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**EFFECTS OF EXTREME GRAVITY AND SEISMIC LOADS ON  
SHORT TO MEDIUM SPAN SLAB-ON-GIRDER STEEL HIGHWAY BRIDGES**

by

**MURAT DİCLELİ**

A thesis submitted to  
the Faculty of Graduate Studies and Research  
in partial fulfillment of the requirements  
for the degree of  
**DOCTORATE OF PHILOSOPHY**  
in Civil Engineering\*

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Ottawa, Canada

May 1993

\* The Ph. D. of Civil Engineering Program  
is a joint program with Carleton University  
administered by the Ottawa-Carleton  
Institute for Civil Engineering



Murat Dicleli, Ottawa, Canada, 1993



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ISBN 0-315-89652-3

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## ABSTRACT

This study addresses two separate but related problems. They are detrimental effects of extreme gravity and seismic loadings, which were not considered in the original design, on slab-on-girder steel bridges.

In the first part of this thesis the effect of extreme gravity loads on slab-on-girder steel bridges is studied. Currently, in many jurisdictions of North America, special permits are issued to extra-heavy vehicles considering only the ultimate capacity of the bridges. Based on this, a large number of special permits have been issued to extra-heavy vehicles, and therefore concerns have been raised since the cumulative effect of such overloads have never been assessed. In this perspective, the ultimate and cumulative effect of such overloads on bridge components is studied.

Typical heavy permit-truck configurations are selected to investigate their effect on steel bridges. As a first step, influence line analyses are conducted to find the ranges of spans of simply supported and continuous bridges for which heavy permit-trucks have the most detrimental effects. In the light of these observations, a number of actual bridges within the estimated span length limits are analyzed to find the bridge members largely affected by such overloads. Next, a literature review is conducted to appraise the state-of-knowledge on the impact of infrequent stress-range excursions produced by heavy loads on fatigue life of bridge members. Recent research has demonstrated that if some stress-range excursions from a variable amplitude stress range spectrum exceed the constant amplitude fatigue limit (i.e. the stress-range magnitude below which fatigue life is deemed to be theoretically infinite), even by a small amount or by a small frequency,

then fatigue life is not infinite anymore. Hence, in this study, for stress range spectrum representative of Ontario trucks loading, the constant amplitude fatigue limit of the design S-N curve is replaced by an extension of the finite life portion of that curve. The effect of variable amplitude loading due to both Ontario truck traffic and heavy permit trucks on fatigue life of bridge members is investigated. Analytical expressions to calculate the fatigue life and the reduction in fatigue life of bridges due to heavy permit-trucks, without the need for a detailed analysis of each bridge, are derived and presented, along with a fatigue-based methodology to assess the reduction in service life of bridges attributable to heavy-permit trucks. Furthermore, as Ontario's normal truck traffic itself sometimes causes stress ranges that exceed the design-fatigue-truck's stress range by up to approximately 70 percent, although not very frequent, it was found that cyclic loading from this normal truck traffic can have a dominant impact on fatigue life of bridge components, bringing the fatigue life from an assumed theoretically infinite life to a finite one. Finally, using the derived equations, a sample permit-policy is presented assuming that a two percent reduction in fatigue life due to heavy permit-trucks is acceptable. This translates into specified numbers of permit-trucks per month for various span ranges.

In the second part, the response of bridge superstructure components to seismic excitations is investigated. Single span simply supported, continuous and multi-span simply supported bridges are studied by varying their geometric and structural properties. Linear and nonlinear dynamic analyses of these bridges are conducted for earthquakes of different characteristic and intensity considering only the superstructure. The forces and displacements of superstructure components which significantly affect the response, are determined as a function of the selected properties and earthquake types. Then, using these results, a methodology is developed for ranking and rapid seismic evaluation of existing steel bridges.

Bearing forces due to seismic excitation in both transverse and longitudinal direction are found to be proportional to the mass of the bridge, span length, and bearings' stiffness. They vary as a function of number of bearings and bridge width.

Bearings with higher stiffness and closer to the edge of the bridge deck are found to attract larger forces than other bearings. However, elastomeric bearings or bearings with small longitudinal stiffness are found to attract almost equal forces regardless of the width of the bridge. The effect of span length, number of spans, column size and steel strength on the seismic response of continuous and multi-span simply supported bridges are studied. It is found that bridges with longer spans generally have smaller seismic capacity. Bridges with smaller number of spans are more vulnerable to seismic excitations in the transverse direction than those with larger number of spans of identical end-to-end length. Bridges with larger column size are found to accommodate larger seismic forces, but their effect on decreasing the transverse displacement of continuous bridges is negligible. It is also found that bridges designed using steel with lower strength are more vulnerable to earthquake excitations. Next, sliding of bridges after the bearings are severed is investigated. High intensity earthquakes are required to slide multi-span simply supported bridges when the bearings at the abutments are severed but single span simply supported and continuous bridges may have considerable sliding displacements depending on the  $A_p/V_p$  ratio of earthquakes. Ground motions with high frequency content or high  $A_p/V_p$  ratio result in very low sliding displacements, whereas ground motions with intense long duration acceleration pulses or low  $A_p/V_p$  ratios can cause remarkable sliding displacements. For the same ratio,  $\mu_f/A_p$ , of friction coefficient to peak ground acceleration, the sliding displacement is found to be linearly proportional to the amplitude of the peak ground acceleration. Additionally, the effect of cross-bracing as a retrofitting element for continuous and multi-span simply supported bridges is investigated. It is found that cross-bracing may be used to increase the seismic capacity of bridges, but the effect of pressure and uplift forces exerted by the bracing elements on the foundations should be considered.

## **ACKNOWLEDGMENT**

I would like to express my deep gratitude to Dr. M. Bruneau, thesis supervisor, for his guidance, encouragement and financial support throughout the course of this study. Thanks are also due to Dr. M. Saatçioğlu for his moral support and valuable advice. The entrance scholarship provided by the School of Graduate Studies of the University of Ottawa and the financial support provided by the Ministry of Transportation of Ontario are highly appreciated.

Special thanks are extended to Brigitte for her moral support.

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## GLOSSARY

|           |                                                                                                                                                               |
|-----------|---------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $A$       | a constant relating the stress range to the number of cycles to failure                                                                                       |
| $A'$      | a constant relating the stress range to the number of cycles to failure in logarithmic form                                                                   |
| $A_{ab}$  | area of the anchor bolt, $m^2$                                                                                                                                |
| $A_D$     | cross-sectional area of the bridge deck, $m^2$                                                                                                                |
| $A_{Ds}$  | shear area of the bridge deck, $m^2$                                                                                                                          |
| $A_e$     | area of the elastomer parallel to the bridge deck, $m^2$                                                                                                      |
| $A_p$     | peak acceleration of the ground motion as a percentage of gravitational acceleration                                                                          |
| $A_{p10}$ | peak ground acceleration as a percentage of gravitational acceleration for a 10 percent probability of exceedance in 50 years                                 |
| $A_{p5}$  | peak ground acceleration as a percentage of gravitational acceleration for a 5 percent probability of exceedance in 50 years                                  |
| $A_{pc}$  | peak acceleration required for the collision of column-fixed decks                                                                                            |
| $A_{ps}$  | peak acceleration of the site as a percentage of gravitational acceleration                                                                                   |
| $ADTT$    | average daily truck traffic                                                                                                                                   |
| $AMF$     | acceleration magnification factor used to magnify the ground motion history to obtain an effective dynamic force for an equivalent SDOF system                |
| $AW$      | axle width, m                                                                                                                                                 |
| $a_i$     | ratio of the frequency of occurrence of stress range, $S_i$ to the number of constant-amplitude-cycles defining the fatigue life at a stress range, $S_{eff}$ |
| $B$       | slope of the finite life portion of the S-N curve                                                                                                             |
| $B_r$     | bearing force demand, kN                                                                                                                                      |
| $B_{rc}$  | capacity of a bearing, kN                                                                                                                                     |
| $B_{ry}$  | resultant bearing force due to seismic excitation in the transverse direction, kN                                                                             |
| $b$       | bridge width, m                                                                                                                                               |
| $b_{rx}$  | longitudinal component of bearing force due to seismic excitation in the longitudinal direction, kN                                                           |
| $b_{rxv}$ | longitudinal component of bearing force due to seismic excitation in the transverse direction, kN                                                             |
| $b_{ry}$  | transverse component of bearing force due to seismic excitation in the transverse direction, kN                                                               |

|                 |                                                                                                                                                                               |
|-----------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $C_1, C_2$      | correlation factors to account for the simultaneous occurrence of seismic excitations in both orthogonal directions and the directional uncertainty of the earthquake motions |
| $C_{av}$        | average of the present cost of the bridges in Ontario, dollars                                                                                                                |
| $\dot{c}$       | viscous damping coefficient                                                                                                                                                   |
| $DLA$           | dynamic load allowance                                                                                                                                                        |
| $DLAR$          | dynamic load allowance ratio                                                                                                                                                  |
| $d$             | depth of a column's cross-section, m                                                                                                                                          |
| $d_c$           | distance of a column to the centerline of the bridge deck, m                                                                                                                  |
| $E_f$           | friction energy, kN·m                                                                                                                                                         |
| $E_f^*$         | scaled friction energy, kN·m                                                                                                                                                  |
| $E_{Ri}$        | relative input energy, kN·m                                                                                                                                                   |
| $E_{Ri}^*$      | scaled relative input energy, kN·m                                                                                                                                            |
| $E_{Rk}$        | relative kinetic energy, kN·m                                                                                                                                                 |
| $E_{Rk}^*$      | scaled relative kinetic energy, kN·m                                                                                                                                          |
| $EJW$           | expansion joint width                                                                                                                                                         |
| $F_i$           | incremental transverse force produced by second order moments, kN                                                                                                             |
| $F_s$           | sliding or friction resistance force, kN                                                                                                                                      |
| $FVPC_{av}$     | future value of the two percent of the average present cost of bridges                                                                                                        |
| $f_E$           | frequency of occurrence of an earthquake for an assumed risk, 1/year                                                                                                          |
| $f_{hpt}$       | frequency of passage of heavy permit-trucks                                                                                                                                   |
| $\bar{f}_{hpt}$ | normalized frequency of passage of heavy permit-trucks                                                                                                                        |
| $f_i$           | frequency of occurrence of stress range, $S_i$ , in a variable amplitude stress range spectrum                                                                                |
| $\bar{f}_i$     | normalized frequency of occurrence of stress range, $S_i$ , in a variable amplitude stress range spectrum                                                                     |
| $G$             | shear modulus of elasticity of steel, kN/m <sup>2</sup>                                                                                                                       |
| $G_r$           | shear modulus of elasticity of elastomer, kN/m <sup>2</sup>                                                                                                                   |
| $g$             | gravitational acceleration, m/sec <sup>2</sup>                                                                                                                                |
| $H$             | seismic force acting on the actual system during the elastic phase; or seismic force acting on a structure, kN                                                                |
| $H^*$           | seismic force acting on the equivalent system during the elastic phase, kN                                                                                                    |
| $h_{bb}$        | height of the bearing bar, m                                                                                                                                                  |
| $h_c$           | column height, m                                                                                                                                                              |
| $I_{bd}$        | bearing damage index                                                                                                                                                          |
| $I_{cd}$        | column damage index                                                                                                                                                           |
| $I_{cx}$        | column's strong axis inertia, m <sup>4</sup>                                                                                                                                  |
| $I_{cy}$        | column's weak axis inertia, m <sup>4</sup>                                                                                                                                    |
| $I_D$           | moment of inertia of the bridge deck about a vertical axis perpendicular to it, m <sup>4</sup>                                                                                |
| $I_d$           | overall damage index of the structure                                                                                                                                         |
| $I_d^*$         | temporary overall damage index of the structure                                                                                                                               |
| $I_i$           | importance index                                                                                                                                                              |
| $I_R$           | ranking index                                                                                                                                                                 |

|               |                                                                                                                                           |
|---------------|-------------------------------------------------------------------------------------------------------------------------------------------|
| $I_{sw}$      | seat width index                                                                                                                          |
| $I_{swL}$     | seat width index in the longitudinal direction                                                                                            |
| $I_{swT}$     | seat width index in the transverse direction                                                                                              |
| $i$           | yearly rate of inflation                                                                                                                  |
| $J_c$         | column's torsional inertia, $m^4$                                                                                                         |
| $K$           | shape factor or the ratio of plastic to elastic section modulus; or effective length factor for columns in the plane of bending           |
| $K_{AFD}$     | stiffness of the abutment-fixed deck, $kN/m$                                                                                              |
| $K_{bL}$      | longitudinal stiffness of bearings-set, $kN/m$                                                                                            |
| $K_{bT}$      | transverse stiffness of bearings-set, $kN/m$                                                                                              |
| $K_{f\theta}$ | rotational stiffness of bearings set, $kN \cdot m/rad$ .                                                                                  |
| $K_{CFD}$     | stiffness of the column-fixed deck, $kN/m$                                                                                                |
| $K_{cL}$      | longitudinal direction stiffness of bridge columns-set, $kN/m$                                                                            |
| $K_{cT}$      | transverse direction stiffness of bridge columns-set, $kN/m$                                                                              |
| $K_{DA}$      | axial stiffness of the deck, $kN/m$                                                                                                       |
| $K_{TT}$      | transverse direction stiffness of the bridge, $kN/m$                                                                                      |
| $k^*$         | generalized stiffness for an equivalent SDOF system, $kN/m$                                                                               |
| $k_1$         | number of bridges in Ontario with spans shorter than or equal to 35 metres                                                                |
| $k_2$         | number of bridges in Ontario with spans longer than 35 metres and shorter than or equal to 50 metres                                      |
| $k_3$         | number of bridges in Ontario with spans longer than 50 metres and shorter than or equal to 75 metres                                      |
| $k_4$         | number of bridges in Ontario with spans longer than 75 metres                                                                             |
| $k_{ab}$      | spring coefficient or stiffness of the anchor bolt, $kN/m$                                                                                |
| $k_{bb}$      | stiffness of the bearing bar in a sliding bearing, $kN/m$                                                                                 |
| $k_{bL}$      | longitudinal stiffness of a bearing, $kN/m$                                                                                               |
| $k_{bT}$      | transverse stiffness of a bearing, $kN/m$                                                                                                 |
| $k_{cL}$      | longitudinal direction stiffness of bridge column, $kN/m$                                                                                 |
| $k_{cT}$      | transverse direction stiffness of bridge column, $kN/m$                                                                                   |
| $k_D$         | stiffness of the bridge deck idealized as a cantilever, $kN/m$                                                                            |
| $k_{f\theta}$ | rotational stiffness of the foundation, $kN \cdot m/rad$ .                                                                                |
| $k_r$         | local rotational spring stiffness, $kN/m$                                                                                                 |
| $k_t$         | local translational spring stiffness, $kN/m$                                                                                              |
| $L$           | span length, $m$                                                                                                                          |
| $L_{av}$      | largest of the average lengths of adjacent spans in a continuous bridge, $m$                                                              |
| $L_T$         | total end-to-end length of a bridge                                                                                                       |
| $LLR$         | live load ratio or the ratio of the heavy permit-truck's flexural stress range to the OHBD live load's flexural stress range              |
| $l_{ab}$      | length of the anchor bolt, $m$                                                                                                            |
| $l_b$         | distance of bearing to the centerline of the bridge deck, $m$                                                                             |
| $I_{bb}$      | moment of inertia of the cross-section of the bearing bar parallel to the deck, about an axis in the bridge's transverse direction, $m^4$ |
| $l_p$         | length between the anchor bolt in tension and tip of the bottom plate, $m$                                                                |

|                 |                                                                                                                         |
|-----------------|-------------------------------------------------------------------------------------------------------------------------|
| $M$             | total moment at a bridge cross-section due to heavy permit-trucks, $\text{kN} \cdot \text{m}$ ; or earthquake magnitude |
| $M^+$           | mid-span moment in girders due to gravity loading, $\text{kN} \cdot \text{m}$                                           |
| $M^-$           | support moment in girders due to gravity loading, $\text{kN} \cdot \text{m}$                                            |
| $M_1$           | smaller of the moments at the column ends, $\text{kN} \cdot \text{m}$                                                   |
| $M_2$           | larger of the moments at the column ends, $\text{kN} \cdot \text{m}$                                                    |
| $M_{ux}$        | maximum strong axis moment prior to column's failure, $\text{kN} \cdot \text{m}$                                        |
| $M_{uy}$        | maximum weak axis moment prior to column's failure, $\text{kN} \cdot \text{m}$                                          |
| $M_D$           | dead load moment, $\text{kN} \cdot \text{m}$                                                                            |
| $M_{Ex}$        | column's longitudinal seismic moment, $\text{kN} \cdot \text{m}$                                                        |
| $M_{Ex1}$       | first order column's longitudinal seismic moment, $\text{kN} \cdot \text{m}$                                            |
| $M_{Exy}$       | column's longitudinal seismic moment due to loading in the transverse direction, $\text{kN} \cdot \text{m}$             |
| $M_{Ey}$        | column's transverse seismic moment, $\text{kN} \cdot \text{m}$                                                          |
| $M_{Ey1}$       | first order column's longitudinal seismic moment, $\text{kN} \cdot \text{m}$                                            |
| $M_{Eyx}$       | column's transverse seismic moment due to loading in the longitudinal direction, $\text{kN} \cdot \text{m}$             |
| $M_e$           | maximum exterior girder moment due to any truck, $\text{kN} \cdot \text{m}$                                             |
| $M_{eff}$       | effective modal mass, ton                                                                                               |
| $(M_e)_{OHBDT}$ | exterior girder moment obtained from OHBDT, $\text{kN} \cdot \text{m}$                                                  |
| $M_i$           | maximum interior girder moment due to any truck, $\text{kN} \cdot \text{m}$                                             |
| $(M_i)_{OHBDT}$ | interior girder moment obtained from OHBDT, $\text{kN} \cdot \text{m}$                                                  |
| $M_L$           | live load moment, $\text{kN} \cdot \text{m}$                                                                            |
| $M_{OHBDT}$     | total moment at a bridge cross-section due to OHBDT-truck, $\text{kN} \cdot \text{m}$                                   |
| $M_{prx}$       | reduced plastic strong axis moment considering the effect of axial force, $\text{kN} \cdot \text{m}$                    |
| $M_{pry}$       | reduced plastic weak axis moment considering the effect of axial force, $\text{kN} \cdot \text{m}$                      |
| $M_{px}$        | plastic strong axis moment for class 1 or 2 sections or yield moment for class 3 sections, $\text{kN} \cdot \text{m}$   |
| $M_{py}$        | plastic weak axis moment for class 1 or 2 sections or yield moment for class 3 sections, $\text{kN} \cdot \text{m}$     |
| $M_R$           | unfactored moment resistance of a member or component, $\text{kN} \cdot \text{m}$                                       |
| $M_r$           | maximum moment range over any girder due to any truck, $\text{kN} \cdot \text{m}$                                       |
| $M_{r-OHBDT}$   | maximum moment range over any girder obtained from OHBDT, $\text{kN} \cdot \text{m}$                                    |
| $M_{rx}$        | maximum resistible strong axis moment in the absence of axial load, $\text{kN} \cdot \text{m}$                          |
| $M_s$           | in-plane support moment, $\text{kN} \cdot \text{m}$                                                                     |
| $M_u$           | moment resistance of a member subject to lateral buckling, $\text{kN} \cdot \text{m}$                                   |
| $M_{x0}$        | first order strong axis moment, $\text{kN} \cdot \text{m}$                                                              |
| $M_{xi}$        | second order strong axis moment obtained from the $i^{\text{th}}$ iteration, $\text{kN} \cdot \text{m}$                 |
| $M_{xy0}$       | first order strong axis moment due to loading in the weak axis direction, $\text{kN} \cdot \text{m}$                    |

|           |                                                                                                                                                                            |
|-----------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $M_{xvi}$ | second order strong axis moment due to loading in the weak axis direction, obtained from the $i^{\text{th}}$ iteration, $\text{kN}\cdot\text{m}$                           |
| $M_{y0}$  | first order weak axis moment, $\text{kN}\cdot\text{m}$                                                                                                                     |
| $M_{yi}$  | second order weak axis moment obtained from the $i^{\text{th}}$ iteration, $\text{kN}\cdot\text{m}$                                                                        |
| $MPT$     | toll to be charged to a permit truck per passage of a bridge                                                                                                               |
| $m$       | number of stress range cycles; or bridge mass, ton                                                                                                                         |
| $m^*$     | generalized mass for an equivalent SDOF system, ton                                                                                                                        |
| $m_D$     | mass of a single deck, ton                                                                                                                                                 |
| $m_{Ex}$  | first order column seismic moment in the longitudinal direction for a unit peak ground acceleration, $\text{kN}\cdot\text{m}$                                              |
| $m_{Exy}$ | first order column seismic moment in the longitudinal direction, due to loading in the transverse direction, for a unit peak ground acceleration, $\text{kN}\cdot\text{m}$ |
| $m_{Ey}$  | first order column seismic moment in the transverse direction for a unit peak ground acceleration, $\text{kN}\cdot\text{m}$                                                |
| $N_{CR}$  | maximum number of cycles that can be applied by Ontario truck traffic                                                                                                      |
| $N_c$     | total number of the applied cycles of the stress range spectrum                                                                                                            |
| $N_{eff}$ | number of constant-amplitude-cycles defining the fatigue life at a stress range, $S_{eff}$                                                                                 |
| $N_{hpt}$ | number of constant-amplitude-cycles of stress range, $S_{hpt}$ , as read from the S-N curve                                                                                |
| $N_i$     | number of constant-amplitude-cycles of stress range, $S_i$ , as read from the S-N curve                                                                                    |
| $N_L$     | number of cycles corresponding to CAFL of the S-N curve                                                                                                                    |
| $N_R$     | reduced fatigue life in cycles                                                                                                                                             |
| $N_{ym}$  | mean fatigue life in years                                                                                                                                                 |
| $N_{ys}$  | safe fatigue life in years                                                                                                                                                 |
| $NT_{av}$ | average number of trucks per year                                                                                                                                          |
| $n_{ab}$  | number of anchor bolts in tension                                                                                                                                          |
| $n_b$     | total number of bridges in Ontario; or number of bearings                                                                                                                  |
| $n_c$     | number of columns                                                                                                                                                          |
| $n_{cs}$  | number of column-sets                                                                                                                                                      |
| $n_{ej}$  | number of expansion joints                                                                                                                                                 |
| $n_{hpt}$ | total number of passages of a heavy permit-truck during the fatigue life time                                                                                              |
| $n_i$     | frequency of occurrence of any stress range, $S_i$                                                                                                                         |
| $n_{pm}$  | number of heavy permit-truck passages per month                                                                                                                            |
| $n_s$     | number of spans                                                                                                                                                            |
| $ODR$     | over design ratio                                                                                                                                                          |
| $P$       | axial load, $\text{kN}$                                                                                                                                                    |
| $P_D$     | axial load due to gravity, $\text{kN}$                                                                                                                                     |
| $P_E$     | seismic axial force in the column for a unit peak ground acceleration, $\text{kN}$                                                                                         |
| $P_{eff}$ | effective dynamic force acting on an equivalent SDOF system, $\text{kN}$                                                                                                   |

|             |                                                                                                                                                            |
|-------------|------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $P_n$       | allowable axial force considering column instability in the absence of moment, kN                                                                          |
| $P_y$       | yield axial load, kN                                                                                                                                       |
| $PC_{av}$   | two percent of the average present cost of the bridges in Ontario, dollars                                                                                 |
| $PR$        | percentage of reduction in fatigue life due to heavy permit-trucks                                                                                         |
| $R$         | sum of the reaction forces on the bearings due to the weight of the bridge, kN                                                                             |
| $R^*$       | modified vertical force acting on an equivalent system and representing the sum of the reaction forces on the bearings due to the weight of the bridge, kN |
| $R_A$       | ratio of peak ground acceleration for a 5 percent probability of exceedance to that for a 10 percent probability of exceedance in 50 years                 |
| $R_h$       | hypocentral distance, km                                                                                                                                   |
| $RS$        | ratio of deck stiffness to the rotational stiffness of bearings-set                                                                                        |
| $r_1$       | scaling factor for ground motion and friction coefficient                                                                                                  |
| $r_2$       | scaling factor for sliding displacement                                                                                                                    |
| $r_x$       | radius of gyration about strong axis of a structural section, m                                                                                            |
| $S$         | elastic section modulus, $m^3$                                                                                                                             |
| $S_{1.0}$   | stress range value when $S'_1=1.0$ , $kN/m^2$                                                                                                              |
| $S_s$       | stress range beyond which there is no increase in the slope of the S-N curve below the CAFL, $kN/m^2$                                                      |
| $S_u$       | absolute spectral acceleration, $m/sec^2$                                                                                                                  |
| $S_{eff}$   | effective stress range, $kN/m^2$                                                                                                                           |
| $S_i$       | $i^{th}$ stress range, $kN/m^2$                                                                                                                            |
| $S'_i$      | ratio of Ontario trucks' stress range to OHBD-fatigue-truck's stress range                                                                                 |
| $S_L$       | allowable stress range, $kN/m^2$                                                                                                                           |
| $S_{max}$   | maximum stress range in the spectrum, $kN/m^2$                                                                                                             |
| $S_{SLS-I}$ | minimum required elastic section modulus if the bridge is designed for SLS-I, $m^3$                                                                        |
| $S_u$       | ultimate strength, $kN/m^2$                                                                                                                                |
| $SF$        | global safety factor                                                                                                                                       |
| $SRC$       | stress reduction coefficient                                                                                                                               |
| $SW$        | minimum required seat width, mm                                                                                                                            |
| $SWL$       | seat width in the longitudinal direction, mm                                                                                                               |
| $SWT$       | seat width in the transverse direction, mm                                                                                                                 |
| $T_1$       | fundamental period, s                                                                                                                                      |
| $T_i$       | incremental torsional moment acting on the expansion joint, $kN \cdot m$                                                                                   |
| $t$         | thickness of elastomeric bearing, m                                                                                                                        |
| $t_f$       | thickness of a column's flange, m                                                                                                                          |
| $u$         | relative displacement of an equivalent SDOF system, m                                                                                                      |
| $\dot{u}$   | relative velocity of an equivalent SDOF system, $m/sec$                                                                                                    |
| $\ddot{u}$  | relative acceleration of an equivalent SDOF system, $m/sec^2$                                                                                              |
| $u_i$       | deformed shape function, m                                                                                                                                 |
| $u_e$       | elastic displacement of an equivalent SDOF system, m                                                                                                       |

|                    |                                                                                                                         |
|--------------------|-------------------------------------------------------------------------------------------------------------------------|
| $\dot{u}_e$        | velocity of an equivalent SDOF system in the elastic phase, m/sec                                                       |
| $u_{ec}$           | elastic displacement of the structure at the column's location                                                          |
| $u_g$              | ground displacement, m                                                                                                  |
| $\ddot{u}_g$       | acceleration of ground motion, m/sec <sup>2</sup>                                                                       |
| $u_s$              | sliding displacement of an equivalent SDOF system,                                                                      |
| $u_t$              | total displacement of an equivalent SDOF system                                                                         |
| $\ddot{u}_t$       | total acceleration of an equivalent SDOF system, m/sec <sup>2</sup>                                                     |
| $V_p$              | peak velocity of the ground motion, m/sec                                                                               |
| $VI_x$             | Incremental column shear force in the longitudinal direction, kN                                                        |
| $VI_{xy}$          | Incremental column shear force in the longitudinal direction due to loading in the transverse direction, kN             |
| $VI_y$             | Incremental column shear force in the transverse direction, kN                                                          |
| $W$                | weight of the truck                                                                                                     |
| $w_f$              | percentage of the weight of a structure transferred to the supports where there is friction resistance                  |
| $x_c$              | the distance of columns-set measured from the abutment, m                                                               |
| $Y$                | Inverse of the S-N curve's slope below the CAFL                                                                         |
| $YI$               | Yearly instalments to pay the future cost of the bridges in Ontario, dollars                                            |
| $Z$                | plastic section modulus, m <sup>3</sup>                                                                                 |
| $Z_u$              | peak acceleration zone parameter                                                                                        |
| $Z_v$              | peak velocity zone parameter                                                                                            |
| $\alpha$           | dimensionless torsional parameter                                                                                       |
| $\alpha_D$         | dead load factor                                                                                                        |
| $\alpha_L$         | live load factor                                                                                                        |
| $\alpha_L^E$       | effective live load factor                                                                                              |
| $\alpha_T$         | coefficient of thermal expansion, 1/C°                                                                                  |
| $\beta$            | ratio of the absolute spectral acceleration, $S_u$ , to the peak ground acceleration $A_p$                              |
| $\beta_{mx}$       | magnification factor for strong axis moment due to seismic excitation in the longitudinal direction                     |
| $\beta_{mxy}$      | magnification factor for strong axis moment due to seismic excitation in the transverse direction                       |
| $\beta_{my}$       | magnification factor for weak axis moment due to seismic excitation in the transverse direction                         |
| $\Delta_{nb}$      | elongation of the anchor bolt in a sliding bearing due to an imposed unit displacement at the tip of the bearing bar, m |
| $\Delta_{bT}$      | transverse displacement of the bearings, m                                                                              |
| $\Delta_{b\theta}$ | displacement at the columns' location due to the rotation of the bridge deck at the support, m                          |
| $\Delta_c$         | displacement at columns' location for a unit peak ground acceleration, m                                                |
| $\Delta_{cL}$      | longitudinal displacement of the columns, m                                                                             |
| $\Delta_{cLR}$     | the ratio of columns' displacement to the displacement of 6 m column                                                    |
| $\Delta_{cT}$      | transverse displacement of the columns, m                                                                               |



|                |                                                                                                                          |
|----------------|--------------------------------------------------------------------------------------------------------------------------|
| $\Delta_d$     | displacement at the columns' location due to flexural and shear deformation of the bridge deck, m                        |
| $\Delta_{ejo}$ | maximum possible expansion joint opening, m                                                                              |
| $\Delta_{ms}$  | midspan displacement, m                                                                                                  |
| $\Delta_{rm}$  | ratio of the displacement of the deck at the column locations to the midspan displacement                                |
| $\Delta_s$     | displacement of the column-fixed deck due to seismic excitation in the longitudinal direction, m                         |
| $\Delta_{x0}$  | first order elastic displacement of a column in the longitudinal direction, m                                            |
| $\Delta_{xy0}$ | first order elastic displacement of a column in the longitudinal direction due to loading in the transverse direction, m |
| $\Delta_{y0}$  | first order elastic displacement of a column in the transverse direction, m                                              |
| $\Delta l$     | incremental displacement of the deck at the columns' location, m                                                         |
| $\Delta l_x$   | Incremental elastic displacement of a column in the longitudinal direction, m                                            |
| $\Delta T$     | temperature difference, C°                                                                                               |
| $\Theta$       | dimensionless flexural parameter                                                                                         |
| $\Theta l$     | Incremental rotation of the deck at the columns' location                                                                |
| $\theta_M$     | negative rotation due to in-plane support moment                                                                         |
| $\theta_s$     | support rotation                                                                                                         |
| $\theta_{s0}$  | uncorrected support rotation or rotation obtained from the first derivative of the mode shape function                   |
| $\lambda_m$    | modified effective slenderness ratio                                                                                     |
| $\lambda_s$    | effective slenderness ratio                                                                                              |
| $\mu_c$        | displacement ductility of columns                                                                                        |
| $\mu_r$        | coefficient of friction                                                                                                  |
| $\sigma_{all}$ | allowable stress, kN/m <sup>2</sup>                                                                                      |
| $\sigma_y$     | yield stress, kN/m <sup>2</sup>                                                                                          |
| $\tau_y$       | shear strength of anchor bolts                                                                                           |
| $\Phi$         | performance factor                                                                                                       |
| $\psi$         | ratio of sliding resistance, $F_s$ , to the effective modal mass, $M_{eff}$ times the peak ground acceleration, $A_p$    |
| $\omega$       | coefficient used to determine equivalent uniform bending effect in beam-columns                                          |
| $\omega_1$     | fundamental circular frequency, rad/s                                                                                    |
| $\mathcal{Q}$  | earthquake excitation factor                                                                                             |

## Abbreviations

|       |                                                  |
|-------|--------------------------------------------------|
| AASHO | American Association of State Highways Officials |
| CAFL  | constant amplitude fatigue limit                 |
| GB    | Grigg Bridge                                     |

|        |                                                      |
|--------|------------------------------------------------------|
| LBFC   | longitudinal bearing force coefficient               |
| MCB-I  | Mud Creek Bridge-I                                   |
| MCB-II | Mud Creek Bridge-II                                  |
| MP1SD  | mean plus one standard deviation                     |
| MTO    | Ministry of Transportation of Ontario                |
| OHBDC  | Ontario Highway Bridge Design Code                   |
| SCWB   | strong column weak beam                              |
| SDOF   | single degree of freedom                             |
| SRB-I  | Saugeen River Bridge-I                               |
| SRB-II | Saugeen River Bridge-II                              |
| SSRB   | South Saugeen River Bridge                           |
| TBFC   | Transverse bearing force coefficient                 |
| UBC    | uniform building code                                |
| ULS    | ultimate limit states                                |
| SLS    | serviceability limit states                          |
| S-N    | stress range amplitude - number of cycles to failure |
| WCSB   | weak column strong beam                              |

# **CHAPTER 1**

## **INTRODUCTION**

Loads acting on bridges can be classified as gravity and lateral loads. Typically, these are vehicles, and earthquake or wind loads. In the case of gravity loads, fatigue and ultimate strength are the two important parameters to be considered in design, whereas strength and energy absorbing capability, key parameters to ensure survival of a structure during a strong earthquake, must be considered in seismic design. Based on the current state of design practice, heavy permit-trucks and severe earthquakes are the extreme loads acting on bridges.

### **1.1 Extreme Gravity Loads**

The permissible weights on axles have been raised at relatively frequent intervals since the vehicle weight regulations were first introduced on Canadian roads. The current maximum vehicle weight legal limit in Ontario is somewhat related to the Ontario Bridge Formula (Jung & Witecki, 1971) which is based on live load capacity of bridge structures and is a function of the weight and axle configuration of trucks. It is introduced to protect highway structures and pavement from damage caused by the overloading of vehicles.

Sometimes, for practical, technical and/or economic reasons, the limitations on legal vehicle weight can be exceeded for heavy indivisible loads. Therefore, special permits are required to move a vehicle in excess of dimensional and weight limits. A rational methodology has been developed by the Ministry of Transportation of Ontario (MTO) to allow a rapid appraisal of permit applications without requiring a detailed evaluation of all the bridges to be used (Agarwal, 1981). Based on this methodology a large number of special permits have been issued to extra-heavy vehicles. This poses a particular problem, as the basis on which those special permits are granted has been established without considering the cumulative effect of such overloads. The problem is also much broader than Ontario as many other Ministries and Departments of Transportation throughout North America have adopted, or are in the process of adopting, similar methodologies to grant special permits to extra heavy trucks without regards to cumulative effects.

Bridge components are subjected to cyclic loading and consequently exposed to potential fatigue failure. The evaluation of the fatigue damage accumulation for bridges is based on the so-called S-N curves, which are the relationships between stress range and number of load cycles to failure. Good bridge engineering practice often consists of selecting a service-induced stress lower than the lower plateau of the S-N curves, i.e. a value below which fatigue life is theoretically infinite. The stress range value at this lower plateau is mostly a function of the type of detailing used and therefore, during the design process, by limiting the details to some preselected types known to have excellent fatigue behaviour, this stress range can be made to match the service design stresses.

The infrequent but extreme loadings now applied to the structures due to the granting of special permits and not anticipated during the original design of the bridges, can potentially lead to stress-ranges exceeding those of the flat lower bound of the S-N curves, initiating the accumulation of damage cycles. Therefore the damage to a bridge caused by a single passage of an extremely heavy vehicle may exceed that caused by many thousands of vehicles complying with the limitations of the vehicle weight

regulations.

In the type of loading mentioned above, where the majority of stress cycles are below the fatigue limit while some of them are above, Miner's rule which relates damage to the life of the structure, to a complex stress pattern in a linear way, may not be applicable anymore. Furthermore, the conventional S-N curve may not be valid either. Therefore, a methodology should be developed to assess the remaining fatigue life of bridge members under variable amplitude long-life fatigue loading. Concurrently, other potential structural problems in bridges subjected to heavy permit-truck loadings should also be identified.

## **1.2 Extreme Lateral Loads**

In addition to the problems caused by heavy permit-trucks, earthquakes also cause structural problems in steel bridges which are vulnerable to ground excitation. Many lessons on the seismic performance of bridges were learned from past earthquakes. These lessons have been used to improve the design codes and to better understand the dynamic behaviour of structures. In particular it is now recognized that: (i) Special attention must be paid to the substructures, foundations and the connections to the superstructure of bridges, and that enough strength and ductility should be provided at the key locations to obtain a good performance during an earthquake; (ii) The transfer of seismic loads (which are lateral forces that act through the centre of mass of the structures) to the ground by the shortest and most direct path will generally assure the best performance; (iii) Symmetry in plan is desirable to minimize rotation about a vertical axis and avoid damaging effects of torsional forces, and; (iv) Various components of a bridge must remain connected together during an earthquake. These criteria, namely simplicity, symmetry and integrity are required to have a good structural form for ductile behaviour.

However, few of the existing steel bridges were designed considering the above criteria. Consequently, many steel bridges could be damaged if subjected to an earthquake, and should therefore be retrofitted to ensure that the infrastructure remains undamaged or has easily repairable damage after the earthquake. Determining the vulnerable components to be retrofitted in each existing bridge individually is possible, but not practical considering the large number of existing steel bridges still in service. Furthermore, it is not economically feasible either to retrofit all the existing steel bridges which would be determined as vulnerable to earthquake excitation. Therefore, a guideline is required to advise as to which bridges are most likely to be problems, and a strategy must be established to prioritize the retrofit of these bridges. This goal may be achieved by establishing what levels of strength and ductility are required for critical bridge components, as a function of structure geometry, structure type and earthquake excitations of various characteristics and intensity.

### **1.3 Objectives and Scope**

This study is composed of two main parts. In the first part, the objective is to investigate the effect of selected typical heavy permit-truck configurations on actual steel bridges. In this respect, fatigue and ultimate strength, two important parameters in bridge design will be studied and the adequacy of existing methodologies used to permit extra-heavy trucks on Ontario bridges will be evaluated. As a first step, influence line analyses will be conducted to find the ranges of span lengths of simply supported and continuous bridges for which heavy permit-trucks could have the most detrimental effects. In the light of these observations a number of actual bridges within the identified critical span length limits will be analyzed to study the effect of heavy permit-trucks on ultimate strength and fatigue life of different bridge members. As a result of these analyses, the most vulnerable bridge components will be identified. The stress ranges produced by normal truck traffic and heavy permit-trucks constitutes a variable amplitude stress range spectrum. The effect of this stress range spectrum on fatigue life of these bridge

components will be investigated. Based on the findings, a methodology will be developed to assess the reduction in service life of the bridges due to normal truck traffic and heavy permit-trucks.

In the second part, the objective is to investigate the response of bridge superstructure components to seismic excitations, and to establish a methodology to rank existing steel bridges with respect to their vulnerability to earthquakes, while providing rapid and accurate assessment of this vulnerability. To accomplish this, single span simply supported, continuous and multi-span simply supported slab-on-girder bridges will be studied by varying their geometric and structural parameters to represent a broad range of existing steel bridges. Linear and non-linear dynamic analyses of these bridges will be conducted for earthquakes of different characteristic and intensity. The strength, and if applicable ductility demand of superstructure components which significantly affect the response, will be determined as a function of the selected parameters and earthquake types. Then, using these results, a methodology will be developed for rapid seismic evaluation and ranking of existing steel bridges. The seismic performance and behavior of foundations and abutments, as well as soil-structure interaction, is beyond the scope of this study.

## **1.4 Review of Previous Study**

### **1.4.1 Literature Review on Fatigue Under Variable Amplitude Loading**

The concept of cumulative fatigue damage under variable amplitude loading was first introduced by Miner (1945). He stated that fatigue damage to the life of a structure is a linear function of the cycle ratio, i.e. the ratio of the number of cycles applied, to the number of cycles to failure. On the contrary, Marco and Starkey (1954) claimed that fatigue damage is an exponential function of the cycle ratio rather than a linear function. However, in practical applications, the more complex cumulative damage rules were not

found to be better than the Miner's simple linear cumulative damage rule (Walter, 1979)

Several researchers tried to determine a constant amplitude effective stress that can be used to define the fatigue damage under variable amplitude loading. Barsom (1973) demonstrated that, an equivalent constant amplitude load, which is the square root of the mean of the squares of the individual load cycles in a variable amplitude stress spectrum, can predict the fatigue damage due to variable amplitude loading. Later, Albrecht and Friedland (1979) showed that a constant amplitude effective stress, which is the cube root of the sum of the cubes of the individual stress cycles, can also predict fatigue damage due to variable amplitude loading. Furthermore, Zwerreman and Frank (1988) demonstrated that the mean levels of the minor cycles within a complex load-time history significantly affect the fatigue damage. In addition, minor cycles below the constant amplitude threshold contribute to damage caused by the complex cycle. They proposed a damage index to be used to convert the complex cycle into an equivalent constant amplitude cycle.

Other researchers attempted to determine the parameters affecting the fatigue strength. Experiments were conducted to identify the variables that influence the fatigue strength of full scale welded details typically used in bridge construction (Fisher et al., 1970; Fisher, 1974). It was found that stress range and the detail type are the only parameters affecting the fatigue strength. Later, Abtahi et al. (1976) demonstrated that the rate of fatigue crack growth under constant amplitude cyclic-load fluctuation can be retarded as a result of the application of a single or multiple tensile load cycles having a peak load greater than that of the constant amplitude cycles. Schilling et al. (1978) showed that the order of occurrence of the cyclic load fluctuations does not affect the average rate of crack growth.

Agarwal and Workowicz (1976) gathered statistical data about the weight and axle configurations of the truck population in Ontario to have information about variable amplitude loading on bridges. From this data a probability density histogram of efforts



in structural members generated by trucks was constructed. The probability density function found to best fit the heavy portion of the truck population is the Rayleigh distribution. Fisher et al. (1983) used this distribution as a load spectrum in the experiments conducted to provide information on evaluating the high cycle fatigue resistance of welded attachments. It was found that, if any of the stress range cycles in a variable amplitude spectrum exceed the constant amplitude fatigue limit (CAFL), then the fatigue life is not infinite anymore and the S-N curve must be extended below the CAFL. Furthermore, Fisher and Keating (1987) reviewed the fatigue test data generated around the world on welded bridge details and formed a database. They observed that as the database for a given detail has increased, the slope of the finite life portion of the S-N curve (i.e. stress range (S) versus number of cycles to failure (N)) tends to stabilize to a value of 1:3. Nevertheless, the slope of the S-N curve below the CAFL is not defined very well and depends on the spectrum shape and the peak stress range (Fisher & Keating, 1989).

Although further research is required to define the slope of the S-N curve below the CAFL as a function of the spectrum shape and maximum stress range, with the current knowledge, a safe determination of the fatigue life is possible by applying the Miner's rule on an S-N curve which has no CAFL.

#### **1.4.2 Literature Review on Non-linear Dynamic Response of Highway Bridges Subjected to Ground Excitation**

After the 1971 San Fernando earthquake, extensive research was conducted to identify the structural components and parameters which significantly affect the seismic response of reinforced concrete bridges. The longitudinal restrainer ties between expansion joints and the strength and distribution of columns in each deck segments were found to be the primary factors affecting the seismic response characteristic of multiple span bridges (Tsang & Penzien, 1973; Penzien et al., 1975). In addition to the ties which

are required between deck segments to retain the integrity of a bridge. adequate lateral reinforcement, end anchorage and splice length are required in reinforced concrete columns to ensure a ductile behavior (Degenkolb, 1978). Adequate restrainers should also be provided at rocker bearings to prevent excessive displacement (Douglas, 1979). The effect of impacting when adjacent sections of a bridge structure collide is also very important since it causes high shear forces at the superstructure-substructure connections such as bearings. Therefore, bridge systems that have non-ductile bearings are vulnerable to eccentric impact loadings; their bearings may fail prior to the development of plastic hinging in the energy absorbing systems (Imbsen and Penzien, 1986).

Another important component is the transverse shear restraint at expansion joints. The absence of this component can lead to a large opening in the expansion joint as a consequence of the rigid body rotation of the bridge deck about a vertical axis (Imbsen and Penzien, 1986).

Non-uniform distribution of column stiffnesses along the bridge and disproportionate flexural and shear strengths of the columns are also important factors negatively affecting bridge response (Priestley, 1988). These factors were responsible for the damage which occurred in some bridges during 1987 Whitter earthquake.

Another parameter, soil-structure interaction, was also found to affect the dynamic response of bridges (Penzien et al., 1975; Chen & Penzien, 1975; Baron & Pong, 1975; Douglas & Reid, 1982). However, much remains to be learned on the behaviour of soil and how it interact with bridges. The degree of accuracy in such analysis depends on good assessments of the parameters which govern the interaction. Therefore, an explicit requirement to address the soil-structure interaction problem has not been included in bridge design codes yet (Seismic, 1987).

Several researchers have tried to correlate computer analysis results with the actual behavior of bridges to improve modelling techniques. Tsang and Penzien (1973)

observed that linear seismic analysis provides a reasonable estimate of the maximum displacement response, however, substantial error may result in predicting the internal forces in the structure. Kawashima and Penzien (1976) demonstrated that the displacement seismic response of a curved bridge under low intensity excitation can be predicted with fairly good accuracy using linear analytical models. On the other hand, the displacement seismic response produced by high intensity excitation can be predicted realistically using a non-linear mathematical model which accounts for the effects of collisions, slippage and tie bars. Douglas (1979) stated that the vertical stiffness of composite girder bridges can be estimated by treating them as continuously composite with an effective modular ratio applied uniformly over the whole bridge deck. This information can be used to model composite slab on girder steel bridges.

