.700
			4	-2.258	0.426	0.151	0.005	0.773
			5	1.693	0.468	0.117	0.005	0.628
			6	-1.946	0.440	0.154	0.005	0.767
			7	-2.853	0.388	0.162	0.004	0.824
			8	-4.348	0.373	0.166	0.004	0.842
	9 Axle Double	527	1	6.999	0.167	0.023	0.002	0.264
			2	-0.455	0.257	0.120	0.003	0.802
			3	-0.844	0.263	0.122	0.003	0.800
			4	0.457	0.317	0.118	0.003	0.724
			5	0.659	0.329	0.116	0.003	0.701
			6	-1.381	0.289	0.122	0.003	0.770
			7	-1.644	0.310	0.126	0.003	0.756
			8	-2.031	0.241	0.127	0.002	0.839
			9	-1.760	0.259	0.124	0.003	0.812

Data Set A-5.2.3 Truck-Weight-Limit Enforcement Costs – (Minnesota DOT, 1991)

For estimating weight enforcement costs, the following data may be used, as appropriate.

Annual cost per enforcement crew: $116,400 Y2000 dollars

Data Set A-5.2.4 Steel Fatigue Repair Costs – NCHRP 1251

NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs

Level I Steel Fatigue Costs

A set of Level I fatigue costs for the details identified in Appendix B has been calculated. A typical member for each detail was identified and an appropriate repair was sized and detailed. A sketch of each detail is given. Each repair was broken into a group of cost items. The cost items with units and unit costs are shown on the next page. A cost sheet is included for each repair.

The repairs are divisible into flange repairs and web repairs (the axial truss member is similar to the flange repairs). Repair sizing was based on different requirements for flanges and webs. A crack in a flange requires a splice capable of replacing the flange at that section. A crack in a web requires a splice covering a somewhat greater depth of the web than the crack capable of replacing the cracked area.

A further distinction between the flange and web repairs concerns variability. The details are intended to be typical and represent an average member. The web repairs are expected to be representative. The flange details are more variable and a Cost Impact Factor table based upon the span length has been provided with each. The tabulated values are an estimate of the changes in repair detail sized based on increased flexural moment with increasing length.

NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs

A lump sum cost of $1000 per repair will be assessed for traffic control and access to the repair location

Item	Unit	Unit cost
Structural Steel[1]	LB	$4.00
Cleaning and Painting[2]	SF	$25.00
Field Drill Holes at Crack Tip	EA	$25.00
Remove Rivets Replace with H.S. Bolts	EA	$50.00
Cut Transverse Connection Plate	EA	$100.00
Traffic Control / Access[3]	LS	$1,000.00

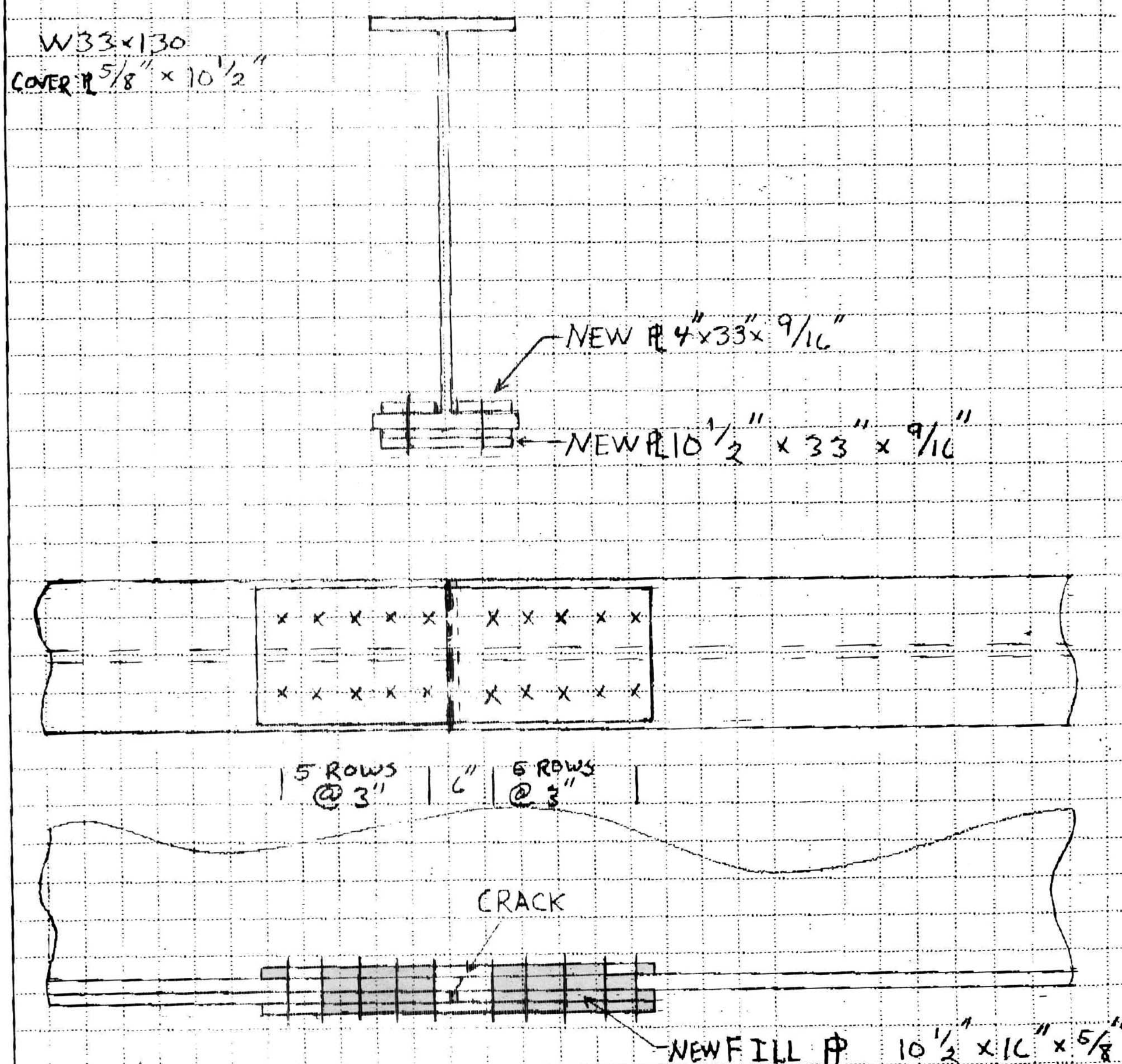
1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.
2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.
3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

Lichtenstein

BRIDGES HIGHWAYS RAILROADS SITE ENGINEERING DAMS

JOB 2035
SHEET NO. 1 OF
CALCULATED BY WE DATE 8/1979
CHECKED BY DATE
SCALE

FATIGUE DETAIL / REPAIR B1
WELDS OF PARTIAL LENGTH COVERPLATES
BASE DETAIL FOR ESTIMATE



NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs
Level I Estimate

Detail B1
Welds of Parial-Length Coverplates
Section based on MCE-LRFR example A1

Item	Quantity	Unit	Unit Cost	Cost
Structural Steel[1]	150	LB	$4.00	$600.00
Cleaning and Painting[2]	6	SF	$25.00	$150.00
Field Drill Holes at Crack Tip	1	EA	$25.00	$25.00

Subtotal: $775.00

Traffic Control / Access (TCA)[3]	1	LS	$1,000.00	$1,000.00

Total Repair Cost: $1,775

Repair Cost = Subtotal x CIF + TCA

Cost Impact Factor (CIF) Applied to the Subtotal Cost

Span	CIF	Total Repair Cost
≤ 50 FT	0.71	$1,551
65 FT	1.00	$1,775
100 FT	1.64	$2,271

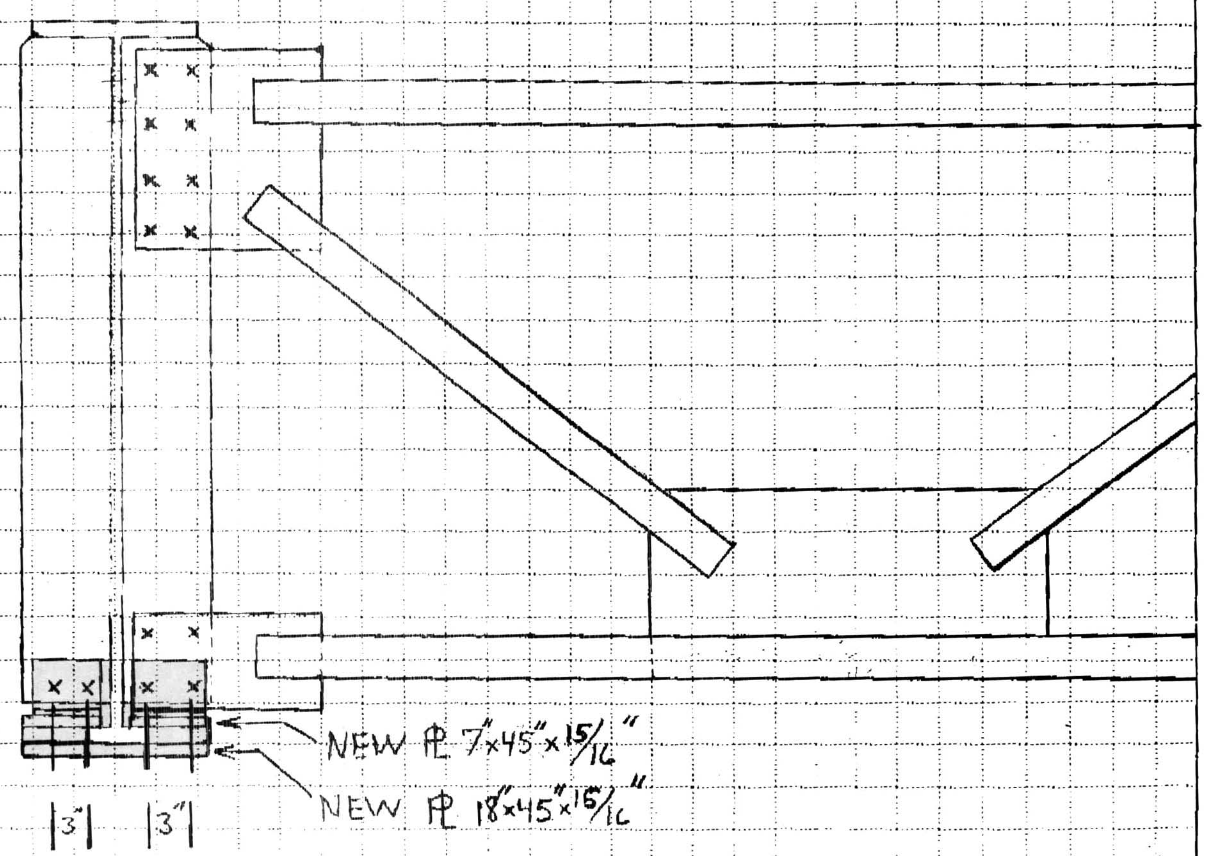
Notes:
1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.
2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.
3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

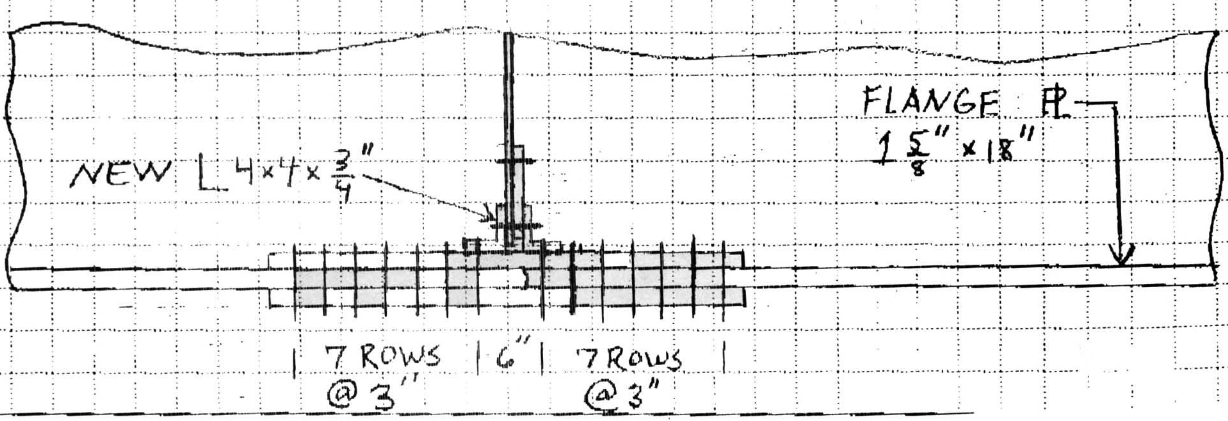
Lichtenstein

BRIDGES HIGHWAYS RAILROADS SITE ENGINEERING DAMS

JOB 2035
SHEET NO. 2 OF
CALCULATED BY WE DATE 8/1999
CHECKED BY DATE
SCALE

FATIGUE DETAIL / REPAIR B2
TRANSVERSE CONNECTION PLATES WELDED
TO THE TENSION FLANGE





NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs
Level I Estimate

Detail B2

Transverse Connection Plates Welded to the Tension Flange

Section based on AISI LRFD Steel Examples, Ex.3

Item	Quantity	Unit	Unit Cost	Cost
Structural Steel[1]	480	LB	$4.00	$1,920.00
Cleaning and Painting[2]	12	SF	$25.00	$300.00
Field Drill Holes at Crack Tip	1	EA	$25.00	$25.00
Cut Transverse Connection Plate	2	EA	$100.00	$200.00

Subtotal: $2,445.00

Traffic Control / Access (TCA)[3]	1	LS	$1,000.00	$1,000.00

Total Repair Cost: $3,445

Repair Cost = Subtotal x CIF + TCA

Cost Impact Factor (CIF) Applied to the Subtotal Cost

Span	CIF	Total Repair Cost
≤ 100 FT	0.75	$2,836
120 FT	1.00	$3,445
140 FT	1.28	$4,127
160 FT	1.63	$4,974
180 FT	2.03	$5,954

Notes:

1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.