Imbsen and Penzien (1979) evaluated the linear response spectrum analysis technique using a non-linear dynamic analysis program for bridges. They recommended that, in elastic analysis, ductility reduction should be made on an individual component basis rather than using an overall structure ductility. They also recommended that seismic design should provide for an increase of approximately 1.5 to 2 in the forces at the abutments derived from elastic analysis if yielding in columns is anticipated. Later, Imbsen and Penzien (1986) compared the nonlinear and linear dynamic analysis results of several multi-span simply supported bridges. They stated that the correlation between elastic and nonlinear analysis results is poor when impacting of the bridge deck occurs. Wilson (1985) studied the dynamic behavior of an existing multi-span highway bridge. He found that a finite element model is able to predict modal frequencies in agreement with measured values only when the expansion joints were assumed to be locked, thereby preventing relative motion between adjacent spans in the longitudinal direction. He stated that such locking may occur possibly as a result of a certain amount of corrosion at the bearing interfaces and an accumulation of windblown debris over a period of years.

The effect of some geometric properties on the seismic response of bridges was also investigated. Research on long curved bridges (Williams & Godden, 1975)

demonstrated that they are potentially very stiff structures in resisting seismic forces compared to straight bridges. However, the curved geometry magnifies the effects of axial forces including tension due to arch action and effects of torsion. Research (Ghobarah & Tso, 1974) on two span skewed bridges supported on intermediate columns and abutments demonstrated that contribution from the torsional response is orders of magnitude larger than the flexural response. An examination of the failure pattern which developed for this kind of bridges during 1971 San Fernando earthquake showed that the outer columns were indeed heavily damaged while the inner columns suffered less damage, confirming the dominance of torsional response.

Considerable research was conducted on the dynamic behaviour of reinforced concrete bridge piers (Ghobarah & Ali, 1988; Priestly, 1985; Priestly, 1988; Priestly & Park, 1987; Saïidi et al., 1988). However, information on dynamic behavior of steel bridges is scarce in the literature. Some of the existing knowledge on the modelling techniques and nonlinear behavior of bridge components will be used throughout this research to generate new information applicable specifically to slab-on-girder steel bridges.

## **CHAPTER 2**

### **EFFECT OF HEAVY PERMIT-TRUCKS: PRELIMINARY SENSITIVITY ASSESSMENT**

#### **2.1 General**

In this study, five vehicle models representative of the extreme gravity loads produced by various types of heavy permit-trucks in Ontario are considered. General properties of the truck models are presented in Table 2.1. In Table 2.2, spacing of the axles and their weights are tabulated for all the trucks. The specifics of these five very different vehicle models have been provided by the MTO, based on their permit-issuing experience. They have been chosen in anticipation that their observed aggregate impact on steel bridges will match that due to a broad spectrum of permit-truck types and configurations. A preliminary sensitivity assessment of the global effect of these loads will be examined in this chapter.

#### **2.2 Methodology**

To determine which spans are most likely to be detrimentally affected by heavy permit-trucks, their global effect on bridges of various spans is studied using simple influence lines concepts. A simply supported and a two span continuous beam are

selected to represent single span and continuous span bridges. Influence line equations of the moment at the midspan of simply supported beam, and at the support and midspan of the continuous beam, are used to calculate the effects of truck models. The span length is kept as a variable parameter in the influence line equations. A computer program is written to obtain the effect of truck models for spans ranging from 1 metre up to 125 metres, the upper limit corresponding to the 1983 Ontario Highway Bridge Design Code (OHBD) stated range of applicability. Although 1 metre is not a realistic bridge span, the results obtained for such small spans can be useful to examine the local effect of heavy permit-trucks on short bridge members such as stringers or cross beams in steel truss bridges.

Each heavy permit-truck is placed alone at its most critical position on the influence lines and the resulting efforts are computed. Similarly, OHBD-truck, OHBD-lane load and, in the case of continuous beam, tailgating OHBD-trucks are individually considered, placed at their most critical position on the influence lines, and the largest calculated resulting efforts are compared with those of heavy permit-trucks.

Two indices are considered to compare the effects of heavy permit-trucks with those of design live loads: The ratio of live load effects and the ratio of total load effects. Total load effect results from the superposition of dead load and live load effects. An average value of 2.25 kN/m per lane is used for dead load in the calculations. It is noteworthy that all comparisons in this section are for a one-lane bridge, i.e. assuming that the lateral distribution of truck loading is critical over an equivalent width of one lane only. The ratio of live load effects is most meaningful when addressing potential fatigue problems; For values greater than one, stress ranges larger than anticipated during design will occur with each heavy permit-truck passage. On the other hand, the ratio of total load effects reflects the erosion of the original safety factor.

### **2.3 Simply Supported Single-Span Bridges**

Centre span moment results are presented. The ratio of live loads and total loads are shown in Figures 2.1 and 2.2 respectively. It is noteworthy that truck model 5 will govern for short spans up to 10 metres, model 1 will govern in the 11-25 metres range, model 2 in the 26-47 metres range, and model 3 thereafter. For very long bridges, the ratio of live loads will fall below one as lane load effect becomes predominant. The most detrimentally affected spans range between 11 to 25 metres for truck models 1 to 4 and 1 to 10 metres for truck model 5. The amplification factor in this first range is 1.25 for live load ratio and 1.15 for total load ratio, whereas in the latter range, the factors can be as large as 1.65 and 1.60.

Based on these results, two different spans appear worthy of considerations. First, the shortest steel bridge span realistically available would be of interest with truck model 5. Then a span of 15-20 metres would be ideal for studies with truck models 1 to 4. Nonetheless, since the detrimental effect of truck 5 is mostly due to its unusually large axle loads, it is anticipated that local problems will also occur in various bridge members irrespectfully of span for this truck model. Therefore, the selection of typical bridges in the 15-20 metres range seems appropriate.

### **2.4 Two-Span Continuous Bridges**

Results for moment at the centre of a span are illustrated in Figures 2.3 and 2.4, while those for the moment over the central support are illustrated in Figures 2.5 and 2.6. For considerations of the moment at centre span, the previously described trends re-occur for all heavy permit-trucks, i.e. the smallest spans are the most sensitive to truck model 5, and truck models 1,2,3 and 4 reach their maximum ratio for spans of roughly 11-25 metres.

For considerations of the moment over the central support, unusual axles spacing plays a more significant role. In fact, truck model 1 does not seem to produce critical results, while models 2, 4 and 5 produce peak results at around 10-12 metres spans with a factor of roughly 1.30, and truck model 3 produces most significant live load amplifications in the 20-29 metres range with a factor of 1.55. Therefore, the study of continuous bridges in the 10-25 metres span ranges seems appropriate to meet the stated objectives.



## **CHAPTER 3**

### **EFFECT OF HEAVY PERMIT-TRUCKS ON BRIDGE MEMBERS**

#### **3.1 General**

In the previous chapter a preliminary study was conducted to determine the range of spans of simply supported and continuous bridges for which heavy permit-trucks could potentially have the most detrimental effects. Five truck models representative of these extreme permit-loads were provided by the MTO. Based on simple influence-line analyses comparing the live-load demand of these trucks with that of the 1983 OHBDC model-truck (OHBD-truck), it was found that permit-trucks 1 to 4 could have detrimental effects on bridges with spans of 15-25 metres, while the potential negative impact of permit-truck 5 was limited to spans shorter than 10 metres; this is true for both simply supported single-span and two-span continuous bridges. Furthermore, it was anticipated that the unusually heavy axle loads of permit-truck 5 could create problems locally irrespective of the span length.

Based on the results obtained in the previous chapter, structural drawings of existing steel bridges of different types and layouts, and of spans within the aforementioned range of interest, were provided by the MTO. Four of these bridges were selected as a representative set. However, they were modified as necessary in order to

investigate the effects of a broader range of parameters. The particulars of each bridge retained and the list of their modifications are presented in Table 3.1.

The Mud Creek Bridge is a simply supported slab-on-girder bridge with a 22 metres span; its curb width has been slightly reduced in both a close-to-original 3 design-lanes construction (Mud Creek Bridge II), and a narrower 2 design-lanes version (Mud Creek Bridge I). The Saugeen River Bridge is a simply supported slab-on-girder 15°-skew bridge with a span of 24 metres (Saugeen River Bridge I); to investigate the influence of skewness on the results, the original 15°-skew was increased to 45° (Saugeen River Bridge II). The South Saugeen River Bridge originally was a 40°-skew three-span continuous slab-on-girder bridge; the skew as well as one-span were eliminated to allow a better understanding of the effects of continuity and possible correlation with the prior influence-line study. Each span has a length of 9.0 metres. Finally, the Grigg bridge is a simply supported through-truss steel bridge with a 28 metres span.

Furthermore, some bridges were found to be either over-designed or under-designed (or grossly under-designed in the Grigg Bridge's case) for the current live-load requirements of the OHBDC, without any consistent patterns. These were therefore re-designed to be in compliance with these requirements, and without excess capacity, i.e. the over-designed bridges were trimmed to meet the minimum strength requirements of the code and the under-designed ones were re-designed and strengthened as necessary. The OHBDC recommended simplified analysis method was adopted for that task.

It is noteworthy that the original character and properties of all bridges were preserved when modified or re-designed.

### 3.2 Methodology

While the semicontinuum method is selected as the preferred analysis technique for modelling the slab-on-girder bridges, in many cases the accuracy of the results is also verified by the grillage analogy method. A typical grillage mesh, here to simulate a two lane simply supported bridge, is shown in Figure 3.1. Comparative studies are conducted, for a two lane, a three lane and a 15° skew simply supported bridges, as well as for a two-span continuous bridge. For the grillage analogy method the program SAP90 (Wilson & Habibullah, 1990) is used, while the semicontinuum method analyses are performed with the programs SECAN1 and SECAN2 (Jaeger & Bakht, 1989). In all cases, the results obtained from both methods are in good agreement. Therefore, all structural analyses thereafter are carried out for slab-on-girder bridges using the semicontinuum programs. It is noteworthy that the curb's contribution to the exterior girder stiffness is considered herein in all analyses.

As normally done for this type of studies for the ultimate limit state, each of the five permit-trucks and the OHBD-truck is placed on the bridges at the longitudinal position producing the maximum moment at a section of interest along the span. Each is also moved laterally across the deck to obtain the maximum interior and exterior girder moments. To obtain the maximum moment in exterior girders, heavy permit-trucks are placed on the bridge in such a way that their wheels touch the curb. By contrast, OHBD-truck is placed on the deck by assuming that it remains entirely within its design lane, i.e. the wheel centerline can not be closer than 60 cm. from the curb, the barrier wall or the next lane.

For serviceability limit state I (fatigue), only one OHBD-truck is placed at the centre of a travelled lane and at the critical longitudinal position, as required by the OHBDC, to calculate the maximum stress-ranges. In particular, to determine the stress-ranges at mid-span of the two span continuous bridge for each of the five permit-trucks and OHBD-truck, the longitudinal positions respectively producing the maximum positive

and negative moments must be both considered; the maximum stress-range is established by summing the absolute values of the stresses resulting from maximum negative and positive moments. Hereafter, for simplicity and consistency, "moment-ranges" will be considered, these being implicitly proportional to stresses.

For both limit states, all the bridge analyses for OHBD-truck are also conducted using the semicontinuum program to confirm the results obtained from the code procedure.

The bending moments as obtained for each truck versus that predicted by the OHBDC recommended simplified analysis method for both exterior and interior girders are selected as meaningful parameters to compare and assess the significance of each load condition and identify the cases for which detrimental effects are anticipated. For the slab-on-girder steel bridges studied, girder sizes are usually dictated by flexural-design requirements rather than by shear considerations. It is found here that the shear strength of the girders typically exceeds by at least fifty percent the design shear force, thus making shear distress improbable even when the bridges are loaded by heavy permit-trucks. Furthermore, as most connections are sized to transmit shear, these findings can be broadly extended to the details. Finally, the reinforced concrete bridge deck's adequacy in resisting local overloads by arching action is well documented and is therefore not questioned either (Batchelor et al., 1978; Batchelor & Hewitt, 1978; Perdikaris et al., 1989).

Therefore, only bending moments are considered to investigate the potential negative impacts of heavy permit-trucks on steel slab-on-girder bridges relative to the effect of OHBDC design loads, but, understandably, a more in-depth survey for all members of the truss bridge is warranted, and performed.

### 3.3 Analysis of the Selected Bridges and Presentation of Findings

#### 3.3.1 Comparative Indices Selected

It was previously indicated that the bending moments are the selected parameters for all subsequent comparison of the effect of various extreme truck load conditions. Thus, the absolute expression of the maximum exterior ( $M_e$ ), interior ( $M_i$ ) girder moments and maximum moment range over any girder ( $M_r$ ), are calculated for each of the five heavy permit-truck models, as well as for the OHBD-truck as obtained from computer analyses (OHBDT) and from the code's simplified method of analysis (OHBDC). It is noteworthy that for the OHBDT and OHBDC analyses, the  $M_r$  values are for truck travelling in the middle of their design lanes to model fatigue under high volume of traffic, but that permit-trucks, fewer in numbers, are still considered at their worst lateral positions. Furthermore in violation of the OHBDC, load factors of 1.0 (instead of 0.8) are considered throughout, for all trucks. However, to provide additional perspective, comparison will also be made between the fatigue results for permit-trucks considered to travel in the middle of the design lanes and OHBD-truck using a load factor of 0.8 (in compliance with the OHBDC requirements).

However, as seen in Tables 3.2 to 3.10 where all findings from this phase of the research are reported, a number of additional ratios can be calculated, each expressing a particular aspect of worthwhile consideration. It is the purpose of this subsection to systematically review all of these expressions which will be used hereafter.

The  $M/M_{OHBDT}$  ratio informs on the respective static demands requested by each of the various trucks as compared to that of the OHBDC; It also allows comparison with the results previously obtained by influence-line analyses.

To verify whether the serviceability limit state has been violated, the ratio  $((DMF \cdot M_r) / (DMF \cdot M_r)_{OHBDT})$  is more realistic, where DMF is the dynamic magnification

factor. As permit-trucks travel at low speeds, their DMFs are typically smaller than those used for OHBDC designs. Values of this ratio exceeding 1 (Table 3.2) are indicative of potential serviceability problems. For information only, the ratio  $((DMF \cdot M_L) / (DMF \cdot M_L)_{(OHBDC)})$  is also calculated (Tables 3.3 to 3.9) for the ultimate limit state.

The protection against ultimate-strength failure can be gauged in two ways: (a) the ultimate limit state equation  $\alpha_D \cdot M_D + \alpha_L \cdot DMF \cdot M_L \leq \phi M_R$ , with  $\alpha_D$  taken as 1.2, can be reversed to determine the effective live load factor ( $\alpha_L^E$ ) available when all other parameters are known, i.e.  $\alpha_L^E = (\phi M_R - 1.2 \cdot M_D) / (DMF \cdot M_L)$ ; (b) the absolute safety margin available if all uncertainties including those addressing the variability of the dead load and material resistances can be transferred into a global safety factor,  $SF = M_R / (M_D + DMF \cdot M_L)$ . The former is more consistent with a Limit States Design formulation while the later, comparable to a Working Stress Design philosophy, is still often used by practising engineers in similar situations.

The effect of heavy permit-trucks on the ultimate limit state of the bridges' exterior girders is also shown in Figures 3.4a and 3.4b in a bar-chart form. There, the bars measure the effect of the factored loads applied to the structure, i.e.  $\alpha_D \cdot M_D + \alpha_L \cdot DMF \cdot M_L$  where  $\alpha_D=1.2$  and  $\alpha_L=1.4$  in Figure 3.4a as commonly used in design, whereas in Figure 3.4b  $\alpha_L$  is assigned a value of 1.15 as suggested by clause 2-5.4.2 of the 1983 OHBDC when there is a better supervision on vehicles such as heavy permit-trucks (controlled vehicle) compared to ordinary trucks (trucks that do not need any specific control). In these figures the dashed line indicates the level of supplied ultimate strength ( $\phi M_R$ ) of the bridges. Given the constraints of design, the supplied capacity is often in "over-strength" when compared to the demands. Thus, Figures 3.4a and 3.4b also allow for a direct comparison between the permit-truck loading levels and that of the original design. Similarly in Figures 3.5a and 3.5b, the effect of heavy permit-trucks on the serviceability limit state of the bridges, as expressed by the previously defined

$((DMF \cdot M_r)/(DMF \cdot M_r)_{OHBDX})$  ratio is illustrated. Values above 1 (dashed line) indicate a violation of the serviceability limit state I (fatigue).

As demonstrated by the influence-line analyses conducted in the previous chapter, the potential detrimental effects attributable to each of the heavy permit-trucks, when compared to the effect of OHBD-truck, is also a function of the span length of the bridge under consideration. In other words, each heavy permit-truck has a largest possible effect on a certain span length defined as the critical span length associated to that truck. Therefore, it is possible, and desirable, to directly magnify the static ( $M/M_{OHBDX}$ ) and dynamic  $((DMF \cdot M)/(DMF \cdot M)_{OHBDX})$  ratios of demand corresponding to each of the various permit-trucks for the six bridges under investigation, to obtain a preliminary assessment of the worst possible conditions that could prevail if the bridges were of their critical span lengths. The magnification factors necessary to meet this goal are presented in Table 3.10 for each truck and all bridges. They are obtained by dividing the  $M/M_{OHBDT}$  ratios at critical span by the corresponding ones at current span. The resulting values of  $M_c/(M_e)_{OHBDX}$  at critical spans, which are obtained by multiplying the previously obtained  $M_c/(M_e)_{OHBDX}$  at current span by those magnification factors, are also presented in that Table. These values are indicative of the largest possible negative impact of each heavy permit-trucks on the exterior girders of the bridges having lengths equal to the critical spans associated to each truck.

### 3.3.2 Mud Creek Bridge (MCB) I and II

The original MCB when evaluated according to the OHBDC was found to be over-designed. Therefore two trimmed-down versions of the MCB for which the girders and cover plates on the bottom flanges have been resized, are used: a 3-design-lanes version (MCB-II) and a narrower 2-design-lanes one (MCB-I).

In order to investigate the effect of the large overhang for this particular bridge, in both cases a curb width smaller than that of the original bridge curb is used. This allows for some truck wheels to be on the cantilever part of the bridge deck.

Two different models are also considered when analyzing the MCB-I in order to investigate the effect of curb stiffness. In Model 1, the curb's contribution to the exterior girder stiffness is taken into consideration, whereas in Model 2 the curb stiffness is neglected and exterior girders are assumed to have the same stiffness as the interior girders. Structural analysis of both models is carried out for the OHBDC's truck loading and the variation of exterior and interior girder moments as a function of the distance from the centerline of the truck's closest wheel-line to the edge of the bridge deck are plotted in Figures 3.2 and 3.3. As seen in these Figures, for OHBD-truck with wheels at the minimum edge distance permitted by the OHBDC, the exterior girder moments are larger for the first model than for the second one (1183 kN-m vs. 1143 kN-m i.e. 4% more), whereas the reverse is true for the interior girder moments (657 kN-m vs. 739 kN-m i.e. 12% less). While Model 1 is more realistic and retained for the following analyses, Model 2 is implicitly equivalent to that adopted by the OHBDC simplified analysis procedure.

The bridge is then analyzed for each of the five heavy permit-truck loads using the semicontinuum program. The results of these analyses for the MCB-I and MCB-II are presented in Tables 3.3 and 3.4 for the ultimate limit states, and in Table 3.2 for the serviceability limit state I. Maximum exterior ( $M_e$ ) and interior ( $M_i$ ) girder moments due to OHBD-truck loading as obtained from computer analyses (OHBDT) and from the code's simplified method of analysis (OHBDC) are also presented in Tables 3.3 and 3.4. It is noteworthy that when comparing these later two values, the simplified code method is seen to predict smaller exterior girder moments and larger interior girder moments than those obtained by the more accurate computer analyses. This large difference can be rationalized partly from : (a) the fact that in the OHBDC simplified analysis procedure,



the curb's contribution to the exterior girder stiffness is neglected and all the girders are assumed to have the same stiffness, and (b) the unusually large overhang of the bridge.

It can be seen from Tables 3.2, 3.3 and 3.4, as well as from Figures 3.4 & 3.5, that heavy permit-trucks have a slightly larger impact on the MCB-I than on MCB-II, but for all practical purposes results are almost identical. The serviceability limit state on the exterior girder is violated for permit-trucks 1 to 4, the worst offenders being permit-trucks 1 and 4 with dynamically magnified live load moments respectively 1.34 and 1.28 times larger than those obtained from the code. No such problems occurred on the interior girders. It is also observed that, the resulting effective live load factor,  $\alpha_L^E$ , is smaller than the code specified value of 1.4 for all permit-trucks except truck 5, the smallest value being 1.13 for permit-truck 1. However, as the weights of permit-trucks are more accurately known and better controlled, thus more reliable than ordinary trucks, these small safety margins could be acceptable and this is reflected in Figure 3.4b where a live load factor of 1.15 is used for all permit-trucks. Rather in light of these comments, the similarly low value of  $\alpha_L^E=1.15$  obtained for the OHBD-truck is a lot more worrisome; it reveals a failure of the OHBDC simplified analysis method to accurately predict the live load demand for its own truck model.

Truck model 5 is the only truck which does not have any detrimental effect on these two bridges. This is in agreement with the influence-line results obtained in the previous study.

Also as the length of these bridges is close to the critical spans associated with permit-trucks 1 to 4, as can be observed from Table 3.10, then the value of the corresponding magnification factors are modest and the largest detrimental effect on the bridges remain essentially unchanged. However, the magnification factor due to truck model 5 is quite large and as a result, exterior girder moments could be up to 1.79 times those corresponding to the OHBDC for MCB-I. Fortunately, the critical span for permit-truck 5 (4 metres) is of no engineering interest. In fact, for spans of 10 metres and larger,

the effect of permit-truck 5 should be no more than that of truck 1. Yet caution is advisable in this case when dealing with progressively smaller spans.

### **3.3.3 Saugeen River Bridge (SRB) I and II**

The SRB-I, the original 15°-skew version of that bridge, was found to have an ultimate strength capacity in excess of that needed when evaluated according to the OHBDC. However, the stress ranges limits corresponding to the fatigue rating for the main girders of that bridge were reached under application of the design-truck live load. Therefore, the performance of this bridge is governed by, and in compliance with, the serviceability limit states I; no re-design was needed. This feature was preserved for the 45°-skew version of this bridge (SRB-II).

Structural analyses of both bridges are carried out for each of the five heavy permit-trucks and the OHBD-truck loads. The results of these analysis for SRB-I and SRB-II are presented in Tables 3.5 and 3.6 respectively for the ultimate limit state and in Table 3.2 for the serviceability limit state I. Again, the moments predicted by the OHBDC simplified analysis procedure are smaller for the exterior girders, and larger for the interior girders, than those obtained by computer analyses for the OHBD-truck. These differences may be partly attributed to the neglect of the curbs' contribution to the exterior girders stiffnesses in the simpler analysis method, but in light of the severe disagreement in these results, it may also largely be a consequence of the skewness of the bridges which is not considered by the OHBDC.

It is also observed that permit-trucks 1, 2 and 4 have the largest detrimental effect on the bridge. Exterior girder moments due to these trucks are 1.22 to 1.27 times larger than the moments obtained using the OHBDC procedure. Although, when the DMF is taken into account, the ultimate strength ratio drops to 1.09 for trucks 1 and 2 and falls

below 1 for truck 4, the ratios remain high for the serviceability limit state I, up to 1.20 for trucks 1 and 2. This is also visually confirmed in Figures 3.5a and 3.5b.

Although the exterior girder live load moments due to the permit-truck loads are generally larger than those obtained from the OHBDC, compliance to the ultimate strength limit state is easily achieved. Since fatigue was the limiting condition in the original design, the bridges already had reserve strength to spare; this can also be observed from Figures 3.4a and 3.4b as well as from the tabulated values of the effective live load factor  $\alpha_L^E$ , all larger than the code specified value of 1.4, thus providing a superior safety margin.

Magnification factors in Table 3.10 are rather modest except for permit-truck model 5. Although this truck does not have any detrimental effect on these 24 metres span bridges, at the critical span length it would produce the largest of all permit-trucks effects. However, a span length of 10 metres and larger would produce an overall demand no worse than that of the OHBD-truck.

### **3.3.4 South Saugeen River Bridge (SSRB)**

The original SSRB is a three-span continuous skew bridge. It was modified into a two-span continuous right bridge and redesigned. In that process, in order to calculate the support and mid-span moments of the exterior and interior girders, equivalent simply supported span lengths of  $0.2(2L)$  and  $0.8L$  were assumed for the negative and positive moment regions respectively in accordance with the OHBDC simplified design procedure.

Structural analysis of the bridge is carried out for each of the heavy permit-trucks and OHBD-truck loads and results are presented in Tables 3.2, 3.7 and 3.8 for the support ( $M^-$ ) and mid-span ( $M^+$ ) moments. For all girders, support and mid-span moments obtained from computer analyses are smaller than those predicted by the OHBDC

simplified analysis method. This difference is due to a combination of the large size of the curb, the small length of the overhang, and the conservative approximation recommended by the OHBDC that the negative and positive moment regions of the continuous bridge be modelled as two separate simply supported bridges.

The SSRB is particular in that it has a curb width of 1.0 metre resulting in the placement of the closest wheel-line of the OHBD-truck 1.60 metre (1.0 m. curb + 0.60 m. distance from curb to centre of tire) away from the bridge's edge. Since the simplified procedure of the OHBDC has been developed assuming a minimum curb width of 0.45 metre and considering a large number of combinations of truck loading, many of which positioned as close as possible from the edge to provide a conservative and safe design criteria (Bakht et al., 1979), it should not come as a surprise that when this code procedure is applied to analyze bridges having wide curbs, it may give conservative results. To illustrate this, the variation of exterior girder positive moment at mid-span of the SSRB is plotted in Figure 3.6 as a function of the lateral position of the first wheel line of the OHBD-truck. When placed at the minimum code specified distance of 1.05 m. (0.45 m. curb + 0.60 m.) away from the edge, the exterior girder moment is 244 kN-m; when placed at the closest distance physically possible of 1.6 metres, the resulting moment decreased to 180 kN-m. This is consistent with values presented in Table 3.8.

As for the moments on the interior girders, differences between values predicted by the OHBDC simplified method and the more accurate computer analyses can again be explained by the neglect in the former of the curb's contribution to stiffness and its influence on the lateral distribution of efforts, as discussed in a previous section. As seen in Tables 3.2, 3.7 and 3.8, none of the permit-trucks have detrimental effect on the bridge when compared with the OHBDC values, and only trucks 3 and 4, for the interior girders negative moment only, are close to OHBD-truck values. Figures 3.4 and 3.5 confirm this.

However recall that, in Figures 3.4a and 3.4b the cross hatched bar shows the ratio of total design moment to the *exterior* girder's factored moment capacity; it is quite below

unity in this case since the *interior* girder governs the design, i.e. for economical reasons, all girders are selected to be identical across the bridge and are sized here based on the dominant interior girder moments.

### 3.3.5 Grigg Bridge

The original bridge, when evaluated according to OHBDC, was found to be severely under-designed and had to be redesigned before being analyzed for each one of the five heavy permit-trucks. Great care was taken to preserve the character of the bridge, especially with regard to built-up sections.

The structural analysis of the bridge is performed for each of the heavy permit-trucks, and all the resulting axial forces in all members of the truss are compared with those predicted by the OHBDC. Although these results are not presented, for conciseness, it is found that none of the truck models have a detrimental effect on any of the truss members.

It is noteworthy that, for this two design-lanes bridge, the load transferred to each truss panel by the critical placing of two OHBD-trucks is larger than that transferred by any of the five permit-truck models. Moment and shear forces in cross beams due to the OHBD-truck are also found to exceed, for similar reasons, those due to the heavy permit-trucks. Although for a one-lane bridge the reverse would have been true (i.e. the axial forces produced by heavy permit-trucks would have exceeded those for the OHBD-truck) it is believed that single-lane bridges are unlikely to be found on routes for which permits are issued.

Stringers (i.e. the simply supported beams spanning between cross beams) are found to be the most vulnerable structural elements in the through-truss bridge (span of the stringers is 4.68 m.). Traditional analysis of statically determinate floor-systems can

be used here to calculate the effects in each stringer under various load conditions, but, alternatively, the portion of the deck between each cross beam can be also modelled as an independent simply supported small slab-on-girder bridge which can be analyzed by the semicontinuum method. Using that later technique, structural analysis of the stringers is carried out for each of the five heavy permit-trucks and OHBD-truck loads as well as by the simplified OHBDC procedure. The ensuing results are presented in Tables 3.2 and 3.9. Here, the results of both the more accurate computer analysis and the simple OHBDC procedure are in excellent agreement. As expected in the light of the stringer's short spans and known results from previous influence line analyses, permit-truck 5 has the most severe effect on the stringers. For ultimate limit states, the absolute exterior girder moment due to truck 5 loading is 1.49 times the OHBDC result; the ratio drops to 1.23, if also considering dynamic effects in the serviceability limit state I as illustrated in Figure 3.5a. Considering the 0.8 load factor used for the OHBD-truck in the serviceability limit state I, the absolute girder moment due to truck 5 loading becomes 1.54 times the OHBDC result as illustrated in Figure 3.5b. This demonstrates the potential for fatigue problem in the stringers.

With regards to ultimate strength limit states, an existing strength reserve for the exterior stringer is adequate to safely resist the larger demands as shown in Figures 3.4a and 3.4b. For the same economical and practical reasons previously mentioned, all stringers are made identical during design. As interior stringers attract larger moments, this explains the existence of this reserve capacity for the exterior stringers. Consequently, none of the permit-truck violate the ultimate strength limit state for this particular truss bridge type.

Finally, the large numbers of Table 3.10 would be misleading in this case; the magnification factors for permit-trucks 1 to 4 would apply to spans unlikely to be encountered for stringers (e.g. 7 metres and more to produce a magnification factor exceeding 1), while any decrease in stringer's span would not increase the results obtained for permit-truck 5 by more than 5%.

### 3.3.6 Additional General Aspects

Most general findings from this limited investigation have already been reported in the previous sections for each of the bridges studied. A few additional comments and observations are presented as follows.

- All above observations were made assuming that the heavy permit-trucks travelled at their permitted low speeds and that the DMF equation proposed by the OHBDC was applicable to these vehicles. Obviously, the permit-truck's DMF are smaller than those used for the faster OHBD-truck, which positively affects the results. Both the compliance of permit-truck drivers with speed restrictions and the reliability of the DMF in this situation are debatable, although the results presented herein are sufficiently complete to allow an assessment of alternative scenarios.
- The emphasis so far has been placed on comparing various internal efforts produced by the application of heavy permit-truck loads with those predicted by the OHBDC simplified analysis procedure. Yet, some engineers may be inclined to use computer programs to more accurately analyze and design bridges. The potentially large difference in results obtained by the two methods has been well demonstrated above. As a result of this, designs may differ. A detailed re-interpretation of the findings, where analysis results for heavy permit-trucks would be compared instead with those more accurately obtained for the OHBD-truck load, has not been conducted. However Figures 3.4 & 3.5 offer a visual guide of the consequences this would have on the safety of the bridges.
- For bridges having more than one design-lane, two OHBD-trucks can be placed closely together with their adjacent wheels loading an interior girder to produce moments much larger than those resulting from a single heavy permit-truck.

Therefore, the absence of detrimental effects due to heavy permit-trucks on the interior girders of slab-on-girder steel bridges should not be surprising.

- All results herein pertain to permit-trucks located in their most unfavourable, and somewhat improbable, lateral position. Potential problems attributable to permit-trucks will progressively be attenuated as they are moved further away from the bridges' edge. This is illustrated in Figure 3.7 for the MCB-I bridge. In this figure, the distance of the first wheel-line of each permit-truck from the bridge edge is determined such that the exterior girder moment due to each truck is equal to the moment due to OHBD-truck for this particular bridge. The wheel-line of permit-trucks 1 to 4 should be placed at least 1.64, 1.28, 1.61 and 1.33 metres from the edge respectively to obtain exterior girder moments smaller than design moments. In this particular case, the span of the bridge considered is such that permit-truck 5 does not have any detrimental effect irrespective of edge distance. Obviously, for ultimate limit states, by considering a reduced live load factor of 1.15 as commonly done for controlled vehicles, trucks could be placed closer to the edge; For serviceability limit state I, where a 0.8 live load factor is applied to the design truck, thus reducing the design moment by 20 percent, permit-trucks would need to be further away from the bridge's edge to produce moments smaller than the design moment.
- The extremely heavy wheel loads but moderate overall weight of permit-truck 5 put it in a class of its own with detrimental effects only on bridges shorter than 10 metres. Although slab-on-girder bridges of such spans are unusual, stringers in steel truss bridges are most often less than 10 metres. From this limited study, stringers are found to be the only structural members affected by permit-truck model 5.
- The above study has been concerned with bridges in compliance with the 1983 edition of the OHBDC. Should the strength of an existing bridge be below that



required by current standards, which is not unlikely given that some of the bridges considered here were originally found to be under-designed by as much as 40%, ultimate strength and/or serviceability distress may appear or may further be compounded if already existing. Alternatively, in an over-designed bridge, both the ultimate strength and/or fatigue may not be critical, even if they are found to be critical in a code-compliant version of the same bridge. Therefore, beyond the comparison of heavy permit-trucks and OHBD-truck demands, a knowledge of the bridge's existing conditions is most important.

- The use of high strength steel for slab-on-girder bridges translates into smaller girder sizes in the design process. This, in terms, implies that fatigue considerations (i.e. serviceability limit state I) are more likely to dictate the sizing of those girders than the ultimate strength limit state. In such cases, bridges may have an ultimate strength reserve adequate to safely accommodate loads larger than anticipated during the original design, while being simultaneously excited by stress ranges exceeding the critical fatigue limit when loaded by trucks heavier than the OHBD-truck.

### 3.4 Summary

Six bridges were evaluated according to the 1983 edition of the OHBDC. Some of them were found to be in compliance with the code, others under-designed or over-designed. As needed the bridges were strengthened or trimmed to meet the minimum requirements of the OHBDC, all while preserving their unique character and features.

These bridges were then analyzed as loaded by five models of heavy permit-trucks and the ensuing results were compared with those obtained using the simplified method of the OHBDC, as well as more accurate computer analyses conducted for direct application of the OHBD-truck model.

None of the permit-trucks considered produced detrimental effects on the interior girders of the bridge studied, i.e. the internal efforts resulting from the application of the permit-truck loads were less than those predicted by the OHBDC simplified design procedure. As for the exterior girders, the following can be concluded with regards to the ultimate limit state. For the slab-on-girder bridges studied:

- The absolute static live load bending moments produced by the permit-trucks are often more than both those obtained from the OHBD-truck or predicted by the OHBDC simplified design method procedure. But, as illustrated in Figure 3.4b, when the dynamic magnification and live load factors for controlled vehicles are considered, heavy permit-trucks do not produce any detrimental effect with regard to ultimate strength.
- When taking the dynamic magnification factors proper to each truck into consideration, the demand for the permit-trucks can still exceed that of the OHBDC-level, but never exceeds significantly that of the OHBD-truck as illustrated in Figure 3.4b.
- The most critical effects of permit-trucks is found to occur on bridges having a combination of small curb-width and large overhang.
- On average, permit-trucks 1 to 4 have the most effect on the bridges studied. Permit-truck 5 will have detrimental effect for spans shorter than 10 metres; for longer spans its effect is less than that of permit-truck 1 and OHBD-truck. Caution is advisable when dealing with progressively smaller spans.
- Potential problems attributable to the permit-trucks will progressively be attenuated as they are moved further away from the bridges' edge.

- The above conclusions will remain valid for under-designed bridges. Hence, detrimental effects incurred due to permit-truck overloads will be no worse than those produced by the OHBD-truck.
- Bridges are likely to have a favourable reserve capacity when the design is governed by fatigue.

For the Grigg truss-bridge:

- Only the stringers are detrimentally affected by the permit-trucks.
- Other bridge components would also be affected if the bridge had one lane. However, it is believed that single lane bridges are unlikely to be found on routes for which permits are issued.
- The absolute static live load bending moments, in the stringers, produced by permit-trucks 1 to 4 are less than those obtained from the OHBD-truck or OHBDC.
- Permit-truck 5 is found to produce bending moments, in the exterior stringers, in excess of those corresponding to both the OHBD-truck and the OHBDC level. However, the stringers of the re-designed Grigg Bridge still have a sufficient capacity to accommodate this higher demand.

In addition, for the exterior girders, the following can be concluded with regards to the serviceability limit state I. For the slab-on-girder bridges studied:

- When located in their worst lateral position, most permit-trucks will produce moment-ranges (i.e. stress-ranges) larger than those obtained from the OHBD-truck or predicted by the OHBDC simplified design procedure.

- For continuous bridges, the conservative assumptions built into the OHBDC simplified design method result in a design apparently adequate to overcome the effect of permit-truck, as illustrated by the results for the SSRB.

For the Grigg truss-bridge:

- Again only the stringers, and only permit-truck 5 are of a concern for this bridge.
- Permit-truck 5 is found to produce dynamic bending moments, in the exterior stringers, largely in excess of both those corresponding to the OHBD-truck and the provided OHBDC design.
- Fatigue distress would be anticipated to occur more rapidly under permit-truck 5 loading than under the OHBD-truck, if the bridge stringers were under-designed relative to the 1983 OHBDC.

As the weight of heavy permit-trucks is more predictable, the live load factor of 1.5 used in original designs produces a strength reserve enabling the bridges to handle the effects of unusual infrequent overloads. Therefore, ultimate strength is not a problem for the slab-on-girder and truss bridges studied. However, fatigue is found to be critical. The design live load factor of 0.8 applied to the OHBD-truck increases the relative effects of heavy permit-trucks and even those of the trucks in normal traffic, subsequently creating more severe fatigue related problems.

## **CHAPTER 4**

### **EFFECT OF VARIABLE AMPLITUDE LOADING ON FATIGUE STRENGTH OF BRIDGE COMPONENTS**

#### **4.1 General**

In the 1983 edition of the OHBDC (Ontario 1983a), an entire section is devoted to the design of steel and aluminium components subjected to repeated application of live loads. Typically, a steel bridge's structural elements and details are designed on the basis of an allowable stress range specified to ensure a satisfactory protection against fatigue-failure. This allowable stress is a function of the number of stress cycles which are likely to be sustained by bridges during their service life as specified according to average daily truck traffic (ADTT). Consequently, the computed stress ranges at each location and detail of interest should be less than their corresponding allowable stress ranges. The resulting compliant steel bridge members and details are assumed to have satisfactory fatigue life.

In Chapter 3, several bridges having spans for which heavy permit-trucks could have nearly the most detrimental effects, were selected. Structural analyses of the bridges were carried out for the truck model recommended by the Ontario Highway Bridge Design Code (OHBD-truck) and five heavy permit-truck models. The semi-continuum analysis technique was used. The stress ranges obtained from these analyses

for each of the five heavy permit-trucks were compared with the design and OHBD-truck stress ranges to identify potential problems regarding the Serviceability Limit State (SLS) I and Ultimate limit State (ULS). For the SLS I, or fatigue limit state condition, it was found that, in some of the bridges, the maximum stress ranges produced in major structural components by the passage of heavy permit-trucks exceeded those due to the OHBD fatigue-truck. This OHBD fatigue-truck model is essentially identical to the OHBD-truck but multiplied by a load factor of 0.8; it represents an average truck train in Ontario. In some cases, the calculated stress ranges due to the heavy permit-trucks considered exceeded the OHBD fatigue-truck's stress ranges by approximately 50%. Conceivably, this could affect the fatigue-life of steel bridge elements, particularly if the design stress range is close to the fatigue limit.

Therefore, to determine the effect of occasional passage of heavy permit-trucks on the fatigue life of existing bridges, a literature review was conducted to appraise the state-of-knowledge on the impact of infrequent stress-range excursions beyond the design limits, particularly when these designs aim at a theoretically infinite fatigue life. Findings from the work are presented in this chapter.

## **4.2 Fatigue**

Fatigue is defined as the failure of a material under repeated or cyclic loading. It is the result of crack propagating from some discontinuity, initial defects or cracks, in the material. Near the discontinuity, internal stresses produced by the loading are larger than in the surrounding material. Hence, local yielding may occur at the discontinuity although the remaining parts of the material are well below the yield stress. Yielding in steel typically produces slip planes at the microscopic level. During the first few cycles of plastic flow, the atoms on each side of the slip plane form new bonds without losing strength in a measurable degree. However, under continued cycling, microscopic sharp cracks will be generated along the slip planes. Once such a crack has been initiated, the

stress concentration at its end promotes further crack growth as the cycling continues. Under the biaxial stress field at its tip, the crack develops in the plane of maximum shear stress to a size detectable by eye. After the initial crack has grown to an appreciable size, the failure process enters a second phase. In this phase the crack continues to spread but travels in a plane normal to the maximum tensile stress. When the uncracked area of the cross section has been reduced in size to the point where it can no longer carry the load, sudden and complete failure takes place (Rolfe and Barsom 1987, Bowes et al. 1984).

#### **4.2.1 Fatigue Damage under Constant Amplitude Loads**

Most fatigue failures in the laboratory have been produced by subjecting a test specimen to a number of approximately sinusoidal stress cycles in which the peak stress amplitude is maintained constant throughout the life of the specimen. The same test is repeated for different peak stress magnitudes. The data generated from such tests when plotted in logarithmic scales, provides a relationship of the form shown in Figure 4.1, typically referred to as S-N curves, i.e. stress range amplitude (S) versus number-of-cycle to failure (N) curves.

As seen in Figure 4.1, the strength under constant amplitude cyclic load is generally less than the ultimate strength  $S_u$ . As the stress is decreased, the number of cycles required to cause failure increases until a stress-range level is attained below which failure does not occur regardless of how many cycles are applied.

It is noteworthy that the tri-linear S-N curve of Figure 4.1 is typically constructed, for the base metal properties and for small size test specimens only, by the following empirical procedure: (a) At N equals one cycle, the specimen obviously fails when ultimate strength is reached. (b) Based on empirical evidence, at  $10^3$  cycles the allowable stress range can be taken as  $0.9S_u$ . (c) At  $10^6$  cycles and beyond, the allowable stress range remains constant: This is called the endurance limit or constant-amplitude fatigue

limit. For a very wide range of steels the value of the endurance limit is close to one-half the ultimate strength. The above procedure defines an ideal S-N curve for the mean fatigue life of the base metal only.

For small size test specimens, this mean S-N curve should be modified to take into account the different factors affecting the fatigue strength, namely: size of specimen, surface finish, notch sensitivity, mean stress, desirable reliability levels and other conditions. (Rolfe and Barsom 1987, Bowes et al. 1984). Although, for civil engineering practical applications using full size specimens, most of the factors are known to have no influence on fatigue behavior, for the sake of completeness, they are still briefly reviewed following.

#### **4.2.1.1 Size of Specimen**

If a fatigue test is conducted under the same stress range for two specimens having different sizes, the volume of the highly stressed region in the larger specimen is more than that of the smaller specimen. Thus, the probability of significant flaws occurring in the highly stressed region is greater in large specimens, and their fatigue strength is correspondingly smaller. Hence, fatigue strength decreases as the size of the specimen increases (Fisher and Mertz 1983).

#### **4.2.1.2 Surface Finish**

Tests were carried out in the past on specimens of the same material having different surface finishes. It was found that the endurance limit is strongly dependent on that finish. Since a rough surface introduces depressions that cause stress concentrations, a reduction in fatigue strength is expected in specimens having rough surfaces. For two small size test specimens of equal surface roughness, the reduction in fatigue strength



also varies with the strength of the material, i.e. high strength materials are more influenced by the surface roughness than the low-strength materials. However, this trend has not proven to be true for full scale beam specimens and details typically used in bridge construction. (Fisher et al. 1970, Fisher 1974).

#### **4.2.1.3 Notch Sensitivity**

Notch sensitivity is a measure of the reduction in strength of a metal caused by the presence of a notch. For instance, low grade cast iron is quite insensitive to notches; this material has so many built-in stress raisers that additional holes and notches have little influence on the fatigue life. Increasing notch sensitivity decreases the fatigue-strength. For small size test specimens the notch sensitivity was found to depend on the ultimate strength, i.e. as ultimate strength increases, so does notch sensitivity, and fatigue-strength decreases. However, here again, this trend has been demonstrated not to occur for full scale steel beam specimens (Fisher et al. 1970, Fisher 1974).

#### **4.2.1.4 Mean Stress**

The magnitude of the mean stress has a considerable effect on the fatigue life of small test specimens. As the mean stress increases, the fatigue life of the specimen decreases. A linear relationship is often used to relate the reduction in fatigue life to the increase of mean stress: Typically, the maximum fatigue life calculated for the zero mean stress decrease linearly to zero when the mean stress increases toward the ultimate strength. However, for full scale beam specimens, all existing test results indicate that the mean stress has again no effect (Fisher et al. 1970, Fisher 1974).

#### **4.2.1.5 Desirable Reliability levels**

The data used to construct the S-N curve has a considerable amount of scatter. This scatter increases as the number of cycles increases along the S-N curve. Should the mean S-N curve be used in a design process to estimate a particular fatigue life at a given stress range, there would be a 50% chance that the resulting structure would not survive for the required number of cycles. Hence, a 95% confidence S-N curve is usually adopted by design codes to ensure adequate safety and consistent levels of reliability in calculating the safe life of a structure.