2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.

3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

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BRIDGES HIGHWAYS ... DAMS

JOB

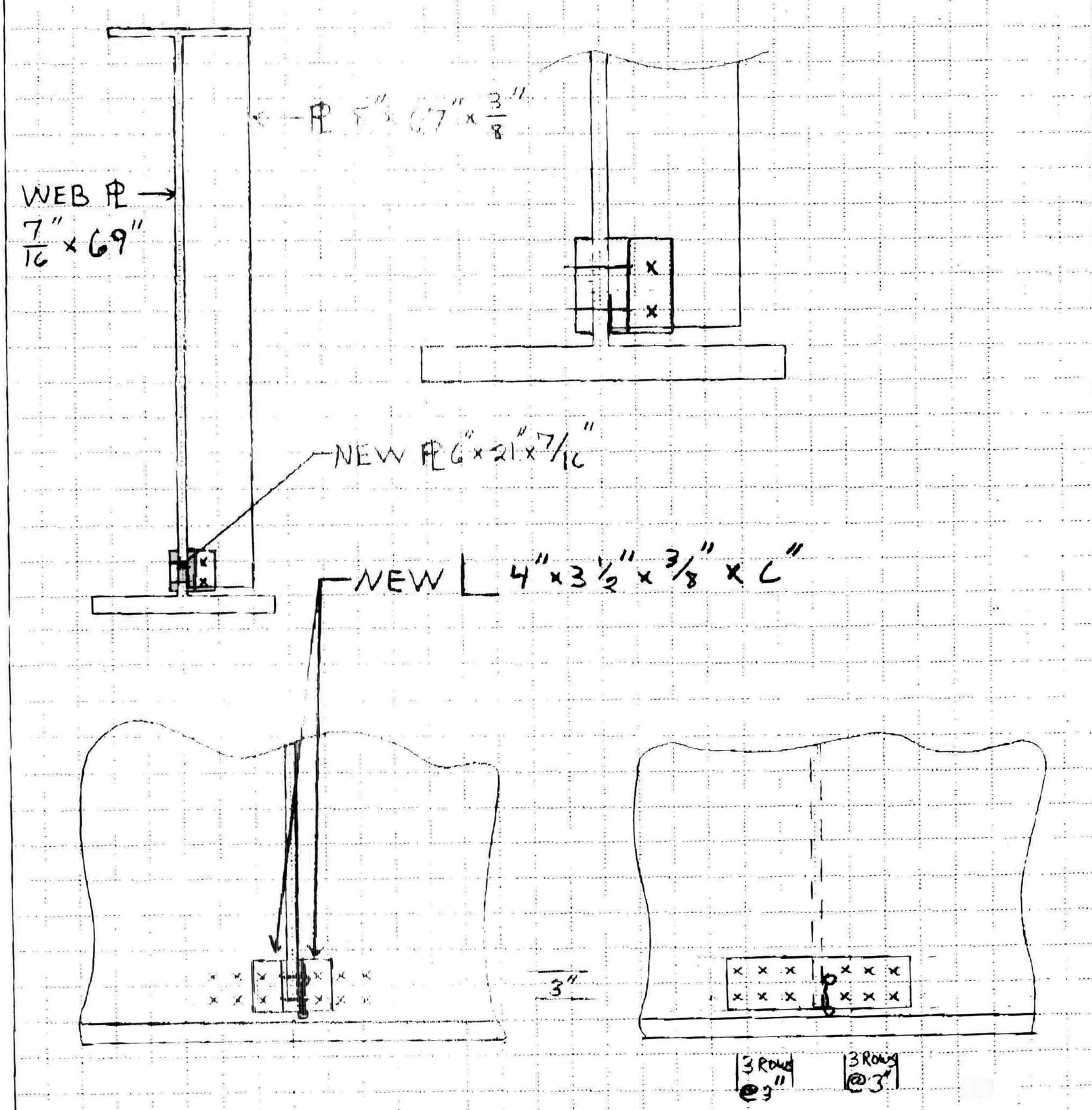
SHEET NO. 3 OF

CALCULATED BY WE DATE 8/1999

CHECKED BY DATE

SCALE

FATIGUE DETAIL / REPAIR B3
WELDED TRANSVERSE STIFFENER



NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs
Level I Estimate

Detail B3
Welded Transverse Stiffeners
Section based on AISI LRFD Steel Examples, Ex.3

Item	Quantity	Unit	Unit Cost	Cost
Structural Steel[1]	60	LB	$4.00	$240.00
Cleaning and Painting[2]	2	SF	$25.00	$50.00
Field Drill Holes at Crack Tip	2	EA	$25.00	$50.00

Subtotal: $340.00

Traffic Control / Access (TCA)[3]	1	LS	$1,000.00	$1,000.00

Total Repair Cost: $1,340

Notes:
1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.
2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.
3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