Other factors, such as residual stresses caused by hot rolling or welding, operating temperature and corrosion also have an influence on fatigue life. These factors should be considered whenever necessary.

#### **4.2.2 Fatigue Damage Under Variable Amplitude Loads**

Live loads due to traffic on bridge structures do not produce stress ranges of constant magnitude. For a given bridge, the maximum values of the stresses in the various bridge elements fluctuate as vehicles of different weights and axle configurations pass over the bridge. Clearly, the traffic load on bridges is of a random nature.

In past surveys (Agarwal and Wolkowicz 1976), statistical data was gathered about the weight and axle configuration of the truck population in Ontario. From this data, a probability density histogram of the efforts in structural members generated by trucks was constructed by passing those truck samples over several influence lines. The probability density function found to best fit the heavy portion of the truck population is the Rayleigh distribution. This function constitutes a variable amplitude stress spectrum for which various constant amplitude stress ranges are assigned a corresponding frequency of occurrence. In other words, the function can be decomposed into  $m$  groups of constant-

amplitude stress ranges. During a given time interval there will be  $n_1$  cycles of stress range  $S_1$ ,  $n_2$  cycles of stress range  $S_2$  and  $n_i$  cycles of stress range  $S_i$  where the  $n_i$ 's are obtained from the corresponding frequency of occurrence of each stress range. Typically, a few cycles of one stress level will be followed by a few cycles of another stress level, in a random manner. Some experiments were conducted to show that the sequence of application of these stresses does not have an effect on the fatigue life (Schilling et al. 1978), if none of the applied stress ranges are below the endurance limit as will be demonstrated in a later section.

To check for safety, each stress range can be considered separately, and the number of cycles of that stress which could be carried safely can be calculated using the S-N curve. Thus,  $S_1$  could be carried safely for  $N_1$  cycles,  $S_2$  for  $N_2$  cycles and  $S_i$  for  $N_i$  cycles. In other words,  $N_i$  is the number of constant-amplitude-cycles of stress range  $S_i$  as read from the S-N curve. For variable-amplitude-loading, according to stress spectrum, if  $S_i$  is applied only  $n_i$  times then only a fraction  $n_i/N_i$  of the constant-amplitude-fatigue life of a structure has been consumed. The application of  $n_i$  cycles of stress range  $S_i$  will damage the structure by the ratio  $n_i/N_i$  of its life. Therefore, the fatigue damage from all stress ranges acting on a structure is cumulative and that structure is deemed safe if:

$$\sum_{i=1}^m \frac{n_i}{N_i} \leq 1 \quad (4.1)$$

This equation, known as Miner's rule (Miner 1945), is used to obtain the cumulative fatigue damage due to cycles of variable amplitude loading.

An effective stress range,  $S_{eff}$ , can be developed using Miner's linear fatigue damage relationship together with the analytical expression of the S-N curve, to allow reading the fatigue life under variable amplitude stress cycles directly from the existing S-N curve obtained from constant amplitude cycle tests.

The equation of the declining portion of the S-N curve (the part between  $10^1$  and  $10^6$  cycles in Fig. 4.1) is :

$$\log S = \log A' - B \log N \quad (4.2)$$

where  $A'$  is a constant and  $B$  is the slope of the S-N curve. Solving Eq. 4.2 for  $N$  gives:

$$N = AS^{-1/B} \quad (4.3)$$

where  $A = (A')^{1/B}$ . Substituting Eq. 4.3 into Eq. 4.1 gives:

$$\sum_{i=1}^m \frac{n_i S_i^{1/B}}{A} = 1 \quad (4.4)$$

Dividing both sides of Eq. 4.4 by the total number of constant-amplitude cycles  $N_{eff}$  defining the fatigue life at a stress range  $S_{eff}$ ,

$$N_{eff} = AS_{eff}^{-1/B} \quad (4.5)$$

and which corresponds to the fatigue-life under variable-amplitude loading, the following equality is obtained:

$$\sum_{i=1}^m \frac{n_i S_i^{1/B}}{AN_{eff}} = \frac{S_{eff}^{1/B}}{A} \quad (4.6)$$

Cancelling the constant term  $A$  on both sides of Eq. 4.6 leads to:

$$S_{eff} = \left( \sum_{i=1}^m a_i S_i^{1/B} \right)^B \quad (4.7)$$

where:

$$a_i = \frac{n_i}{N_{eff}} \quad (4.8)$$

Recent fatigue test of details under variable amplitude loading and the construction of a database of the fatigue-performance of various details has confirmed that the slope of the S-N curve tends indeed to a value of 1:3 (Fisher and Keating 1987). Therefore, for steel, the finite life portion of the conventional S-N curves has a slope of 1:3, i.e.  $B=1/3$ . Substituting this value into Eq. 4.7, the effective stress range is obtained for variable amplitude loading.

$$S_{eff} = \left( \sum_{i=1}^m a_i S_i^3 \right)^{1/3} \quad (4.9)$$

Therefore, the fatigue life in cycles at this effective stress range is equal to that due to the application of the different stress ranges of frequency and magnitude specified by the stress spectrum. Using this effective stress range concept and the usual S-N curve obtained from constant amplitude fatigue test, fatigue life of a structure subjected to variable amplitude loading can easily be assessed.

#### 4.2.3 High Cycle Fatigue Under Variable Amplitude Loading

A substantial amount of experimental data on the constant amplitude fatigue life of steel beams and details typically used in bridge construction has been obtained in the USA under the sponsorship of the National Co-operative Highway Research program. Tests were conducted on full-scale beams with welded details. Regression analyses on the test data indicated that only two variables significantly influence the fatigue strength of full scale welded details: the stress range and the detail type. While mean stress and steel type were found to affect the fatigue strength of small test specimen, tests conducted on full-size welded details indicated that these factors are of no significance (Fisher et al. 1970, Fisher 1974). The residual stresses present in full size rolled and welded steel structures are partly accountable for this difference.

As part of the same research program, other tests were conducted to provide information on the fatigue resistance of welded attachments subjected to variable amplitude fatigue loading. Most of the research concentrated on experimental studies of welded attachments subjected, in a random sequence, to variable amplitude stresses. A Rayleigh type distribution was selected as a load spectra and only a few stress-cycles exceeded the constant amplitude fatigue limit (CAFL). In a first phase of this research programme, eight full-size beams with web attachments and cover plates were tested (Fisher and Mertz 1983). Tests on another series of beams were also recently completed for which results are yet to be published (J. W. Fisher, personal communication 1992).

In the above tests, the frequency of occurrence of stress cycles above the CAFL was varied from 0.01 to 11.72 percent. The magnitude of the peak stress range in the stress spectrum was made to exceed the CAFL by up to 38 percent. The magnitudes of these loads beyond the CAFL are comparable to those that would be expected on actual bridges.

The tests by Fisher and Mertz (1983) also demonstrated that the existence of a fatigue limit below which no fatigue crack propagation occurs is ensured only if none of the stress range cycles exceed the CAFL. Should this condition be violated, crack propagation will likely occur and fatigue life should be based on the assumption that all stress cycles contribute to fatigue damage. As a consequence, the CAFL concept is not valid anymore and the S-N curve must be extended below the CAFL. Tests on OHBDC fatigue details category E and F, such as cover plate details and thick web attachments, suggested that this extension be of the same 1:3 slope, but similar tests on type C details, such as vertical stiffeners indicated that a slope of 1:5 is more appropriate.

It is noteworthy that, in these tests, two of the cover plated details were subjected to a stress range spectrum which contained no stress ranges exceeding the CAFL but a maximum stress range close to it. Yet, even in these two cases, limited crack propagation

occurred. This is likely attributable to the difficulty in precisely defining the CAFL, and emphasizes that even a small exceedance of the CAFL can initiate crack propagation.

Stress-range cycles of magnitude less than the CAFL contribute to fatigue damage by propagating the cracks initiated by the stress-range cycles that exceeded the CAFL. Therefore, the sequence of the applied stress-range cycles is important. However, it appears that some of the stress-range cycles below a certain amplitude do not contribute to the fatigue damage (Fisher and Keating 1989). If these small cycles are assumed to cause the same fatigue damage as if the CAFL did not exist, then the aforementioned extension of the S-N curve can be seen as defining a lower bound for fatigue life.

The inclusion or non-inclusion of previously deemed non-contributing stress cycles in the fatigue life estimate can affect the shape of the new extension of the S-N curve below the CAFL. As a greater portion of the smaller stress cycles are included in the test spectrum, the effective stress range decreases and the number of cycles to failure increases. The resistance curve becomes a function of the stress range spectrum and the cycle counting method. For some types of load spectra, the actual S-N curve could in fact lie between the CAFL (upper bound) and the linear extension (lower bound) of the finite life portion of the S-N curve. This leads to the conclusion that, the difference between the upper and lower bound S-N curves below the CAFL depends primarily on the peak stress-range and the shape of the probability-density curve defining the stress spectrum. Therefore the established fatigue behaviour becomes unique to the test conditions. However, for the Rayleigh Spectra tested by Fisher (1983) and representative of existing truck population, the linear extension was found to be most adequate.

For the fatigue analysis of details subjected to variable amplitude loading two parameters must be considered: the effective stress range ( $S_{eff}$ ) and the maximum stress range ( $S_{max}$ ). Three different situations can be encountered.

- i) The effective stress range is above the CAFL;

- ii) The effective stress range is below the CAFL, but the maximum stress is above the CAFL;
- iii) Both the effective stress range and the maximum stress range are below the CAFL.

For case (i) in which the effective stress range is greater than the CAFL, this effective stress range is used as an equivalent constant amplitude stress range in conjunction with the constant amplitude S-N curve to determine the fatigue life. In case (ii) the effective stress range is below the CAFL, yet the maximum stress range exceeds this limit. Fisher and Keating (1987) demonstrated that the effective stress range may be used in conjunction with a straight line extension of the sloping portion of the constant amplitude S-N curve to determine the fatigue life. This procedure assumes that CAFL does not exist anymore and that fatigue life is now finite. For case (iii), since the whole stress range spectrum is below the CAFL, none of the stress ranges should be damaging and no fatigue crack propagation is expected.

#### **4.3 Implications of Results**

Using the data from the survey of the truck population in Ontario (Agarwal and Wolkowicz 1976), the structural effects caused by traffic loading on highway bridges were studied and a procedure for obtaining the statistical prediction of the largest effects caused by traffic loading was developed (Harman and Davenport 1979).

For the truck model constructed from this probabilistic work, and the applicable ultimate limit state load factors, there should be only a very small probability that the actual load effects over the lifetime of the bridge exceed the design resistance. In other words, for an ultimate limit state, the mean largest live load effect is calibrated based on a period equal to a bridge life time. In contrast, most serviceability limit states are



anticipated to occasionally occur. Fatigue is such a limit state that many repetitions of the stress ranges are required to cause a failure.

In the OHBDC, the Serviceability Limit State I (fatigue) loading model corresponds to 80% of one OHBD truck located at the centre of the travelled lane. As the OHBD truck is a truck model based on the maximum observed load (MOL) level, when multiplied by 0.8, its weight is reduced to a value approximately equal to the legal load limit according to the Ontario Bridge Formula (OBF); this corresponds to a load level close to the legal load limit in an average truck train in Ontario. The relationship between the MOL and the OBF is illustrated in Figure 4.2 (Jung and Witecki 1971, Ontario 1983b). This fatigue load level, on major roads, is expected to be exceeded once every five minutes.

It is possible, and customary, to design bridges according to the 1983 edition of OHBDC such that they will have an assumed theoretical infinite fatigue life. However, the structural effects produced by some trucks in Ontario can exceed those due to the OHBD-fatigue-truck model. Consequently, the normal Ontario truck traffic itself, in the absence of heavy permit-trucks, can create a fatigue problem: as previously mentioned, if even only a small number of stress ranges exceed the CAFL, then the fatigue life is not infinite anymore.

It has been speculated that the magnitude of the maximum stress-range above the CAFL may affect the slope by which the finite life portion of the S-N curve is extended below the CAFL, i.e. more severe exceedance could cause larger slopes (Fisher and Keating 1989). In that perspective, heavy permit-trucks that can cause higher stress-ranges in bridge elements than the OHBD-fatigue-truck may, depending on their frequency of passage, produce more severe fatigue problems i.e. more severe shortening of the fatigue life. This remains to be investigated.

It is noteworthy that the fatigue truck model is assumed to travel in the middle of a design lane. However, the lateral position of trucks across the bridge deck is a random variable. If a truck shifts from its assumed position towards or away from the shoulder, then it may cause larger stresses in exterior or interior girders. The positioning of the truck in the middle of a design-lane is logical, in an average sense, to assess fatigue life. However, since the fatigue life is not infinite even if only 0.01 percent of the stress cycles in a variable amplitude spectrum exceeds the constant amplitude fatigue limit, this further compounds the aforementioned problem.

Finally as the sequence of loading was demonstrated to be very important, it should be obvious that, until a truck heavier than the fatigue design truck has passed over a bridge and until a stress-range exceeds the CAFL, all previous stress cycles can be neglected in the calculation of the remaining fatigue life.

#### **4.4 Summary**

- Recent research demonstrates that if stress ranges from a variable amplitude stress spectrum exceeds the CAFL, even by a small amount or by a small frequency, then the fatigue life is not infinite anymore. For load spectra representative of truck loading, the CAFL can be replaced by an extension of the finite life portion of the S-N curve with a slope of 1:3 for OHBDC fatigue details category E or F, and 1:5 for category C. Information on other fatigue detail categories is not available in the existing literature.
- The load effect of a certain percentage of the truck population in Ontario exceeds that of the OHBD-fatigue truck model. Therefore, the fatigue life of some bridges designed according to 1983 edition of the OHBDC is likely not infinite in spite of the contrary design intent. Furthermore, as the fatigue truck model considered in design is not usually placed on the bridge deck at a position causing the largest

live load effect, this adds to the probability that the design stress ranges can be exceeded.

- Although the stress ranges due to heavy permit trucks were sometimes found to exceed the design stress ranges by approximately 50%, the disappearance of the CAFL can be equally attributable to overweight conditions in the normal traffic. Therefore the comparative effect of heavy permit-trucks passages in shortening fatigue-life can be easily calculated by adopting the new modified S-N curve.

## CHAPTER 5

### DEVELOPMENT OF A FATIGUE BASED METHODOLOGY TO PERMIT HEAVY TRUCKS

#### 5.1 Estimation of The Fatigue Life

The variable amplitude stress range spectrum is composed of  $m$  various stress range  $S_i$  of constant amplitude, each assigned a corresponding frequency of occurrence  $f_i$ . These frequencies can be normalized by dividing each of them by their sum to obtain:

$$\sum_{i=1}^m f_i = 1 \quad (5.1)$$

Every stress range is then considered separately, and the number of cycles  $N_i$  of each stress range which could be carried safely is calculated using either Eqs. 5.2 or 5.3 which define two regions, the finite life portion and the part below CAFL, of the S-N curve.

$$N_i = N_L \left( \frac{S_L}{S_i} \right)^3 \quad (5.2)$$

$$N_i = N_L \left( \frac{S_L}{S_i} \right)^Y \quad (5.3)$$

$S_L$  and  $N_L$  in the above equations are respectively the allowable stress range and number of cycles corresponding to the CAFL and vary for various detail categories, while  $Y$  in

Eq. 5.3 stands for the inverse of the S-N curve's slope below the CAFL. Eq. 5.2 should be used for stress ranges larger than  $S_L$  otherwise Eq. 5.3 should be used.

The number of application  $n_i$  of each stress cycle during the fatigue life time can be obtained by multiplying its frequency by the total number of applied cycles  $N_c$ :

$$n_i = N_c f_i \quad (5.4)$$

Substituting Eq. 5.4 in Miner's cumulative damage equation, the safe fatigue life, in cycles, of a detail can be calculated as:

$$N_c = \frac{1}{\sum_{i=1}^m \frac{f_i}{N_i}} \quad (5.5)$$

Assuming that each truck passage causes one stress cycle, the safe fatigue life  $N_{ys}$  in years is calculated by the following equation;

$$N_{ys} = \frac{N_c}{ADTT \times 365.25} \quad (5.6)$$

where, ADTT is the average daily truck traffic volume and 365.25 is the number of days in a year.

It is noteworthy that, the calculated fatigue life in years depends on the adequacy of the stress range spectrum in reflecting the actual traffic and on how reliably the truck traffic volume is represented by the ADTT. Furthermore, at the component level, the distribution of efforts depends on the geometric and structural properties of the bridge as well as the axle width of the truck (Bakht and Jeager, 1985). Therefore, for a more accurate estimation of the fatigue life, a stress range spectrum reflecting the efforts in the components of interest should be employed in the calculations. However, a spectrum reflecting the overall stress range at bridge cross-section is generally accepted and can be

used to estimate the fatigue life of major structural components such as steel girders. Moreover, in the derivation of Eq. 5.6 it was assumed that each truck passage causes one stress cycle. Obviously, this is not true for cross beams and stringers in truss bridges. For cross beams, number of stress range cycles due to each truck is equal to the number of axles. However, the stress ranges caused by each axle may not be the same. In the case of stringers the situation is more complicated. The number of stress range cycles is a function of the number of axles, axle spacing and length of the stringer. The stress ranges in this case may not be the same for each cycle either.

### 5.1.1 Mean Versus Safe Fatigue Life

To estimate the safe fatigue life,  $S_L$  and  $N_L$  in Eqs. 5.2 and 5.3 should be obtained from the 95% confidence allowable S-N curve usually adopted by design codes to ensure adequate safety and consistent levels of reliability. To obtain the mean fatigue life, the ratio of mean to allowable stress range can be used. It was proven that, for all detail categories, this ratio does not vary greatly and averages to 1.243 (Moses et al., 1987). Because of the power of 3 in the S-N curve, the corresponding ratio of mean to safe lives is equal to 1.243 cubed, or 1.92. Consequently, the mean life  $N_{ym}$  in years can be obtained by multiplying Eq. 5.6 by 1.92;

$$N_{ym} = 1.92 \times \frac{N_c}{ADTT \times 365.25} \quad (5.7)$$

This last equation is valid provided that the slope of the S-N curve below the CAFL is also 1:3 as in the finite life portion. Otherwise, to calculate  $N_{ym}$ , Eqs. 5.2 and 5.3 must first be multiplied respectively by the cube and Y<sup>th</sup> power of the constant 1.243. The new  $N_i$ 's thus obtained can then be substituted in Eq. 5.5 to calculate the fatigue life in cycles  $N_f$ , and Eq. 5.6 would directly give the mean fatigue life in years ( $N_{ym}$ ) instead of the safe fatigue life ( $N_{ys}$ ). Nevertheless, this is necessary only if the maximum stress range in the spectrum exceeds  $1.243 \cdot S_L$ ; otherwise the mean fatigue life is theoretically infinite.

### 5.1.2 Effect of Over Design on Fatigue Life

A steel bridge's structural elements and details are typically designed on the basis of an allowable stress range specified to ensure a satisfactory protection against fatigue-failure. It is the intent of the 1983 edition of the OHBDC that the computed stress ranges at each location and detail of interest should be less than or equal to their corresponding allowable stress ranges  $\sigma_{all}$ . However, in many instances, depending on the type of fatigue detail used and its location, the design may be governed by the ULS rather than the SLS-I. Should this be the case, the resulting stress ranges would be lower than the allowable stress ranges specified by the SLS-I, and the fatigue life predicted by Eqs. 5.6 and 5.7, which are derived on the basis that design is governed by the SLS-I, would be lower than the actual one. Therefore, these equations can be improved if multiplied by the cube of the over design ratio, *ODR*, described in the subsequent part of this chapter.

A steel bridge's girder is designed for fatigue using the following equation;

$$\frac{0.8M_L}{S_{SLS-I}} \leq \sigma_{all} \quad (5.8)$$

where 0.8 and  $M_L$  are respectively the live load factor and the live load moment and  $S_{SLS-I}$  is the minimum required elastic section modulus if the bridge is designed for SLS-I. On the other hand, a steel bridge's girder is designed for ultimate strength such that;

$$\frac{1.2M_D + 1.4M_L}{Z} \leq \sigma_y \quad (5.9)$$

where  $M_D$  is the dead load moment and  $Z$  is the plastic section modulus. The constants 1.2 and 1.4 are the dead and live load factors respectively. The plastic section modulus

is related to the elastic section modulus  $S$  by the following equation.

$$K = \frac{Z}{S} \quad (5.10)$$

For typical structural sections used in bridge construction the shape factor  $K$  has an average value of 1.14 (Horne and Morris, 1982). Assuming that for slab on girder bridges of 10-50 metres spans, the live load moment is on average 20 percent larger than the dead load moment, and if the unfactored live load moment range calculated in fatigue design is assumed to be equal to the live load moment calculated in ultimate strength design (i.e. no stress reversals), then Eqs. 5.8 and 5.9 can be combined to provide a relationship between  $S$  and  $S_{SLS-I}$  and consequently an expression for the  $ODR$ . The resulting equation and definition for this  $ODR$  is:

$$ODR = \frac{S}{S_{SLS-I}} = \frac{2.63 \sigma_{all}}{\sigma_Y} \quad (5.11)$$

Clearly, the  $ODR$  depends on the steel grade and type of detail used in the design as Eq. 5.11 is a function of the design yield strength and allowable stress range defined for each detail category. Value of  $ODR$  smaller than 1 indicates that the design is governed by fatigue, and that Eqs. 5.6 and 5.7 are applicable, whereas, a value greater than 1 indicates that the design is governed by ultimate strength and therefore the calculated SLS-I stress ranges are less than the allowable stress ranges specified in the code. In other words, CAFL or  $S_L$  is  $ODR$  times higher than the calculated stress ranges. As observed from Eq. 5.2, in an S-N curve of constant slope of 1:3, the number of cycles to failure, hence the fatigue life is proportional to the cube of the  $S_L$ . Thus, Eqs. 5.6 and 5.7 should be multiplied by the cube of the  $ODR$  to obtain a correct estimate of the safe and mean fatigue life respectively.

The above procedure is not applicable if the slope of the S-N curve changes below the CAFL. To calculate the safe fatigue life in this case, Eqs. 5.2 and 5.3 must first be multiplied respectively by the cube and  $Y^{th}$  power of the  $ODR$ . The new  $N_f$ 's thus



obtained can then be substituted in Eq. 5.5 to calculate the fatigue life in cycles  $N_C$ , and Eq. 5.6 would directly give the safe fatigue life in years. Nevertheless, this is necessary only if the maximum stress range in the spectrum exceeds  $ODR \times S_L$ ; otherwise the fatigue life is theoretically infinite. To calculate the mean fatigue life, the same procedure applies with the exception that, Eqs. 5.2 and 5.3 must also be multiplied respectively by the cube and  $Y^{\text{th}}$  power of 1.243, if the maximum stress range in the spectrum exceeds  $1.243 \cdot ODR \cdot S_L$ ; otherwise the fatigue life is theoretically infinite.

Using Eq. 5.11, Table 5.1 is constructed for various detail categories and design yield capacities. As observed from the table, for detail categories c to f, the ODR are less than 1 for all the design yield limits considered. Hence, design is governed by fatigue for these detail categories. Whereas, if detail category a is used, design is always governed by ultimate strength since  $ODR$  is larger than 1 for all the design yield limits considered. For detail category b, the design is governed by fatigue for design yield capacities greater than 300 MPa, for the assumption adopted for the ratio of live load to dead load moments. As will be illustrated later, in the average spectrum used in fatigue life calculations, the maximum stress range is  $1.71825 \cdot S_L$ . Therefore, for the over design ratios greater than this value, fatigue life is theoretically infinite. It is observed from Table 5.1 that if detail a is used with a steel of 250 MPa design yield capacity, then the  $ODR$  is 1.74. This makes the CAFL stress larger than the peak stress range in the average spectrum, leading to a theoretically infinite fatigue life.

## 5.2 Effect of Ontario Truck Traffic on Fatigue Life of Steel Bridges

In a previous phase of this research, it was noted that the structural effects produced by some trucks in Ontario can exceed those due to the OHBD fatigue-truck model. Consequently, the normal Ontario truck traffic itself, in the absence of heavy permit-trucks, could create a fatigue problem since, as previously mentioned, if even only

a small number of stress ranges exceed the CAFL, then the fatigue life is not infinite anymore. This is investigated following.

From the sample of 6375 trucks weighed and measured to study the distribution of truck traffic in Ontario (Agarwal and Wolkowicz, 1976), flexural stress range spectra for spans ranging from 10 metres up to 100 metres were constructed by passing these trucks over the moment influence lines corresponding to each span. These spectra represent the effects of the heavy portion of the Ontario truck traffic which constitutes approximately 35 percent of the total truck population. The mean spectrum of the heavy portion of the truck traffic, obtained by averaging all these spectra, is illustrated in Figure 5.1. The horizontal axis of the spectrum is the ratio,  $S'_i$ , of the Ontario trucks' stress range to the OHBD-fatigue truck's stress range, and the vertical axis is the frequency of occurrence,  $f_i$ , of these stress range ratios. To obtain the magnitude of stress range  $S_i$  due to the various Ontario trucks in that spectrum, the  $S'_i$  ratios need only be multiplied by the OHBD-fatigue truck's stress ranges for a given span. Should the design be governed by fatigue, the magnitude of the stress ranges due to these same Ontario trucks can instead be obtained more directly by multiplying the stress ratios  $S'_i$  by  $S_L$  since OHBD-fatigue truck's stress range at a detail of interest is equal to its corresponding allowable stress range at the CAFL of the S-N curve i.e.  $S_L$ . As observed from the spectrum, while the most frequent stress ranges are those between 65 and 95 percent of the OHBD-fatigue truck's stress range, some are up to 70% larger than the OHBD-fatigue truck's stress range albeit with a small frequency. Yet all trucks producing stress range ratios larger than 1.0 are responsible for eliminating the CAFL which subsequently result in a considerable decrease in fatigue life.

In Figure 5.2, an average stress range spectrum which represents the effect of total truck traffic in Ontario is illustrated. In this spectrum, the most frequent stress ranges are those between 30 and 60 percent of the OHBD-fatigue truck's stress range. Since this spectrum represent the effect of total truck traffic rather than the heavy portion, it is more

realistic and will be used exclusively in this study to estimate fatigue lives except in the remainder of this subsection where the effect of both spectra on the estimation of fatigue life will be briefly investigated, and in Section 5.4 where effects of various parameters are investigated.

In Figure 5.3, the mean fatigue life in years, due to the heavy portion of Ontario's truck traffic, is plotted for different S-N curve's slopes below the CAFL as a function of a scale factor,  $S_{i,0}/S_L$ , where  $S_{i,0}$  is the stress range value when  $S'_i = 1.0$ . This scale factor relates the amplitude of the stress ranges in the spectrum to the CAFL. Here, no stress range exceeds the CAFL when the scale factor is approximately 0.60. An ADTT of 4000 is assumed for the total truck traffic. Since the heavy portion of the Ontario's truck traffic is around 35 percent of the total truck traffic, an ADTT of  $0.35 \times 4000$  or 1400 is used in conjunction with the heavy portion of the Ontario's truck traffic's stress range spectrum to estimate the mean fatigue life in years. Clearly, from Figure 5.3, fatigue life increases as the slope below CAFL decreases, as expected intuitively. However, the differences are considerable, particularly as the amplitude of the average spectrum is scaled down. For example for a 1.0 scale factor (unscaled stress range spectrum), the mean fatigue life of 300 years when the slope below CAFL is zero, drops down to 53 years when the slope below CAFL is increased to 1:3. Intermediate values and their sensitivity to slope can be read from Figure 5.3.

As mentioned before, the slope below CAFL and consequently fatigue life, depends on the amplitude of the maximum stress range in the spectrum. Although a relationship between this stress range and slope has not been experimentally defined yet, its existence is logical and can be simply modelled to analytically investigate its consequences; this will be done in a later section. It is noteworthy that, even for zero slope, the few stress ranges exceeding the CAFL contribute to a calculable fatigue life of 300 years, in spite of the theoretically infinite fatigue life implied by the OHBDC design rules. This further shows that the amplitude of the stress range spectrum is a very important parameter affecting the fatigue life.

To study the effect of Ontario's normal truck traffic on fatigue life of bridges, the same plot as in Figure 5.3 is regenerated in Figure 5.4 using instead the average stress range spectrum which represents the effect of total truck traffic in Ontario, for a corresponding ADTT of 4000. The same trend is observed here as in Figure 5.3. The differences in predicted life typically attributable to the choice of spectrum are best illustrated by Figure 5.5, where the mean fatigue life for detail type f is plotted for various slopes using both the stress range spectrum which represent the heavy portion of truck traffic and that of the total truck traffic. It is observed that the mean fatigue life obtained using the spectrum for the total truck traffic is lower than that obtained using the spectrum for heavy portion of the truck traffic. However, the difference decreases as the slope below CAFL decreases and no difference is observed when the slope below the CAFL is zero. This is logical, as the dissimilarity between the two spectra only occurs for the stress ranges below the CAFL which do not contribute to the fatigue damage when the slope is zero unless the  $S_{L0}/S_L$  ratio become exceptionally large. This also further demonstrates how fatigue life predictions are sensitive to the shape of the stress range spectrum.

In summary, the effect of Ontario truck traffic is to bring the fatigue life from an assumed theoretically infinite life to a finite life since some of the stress ranges in the spectrum exceeds the CAFL. The slope below the CAFL has a considerable impact on the estimation of fatigue life. This slope is believed to depend on the magnitude of the peak stress amplitude.

In light of the considerable decrease in fatigue life due to the Ontario normal truck traffic alone, one may wonder why there has been no fatigue-related bridge failures in Ontario yet. This can be partly explained by various factors:

- (i) Bridges older than fifty years are often posted and passage of trucks heavier than a certain weight limit is prohibited;

- (ii) Old bridges on major roads are often replaced by larger new bridges, or strengthened if new lanes can be added to increase their capacity, while most of the old bridges on secondary highways carry a much smaller volume of traffic.
- (iii) The ADTT has not been constant for every bridge over the years. The current traffic volume is much larger than when most bridges were opened. The history of this ADTT should be accurately known to obtain a reliable estimate of the fatigue life for a given bridge.
- (iv) The stress range spectrum defined above represents the frequency distribution of the effect of current Ontario trucks. These have become substantially heavier with the recent developments in the trucking industry in Canada, over the last 20 years. Therefore, for old bridges only conservative fatigue life estimates are possible using this spectrum. Instead, a spectrum of the old truck traffic could be used over its applicable period of time to calculate the corresponding fatigue damage, and the current truck traffic spectrum could be used to calculate the remaining fatigue life. Alternatively, multi-step spectra or other evolutionary spectra could be used provided they can be reliably constructed.
- (v) When bridges are designed, the selection of member sizes is sometimes governed by ultimate strength instead of fatigue. Depending on the degree of over-design with respect to the SLS-I, fatigue life may even be infinite if no stress range in the spectrum exceeds the CAFL.
- (vi) Generally, in the high cycle portion of the the S-N curves, the experimental data are highly dispersed above the mean curve, and therefore some bridges may have much longer life than expected.

For a quick estimate of the safe and mean fatigue lives for all detail categories under the Ontario normal truck traffic, Table 5.2 is constructed using a constant lower

bound slope of 1:3 in the S-N curve and the average flexural stress range spectrum illustrated in Figure 5.2. The equations in Table 5.2 should be multiplied by the cube of the *ODR* if the design is governed by ULS. As mentioned above the safe and mean fatigue lives obtained using the equations in Table 5.2 are only minimum values. Fatigue life may be larger if the slope of the S-N curve below the CAFL is smaller than 1:3. In Figures 5.6 and 5.7 the safe and mean fatigue life are respectively plotted as a function of the ADTT for various detail categories and a slope of 1:3 only. As observed from these figures, bridge life is highly dependent on the ADTT volume. Hence, for a close approximation of the fatigue life, an ADTT over the bridge life time should be estimated accurately.

### 5.3 Estimation of Reduction in Fatigue Life Due to Heavy Permit-Trucks

Frequency of passage,  $f_{hpt}$ , of the heavy permit-truck, which is expressed as a percentage of the current truck traffic, and the magnitude of stress range,  $S_{hpt}$ , of the heavy permit-truck are the only live load parameters affecting the reduction in fatigue life. The same procedure, described for the calculation of fatigue life under normal traffic, can be used to obtain the reduced fatigue life  $N_R$ , if the frequency and stress range of the heavy permit-truck are included in the aforementioned spectrum. Doing so, as a first step, the frequencies of the stress range spectrum must be re-normalized by including the frequency of the heavy permit-truck:

$$f'_i = \frac{f_i}{1+f_{hpt}} \quad (5.12)$$

$$f'_{hpt} = \frac{f_{hpt}}{1+f_{hpt}} \quad (5.13)$$

Then, Eq. 5.1 can be presented in a different form using Eqs 5.12 and 5.13 :

$$f'_{hpt} + \sum_{i=1}^m f'_i = \frac{1}{1+f_{hpt}} \left[ f_{hpt} + \sum_{i=1}^m f_i \right] = 1 \quad (5.14)$$

Next, the reduced fatigue life in cycles is calculated using Eqs. 5.4 and 5.14 in conjunction with Miner's cumulative damage equation:

$$N_R = \frac{1+f_{hpt}}{\frac{f_{hpt}}{N_{hpt}} + \sum_{i=1}^m \frac{f_i}{N_i}} \quad (5.15)$$

$N_{hpt}$  in Eq. 5.15 is the number of cycles of stress range  $S_{hpt}$  which could be carried safely if applied alone and can be obtained from either Eq. 5.2 or Eq. 5.3 depending on the magnitude of the stress range. The normalized frequency of the heavy permit-truck is multiplied by the reduced fatigue life in cycles to obtain;

$$n_{hpt} = \frac{f_{hpt}}{1+f_{hpt}} N_R \quad (5.16)$$

where  $n_{hpt}$  is the total number of passages of a heavy permit-truck during the fatigue life time. Then, the number of heavy permit-truck passages per month,  $n_{pm}$ , corresponding to a specified frequency is obtained from the following equation:

$$n_{pm} = \frac{n_{hpt} ADTT \ 30.4375}{N_R} \quad (5.17)$$

The constant 30.4375 in the above equation is the average number of days in a month. The frequency of the heavy permit-truck can also be expressed as a function of the

allowable number of passages of these trucks per month if Eq. 5.16 is substituted into Eq. 5.17:

$$f_{hpt} = \frac{n_{pm}}{30.4375 ADTT - n_{pm}} \quad (5.18)$$

The reduced fatigue life in cycles,  $N_R$ , includes also the cycles applied by heavy permit-truck. To obtain the maximum number of cycles  $N_{CR}$  that can be applied by the Ontario truck traffic (spectrum without the heavy permit truck) during the reduced fatigue life time, the number of cycles of heavy permit-truck,  $n_{hpt}$ , should be subtracted from the reduced life:

$$N_{CR} = N_R - n_{hpt} \quad (5.19)$$

The difference between  $N_r$  and  $N_{CR}$  gives the reduction in fatigue life in cycles. Finally, the percentage of reduction,  $PR$ , in fatigue life due to heavy permit-truck is calculated as:

$$PR = \frac{N_c - (N_R - n_{hpt})}{N_c} 100 \quad (5.20)$$

#### 5.4 Effects of Various Parameters on Percent Reduction in Fatigue Life

The tolerable reduction in fatigue life due to heavy permit-trucks depends on the actual life under normal traffic. If a bridge's fatigue life is calculated as 150 years, then 50 years reduction may be acceptable. On the other hand, if a bridge has a 40 years of fatigue life then even a few years of reduction in fatigue life may not be admissible. Therefore, percent reduction in fatigue life is chosen as a more meaningful dimensionless variable to study the effects of various parameters such as type of the stress range spectrum, detail type, amplitude of the heavy permit-truck's stress range, slope of the S-N curve below the CAFL and the ADTT. The concept is that only a relative loss of service



life on the entire bridge inventory can be managed. The procedure described in the previous sections of this thesis is used to write a computer program to investigate the effects of these parameters.

#### **5.4.1 Stress Range Spectrum**

In Figure 5.8, percent reduction in fatigue life of a bridge girder with d type detail is plotted for spectra obtained for spans ranging up to 100 metres as well as for the average of these spectra, as a function of the stress range due to heavy permit-trucks. These spectra represent the effect of heavy portion of the truck population. The frequency of the application of the heavy permit-truck's stress range is chosen as 100 per month. An ADTT of 1000 is used in the calculations. As observed from the figure, the difference in percent reduction in fatigue life for various spectra increases as the magnitude of the stress range increases, yet the difference is in acceptable limits to enable the use of a mean spectrum as plotted by a thick solid line in the figure. Hereafter, the average spectrum will be used in all the calculations.

#### **5.4.2 Type of Detail**

Percent reduction in fatigue life for various detail categories, as a function of the stress range due to heavy permit-trucks is illustrated in Figure 5.9. Here again, the frequency of the application of the heavy permit-truck's stress range is chosen as 100 per month. The average spectrum which represents the effect of heavy portion of the Ontario's truck traffic is employed in the calculations with an ADTT of 1000. A slope of 1:5 below the CAFL is used in the calculation of the percent reduction in fatigue life. As observed from the figure, percentage of reduction in fatigue life does not depend on the detail type. This result is expected because the S-N curves of all detail categories are parallel to each other and the bridge components are designed for an allowable stress

which corresponds to the CAFL's stress. Although this is formally proven in section 5.5.1, it can also be conceptually explained. Consider two steel girders with different detail categories, in two similar bridges of the same length and load distribution characteristics. Assume that these girders are both designed for SLS-I such that the design stress ranges matches the allowable stress ranges or  $S_L$  at each corresponding detail category. The girder with a detail type of smaller allowable stress will have a larger section modulus, hence the heavy permit-trucks' stress range in this girder will be smaller. On the contrary, the girder with a detail type of larger allowable stress will have a smaller section modulus. Obviously, the heavy permit-trucks' stress range in this girder will be larger. However, the relative magnitude of the heavy permit-trucks' stress range with respect to CAFL will be the same in both girders. Consequently, the reduction in fatigue life will also be the same in both girders.

#### **5.4.3 Magnitude and Frequency of The Heavy Permit-Truck's Stress Range**

In Figure 5.10, the percent reduction in fatigue life is plotted for different slopes of the S-N curves below the CAFL as a function of the stress range due to heavy permit-trucks. The S-N curve for detail d is used. The frequency of the application of the heavy permit-truck's stress range is chosen as 100 per month. The spectrum representing the effect of the heavy portion of the truck traffic is used again with an ADTT of 1000. As observed from the figure the percent reduction in service life has a polynomial variation as a function of the stress range. This is because the functional relation between the number of cycles to failure and the stress range is also polynomial. For an S-N curve of a constant slope of 1:3, percent reduction in fatigue life is proportional to the cube of the heavy permit-truck's stress range.

Figure 5.11 demonstrates the variation of the percent reduction in fatigue life as a function of the number of passages of heavy permit-trucks per month. The figure is generated using the flexural stress ranges of these trucks as obtained from an influence

line analysis for 20 metrespan. The variation seems to be an almost linearly increasing function of the frequency of passage.

#### **5.4.4 Slope of The S-N Curve Below CAFL**

In Figure 5.12, the percent reduction in fatigue life is plotted for five different stress ranges, as a function of the slope of the S-N curve below the CAFL. These stress ranges are defined as a fraction of the CAFL. Again, the frequency of the application of the stress ranges is chosen as 100 per month. The average spectrum that represents the effect of heavy portion of the truck traffic is used in the calculations with an ADTT of 1000. The result shows that as the slope decreases reduction in fatigue life linearly increases.

#### **5.4.5 ADTT**

Percent reduction in service life as a function of the ADTT is demonstrated for various detail categories in Figure 5.13. The plot is generated for five different stress ranges which are defined as a fraction of the CAFL. The frequency of these stress ranges is chosen as 100 per month. Here again, the average spectrum that represent the effect of heavy portion of the truck traffic is used. It is observed from the figure that fatigue life is inversely proportional to the ADTT. Hence, the reduction in fatigue life increases as the ADTT decreases. This is expected because the number of application of the heavy permit-truck's stress ranges is assumed constant (100 per month) regardless of the ADTT. Consequently, as the ADTT decreases, frequency of the heavy permit-truck's stress ranges relatively increases. This obviously leads to a higher reduction in fatigue life as the ADTT decreases.

## 5.5 Methodology to Permit Heavy Trucks

### 5.5.1 Development

A flexible methodology which considers the degrees of liberalism in regulations of heavy-truck permits and the change in the volume of the normal traffic as well as the weight of the permit vehicles, should be developed to assess the reduction in service life of the bridges due to heavy permit-trucks. The objective is to obtain the allowable number of passages of a heavy permit-truck per month, for a specified percentage of reduction in fatigue life and the significance of this percentage in real terms. Considering these constraints, an equation is originated to be used for issuing permit to heavy trucks. In this equation, three parameters: the tolerable percentage of reduction in fatigue life, ADTT and the type and weight of the heavy permit-trucks are chosen as variable, while the type of spectrum is kept constant. However, if the spectrum type changes in the future, it will be possible to modify the equation for the new spectrum.

In order to obtain the equation mentioned above, first, Eq. 5.15 is substituted into Eq. 5.20 and simplified to obtain:

$$f_{hpt} = N_{hpt} \left( \frac{100}{N_c (100 - PR)} - \sum_{i=1}^m \frac{f_i}{N_i} \right) \quad (5.21)$$

Then, Eq. 5.5 is substituted into Eq. 5.21 and simplified to obtain:

$$f_{hpt} = \frac{N_{hpt} PR}{N_c (100 - PR)} \quad (5.22)$$

Then, from Eqs. 5.18 and 5.22 the allowable number of a certain type of heavy permit-truck per month is obtained as:

$$n_{pm} = \frac{N_{hpt} PR ADTT 30.4375}{N_c (100 - PR) + N_{hpt} PR} \quad (5.23)$$

The above equation is a general expression that can be used to estimate the allowable number of permits issued to a certain type of heavy permit-truck per month for a specified percentage of reduction in fatigue life. However, this equation can further be simplified for practical applications that do not require a detailed consideration of all the variables.

The five vehicle models considered in the previous phase of this research programme are representative of the types of heavy permit-trucks in Ontario. Each truck is currently assigned a specific weight when it is fully loaded. These weights may increase with time as the trucking industry is developing very fast. On the other hand, not all the trucks are always loaded to their full capacity. Consequently, the weight of these trucks is subject to change. Besides this, the stress ranges in structural components due to these trucks varies significantly with the span length of the bridge. Therefore, the weight of these trucks as well as the span length of the bridge should be variable in the equation to be derived.

For added flexibility, in the subsequent application of the methodology the following tools are developed using all the heavy-truck models as arbitrarily normalized to have a gross weight of 1000 kN. However, the known proportional distribution of this 1000 kN weight among the axles of each permit-truck model is respected in all cases. In a previous chapter, influence line analyses of simply supported and continuous spans

ranging up to 125 metres were conducted for each truck model as well as the OHBD live load to obtain the live load ratios, *LLR*, which are the ratio of the heavy permit-trucks' flexural stress ranges to the OHBD live load's flexural stress ranges. Then the results were plotted for simply supported and continuous bridges. In the case of simply supported bridges, the previously obtained *LLR* are scaled with respect to 1000 kN weight and also divided by 0.8 to account for fatigue live load. The resulting curves are illustrated in Figure 5.14. In the case of continuous bridges, ignoring the contribution of the concrete slab in composite sections, the design is governed by the largest of negative and positive design moment ranges resulting in a uniform section along the continuous span. Therefore, the largest of *LLR* for negative and positive moment ranges are picked up and then scaled with respect to 1000 kN weight. The *LLR* are also divided by 0.8 to account for fatigue live load. The resulting envelope curves are plotted in Figure 5.15. For a truck model of a different given weight, *W*, its *LLR* as per the figures need only be multiplied by 0.001*W*.

Heavy permit-trucks usually travel with a slower speed than the normal trucks, consequently, the dynamic load allowance, *DLA*, for these trucks is smaller than those of normal trucks. To take into consideration this effect, the *LLR* obtained from Figures 5.14 or 5.15 should be multiplied by the dynamic load allowance ratio, *DLAR*, which is a function of speed, *V* and *DLA*. The function is expressed as follows:

$$DLAR = \left\{ \begin{array}{ll} \frac{1 + 0.3 DLA}{1 + DLA} & \text{if } V \leq 10 \text{ km/h} \\ \frac{1 + 0.5 DLA}{1 + DLA} & \text{if } 10 < V \leq 25 \text{ km/h} \\ 1 & \text{if } V > 25 \text{ km/h} \end{array} \right\} \quad (5.24)$$

The procedure in the 1983 edition of OHBDC can be used to obtain the *DLA*. *DLA* is a function of the first natural frequency of the bridge superstructure and varies between 0.2 and 0.4, therefore, for practical purposes an average value of 0.3 could be considered.

Heavy permit trucks may have axles wider than the standard 1.8 metres axle width of the OHBDC-fatigue truck. It is known that the distribution of flexural moments among the girders in slab on girder bridges is a function of the axle width (Bakht and Jaeger, 1985). This should also be considered in the methodology.