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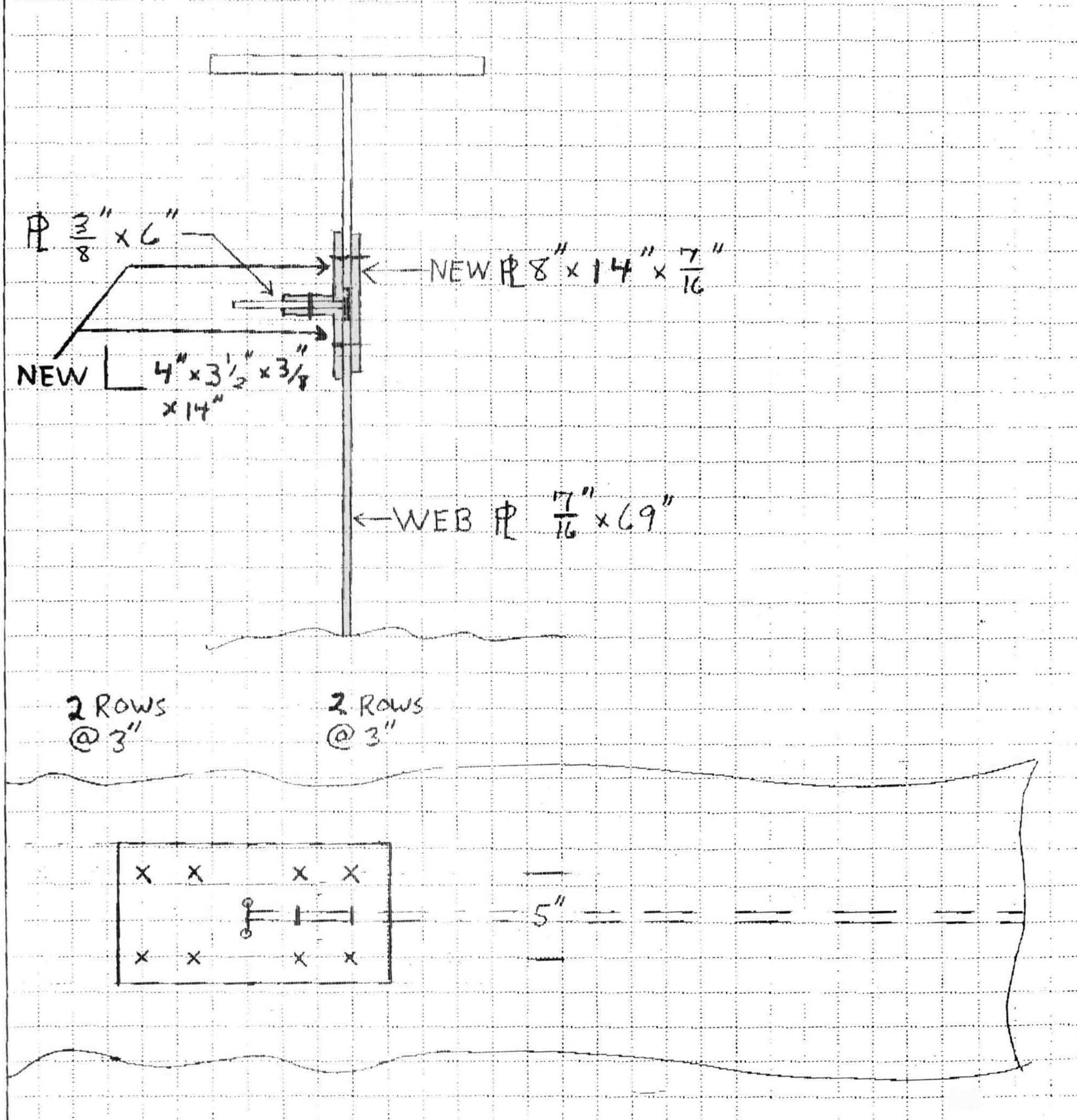
BRIDGES HIGHWAYS RAILROADS SITE ENGINEERING DAMS

JOB 2035
SHEET NO. 4 OF
CALCULATED BY WE DATE 8/1999
CHECKED BY DATE
SCALE

FATIGUE DETAIL / REPAIR B4

TERMINATION OF WELDED LONGITUDINAL WEB STIFFENERS



NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs
Level I Estimate

Detail B4
Termination of Welded Longitudinal Web Stiffeners
Section based on AISI LRFD Steel Examples, Ex.3

Item	Quantity	Unit	Unit Cost	Cost
Structural Steel[1]	65	LB	$4.00	$260.00
Cleaning and Painting[2]	2	SF	$25.00	$50.00
Field Drill Holes at Crack Tip	2	EA	$25.00	$50.00

Subtotal: $360.00

Traffic Control / Access (TCA)[3]	1	LS	$1,000.00	$1,000.00

Total Repair Cost: $1,360

Notes:
1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.
2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.
3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

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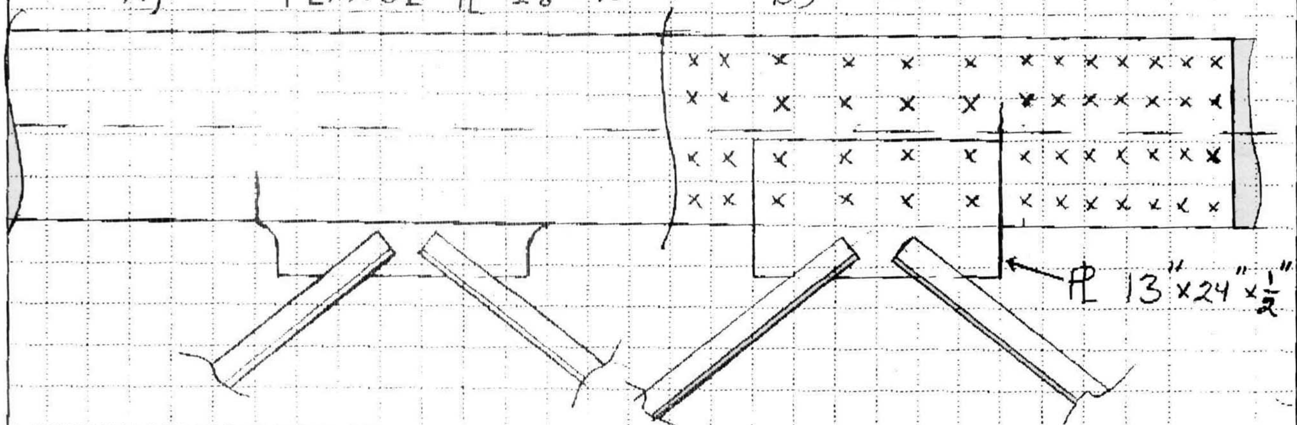
BRIDGES HIGHWAYS RAILROADS SITE ENGINEERING DAMS

JOB ______
SHEET NO. 5 OF ______
CALCULATED BY WE DATE 8/1999
CHECKED BY ______ DATE ______
SCALE ______

FATIGUE DETAIL / REPAIR B5
WELDED LONGITUDINAL CONNECTION PLATES

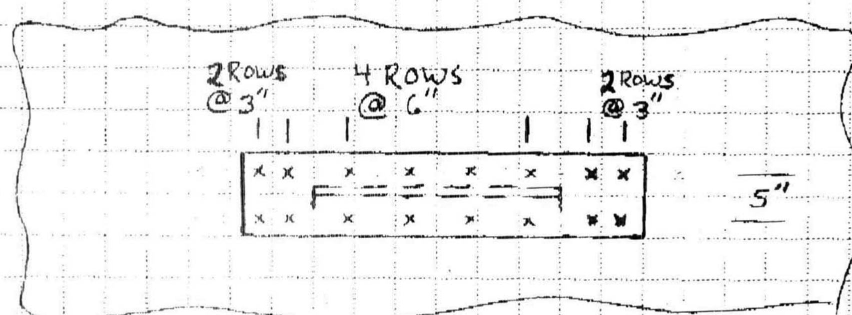
UNDERSIDE OF BOTTOM FLANGE

A) FLANGE PL $1\frac{5}{8}" \times 18"$ B)



C)

WEB PL
$\frac{7}{16}" \times 69"$



NEW PLs

A) 1: PL $18" \times 69" \times \frac{15}{16}"$
2: PL $7" \times 69" \times \frac{15}{16}"$

C) 1: PL $8" \times 39" \times \frac{7}{16}"$
2: L $4" \times 3\frac{1}{2}" \times \frac{3}{8}" \times 39"$

B) 1: PL $18" \times 69" \times \frac{15}{16}"$
2: PL $7" \times 69" \times \frac{15}{16}"$
1: PL $16" \times 69" \times \frac{1}{2}"$
2: PL $8" \times 22\frac{1}{2}" \times \frac{1}{2}"$

NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs
Level I Estimate

Detail B5a

Welded Longitudinal Connection Plate, Flange Connection

Section based on AISI LRFD Steel Examples, Ex.3

Item	Quantity	Unit	Unit Cost	Cost
Structural Steel[1]	660	LB	$4.00	$2,640.00
Cleaning and Painting[2]	29	SF	$25.00	$725.00
Field Drill Holes at Crack Tip	2	EA	$25.00	$50.00

Subtotal: $3,415.00

Traffic Control / Access (TCA)[3]	1	LS	$1,000.00	$1,000.00

Total Repair Cost: $4,415

Repair Cost = Subtotal x CIF + TCA

Cost Impact Factor (CIF) Applied to the Subtotal Cost

Span	CIF	Total Repair Cost
≤ 100 FT	0.75	$3,564
120 FT	1.00	$4,415
140 FT	1.28	$5,367
160 FT	1.63	$6,550
180 FT	2.03	$7,920

Notes:

1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.

2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.

3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs
Level I Estimate

Detail B5b
Welded Longitudinal Connection Plate, Flange Connection
Section based on AISI LRFD Steel Examples, Ex.3

Item	Quantity	Unit	Unit Cost	Cost
Structural Steel[1]	810	LB	$4.00	$3,240.00
Cleaning and Painting[2]	29	SF	$25.00	$725.00
Field Drill Holes at Crack Tip	2	EA	$25.00	$50.00

Subtotal: $4,015.00

Traffic Control / Access (TCA)[3]	1	LS	$1,000.00	$1,000.00

Total Repair Cost: $5,015

Repair Cost = Subtotal x CIF + TCA

Cost Impact Factor (CIF) Applied to the Subtotal Cost

Span	CIF	Total Repair Cost
≤ 100 FT	0.75	$4,015
120 FT	1.00	$5,015
140 FT	1.28	$6,135
160 FT	1.63	$7,525
180 FT	2.03	$9,136

Notes:

1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.

2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.

3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

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BRIDGES HIGHWAYS RAILROADS SITE ENGINEERING DAMS

JOB
SHEET NO. 6 OF
CALCULATED BY WE DATE 8/1999
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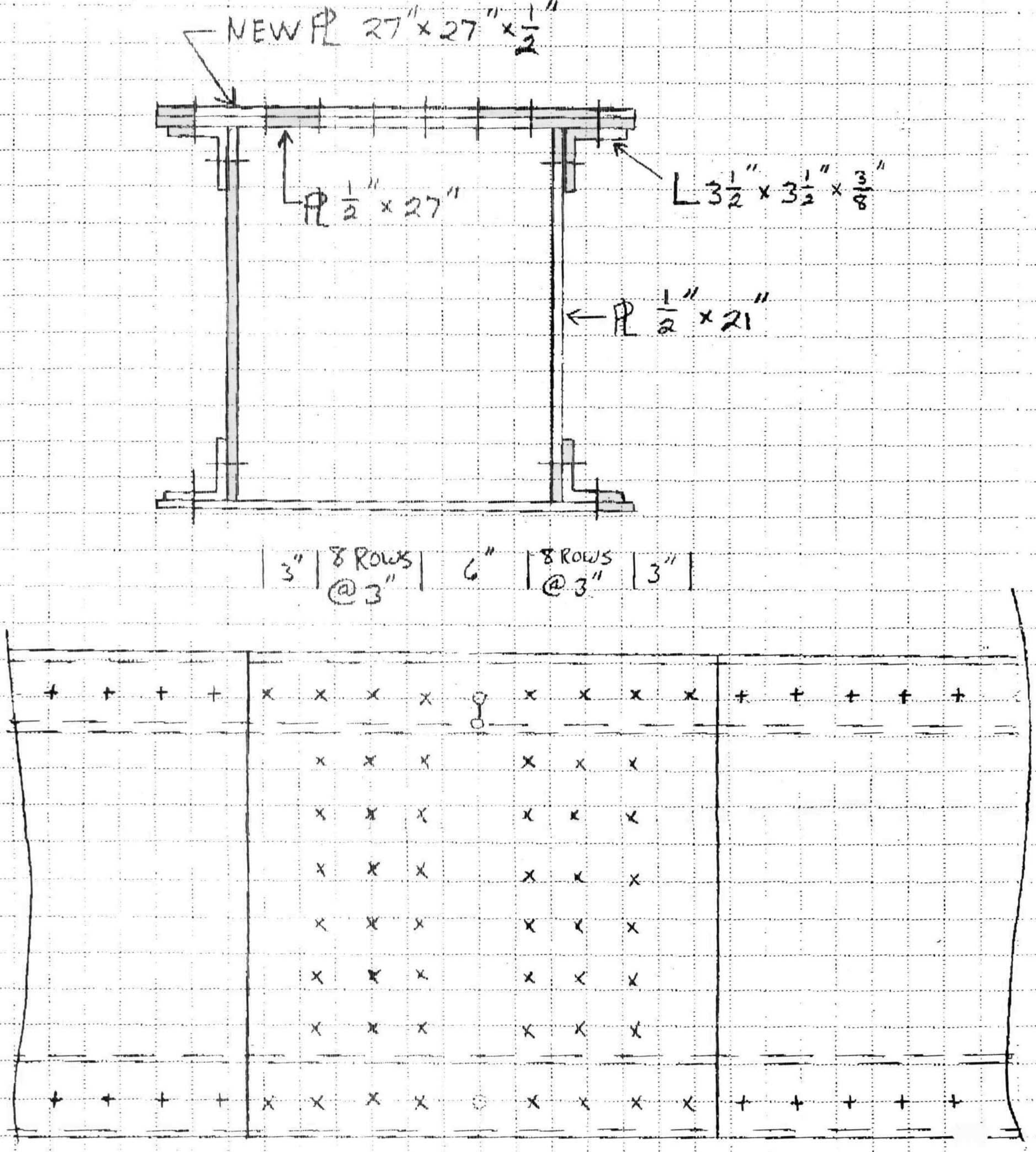
SCALE

FATIGUE DETAIL/REPAIR B6
RIVETED TRUSS MEMBERS



NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs
Level I Estimate

Detail B6
Riveted Truss Members
Section based on modified version of MCE-LRFR example A6.