In the 1983 edition of the OHBDC, two dimensionless torsional and flexural parameters,  $\alpha$  and  $\Theta$ , quantify the load distribution characteristics of a bridge. A bridge with small value of  $\alpha$  and large value of  $\Theta$  has poor lateral load-distribution characteristics, whereas a bridge with large  $\alpha$  and small  $\Theta$  has a superior lateral load-distribution characteristic, i.e resistance sharing among the girders tend to be more uniform. The methodology developed by Bakht B. and Jaeger L. G. (1984) for calculating the allowable increase in vehicle weight as a function of axle width is used as a tool to find an expression which accounts for the percent decrease in stress as the axle width increases. For typical slab on girder bridges,  $\alpha$  varies between 0.06 and 0.30 and  $\Theta$  between 0.5 and 2.0. Thus, two-lane bridges of 8.0 metres widths and with the worst ( $\alpha=0.06$ ,  $\Theta=2.0$ ) and best ( $\alpha=0.30$ ,  $\Theta=0.5$ ) lateral load-distribution characteristics are analyzed by the semi-continuum method, using the program SECANI. The OHBDC-fatigue truck is placed at the centre of one of the lanes as required by the OHBDC. Then, the axle width is increased from 1.8 metres up to 3.0 metres, and both the interior and exterior girder moments are calculated for each case. The resulting moments are taken as upper and lower bound values for slab on girder bridges. It is found that the change in the exterior girder moment is not significant over the range of  $\alpha$  and  $\Theta$ . The interior girder moment, however, decreases almost linearly as the axle width increases, the percentage of decrease being more in the bridge with the worst load distribution characteristic. This demonstrates that the percentage of decrease in moment is a function

of  $\alpha$  and  $\Theta$ . Consequently, the results obtained from the analysis of the bridge with the best load distribution characteristic are used to derive the following analytical expression:

$$SRC = 1 - 0.07 (AW - 1.8) \quad (5.25)$$

where, *SRC* stands for stress reduction coefficient and *AW* is the axle width in metres. The above equation has been developed for vehicles with two lines of wheels, and would produce conservative values should the width between the outermost wheels be used for vehicles having more than two lines of wheels. Although the equation is developed for bridges with only two lanes, it can be safely used for bridges with larger number of lanes.

If the axle width of the heavy permit-truck is larger than 1.8 metres, the *LLR* should be multiplied by the *SRC* to obtain a close approximation of the heavy permit-truck's stress in the girders of the slab on girder bridges. However, as *SRC* values are generally close to 1.0, this added refinement can be optional.

The following comparison is done to assess the reliability of Eqs 5.24 and 5.25 derived to obtain stress ranges of heavy permit trucks. The ratio of the SLS-I stress range due to heavy permit-truck 5 to that due to OHBDC-truck was obtained as 1.23 as illustrated in Table 3.2 for the stringers of Grigg bridge. From Figure 2.1, the *LLR* is obtained as 1.56. Also from Table 2.1, the axle width and speed limit of the truck is obtained as 1.8 m. and 10 km/h respectively. Assuming a *DLA* of 0.3, the *DLAR* is calculated as 0.838 using Eq. 5.24. Substituting the axle width in Eq. 5.25, *SRC* is obtained as 1.00. Next, *LLR* is multiplied by *DLAR* and *SRC* respectively and the ratio of stress ranges is calculated as 1.30. As demonstrated, the difference between the actual and approximated stress range is acceptable, hence, Eqs. 5.24 and 5.25 can safely be used to obtain the heavy permit truck stress ranges.

The structural components of steel bridges are designed for fatigue on the basis that the computed stress ranges at each location and detail of interest is nearly equal to



the corresponding allowable stress which is the CAFL stress. Consequently, the fatigue live load stress is approximately equal to  $S_L$ . Using this information, the heavy permit-truck's stress is depicted by the following equation:

$$S_{hpt} = S_L \frac{LLR \cdot DLAR \cdot SRC}{1000} \frac{W}{1000} \quad (5.26)$$

If the slope of the S-N curve below the CAFL is assumed to have a lower bound value of 1:3, Eq. 5.26 can be substituted in Eq. 5.2 to obtain  $N_{hpt}$  as:

$$N_{hpt} = N_L \left( \frac{1000}{LLR \cdot DLAR \cdot SRC \cdot W} \right)^3 \quad (5.27)$$

Then, Eq. 5.27 is substituted into Eq. 5.23 and simplified to obtain:

$$n_{pm} = \frac{30.4375 \times 10^9 N_L ADTT PR}{N_c (100 - PR) (LLR \cdot DLAR \cdot SRC \cdot W)^3 + N_L PR (1000)^3} \quad (5.28)$$

It can be mathematically proven that the percentage of reduction in fatigue life is independent of the detail type. To illustrate this, first substitute Eq. 5.16 into Eq. 5.20 to obtain Eq. 5.29 as:

$$PR = 100 \left( 1 - \frac{N_R}{N_C} \left( 1 - \frac{f_{hpt}}{1+f_{hpt}} \right) \right) \quad (5.29)$$

Then, substitute Eq. 5.5 into Eq 5.15 to obtain:

$$N_R = \frac{N_C N_{hpt} (1+f_{hpt})}{f_{hpt} N_C + N_{hpt}} \quad (5.30)$$

Next, substitute the above equation into Eq. 5.29 to obtain Eq. 5.31 as:

$$PR = 100 \left( 1 - \frac{N_{hpt}}{f_{hpt} N_C + N_{hpt}} \right) \quad (5.31)$$

Assume that the stress range spectrum is composed of  $m_i$  stress cycles less than or equal to  $S_L$ , and  $m-m_i$  stress cycles greater than  $S_L$ . Then, for the general case of an S-N curve

with different slopes above and below the CAFL, Eqs. 5.2 and 5.3 are substituted in Eq. 5.5 and arranged to obtain the fatigue life in cycles as:

$$N_C = \frac{1}{\sum_{i=1}^{m_1} \left( \frac{f_i}{N_L \left( \frac{S_L}{S_i} \right)^Y} \right) + \sum_{i=m_1+1}^m \left( \frac{f_i}{N_L \left( \frac{S_L}{S_i} \right)^3} \right)} \quad (5.32)$$

Knowing that:

$$N_{hpt} = N_L \left( \frac{S_L}{S_{hpt}} \right)^3 \quad (5.33)$$

Eqs. 5.32 and 5.33 are substituted into Eq. 5.31 and simplified to obtain:

$$PR = 100 \left( 1 - \frac{\left( \frac{S_L}{S_{hpt}} \right)^3}{\frac{1}{f_{hpt} \left[ \sum_{i=1}^{m_1} f_i \left( \frac{S_i}{S_L} \right)^Y + \sum_{i=m_1+1}^m f_i \left( \frac{S_i}{S_L} \right)^3 \right]} + \left( \frac{S_L}{S_{hpt}} \right)^3} \right) \quad (5.34)$$

As mentioned before the stress range  $S_i$  can be obtained by multiplying the dimensionless ratio  $S'_i$  by  $S_L$  if the design is governed by SLS-I. Knowing this and substituting Eq. 5.26 into Eq. 5.34 the following equation is obtained:

$$PR = 100 \left( 1 - \frac{\left( \frac{1000}{LLR \ DLAR \ SRC \ W} \right)^3}{\frac{1}{f_{hpt} \left[ \sum_{i=1}^{m_1} f_i (S'_i)^Y + \sum_{i=m_1+1}^m f_i (S'_i)^3 \right]} + \left( \frac{1000}{LLR \ DLAR \ SRC \ W} \right)^3} \right) \quad (5.35)$$

This proves that percent reduction in fatigue life is independent of the detail type since the equation does not contain any of  $N_L$  and  $S_L$ . It was also illustrated that slight

variations in spectrum shape, within limits, do not have a significant effect on the percentage of reduction in fatigue life. Therefore, the values of  $N_c$  and  $N_L$  obtained for any detail type and the average spectrum, can be substituted in Eq. 5.28. The values of  $N_L$  and  $N_c$  for detail category a are 2,000,000 and 9,223,786 respectively. Substituting these values in Eq. 5.28,  $n_{pm}$  is obtained as:

$$n_{pm} = \frac{6.0875 \times 10^{16} ADTT PR}{9.223786 \times 10^6 (100 - PR) (LLR DLAR SRC W)^3 + 2 \times 10^{15} PR} \quad (5.36)$$

Rounding the numbers and further simplifying, Eq. 5.36 takes the form:

$$n_{pm} = \frac{6 \times 10^{10} ADTT PR}{9.2 (100 - PR) (LLR DLAR SRC W)^3 + 2 \times 10^9 PR} \quad (5.37)$$

The above equation can be used in conjunction with Figures 5.14 and 5.15 to calculate the allowable number of heavy permit-trucks per month for a specified truck type and weight, span length as well as the percent reduction in fatigue life.

Equation 5.37 can easily be extended to calculate the aggregate fatigue damage for various types of permit-trucks. Consider  $m$  types of heavy trucks to be permitted. Accordingly, there will be  $n_1$  number of truck 1,  $n_2$  number of truck 2 and  $n_i$  number of truck  $i$ . For a specified percentage of reduction in fatigue life, the allowable number of each truck,  $(n_{pm})_i$ , that can be permitted alone is calculated using Eq. 5.37. If truck 1 is permitted only  $n_1$  times, then only a fraction,  $n_1 / (n_{pm})_1$ , of the allowable reduction in life is consumed. Similar to Miner's rule, the specified period of fatigue life is consumed

when;

$$\sum_{i=1}^m \frac{n_i}{(n_{pm})_i} = 1 \quad (5.38)$$

The above equation can be used to permit various number of heavy trucks which have different effects.

### 5.5.2 Example 1

A number of mobile cranes (permit-truck type 1) need to travel between two construction sites, and be allowed to pass several times over a simply supported slab on girder bridge of 20 metres span. The bridge girders have fatigue detail category c. The estimated ADTT on the bridge is 1450. Given the following information, find the number of mobile cranes that can be permitted within a month.

Truck weight : 790 kN.

Axle width : 2.5 m.

Max. speed : 25 km/h

From Table 5.2, the mean fatigue life for detail category c is obtained as;

$$N_{my} = \frac{109,000}{ADTT} = \frac{109,000}{1450} = 75.17 \text{ years}$$

Assuming the MTO, as a policy wishes that the bridge be able to provide a minimum service life of 75 years as calculated by current traffic volume, in anticipation that larger

volumes will occur in the future. Therefore, the allowable percent reduction in service life is calculated as:

$$PR = \frac{75.17 - 75}{75.17} \times 100 = 0.23\%$$

The *LLR* is obtained from Figure 5.14 as 2.4. Assuming an average value of 0.3 for the *DLA*, *DLAR* is calculated using Eq. 5.24;

$$DLAR = \frac{1 + 0.5 DLA}{1 + DLA} = \frac{1 + 0.5 \times 0.3}{1 + 0.3} = 0.885$$

Since the axle width of the truck is larger than 1.8 m., *SRC* is calculated using Eq. 5.25 as;

$$SRC = 1 - 0.07 (A - 1.8) = 1 - 0.07 \times (2.5 - 1.8) = 0.951$$

Substituting the above values in Eq. 5.37, the number of mobile cranes that can be permitted is obtained as;

$$n_{pm} = \frac{6.10^{10} \times 1450 \times 0.23}{9.2 \times (100 - 0.23) (2.4 \times 0.885 \times 0.951 \times 790)^3 + 2 \times 10^9 \times 0.23} = 5.36$$

Maximum 5 mobile cranes per month can be allowed to pass over the bridge.

### 5.5.3 Example 2

MTO decides a policy to adopt 2 percent reduction in life of all bridges due to heavy permit-trucks. Consider a simply supported slab on girder steel bridge of 20 metres span. Within a given month, there is a request for the mobile crane in Example 1 to cross the bridge 12 times. There is another request for a float (permit-truck type 4) of 1600 kN weight to cross the bridge 10 times. Find if more trucks can be allowed to cross the bridge for the current month. (ADTT = 1200)

For the mobile crane:

$$LLR = 2.40$$

$$DLAR = 0.885$$

$$SRC = 0.951$$

$$W = 790 \text{ kN}$$

substituting these variables in Eq. 5.37 the allowable number of mobile cranes,  $(n_{pm})_1$  is calculated as 39.

For the float:

$$LLR = 1.09$$

$$DLAR = 0.885$$

$$SRC = 1.000$$

$$W = 1600 \text{ kN}$$

substituting the above variables in Eq. 5.37 the allowable number of floats,  $(n_{pm})_2$  is calculated as 43. Then, the allowable number of trucks per month and the number of intended passages of these trucks are substituted in Eq. 5.38 to obtain the consumed portion of the allowable reduction in fatigue life as;

$$\frac{12}{39} + \frac{10}{43} = 0.54$$

Only 54 percent of the allowable reduction in fatigue life will be consumed. Therefore, more truck can be allowed to cross the bridge for the current month. If no other trucks cross the bridge, the remaining 46 percent can be carried over the next month.

## 5.6 Sensitivity Analysis

### 5.6.1 Variable S-N Curve Slope

In a previous chapter, it was stated that the S-N curve slope below the CAFL depends on the peak stress range and the shape of the stress range spectrum. Consequently, a limited sensitivity analysis is conducted following to investigate the impact of the spectrum's maximum stress range magnitude,  $S_{max}$ , on fatigue life assuming that the slope below the CAFL is linearly proportional to the maximum stress range in the spectrum. To define this linear relationship, it is assumed that, when the maximum stress range in the spectrum is less than or equal to  $S_L$ , the S-N curve slope below the CAFL is zero, and this slope reaches its 1:3 upper bound value at a stress range of  $S_3$ . Beyond  $S_3$ , there is no further increase in the slope. Since in their tests, Fisher and Mertz, (1983) obtained an S-N curve slope of 1:3 below the CAFL using a maximum stress range of  $1.38 \cdot S_L$ , failing the availability of more reliable data, this value will be taken as the upper bound stress range  $S_3$ .

In Figure 5.16, the safe fatigue life for a fatigue detail category d is plotted as a function of  $S_{max}/S_L$ . The spectrum of the total truck traffic in Ontario is used with an ADTT of 3000. Knowing that the stress ranges above the CAFL are only responsible for reducing the fatigue life from an infinite to a finite life, and to preserve the property of the spectrum, only the tail portion, i.e. the part beyond  $S_L$  of the spectrum is scaled such that  $S_{max}$  in the spectrum is varied between  $1.01 \cdot S_L$  and  $1.7 \cdot S_L$ ,  $1.7 \cdot S_L$  being the actual maximum value in that spectrum (Figure 5.2). By scaling the stress ranges larger than  $S_L$ , only the tail portion of the spectrum is varied, i.e. shortened or elongated. Safe life is calculated using the corresponding S-N curve slope defined by  $S_{max}$  as per the above procedure. As seen in Figure 5.16, for the variable slope case, the fatigue life increases as the amplitude of the maximum stress in the spectrum decreases. The fatigue life is almost constant beyond the upper bound stress range  $S_3$  where the slope remains as 1:3. The difference in fatigue life between the constant and variable slope cases increases as

the maximum stress range decreases. This is as a consequence of the decrease in the slope of the S-N curve below the CAFL with decreasing stress range for the variable slope case.

As the Ontario truck traffic spectrum contains stress ranges 70 percent larger than  $S_L$ , the constant 1:3 slope and variable slope results are identical and Eq. 5.37 remains unchanged and applicable without presumption of extreme conservatism.

### 5.6.2 Impact of Infrequent Extreme Stresses

Fatigue damage due to Ontario trucks with stress ranges larger than the OHBD-fatigue truck's stress range is investigated in this section for a constant 1:3 S-N curve slope. In Figure 5.17, the cumulative percentage of fatigue damage is plotted as a function of the ratio,  $S'_i$ , of Ontario trucks' stress range to the OHBD-fatigue truck's stress range. This cumulative percentage of fatigue damage is calculated from right to left to sum-up the effects of all trucks more damaging than a given  $S'_i$  value. As seen in the figure, the total contribution of all the stress range ratios,  $S'_i$  larger than 1.2, to the fatigue damage is less than 0.7 percent. Thus, the most important effect of stress range ratios larger than 1.0 is not their contribution to the cumulative fatigue damage, but rather their alteration of fatigue life from infinite to finite.

This also explains why, in Figure 5.16, there is not a notable change in fatigue life when the slope below CAFL is assumed to remain constant. This is actually due to the very small frequencies of the stress ranges greater than  $S_L$  which do not have a significant contribution to the fatigue damage.



## 5.7 Permit Policy

### 5.7.1 Tolerance of Permit Trucks

It is possible, using Eq. 5.37, to formulate a number of different policies governing the emission of special permits for extra-heavy trucks. Yet, for any such guideline, an apriori decision must be made on the tolerable fatigue-life reduction attributable solely to heavy permit-trucks, and a schedule of permit fees must be established to recoup the progressive loss of serviceability. A simple and effective permit policy is proposed following, upon which more elaborate strategies can be modelled as needed. It is also useful to illustrate the relative fatigue damage cost-burden to be assumed by permit trucks and regular truck-traffic as well as its actual impact on permit fees.

The number of permissible heavy permit-trucks on a given bridge is a function of the assumed service life of the bridge. OHBDC requires a minimum service life of 50 years, however, there are bridges that have been serving for more than 50 years. It has already been shown that percentage reduction of service life is a more meaningful and manageable parameter than the absolute reduction in number of years. A reasonable percentage of reduction should therefore be assumed. Since a one year reduction in 50 years service life seems reasonable considering the practical and economical consequences of permitting heavy trucks, a 2 percent reduction in service life is adopted.

ADTT, weight and axle width of the trucks, as well as the DLA are the factors affecting the number of trucks that can be permitted. Conservative values can be assigned to these variables to establish a policy to permit heavy trucks. It was mentioned before that, as the ADTT decreases the number of heavy trucks to be permitted decreases for a specified percentage of reduction in life. Therefore a lower bound value of ADTT which is 1000 for class A highways is assumed. The weights of the 5 truck models provided by the MTO are taken as a basis, and the trucks which causes the most detrimental effect for a specified span length are considered only. In other words, the envelope values of

the LLR obtained previously are used. However, these LLR are divided by 0.8 to account for the effect of OHBD-fatigue truck. Furthermore axle width and the DLA are assumed to be the same as those of the normal trucks to be on the safe side.

The above assumed values are substituted in Eq. 5.37 to obtain the allowable number of heavy trucks per month for various span ranges. For slab on girder steel bridges, 15, 30, 45 and 60 heavy trucks per month may be permitted for spans shorter than 35 metres, 35 to 50 metres, 50 to 75 metres, and longer than 75 metres respectively.

Truck model 5 has the most detrimental effect on stringers up to 10 metres in steel truss bridges. Since this truck has only 2 axles and the distance between the axles is about 11 metres (longer than usual length of stringers), two cycles per passage is considered appropriate. Accordingly, for steel truss bridges, 8, 12, and 33 heavy trucks per month may be permitted for stringer spans less than 7 metres, 7 to 10 metres and more than 10 metres respectively.

It is noteworthy that, Ontario's truck traffic itself causes stress ranges that exceed the OHBD-fatigue truck's stress range. Some of these stress ranges, although not very frequent, exceeds the design stress ranges by approximately 70 percent. Therefore, including the stress range of heavy permit-trucks in the spectrum does not have much effect. Consequently, the reduction in life caused by the heavy permit-trucks is not considerable, and a large number of heavy trucks can be permitted. This can also be observed from Eq. 5.23. The fatigue life in cycles,  $N_c$ , under normal truck traffic is in the denominator of the equation. Obviously, as the amplitude of the stress ranges in the spectrum increases, the fatigue life in cycles decreases; this consequently results in a larger number of heavy trucks permitted for a constant percentage of reduction in life. Thus, even in the absence of heavy permit-trucks, due to the high amplitude stress ranges in the spectrum, fatigue life is shortened. Therefore, Ontario's truck traffic itself creates fatigue-related problems in bridges which would have been otherwise thought to have an infinite fatigue life.

### 5.7.2 Fatigue-Related Schedule of Permit Fees

Bridges are investments intended to provide a certain service life. Therefore, any percentage reduction in this life is financially equivalent to the same percentage of the investment's cost. Accordingly, the reduction in the life of the bridges in Ontario should be compensated by the trucks which have detrimental effect on the bridges. In the case of heavy permit-trucks, for 2 percent reduction in fatigue life, the allowable number of trucks to be permitted per month was calculated for various span ranges. Using this information, the toll that should be charged to each heavy permit-truck per passage of a bridge, to compensate the reduction in life, can be calculated as follows.

Using the database available, the average of the present cost,  $C_{av}$ , of the bridges in Ontario can be calculated. The 2 percent of this average cost,  $PC_{av}$ , will be paid by the heavy permit-trucks during the bridge service life. Assuming a bridge service life of 50 years, the future value of  $PC_{av}$  is calculated as;

$$FVPC_{av} = PC_{av} (1 + i)^{50} \quad (5.39)$$

where  $i$  is the yearly rate of inflation. This future cost will be paid by the trucks in 50 years by yearly instalments which will increase with the inflation rate. This can be expressed mathematically as;

$$YI \frac{(1 + i)^{50} - 1}{i} = FVPC_{av} \quad (5.40)$$

where  $YI$  is the present time instalment or the instalment for the current year. Knowing the allowable number of heavy permit-trucks per month for specified span ranges, an

average number of trucks per year,  $NT_{av}$ , can be calculated as;

$$NT_{av} = 12 \times \frac{15k_1 + 30k_2 + 45k_3 + 60k_4}{n_b} \quad (5.41)$$

where,  $k_1$  is the number of bridges with spans less than or equal to 35 metres,  $k_2$  is the number of bridges with spans greater than 35 metres and less than or equal to 50 metres,  $k_3$  is the number of bridges with spans greater than 50 metres and less than or equal to 75 metres,  $k_4$  is the number of bridges with spans greater than 75 metres and  $n_b$  is the total number of bridges in Ontario. Using Eqs. 5.39, 5.40 and 5.41 the toll to be charged to a permit-truck per passage of a bridge is expressed as;

$$MPT = \frac{YI}{NT_{av}} = \frac{PC_{av} i (1 + i)^{50}}{NT_{av} ((1 + i)^{50} - 1)} \quad (5.42)$$

This calculated amount of money should be multiplied by the number of bridges on the route when charging the heavy permit-trucks.

For the specified allowable number of permits, the heavy permit-trucks are responsible only for the 2 percent reduction in life, while the Ontario truck traffic is responsible for the 98 percent of the life hence the cost. The same logic can be applied to find the amount of toll to be paid yearly by the normal trucks. In this case, instead of the 2 percent of the cost, 98 percent of the cost will be used in the above equation and the total number of trucks in Ontario will be substituted in the equation instead of the average number of heavy permit-trucks. The obtained result is only for per bridge. Therefore, it should be multiplied by an average number of times a typical Ontario truck uses the bridges within a year.

### 5.7.2.1 Example 1

The average current cost of the superstructure of a slab-on-girder steel bridge in Ontario is 250,000 dollars. The total number of municipal and provincial steel bridges,  $n_b$ , in Ontario is approximately 3500. It is assumed that: (i)  $k_1$ ,  $k_2$ ,  $k_3$  and  $k_4$  are 2350, 1000, 100 and 50 respectively; (ii) inflation is 6 percent; (iii) a heavy permit-truck will cross 15 bridges on its route. Calculate the toll to be charged to the truck.

Two percent of the cost,  $PC_{av}$  is 5000 dollars which must be paid by all heavy permit-trucks during the service life. Substituting the given values in Eq. 5.41, the average number of heavy trucks to be permitted is found as 249 per year. Using Eq. 5.42 the toll to be charged is calculated as 1.28 dollars per passage per bridge. Since this truck will cross 15 bridges on its trip, the toll to be charged is  $15 \cdot 1.28$ , i.e. 19.2 dollars.

### 5.7.2.2 Example 2

Here, the same average cost of a steel bridge superstructure in Ontario is used i.e., 250,000 dollars. Again the total number of steel bridges,  $n_b$ , in Ontario is 3500. The total number of registered trucks in Ontario is 130,000. The ADTT per bridge in Ontario is estimated as 1000. Assuming an inflation rate of 6 percent, calculate the toll to be charged to each normal truck for the current year.

Ninety eight percent of the average cost of a bridge,  $PC_{av}$  is 245,000 dollars which will be paid by 130,000 trucks using Ontario bridges. Substituting these values in Eq. 5.42, the toll per truck and per bridge is estimated as 0.12 dollars. The average number of trucks crossing a bridge per year being  $1000 \cdot 365$ , or 365,000, therefore, the average number of times a truck uses a bridge is  $3500 \cdot 365,000/130,000$  or 9827. Therefore, the toll that a normal truck should pay per year is  $0.12 \cdot 9827$  or 1179 dollars.

## **CHAPTER 6**

### **INTRODUCTION TO SEISMIC PERFORMANCE OF EXISTING SHORT TO MEDIUM SPAN SLAB-ON-GIRDER STEEL BRIDGES**

#### **6.1 General**

Knowledge about Canadian seismicity is relatively recent and the vast majority of existing bridges have not been designed to resist earthquakes. Furthermore, the seismic-resistant provisions of the current Canadian design standards for highway bridges are still deceptively primitive and may, in many cases, be totally ineffective in ensuring a satisfactory seismic-performance. A complacent attitude long existed regarding these severe deficiencies in Canada, and the necessity to address this problem was first realized only shortly before the Loma Prieta (San Francisco) earthquake of 1989, and clearly articulated only following it. During that moderate earthquake of Richter Magnitude 7.0, a number of bridges failed, as far as 100 km away from the epicentral region, at a great loss of life and with large indirect costs and economic losses to the region. Although numerous bridges also failed in previous North American earthquakes (e.g. San Fernando California 1971, and Whittier 1987), the life-safety hazards that exist due to the poor seismic-resistance of existing bridges has only been fully appreciated in 1989, when images of the failures sustained by the San Francisco Bay Area's bridge infrastructure, and number of deaths resulting from these failures, were broadcast worldwide.

It is noteworthy that earthquakes of equal or greater intensity than that of Loma Prieta are anticipated in many regions of Canada. Unfortunately, the level of awareness to the existing seismic threats is considerably lower in Canada than in the San Francisco Bay Area, and Canadians are unquestionably less prepared to survive an earthquake than Californians. Hence, the economic disruption and loss of life anticipated in a Canadian earthquake could be very severe, particularly in light of the little redundancy of the Canadian network of transportation links.

In this perspective, the existing older infrastructure constitutes the largest seismic hazards. There is now a heightened interest in the potential seismic risk to the infrastructure in many parts of Canada. The perceived risk to old structures is great, and agencies responsible for the transportation systems are faced with the daunting prospect of retrofitting or replacing many bridges, at great expense.

The appropriate balance between risk and cost is one in which the cost related to future consequences of seismic damage is compared with the cost of retrofits. This kind of economic decision making can only be carried out if there is a good understanding of the risk to the structure, the possible damage states that exist, the consequences of those damage states, the means to control the structural behaviour to mitigate those consequences by retrofitting, and the cost of such retrofits. It is essential to have the means to assess the current risk (assuming no retrofit is performed) before trying to decide upon how to mitigate that risk through retrofits. The seismic-performance of a bridge is an intricate function of its seismic exposure, its particular geometric, detailing and dynamic characteristics, and the level of conservatism present in its design and/or construction. There currently exists no broadly available methodology to assist the transportation agencies in assessing this risk and/or to priority-rank bridges considering these above factors. Therefore, a guideline is required to identify which bridges are most likely to be a problem, and a good strategy should be established to prioritize the retrofit of old bridges. This may be achieved by determining the required strength and ductility demand for critical bridge components, as a function of structure geometry, structure type

and earthquake excitations of various characteristics and intensity representing the ones in Eastern and Western parts of Canada.

Throughout this study, the development of new knowledge and models of the ultimate behaviour of existing bridges having deficient components will enable structural engineers to perform more accurate non-linear inelastic analyses and behavioral studies of individual bridges, which may lead to savings in retrofit costs and even demonstrate, in some instances, that retrofits may not be needed.

## **6.2 Introductory Information About Some Bridge Components**

The strength of the superstructure components, namely girders and the concrete deck are governed by gravity loading. These components are very rigid and have enough strength reserve to tolerate high intensity earthquake loading. For seismic loads, the most vulnerable elements in a bridge are the connections, columns, foundations and abutments.

Connections are very important for the continuity of the superstructure since various components of a bridge must remain attached together during an earthquake to distribute in-plane forces to the piers and abutments and to preserve structural integrity. Bearings and tie bars are examples of connection elements. Tie bars function as links to provide axial continuity between the decks of multi span simply supported bridges. They do not exist in old highway bridges. They are used in the USA for the design of modern highway bridges and are also usually used to retrofit old bridges, but generally are not used in Canada and are not required by the Canadian bridge design code. Bearings are used to transfer vertical and horizontal loads between bridge superstructures and substructures. They are also used to prevent restraint and restraint-induced loads. The fixed bearings permit the supported superstructure to rotate about a horizontal axis perpendicular to the direction of bridge span and prevent transverse and longitudinal



translation. Expansion bearings perform the same functions as fixed bearings except they are designed to move longitudinally to accommodate changes in length due to temperature variations and other effects. Rocker and sliding bearings are mostly used in old steel bridges and are not ductile components (Libby, 1979). They are the most vulnerable types of bearings (Seismic..., 1987). Therefore, special attention should be paid to these components of old bridges and enough strength should exist to ensure a good performance during an earthquake. If needed, these components can easily be retrofitted or replaced. Therefore, the strength demand of these components will be studied.

Steel columns and piers are bridge components which transfer gravity and earthquake forces to the foundations. They have high energy absorbing capacity if designed to have adequate ductility. Failure of these bridge components may result in total collapse of the structure. It is possible to retrofit these components to have adequate strength and if possible, ductility. Therefore, the strength and ductility demand of these components will be studied.

Foundations resist the forces transmitted by the columns or piers, and abutments resist the forces transmitted by the bearings. It is difficult and expensive to retrofit these elements since they lie under the soil. Moreover, the behaviour of these elements is more difficultly predictable as their interaction with soil is complex and difficult to model reliably. Therefore, in this study, the seismic response of the bridges will be investigated assuming that these components preserve their integrity.

Finally, even if the abutments or columns can survive an earthquake without any damage, if the seat width on these elements is not adequately wide, the bridge deck may fall off its support and get damaged. Accordingly, the displacement of the bridge deck on these supports will also be studied.

### 6.3 Description and Properties of the Selected Bridge Types

Three common types of slab-on-girder symmetric steel bridges are studied. The most elementary type is a simply supported bridge with the deck supported by fixed and expansion bearings respectively on the left and right abutments as shown in Figure 6.1. Another type is a two span continuous bridge shown in Figure 6.2. The deck of the bridge is supported at the ends by fixed and expansion bearings on each abutment, and at the middle by columns distributed across the deck, one under each steel girder. The columns are assumed to be rigidly connected to the steel girders but hinged at the foundation, a commonly used type of connection to simplify the design and/or to prevent damage to footings. The third type is a two span simply supported bridge. As shown in Figure 6.3, the span of this bridge is supported at one end by fixed bearings resting on the abutment and at the other end by expansion bearings resting on the columns distributed across the deck. The right span is supported at one end by fixed bearings resting on the same columns which support the left span and at the other end by expansion bearings resting on the abutment. Since the bridge spans are both simply supported on the columns, to obtain a stable structure, columns' ends must be rigidly fixed to the foundation. For both continuous and multi-span simply supported bridges, the strong direction of the columns is set parallel to the longitudinal direction of the bridge.

Span length, deck width and number of spans are taken as geometric parameters to investigate their effect on the seismic response. Knowing that short to medium span bridges are common in existing steel bridges, spans ranging between 20 and 60 metres are studied. To study the effect of deck width on the seismic response, only two lane and three lane bridges are considered. In the case of two lane simply supported bridges, the deck width and girder spacing are also changed to investigate their effect on bearing forces. In addition to the two span continuous and simply supported bridges, those with two lanes and 3, 4, 5 and 6 spans are also considered to investigate the effect of number of spans on the seismic response.

Only the type of bearings which were commonly used in existing steel bridges are considered. These are sliding and pinned rocker bearings (Libby, 1979). However, pinned rocker bearings were mostly used for long span bridges (Heins and Firmage, 1979) and therefore, are beyond the scope of this study. The expansion and fixed types of sliding bearings are shown in Figures 6.4 and 6.5 respectively. The expansion type of sliding bearings is composed of a top plate, a bottom plate, a bearing bar welded to the bottom plate and supporting the top plate, two rectangular steel bars (transverse stopper bars) welded to the top plate to prevent transverse movement and anchor bolts connecting the bottom plate to the concrete abutment. As seen in Figure 6.5, the fixed type additionally has two rectangular steel bars (longitudinal stopper bars) welded on the top plate to prevent longitudinal movement. The movement of the bridge deck is basically prevented by the interaction of the rectangular steel bars with the bearing bar upon sliding. The weakest components in these type of bearings are the anchor bolts. The maximum capacity of this type of bearing is therefore governed by the shear capacity of the anchor bolts. In addition to sliding bearings, elastomeric bearings are also considered in this study, but only in the case of simply supported bridges to illustrate their effects on the seismic response and to compare them with the more traditional older type bearings.

Most of the old highway steel bridges were constructed in the 1960's. Accordingly, to reflect the expected seismic performance of existing highway steel bridges, all the bridges considered in this study are designed to be in compliance with the gravity loadings and strength requirements of the 1961 edition of the American Association of State Highways Officials (AASHO) code (American..., 1961) to represent typical old steel bridges. For the three bridge types mentioned previously, five span lengths are considered; 20, 30, 40, 50 and 60 metres. It is noteworthy that, for continuous and multispan simply supported bridges, each individual span is assumed to have the above lengths. Additionally, for each span two and three lane decks are also considered. The two lane deck has eight metres width and is supported by four girders spaced at two metres intervals. The three lane deck has twelve metres width and is

supported by six girders spaced also at two metres intervals. A one metre overhang is assumed at both sides of the decks for all the bridges. The deck is assumed to be 27 cm thick, i.e. a concrete slab of 20 cm and 7 cm of pavement. A minimum of five metres vertical clearance over under-passing roadway is required by the 1961 edition of the AASHTO code. Consequently, considering the existing variations in vertical clearance for different bridges, the columns of the continuous and multispan simply supported bridges are assumed to have an average length of six metres. Moreover, these columns will be assumed to be Class 2 in the subsequent seismic analyses irrespective of the obtained design, to be able to directly compare the obtained seismic capacities of the bridges excluding the effect of column's class as an additional parameter. This is a reasonable assumption as Class 3 columns were found in only a few cases.

In the design, for all bridges, 20 MPa concrete strength is assumed for the deck and 350 MPa steel is assumed for the steel girders and columns. However, steel strength is varied in the case of a two lane continuous bridge with two spans of 60 metres each, to investigate its effect on the seismic response. All bridges are designed considering full composite action between the deck and steel beams as this condition is most common in existing slab-on-girder bridges. The geometric and structural properties of the designed bridges are listed in Tables 6.1 and 6.2 respectively.

#### **6.4 Description of the Earthquake Loading**

Ground motions can be characterized by the peak acceleration to peak velocity ratio,  $A_p/V_p$ , where  $A_p$  is expressed in units of the gravitational acceleration and  $V_p$  is expressed in metres per second (Zhu et. al., 1988). Ground motions with very high frequency content would produce high  $A_p/V_p$  ratios and relatively small spectral acceleration values at moderate to long periods, whereas the ground motions with intense long duration acceleration pulses would generally lead to low  $A_p/V_p$  ratios and pronounced spectral acceleration values at moderate to long periods. The ground motions with highly

irregular acceleration patterns result in intermediate  $A_p/V_p$  ratios and moderate spectral acceleration values in the medium to long period range.

Accordingly, earthquake zones in the latest Canadian isoseismic maps, developed for the National Building Code of Canada (Basham et. al, 1983; National ..., 1990) are represented by two parameters,  $Z_a$  and  $Z_v$ , the peak acceleration and velocity zones respectively. There are three design spectra representing the zones where  $Z_a < Z_v$ ,  $Z_a = Z_v$  and  $Z_a > Z_v$ . In Eastern Canada, generally,  $Z_a \geq Z_v$  or,  $A_p \geq V_p$  and in Western Canada  $Z_a \leq Z_v$  or  $A_p \leq V_p$  (Heidebrecht et. al., 1983). As the peak ground velocity gets larger relative to the peak ground acceleration, higher spectral acceleration values are obtained in the long period region of the spectrum while spectral acceleration values almost remain the same in the short period region for all  $A_p/V_p$  ratios.

For the purpose of this study, two types of dynamic analyses are conducted, a linear response spectrum analysis and a nonlinear time history analysis. For the elastic analyses, the design spectrum with  $Z_a = Z_v$  is taken for Eastern Canada as a conservative envelope of all cases for which  $Z_a \geq Z_v$ . Similarly, for Western Canada the design spectrum with  $Z_a < Z_v$  is taken as this conservative envelope to study the response of steel bridges.

For the nonlinear time history analyses, four Western United States earthquakes all recorded on rock or stiff soil are chosen to represent Western Canada earthquakes. The smoothed acceleration spectrum which matches the mean plus one standard deviation (MPISD) of the spectra of these earthquakes for 5 percent damping (Dicleli, 1989) is considered for comparison with the Western Canada design spectrum. In Figure 6.6 this spectrum is compared with the Uniform Building Code (UBC) spectrum used for dynamic analyses of structures in the absence of a specific site earthquake, (International..., 1991). The vertical axis in Figure 6.6 is,  $\beta$ , which is the ratio of the absolute spectral acceleration,  $S_a$ , to the peak ground acceleration  $A_p$ . It is observed that the MPISD spectrum of Western United States earthquakes is larger than the UBC spectrum in the

medium to long period region but a bit smaller in the short period region. Agreement is otherwise very good.

For Eastern Canada, several earthquake records of the same characteristics are available. Two of these earthquake records, one on bedrock and another on alluvion deposit are considered for the non-linear time history analyses. An average spectrum would not be suitable for comparison with the design spectrum due to small number of real (i.e. non-synthetic) earthquake records available for Eastern Canada. Therefore, the pseudo-acceleration response spectrum of each earthquake is generated separately.

Additionally, a record from the Erzincan earthquake, which has occurred in Eastern Türkiye (Turkey), is also considered in one of the analyses due to its interesting acceleration time history characteristics, which are: the concentration of nearly all input energy over a short time interval, its low frequency content, and the presence of three successive large acceleration pulses (Saatcioglu and Bruneau, 1993).

The properties of all the earthquake motions used in this study are shown in Table 6.3. The response spectra of the earthquake records considered, and the design response spectra are illustrated in Figure 6.7. As seen in the figure, for the Eastern and Western Canada design response spectra,  $\beta$  in the short period range is represented by a flat plateau of magnitude 3. This plateau continues up to 0.25 second for the Eastern Canada spectrum and 0.5 second for the Western Canada spectrum. This is a conservative assumption since the true  $\beta$  must start from 1.0 at a period of 0, i.e. when a structure is infinitely rigid, its spectral acceleration is equal to peak ground acceleration. Therefore, there is a considerable difference between the design and other spectra in the very short period ranges. In the long period range, the magnitudes of  $\beta$  for the two Eastern Canada earthquake records are negligible compared to that of the Eastern Canada design response spectrum. This is partly due to the shorter duration of the existing records compared to that expected of future earthquakes in that region. As observed in Figure 6.7, the Eastern Canada earthquakes considered in this research excites mostly structures with short

periods, yet this is not the case for Western United states earthquakes. The shape of the MP1SD spectrum of four Western United states earthquakes matches closely that of Western Canada design spectrum, however the amplitude of spectral accelerations are 84 percent of the design Spectrum.

## **CHAPTER 7**

### **SINGLE SPAN SIMPLY SUPPORTED SLAB-ON-GIRDER STEEL BRIDGES**

#### **7.1 General**

A simply supported bridge is a deck spanning between two abutments. The deck is generally attached to one abutment by a fixed bearing (hinge) and on the other abutment it is supported by an expansion bearing (roller). The deck is composed of deep steel girders supporting a thick reinforced concrete slab. It is a very stiff element in its horizontal plane which has adequate strength reserve to resist seismic loads elastically. Therefore, the seismic behavior of a simply supported bridge is linear until the bearings are damaged, then the bridge deck may slide in both directions and collide with the abutment walls or fall off its support.

Generally, short to medium span old steel bridges are supported by sliding-bearings. The anchor bolts in these bearings are vulnerable to earthquakes since they were not designed to resist seismic forces. In the longitudinal direction, when the anchor bolts at the fixed bearings are severed, the deck is free to slide. As seen in Figure 7.1, should such a failure happen, frictional forces will develop between the bottom plate and the concrete abutment at the fixed bearing, as well as, to a lesser extent, between the top plate and the bearing bar at the expansion bearing. Collision of the deck with the abutment walls may occur repeatedly when the friction resistance at the bearings is



exceeded. Since there are abutment walls at each end of the bridge, the movement of the deck in the longitudinal direction is restricted by the width of the seat gap or expansion joint which is generally no more than 4-5 cm for the span ranges considered herein. Therefore, assuming that other local failure modes not considered in this study are inconsequential, the bridge deck can not fall off its support, unless;

- i) the abutments are severely damaged or displaced excessively;
- ii) abutment's edge fails locally when the bearings slide to a point near the edge where insufficient bearing resistance can be provided to resist the normal reaction forces due to gravity loading, either due to a reduction of bearing surface increasing the effective stresses to exceed the material's capacities, or by the absence of adequate detailing to account for this shift in load application point;
- iii) the distance between the bearing centerline and the support edge is less than the expansion joint width.

In the transverse direction, when the anchor bolts are severed, friction forces are produced between the bottom plate and the concrete abutment's surface upon sliding, as illustrated in Figure 7.2. When these forces are exceeded, the bridge deck slides, and it may fall off its support if there is not adequate space between the exterior bearing centerline and the support edge, or if the abutment's edge fails locally for the same reasons stated above. Should that happen, the portion of the gravity load previously supported by the exterior girder is transferred to the nearest interior girder by cantilever action of the slab. Assuming that a typical slab-on-girder steel bridge has an average of 2 metres girder spacing, 1 metre of slab extending beyond the exterior girder, a 200 mm thick slab with 70 mm of pavement, a 150 mm high curb of 1.5 metre width, a concrete railing of 150 mm width and 600 mm height, and an average girder weight of 3 kN/m, the unfactored shear and moment acting on the slab when an exterior girder falls off its support is found to be 30 kN/m and 53 kN·m/m. To resist these loads, a minimum

percentage of transverse top reinforcement of 0.45% would be required, assuming that 400 MPa steel is used to reinforce the slab. Should a slab not have the needed percentage of transverse steel, the exterior portion of the bridge deck could be severely damaged and make the bridge unusable. Therefore, sliding in the transverse direction seems critical and is studied in the subsequent parts of this chapter.

## **7.2 Description of the Analytical Models**

For the purpose of this study, two types of dynamic analyses are performed. The first type is a linear elastic response-spectrum analysis. The objective in performing this analysis is to find the bearings' transverse and longitudinal forces and maximum displacement or rotation of the deck at the midspan and at the bearings. The program SAP90 (Wilson, and Habibullah, 1990) is used for this purpose. The other type of analysis performed is a nonlinear time-history analysis capable of accounting for the nonlinear response at the expansion joints, namely impact and friction. The objective in performing this analysis is to find the maximum transverse sliding displacement at the bearings as a function of various earthquake intensities, friction coefficients and span length. The program NEABS (Penzien et. al., 1981) is used for this purpose. It is noteworthy that this program needed modifications in order to be used in computers operating with MS-DOS or UNIX systems.

### **7.2.1 Linear Elastic Modelling of Bridges**

For the elastic case, the bridge span is divided into 10 beam segments. The mass of the bridge is divided and lumped at each node linking these beams. The supports are modelled by summing the stiffnesses of each of the bearings and lumping their overall effect at the centerline of the bridge by using equivalent rotational and translational springs.

The stiffness of the fixed type of sliding-bearings in the longitudinal direction is formulated below, considering the following two assumptions : (i) the anchor bolts are assumed to be well embedded in the abutment, therefore their slippage is not considered and; (ii) the bottom plate in sliding bearings is assumed as rigid since it is a relatively thick element and is stiffened against bending by the bearing bar. Accordingly the longitudinal stiffness of the sliding bearings is determined considering only the contributions from bending of the bearing bar and elongation of the anchor bolts as seen in Figure 7.3. The spring coefficient for each of these contributions is determined by imposing a unit horizontal displacement at the tip of the bearing bar. The bearing bar is modelled as a cantilever and its stiffness,  $k_{bb}$ , is calculated as;

$$k_{bb} = \frac{3 E I_{bb}}{h_{bb}^3} \quad (7.1)$$

where,  $E$  is the modulus of elasticity of steel,  $I_{bb}$  is the moment of inertia of the cross-section of the bearing bar parallel to the deck, about an axis in the bridge's transverse direction, and  $h_{bb}$  is the height of the bearing bar. The fixed bearing is assumed to rotate about the tip of the bottom plate and the elongation of each anchor bolt,  $\Delta_{ab}$ , due to an imposed unit displacement at the tip of the bearing bar is calculated using the similarity of triangles as seen in Figure 7.3;

$$\Delta_{ab} = \frac{l_p}{h_{bb}} \quad (7.2)$$

where,  $l_p$  is the length between the anchor bolt in tension and tip of the bottom plate. Then, the spring coefficient of each anchor bolt is calculated as;

$$k_{ab} = \frac{E A_{ab}}{l_{ab}} \Delta_{ab} = \frac{l_p E A_{ab}}{h_{bb} l_{ab}} \quad (7.3)$$

$A_{ab}$  and  $l_{ab}$  in the equation above are respectively the area and the length of the anchor bolt. An equivalent spring coefficient for all effective anchor bolts in the bearing is the sum of the spring coefficients of each anchor bolt in tension. Obviously, the bearing bar

and the anchor bolts as per the above model are acting as springs connected in series, and the longitudinal spring coefficient,  $k_{bL}$ , for each bearing is obtained as;

$$k_{bL} = \frac{k_{bb} \sum_{i=1}^{n_{ab}} k_{ab_i}}{k_{bb} + \sum_{i=1}^{n_{ab}} k_{ab_i}} \quad (7.4)$$

where, in the above equation,  $n_{ab}$  is the number of anchor bolts in tension. By using this spring coefficient for each bearing, with a bearing located under each girder as normally done, the longitudinal effect of the bearings-set is transformed into one translational spring parallel to the span and one rotational spring about a vertical axis perpendicular to the bridge deck. Both springs are located at the centerline of the bridge deck and at the one end of the bridge where fixed bearings are present. In the transverse direction sliding-bearings are very rigid, hence, the bearing stiffness,  $k_{bT}$ , in this direction need not to be calculated.