Item	Quantity	Unit	Unit Cost	Cost
Structural Steel[1]	160	LB	$4.00	$640.00
Cleaning and Painting[2]	6	SF	$25.00	$150.00
Field Drill Holes at Crack Tip	1	EA	$25.00	$25.00
Remove Rivets Replace with H.S. Bolts	18	EA	$50.00	$900.00

Subtotal: $1,075.00

Traffic Control / Access (TCA)[3]	1	LS	$1,000.00	$1,000.00

Total Repair Cost: $2,075

Repair Cost = Subtotal x CIF + TCA

Cost Impact Factor (CIF) Applied to the Subtotal Cost

Span	CIF	Total Repair Cost
< 120 FT	0.75	$1,806
120 - 200 FT	1.00	$2,075
>200 FT	1.20	$2,290

Notes:
1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.
2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.
3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

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BRIDGES HIGHWAYS RAILROADS SITE ENGINEERING DAMS

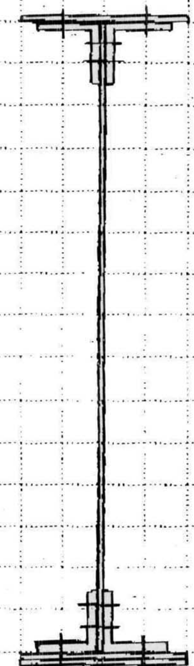
JOB ______
SHEET NO. 7 OF ______
CALCULATED BY WE DATE 8/1999
CHECKED BY ______ DATE ______
SCALE 1" = 16"

FATIGUE DETAIL / REPAIR B7 RIVETED BUILT-UP GIRDER

Floorbeam

rivets @ 6" cc in Cov. plate

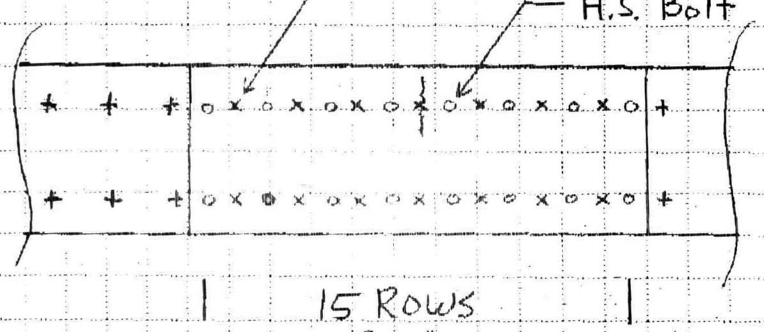
COVER PL 16" × 7/16"
L 6" × 6" × 3/4"
WEB PL 60½" × 3/8"



RIVET REMOVED
REPLACED w/ H.S. BOLT

NEW PL 16" × 45" × 7/16"

H.S. Bolt



15 ROWS
@ 3"

NCHRP 12-51
Effect of Truck Weight on Bridge Network Costs
Level I Estimate

Detail B7

Riveted Built-Up Girders

Section based on MCE-LRFR example A6.

Item	Quantity	Unit	Unit Cost	Cost
Structural Steel[1]	120	LB	$4.00	$480.00
Cleaning and Painting[2]	6	SF	$25.00	$150.00
Field Drill Holes at Crack Tip	2	EA	$25.00	$50.00
Remove Rivets Replace with H.S. Bolts	14	EA	$50.00	$700.00

Subtotal: $1,380.00

Traffic Control / Access (TCA)[3]	1	LS	$1,000.00	$1,000.00

Total Repair Cost: $2,380

Repair Cost = Subtotal x CIF + TCA

Cost Impact Factor (CIF) Applied to the Subtotal Cost

Span	CIF	Total Repair Cost
≤ 65 FT	0.51	$1,709
100 FT	1.00	$2,380
120 FT	1.33	$2,838
140 FT	1.70	$3,351
160 FT	2.16	$3,987
180 FT	2.70	$4,724

Notes:

1 - Includes cost for new high strength bolts in new holes.
Weight of bolts has been added to weight of structural steel.

2 - Includes cleaning, priming, and painting of existing steel repair areas only.
Painting of new steel included in cost of structural steel.

3 - A nominal cost has been added. Cost is highly variable depending on site conditions.

Data Set A-5.2.5 RC Deck Concrete Overlay Costs – NCHRP 1251

Data Set A-5.2.5 RC Deck Concrete Overlay Costs - NCHRP 1251

Cost $ per sq ft	State	Cost $ per sq ft	State
$23	Arizona	$28	Missouri
$28	California	$22	Oklahoma
$20	Idaho	$48	New York
		$41	Rhode Island

For states not listed above, the following cost indices may be used to estimate the cost. For example, for New Mexico, use the unit cost $23 for Arizona and multiply it by a coefficient of 95/95.1 = .9989 to find the unit cost of $22.98 for New Mexico. (New Mexico index = 95.0 and Arizona index = 95.1)

Cost Indices for Concrete Construction Averaged Based on R.S.Means City Cost Indices			
Cost Index	State	Cost Index	State
85.5	Alabama	95.1	Missouri
141.9	Alaska	93.6	Montana
95.1	Arizona	86.9	Nebraska
86.4	Arkansas	105.2	Nevada
113.6	California	90.6	New Hampshire
94.8	Colorado	107.3	New Jersey
99.1	Connecticut	95.0	New Mexico
106.7	Delaware	85.6	North Carolina
93.8	District of Columbia	105.5	Now York
88.0	Florida	99.2	Ohio
87.1	Georgia	86.5	Oklahoma
117.7	Hawaii	107.5	Oregon
97.7	Idaho	102.2	Pennsylvania
98.1	Illinois	105.2	Rhode Island
97.9	Indiana	80.9	South Carolina
94.0	Iowa	80.6	South Dakota
87.0	Kansas	83.1	Tennessee
88.5	Kentucky	87.6	Texas
83.7	Louisiana	89.6	Utah
92.8	Maine	87.1	Virginia
101.0	Maryland	94.1	Vermont
110.9	Massachusetts	101.9	Washington
99.2	Michigan	103.6	West Virginia
97.6	Minnesota	96.8	Wisconsin
82.9	Mississippi	87.5	Wyoming

Data Set A-5.2.6 General New Bridge Costs – FHWA

Data Set A-5.2.6 General New Broidge Costs - FHWA				
Bridge Construction Unit Cost $ Per Square Foot - Federal Aid Highways				
	STATE	1995	1996	1997
1	CONNECTICUT	133	103	183
	MAINE	101	106	98
	MASSACHUSETT S	113	96	109
	NEW HAMPSHI RE	97	109	142
	NEW JERSEY	117	86	141
	NEW YORK	136	99	117
	RHODE ISLAND	N/A	N/A	172
	VERMONT	110	177	86
	PUE1RTO RICO	75	66	66
2	DELAWARE	114	80	117
	MARYLAND	91	82	76
	PENNSYLVANIA	111	119	109
	VIRGINIA	63	70	75
	WEST VIRGINIA	98	95	114
	DIST OF COLUMBI			
3	ALABAMA	48	42	44
	FLORIDA	47	58	56
	GEORGIA	50	50	39
	KENTUCKY	44	57	62
	MISSISSIPPI	36	45	39
	NORTH CAROLI NA	61	56	64
	SOUTH CAROLI NA	49	46	53
	TENNESSEE	46	45	55
4	ILLINOIS	71	73	69
	INDIANA	51	58	65
	MICHGAN	67	74	79
	MINNESOTA	53	58	58
	OHIO	66	63	66
	WISCONSI N	38	43	45
5	ARKANSAS	48	46	49
	LOUISIANA	32	38	36
	NEW MEXICO	52	70	56
	OKLAHOMA	37	36	43
	TEXAS	34	35	35
6	IOWA	40	40	40
	KANSAS	48	49	50
	MISSOURI	54	55	58
	NEBRASKA	50	60	53
7	COLORADO	47	55	52
	MONTANA	71	65	54
	NORTH DAKOTA	39	60	67
	SOUTH DAKOTA	48	42	49
	UTAH	49	58	64
	WYOMING	55	67	60
8	ARIZONA	52	48	62
	CALIFORNIA	83	69	71
	HAWAII	155	N/A	N/A
	NEVADA	46	58	102
9	ALASKA	123	141	141
	IDOHO	69	65	68
	OREGON	58	68	90
	WASHINGTON	76	90	98

Data Set A-5.2.7 Relative New Bridge Costs for Incremental Design Loads – FHWA

Data Set A-5.2.7 New Bridge Cost Ratios for Incremental Design Loads FHWA and Moses 1989

Note: For design loads not listed here, interpolation or extrapolation may be used as appropriate. The cost ratios in Table A-5.2.7.1 may also be used as appropriate.

All data are given by Dr. James Saklas except those in Table A-5.2.7.1 taken from (Moses 1989)

Table A-5.2.7.1 Cost Ratios for HS-30 Design Load

Bridge Type		Design Load HS-20	Design Load HS-30
Reinforced Concrete Slabs		1.000	#
Reinforced Ocncrete T Beams		1.000	#
Prestress Concretet I-Beams	≤ 60 ft Spans	1.000	#
	>60 ft Spans	1.000	#
Steel Girders		1.000	#

Table A-5.2.7.2 Reinforced Concrete Slab Bridge (Simple) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
30	HS 25	1.026	1.046	1.039
	HS 22.5	1.011	1.000	1.003
	HS 20	1.000	1.000	1.000
40	HS 25	1.042	1.036	1.037
	HS 22.5	1.019	1.000	1.005
	HS 20	1.000	1.000	1.000
50	HS 25	1.039	1.030	1.031
	HS 22.5	1.024	1.030	1.028
	HS 20	1.000	1.000	1.000

Table A-5.2.7.3 Reinforced Concrete Slab Bridge (Continuous) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
30	HS 25	1.008	1.113	1.075
	HS 22.5	1.015	1.059	1.043
	HS 20	1.000	1.000	1.000
40	HS 25	1.065	1.092	1.085
	HS 22.5	1.032	1.042	1.039
	HS 20	1.000	1.000	1.000
50	HS 25	1.043	1.039	1.040
	HS 22.5	1.035	1.035	1.035
	HS 20	1.000	1.000	1.000
60	HS 25	1.044	1.056	1.054
	HS 22.5	1.015	1.031	1.027
	HS 20	1.000	1.000	1.000

Table A-5.2.7.4 Prestressed Concrete Slab Bridge (Simple) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
30	HS 25	1.020	1.050	1.039
	HS 22.5	1.015	1.023	1.020
	HS 20	1.000	1.000	1.000
40	HS 25	1.056	1.014	1.055
	HS 22.5	1.032	0.994	1.034
	HS 20	1.000	1.000	1.000
50	HS 25	1.068	1.051	1.055
	HS 22.5	1.052	1.000	1.011
	HS 20	1.000	1.000	1.000

Table A-5.2.7.5 Prestressed Concrete Slab Bridge (Continuous) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
30	HS 25	1.052	1.057	1.055
	HS 22.5	1.012	1.030	1.024
	HS 20	1.000	1.000	1.000
40	HS 25	1.055	1.087	1.780
	HS 22.5	1.026	1.039	1.035
	HS 20	1.000	1.000	1.000
50	HS 25	1.042	1.050	1.048
	HS 22.5	1.018	1.024	1.022
	HS 20	1.000	1.000	1.000
60	HS 25	1.049	1.046	1.047
	HS 22.5	1.018	1.014	1.015
	HS 20	1.000	1.000	1.000
70	HS 25	1.033	1.025	1.027
	HS 22.5	1.017	1.013	1.014
	HS 20	1.000	1.000	1.000

Table A-5.2.7.6 Reinforced Concrete T-beam Bridge (Simple) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
30	HS 25	1.047	1.044	1.045
	HS 22.5	1.020	1.016	1.018
	HS 20	1.000	1.000	1.000
40	HS 25	1.052	1.035	1.041
	HS 22.5	1.019	1.018	1.018
	HS 20	1.000	1.000	1.000
50	HS 25	1.051	1.043	1.045
	HS 22.5	1.031	1.020	1.023
	HS 20	1.000	1.000	1.000
60	HS 25	1.108	1.054	1.070

	HS 22.5	1.081	1.038	1.050
	HS 20	1.000	1.000	1.000
70	HS 25	1.095	1.050	1.061
	HS 22.5	1.044	1.035	1.038
	HS 20	1.000	1.000	1.000

Table A-5.2.7.7 Reinforced Concrete T-beam Bridge (Continuous) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
30	HS 25	1.036	1.030	1.033
	HS 22.5	1.016	1.009	1.012
	HS 20	1.000	1.000	1.000
40	HS 25	1.045	1.056	1.052
	HS 22.5	1.024	1.036	1.032
	HS 20	1.000	1.000	1.000
50	HS 25	1.052	1.057	1.055
	HS 22.5	1.031	1.021	1.024
	HS 20	1.000	1.000	1.000
60	HS 25	1.055	1.080	1.073
	HS 22.5	1.021	1.030	1.027
	HS 20	1.000	1.000	1.000
70	HS 25	1.110	1.072	1.082
	HS 22.5	1.017	1.016	1.016
	HS 20	1.000	1.000	1.000
80	HS 25	1.116	1.108	1.109
	HS 22.5	1.046	1.041	1.043
	HS 20	1.000	1.000	1.000
90	HS 25	1.070	1.098	1.095
	HS 22.5	1.028	1.042	1.041
	HS 20	1.028	1.000	1.000
100	HS 25	1.062	1.052	1.054
	HS 22.5	1.040	1.035	1.036
	HS 20	1.000	1.000	1.000