In the case of elastomeric bearings, the longitudinal and the transverse stiffnesses  $k_{bL}$  and  $k_{bT}$  are calculated as;

$$k_{bL} = k_{bT} = \frac{G_e A_e}{t} \quad (7.5)$$

where,  $G_e$  is the shear modulus of elasticity of the elastomer,  $A_e$  is the area of the elastomer parallel to the bridge deck, and  $t$  is the thickness of the elastomeric bearing. Using this spring coefficient for each bearing again located under each girder, the longitudinal and transverse effect of the bearings-set are transformed into two translational springs parallel and perpendicular to the bridge span and one rotational spring about a vertical axis perpendicular to the bridge deck. These springs are located at the centerline of the bridge deck at each end of the bridge.

For both sliding and elastomeric type of bearings, the translational stiffnesses of the bearings-set,  $K_{bL}$  in the longitudinal and  $K_{bT}$  in the transverse direction are obtained by summing the stiffnesses of all the bearings:

$$K_{bL} = \sum_{i=1}^{n_b} k_{bL_i} \quad (7.6)$$

$$K_{bT} = \sum_{i=1}^{n_b} k_{bT_i} \quad (7.7)$$

where  $n_b$  is the number of bearings. The bridge deck is assumed to have infinite in-plane rigidity in the longitudinal direction (i.e. axial deformation of the deck at bearings locations is neglected). Then, a unit rotation is imposed at the centerline of the bridge deck and the bearing forces in the longitudinal direction are obtained by multiplying the stiffness of each bearing by its distance,  $l_{bi}$ , to the centerline of the bridge deck, as illustrated in Figure 7.4. The moment of these forces about the centerline of the bridge deck is the rotational stiffness,  $K_{b\theta}$ , of the bearings-set expressed as follows;

$$K_{b\theta} = \sum_{i=1}^{n_b} k_{b_i} l_{b_i}^2 \quad (7.8)$$

### 7.2.2 Non-linear Inelastic Modelling of Bridges

For the inelastic case, the sliding of the bridge deck in the transverse direction is investigated. Due to the difficulty in directly modelling the sliding of the bridge in the transverse direction using the program NEABS, a simpler equivalent model is used to simulate the nonlinear behavior of the actual structure. The following two sub-sections address respectively the assumptions in modelling and the model itself.

### 7.2.2.1 Assumptions in Modelling

The ultimate strength of bearings is obviously larger than their frictional resistance alone, i.e. a force larger than the bearing strength is required first to actually rupture the anchor bolts since bearing and friction are co-existent prior to the initiation of pure sliding, as illustrated in Figure 7.5. However, this aspect of behavior is not included in the model nor is it accounted for by the computer program used. Instead, the bearings are assumed to be immediately damaged and their initial contribution is ignored. Since the peak of ultimate bearing resistance prior to anchor bolt failure, as illustrated in Figure 7.5, occurs only once and does not affect significantly the total amount of energy dissipated through sliding, refinements of the hysteresis models are unwarranted.

In the non-linear inelastic model, free rotation at both ends of the bridge is assumed. Although, some rotational resistance is produced by the longitudinal direction friction forces at the end of the bridge where fixed bearings are located, results from elastic analyses showed that very high longitudinal forces are produced in the bearings by the in-plane end moment at that support, and these forces can easily overcome the aforementioned resistance and rotate the bridge's end. For example a 2-lane 40 metres long simply supported bridge was analyzed using Western Canada design spectrum scaled to 0.2g peak acceleration. The calculated longitudinal force at the exterior bearing, assuming restraint against rotation at one bridge's end, was 781 kN. The frictional resistance at the same bearing was also calculated by multiplying the bearing reaction force due to dead load by a friction coefficient of 0.4, appropriate for friction between the steel bottom plate of the bearing and the concrete abutment's surface (PCI, 1971; CPCI, 1989). The resulting frictional resistance was 140 kN, at least five times smaller than the bearing longitudinal force due to seismic loading. Results from elastic analyses also showed that the longitudinal displacements at the bearings due to free rotation at the support are very small. This shows that, only a negligible amount of energy is dissipated when the bridge end rotates. Consequently, the assumptions made in modelling do not produce significant errors in simulating the behavior of the actual system.

### 7.2.2.2 Description of the Model

From the elastic analyses, it is found that the first or fundamental transverse direction mode is dominant. Knowing that the bridge deck always stays elastic before and after the sliding of bearings, the structure can be idealized as a single degree of freedom (SDOF) system which slides after the friction resistance is exceeded. It is customary to represent the mode shape of the real system by the following trigonometric function;

$$\Phi(x) = \sin \frac{\pi x}{L} \quad (7.9)$$

where  $L$  is the span length. Then, the generalised mass  $m^*$ , stiffness  $k^*$  and the effective force  $P_{eff}$  for the equivalent model are calculated as (Clough and Penzien, 1975);

$$m^* = \int_0^L \frac{m}{L} (\Phi(x))^2 dx = \int_0^L \frac{m}{L} \sin^2 \frac{\pi x}{L} dx = \frac{m}{2} \quad (7.10)$$

$$k^* = \int_0^L EI_D \left( \frac{d^2 \Phi(x)}{dx^2} \right)^2 dx = \int_0^L EI_D \left( \frac{\pi^2}{L^2} \sin \frac{\pi x}{L} \right)^2 dx = \frac{\pi^4 EI_D}{2 L^3} \quad (7.11)$$

$$P_{eff} = \ddot{u}_g \int_0^L \frac{m}{L} \Phi(x) dx = \ddot{u}_g \int_0^L \frac{m}{L} \sin \frac{\pi x}{L} dx = \frac{2m}{\pi} \ddot{u}_g \quad (7.12)$$

where  $m$  is the mass of the bridge,  $I_D$  is the moment of inertia of the bridge deck about a vertical axis perpendicular to the deck and  $\ddot{u}_g$  is the acceleration of the ground motion. Normally, when a SDOF system with a mass,  $m^*$ , is subjected to ground motion, the force acting on the system is its mass times the ground acceleration, i.e.  $m^* \ddot{u}_g$ . Note that for the simplified equivalent system considered, the effective force expressed above must act on the system, not  $m^* \ddot{u}_g$ . Therefore, the ground acceleration history is corrected by an

acceleration modification factor,  $AMF$ , as expressed below and then used in NEABS as acceleration time history input for the equivalent SDOF system.

$$AMF = \frac{P_{eff}}{m^* \ddot{u}_g} = \frac{4}{\pi} \quad (7.13)$$

The idealized structure model and the actual system are illustrated in Figure 7.6. The simplified equivalent model basically consists of three nodes placed sequentially on a straight line. The bridge deck is modelled using a NEABS beam element with negligible flexural stiffness but with axial stiffness equal to  $k^*$ . An expansion joint element with a sliding elastic sub-element is used for the model. The expansion joint and beam elements are connected in series between nodes 1 and 2 and nodes 2 and 3 respectively. While all the degrees of freedom of node 1 are fixed, other nodes are set free to displace in the longitudinal direction, and the generalized mass  $m^*$  is lumped on node 3.

In the program NEABS, the behavior of the sliding sub-element is characterized as elasto-plastic. Elastic behavior exists until the maximum Coulomb friction force, which is equal to the reaction force on the expansion joint times the friction coefficient, is exceeded after which plastic behavior occurs. In-plane elastic forces acting on the expansion joint are related to the elastic deformations by a friction stiffness input by the user. This friction stiffness is actually the stiffness of the bearing element. Elastic restoring forces occur when the bearing element is deformed, and continue until the Coulomb friction forces are exceeded. The magnitude of the Coulomb friction forces varies depending on the magnitude of the reaction force which is the sum of the gravity and seismic forces acting on the bearings. This reaction force is assumed to remain constant during the incremental time interval and it is monitored at the beginning of each time step to determine the frictional sliding resistance. The friction force always acts in a direction opposite from that of relative velocity.



A high value is assigned to the friction stiffness for the sliding friction sub-element in the program to simulate rigid plastic friction force-displacement hysteresis, since the bearings are rigid in the transverse direction.

A structure slides when the ratio of the seismic force acting on the structure to Coulomb friction force exceeds 1. For a constant value of friction coefficient, the magnitude of Coulomb friction force is proportional to the reaction force, and therefore sliding of the bridge is proportional to the ratio of the seismic force acting on the system to the reaction force. However, the seismic forces acting on the actual bridge and the simplified equivalent model are not identical as demonstrated earlier. Therefore, the reaction force due to gravity loading on the actual system can not be applied on the simplified equivalent system. Accordingly, a modified vertical force,  $R^*$ , which represent the sum of the reaction forces on the bearings due to the weight of the bridge is applied on the simplified equivalent system. The ratio of  $R^*$  to the reaction force,  $R$ , on the real system should be equal to the ratio of seismic force acting on the equivalent model to that acting on the real system, so that, sliding occurs at the same time in the equivalent system as it would occur in the real system. The seismic force acting on the equivalent model, before sliding occurs, is expressed as;

$$H^* = (AMF S_a) m^* = \frac{2}{\pi} m S_a \quad (7.14)$$

where  $S_a$  is the pseudo-acceleration of the structure before sliding. The seismic force acting on the real system in the elastic phase is expressed as (Clough and Penzien, 1975);

$$H = \frac{\mathcal{Q}^2}{m^*} S_a \quad (7.15)$$

where the earthquake excitation factor,  $\mathcal{Q}$ , is expressed as;

$$\mathcal{Q} = \int_0^L \frac{m}{L} \Phi(x) dx = \int_0^L \frac{m}{L} \sin \frac{\pi x}{L} dx = \frac{2}{\pi} m \quad (7.16)$$

Substituting Eqs. 7.10 and 7.16 in Eq. 7.15, the total seismic force acting on the real system is expressed as;

$$H = \frac{8}{\pi^2} m S_a \quad (7.17)$$

Then, the modified vertical force is expressed as follows;

$$R^* = \frac{H^*}{H} R = \frac{\pi}{4} R \quad (7.18)$$

This modified vertical force is applied on the beam node connected to the expansion joint in the equivalent model.

These modified parameters together with the modified ground acceleration history are input to the program to obtain the sliding displacement of simply supported bridges in the transverse direction. The program can use mass and stiffness proportional Rayleigh damping. Here, only mass proportional damping is considered in the analyses and a five percent damping ratio is assumed. The first mode circular frequency is used to obtain the mass proportional Rayleigh damping coefficient.

### **7.3 Analyses and Presentation of Results**

#### **7.3.1 Linear Elastic Response Spectrum Analysis**

In all analyses, full composite action between the deck and steel beams is assumed for the response in the longitudinal direction. In the transverse direction, full composite action is also considered assuming that sufficient number of diaphragm beams exist along the span.

The deck of a simply supported bridge, when subjected to gravity loads, is free to rotate at both ends about a horizontal axis perpendicular to the bridge span. When the bridge is subjected to transverse loads, the support rotation about a vertical axis perpendicular to the bridge deck is resisted by the fixed bearings located at one end of the bridge. The degree of this resistance is proportional to the longitudinal stiffness of the bearings. Consequently, bearings with various longitudinal stiffness result in different rotational stiffness, hence different end fixity conditions which may affect the response significantly. Therefore, the effect of bearing type and stiffness will be investigated in the subsequent sections.

#### **7.3.1.1 Description of Bearing Types Used in the Analyses**

Elastomeric bearings are frequently used in the construction of modern highway bridges. They are exceptionally flexible compared to the types of bearings used in old bridges. Their very small stiffness in the longitudinal direction reduces the rotational stiffness considerably. Unlike the other types of bearings, they also have a very small stiffness in the transverse direction which adds more flexibility to the structure. A 4,000 kN/m stiffness per elastomeric bearing is a representative value for the span ranges considered in this study (Heins and Firmage, 1979), and is therefore chosen hereafter. This stiffness is then used to illustrate the effect of this type of bearings on the seismic response of short to medium span simply supported bridges.

Sliding-bearings are one of the most commonly used type of bearings in short to medium span old bridges. The longitudinal stiffness of these bearings is much higher than that of elastomeric bearings. They are also very rigid in the transverse direction. Knowing that the stiffness of the bearing bar in a sliding-bearing is high compared to that of the anchor bolts, the flexibility of these types of bearings in the longitudinal direction is controlled primarily by the number of anchor bolts, their length and diameter. In the

1961 edition of the AASHTO code, the minimum required number of bolts, their length and diameter are recommended for various span ranges. These are;

- Spans 50 feet in length or less; 2 bolts, 1" in diameter, 10" embedment
- Spans 52 to 100 feet ; 2 bolts, 1¼" in diameter, 12" embedment
- Spans 101 to 150 feet ; 2 bolts, 1½" in diameter, 15" embedment
- Spans greater than 150 feet ; 4 bolts, 1½" in diameter, 15" embedment

Thus, based on these recommendations, two bolts of 32 mm diameter per bearing and 4 bolts of 38 mm diameter per bearing are taken as lower and upper bounds respectively for the span ranges considered in this research. An average size bearing bar of 100x400x200 mm (Width x Length x Height) is also considered for the span ranges studied. The bearing plates are assumed to be 250 mm wide parallel to the span. A distance of 170 mm is also assumed between anchor bolts in the longitudinal direction. Using this information and the bearing stiffness model described earlier, the stiffness is calculated as 400,000 kN/m for the bearing with two bolts, and 800,000 kN/m for the other bearing. However, as bearings with four bolts are more commonly used, the calculated stiffness of 800,000 kN/m can be taken as a representative value for the span ranges studied.

Other than the above bearing stiffnesses, the theoretical cases of bearings with zero and infinite rotational stiffness are also considered to investigate the effect of bearing flexibility on the seismic response. Infinite rotational stiffness occurs when the fixed bearing is infinitely rigid in the longitudinal direction. Zero rotational stiffness occurs when the fixed bearing has no longitudinal stiffness (both side of the bridge has expansion bearings). Some types of pinned rocker bearings may approach the later limit. Zero rotational stiffness can also occur when bearings are damaged. In both cases transverse stiffness is assumed to be infinitely rigid.

### 7.3.1.2 Transverse Direction Seismic Response of Bridges

When a simply supported bridge is loaded in the transverse direction, the ends of the deck at each abutment try to rotate about a vertical axis perpendicular to the bridge. Due to this rotation at the support, one corner of the end of the bridge deck moves toward the abutment wall while the other corner of that same end moves away as shown in Figure 7.4. It is found from the elastic response spectrum analyses that the rotation at the bridge end is maximum when the rotational stiffness of the bearings-set is zero. Considering this unfavourable case, and assuming that the deck has infinite axial stiffness, the corner displacements at one end of the bridge deck are calculated by multiplying the rotation of the support obtained using Western Canada design spectrum, by the half width of the deck. These displacements are compared with the expansion joint width to check if collision of the deck with the abutment wall occurs. For a peak ground acceleration of 0.5g, the displacements at the corner of the bridge decks are calculated. The obtained displacements, for all the bridges considered, are much less than the width of the expansion joints shown in Table 6.1. For example for a 2-lane, 40 metres span simply supported bridge, the expansion joint width is 30 mm but the calculated displacement is 4.3 mm. Therefore, the collision of the deck with the abutment wall is not probable in the case of transverse seismic loading. It is noteworthy that, the support rotation of 2-lane bridges are almost twice those of 3-lane bridges.

Response spectrum analyses of five 2-lane and five 3-lane simply supported bridges described in the previous chapter are conducted in the transverse direction for five different bearing stiffnesses using Eastern and Western Canada design spectra, MPISD spectrum of Western United States and the response spectra of two Eastern Canada earthquakes.

As illustrated in Figure 7.7, the forces,  $B_r$ , in fixed type of sliding-bearings are calculated first by dividing the total reaction force at the bridge abutment by the number of bearings, then, summing vectorially this force and the longitudinal force produced by

the resistance to rotation at the bearings. As seen in that figure, bearings farthest to the bridge centerline attract larger seismic forces, and are therefore the most critical bearings. Obviously, the expansion type of sliding-bearings does not attract as much forces as the fixed type since the bridge deck can freely rotate about an axis perpendicular to it. Therefore, in that case, the bearing forces in the transverse direction are calculated simply by dividing the total force on the bridge abutment by the number of bearings. For the elastomeric bearings, the same procedure as for the fixed type of sliding-bearings is applied to obtain the bearing forces at both ends of the bridge due to seismic loading in the transverse direction.

The fundamental periods of the bridges considered in this study are illustrated as a function of span length in Figures 7.8 to 7.10 in two principal directions. The effect of end fixity on the transverse direction fundamental periods of two and three lane bridges is illustrated in Figures 7.8 and 7.9 respectively. The following is observed;

- For all bearing types, fundamental period increases as the span length increases
- As expected, elastomeric bearings have the lowest transverse, longitudinal and rotational stiffnesses. Therefore, the fundamental period is largest and varies between 0.38 and 0.77 second for the ranges of spans considered. Actually, these high fundamental periods are due to the very low transverse stiffness of the bearings and the magnitude of the rotational stiffness has little effect.
- When zero longitudinal or rotational but infinite transverse stiffness is considered, the fundamental period ranges between 0.045 and 0.31 second. The remarkable drop in the fundamental period is mainly due to the increase in the transverse stiffness of the bearings.

- The bridges have the lowest period when infinite longitudinal and transverse stiffness is assumed for the bearings, and the fundamental period, in this case, varies between 0.036 and 0.21 second for the range of spans considered.
- The periods resulting from sliding-bearings are between the periods of the two later cases. It is noteworthy that, as the stiffness of the bearings increases, the fundamental period decreases; this generally results in the attraction of larger seismic forces.

The effect of end fixity on the transverse bearing forces is illustrated in Figures 7.11 and 7.12 for two and three lane simply supported bridges respectively. In these figures, the maximum transverse bearing force coefficient (TBFC) is plotted as a function of span length for various bearing stiffnesses. TBFC is a dimensionless parameter and obtained by dividing the maximum of the resultant bearing forces due to seismic loading in transverse direction, by the bridge mass and the peak acceleration of the ground motion. For the time being, Eastern Canada design spectrum is arbitrarily chosen for the analyses. It is observed that as the longitudinal stiffness of the bearings increases, the TBFC becomes larger. The difference between the forces in bearings of various stiffnesses increases with span length. For all types of bearings, except for elastomeric bearings and bearings with zero rotational stiffness, the TBFC also increases as the span length increases. This is mostly due to the corresponding increase of the in-plane end moment at the support.

In the case of bearings with zero or very small longitudinal stiffness but infinite transverse stiffness, the TBFC is almost constant but bearing forces actually increase with span length since the mass of the bridge increases as the span gets longer. In the case of elastomeric bearings, the much longer period of the bridge is such that it falls in the declining part of the response spectra. This results in attraction of smaller seismic forces, hence smaller bearing forces, which explains why the TBFC is very small compared to those of other types of bearings. It is also found that TBFC decreases slightly as the span

length increases for elastomeric bearings. This is again associated with the increase in the fundamental period as the span length increases, which in turn results in less spectral acceleration values. Also, because elastomeric bearings are very flexible compared to typical bridge decks, the bridge behaves like a SDOF system. It is noteworthy that the large increases in the period of these bridges are not dominantly due to the increasing flexibility of the bridge with increasing span length but rather due to increases in the mass of the bridge.

As an overall trend, transverse bearing forces are proportional to the mass of the bridge, hence to the span length, and they become larger as the bearings get stiffer.

In Figure 7.13, ratios of the bearing forces of 2-lane to 3-lane bridges are plotted as a function of span length. For bearings of infinite longitudinal or rotational stiffness, the bearing forces in 2-lane bridges are approximately 50 percent larger than the bearing forces in 3-lane bridges. Since 3-lane bridges are wider than 2-lane bridges, the distance between the bearings and the centerline of the bridge deck is larger, hence, the moment arms of the bearings in 3-lane bridges are longer. Additionally, 3-lane bridges have more bearings than 2-lane bridges. Consequently, the bearings in 3-lane bridges resist the same force and moment with less effort than the bearings in 2-lane bridges. Thus, the TBFC obtained for narrower bridges can be used conservatively for wider bridges.

Also, as seen in Figure 6.7, the transverse direction fundamental periods of all the bridges with rigid bearings, fall in the small period range where the Canadian elastic design spectra conservatively have a flat plateau of  $3 \cdot A_p$  amplitude. Accordingly, the nearly constant ratio of bearing forces in Figure 7.13 is mainly due to the shape of the Eastern Canada design spectrum used in the analyses over the range of periods considered.

For bearings with zero longitudinal stiffness, the ratio of bearing forces of 2-lane to 3-lane bridges is also almost constant. This is again due to the fundamental periods



of all the 2-lane and 3-lane bridges which fall in the small period range of the design spectrum for the range of spans considered; consequently, all the bridges have the same spectral amplitudes. Although 3-lane bridges are heavier than 2-lane bridges (50% more for the bridges considered), they also have more bearings (6 for 3-lane and 4 for 2-lane). Consequently, the tributary mass and the force per bearing is the same as in 2-lane bridges.

In the case of sliding-bearings, the ratio of bearing forces of 2-lane to 3-lane bridges increases with span length. This is principally due to the increase in the end moment. It is noteworthy that, the ratio of bearing forces increases up to 50 metres length, then, decreases slightly. This is due to the longer period of the 2-lane bridges which fall beyond the flat plateau of the spectrum, resulting in spectral acceleration values less than those of 3-lane bridges.

In summary, the bearing force due to seismic loading in the transverse direction, is a function of the stiffness of the bearings. Simply supported slab-on-girder steel bridges, which have elastomeric bearings or bearings with small longitudinal stiffness, attract almost equal forces regardless of the number of lanes they have. However, in multilane simply supported slab-on-girder steel bridges, which have sliding-bearings, transverse forces in bearings decrease as the number of lanes increases.

In Figure 7.14, the TBFC obtained using two Eastern Canada earthquakes, Baie-St-Paul recorded on alluvion deposit and Chicoutimi-Nord recorded on rock, are compared with those obtained using Eastern Canada design spectrum for two and three lane bridges. Bridges with infinite rotational stiffness at the supports, have very small period. Therefore, infinite rotational stiffness is assumed for the response in transverse direction to illustrate the effect of the difference between the shape of the design and other spectra in the small period region. As expected, the results obtained using the Eastern Canada design spectrum are higher than those obtained using other spectra.

In Figure 7.15 the bearing forces are plotted as a function of span length for Western Canada design spectrum and MPISD spectrum of Western United States earthquakes. It is observed that, the TBFC obtained using the Western Canada design spectrum are higher than those obtained using the MPISD spectrum of Western United States earthquakes. The difference becomes more remarkable for short span bridges of periods less than 0.2 second. This is a consequence of the difference between the shape of the spectra in the small period region. In the MPISD spectrum of Western United States earthquakes, the spectral acceleration linearly increases up to 2.5 times the peak ground acceleration at a period of 0.2 second, while the spectral acceleration is constant and equal to 3.0 times the peak ground acceleration up to a period of 0.5 second in the Western Canada design spectrum.

#### **7.3.1.3 Longitudinal Direction Seismic Response of Bridges**

The effect of bearing stiffness on the longitudinal direction fundamental period is illustrated in Figure 7.10. The same trend is observed as in the case of transverse direction. The fundamental period varies between 0.4 and 0.76 in the case elastomeric bearings and between 0.027 and 0.072 in the case of rigid bearings for the range of spans considered. The difference in the fundamental period for bearings of various stiffness is more remarkable in the longitudinal direction and gets larger as the span length increases.

Response spectrum analyses of the five 2-lane and five 3-lane simply supported bridges described in the previous chapter are conducted in the longitudinal direction for four different bearing stiffnesses using Eastern Canada response spectra. In Figures 7.16 and 7.17 longitudinal bearing force coefficient (LBFC) for two and three lane bridges respectively, is plotted as a function of span length for various bearing types. LBFC is a dimensionless parameter and it is obtained by dividing the bearing force due to seismic loading in longitudinal direction, by the bridge mass and the peak acceleration of the

ground motion. For all types of bearings, the total longitudinal force is shared equally by each bearing supporting the deck, provided that they have the same stiffness.

Elastomeric bearings have the smallest LBFC compared to other types of bearings. This is a consequence of the particularly large first mode period due to low stiffness of the bearings, which effectively attracts smaller seismic forces. It is also observed that LBFC decreases with span length. However, in this case, this does not correspond to a reduction in the bearing forces; actually the bearing forces, which are obtained by multiplying the LBFC by the mass and peak acceleration, increase with span length as the mass of the bridge also increases.

It is observed that, the two-bolts-sliding-bearing has the highest LBFC. This is because the effective modal mass of bridges with these bearings is larger than that of bridges with other types of bearings except the ones with elastomeric bearings, and the period of the bridge is small enough to fall in the constant part of the design spectrum where the spectral acceleration is the highest. Although the LBFC for 2-lane bridges are larger than those of 3-lane bridges, as illustrated in Figure 7.18, the actual bearing forces are equal since each bearing has the same tributary mass.

In summary, when a bridge is subjected to seismic excitation in the longitudinal direction, bearing forces increase as the span length increases and the number of lanes does not have any effect on the bearing forces for the bridges considered in this study. Actually, longitudinal bearing force is proportional to the tributary mass distributed to each bearing. Accordingly, knowing that there is a bearing under each steel girder, bridges with equal girder spacing and equal mass per unit area have the same longitudinal bearing force regardless of the number of lanes.

Assuming that the bridges have sliding-bearings with two bolts (i.e. stiffness of 400,000 kN/m), longitudinal displacements are calculated for the Western Canada design spectrum scaled to 0.5g. These displacements are found to be very small relative to the

expansion joint widths reported in Table 6.1. For example, the expansion joint width of a 2-lane 60 metres bridge is 40 mm while the longitudinal displacement is 8.9 mm. These small displacements are mainly due to the high axial stiffness of the bridge deck. Therefore, unless the failure of the fixed bearings takes place, the collision of the deck with the abutment wall is unlikely in the case of sliding-bearings. The longitudinal displacements are also calculated assuming that the bridge has elastomeric bearings, for the same Western Canada design spectrum scaled to 0.5g. This time, the displacements are found to be larger than the expansion joint width for the range of spans considered. For example, the expansion joint width of a 2-lane 40 metres bridge is 30 mm while the longitudinal displacement is 88 mm. Therefore, collision of the deck with the abutment walls may take place in the case of a high intensity earthquake when elastomeric bearings are used.

#### **7.3.1.4 Effect of Deck Width and Girder Spacing on Transverse and Longitudinal Bearing Forces**

The 2-lane and 3-lane simply supported bridges considered in this study have respectively 8 and 12 metres width and 2 metres girder spacing. In reality, the deck width and girder spacing may vary and therefore affect the bearing forces due to seismic excitation. To investigate this effect, other forty metres span, 2-lane simply supported bridges are studied; the practical limits of deck width and girder spacing for such bridges are assumed to vary between 6 and 10 metres and 1.5 and 3 metres respectively. Accordingly, four bridges with various combinations of deck width and girder spacing are considered. These are bridges with:

- 1) Deck width = 6 m, Girder spacing = 1.5 m, Number of girders = 4
- 2) Deck width = 6 m, Girder spacing = 3.0 m, Number of girders = 3
- 3) Deck width = 10 m, Girder spacing = 1.5 m, Number of girders = 7
- 4) Deck width = 10 m, Girder spacing = 3.0 m, Number of girders = 4

These bridges are first designed to be in compliance with the 1961 AASHO, and then analyzed using Eastern Canada response spectrum assuming that they have sliding-bearings with four bolts. The results are compared with that of the previously considered 2-lane 40 metres bridge with 8 metres deck width and 2 metres girder spacing.

First, investigating the response for excitation in the transverse direction, it is found that the fundamental periods of the 1<sup>st</sup> and 2<sup>nd</sup> bridges are respectively 20 percent larger and almost identical, while those of both the 3<sup>rd</sup> and 4<sup>th</sup> bridges are 27 percent smaller. These periods fall in the constant part of the response spectrum and therefore all bridges undergo the same spectral acceleration.

The transverse bearing forces are 12 percent larger and almost identical respectively for the 1<sup>st</sup> and 2<sup>nd</sup> bridges while they are 27 and 12 percent smaller for the 3<sup>rd</sup> and 4<sup>th</sup> bridges respectively. These limited results indicate that the bearing forces of bridges with decks wider than 4 metres per lane are smaller, but the difference is not very large. Furthermore, narrower bridges can have larger bearing forces, but a difference of no more than 12 percent was obtained for the worst case considered.

However, the fundamental periods of narrower bridges are generally smaller while for longer span bridges periods may fall in the descending part of the response spectrum resulting in less seismic forces hence less bearing forces. Hence, a remarkable difference between the TBFC of these bridges is likely, particularly since the TBFC includes only the mass and peak acceleration and neglects the effects of number of bearings, distance of bearings to the centerline of the bridge deck and rotational stiffness of the bearings-set relative to the deck stiffness. To provide this additional refinement, a new expression will be introduced in Chapter 10 to obtain the bearing forces considering the effects of the above-mentioned factors and the spectrum shape.

For the same bridges subjected to longitudinal earthquake excitation, the results show that the periods of all these bridges are very close and no more than 0.07 second. These periods fall in the constant part of the response spectrum and therefore, the bridges undergo identical spectral accelerations. Results show that bearing forces are larger mostly in wider bridges with fewer bearings. Again a remarkable discrepancy is found between the bearing forces and the LBFC of the bridges due to different mass and number of bearings. Again a new expression will be introduced in Chapter 10 to obtain the correct bearing forces, taking into account the number and stiffness of the bearings as well as the spectrum shape in addition to the peak ground acceleration and mass of the bridge.

### 7.3.2 Nonlinear Time History Analysis

#### 7.3.2.1 Minimum Peak Ground Acceleration Required for Sliding

The sliding of the bridge in the transverse direction after the bearings are damaged, may take place depending on the intensity of the earthquake. For sliding to occur, the total transverse seismic load,  $H$ , as expressed by Eq. 7.17 must be at least equal to the sliding resistance force,  $F_s$ , described for single span simply supported bridges as;

$$F_s = \mu_f m g \quad (7.19)$$

where  $\mu_f$  is the coefficient of friction and  $g$  is the gravitational acceleration. Expressing the absolute spectral acceleration,  $S_a$  as  $\beta A_p$ , where  $\beta$  is the ratio of the absolute spectral acceleration to the peak ground acceleration, and using Eqs. 7.17 and 7.19, the minimum

value of peak ground acceleration required for sliding is expressed by the following equation:

$$A_p = \frac{\pi^2}{8\beta} \mu_f g \quad (7.20)$$

As observed in Figures 7.8 and 7.9 the fundamental periods of all bridges increase with span length. These periods, except those of the bridges with elastomeric bearings, fall in the small period range of the response spectra. As seen in Figure 6.7, in that portion of the response spectra,  $\beta$  increases as the period increases. Thus, since  $\beta$  is in the denominator of Eq. 7.20, the minimum peak ground acceleration required for sliding decreases as the span length increases. This is illustrated numerically using the MPISD spectrum of Western United States earthquakes and a friction coefficient of 0.4 between the steel plates and the concrete seat (PCI, 1971; CPCI, 1989) for sliding-bearings: For 20 metres span 2-lane simply supported bridge, ( $T_1 \approx 0.045$  sec,  $\beta = 1.34$ ) the minimum required peak ground acceleration for sliding is 0.37g while for a 60 metres span 2-lane simply supported bridge ( $T_1 = 0.31$  sec.,  $\beta = 2.5$ ) it is 0.20g.

As observed in Figures 7.8 and 7.9, the transverse fundamental period of bridges with elastomeric bearings are much higher than those of bridges with other types of bearings, and fall in the descending part of the response spectra. Therefore, these bridges attract less seismic force. Furthermore, the friction coefficient is 0.8 between elastomer and concrete (PCI, 1971; CPCI, 1989), twice as much as the friction coefficient between steel and concrete in the case of sliding-bearings. Accordingly, a very high intensity earthquake is required to slide bridges with elastomeric bearings and therefore, their sliding is highly unlikely, and not considered in this study.

It is noteworthy that the peak ground acceleration required for sliding may be affected by vertical ground acceleration that reduces or increases the reaction forces due to gravity loading hence the friction resistance. The correlation between the horizontal and vertical ground movements is complex and not well established, but, simplistically,

considering a zero-mean average, the effect of vertical ground movement on sliding is ignored. Additionally, due to weathering conditions, the abutment floor and bottom of the bearing plates may not be smooth and therefore higher friction forces than expected may be produced; conservatively this is neglected in this study.

### **7.3.2.2 Sliding of Bridges in the Transverse Direction**

Five 2-lane and five 3-lane simply supported bridges described in the previous chapter are analyzed for five different coefficient of friction using six different earthquakes, namely the four Western United States and two Eastern Canada listed in Table 6.3. Sliding displacements at the support of 2-lane and 3-lane simply supported bridges are plotted as a function of span length for various friction coefficient to peak ground acceleration ratios,  $\mu_f/A_p$  ( $A_p$  is expressed as a percentage of  $g$ ), in Figures 7.19 to 7.22. In these figures, the vertical axis is the support sliding displacement in mm per unit peak ground acceleration expressed as a percentage of gravitational acceleration and the horizontal axis is the span length. A total of 300 cases are analyzed using the program NEABS to visually express the relationship between sliding displacement and peak ground acceleration, friction coefficient as well as the span length for various earthquakes. In the analyses, the peak ground acceleration of all the earthquakes used, are scaled to 0.4g while the coefficients of friction is varied between 0.1 and 0.8. Additionally, keeping the  $\mu_f/A_p$  ratio constant, but changing the magnitude of the peak ground acceleration, 60 new cases are also analyzed using the same 6 earthquakes, 2 different  $A_p$  and 5 bridges of different span length, to obtain the relationship between the magnitude of the peak ground acceleration and sliding displacement. It is found that, for the same  $\mu_f/A_p$  ratio, the sliding displacement is linearly proportional to the amplitude of the peak ground acceleration. This is explained in the following section.



### 7.3.2.3 Energy Approach to Sliding

Consider that the previously defined equivalent SDOF system shown in Figure 7.6 is subjected to a ground motion. The total and relative displacements,  $u_t$  and  $u$ , of the mass of the system are expressed by the following equations;

$$u_t = u_g + u_s + u_e \quad (7.21)$$

$$u = u_s + u_e \quad (7.22)$$

where  $u_g$  is the ground displacement, and  $u_s$  and  $u_e$  are respectively the sliding and elastic displacements of the mass. Identical and valid expressions can be obtained for velocities and accelerations simply by putting single and double dots above the variables representing the displacements. Considering the equilibrium of the mass of the system just before sliding, the equation of motion is expressed as;

$$m \ddot{u}_t + c \dot{u}_e + k u_e = 0 \quad (7.23)$$

where  $c$  is the viscous damping coefficient. During sliding, the sum of damping and elastic restoring forces is constant and equal to the friction resistance  $F_s$ . Knowing this and expressing the total acceleration as the sum of the relative and ground acceleration, the above equation is rewritten as;

$$m \ddot{u} + F_s = -m \ddot{u}_g \quad (7.24)$$

The friction resistance for the equivalent SDOF system is expressed as;

$$F_s = \mu_f R^* \quad (7.25)$$

Substitute the above equation into Eq. 7.24 and integrate with respect to the relative displacement:

$$\int m^* \ddot{u} \, du + \int \mu_f R^* \, du = - \int m^* \ddot{u}_g \, du \quad (7.26)$$

The first term on the left hand side of the above equation is the relative kinetic energy of the system expressed as:

$$E_{Rk} = \int m^* \ddot{u} \, du = \int m^* \frac{d\dot{u}}{dt} \, du = \int m^* \dot{u} \, d\dot{u} = m^* \frac{\dot{u}^2}{2} \quad (7.27)$$

During sliding, for an undamped system, elastic displacement stays as it was just before sliding and therefore the incremental change in relative displacement,  $du$ , is actually equal to the incremental change in sliding displacement,  $du_s$ . Consequently, the second term on the left hand side of Eq. 7.26 is the energy dissipated by friction and expressed in the following form:

$$E_f = \int \mu_f R^* \, du_s = \mu_f R^* u_s \quad (7.28)$$

The term on the right hand side of Eq. 7.26 is the relative input energy,  $E_{Ri}$ , of the system. This input energy represents the work done by the seismic force  $m^* \ddot{u}_g$ , on the

equivalent SDOF system. Using these energy terms, the energy dissipated by friction is expressed as;

$$E_f = E_{Ri} - E_{Rk} \quad (7.29)$$

Assume that the friction coefficient and the ground motion are scaled by a factor  $r_1$  and due to this, the relative or sliding displacement increases  $r_2$  times. The new scaled energy terms  $E_f^*$ ,  $E_{Ri}^*$  and  $E_{Rk}^*$ , respectively for friction, relative input and relative kinetic energy are expressed as follows;

$$E_f^* = (r_1 \mu_f) R^*(r_2 u_s) = r_1 r_2 \mu_f R^* u_s = r_1 r_2 E_f \quad (7.30)$$

$$E_{Ri}^* = -\int m^* (r_1 \ddot{u}_g) (r_2 du) = -r_1 r_2 \int m^* \ddot{u}_g du = r_1 r_2 E_{Ri} \quad (7.31)$$

$$E_{Rk}^* = m^* \frac{(r_2 \dot{u})^2}{2} = r_2^2 m^* \frac{\dot{u}^2}{2} = r_2^2 E_{Rk} \quad (7.32)$$

The new friction energy is equal to the new relative input energy minus the new relative kinetic energy, i.e.;

$$r_1 r_2 E_f = r_1 r_2 E_{Ri} - r_2^2 E_{Rk} \quad (7.33)$$

Simplifying the above equation;

$$E_f = E_{Ri} - \frac{r_2}{r_1} E_{Rk} \quad (7.34)$$

Comparing the above equation with Eq. 7.29, for energy equilibrium,  $r_2$  should be equal to  $r_1$ . In other words, when the friction coefficient and the ground motion are scaled by a factor  $r_1$ , the sliding displacement increases by the same factor. This also proves that, for the same ratio of friction coefficient to peak ground acceleration, the sliding displacement of a structural system is linearly proportional to the magnitude of the peak

ground acceleration. The small levels of damping present in steel bridges do not affect meaningfully the above results.

For example, a 60 metres span, two lane simply supported bridge with  $\mu_f=0.1$  is subjected to the S69E component of the 1952 Taft earthquake scaled to  $A_p=0.2g$ . The  $\mu/A_p$  ratio is, 0.1 divided by 0.2, i.e. 0.5, and the sliding displacement is 63 mm. If the same bridge, but with  $\mu_f=0.2$ , is subjected to the same earthquake now scaled to  $A_p=0.4g$ , the same  $\mu/A_p$  ratio of 0.5 is obtained. However, the sliding displacement in this second case is calculated as 125 mm, twice the displacement obtained for the smaller peak ground acceleration. In both cases, the resulting  $u_s/A_p$  is 315 as predicted by the above proposed normalization method.

#### **7.3.2.4 Interpretation of the Results**

The transverse direction sliding displacements of 2 and 3 lanes bridges for Western United States earthquakes are depicted respectively in Figures 7.19 and 7.20. The sliding displacements are obtained by averaging the results obtained using the four Western United States earthquakes. It is observed that, for increasing span length and decreasing  $\mu_f/A_p$  ratio, the sliding displacement increases.

When the bearings are damaged, both ends of the bridges are free to rotate. Figures 7.8 and 7.9 demonstrate that the fundamental periods of these bridges are very small. Therefore, they fall in a part of the response spectra where the amplitude increases with period as seen in Figure 6.7. For the range of spans considered, bridges with longer span have larger periods, and therefore, more force is applied on the structure. This subsequently results in a higher amount of energy to be dissipated by friction. Therefore, the increase in sliding displacement with span length is due to the increase in input energy as the span gets longer.

As seen in Figures 7.19 and 7.20, sliding displacements of 3 lane bridges are smaller than those of 2 lane bridges. Since wider bridges have even smaller periods, the attracted seismic force is smaller than for narrower bridges of the same length for the range of spans considered.

Figures 7.21 and 7.22 are respectively for the transverse sliding displacements of 2 and 3 lanes bridges for Eastern Canada earthquakes. Almost the same trend as for Western United States earthquakes is observed in the case of Baie-St-Paul earthquake. However, in the case of Chicoutimi-Nord earthquake, sliding displacements start decreasing for bridges of 50 metres span and longer ( $T_1 > 0.20$  sec.). As observed from the earthquake spectrum of Figure 6.7, for structures with fundamental periods longer than 0.2 second, the force applied on the structure decreases, hence the sliding displacement decreases.

The sliding displacements obtained using Eastern Canada earthquakes are smaller than those obtained using Western United States earthquakes, yet the displacements are not considerable in both cases. For instance, using a friction coefficient of 0.4 between steel and concrete (sliding-bearings) and a peak ground acceleration of  $0.4g$ , ( $\mu_f/A_p = 1.0$ ) the maximum transverse sliding displacement for a 60 metres simply supported 2-lane bridge is 27 mm in Western United States or Canada.

#### **7.3.2.5 Effect of $A_p/V_p$ Ratio of Earthquakes on Sliding Displacements**

Ground motions are characterized by their  $A_p/V_p$  ratio. In this section the effect of this ratio on the magnitude of sliding displacement is investigated. Sliding displacement time history of a 2-lane, 60 metres bridge is conducted using two different earthquakes: the 1940 El Centro S00E component and 1992 Erzincan, Türkiye NS component. The El Centro earthquake has highly irregular acceleration patterns, i.e. a mixture of short and medium duration pulses, and therefore has an intermediate  $A_p/V_p$

ratio, whereas the Erzincan earthquake has intense long duration acceleration pulses which results in a low  $A_p/V_p$  ratio. The peak accelerations of both earthquakes are scaled to 0.40g. The full acceleration records of both earthquakes are used in the analyses, but the maximum sliding displacements occurred within the first 10 seconds of both earthquakes. Therefore, only the first 10 seconds of the acceleration records and the sliding displacement histories are plotted in Figures 7.23 and 7.24 for the El Centro earthquake, and in Figures 7.25 and 7.26 for the Erzincan earthquake. The maximum sliding displacement obtained from the El Centro earthquake is 54 mm and it is 192 mm in the case of Erzincan earthquake. As seen in Figure 7.23, the El centro earthquake contains generally high frequency, irregular acceleration pulses, which load and unload the structure in short time periods. Therefore, once the structure starts to slide, this sliding motion cannot be sustained for a long time because the force applied on the system remains above the threshold of friction only for a short duration. Accordingly, in Figure 7.24, there is an irregularly increasing and decreasing sliding displacement plot, i.e. after a remarkable sliding as a result of a relatively longer duration pulse, the structure exerts small sliding displacements back and forth. On the contrary, as observed in Figure 7.25, the Erzincan earthquake contains pulses of long duration. Therefore, once the structure starts to slide, the motion can be sustained for a longer time, since the force applied on the system remains above the threshold of friction for a long duration. Therefore, in Figure 7.26, a smoothly increasing sliding displacement is observed. Note that Figures 7.24 and 7.26 have different displacement scale and, as such, should not be misinterpreted.

Additionally, the sliding displacements obtained for the same bridge from the two Eastern Canada earthquakes used in this research, are compared with the those obtained from the El Centro and Erzincan earthquakes. These Eastern Canada earthquakes contain very high frequency acceleration pulses, thus very high  $A_p/V_p$  ratios. As expected, the sliding displacements obtained from these earthquakes are not considerable and much less than the displacements calculated above, being 17 mm and 14 mm for Chicoutimi-Nord

and Baie-St-Paul earthquake records respectively, when peak ground accelerations are consistently scaled to 0.4g.

In fact the distribution of the energy content of the earthquake which is related to the area under the acceleration time history, i.e. the velocity, can be an indication of the propensity of an earthquake to cause high sliding displacements. For earthquake records such as the Erzincan one, energy is concentrated over a short time period in three low frequency, big acceleration pulses; hence, it produces high values of sliding displacements. However, for earthquake records like Chicoutimi-Nord and Baie-St-Paul, the energy is distributed over a longer time period in high frequency pulses; hence, they are not as effective as the Erzincan earthquake. In other words, ground motions with high frequency content or high  $A_p/V_p$  ratio produce very low sliding displacements, whereas ground motions with intense long duration acceleration pulses which have low  $A_p/V_p$  ratios can cause remarkable sliding displacements. Ground motions with highly irregular acceleration pulses and intermediate  $A_p/V_p$  ratios causes medium sliding displacements. As mentioned in the previous chapter, Western Canada earthquakes have intermediate to low  $A_p/V_p$  ratios. Consequently, bridges in the west part of Canada may potentially be subjected to higher sliding displacements than those in the east part of Canada where earthquakes are known to have intermediate to high  $A_p/V_p$  ratios.