Table A-5.2.7.8 Prestressed Concrete Beam Bridge (Precast) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
30	HS 25	1.019	1.000	1.009
	HS 22.5	1.003	1.000	1.001
	HS 20	1.000	1.000	1.000
40	HS 25	1.029	1.000	1.012
	HS 22.5	1.015	1.000	1.007
	HS 20	1.000	1.000	1.000
50	HS 25	1.038	1.000	1.014
	HS 22.5	1.014	1.000	1.005
	HS 20	1.000	1.000	1.000
60	HS 25	1.014	1.000	1.005
	HS 22.5	1.002	1.000	1.001
	HS 20	1.000	1.000	1.000
70	HS 25	1.032	1.000	1.011
	HS 22.5	1.012	1.000	1.003
	HS 20	1.000	1.000	1.000
80	HS 25	1.033	1.000	1.012
	HS 22.5	1.022	1.000	1.007
	HS 20	1.000	1.000	1.000
90	HS 25	1.027	1.000	1.010
	HS 22.5	1.009	1.000	1.003
	HS 20	1.000	1.000	1.000
100	HS 25	1.026	1.000	1.009
	HS 22.5	1.018	1.000	1.006
	HS 20	1.000	1.000	1.000
120	HS 25	1.023	1.000	1.007
	HS 22.5	1.015	1.000	1.004
	HS 20	1.000	1.000	1.000
140	HS 25	1.025	1.000	1.006
	HS 22.5	1.018	1.000	1.005
	HS 20	1.000	1.000	1.000

Table A-5.2.7.9 Prestressed Concrete Multicell Box Girder Bridge Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
80	HS 25	1.021	1.016	1.017
	HS 22.5	1.010	1.008	1.009
	HS 20	1.000	1.000	1.000
90	HS 25	1.025	1.020	1.021
	HS 22.5	1.012	1.010	1.011
	HS 20	1.033	1.001	1.007
100	HS 25	1.018	1.018	1.018
	HS 22.5	1.010	1.009	1.009
	HS 20	1.000	1.000	1.000
120	HS 25	1.018	1.018	1.018
	HS 22.5	1.010	1.009	1.010
	HS 20	1.000	1.000	1.000
140	HS 25	1.032	1.018	1.020
	HS 22.5	1.016	1.009	1.010
	HS 20	1.000	1.000	1.000
160	HS 25	1.017	1.016	1.016
	HS 22.5	1.008	1.008	1.008
	HS 20	1.000	1.000	1.000
180	HS 25	1.068	1.017	1.024
	HS 22.5	1.010	1.008	1.009
	HS 20	1.000	1.000	1.000
200	HS 25	1.021	1.015	1.016
	HS 22.5	1.010	1.008	1.008
	HS 20	1.002	1.000	1.000
220	HS 25	1.017	1.015	1.016
	HS 22.5	1.009	1.008	1.008
	HS 20	1.000	1.000	1.000
240	HS 25	1.016	1.015	1.015
	HS 22.5	1.008	1.008	1.008
	HS 20	1.000	1.000	1.000

Table A-5.2.7.10 Steel Rolled Beam Bridge Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
30	HS 25	1.036	1.040	1.038
	HS 22.5	1.005	1.040	1.024
	HS 20	1.000	1.000	1.000
40	HS 25	1.044	1.043	1.043
	HS 22.5	1.015	1.043	1.033
	HS 20		1.000	1.000
50	HS 25	1.019	1.057	1.046
	HS 22.5	1.014	1.057	1.045
	HS 20	1.000	1.000	1.000
60	HS 25	1.017	1.041	1.035
	HS 22.5	1.010	1.000	1.002
	HS 20	1.000	1.000	1.000
70	HS 25	1.025	1.122	1.101
	HS 22.5	1.011	1.054	1.045
	HS 20	1.000	1.000	1.000
80	HS 25	1.025	1.090	1.079
	HS 22.5	1.012	1.045	1.039
	HS 20	1.000	1.012	1.000

Table A-5.2.7.11 Steel Girder Bridge (simple) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
50	HS 25	1.021	1.063	1.049
	HS 22.5	1.052	1.031	1.038
	HS 20	1.000	1.000	1.000
60	HS 25	1.013	1.057	1.023
	HS 22.5	1.000	1.028	1.000
	HS 20	1.000	1.000	0.980
70	HS 25	1.019	1.049	1.041
	HS 22.5	1.010	1.024	1.021
	HS 20	1.000	1.000	1.000
80	HS 25	1.017	1.044	1.038
	HS 22.5	1.008	1.022	1.019
	HS 20	1.000	1.000	1.000
90	HS 25	1.500	1.039	1.026
	HS 22.5	1.013	1.020	1.002
	HS 20	1.000	1.000	0.985
100	HS 25	1.013	1.040	1.027
	HS 22.5	1.006	1.018	1.008
	HS 20	1.000	1.000	0.992
120	HS 25	1.012	1.044	1.039
	HS 22.5	1.007	1.022	1.019
	HS 20	1.000	1.000	1.000

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Table A-5.2.7.11 continued

140	HS 25	1.012	1.044	1.040
	HS 22.5	1.007	1.022	1.020
	HS 20	1.000	1.000	1.000
160	HS 25	1.012	1.041	1.037
	HS 22.5	1.006	1.022	1.020
	HS 20	1.000	1.000	1.000
180	HS 25	1.010	1.039	1.035
	HS 22.5	1.006	1.020	1.019
	HS 20	1.000	1.000	1.000
200	HS 25	1.019	1.033	1.032
	HS 22.5	1.013	1.016	1.015
	HS 20	1.000	1.000	1.000
220	HS 25	1.032	1.032	1.032
	HS 22.5	1.016	1.017	1.017
	HS 20	1.000	1.000	1.000
240	HS 25	1.017	1.032	1.030
	HS 22.5	1.005	1.016	1.016
	HS 20	1.000	1.000	1.000

Table A-5.2.7.12 Steel Girder Bridge (continuous) Cost Ratios

Span (ft)	Design Load	Substructure Cost Ratio	Superstructure Cost Ratio	Total Cost Ratio
50	HS 25	1.028	1.052	1.044
	HS 22.5	1.014	1.028	1.023
	HS 20	1.000	1.000	1.000
60	HS 25	1.014	1.040	1.032
	HS 22.5	1.007	1.020	1.016
	HS 20	1.000	1.000	1.000
70	HS 25	1.028	1.036	1.034
	HS 22.5	1.014	1.018	1.017
	HS 20	1.000	1.000	1.000