## 7.4 Summary

The findings on the seismic behavior of simply supported bridges are summarized as follows;

- As an overall trend, bearing forces due to seismic loading in both transverse and longitudinal directions are proportional to the mass of the bridge, hence, to the span length and they also become larger as the bearings get stiffer.

- The bearing forces due to seismic loading in transverse direction, are functions of the stiffness of the bearings. Bearings with higher longitudinal stiffness and closer to the edge of the bridge deck attract more forces than other bearings.
- Simply supported slab-on-girder steel bridges, which have elastomeric bearings or bearings with small longitudinal stiffness, attract almost equal forces regardless of the number of lanes they have. However, in multilane simply supported slab-on-girder steel bridges which have sliding-bearings, bearing forces due to seismic loading in transverse direction decrease as the number of lane increases. In fact, bearings in wider bridges attract smaller forces.
- Bearing forces due to seismic loading in longitudinal direction are highly dependent on the number of bearings and mass of the bridge. Bearing forces are larger in wider bridges and/or bridges with smaller number of bearings.
- For the same ratio of friction coefficient to peak ground acceleration, the sliding displacement of a structural system is linearly proportional to the amplitude of the peak ground acceleration.
- For increasing span length and decreasing  $\mu_f/A_p$  ratio, the sliding displacement increases.
- Sliding displacements of 3 lane bridges are smaller than those of 2 lane bridges. Nevertheless, the displacements are not considerable in both cases for the earthquakes and range of spans considered.
- The distribution of the energy content of an earthquake which is related to the area under the acceleration time history can be an indication of propensity of an earthquake to cause high sliding displacements. Ground motions with high frequency content or high  $A_p/V_p$  ratio produce very low sliding displacements,



whereas ground motions with intense long duration acceleration pulses or low  $A_p/V_p$  ratios can cause remarkable sliding displacements. Ground motions with highly irregular acceleration pulses which have intermediate  $A_p/V_p$  ratios produce medium sliding displacements. Accordingly, Bridges in Western Canada may potentially be subjected to higher sliding displacements than those in Eastern Canada.

## **CHAPTER 8**

### **CONTINUOUS SLAB-ON-GIRDER STEEL BRIDGES**

#### **8.1 Initial Comments on Behavior**

The deck of the continuous bridges considered in this study, is typically attached to one abutment by a fixed bearing and supported on the other abutment by an expansion bearing. It is also supported by steel columns rigidly connected to each steel girder at midspan to make a moment resisting steel frame. A simple connection is assumed at the base of each column.

Moment resisting structural steel frames are often used in the construction of buildings in seismic regions. Their excellent energy dissipation capability is well established and reliable provided stable hinging mechanisms can develop. To this end, plastic hinges should ideally develop in beams and joints to prevent localized single-storey collapse mechanisms and/or excessive damage to columns, the main gravity-load carrying structural elements. By providing strong columns and weaker beams, yielding can be ensured to occur in the beams and joints prior to possible yield in columns. Moreover, compact sections are used to preclude local buckling of beams under large plastic rotations and to obtain full and repeatable hysteresis loops for energy dissipation.

Unfortunately, in the case of slab-on-girder continuous steel bridges, the beam sizes tend to be substantially bigger than the column sizes, and yielding may occur in columns instead of in beams. Moreover, steel columns used in existing bridges of that type are not necessarily compact. Yielding in columns and beams differ in that columns must safely carry axial load while resisting large plastic moments. Therefore, it may be difficult to obtain complete hysteresis loops and even dissipate any energy due to stability problems.

A review of the literature on the cyclic behavior of steel members revealed that such research has concentrated solely on building elements. The bulk of research considered the strong column weak beam (SCWB) behavior of building frames; only a few researchers addressed the weak column strong beam (WCSB) behavior more typical of steel bridges. Popov et al. (1975) tested two different 2 metres long W-shape sections, a W200x71 and a W200x42 in a sub-assembly of a building frame. These tested columns were compact sections as per AISC (1988) designation and class 1 by the CISC (1991) designation, with slenderness ratios of 38 and 48 about their weak axis, 22 and 23 about their strong axis for the W200x71 and W200x42 sections respectively. It is noteworthy that class 1 sections are capable to sustain large inelastic deformations without any local instability contrary to higher class sections (class 2 and 3). The columns were braced to prevent lateral buckling about their weak axis when loaded perpendicularly to their strong axis (i.e. to produce strong axis bending). Sway-frame loading was applied to the sub-assemblies under constant column axial load. Test results showed that the cyclic behavior of the tested specimens is a function of both the applied axial load to yield axial load ratio, i.e.  $P/P_y$ , and the magnitude of inter-storey drift. Sudden failure was observed in specimens having  $P/P_y$  ratios larger than 0.5. Columns loaded with high axial load tend to become "C-shaped" about their strong axis and failed to strengthen out on load reversal. However, a good cyclic behavior was observed for columns with lower  $P/P_y$  ratios. Later, Takanashi and Ohi (1984) performed a shaking table test on a small scale three story WCSB frame, and it collapsed on the shaking table during the test. Uchida et. al. (1992) conducted shaking table and static moment loading tests of steel cantilever

beam-columns of H-shaped section about their strong axis. They observed that all specimens collapsed about their weak axis due to lateral instability. The lateral instability behavior observed in the shaking table tests seemed more severe than that observed in the static moment loading tests. Recently, Schneider and Roeder (1992), examined the impact of inelastic deformation in columns and panel zones, on the seismic performance of moment resisting steel frames. They tested five moment resisting steel building frames designed according to 1988 edition of the UBC. They found that axial loads prevented initial local buckles from recovering upon load reversal and accentuated flange and web buckling in the column. Stable hysteretic behavior was exhibited at an axial load ratio of  $0.2 P/P_y$ . However, frame strength began to deteriorate rapidly upon an axial load increase to  $0.30 P/P_y$ . Slenderness of the flanges and web had significant influence on the hysteretic behavior of the frames. Slender flanges and web led to more rapid deterioration in hysteretic behavior than for stockier sections, particularly in the presence of axial load. It is noteworthy that these results were based on 1/2 scale tests. Smaller members may offer better performance than the full-scale counterpart. Additionally, welded connections in the model frames were of much higher quality than obtained in most standard construction.

The dead load to yield axial load ratios of the columns of the continuous bridges studied here varies between 0.13 and 0.31. Accordingly, one could expect that some of these columns are able to develop full hysteresis loops based on by Schneider and Roeder (1992)'s findings for steel building columns with  $P/P_y$  ratio of 0.2. However, bridges generally have much longer columns than those commonly used in buildings; the slenderness ratios of bridge columns are also much higher. For example, the continuous bridges studied have columns with weak axis slenderness ratios between 78 and 120. Furthermore, most of the columns in old steel bridges were not designed with intent to absorb energy through cyclic inelastic deformations and therefore are often class 2 or 3 sections. Finally, unlike the columns tested by Popov (1975), bridge columns are not braced laterally, and depending on their slenderness, lateral torsional buckling is possible under strong axis bending, as observed by Uchida et. al. (1992). Considering the above

factors and the lack of information about the behavior of steel bridge columns, in this study, they are conservatively assumed to fail as soon as the capacity delimited by statically derived interaction curve is reached. The stability interaction equations proposed by L. Duan and W.-F. Chen (1989) are used for this purpose. Further experimental research is needed to accurately determine the level of actual inelastic cyclic deformations bridge columns can accommodate.

The present AISC-LRFD, AISC-ASD and CISC linear interaction equations are simple to use, but they have some weaknesses. In particular some of these stability interaction equations for a very short member, do not always correctly reduce to the strength equation. However, the equations proposed by Duan and Chen, (1989) have a smooth transition between stability and strength interaction equations since they are continuous and therefore more convenient to use. Duan and Chen have done an extensive evaluation of the proposed interaction equations as well as the AISC equations and compared with exact inelastic solutions of I-section beam-columns and with the results of 81 full-scale tests of biaxially loaded beam-columns of I-section. It was found that the load carrying capacity of steel beam-columns could be estimated simply and more accurately by the proposed interaction equations. Consequently, due to their simplicity and accuracy, the interaction equations proposed by Duan and Chen, (1989) are used in this study to define the failure of the columns.

The deck is again assumed to behave elastically during earthquakes because of its high in-plane strength reserve. If there is no damage to the abutments, bearings and foundations, the seismic capacity of continuous bridges is governed by the capacity of the columns. However, if the bearings are also damaged, then the capacity may either be governed by sliding displacement of the bridge deck or by the capacity of the columns. Foundations and abutments damage is beyond the scope of this study.

In the longitudinal direction, when the anchor bolts of fixed bearings are severed, the deck is free to slide. In the case of sliding-bearings, as seen in Figure 7.1, frictional

forces are produced between the bottom plate and the concrete abutment at the fixed bearing, as well as between the top plate and the bearing bar at the expansion bearing. Collision of the deck with the abutment walls may occur repeatedly when the friction resistance of the bearings is exceeded. Because there are abutment walls at both ends of the bridge, the movement of the deck in the longitudinal direction is restricted by the width of the expansion joint. Therefore, the bridge deck can not fall off its support in the longitudinal direction unless one or more of the followings happen :

- i) The abutments are severely damaged or displaced excessively;
- ii) The column's foundations settle excessively, particularly when soil liquefaction occurs;
- iii) Columns are excessively damaged;
- iv) abutment's edge fails locally when the bearings slide to a point near the edge where insufficient bearing resistance can be provided to resist the normal reaction forces due to gravity loading, either due to a reduction of bearing surface increasing the effective stresses to exceed the material's capacities, or by the absence of adequate detailing to account for this shift in load application point;
- v) the distance between the bearing centerline and the support edge is less than the expansion joint width.

As before, other local failure modes are not considered above.

In the transverse direction, when the anchor bolts are severed, friction forces between the bottom plate and the concrete abutment floor are produced upon movement as seen in Figure 7.2. When these forces are exceeded, the bridge deck slides, and it may fall off its support if there is not adequate space between the exterior bearing centerline

and the support edge or if the abutment's edge fail locally for the same reasons stated above. Due to excessive sliding displacement, the columns may also be damaged before the bridge deck falls off the support.

## **8.2 Description of the Analytical Models**

Two types of dynamic analyses are performed. The first type is a linear response-spectrum analysis. The objective in performing this analysis is to find the bearings' transverse and longitudinal forces, column moment and shear forces and the maximum elastic displacement or rotation of the deck at the midspan and at the bearings. The program SAP90 is again used for this purpose. The other type of analysis performed is a nonlinear time-history analysis. The objective in performing this analysis is to find the maximum transverse sliding displacement at the bearings, and displacement at the midspan, as well as the maximum peak ground acceleration that can be reached prior to columns' failure due to instability, as a function of various friction coefficients, span length and number of lanes. The program NEABS is used for this purpose.

### **8.2.1 Linear Elastic Modelling of Bridges**

The linear elastic model of the continuous bridges is illustrated in Figure 8.1. The deck is divided into 10 segments, and each segment is modelled as a 3-dimensional beam element. The mass of the bridge is lumped at the nodes linking the beam elements. The stiffness of the deck relative to the stiffness of the columns is very high. Therefore, it is appropriate to assume the end of the columns connected to the deck as infinitely rigidly fixed. The in-plane rigidity of the bridge deck is also very high. Accordingly, a rigid bar is used to model; (i) the interaction between the axial deformation of the columns and the torsional rotation of the bridge deck, (ii) the interaction between the midspan rotation of the bridge deck and torsional rotation of the columns-set due to unsymmetrical support

conditions and/or the small contribution of the second and higher modes of vibration. This rigid bar is connected to the beam element at the midspan and oriented in the transverse direction. The bridge columns are then connected to this rigid bar. In Figure 8.2, the second mode shape is used to illustrate the function of this rigid bar.

The supports are modelled by summing the stiffnesses of each of the bearings and lumping their overall effect at the centerline of the bridge deck by using equivalent rotational and translational springs as described in the previous chapter.

A procedure similar to that described above is followed for the linear elastic modelling of four continuous bridges with 3,4,5 and 6 spans designed to investigate the effect of number of spans on the seismic response. Each span is divided into 5 segments, and each segment is modelled as a 3-dimensional beam element. The mass of the bridge is divided and lumped at the nodes linking the beam elements. A rigid bar at each columns-set location is used to model the interaction between the deformation of columns and bridge deck.

## 8.2.2 Non-linear Inelastic Modelling of Bridges

For the inelastic case, the sliding of the bridge deck in the transverse direction, after the bearings are damaged, is investigated. A non-linear simplified equivalent model similar to that of single span simply supported bridges is used for continuous bridges. From the elastic analyses, the fundamental transverse mode is found to be dominant. Thus, as columns do not have a significant effect on the dynamic behavior of the continuous slab-on-girder steel bridges due to their very low stiffnesses relative to the in-plane stiffness of the deck, the same shape function expressed by Eq. 7.9 is used. The deck is considered separately and the generalised mass  $m^*$ , stiffness  $k^*$  and the effective force  $P_{eff}$  for the equivalent system are calculated using Eqs. 7.10, 7.11 and 7.12. The total end-to-end length,  $L_T$ , of the bridge is used in these equations. The stiffness of each



column in the transverse direction is calculated assuming free rotation at the base and fixed condition at the connection to the deck. Then, the stiffnesses of all the columns are summed up to obtain an equivalent single column stiffness. Using these, the equivalent model of the structure shown in Figure 8.3 is constructed. This model basically consists of three nodes placed sequentially on a straight line. The bridge deck is modelled as a beam element with negligible flexural stiffness but with axial stiffness equal to  $k^*$ . Because the axial forces in columns due to seismic excitation are negligible relative to those due to gravity loading, the equivalent single column is also modelled as a beam element with negligible flexural stiffness but with axial stiffness equal to the originally calculated flexural stiffness. An expansion joint element with a sliding elastic subelement is used for the model. The expansion joint and beam elements are connected in series between nodes 1 and 2 and nodes 2 and 3 respectively. The equivalent single column is connected between nodes 1 and 3 parallel to the deck element so that when the Coulomb friction forces are exceeded, it is compliant to sliding of the bridge deck. Node 1 is fixed in all directions while nodes 2 and 3 are set free to displace in the longitudinal direction. Then, the generalized mass  $m^*$  is lumped on node 3. The modified vertical reaction force,  $R^*$ , is calculated using Eq. 7.18 and placed on node 2. Then, the ground motion is multiplied by  $AMF$  as defined in the previous chapter, and used as acceleration time history input for the equivalent system. Only mass proportional damping is considered in the analyses. A five percent damping ratio is assumed. The first mode circular frequency is used to obtain the mass proportional Rayleigh damping coefficient.

Additionally, a three span continuous bridge is used to investigate the effect of number of spans on the inelastic seismic behavior in the transverse direction. The same procedure as above is followed to model the deck of the three span continuous bridge. However, the columns are modelled slightly differently. The actual system and the model are illustrated in Figure 8.4. As seen in the figure, the columns of the three span continuous bridge are located at  $1/3$  and  $2/3$  of the total end-to-end length, therefore, their displacement is not equal to the midspan displacement. The ratio,  $\Delta_m$ , of the displacement of the deck at the column locations to the midspan displacement is

calculated by substituting  $L/3$  in place of  $x$  in the shape function defined in Eq 7.9. Then, an additional node (node 4 in the model shown in Figure 8.4) is located on the equivalent beam connected between nodes 2 and 3. Knowing that the axial deformation of the beam element in the model increases linearly along its length, the ordinate of this node is set at  $\Delta_m$  of its total length. Then, the weak axis moment of inertias of all the columns are summed up to obtain an equivalent single column inertia. The rotation at the additional node is fixed and the equivalent single column is connected to this node perpendicular to the equivalent beam representing the bridge deck. Only the translational degrees of freedom are fixed at the far end of the column since hinge connection is assumed at the base. This model is used to obtain the sliding displacement of the three span continuous bridge in the transverse direction.

### **8.3 Analyses and Presentation of Results**

#### **8.3.1 Elastic Response Spectrum Analysis**

As in the case of single span simply supported bridges, full composite action between the deck and the steel beams is assumed for the response in the longitudinal direction. In the transverse direction, full composite action is also considered assuming that sufficient number of diaphragm beams exist along the span. The bridges are assumed to have sliding-bearings as commonly found in short to medium span old bridges. An average longitudinal stiffness of 800,000 kN/m for the bearings is assumed for the range of spans considered in the analyses. Response spectrum analyses of 2-lane and 3-lane bridges considered in this study are conducted in the transverse and longitudinal direction. As observed in Figure 8.5, the transverse direction fundamental periods of 2-lane and 3-lane bridges are found to vary between 0.14 and 0.93 second and 0.10 and 0.63 second respectively. The longitudinal direction periods of all the bridges are also calculated and they are found to be less than 0.2 second.

### 8.3.1.1 Bearing Forces and Expansion Joint Displacements

The TBFC is plotted as a function of span length for Eastern and Western Canada design spectra in Figure 8.6. It is observed that the TBFC increases as the span length increases. This is mostly due to the increasing in-plane support moment with length. However, the rate of increase in TBFC reduces as the span length increases since the fundamental period eventually fall in the descending part of the spectra resulting in smaller seismic forces. Although the TBFC in 2-lane bridges seem much larger than those in 3-lane bridges, actually the difference between the bearing forces of 2-lane and 3-lane bridges is about 30 percent for spans up to 30 metres and it is about 10 percent for longer spans. As an overall trend, transverse bearing forces are proportional to the mass of the bridge, hence to the span, and are larger in 2-lane bridges.

In Figure 8.7, LBFC is plotted for two and three lane bridges as a function of span length. Since the periods of the bridges in the longitudinal direction are all less than 0.25 second, the LBFC obtained using Eastern or Western Canada design spectrum are identical. The total longitudinal force is shared equally by each bearing since they have the same stiffness. Although the LBFC for 2-lane bridges is 50 percent larger than those of 3-lane bridges, infact, the individual bearing forces are identical. The constant LBFC in Figure 8.7 should not be misinterpreted; actually, the bearing forces increases with span length since the total mass of the bridge also increases as the span gets longer. Similar to simply supported bridges; (i) as the bridge length increases, longitudinal bearing forces also increase and (ii) the number of lanes does not have any effect on the longitudinal bearing forces for the bridges considered in this study.

Finally, The displacements at the corners of the bridge decks due to seismic excitation in the transverse direction are calculated using Western Canada design spectrum scaled to 0.5g peak ground acceleration. It is observed that these displacements are less than the width of the expansion joint presented in Table 6.1 for all the bridges studied. Therefore, the collision of the deck with the abutment wall is not probable. The

displacement of the bridge deck at the support due to seismic excitation in the longitudinal direction is very small due to the high axial stiffness of the bridge deck and stiffness of the bearings. Unless the failure of the fixed bearings takes place, the collision of the deck with the abutment wall is also unlikely.

### 8.3.1.2 Response of Columns in the Transverse Direction

Second order analysis is conducted to find the magnified seismic moment in the columns due to loading in the transverse direction. The first order displacement at the tip of the column is calculated. Then, the total axial load is multiplied by this displacement to find the second order moment. This moment is divided by the column height and an additional shear force acting on each column is found. These shear forces are summed up and applied to the deck at the centre span. However, since the sway of the bridge is prevented mostly by the bearings which resist more than 98 percent of the seismic force, and since the bridge deck is a very stiff element, the extra lateral force produced by the second order moments in the columns does not cause a significant additional lateral displacement. Thus, one iteration is found adequate to calculate the second order moment. Accordingly, the column transverse seismic moment,  $M_{Ey}$ , for a specified peak ground acceleration,  $A_p$ , is expressed as;

$$M_{Ey} = A_p ( m_{Ey} + P_D \Delta_c ) + A_p^2 P_E \Delta_c \quad (8.1)$$

where  $m_{Ey}$ ,  $P_E$  and  $\Delta_c$  are respectively the first order transverse seismic moment, seismic axial force and displacement at the columns' location for a unit peak ground acceleration and  $P_D$  is the axial force in the column due to gravity loading. Considering column's instability, the maximum weak axis moment,  $M_{ny}$ , prior to column failure is expressed by

the following equations (Duan and Chen, 1989):

$$M_{xy} = M_{py} \left[ 1 - \left( \frac{P}{P_n} \right)^\xi \right] \quad (8.2)$$

$$\xi = 3.0 + 0.035\lambda_m \geq 1.0 \quad (8.3)$$

$$\lambda_m = \frac{Kh_c}{r_x} \left( \frac{M_1}{M_2} - 1 \right) \quad (8.4)$$

where  $M_{py}$  is the plastic weak axis moment for class 1 or 2 sections or yield moment for class 3 sections,  $P$  is the axial load applied on the column and is equal to the sum of the axial loads due to seismic and gravity loading,  $P_n$  is the allowable axial force considering column instability in the absence of moment,  $K$  is the effective length factor in the plane of bending which is equal to 1.0 in this case,  $h_c$  is the height of the column,  $r_x$  is the radius of gyration about the strong axis and  $M_1$  and  $M_2$  are the smaller and larger of the moments at the column ends.  $M_1/M_2$  is positive when the member is bent in reverse curvature and negative when bent in single curvature. The calculated seismic moment should not exceed the maximum moment defined above to prevent failure of the column due to instability. Therefore, equating Eq. 8.1 to Eq. 8.2, the following expression is obtained as a function of peak ground acceleration that can be reached prior to failure, (maximum resistible peak ground acceleration);

$$P_E \Delta_c A_p^2 + (m_{Ey} + P_D \Delta_c) A_p - M_{py} \left[ 1 - \left( \frac{P_D + P_E A_p}{P_n} \right)^\xi \right] = 0 \quad (8.5)$$

Using the above equation and the response spectrum analyses results obtained for various spans and number of lanes, the maximum resistible peak ground acceleration prior to column failure is plotted as a function of span length in Figure 8.8. As seen in that figure, this peak ground acceleration is larger for 3-lane continuous bridges than for 2-

lane continuous bridges. Also, 2-lane bridges up to 40 metres and 3-lane bridges up to 50 metres can survive rather strong motion earthquakes without any damage to columns provided that bearings are not ruptured at the abutments.

The practical range of variation of girder spacing in slab-on-girder bridges is relatively small and consequently, for a given span length, the gravity load resisted by each column is not an important issue for the types of bridges considered here. Similarly, column sizes are not affected much by the number of lanes in bridges designed considering only gravity loading. For example, the 2 and 3-lane bridges considered in this study have the same girder spacing and therefore the same column sizes. However, as the number of lanes increases, the bridge deck gets wider and stiffer in the transverse direction: the attracted seismic forces may be larger in bridges with larger number of lanes, but the displacements are actually much smaller since the stiffness of the deck is proportional to the cube of the transverse dimension. Accordingly, the calculated first and second order forces in columns are much smaller in wider bridges, and since column sizes do not vary considerably with the bridge width, wider bridges can resist larger seismic forces as seen in Figure 8.8. On the contrary, increasing span length has a negative impact on the seismic capacity. Longer bridges may attract smaller seismic forces due to their long periods which fall in the descending part of the response spectra. However, the lateral displacement of the deck, which is proportional to fourth power of the span length, is much larger. Therefore, the calculated first and second order forces in columns are larger in longer bridges, which, as a result, can only resist smaller seismic forces, as seen in Figure 8.8.

#### **8.3.1.3 Effect of Column Size on the Seismic Capacity**

The effect of flexural and axial capacity of columns on the seismic performance of bridges is studied. The response spectrum analysis of a 30 metres span, 2-lane continuous bridge is conducted in the transverse direction for various column sizes using

Western Canada design spectrum scaled to 0.4g peak ground acceleration. Column sizes are varied between W310x79 and WWF450x228 even though axial gravity loading is unchanged. It is noteworthy that column sizes were chosen such that, flexural capacity of all the selected columns in both principal directions increases. However, the axial capacity of the columns up to size W310x202, increases, then it becomes relatively smaller for the other larger column sizes considered.

As seen in Figure 8.9, bridges with columns having larger flexural and axial capacity are able to sustain larger seismic forces. For example, when the size of the bridge column is increased from W310x79 to W310x202, the seismic capacity is nearly doubled, however, the increase is not as much for other larger sections considered due to their relatively smaller axial capacity.

Reduction of the  $P/P_n$  ratio in Eq. 8.2 is largely accountable for this increase. Indeed, analyses show that column size does not significantly decrease the deflection of the bridge deck, although it enhances the seismic capacity. In fact, here, the calculated column seismic moments are found to be directly proportional to the column stiffnesses. Hence, although column stiffness (and indirectly strength) increases with size, so does the applied seismic moment. Therefore, the increase in the seismic capacity observed above is primarily due to the increase in the column cross-sectional area, and axial load capacity which reduces the negative effect of applied axial load on the moment capacity. For example, as seen in Figure 8.9, W360x179 has larger flexural capacity than W310x202, but a smaller seismic capacity due to its smaller cross-sectional area.

The effect of column size on the percentage of the total seismic shear force attracted by columns is illustrated in Figure 8.10. As seen in the figure, this percentage increases along with column size as expected, but all percentages remain small as most of the shear is taken by the bearings.

#### **8.3.1.4 Effect of Steel Strength on the Seismic Capacity**

All the bridges considered in this study were designed using 350 MPa steel. The effect of steel strength on the seismic performance of continuous steel bridges is investigated in this section. The 2-lane 60 metres span continuous bridge is redesigned using yield strengths of 250, 300 and 400 MPa. When lower strength steel is used, the weight of the deck increases due to the larger size of steel girders. This increase in weight is 10 and 3.5 percent for the 250 MPa and 300 MPa steels respectively. Whereas, the weight decreases by 2.2 percent when 400 MPa steel is used. Such increase in dead load and decrease in steel strength result in larger column sizes; on the contrary, when higher strength steel is used, the dead load is reduced and smaller column sizes are obtained. The resulting column sizes are W310x143, W310x129 and W310x107 for 250, 300 and 400 MPa steels respectively. The column size used in the original design is W310x118.

Response spectrum analyses of these modified structures are conducted using Western Canada design spectrum. The results presented in Figure 8.11 show the maximum resistible peak ground acceleration as a function of steel strength considering column instability. It is observed that, as the steel strength decreases, the seismic capacity of the column decreases. This reduction in seismic capacity is 7 and 24 percent for 300 and 250 MPa steels respectively. The seismic capacity is increased by 8 percent when 400 MPa steel is used.

#### **8.3.1.5 Effect of Number of Spans on the Seismic Capacity**

Two lane continuous bridges of 3, 4, 5 and 6 spans are designed. Each span is assumed to be 20 metres in length. As the number of spans increases, the variation of the gravity loading stresses in the girders and columns becomes negligible. Therefore, the girder and column sizes of all the designed continuous bridges with number of spans



larger than two, are identical. The girder and column sizes for these bridges are respectively WWF900x169 and W310x67, whereas they were WWF900x192 and W310x74 for the two span continuous bridge. Response spectrum analyses of all the bridges are conducted using Western Canada design spectrum. The results are illustrated in Figures 8.12 and 8.13.

In Figure 8.12 the maximum resistible peak ground acceleration is plotted for various number of spans. It is observed that, as the number of spans increases, the seismic capacity rapidly decreases. This is mainly due to the increased mass and flexibility of the structure which subsequently results in higher displacements at the column locations for a given peak ground acceleration.

In Figure 8.13, the ratio of the maximum resistible peak ground acceleration of multiple span bridges to that of two span bridges of equal total length, is illustrated. Generally, multiple span bridges can accommodate peak ground accelerations larger than those obtained for two span bridges of identical end-to-end length. It is noticed that, bridges with even number of spans are more vulnerable to seismic excitations than those with odd number of spans. In the case of bridges with even number of spans, there is always a column located at the midspan where the displacement is high. However, in the case of odd number of spans, the columns are located away from the midspan, hence they have less displacement. This effect vanishes as the span length and number of span increases.

#### **8.3.1.6 Longitudinal Direction Response**

Assuming that the bearings are damaged but abutments and column footings conserve their integrity, the bridge deck may displace in the longitudinal direction until it collides with the abutment walls. The structure may survive the earthquake if its columns are able to displace by as much as the width of the expansion joints (values

listed in Table 6.1) without any damage. The columns may safely overcome that much displacement if the applied moments are less than the maximum strong axis moments that initiate instability. These strong axis moments, for all the columns, are obtained using the following stability interaction equations proposed by Duan and Chen (1989):

$$M_{ax} = M_{rx} \left[ 1 - \left( \frac{P}{P_n} \right)^\eta \right] \quad (8.6)$$

$$\eta = 1.3 + 0.002\lambda_x \quad (8.7)$$

$$\lambda_x = \frac{Kh_c}{r_x} \quad (8.8)$$

where,  $M_{rx}$  is the ultimate strong axis moment capacity in the absence of axial load, and is defined as (Kulak et.al., 1988);

$$M_{rx} = \begin{cases} M_u & \text{if } M_u \leq \frac{2}{3} M_{px} \\ 1.15 M_{px} \left( 1 - \frac{0.28 M_{px}}{M_u} \right) \leq M_{px} & \text{if } M_u > \frac{2}{3} M_{px} \end{cases} \quad (8.9)$$

$$M_u = \frac{\pi}{\omega h_c} \sqrt{E I_{cy} G J_c + \left( \frac{\pi E}{h_c} I_{cy} \frac{(d-t_f)}{2} \right)^2} \quad (8.10)$$

$$\omega = 0.6 + 0.4 \frac{M_1}{M_2} \quad (8.11)$$

In the above equations,  $M_u$  is moment resistance of a member subject to lateral torsional buckling,  $M_{px}$  is the plastic strong axis moment for class 1 or 2 sections or yield moment for class 3 sections,  $I_{cy}$  and  $J_c$  are respectively the weak axis and torsional inertia of the column,  $d$  is the depth of the column section,  $t_f$  is the thickness of the column flange and

$G$  is the shear modulus of elasticity of steel. Assuming that the bearings are damaged, and equating the expansion joint width to the maximum displacement at deck level, the first order column moment due to this displacement is calculated for all the bridges considered in this study. Second order analyses are also performed by multiplying these imposed displacements by the axial forces due to gravity loading. Only one iteration is carried out, knowing that the bridge deck cannot displace more than the expansion joint width. First order and second order moments are summed up and the ensuing final moments are compared with the ultimate strong axis moment capacities obtained considering column instability. It is found that only the columns in 2 and 3-lane continuous bridges of spans longer than 50 metres, cannot accommodate a displacement equal to the expansion joint width.

In the design performed for this study, the expansion joint width is calculated considering the difference between the maximum and minimum annual temperatures, and may be different in various climates. Therefore, the following procedure is applied to calculate the maximum permissible longitudinal displacement as a function of span length regardless of the expansion joint width.

The columns used in the continuous bridges considered in this study are assumed to be hinged at the base and fixed at the top to the rigid decks of the bridges. Therefore, in sway mode, each column can be modelled as a reversed cantilever with a longitudinal direction stiffness  $k_{cL}$  and a length  $h_c$ . This length is defined as the distance to the point of inflection, and in this case it is equal to the full height of the column. The magnified moment,  $\beta_{mx}M_{x0}$ , considering the second order effects is expressed as;

$$\beta_{mx}M_{x0} = M_{x0} + M_{x1} + M_{x2} + M_{x3} + \dots + M_{xn} \quad (8.12)$$

where  $M_{x0}$  is the first order moment, and  $M_{x1}$ ,  $M_{x2}$ ,  $M_{x3}$ ,  $M_{xn}$  are the second order moments obtained respectively from the 1<sup>st</sup>, 2<sup>nd</sup>, 3<sup>rd</sup> and n<sup>th</sup> iteration. In the first iteration, the calculated first order elastic displacement,  $\Delta_{x0}$ , is multiplied by the axial force and the

second order moment is obtained. In the subsequent iterations, the second order moment obtained in the previous iteration is divided by the column length and an incremental lateral force,  $VI_x$ , is found. This incremental force is then divided by the column stiffness and a new incremental displacement,  $\Delta I_x$ , is determined. Then, this displacement is multiplied by the axial force to obtain the new incremental moment. The above procedure is illustrated for the first three steps as follows;

**Step 1 :**

The first order moment is expressed as;

$$M_{x0} = k_{cL} h_c \Delta_{x0} \quad (8.13)$$

Then, the second order moment, incremental shear and displacement are calculated as;

$$M_{x1} = P \Delta_{x0} \quad (8.14)$$

$$VI_{x1} = \frac{M_{x1}}{h_c} = \frac{P \Delta_{x0}}{h_c} \quad (8.15)$$

$$\Delta I_{x1} = \frac{VI_{x1}}{k_{cL}} = \frac{P \Delta_{x0}}{k_{cL} h_c} \quad (8.16)$$

**Step 2 :**

$$M_{x2} = P \Delta I_{x1} = \frac{P^2 \Delta_{x0}}{k_{cL} h_c} \quad (8.17)$$

$$VI_{x2} = \frac{M_{x2}}{h_c} = \frac{P^2 \Delta_{x0}}{k_{cL} h_c^2} \quad (8.18)$$

$$\Delta I_{x2} = \frac{VI_{x2}}{k_{cL}} = \frac{P^2 \Delta_{x0}}{k_{cL}^2 h_c^2} \quad (8.19)$$

Step 3 :

$$M_{x3} = P \Delta I_{x2} = \frac{P^3 \Delta_{x0}}{(k_{cL} h_c)^2} \quad (8.20)$$

$$VI_{x3} = \frac{M_{x3}}{h_c} = \frac{P^3 \Delta_{x0}}{k_{cL}^2 h_c^3} \quad (8.21)$$

$$\Delta I_{x3} = \frac{VI_{x3}}{k_{cL}} = \frac{P^3 \Delta_{x0}}{k_{cL}^3 h_c^3} \quad (8.22)$$

Substituting the moments obtained in the above steps in Eq. 8.12, the following equation is obtained;

$$\beta_{mx} M_{x0} = h_c k_{cL} \Delta_{x0} + P \Delta_{x0} + \frac{P^2}{(k_{cL} h_c)} \Delta_{x0} + \frac{P^3}{(k_{cL} h_c)^2} \Delta_{x0} + \dots + \frac{P^n}{(k_{cL} h_c)^{n-1}} \Delta_{x0} \quad (8.23)$$

The above equation is expressed in another form as;

$$\beta_{mx} M_{x0} = h_c k_{cL} \Delta_{x0} \left( 1 + \frac{P}{k_{cL} h_c} + \frac{P^2}{(k_{cL} h_c)^2} + \frac{P^3}{(k_{cL} h_c)^3} + \dots + \frac{P^n}{(k_{cL} h_c)^n} \right) \quad (8.24)$$

Then, the above equation is represented in series form as;

$$\beta_{mx} M_{x0} = \sum_{i=0}^n \left( \frac{P}{k_{cL} h_c} \right)^i k_{cL} h_c \Delta_{x0} \quad (8.25)$$

As the number of iterations increases, more accuracy is obtained. Therefore the limit of the above equation as  $n$  approaches infinity is the most accurate representation of the magnified moment.

$$\beta_{mx} M_{x0} = \lim_{n \rightarrow \infty} \sum_{i=0}^n \left( \frac{P}{k_{cL} h_c} \right)^i h_c k_{cL} \Delta_{x0} = \frac{h_c k_{cL} \Delta_{x0}}{1 - \frac{P}{k_{cL} h_c}} \quad (8.26)$$

The above equation is true if  $P/k_{cL} h_c$  is less than 1.0 and can be used safely to obtain the magnified elastic sway moment of a column. Knowing that the magnified elastic moment should not exceed the maximum strong axis moment,  $M_{ux}$ , that initiate instability, and using the above equation, the maximum applicable first order elastic displacement is expressed as;

$$\Delta_{x0} = \left( \frac{1}{k_{cL} h_c} - \frac{P}{(k_{cL} h_c)^2} \right) M_{ux} \quad (8.27)$$

The strong axis moments, stiffnesses and heights of the continuous steel bridge columns considered in this study are substituted in the above equation to obtain the longitudinal displacement as a function of span length. The result is illustrated in Figure 8.14. The width of the expansion joints of the bridges considered in this study are also plotted in the same figure. As seen in the figure, shorter span bridges can accommodate larger displacements due to lower axial loads on the columns which subsequently results in smaller second order moments. Furthermore, shorter span bridges have smaller expansion joint width which adds an extra safety. Bridges of spans longer than delimited by the intersection of the lines representing the expansion joint width and the calculated maximum permissible first order displacement, may be a problem if the bearings are damaged. However, the friction forces between the steel plate and the concrete abutment

floor at the fixed abutment may dissipate energy, resulting in less displacement than expected. This will be investigated in Section 8.3.2.2

### 8.3.2 Nonlinear Time History Analysis

#### 8.3.2.1 Transverse Direction Response

The sliding of continuous bridges in the transverse direction, after the bearings are damaged, may take place depending on the intensity of the earthquake. For sliding to occur, the total transverse seismic load,  $H$ , expressed in Eq. 7.17 must be at least equal to the sliding resistance force,  $F_s$ . By analogy with the distribution of reaction forces in a two span continuous beam, 62.5 percent of the bridge weight is supported by the columns at the midspan and the remaining 37.5 percent is transferred to the bearings. The sliding resistance force is obtained by multiplying the coefficient of friction at the abutments by the weight transferred to the bearings:

$$F_s = \frac{3}{8} \mu_f m g \quad (8.28)$$

Expressing the absolute spectral acceleration as a fraction of the peak ground acceleration, and using Eqs. 7.17 and 8.28, the minimum value of peak ground acceleration required for sliding is represented by the following equation:

$$A_p = \frac{3\pi^2}{64\beta} \mu_f g \quad (8.29)$$

For example, using the MPISD spectrum of Western United States earthquakes and a friction coefficient of 0.4 between the steel plates and the concrete seat, the minimum required peak ground acceleration for sliding is 0.074g for a 2-lane continuous bridge with two spans of 30 metres each ( $T_1 = 0.29$  sec,  $\beta = 2.5$ ), while for a 2-lane continuous bridge with two spans of 60 metres each ( $T_1 = 0.93$  sec,  $\beta = 1.34$ ) it is 0.140g.

Generally, the peak ground acceleration required for sliding, increases with span length for the continuous bridges considered.

Inelastic dynamic analyses of the continuous bridges considered in this study are conducted for 4 Western United States earthquakes to investigate the effect of damage to bearings on the seismic capacity of the structure. In the previous chapter, Eastern Canada earthquakes were not found very effective in causing sliding and therefore are not considered in the subsequent analyses. When the bearings are damaged, the column moment in continuous bridges is a function of the sum of the sliding displacement at the supports and the elastic displacement of the deck at the column location. The sliding displacement varies with the earthquake's peak acceleration, friction coefficient at the bearings, and span length. Unfortunately, the relationship between these parameters is highly nonlinear. Therefore, more than 400 analyses were performed using the program NEABS to establish the relationship between the maximum resistible peak ground acceleration and span length for various friction coefficients. This relationship is determined using the average of results obtained from 4 Western United States earthquakes and considering the stability of columns in the transverse direction. The results are presented in Figure 8.15. As seen in that Figure, the maximum resistible peak ground accelerations are larger for 3-lane bridges than those for two lane bridges.

Generally, as the friction coefficient increases, so does the maximum resistible peak ground acceleration. Therefore, bridges with higher friction coefficient at the bearings, can sustain bigger earthquakes. Assuming a friction coefficient of 0.4 for old bridges (sliding-bearings), 3-lane continuous bridges of spans shorter than 40 metres and 2-lane continuous bridges of spans shorter than 30 metres may survive high intensity earthquakes even if their bearings are damaged provided that during the earthquake other structural elements preserve their integrity.

Neglecting column instability, sliding displacements at the support of 2-lane and 3-lane simply supported bridges are plotted as a function of span length for various



friction coefficients. The results for S00E component of 1940 El Centro earthquake scaled to 0.4g are presented in Figure 8.16. It is observed that, as a general trend :

(i) for increasing span length and decreasing friction coefficient, the sliding displacement increases, and; (ii) the sliding displacements of 3-lane bridges are smaller than those of 2-lane bridges. It is noteworthy that the sliding displacements illustrated in Figure 8.16, for spans longer than 30 metres can actually not be attained since the bridge columns cannot accommodate an earthquake of 0.4g as seen in Figure 8.15.

It is also observed that the sliding displacements of continuous bridges with spans shorter than 30 metres are nearly twice those of simply supported bridges of equal total end-to-end length. Therefore, the sliding displacements of continuous bridges can be conservatively obtained by multiplying the displacements in Figure 7.20 by 2.

Finally, although columns in continuous bridges have some elastic restoring energy when they are displaced, the same normalization principles derived in the previous chapter could be used to express results in a non-dimensional format, since relatively small stiffness of the columns results in negligible elastic restoring energy after sliding. However, this is not necessary, as results for continuous bridges can safely be extrapolated from those for simple span bridges as indicated above.

#### **8.3.2.2 Effect of Column Stiffness on the Sliding Displacement**

The effect of column stiffness on the transverse sliding displacement of the bridges is studied. Columns of different stiffnesses are considered for the same 2-lane, 60 metres span simply supported bridge studied in the previous chapter. Here, the distribution of the gravity loading to the bearings is assumed not to be affected by the columns, i.e., the same reaction forces are used in the analysis without any modification. The S00E component of 1940 El Centro earthquake scaled to 0.4g is used in the analysis. The

results are presented in Figure 8.17 in terms of sliding displacement as a function of increased column weak axis inertia. As seen in the figure, the displacement of the simply supported bridge is reduced from 54 mm to 42 mm when this weak axis inertia is increased by  $40 \times 10^6 \text{ mm}^4$ . Therefore, it appears that increases in the column's stiffness only causes a slight decrease in sliding displacement.

Additionally, a 2-lane, 30 metres span continuous bridge is also analyzed for the same earthquake and for two different friction coefficients. Although the simply supported bridge and the continuous bridge have the same total span length (i.e. 60 m.), the later has larger sliding displacement. This is a consequence of the smaller reaction force acting on the bearings of the continuous bridge due to the presence of columns. In the same figure the sliding displacement of the continuous bridge for a friction coefficient of 0.4 is also illustrated. It is observed that the sliding displacement for the friction coefficient of 0.1 is roughly 65 percent more than that for the 0.4 friction coefficient.

### **8.3.2.3 Effect of Column Size on the Seismic Capacity**

Assuming that the bearings are damaged, the effect of column size on the seismic capacity is studied. The nonlinear time history analysis of a 30 metres span, 2-lane continuous bridges is conducted in the transverse direction for various column sizes using S00E component of El Centro earthquake scaled to 0.4g. A sliding friction coefficient of 0.1 is used in the analysis. The ratio of the maximum resistible moment prior to column instability to the seismically-induced moment, is plotted for different column sizes in Figure 8.18. As seen in the figure, when the size of the bridge column is increased from W310x79 to W310x202, this ratio doubles, largely because the ultimate capacity becomes almost 70 percent larger than the original one. It was demonstrated earlier that column size does not have a significant effect on sliding displacement of the bridge deck. Here again, the column seismic moments are directly proportional to the column stiffness and increases in the ultimate capacity are primarily due to the increase in the column

cross-section, hence the axial load capacity, which reduces the negative effect of applied axial load on the moment capacity, i.e. the same phenomenon previously described in Section 8.3.1.3.

#### **8.3.2.4 Effect of Number of Spans on the Transverse Sliding Displacement**

To briefly investigate the effect of number of spans on the seismic response, a continuous bridge with three spans of 20 metres each (60 metres total end-to-end length) is designed. The nonlinear dynamic analysis of this bridge is conducted using S00E component of 1940 El Centro earthquake scaled to 0.4g peak ground acceleration, and friction coefficients of 0.2 and 0.4. The resulting sliding displacements at the support are 82 and 61 mm respectively. These sliding displacements are nearly three times the displacements of a continuous bridge with two spans of 20 metres each (40 metres total end-to-end length). This is expected because the three span continuous bridge is about fifty percent heavier than the two span continuous bridge, hence, it attracts more seismic force and undergoes larger sliding displacements.

Additionally, the sliding displacements of the three span continuous bridge are also compared with those of a continuous bridge with two spans of 30 metres each (60 metres total end-to-end length again). The two bridges have equal total end-to-end lengths, but, still, the sliding displacements of the three span bridge are 2.0 and 1.2 times larger than those of the two span bridge, for friction coefficients of 0.2 and 0.4 respectively. Logically, as the number of spans increases the proportion of the total gravity load transferred to the abutments decreases. Consequently, bridges with a larger number of spans have larger sliding displacements at the abutments compared to bridges of equivalent total end-to-end length but smaller number of spans. These large sliding displacements may reduce the seismic capacity of the structure since the columns may not be capable to accommodate that much displacement.

### 8.3.2.5 Longitudinal Direction Response

In a previous section, assuming the bearings were damaged, the maximum longitudinal displacements before the columns fail due to instability were calculated as a function of span length. Then, these displacements were compared with the width of the expansion joints. Continuous bridges with spans longer than 50 metres were found to have expansion joint widths larger than the calculated displacements and therefore were not considered to be safe. Fortunately, when bridges slide in the longitudinal direction, friction forces at the fixed and expansion bearings are produced as shown in Figure 7.1. The friction between the bearing bar and the top steel plate at the expansion bearing is negligible, but the friction forces between the steel plate and the concrete abutment floor at the fixed bearing may dissipate energy, limiting the magnitude of longitudinal displacements to acceptable values, even for a high intensity earthquake. To illustrate this, sliding displacement of a continuous bridge with two spans of 60 metres each is calculated considering and ignoring friction. The following method is used for this purpose.

The fundamental period and the ratio of total elastic seismic force to total friction resistance are the only parameters which control the sliding displacement of a structure for a given ground acceleration history. Note that the friction resistance is equal to the vertical reaction force times the friction coefficient. Two different structures can have the same sliding displacements if (i) they have identical periods, (ii) their elastic seismic force to vertical reaction force ratios are the same, and (iii) they have the same friction coefficient. Knowing this, by matching parameters, transverse sliding displacements obtained for simply supported bridges in the previous chapter may be used to predict longitudinal sliding displacement of continuous bridges. Figure 7.20 illustrates the sliding displacement of 2-lane simply supported bridges for various  $\mu_f/A_p$  ratios as a function of span length. The transverse fundamental period of the 2-lane 50 metres span simply supported bridge is very close to the longitudinal fundamental period of the continuous bridge with two spans of 60 metres each. Thus, neglecting the effect of columns'

stiffness in the continuous bridge, the longitudinal sliding displacement for a specified  $\mu/A_p$  ratio can be obtained conservatively from Figure 7.20. In order to use this figure both bridges must have the same elastic seismic force to friction resistance ratio. However, the elastic seismic force to vertical reaction force ratio is different in both bridges. Therefore, the friction coefficient used for the continuous bridge should be modified to obtain the same elastic seismic force to friction resistance ratio. The elastic seismic force to vertical reaction force ratio for a unit spectral acceleration (i.e.  $S_a = 1.0$ ) is 0.08263 for the simply supported bridge and 0.51648 for the continuous bridge. A friction coefficient of 0.4 between the steel plate and concrete abutment floor is assumed. This friction coefficient is first multiplied by 0.08263 and then divided by 0.51648 to obtain an equivalent friction coefficient to be used in Figure 7.20. Using the calculated equivalent friction coefficient and an arbitrary peak ground acceleration of 0.26g ( $\mu_y/A_p = 0.25$ ), a maximum sliding displacement of 40 mm is obtained. As previously calculated a longitudinal displacement of 49 mm is actually required to fail the column due to instability. Repeating the above procedure would demonstrate that, considering friction, a peak ground acceleration of approximately 0.30g would lead to this displacement. If the friction was ignored, the columns alone would be grossly inadequate in resisting lateral forces; for example, neglecting instability considerations, a peak ground acceleration of 0.30g would produce a maximum longitudinal displacement of 335 mm. It is noteworthy that to produce identical sliding displacements a 60 metres simple span bridge requires a much higher peak acceleration than a 50 metres span continuous bridge.

#### 8.4 Summary

In this section findings for the seismic behavior of continuous bridges are summarized. The response of bridges to earthquake excitations is investigated considering two possible cases. First, the bearings are assumed to survive the earthquake without any damage and the response of other structural components are investigated. As a second

possible case, the bearings are assumed to be damaged and the impact of this on the seismic response of the structure is studied.

#### **8.4.1 Undamaged Bearings**

Bearing forces due to seismic excitation in both transverse and longitudinal direction are proportional to the mass of the bridge, hence, to the span length. Their magnitude is independent of the number of lanes when subjected to seismic excitation in the longitudinal direction. However, when subjected to seismic excitation in the transverse direction, their magnitude decreases as the number of lanes increases. If bearings are not damaged, 2-lane bridges up to 40 metres and 3-lane bridges up to 50 metres span are found capable to survive earthquakes with 0.4g peak acceleration without any damage to columns.

The effect of various geometric and structural properties of continuous bridges on the seismic response is also investigated. It is found that :

- As the number of spans increases, the seismic capacity decreases. This is mainly due to the increased mass and flexibility of the structure which produce higher displacements at the column locations, resulting in failure of the columns due to instability.
- Multiple span continuous bridges can generally accommodate larger peak ground accelerations than two span continuous bridges of equal end-to-end length.
- Bridges with even number of spans are found to be slightly more vulnerable to seismic excitations than those with odd number of spans. However, this affect vanishes as the span length and number of spans increases.

- Column size does not have a significant effect on decreasing the deflection of the bridge deck, however, larger columns increase the seismic capacity, largely as a consequence of the increased axial area and its favourable effect on the axial-force-flexural-moment interaction diagram.
- The percentage of shear attracted by columns increases as the column size increases, however, these shear forces remain small as most of the seismic shear is resisted by the bearings.
- Bridges designed using lower strength steel are slightly more vulnerable to earthquake excitations. This is a consequence of the increased dead load of the superstructure which subsequently results in higher axial load on the columns and larger columns which attract more moment.

#### **8.4.2 Damaged Bearings**

As a second case the bearings are assumed to be damaged and the effect on the seismic response is investigated. The findings are presented below.

The magnitude of the friction coefficient at the bearings is found to have a considerable impact on the seismic capacity of the structure. As this friction coefficient increases, so does the peak ground acceleration required to initiate sliding. Therefore, bridges with higher friction coefficients can resist more severe earthquakes before sliding. Magnitude of the friction coefficient also affects the amount of sliding. Generally, as it increases, the maximum sliding displacement of the bridge decreases. This subsequently produces smaller displacement at the column locations, and even more severe earthquakes are needed to fail the columns. For bridges with sliding-bearings, it is found that 2-lane bridges of spans shorter than 30 metres and 3-lane bridges of spans shorter than 40 metres

may survive earthquakes of 0.35g peak ground acceleration even if the bearings are damaged, provided that they have adequate seat width in the transverse direction.

Additionally the effect of some geometric and structural parameters on the seismic response is also investigated. The following is observed :

- Generally, for increasing span length and decreasing friction coefficient, the sliding displacement increases.
- Sliding displacements of 3 lane bridges are smaller than those of 2 lane bridges.
- The increase in column stiffness causes a slight decrease in sliding displacement.
- The sliding displacement of bridges with larger number of spans is more than that of bridges with identical end-to-end length but smaller number of spans. This is mainly due to the reduced fraction of gravity load on the abutments which results in less friction resistance.

As a general trend, regardless of the conditions of bearings (damaged or undamaged), the peak ground acceleration required to damage the structure is found to be larger for 3-lane than 2-lane continuous bridges.



## **CHAPTER 9**

### **MULTI-SPAN SIMPLY SUPPORTED SLAB-ON-GIRDER STEEL BRIDGES**

The multi-span simply supported bridges considered in this study are assumed to have two spans as in the case of continuous bridges. However, the effect of number of spans on the seismic behavior is investigated separately. The left deck of the two-span simply supported bridge, as seen in Figure 6.3, is attached to the left abutment by a fixed bearing and at the other end it is supported by expansion bearings on the columns placed under each steel girder. These columns are assumed to be connected rigidly to the foundation. The left end of the right deck is supported by fixed bearings on each column and the other end is supported by expansion bearings on the right abutment. It is noteworthy that, in some bridges, restrainer bars may be used at the expansion bearings to prevent the decks from falling off their support in the case of excessive longitudinal movements. For the bridges considered in this study, no restrainer bars are assumed to exist at the expansion bearings, a situation representative of existing bridges in Canada.

The bridges are assumed to have sliding-bearings with two and four bolts when located on the abutments. When located on the columns, the bottom plate is either welded to the top of the column or connected by short bolts to another plate welded to the column. Accordingly, the stiffness of the bearings are calculated considering only the flexural stiffness of the bearing bar. The same bearing dimensions are used as in Chapters 7 and 8. Due to the discontinuity between the decks, the stiffness of the

structure may be affected by the stiffness of the bearings. Therefore, other than sliding-bearings, bearings-sets with zero and infinite rotational stiffness are also considered. Infinite rotational stiffness occurs when the fixed bearings are infinitely rigid in their longitudinal direction. Zero rotational stiffness occurs when the fixed bearings have no longitudinal stiffness. Some types of pinned rocker bearings approach the later limit. Zero rotational stiffness can also occur when bearings are damaged. In all the cases transverse stiffness is assumed to be infinitely rigid.

The deck is assumed to function elastically during a ground motion because of its high in-plane strength reserve. If there is not any damage to the abutments, bearings and foundations, the seismic capacity of these bridges is governed by the capacity of the columns in both directions or the opening of the expansion joints until the deck falls off its support. Foundations and abutments deformation and damage are beyond the scope of this study.

## **9.1 Initial Comments on Behavior**

### **9.1.1 Behavior in the Longitudinal Direction**

In the longitudinal direction, when a multi-span simply supported bridge is subjected to seismic excitation, the friction forces created at the expansion bearings are negligible due to the very low friction coefficient on the contact surfaces, since these bearings are designed intentionally to slide without significant resistance. Therefore, there is not any interaction between the spans until collision occurs between neighbouring decks. Accordingly, these bridges are assumed to be composed of many separate sub-structures in the longitudinal direction. Basically, these sub-structures are of two types. The first type is defined as a deck located at one end of the bridge and supported by fixed bearings on the abutment. In the subsequent sections, it will be referred to as abutment-fixed deck. The other type is what constitutes the remaining parts of the bridge, i.e., a

deck is supported by fixed bearings on the columns at one end and supported by expansion bearings either on the abutment or on other columns at the other end, depending on its location. It will be referred to as column-fixed deck in the subsequent sections. The former type of sub-structure is axially very rigid and its longitudinal displacement is negligible. On the contrary, the latter type is very flexible since the longitudinal movement of the deck is resisted only by the columns. Consequently, the displacements can be large.

When a two-span simply supported bridge is subjected to seismic excitation in the longitudinal direction, collision of the column-fixed deck with the abutment walls and with the abutment-fixed deck may take place repeatedly. Because there is a deck fixed to the abutment on one side and the abutment wall on the other side of the bridge, the movement of the column-fixed deck in the longitudinal direction is restricted by the width of the expansion joint which is roughly a maximum of 4-5 cm for the ranges of spans considered. Therefore, bridge decks cannot fall off their supports unless one or more of the followings happen :

- i) The abutments are severely damaged or displace excessively;
- ii) The columns' foundations settle excessively, particularly when soil liquefaction occurs;
- iii) Columns are excessively damaged
- iv) The abutment edge can not resist the reaction forces due to gravity loading which are brought close to the edge due to sliding
- v) The edge of the plate welded to the top of the column to support the bearings is not stiffened adequately and therefore can not resist the reaction forces due to gravity loading which are brought close to the edge, either due to movement of

the expansion bearing in the case of rocker bearings, or due to sliding of the bearing bar in a sliding-bearing if the bearing bar ruptures from its bottom plate when high longitudinal shear forces are produced by the collision of the column-fixed deck with the abutment or abutment-fixed deck;

- vi) The longitudinal distance between the centerline of the bearing bar in an expansion bearing and tip of the steel girder is less than the expansion joint width (normally it should not be designed like this)
- vii) The distance between the bearing centerline on the roller side and the support edge is less than the expansion joint width if the bearings are not damaged or less than two times the expansion joint width if the bearings are damaged.

Other local failure modes are not considered above.

When bridges with more than two spans are subjected to seismic excitation, the movements of all identical column-fixed decks are in phase until there is collision either between the column-fixed deck and the abutment wall at one end of the bridge, or between the column-fixed and abutment-fixed decks at the other end of the bridge. It is noteworthy that, in the case of long multi-span simply supported bridges, due to different soil conditions under the supports, and the time lag between the arrival of seismic waves to the supports located at a distance, the column-fixed decks may not vibrate in-phase. In fact, the response of such long structures are found to be affected by the phase differences in the input ground motions applied to the bridge foundations, and the nature of this response to the travelling seismic waves is found to be strongly dependent on the direction of incidence as well as on the excitation frequency of the seismic shear waves (Werner et al., 1979).

Consider a three span simply supported bridge subjected to seismic excitation. The bridge has one abutment-fixed and two column-fixed decks of identical length and

therefore identical expansion joint widths. Assuming that there is no damage to the bearings, columns, abutments and foundations, and allowing impact between the decks, the maximum possible longitudinal relative displacement between the decks is two times the expansion joint width. For example, this may happen if the two column-fixed decks move in opposite directions, i.e, if they are 180° out of phase. Now consider a four span simply supported bridge which has one abutment-fixed and three column-fixed decks of identical lengths. If the first column-fixed deck moves toward the abutment-fixed deck as much as the expansion joint width (this is the maximum it can move since it will eventually be stopped by the abutment-fixed-deck), then the second column-fixed deck can move twice the expansion joint width in the same direction. If the third column-fixed-deck moves as much as the expansion joint width in the opposite direction (this is the maximum it can move since it will be stopped by the abutment wall), the maximum possible opening of expansion joint between the second and the third decks is three times the expansion joint width. Different deformation configurations which result in the same maximum opening in the other expansion joints are also possible.

The above examples can be generalized to state that, if the bearings are not damaged, the maximum possible expansion joint opening in a multi-span simply supported bridge with identical expansion joint widths is  $(n_j - 1)$  times the expansion joint width, where  $n_j$  is the number of expansion joints. This is obviously a worst case scenario which requires that column-fixed decks oscillate according to a very particular pattern. The seismic response of multi-span simply supported bridges is a very random event which is a function of the number of spans, the period of each individual span, the width of the expansion joint, the frequency characteristics as well as the intensity of the earthquake excitation and, for bridges with large end-to-end length, the velocity of the seismic waves which itself depends on the type of soil. Considering these factors, the accurate prediction of longitudinal displacements is very difficult. Kawashima and Penzien (1976, 1979) attempted to correlate the seismic response of a multi-span bridge obtained from experimental studies to that obtained analytically. Their results did not show a good correlation in expansion joints openings when the intensity of the seismic

excitation was increased. Analytical case studies on the seismic response of multi-span simply supported bridges, have been performed by some researchers (Tseng and Penzien, 1973; Penzien et. al, 1975; Zimmerman and Brittain, 1981; Imbsen and Penzien, 1986). In a subsequent section, the expansion joint openings obtained from these case studies will be compared to the maximum possible expansion joint openings described above.

In the longitudinal direction, if there is no collision between the decks, then the force acting on a fixed bearing connected to a steel column is equal to the shear force in that column. This force is much smaller than the force acting on a bearing fixed to an abutment, and in most cases, as will be demonstrated in a subsequent section, the column reaches its capacity before the bearings are damaged if collision does not occur. However, the impact forces produced by collision of a deck with another deck or with the abutment may create high axial forces in the deck. These forces are transferred as shear forces to the bearings and may cause damage. This damage can be in many different ways. For example, in a sliding-bearing, If the rectangular stopper bars at the top plate are sheared and separated from the plate, the deck is free to slide on the bearing bar without a significant friction resistance. In that case the deck may slide off the bearing bar, damage the columns and fall off its support. The structure would be damaged in a similar way if the bearing bar is separated from the bottom plate or if the bottom plate is separated from the column.

### **9.1.2 Behavior in the Transverse Direction**

Consider a two-span simply supported bridge. When the bridge displaces in the transverse direction, the abutment-fixed and column-fixed decks rotate. Due to this rotation, the corners of the two decks approach each other and eventually collide provided that (i) the intensity of the earthquake is adequate to cause that much displacement, (ii) the columns are not damaged before the collision takes place or (iii) the decks do not fall off their support due to the excessive opening in the expansion joint. The intensity of this

collision depends on the relative longitudinal direction velocity between the two adjacent decks. However, the collision due to rotation of adjacent decks may not happen before the steel columns are damaged. This will be investigated in the subsequent sections.

Consider the same two span simply supported bridge, in the transverse direction, the force acting on the bearings attached to the columns is equal to the shear force in these columns as long as collision between adjacent decks does not occur. Considering the relative capacities of those bearings and columns, it is concluded that failure of the columns, by far, is more likely. Accordingly, only damage to the bearings located at the abutments will be considered in this study. Obviously once the bearings fail, the rotational stiffness of the fixed-bearings-set becomes negligible. In this case, the decks spanning between the columns and abutments will act almost like a mechanism and the transverse stiffness of the structure will be reduced considerably. Additionally, this mechanism action may transmit less force to the bearings at the abutments, and the resulting bearing force may not be big enough to cause sliding of the structure. Even though the period of the structure elongates in the process with less seismic forces attracted into the structure, the transverse displacement of the columns greatly increases and may trigger structural failure faster. This will be investigated in the subsequent sections.

## **9.2 Description of the Analytical Model**

A linear elastic response-spectrum analysis in the transverse direction is conducted. The objective in performing this analysis is to find the bearings' transverse forces and forces in the columns. The program SAP90 is used for this purpose. A non-linear time history analysis is not needed here since, as will be demonstrated, linear response spectrum analyses results show that in 85% of the cases studied, either columns reach their ultimate capacity prior to impact between the two decks, or the peak accelerations required to produce collision are very high.

Full composite action between the deck and the steel beams is assumed. In the transverse direction, each span rotates separately about a vertical axis perpendicular to the bridge deck, therefore, the continuity of multi-span simply supported bridges is provided only by the simple connections of the decks to the abutments and columns. For this reason, in the mathematical model shown in Figure 9.1, the two span simply supported bridge is divided into two parts by imposing a gap equal to the width of the expansion joint between the abutment-fixed and column-fixed decks. Then, each deck is divided into 5 segments, and each segment is modelled as a 3-dimensional beam element. The mass of the bridge is divided and lumped at the nodes linking the beam elements. A rigid bar which has a length equal to the width of the bridge deck is connected to the tip of the beam element at the end of the column-fixed deck where the columns are located and oriented in the transverse direction. Assuming that the deck has infinite in-plane rigidity, this rigid bar is used to model the interaction between the translation of the columns in the transverse and longitudinal direction, and the translation as well as the rotation of the column-fixed deck about a vertical axis. The function of this rigid bar is illustrated in Figure 9.2 using the second transverse vibration mode shape of the structure. First, the bearing elements are connected to this rigid bar at the column locations and moments about the longitudinal and transverse axes are released at the upper end of the bearing elements. The bridge columns are then connected to these bearings. To measure the relative longitudinal displacement between the adjacent decks at the expansion joint, another rigid bar which has a length equal to the width of the bridge deck is connected to the tip of the beam element at the end of the abutment-fixed deck and oriented in the transverse direction. For compatibility, the transverse displacement and torsional rotation about an axis parallel to the bridge, of the abutment-fixed deck are constrained to be identical to those of the column-fixed deck at the mid-span end nodes of both decks. The translational degree of freedom of the node at the end of the abutment-fixed deck is set free in the longitudinal direction to allow the expansion of the deck.

The bearings at the abutments are modelled by summing the stiffnesses of each one and lumping their overall effect at the centerline of the bridge deck by using



equivalent rotational and translational springs as described in Chapter 7. In the case of sliding-bearings on the columns, their longitudinal stiffness is calculated considering only the deformation of the bearing bar and their deformation in the transverse direction is neglected.

Additionally, 3, 4 and 5 span simply supported bridges are modelled to investigate the effect of number of spans on the seismic response. A procedure similar to that of 2 span simply supported bridge is followed in modelling the multi-span simply supported bridges. In the mathematical models of these bridges, rigid bars are used at each expansion joint. The function of these rigid bars is illustrated in Figure 9.3, using the third vibration mode shape of a 5 span simply supported bridge.

### **9.3 Transverse Direction Response**

Response spectrum analyses of five 2-lane and five 3-lane, two span simply supported bridges are conducted for four different bearing types using three different response spectra; MPI SD spectrum of Western USA earthquakes and Western and Eastern Canada design spectra. A total of 120 cases are analyzed using the program SAP90 to visually express the bearing forces and maximum resistible peak ground accelerations considering columns' instability, as a function of span length for various earthquake spectra. Transverse direction fundamental periods of the 2 and 3 lane bridges are shown in Figures 9.4 and 9.5 respectively and the longitudinal direction fundamental periods of the column-fixed decks are shown in Figure 9.6. As seen in these figures the periods of the structures are highly dependent on the stiffness of the bearings used. For example, transverse direction fundamental periods of 2 lane bridges range between 1.7 and 2.2 seconds when bearings with zero rotational stiffness are used and they range between 0.18 and 1.13 seconds when bearings with infinite rotational stiffness are used. The transverse direction fundamental periods of 2 and 3 lane bridges are very close when bearings at the abutments have zero rotational stiffness, but the difference becomes larger as the bearings

stiffness increases. This is a consequence of the increasing contribution of the deck stiffness to the response as rotational stiffness increases.

In the subsequent sections, the effect of bearing stiffness on the overall stiffness of the structure is investigated, analytical expressions to magnify the seismic moments in the columns are derived, bridges with 3, 4 and 5 spans of 40 metres each are analyzed to investigate the effect of number of spans on the seismic response, and the effect of column size and height on the seismic response and capacity is investigated.

### **9.3.1 Effect of Bearing Stiffness on the Overall Stiffness of the structure**

Consider a two span simply supported bridge. The bridge is composed of an abutment-fixed and a column-fixed deck. The rotational and transverse direction translational stiffness of the fixed bearings-set and the stiffness of the bridge deck contributes to the overall stiffness of the abutment-fixed deck. Knowing this, the abutment-fixed deck is idealized as a beam connected to a rotational and a translational spring. When a unit load is applied at the tip of this beam, the total displacement at the load location is contributed by the transverse displacement,  $\Delta_{bT}$  of the bearings at the abutment, the displacement,  $\Delta_{br}$ , due to the rotation of the bridge deck at the support and the displacement,  $\Delta_d$ , due to flexural and shear deformation of the bridge deck idealized as a cantilever beam. The transverse displacement of the bearings due to a unit load applied at the tip of the beam is inverse of the bearings' total transverse stiffness,  $K_{bT}$  and

similarly the displacement of the cantilever beam is the inverse of the deck stiffness,  $k_D$ , where;

$$k_D = \frac{1}{\frac{L^3}{3EI_D} + \frac{L}{GA_{Ds}}} \quad (9.1)$$

In the above equation  $I_D$  and  $A_{Ds}$  are respectively the moment of inertia and shear area of the bridge deck. The unit load applied at the tip of abutment-fixed deck creates a moment equal to  $1 \cdot L$  at the support. Knowing this, the rotation of the deck at the abutment is calculated by dividing this moment by the rotational stiffness of the bearings-set,  $K_{b\theta}$ . Then, the displacement at the tip of the deck is the span length times this rotation and it is expressed as;

$$\Delta_{b\theta} = \frac{L^2}{K_{b\theta}} \quad (9.2)$$

The sum of these displacements due to a unit load at the tip of this idealized cantilever gives the flexibility of the abutment-fixed deck. The stiffness,  $K_{AFD}$ , of the abutment-fixed deck is the inverse of this flexibility and it is expressed as;

$$K_{AFD} = \frac{1}{\frac{1}{K_{bT}} + \frac{L^2}{K_{b\theta}} + \frac{1}{k_D}} \quad (9.3)$$

The translation and rotation of the column-fixed deck at the columns location is resisted by the rotational and translational stiffness of the columns-set, transverse stiffness of the bearings supporting the deck at the abutment and the stiffness of the deck itself. To calculate the translational stiffness of the columns-set, first, each column and the bearing located on, are idealized as two springs connected in series. This modelling is valid here, since all columns and bearings are identical among each other, and bearings

and columns not separated by spreader beams. Then, an equivalent transverse and longitudinal stiffness,  $k_{cT}$  and  $k_{cL}$ , for each column-bearing assembly is calculated as;

$$k_{cT} = \frac{1}{\frac{1}{k_{bT}} + \frac{h_c^3}{3EI_{cy}}} \quad (9.4)$$

$$k_{cL} = \frac{1}{\frac{1}{k_{bL}} + \frac{h_c^3}{3EI_{cx}}} \quad (9.5)$$

where,  $I_{cy}$  and  $I_{cx}$  are respectively the weak and strong axis inertias of the columns and  $k_{bT}$  and  $k_{bL}$  are the stiffness of an individual bearing in the transverse and longitudinal direction respectively. Assuming that all columns have the same size at a bent, the translational and longitudinal stiffness of the columns-set,  $K_{cT}$  and  $K_{cL}$ , are obtained by multiplying the stiffness of a column-bearing assembly by the number of columns,  $nc$ . Then, assuming that the deck has infinite in-plane rigidity in the longitudinal direction, the rotational stiffness of the columns-set is obtained as;

$$K_{c\theta} = nc J_c + \sum_{i=1}^{nc} k_{cL} d_{ci}^2 \quad (9.6)$$

where  $J_c$  is the torsional inertia of a column and  $d_c$  is the distance of a column to the centerline of the bridge deck. Then, the stiffness,  $K_{cFD}$ , of the column-fixed-deck is defined as;

$$K_{cFD} = K_{cT} + \frac{1}{\frac{L^2}{K_{c\theta}} + \frac{1}{k_D}} \quad (9.7)$$

The transverse stiffness,  $K_{TT}$ , of the bridge is the sum of the stiffnesses of the abutment-fixed and column-fixed decks. This stiffness is highly dependent on the

stiffness of the bearings. As observed in Eq. 9.3, stiffness of the abutment-fixed deck can contribute to the overall stiffness only if a rotational and a translational resistance is developed by the fixed bearings-set at the abutments. For example, if the rotational stiffness of the fixed bearings-set is reduced to zero (this may happen if the bearings are damaged or if they have negligible or zero longitudinal stiffness), then the stiffness of the abutment-fixed-deck becomes zero. Should this be the case, that part of the structure behaves like a mechanism. Therefore, the rotational and translational stiffnesses of the fixed bearings-set located on the abutment are very important parameters that affect the overall stiffness, henceforth the period, of two span simply supported bridges.

If the number of spans are more than two, the effect of bearing stiffness is more localized, and it eventually vanishes with increasing number of spans. For example, in a 3 span simply supported bridge, the stiffness of bearings at the abutment affects the displacement of the abutment-fixed deck, and therefore, only the displacement of the first nearest columns-set. The relative transverse displacement of the other columns-set will not be affected.

It is noteworthy that the total stiffness of the structure is largely affected by the equivalent transverse stiffness of the columns which is partly a function of the stiffness of the bearings located on the columns. For example, bearings with small stiffness, such as elastomeric bearings, may affect the overall stiffness of the structure considerably, but, other less flexible bearings do not. In that latter case, the total stiffness of the structure is mostly affected by the stiffness of the columns, especially in bridges with more than two spans.

### **9.3.2 Derivation of Moment Magnification Factors For Columns**

Consider a two span simply supported bridge. When it is subjected to seismic excitation in the transverse direction, the columns displace in both transverse and

longitudinal directions. The longitudinal displacement of the columns is due to the rigid body rotation of the column-fixed deck about a vertical axis and is proportional to the distance of the columns to the centerline of the bridge deck. Consequently, first and second order moments are generated in both directions. Considering the second order effects, the transverse and longitudinal direction magnified moments,  $\beta_{my}M_{y0}$  and  $\beta_{mx}M_{x0}$  are expressed as;

$$\beta_{my}M_{y0} = M_{y0} + M_{y1} + M_{y2} + M_{y3} + \dots + M_{yn} \quad (9.8)$$

$$\beta_{mx}M_{x0} = M_{x0} + M_{x1} + M_{x2} + M_{x3} + \dots + M_{xn} \quad (9.9)$$

where  $M_{y0}$  is the first order transverse direction moment, and  $M_{y1}$ ,  $M_{y2}$ ,  $M_{y3}$ ,  $M_{yn}$  are the second order moments obtained respectively from the 1<sup>st</sup>, 2<sup>nd</sup>, 3<sup>rd</sup> and n<sup>th</sup> iteration. The same definition applies for the longitudinal direction moment for each column. Here,  $M_{xy}$  stands for the longitudinal direction moment produced by transverse direction seismic excitation. To obtain the transverse direction magnified moments, as an initial iteration, the calculated first order elastic displacement,  $\Delta_{y0}$ , at the columns location is multiplied by the axial force and a second order moment for all the columns is obtained. Similarly, to obtain the longitudinal direction magnified moment for each column, the calculated first order elastic displacement  $\Delta_{xyj}$  of column  $j$  is multiplied by the axial force on the column and the second order moment is obtained. The same procedure is repeated for other columns across the deck to obtain the second order moments in the longitudinal direction. In the subsequent iterations, the second order transverse and longitudinal direction moments obtained in the previous step are divided by the moment arm and incremental shear forces are found for each column in both transverse and longitudinal directions. The incremental shear forces resulting from the transverse direction second order moments are summed up and a new incremental transverse loading,  $F_p$ , is obtained. This incremental load is then applied on the structure at the columns-set location.

The shear forces produced from longitudinal direction second order moments are multiplied by the distance,  $d_{cp}$ , of each column to the centerline of the bridge deck and an in-plane incremental torsional moment,  $T_i$ , acting on the expansion joint, is obtained. When the incremental transverse force,  $F_i$ , is applied on the structure alone, it produces a transverse displacement and a rotation. Similarly, when the incremental in-plane torsional moment,  $T_i$ , is applied on the structure alone, it also produces a transverse displacement and a rotation. The torsional stiffness of each individual column is neglected, and the torsional stiffness of the columns-set is negligible compared to the stiffness of the deck. Therefore, it is appropriate to assume the column-fixed deck as rigid. Then, the rotation of the deck at the columns location is identical to the rotation at the support and it is simply obtained by dividing the displacement at the columns-set location by the span length. Knowing this, and considering the contributions from both loading, the new incremental midspan displacement,  $\Delta I$ , and the new incremental rotation,  $\Theta I$ , are expressed as follows:

$$\Delta I = \frac{F_i}{K_{TT}} + \frac{T_i}{L K_{TT}} \quad (9.10)$$

$$\Theta I = \frac{F_i}{L K_{TT}} + \frac{T_i}{L^2 K_{TT}} \quad (9.11)$$

This incremental displacement is multiplied by the axial force to obtain the new transverse direction incremental moment. Then, the incremental rotation is multiplied by the distances of the columns to the centerline of the bridge deck and the new longitudinal direction incremental displacements of the columns are obtained. This new incremental displacements are multiplied by the axial forces to obtain the new longitudinal direction incremental moment for each column. The above procedure is illustrated for the first

three steps as follows:

**Step 1 :**

The second order incremental transverse moment and shear as well as the incremental transverse loading are calculated as:

$$M_{yl} = P \Delta_{y0} \quad (9.12)$$

$$VI_{yl} = \frac{M_{yl}}{h_c} = \frac{P \Delta_{y0}}{h_c} \quad (9.13)$$

$$F_1 = nc VI_{yl} = \frac{nc P \Delta_{y0}}{h_c} \quad (9.14)$$

The second order longitudinal moment and incremental shear,  $VI_{xylj}$  of a column j as well as the incremental torsional moment are calculated as;

$$M_{xylj} = P \Delta_{xylj} = P \Theta_0 d_{cj} = P \frac{\Delta_{y0}}{L} d_{cj} \quad (9.15)$$

$$VI_{xylj} = \frac{M_{xylj}}{h_c} = \frac{P \Delta_{y0} d_{cj}}{L h_c} \quad (9.16)$$

$$T_1 = \sum_{j=1}^{nc} VI_{xylj} d_{cj} \quad (9.17)$$

Substituting Eq. 9.16 into 9.17 the incremental torsional moment is expressed as;

$$T_1 = \frac{P \Delta_{y0}}{L h_c} \sum_{j=1}^{nc} d_{cj}^2 \quad (9.18)$$



Then, Eqs. 9.14 and 9.18 are substituted into Eqs. 9.10 and 9.11 to obtain the incremental transverse displacement and rotation as;

$$\Delta I_1 = \frac{P \Delta_{y0}}{K_{TT} h_c} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) \quad (9.19)$$

$$\Theta I_1 = \frac{P \Delta_{y0}}{L K_{TT} h_c} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) \quad (9.20)$$

### Step 2 :

The second order incremental transverse moment and shear as well as the incremental transverse loading in the second step are defined as;

$$M_{y2} = P \Delta I_1 = \frac{P^2 \Delta_{y0}}{K_{TT} h_c} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) \quad (9.21)$$

$$VI_{y2} = \frac{M_{y2}}{h_c} = \frac{P^2 \Delta_{y0}}{K_{TT} h_c^2} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) \quad (9.22)$$

$$F_2 = nc VI_{y2} = \frac{P^2 \Delta_{y0}}{K_{TT} h_c^2} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) nc \quad (9.23)$$

The second order longitudinal moment and incremental shear of a column j as well as the

incremental torsional moment in the second step are calculated as;

$$M_{xy2j} = P \Theta I_1 d_{cj} = \frac{P^2 \Delta_{y0}}{K_{TT} h_c} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) \frac{d_{cj}}{L} \quad (9.24)$$

$$VI_{xy2j} = \frac{M_{xy2j}}{h_c} = \frac{P^2 \Delta_{y0}}{K_{TT} h_c^2} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) \frac{d_{cj}}{L} \quad (9.25)$$

$$T_2 = \sum_{j=1}^{nc} VI_{xy2j} d_{cj} \quad (9.26)$$

Substituting Eq. 9.25 into 9.26 the incremental torsional moment in the second step is expressed as;

$$T_2 = \frac{P^2 \Delta_{y0}}{K_{TT} h_c^2} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) \frac{\sum_{j=1}^{nc} d_{cj}^2}{L} \quad (9.27)$$

Then, Eqs. 9.23 and 9.27 are substituted into Eqs. 9.10 and 9.11 and arranged to obtain the incremental transverse displacement and rotation as;

$$\Delta I_2 = \frac{P^2 \Delta_{y0}}{K_{TT}^2 h_c^2} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right)^2 \quad (9.28)$$

$$\Theta I_2 = \frac{P^2 \Delta_{y0}}{L K_{TT}^2 h_c^2} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right)^2 \quad (9.29)$$

**Step 3 :**

The second order incremental transverse moment in the third step is expressed as;

$$M_{y3} = P \Delta I_2 = \frac{P^3 \Delta_{y0}}{K_{TT}^2 h_c^2} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right)^2 \quad (9.30)$$

and the second order longitudinal moment of a column j in the third step is expressed as;

$$M_{xy3j} = P \Theta I_2 d_{cj} = \frac{P^3 \Delta_{y0}}{K_{TT}^2 h_c^2} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right)^2 \frac{d_{cj}}{L} \quad (9.31)$$

Substituting the moments obtained in the above steps in Eqs. 9.8 and 9.9 the following expressions are obtained;

$$\beta_{my} M_{y0} = M_{y0} + P \Delta_{y0} + P \Delta_{y0} \lambda + P \Delta_{y0} \lambda^2 + \dots + P \Delta_{y0} \lambda^{n-1} \quad (9.32)$$

$$\beta_{mxy} M_{xy0j} = M_{xy0j} + \frac{P \Delta_{y0} d_{cj}}{L} + \frac{P \Delta_{y0} d_{cj}}{L} \lambda + \frac{P \Delta_{y0} d_{cj}}{L} \lambda^2 + \dots + \frac{P \Delta_{y0} d_{cj}}{L} \lambda^{n-1} \quad (9.33)$$

where,

$$\lambda = \frac{P}{K_{TT} h_c} \left( nc + \frac{\sum_{j=1}^{nc} d_{cj}^2}{L^2} \right) \quad (9.34)$$

Dividing both sides of Eq. 9.32 by  $M_{y0}$  and Eq. 9.33 by  $M_{xy0j}$  and representing them in series form, the following equations are obtained;

$$\beta_{my} = 1 + \frac{P \Delta_{y0}}{M_{y0}} \sum_{i=1}^n \lambda^{i-1} \quad (9.35)$$

$$\beta_{mxy} = 1 + \frac{P \Delta_{y0} d_{cj}}{L M_{xy0j}} \sum_{i=1}^n \lambda^{i-1} \quad (9.36)$$

$M_{y0}$  and  $M_{xy0j}$  are defined in the following forms;

$$M_{y0} = \Delta_{y0} k_{cT} h_c \quad (9.37)$$

$$M_{xy0j} = \frac{\Delta_{y0}}{L} d_{cj} k_{cL} h_c \quad (9.38)$$

Equations 9.37 and 9.38 are substituted respectively in Eqs. 9.35 and 9.36 and the magnification factors are expressed as;

$$\beta_{my} = 1 + \frac{P}{k_{cT} h_c} \sum_{i=1}^n \lambda^{i-1} \quad (9.39)$$

$$\beta_{mxy} = 1 + \frac{P}{k_{cL} h_c} \sum_{i=1}^n \lambda^{i-1} \quad (9.40)$$

As the number of iterations increases, more accuracy is obtained. Therefore, the limits of the above equations as  $n$  approaches infinity are the most accurate representations of the moment magnification factors.

$$\beta_{my} = \lim_{n \rightarrow \infty} \left( 1 + \frac{P}{k_{cT} h_c} \sum_{i=1}^n \lambda^{i-1} \right) \quad (9.41)$$

$$\beta_{my} = 1 + \frac{P}{k_{cT} h_c} \frac{1}{1 - \lambda} \quad (9.42)$$

Similarly,

$$\beta_{mxy} = 1 + \frac{P}{k_{cL} h_c} \frac{1}{1 - \lambda} \quad (9.43)$$

Note that the above equations are valid if  $\lambda$  is less than 1.0.

Although these equations are derived for 2 span simply supported bridges, they can equally be used to obtain the magnified elastic sway moment of multi-span simply supported bridge columns. In fact, the applicability of these equations for bridges with more than two spans is due to the discontinuous nature of the structure. In other words, a transverse displacement imposed at one of the columns-set is negligibly effects the adjacent ones, and therefore only the stiffness of the two neighbouring decks contributes to the overall stiffness of that degree-of-freedom. Note that the transverse stiffness may change depending on the location of the columns-set. If the moment magnification factors for columns which are not adjacent to the abutment-fixed deck is required then only the stiffness of the column fixed deck contributes to the overall stiffness.

### 9.3.3 Bearing Forces at the Abutments

The TBFC for various bearing types are plotted as a function of span length in Figures 9.7, 9.8 and 9.9 using respectively MP1SD spectrum of Western USA earthquakes and Eastern and Western Canada design spectra. As seen in these figures, in the case of bearings-set with infinite rotational stiffness, the TBFC and bearing forces increase with span length for spans up to 40 metres when MP1SD spectrum of Western USA earthquakes or Western Canada design spectrum is used in the analyses and they increase for spans up to 30 metres when Eastern Canada design spectrum is used in the analyses. Beyond this range the TBFC decreases, but the bearing forces in fact increases. Within the above-mentioned span ranges, the periods of the structures fall in the constant part of the response spectra and therefore the attracted spectral accelerations are constant.

Consequently, the increase in TBFC results directly from the increase, with span length, of in-plane moment at the support. Beyond these specified span ranges, the periods of the structures fall in the descending part of the response spectra and therefore the attracted spectral accelerations become less. Although the in-plane moment at the support still increases with span, this increase is offset by the reduction in spectral acceleration. Consequently, TBFC slightly decrease beyond the aforementioned span ranges, but, again, bearing forces still increase.

It is noteworthy that the rate of reduction of the spectral acceleration as a function of period, in the descending part of the spectrum, is more in the MP1SD spectrum of western USA earthquakes than that in Canadian design spectra. Therefore, when MP1SD spectrum of Western USA earthquakes is used in the analyses, in spite of the increase of in-plane moment at the support, the bearing forces do not change very much beyond the specified span ranges. However, for the other spectra, the increase in bearing forces is more remarkable. For other types of bearings, the TBFC slightly increase with span length for spans up to 30 metres. Beyond this range the TBFC decreases. The bearing forces results (not presented here) are almost constant for spans larger than 30 and 40 metres respectively for 2 and 3 lane bridges when MP1SD spectrum of Western USA earthquakes is used in the analyses. When other spectra are used in the analyses, although the bearing forces increase with span length, this increase is not large. For example, the bearing force in a 2-lane 60 metres span bridge is only 16 percent larger than that in a 2-lane 30 metres span bridge when sliding bearing with 2 bolts are used.

The forces in bearings-set with zero rotational stiffness are nearly zero. This is a consequence of the relatively small spectral accelerations attracted by the structure due its high period and absence of moment at the support, but more importantly, it is largely attributable to the bridge's behavior where decks can rotate freely about their supports at the abutments and act as a mechanism.

In Figure 9.10 the ratio of bearings forces of 2 to 3-lane bridges is plotted as a function of span length. It is observed that for bearings-set with zero rotational stiffness this ratio is almost constant and approximately equal to 1. However, for other types of bearings, the ratio is larger than 1 for short span bridges and gradually decreases with increasing span length. When sliding-bearings are used, the bearing forces in 2-lane bridges are larger than those in 3-lane bridges up to 40 metres span and then they become smaller with increasing span length, but the difference is not very large (about 20% for 60 metres span bridge). In the case of bearings with infinite rotational stiffness, the forces in the bearings of 2-lane bridges are always larger than those in 3-lane bridges for the range of spans considered but the difference gets smaller as the span length increases.

### 9.3.3 Response of Columns in the Transverse Direction

When multi-span simply supported bridges are subjected to seismic excitation in the transverse direction, moments in both transverse and longitudinal directions are produced in the columns respectively due to translation and rigid body rotation of the column-fixed deck about a vertical axis. The resulting magnified transverse and longitudinal direction seismic moments  $M_{Ey}$  and  $M_{Exy}$  are expressed as;

$$M_{Ey} = \beta_{my} m_{Ey} A_p \quad (9.44)$$

$$M_{Exy} = \beta_{mxy} m_{Exy} A_p \quad (9.45)$$

where  $m_{Ey}$  and  $m_{Exy}$  are respectively transverse and longitudinal direction seismic moments in the columns for a unit peak ground acceleration. The maximum resistible peak ground acceleration that can be reached prior to column instability is obtained using the biaxial

stability interaction equations proposed by Duan and Chen (1989).

$$\left( \frac{M_{Exy}}{M_{ax}} \right)^2 + \left( \frac{M_{Ey}}{M_{ay}} \right)^\alpha \leq 1.0 \quad (9.46)$$

$$\alpha = 1.2 + 2 \frac{P}{P_y} - 0.03\lambda_x \geq 1 \quad (9.47)$$

$M_{ax}$ ,  $M_{ay}$ ,  $P_y$  and  $\lambda_x$  have already been defined in Chapter 8. Note that the axial forces in the columns produced by seismic excitation in the transverse direction are zero since the connection between the deck and each column is a hinge. Therefore, only the axial forces due to gravity loading act on the columns. Knowing this and substituting Eqs 8.2 and 8.6 together with Eqs. 9.44 and 9.45 into the biaxial stability interaction equation and arranging, the following expression as a function of peak ground acceleration is obtained.

$$\left( \frac{\beta_{mxy} m_{Exy} A_p}{M_{rx} \left( 1 - \left( \frac{P_D}{P_n} \right)^\eta \right)} \right)^2 + \left( \frac{\beta_{my} m_{Ey} A_p}{M_{ry} \left( 1 - \left( \frac{P_D}{P_n} \right)^\xi \right)} \right)^\alpha - 1 = 0 \quad (9.48)$$

Using the above equation and the response spectrum analyses results obtained for various spans, number of lanes and bearing types, the maximum resistible peak ground accelerations prior to columns failure are obtained. The results are plotted as a function of span length in Figures 9.11, 9.12 and 9.13 respectively for MP1SD spectrum of Western USA earthquakes and Western and Eastern Canada design spectra. As seen in these figures, the maximum resistible peak ground accelerations for 3-lane bridges are larger than those for 2-lane bridges when bearings that develop rotational resistance at the supports are used. This is actually a consequence of the smaller lateral displacements of the columns-set of bridges with wider decks that have larger in-plane stiffness. It is noteworthy that the contribution of the deck stiffness to the overall transverse stiffness is highly dependent on the rotational resistance created by the bearings in the case of 2 span simply supported bridges. This contribution becomes more pronounced when



bearings with larger stiffness are used at the abutments and therefore even less lateral displacement is produced at the columns location. Consequently, the difference between the maximum resistible peak ground accelerations of 2 and 3-lane bridges becomes larger as the rotational stiffness of the bearings-set increases.

Note that when the bearings-set have zero rotational stiffness, the stiffness of the abutment-fixed deck does not contribute to the lateral stiffness of the structure. Therefore, the maximum resistible peak ground accelerations are identical for 2 and 3 lane bridges and very low.

As observed in Figures 9.11, 9.12 and 9.13, increasing span length has a negative impact on the seismic capacity. Although longer bridges may attract smaller spectral accelerations due to their long periods which fall in the descending part of the response spectra, their displacement at the columns location is very large. Therefore, the calculated first and second order forces in columns become dominant in longer bridges, as a result, these bridges can only resist smaller seismic forces.

It is noteworthy that the presented maximum resistible peak ground accelerations are those corresponding to the most vulnerable columns. Transverse and longitudinal direction seismic moments are produced in the columns due to transverse displacement and rigid body rotation of the column-fixed deck about an axis perpendicular to it. The transverse direction seismic moments are identical for all the columns since they have the same transverse displacement due to high axial stiffness of the deck in both principle directions. However, the longitudinal displacement of the columns is proportional to their distance to the centerline of the bridge deck. Accordingly, columns closer to the edge of the deck are the most vulnerable ones; incidentally this effect is even more pronounced in skewed bridges (Ghobarah and Tso, 1974).

### 9.3.4 Effect of Column Size on the Seismic Response

The effect of the capacity of steel columns on the seismic performance of bridges is studied. The response spectrum analysis of a 2-lane simply supported bridge with 2 spans of 40 metres each is conducted in the transverse direction for various column sizes using MP1SD spectrum of Western United States earthquakes. The bridge is assumed to have bearings with zero rotational stiffness at the abutments. Four different column sizes, WWF350x155, WWF400x178, WWF450x228 and WWF500x276 are used in the analyses. The obtained transverse and longitudinal direction seismic moments are substituted in Eq. 9.48 and maximum resistible peak ground accelerations considering column instability are obtained for each case. The results are illustrated in Figure 9.14. As seen in the figure, when the size of the bridge column increases from WWF350x155 to WWF500x276, the seismic capacity nearly doubles. Analyses show that the increase in column size reduces the midspan deflection significantly when the columns are the dominant lateral load resisting structural elements as is the case here. The midspan deflection for the bridge with the largest column size is 3 times smaller than that of the bridge with the smallest column size.

The fundamental period of the structure also reduces when the column size increases. For example, when the column size increases from WWF350x155 to WWF500x276, the period reduces by 50 percent. This reduction in period results in higher seismic forces, hence higher forces in the bearings. The difference in bearing forces of the cases considered, from the smallest to largest column sizes, is 33 percent. However, considering the magnitude of the bearing forces for bearings-set with zero rotational stiffness, the increased bearing forces are still not very significant. The percentage of shear force attracted by columns also increases with increasing column size but the difference is not very significant either.

Additionally, sliding bearings with 4 bolts are introduced to the same bridge model and analyzed using two different column sizes, WWF350x155 and WWF500x276.

Taking all effects into account by substituting the larger column (WWF500x276) to the bridge originally with the smaller one (WWF350x155), the period of the bridge is reduced by 21 percent attracting larger seismic forces to the structure. In the midspan, displacement reduced by 22 percent, seismic capacity increased only by 14 percent and bearing forces are reduced by 17 percent. These results showed that the effect of column size is more pronounced for two span simply supported bridges with bearings-set of zero rotational stiffness, as expected, since the deck in these bridges does not contribute to the overall transverse stiffness of the structure and lateral resistance is provided only by the columns.

For bridges with number of spans larger than 2, the fixed-bearings at the abutment affect only the displacement of the abutment-fixed deck and, therefore, only the columns adjacent to it. Accordingly, the effect of column size may still be significant in these bridges regardless of the bearing stiffness. This will be investigated in section 9.3.6.

### **9.3.5 Effect of Column Length on the Seismic Response**

The effect of the length of columns on the seismic performance of bridges is studied. The columns in a 2-lane simply supported bridge with 2 spans of 20 metres each are assigned different lengths and redesigned accordingly. For lengths of 6, 7, 8, 9 and 10 metres, the obtained redesigned, sections are respectively W310x129, W360x134, W360x147, WWF400x157 and WWF400x178. Then, response spectrum analyses of the bridges with different column sizes are conducted in the transverse direction using Eastern Canada design spectrum. The bridges are assumed to have bearings with zero rotational stiffness at the abutments. The obtained transverse and longitudinal direction seismic moments in the columns with their corresponding magnification factors are substituted in Eq. 9.48 and maximum resistible peak ground accelerations considering column instability are obtained for each case. The percentage of increase in the seismic capacity of bridges relative to that of the bridge with 6 metres column is plotted in Figure 9.15 as a function

of column length. As seen in the figure, there are sporadic jumps in capacity when the column length is increased, a maximum increase of 16 percent is reached for the 9 metres column. Considering the magnitude of maximum resistible peak ground acceleration for bridges with bearings-set of zero rotational stiffness, this increase is not important. In fact, this small increase results from (i) the considerable reduction in fundamental period of the structure with increasing column length which subsequently results in less seismic forces, (ii) the increased flexural capacity of the column as the section becomes bigger with increasing length to prevent buckling under gravity loading, (iii) the reduction in the relative magnitude of seismic moments with respect to the capacity of the column.

The percentage of increase in expansion joint displacements relative to that of the bridge with 6 metres column is plotted as a function of column length in Figure 9.16. As seen in the figure, expansion joint displacements increase by as much as 68 percent for the longest column used in the analyses, in spite of the very small increase in the seismic capacity. This shows that, bridges with longer columns can be more seismically vulnerable than those with shorter columns; the large openings in the expansion joints more easily lead to failure of the structure if the support width is exceeded. Additionally, adjacent bridge decks may collide more easily due to these large openings in the expansion joints, resulting in high forces at the abutments and expansion joints.

It is noteworthy that although there is also a reduction of bearing forces in bridges with longer columns, it is not very significant. For example, the difference between the transverse bearing forces of bridges with 6 and 10 metres columns is only 5 percent.

#### **9.3.6 Effect of Number of Spans on the Seismic Response**

Two lane simply supported bridges with 2, 3, 4, and 5 spans are considered. Each span is assumed to have 40 metres length. Since the decks are simply supported on the

columns, the girder and column sizes of all the multi-span simply supported bridges considered are identical. The bridges are assumed to have two types of bearings; (i) sliding-bearings with 4 bolts and (ii) bearings-set with zero rotational stiffness. For each type of bearings, response spectrum analyses of the bridges are conducted using MP1SD spectrum of Western USA earthquakes.

Analyses results show that in the case of bridges with bearings-set of zero rotational stiffness, the contribution of the first mode is much more than that of the other modes. In the case of bridges with sliding-bearings, the contribution of higher modes is more predominant. In both cases the contribution of higher modes increases with number of spans. Results show that, the number of modes to be considered in the analyses should be at least equal to the total number of spans to obtain an accurate prediction of the response in the transverse direction.

The fundamental periods of the structures also increase with number of spans, however, it is not significant in the case of bridges with bearings-set of zero rotational stiffness. For bridges with other type of bearings the difference between the fundamental periods of 2 and 3 span bridges is more than two times, but for bridges with larger number of spans the difference is much smaller.

The maximum transverse direction displacement of bridges with different number of spans are compared. It is found that, for bridges with bearings-set of zero rotational stiffness, the variation of maximum transverse displacement as a function of number of spans is not significant. For bridges with sliding-bearings, the maximum transverse displacement of the 2-span bridge is much smaller than the others, however the displacements of bridges with higher number of spans are comparable. Empirically, it is found that the maximum transverse displacement of a bridge with  $n_s$  number of spans, supported by bearings-set with high rotational stiffness, is close to that of a bridge with  $n_s-1$  number of spans supported by bearings-set with zero rotational stiffness. For example, the maximum transverse displacement of the 5 span bridge with sliding-bearings

is 0.1041 metre and that of the 4 span bridge with bearings-set of zero rotational stiffness is 0.1030 metre.

From the response spectrum analyses, the transverse and longitudinal direction seismic moments of the columns are obtained. It is found that, in the case of bridges with sliding-bearings, the moments in the columns adjacent to the abutment-fixed deck are lower than those of the other columns. This results from the smaller deformation of the abutment-fixed deck due to the combined rotational resistance of the bearings-set and the transverse resistance of the deck. For each bridge, only the columns with largest seismic moments are considered. These moments are substituted in Eq. 9.48 with their magnification factors and the maximum resistible peak ground acceleration considering column instability is obtained for each bridge. The results are illustrated in Figure 9.17. As seen in the figure, the 2 span bridge has a relatively high capacity when sliding-bearings are used. However, the capacity rapidly drops to more than half when bearings-set with zero rotational stiffness are used. The effect of bearing stiffness on the seismic capacity diminishes for bridges with number of spans greater than 2. The seismic capacity of these bridges is almost identical. In fact, the seismic capacity of multi-span simply supported bridges, regardless of the bearing stiffness, is nearly identical to that of the 2 span bridge with bearings-set of zero rotational stiffness. Therefore, the seismic capacity of these bridges in the transverse direction can be predicted by that of the 2 span bridge with bearings-set of zero rotational stiffness of identical individual span lengths. Accordingly, Figures 9.11 to 9.13 can be used to obtain the seismic capacity of multi-span simply supported steel bridges.

Additionally, the response of a 3-span, simply supported bridge with 120 metres total end-to-end length is compared to that of a 2-span simply supported bridge of identical end-to-end length. It has been demonstrated earlier that bearing stiffness has a considerable effect on the response of 2-span bridges. Therefore, the bridges are assumed to have bearings-set with zero rotational stiffness. It is found that the two span bridge

is more vulnerable to seismic excitations than the three span bridge of identical end-to-end length. This is a result of several factors;

- i) Since the length of each individual span is longer in bridges with smaller number of spans, larger forces due to gravity loading is produced in the superstructure components. This results in larger member sizes. Therefore, the total mass of bridges with smaller number of spans is larger. This produces higher seismic forces in the structure.
- ii) The axial forces due to gravity loading in the columns of bridges with smaller number of spans are larger, therefore its negative impact on the flexural capacity is more significant.
- iii) The stiffness of the columns in bridges with smaller number of spans is larger, therefore greater seismic moments are attracted.
- iv) The number of columns-set in bridges with smaller number of spans is less than that of bridges with larger number of spans. Consequently, their lateral resistance is not as much. This results in larger transverse displacements and larger first and second order seismic moments in the columns.
- v) The bearing forces in bridges with smaller number of spans are larger. This results from the increased proportion of mass per bearing since the length of each individual span is longer in bridges with smaller number of spans.

It is noteworthy that the maximum resistible peak ground accelerations obtained from the analyses are very small regardless of the number of spans. These results are actually not surprising. Imbsen and Penzien (1986) studied the seismic response of some existing multi-span simply supported reinforced concrete bridges. One of these bridges was the Fields Landing Overhead which suffered major damage during the Trinidad-

Offshore, California Earthquake of November 8, 1980. They estimated that a peak ground acceleration of about 0.1 to 0.2g occurred at the bridge site during the earthquake. Furthermore, Linguini et al. (1979), categorized multi-span simply supported bridges as unsound i.e, certain failure, if not retrofitted. Results of the current study agree with these findings.

### **9.3.7 Effect of Condition of Bearings and Expansion Joints on the Seismic Response**

In reality, the ideal case of zero rotational stiffness at the expansion joints may not exist and this has a considerable impact on the prediction of the seismic capacity of bridges sensitive to bearing stiffness. Douglas and Reid (1982) conducted a dynamic quick release test of a five span 120 metres long bridge supported on elastomeric bearings to investigate its dynamic properties. They compared the test results with the analytical ones. Results showed that the calculated values of the combined abutment-bearings stiffness was 15-20 times smaller than the results obtained from the test. They concluded that this large difference was caused by the presence of wind-blown sand and gravel deposited around the bearings and between the end of the deck and the abutment wall. Wilson (1985) studied the dynamic behavior of an existing multi-span concrete highway bridge. He observed that the dynamic response is affected by the condition of the expansion joints. He concluded that this effect was produced by locking of the expansion joints possibly as a result of a certain amount of corrosion at the bearing interfaces and an accumulation of wind-blown debris over a period of years. These show that, rotational resistance at both fixed and expansion bearings may be generated by weathering conditions. Therefore, the maximum resistible peak ground accelerations obtained from the analyses considering bearings-set with zero rotational stiffness may be too conservative. It is possible that multi-span simply supported bridges could resist higher seismic forces than calculated.



The width of the expansion joint is also an important parameter affecting the seismic response and capacity considerably. Consider a multi-span simply supported bridge subjected to seismic excitation in the transverse direction. If the expansion joints widths are reduced by weathering conditions (e.g. accumulation of wind-blown debris), then after a small transverse displacement the corners of the adjacent decks will get into contact. If the bridge attempts to displace further, flexural and axial stresses will be developed in the decks since they can not rotate freely at the expansion joints. Consequently, the stiffness of the decks will contribute to the overall stiffness of the structure. This will reduce the transverse displacements and rotations of the bridge decks as well as the displacements and the transverse and longitudinal direction seismic moments in the columns. Overall, the structure may sustain larger seismic excitations before the columns suffer any damage.

#### **9.3.8 Effect of Damage to Bearings on the Seismic Response**

Analyses showed that fixed bearings with high longitudinal stiffness, such as sliding bearings, attract large forces since they resist the rotation of the deck at the abutment and therefore may be damaged. In this section, the impact of damage to bearings on the seismic response and capacity is investigated.

Two span simply supported bridges are used for that purpose. When the fixed bearings at one of the abutments are damaged, the bridge deck is free to rotate (zero rotational stiffness). Then, the abutment-fixed and column-fixed decks act as a mechanism and the only transverse resistance is provided by the columns. For sliding to occur, the seismic forces produced at the supports should be at least equal to the friction

resistance force expressed as:

$$F_s = \frac{mg}{4} \mu_f \quad (9.49)$$

The seismic reaction forces at the abutments, for a unit peak ground acceleration are obtained using MP1SD spectrum of western USA earthquakes. The friction resistance force expressed above is divided by these forces to obtain the peak ground accelerations required for sliding. The results for different friction coefficients are plotted in Figure 9.18 together with the maximum resistible peak ground accelerations considering column instability, as a function of span length. As seen in the figure, the maximum resistible peak ground accelerations are much smaller than the peak ground accelerations required to initiate sliding, even considering the friction forces produced between steel and steel and between concrete and steel. This shows that the columns will be damaged much before any sliding occurs at the bearings. Moreover, very high peak ground accelerations are required to slide the bridge even if the columns could remain undamaged. For example, for a 2-lane simply supported bridge with two spans of 30 metres each, peak ground accelerations of 0.54g and 0.86g are required to initiate sliding, considering respectively the friction between steel and steel and steel and concrete. Therefore, sliding of multi-span simply supported bridges in the transverse direction is not a serious problem and not considered in this study.

Therefore, the most important impact of damage to bearings remains the reduction of the seismic capacity of two span simply supported bridges; When the bearings are damaged, the resistance of the bridge deck to rotation at the support diminish and higher displacements at the column locations are produced. This increases the seismic moments in the columns and consequently decreases the seismic capacity of the bridge.

## 9.4 Longitudinal Direction Response

In the longitudinal direction, when a multi-span simply supported bridge is subjected to seismic excitation, there is not any interaction between the spans until the neighbouring decks collide. The abutment-fixed deck is very rigid longitudinally, therefore, its longitudinal displacement is negligible. However, any column-fixed deck is very flexible since the longitudinal movement of the deck is resisted only by the steel columns. Consequently, these displacements can be very high and the decks may collide before the columns are damaged. This will be investigated following.

The maximum longitudinal displacement,  $\Delta_{cL}$ , that the columns can accommodate before failure is calculated by dividing the maximum resistible strong axis moment obtained using Eq. 8.6, by the column's height and its longitudinal stiffness.

$$\Delta_{cL} = \frac{M_{ax}}{h_c k_{cl}} \quad (9.50)$$

These displacements and the expansion joint widths of the bridges considered in this study are plotted in Figure 9.19 as a function of span length. It is observed that the columns can easily sustain longitudinal displacements much larger than the expansion joint width. This shows that collision of the decks are inevitable before the columns are damaged.

### 9.4.1 Bridges With Two Spans

For the two-span simply supported bridges considered in this study, since there is an abutment-fixed deck on one side and an abutment wall on the other side of the bridge, the movement of the column-fixed deck in the longitudinal direction is restricted by the width of the expansion joint, which is roughly a maximum of 4-5 cm for the ranges of spans considered. Figure 9.19 already illustrated that the steel bridge columns can

accommodate that much displacement and that impact between two adjacent sections of the bridge may occur for a given slightly intense earthquake. Should that occurs, the total seismic load of the bridge deck will be transmitted to the foundation primarily through the impact between the deck and abutment, with the columns carrying only a small percentage of the total load (Chen and Penzien, 1979). However, impacting between the two adjacent sections of a bridge superstructure upon collision causes high shear forces in the bearings and therefore these components can fail before the columns (Zimmerman and Brittain, 1981; Imbsen and Penzien, 1986). Such failure could make the structural system unstable since the superstructure is disconnected from the columns and the simply supported decks may fall off their support if the seat width is not adequate (Zimmerman and Brittain, 1981). After the bearings are damaged, each individual deck can potentially have a maximum displacement equal to the sum of the expansion joints width.

There are two expansion joints in a two span simply supported bridge and the sum of their widths can equal up to a maximum of 8-10 cm for the range of spans considered. In Figure 9.19 the sum of the expansion joints width is plotted, together with the maximum longitudinal displacement that the columns can accommodate before failure. It is observed that steel columns can accommodate displacement as large as twice the expansion joint width. Therefore, 2 span bridges are considered to be longitudinally safe if (i) other components of the bridge are not damaged (ii) the seat width at the supports is adequate and (iii) the deformation of the abutments are negligible.

#### **9.4.2 Multi-Span Bridges**

As the number of spans increases, the sum of the expansion joints widths also increases. This sum may be larger than the maximum displacement that the steel columns can accommodate before failure. In this case the safety of the columns in the longitudinal direction cannot be ensured since the system can potentially have a maximum displacement equal to the sum of the expansion joint widths. In fact, the maximum

opening in an expansion joint is found to be a function of the frequency of the earthquake (Zimmerman and Brittain, 1981), and expansion joint openings equal to the sum of the total expansion joint widths can occur (Zimmerman and Brittain, 1981; Imbsen and Penzien, 1986). However, if the sum of the expansion joint widths are less than the maximum column displacements obtained using Eq. 9.50, then the columns are expected to survive seismic excitations in the longitudinal direction. Using this information, the number of spans for which columns are considered to be safe are plotted in Figure 9.20 as a function of span length. As seen in that figure, the columns in multi-span simply supported bridges of 20 metres spans with up to 6 spans are considered to be safe, while 60 metres bridges with only a single span (without column) are considered to be safe in the longitudinal direction.

#### **9.4.3 Effect of Damage to Bearings on the Seismic Response**

It is noteworthy that, the loss of support due to damage to bearings has been responsible for several multi-span simply supported bridge failures in the past. The San Fernando California earthquake of 1971, the Guatemala earthquake of 1976, and the Eureka California earthquake of 1980, are some examples of earthquakes in which bridge collapse resulted from bearing failure (Seismic, 1987). Even relatively minor earthquakes have caused failure of anchor bolts, keeper bar bolts and welds (Seismic, 1987). This results from the impacting between the two adjacent sections of a bridge superstructure upon collision which causes high shear forces in the bearings (Zimmerman and Brittain, 1981; Imbsen and Penzien, 1986). When the bearings get damaged due to the effect of impact, the bridge becomes unstable in the longitudinal as well as in the transverse direction since the deck is disconnected from the columns, and eventually the spans fall off their supports if there is not adequate seat width. Therefore, impacting should certainly be prevented to avoid damage to bearings and to the structure.

#### 9.4.4 Minimum Peak Ground Acceleration For Collision to Occur

Neglecting the effect of travelling seismic waves for the range of spans considered, when multi-span simply supported bridges with identical column-fixed decks are subjected to seismic excitation, the movements of all the column-fixed decks are in phase until there is a collision. The collision takes place when the deck displacement is equal to the expansion joint width,  $EJW$ . Note that the displacement of the column-fixed deck,  $\Delta_s$ , produced by seismic forces is magnified by second order effects. For impact to occur this magnified displacement should be equal to the expansion joint width; This is mathematically expressed using the magnification term at the right hand side of Eq. 8.26;

$$EJW = \left[ \frac{1}{1 - \frac{P}{k_{cL}h_c}} \right] \Delta_s \quad (9.51)$$

The seismic force,  $H$ , acting on the column fixed deck is expressed as;

$$H = m_D S_a = m_D A_p \beta \quad (9.52)$$

where  $m_D$  is the mass of the deck. Then, the displacement produced by this seismic force is defined as;

$$\Delta_s = \frac{m_D A_p \beta}{K_{cL}} \quad (9.53)$$

Substituting the above equation into Eq. 9.51, the minimum peak ground acceleration for first impacting to occur can be expressed as a fraction of gravitational acceleration as;

$$A_p = \frac{K_{cL} EJW}{\beta m_D g} \left( 1 - \frac{P}{k_{cL}h_c} \right) \quad (9.54)$$

Using the above equation, the minimum peak ground acceleration required to produce collision is plotted in Figure 9.21 as a function of span length, using 3 different spectra. As seen in that figure, the peak ground accelerations needed to produce collision increase

with span length but are all less than 0.1g. It is noteworthy that, the first impact occurs either between the column-fixed deck and the abutment wall at one end of the bridge or between the column-fixed and the abutment-fixed decks at the other end of the bridge, depending on the direction of maximum deflection of the column-fixed deck during the earthquake. These results assume that all the column-fixed decks have identical periods in the longitudinal direction. However, multi-span simply supported bridges may be composed of spans with different lengths, and most likely the periods of the column-fixed decks will be different. Then, the motion of each span depends on its period. As previously indicated, the seismic response of such bridges is a very complex random event, for which generalizations do not appear possible. Therefore a complete time history analysis is required for each specific case.

## **9.5 Minimum Required Seat width at the Expansion Joints**

Analytical expressions for the minimum required seat width at the expansion joints will be derived considering two criteria. The first one is the maximum expansion joint opening that a multi-span simply supported bridge can potentially have. This is the total sum of the expansion joints widths neglecting the deformation of the abutments. The second one is the maximum possible expansion joint opening that a bridge can have before the columns are damaged, considering the response in both principal directions. The second case may govern if the sum of the expansion joint widths is larger than the maximum expansion joint opening before the columns are damaged. In any case, the smaller of these two criteria will define the maximum possible expansion joint opening in a multi-span simply supported bridge.

### 9.5.1 Maximum Possible Expansion Joint Openings

Maximum possible expansion joint openings of a two span and a four span simply supported bridge in the longitudinal direction are illustrated schematically in Figure 9.22. For the two span bridge, the maximum opening is equal to the expansion joint width since the movement of the column-fixed deck is restricted by the abutment-fixed deck at the left side and the abutment wall at the right side. For the same reason, the four span bridge shown in the figure can have a maximum expansion joint opening equal to three times the expansion joint width, assuming that all the joints have the same width.

If the same mentioned two and four span simply supported bridges displace in the transverse direction, due to the rotation of each span, the corners of the decks at the expansion joints also displace longitudinally, and, eventually, will get in contact with each other and with the abutment walls. At that point, neglecting the deformation of the abutments, further opening of the expansion joints will not be possible. Similar to the longitudinal direction case, the maximum opening in the expansion joint at the column locations is equal to the expansion joint width for the two span bridge, and the same maximum opening in the expansion joints at any one of the column locations is equal to three times the expansion joint width for the four span bridge. Accordingly, for a bridge with identical expansion joint widths, the maximum possible expansion joint opening,  $\Delta_{ejo}$ , is given by the following equation;

$$\Delta_{ejo} = EJW ( n_{ej} - 1 ) \quad (9.55)$$

where  $n_{ej}$  is the number of expansion joints.

The expansion joint openings obtained from the case studies on multi-span simply supported bridges conducted by Tseng and Penzien (1973), Zimmerman and Brittain, (1981), and Imbsen and Penzien (1986) are compared with the maximum possible expansion joint openings obtained using the above equation. Most of these results



reported in the literature were close to the maximum possible as predicted by the above equation. This confirms that multi-span simply supported bridges can potentially have seismically-induced expansion joint openings nearly reaching the maximum physically possible.

If the fixed bearings at the left abutment seen in Figure 9.22 are also damaged, the abutment-fixed deck is free to slide in the longitudinal direction. Then, the deck can potentially have a maximum displacement equal to the sum of the expansion joints widths. Accordingly, assuming that the bearings are damaged, the maximum possible expansion joint opening for bridges with identical expansion joint widths is defined as;

$$\Delta_{ejo} = EJW n_{ej} \quad (9.56)$$

The designed width of each expansion joint is determined by multiplying the span length by the coefficient of thermal expansion,  $\alpha_T$ , and the temperature difference  $\Delta T$ . Using this information, the sum of the expansion joints widths, i.e. the maximum possible expansion joint opening is expressed by the following equation ;

$$\Delta_{ejo} = \alpha_T \Delta T \sum_{i=1}^{N_s} L_i = \alpha_T \Delta T L_T \quad (9.57)$$

where  $N_s$  is the number of spans,  $L_i$  is length of an individual span  $i$  and  $L_T$  is the total end-to-end length of the bridge. Assuming a maximum temperature difference of 70 C° in Canada and using  $12 \times 10^{-6}$  per C° for the coefficient of thermal expansion of steel and concrete, the maximum possible expansion joint opening in mm is expressed as;

$$\Delta_{ejo} = 0.84 L_T \quad (9.58)$$

In the above equation  $L_T$  should be in metres and  $\Delta_{ejo}$  in millimetres.

Depending on the temperature difference, the bridge spans contract or elongate. When the temperature is at its minimum design value, the expansion bearing is closer to the seat edge. Therefore, an allowance for temperature difference should be provided when evaluating the adequacy of the seat width against seismic actions. This allowance is conservatively calculated by multiplying the length,  $L_i$ , of the span supported by the expansion bearings at expansion joint  $i$  by the same coefficient of thermal expansion and temperature difference. Additionally, a certain distance between the edge of the support and the centerline of the bearings should be provided to prevent failure of the structure due to local damage at the support edge. A distance of 50 mm is considered to be appropriate, but the engineers must determine it, based on the particulars of detailing for each individual case, if more generous contingency is necessary. Using the above information, the minimum required seat width  $SW_i$  (in mm) at expansion joint  $i$  is expressed by the following equation;

$$SW_i = 50 + 0.84 ( L_T + L_i ) \quad (9.59)$$

Note that the above equation gives an estimate of the minimum required distance between the centerline of the bearing and the edge of the support to prevent bridge decks from falling off their seats.

### **9.5.2 Maximum Expansion Joint Openings Due To Longitudinal Direction Displacements of Columns**

In this section, the maximum possible expansion joint openings due to seismic excitation in the longitudinal direction is studied. The sum of the expansion joint widths is assumed to be larger than the expansion joint openings due to the maximum relative displacement of the columns before they are damaged. Accordingly, the maximum expansion joint openings are controlled by the maximum deformation capacity of the columns prior to failure. The maximum possible expansion joint openings due to relative

displacement of columns are illustrated in Figure 9.23 for a two span and a multi-span simply supported bridge. For the two span simply supported bridge the expansion joint opening is equal to the maximum displacement of the column before failure. For the multi-span simply supported bridge, the opening of the  $i^{\text{th}}$  expansion joint is equal to the sum of the maximum displacements of columns  $i-1$ , and  $i$  before failure. As a general rule, when considering the opening of an expansion joint, the column at the expansion joint of interest and the neighbouring column connected to the deck by fixed bearing should always be considered. For example, in the system shown in Figure 9.23, the decks on the left side of the expansion joints are connected by fixed bearings to the columns, therefore, the columns on the left side of the expansion joints of interest are considered.

The maximum column displacement before failure is obtained using Eq. 9.50. Then the expansion joint opening due to relative displacement of columns is expressed as;

$$\Delta_{ejoi} = \Delta_{cL_{i-1}} + \Delta_{cL_i} \quad (9.60)$$

Considering the movement of the bridge deck due to temperature difference and providing again a distance of 50 mm between the edge of the support and the centerline of the bearings to prevent failure of the structure due to local damage at the support edge, the minimum required seat width,  $SW_i$  at expansion joint  $i$  is expressed by the following equation:

$$SW_i = 50 + 0.84 L_i + \Delta_{cL_{i-1}} + \Delta_{cL_i} \quad (9.61)$$

Using Eq. 9.50, the above equation is expressed in a more explicit form as;

$$SW_i = 50 + 0.84 L_i + \frac{M_{ax_{i-1}}}{k_{cL_{i-1}}} + \frac{M_{ax_i}}{k_{cL_i}} \quad (9.62)$$

where  $M_{ax}$  is obtained from Eq. 8.6.

For practical applications, the above equation needs to be simplified and represented as a function of column height and span length. Using the maximum possible column displacements before failure already calculated for the bridges considered in this study, determined by Eq. 9.50 and plotted in Figure 9.19, a simplified function is derived for columns with 6 metres height to express the maximum possible column displacement (in millimetre) as a function of span length (in metre).

$$\Delta_{cL} = 150 - L \quad (9.63)$$

The above equation is plotted in Figure 9.24 along with the data obtained from the previous analyses. As seen in the figure, the approximate function yields slightly larger column displacements, but this is conservative since these displacements are used to determine the needed seat width.

So far, Eq. 9.63 can only be used to determine the maximum displacement of columns with 6 metres height as a function of span length. Therefore, it should be expanded to include also the column height as a variable. In a previous section, the columns in a 2-lane simply supported bridge with 2 spans of 20 metres each were assigned different lengths and redesigned accordingly. They were assumed to have 6, 7, 8, 9 and 10 metres lengths and corresponding sections of respectively W310x129, W360x134, W360x147, WWF400x157 and WWF400x178. To increase the size of the available data, the columns in a 2-lane simply supported bridge with 2 spans of 40 metres each are assigned 6, 7, 8, 9 and 10 metres length and redesigned. The resulting new column sections are respectively WWF350x155, WWF400x157, WWF400x202, WWF450x201 and WWF450x228. The maximum longitudinal displacements of these columns before failure are calculated using Eq. 9.50. Then, for each span, the ratio,  $\Delta_{cLR}$ , of this displacement for the longer columns to 6 metres long columns is calculated. Similar ratios are obtained for both the 20 and 40 metres spans bridges. Results, plotted

in Figure 9.25, show that  $\Delta_{cLR}$  is independent of the span length. Moreover, the following simpler approximate linear function is proposed to represent  $\Delta_{cLR}$  ;

$$\Delta_{cLR} = \frac{h_c}{5} \quad (9.64)$$

This function is also plotted in Figure 9.25 to illustrate the good match with the analyses result. As seen in the figure the proposed function is a safe upper bound to the calculated ratios.

The analytical expression for column displacements in Eq. 9.63 was derived considering a column height of 6 metres and the above function gives the magnitude of column displacements relative to that of 6 metres column. Therefore, the multiplication of these two functions gives the maximum possible displacements of the columns as a function of span length and column height;

$$\Delta_{cL} = ( 150 - L ) \frac{h_c}{5} \quad (9.65)$$

The above function is plotted in Figure 9.26 together with some of the analytical results denoted by solid circles. As seen in that figure, although the above equation yields conservative values, the difference is not major.

Note that, the axial forces on the columns of a multi-span simply supported bridge are obtained by summing the reaction forces from the two neighbouring spans supported by the columns. Then, the column sizes are determined using these axial forces which are proportional to the span length. Accordingly, for multi-span simply supported bridges with different individual span lengths, the average of the two adjacent spans supported

by the column can be used in the above equation. Knowing this, the above equation is represented in a more general form as:

$$\Delta_{cL_i} = \left( 150 - \frac{L_i + L_{i+1}}{2} \right) \frac{h_{c_i}}{5} \quad (9.66)$$

where,  $L_i$  and  $L_{i+1}$  are the lengths of two adjacent spans supported by column  $i$ . The above equation is substituted in Eq. 9.61 and rearranged to obtain the minimum required seat width as a function of span length and column height.

$$SW_i = 50 + 0.84 L_i + \left( 30 - \frac{L_{i-1} + L_i}{10} \right) h_{c_{i-1}} + \left( 30 - \frac{L_i + L_{i+1}}{10} \right) h_{c_i} \quad (9.67)$$

### 9.5.3 Maximum Expansion Joint Openings Due To Transverse Direction Displacements of Columns

In this section, the maximum possible expansion joint openings due to seismic excitation in the transverse direction is studied. Potentially, large openings in the expansion joints may develop due to rotation of the decks when the bridge displaces in the transverse direction. The sum of the expansion joint widths is assumed to be larger than the expansion joint openings due to maximum relative displacement of the columns before they are damaged. Accordingly, the maximum expansion joint openings are controlled by the maximum deformation capacity of the columns before failure. The expansion joint openings of two, three and multi-span bridges are shown in Figures, 9.27, 9.28 and 9.29 respectively. First, consider the two span simply supported bridge. The maximum rotations of the decks are obtained by dividing the maximum transverse displacement of the column,  $\Delta_{CT}$ , prior to failure by their span lengths. The maximum expansion joint opening is then obtained by multiplying these rotations by the distance of the exterior bearing to the centerline of bridge deck and summing up the results.

Accordingly, for a two span simply supported bridge, the maximum expansion joint opening is expressed as;

$$\Delta_{ej0} = \Delta_{cT} l_b \left( \frac{1}{L_1} + \frac{1}{L_2} \right) \quad (9.68)$$

Now consider the three span simply supported bridge shown in Figure 9.28. The assumed deformed geometry in the figure produces the maximum rotation at the second deck hence a maximum expansion joint opening. In fact, this deformed geometry may also occur in multi-span simply supported bridges where the contribution of higher modes is significant. An example of one such assumed deformed shape is seen in Figure 9.3 which shows the third mode shape of a five span simply supported bridge. The rotation of the second deck in the three span simply supported bridge is controlled by the maximum transverse displacement of the columns before failure at each end of the deck. This rotation is obtained by dividing the sum of the maximum transverse displacements of the columns by the span length. Then, the expansion joint openings are obtained by multiplying the rotations of the decks at each side of the expansion joints by the distance of the exterior bearing to the centerline of the bridge deck and summing up the results. For example, for the expansion joint 1 shown in Figure 9.28, the maximum possible expansion joint opening is expressed as;

$$\Delta_{ej0_1} = l_b \left( \frac{\Delta_{cT_1}}{L_1} + \frac{\Delta_{cT_1} + \Delta_{cT_2}}{L_2} \right) \quad (9.69)$$

Following the same procedure defined for two and three span simply supported bridges, the maximum opening of an expansion joint  $i$  in a multispan simply supported bridge shown in Figure 9.29 is expressed as:

$$\Delta_{\phi o_i} = l_b \left( \frac{\Delta_{cT_{i-1}} + \Delta_{cT_i}}{L_i} + \frac{\Delta_{cT_i} + \Delta_{cT_{i+1}}}{L_{i+1}} \right) \quad (9.70)$$

Assuming that the exterior bearing is close to the bridge edge,  $l_b$  in the above equation can be replaced by the half of the width,  $b$ , of the bridge deck. Then, as before, considering longitudinal movements of the bridge deck due to temperature variations and providing a distance of 50 mm between the edge of the support and the centerline of the bearings to prevent failure of the structure due to local damage at the support edge, the minimum required seat width,  $SW_i$  at expansion joint  $i$  is expressed by the following equation;

$$SW_i = 50 + 0.84 L_i + \frac{b}{2} \left( \frac{\Delta_{cT_{i-1}} + \Delta_{cT_i}}{L_i} + \frac{\Delta_{cT_i} + \Delta_{cT_{i+1}}}{L_{i+1}} \right) \quad (9.71)$$

Due to transverse displacement and rotation of the decks, the columns have moments in both transverse and longitudinal directions. For simplicity, and conservatively, the moment produced in the longitudinal direction is not considered. Accordingly, to obtain the maximum transverse displacement before the columns are damaged, Eq. 9.50 is again used, but  $M_{xx}$  and  $k_{xL}$  in the equation are replaced respectively by  $M_{yy}$  and  $k_{yT}$ . This modified equation is substituted in the above equation to express the minimum required seat width in a more explicit form as;

$$SW_i = 50 + 0.84L_i + \frac{b}{2} \left[ \frac{1}{L_i} \left( \frac{M_{\phi y_{i-1}}}{k_{cT_{i-1}}} + \frac{M_{\phi y_i}}{k_{cT_i}} \right) + \frac{1}{L_{i+1}} \left( \frac{M_{\phi y_i}}{k_{cT_i}} + \frac{M_{\phi y_{i+1}}}{k_{cT_{i+1}}} \right) \right] \quad (9.72)$$

where  $M_{\phi y}$  in the above equation is obtained from Eq. 8.2



For practical applications, the above equation is again simplified and represented as a function of column height and span length. A procedure similar to the one described in Section 9.5.2 is followed. First, the maximum transverse displacements before failure of the columns are calculated for the bridges considered in this study. Using this data, the following linear function is derived for columns with 6 metres height to express the maximum possible column transverse displacement (in millimetre) as a function of span length in (metre).

$$\Delta_{cL} = 250 - \frac{3}{2}L \quad (9.73)$$

Then, using the previously designed columns with different lengths, the ratios,  $\Delta_{cTR}$ , of the columns' transverse displacements to that of 6 metres columns are calculated for two bridges with two spans of 20 metres and 40 metres. Similar ratios are again obtained for both bridges which shows that  $\Delta_{cTR}$  is still independent of span length. Using these ratios, an equation identical to Eq. 9.64 is obtained. The analytical expression for column displacements in Eq. 9.73 was derived considering a column height of 6 metres and Eq. 9.64 gives the magnitude of column displacements relative to 6 metres column. Therefore, the multiplication of these two equations gives the maximum transverse displacements before the failure of the columns, as a function of span length and column height;

$$\Delta_{cT} = \left( 250 - \frac{3}{2}L \right) \frac{h_c}{5} \quad (9.74)$$

The above approximate function is plotted in Figure 9.30 together with the exact analytical results denoted by solid circles. As seen in the figure, although the above equation yields conservative values, the difference is not very large especially for longer columns where buckling is more critical.

Finally, for multi-span simply supported bridges with different individual span length, the average of the two adjacent spans supported by the column can be used in the

above equation. Knowing this, the above equation is represented in a more general form as:

$$\Delta_{cT_i} = \left( 250 - \frac{3}{4} ( L_i + L_{i+1} ) \right) \frac{h_c}{5} \quad (9.75)$$

where,  $L_i$  and  $L_{i+1}$  are the lengths of two adjacent spans supported by column  $i$ . The above equation is substituted in Eq. 9.71 and rearranged to obtain the minimum required seat width as a function of span length and column height.

$$\begin{aligned} SW_i = & 50 + 0.84 L_i + \frac{b}{2} \left( \frac{50}{L_i} - 0.15 \left( 1 + \frac{L_{i-1}}{L_i} \right) \right) h_{c_{i-1}} + \\ & \frac{b}{2} \left( \frac{50}{L_i} + \frac{50}{L_{i+1}} - 0.15 \left( 2 + \frac{L_{i+1}}{L_i} + \frac{L_i}{L_{i+1}} \right) \right) h_{c_i} + \\ & \frac{b}{2} \left( \frac{50}{L_{i+1}} - 0.15 \left( 1 + \frac{L_{i+2}}{L_{i+1}} \right) \right) h_{c_{i+1}} \end{aligned} \quad (9.76)$$

### 9.5.3 Interpretation of Derived Equations for Minimum Seat Width

Three equations are derived to define the minimum required seat width for multi-span simply supported bridges. Eqs. 9.67 and 9.76 are derived considering the maximum displacement of the columns prior to failure due to longitudinal and transverse direction seismic excitations respectively. The purpose of these equations is to determine if a bridge is likely to fall off its supports before the columns reach their capacity. Since it is very difficult to predict the direction of the seismic excitations relative to the principal directions of the structure, the larger of the results obtained from these equations must be used to calculate the minimum required seat width considering the maximum displacement of the columns. However, neglecting the deformation of the abutments, the expansion joint opening defined by the maximum displacement of the columns cannot be larger than the sum of the expansion joint widths. Therefore, the larger of the results

obtained from Eqs. 9.67 or 9.76 should be compared with the result obtained from Eq. 9.59 and the smaller of these defines the minimum required seat width.

For bridges of identical individual span length and column height, Eqs. 9.59, 9.67 and 9.76 are plotted in Figure 9.31 as a function of span length for various number of spans considering a deck width of 8 and a column height of 6 metres. As seen in that figure, Eq. 9.76 provides results smaller than those of Eq. 9.67 for all the cases considered. Eq. 9.76 may yield larger seat widths only for wide and short span bridges where the deck rotations hence the expansion joint openings are larger. However, for short span bridges, Eq. 9.59 yields results smaller than those of Eq. 9.76 and therefore it governs. Accordingly, Eq. 9.76 need not be used in most of the cases. As seen in the figure, Eq. 9.59 governs for bridges with short and small number of spans. As the span length and number of spans increase Eq. 9.59 yields larger seat widths than those of Eq. 9.67 and therefore Eq. 9.67 governs in these cases.

It is noteworthy that Eqs. 9.67 and 9.76 can only be used for the ranges of spans considered in this study. For longer span bridges, Eqs. 9.62 and 9.72 can be used instead. When calculating the seat width on the abutments, conservatively, the height of the abutment can be substituted in Eqs. 9.67 and 9.76.

#### **9.5.4 Minimum Seat Width Considering Foundation and Abutment Flexibility**

Eqs. 9.67 and 9.76 are derived considering that the columns are fixed at the foundation. Due to unfavourable soil conditions, the columns may rotate and displace at the foundation, resulting in larger expansion joint openings. In this case, Eqs. 9.62 and 9.72 can directly be used instead of Eqs. 9.67 and 9.76, but the column stiffness must be calculated considering the flexibility of the foundation. For example, in the longitudinal direction, neglecting the stiffness of the bearings on the column, and considering only the

rotational stiffness of the foundation, the longitudinal stiffness of the column is expressed as;

$$k_{cL} = \frac{3EI_{cx} k_{\theta f}}{3EI_{cx} h_c^2 + k_{\theta f} h_c^3} \quad (9.77)$$

where  $k_{\theta f}$  is the rotational stiffness of the foundation.

It is noteworthy that if the abutments of an existing bridge are expected to have large deformations when subjected to seismic excitations, Eq. 9.59 must be neglected, and instead, the larger of the results obtained from Eqs. 9.67 and 9.76 is directly used to calculate the minimum required seat width.

#### 9.5.5 Minimum Seat Width Considering Inelastic Deformation of Columns

If future research on compact steel bridge columns shows that they have some inelastic deformation capability, then Eqs. 9.62 and 9.72 should be modified to account for this. First, the reduced transverse and longitudinal direction plastic moments,  $M_{pry}$  and  $M_{prx}$ , considering the effect of axial force are calculated using the following equations (Duan and Chen, 1989);

$$M_{pry} = M_{py} \left( 1 - \left( \frac{P_D}{P_y} \right)^3 \right) \quad (9.78)$$

$$M_{prx} = M_{px} \left( 1 - \left( \frac{P_D}{P_y} \right)^{1.3} \right) \quad (9.79)$$

The deformation of the columns at their plastic capacity is calculated by dividing the plastic moments by the lateral stiffness of the columns. Then, the maximum deformation

of the columns prior to failure is determined by multiplying these deformations by the displacement ductility,  $\mu_c$ , of the columns. Substituting these maximum deformations in Eqs. 9.62 and 9.72 the minimum required seat widths in the longitudinal and transverse directions are expressed respectively in the following forms;

$$SW_l = 50 + 0.84 L_i + \mu_c \left( \frac{M_{prx_{i-1}}}{k_{cL_{i-1}}} + \frac{M_{prx_i}}{k_{cL_i}} \right) \quad (9.80)$$

$$SW_t = 50 + 0.84 L_i + \mu_c \frac{b}{2} \left[ \frac{1}{L_i} \left( \frac{M_{pry_{i-1}}}{k_{cT_{i-1}}} + \frac{M_{pry_i}}{k_{cT_i}} \right) + \frac{1}{L_{i+1}} \left( \frac{M_{pry_i}}{k_{cT_i}} + \frac{M_{pry_{i+1}}}{k_{cT_{i+1}}} \right) \right] \quad (9.81)$$

## 9.6 Summary

In this section findings for the transverse and longitudinal seismic response of multi-span simply supported bridges are summarized. The response of these bridges is investigated considering several different geometric and structural parameters. The findings are summarized following.

### 9.6.1 Response in the Transverse Direction

Bearing stiffness is found to considerably affect the seismic response of two span simply supported bridges. The transverse direction periods of these bridges are highly dependent on the stiffness of the bearings used. In particular it is observed that:

- The transverse direction fundamental periods of 2 and 3 lane bridges are very close when bearings-set at the abutments have zero rotational stiffness, but the difference becomes larger as the stiffness increases.
- For bridges with more than two spans, the effect of bearing stiffness is localized and it vanishes with increasing number of spans.
- When the bearings-set have very small rotational stiffness, the total stiffness of the structure is only affected by the combined stiffness of the columns and the bearings supported on these columns, and the contribution of the deck stiffness is negligible.

Beyond the stiffness and period issues, level of force acting on the bearings of two span simply supported bridges is also very important. These forces were showed to be affected as follows:

- For bearings-set of infinite rotational stiffness, the forces in the bearings at the abutments increase with span length significantly for spans up to 30-40 metres, beyond this range, the increase is not as much.
- For bearings-set of infinite rotational stiffness, the forces in the bearings of 2-lane bridges are always larger than those in 3-lane bridges for the range of spans considered, but the difference gets smaller as the span length increases.
- When sliding-bearings are used, the bearing forces at the abutments of 2 lane bridges are larger than those in 3-lane bridges up to 40 metres span and then they become smaller with increasing span length, but the difference is not very large.
- The forces in bearings-set with zero rotational stiffness are approximately the same for 2 and 3 lane bridges. However, they are negligibly small compared to

the forces in other types of bearings. The bearing forces at the abutments of two span simply supported bridges can conservatively be assumed to be identical to the forces in the bearings of multi-span simply supported bridges.

The effect of various factors on the seismic capacity of multi-span simply supported bridges considering column instability is also studied. The followings are observed:

- For two span simply supported bridges, the maximum resistible peak ground accelerations for 3-lane bridges are larger than those for 2-lane bridges when bearings that develop rotational resistance at the supports are used. The difference becomes more pronounced as the rotational stiffness of the bearings-set increases. However, this effect vanishes for bridges with larger number of spans.
- For bridges with bearings-set of zero rotational stiffness, the maximum resistible peak ground accelerations are identical for 2 and 3 lane bridges. However, they are greatly reduced compared to those of the bridges with other types of bearings considered in this study. It is found that bridges with this type of bearings may be damaged by earthquakes of peak accelerations less than 0.20g.
- The condition of the expansion joints may have a considerable impact on the seismic capacity. If the width of the expansion joint in a multi-span simply supported bridge is reduced by weathering conditions or the movement of its bearings are prevented by rust, the structure may sustain larger earthquakes before the columns suffer damage.
- Increasing span length is also found to have a negative impact on the seismic capacity due to high moments exerted on the columns.

The effect of column size and height on the seismic response of multi-span simply supported bridges is studied. The followings are observed:

- The increase in column size reduced the midspan deflection and expansion joint openings, and increased the seismic capacity significantly.
- Although a slight increase in the seismic capacity with column length is found, bridges with longer columns are actually not superior to ones with shorter columns since large openings in the expansion joints occur.
- Columns closer to the edge of the deck are found to be the most vulnerable ones. This effect is pronounced more in skewed bridges (Ghobarah and Tso, 1974).

Furthermore, the effect of number of spans on the seismic response of multi-span simply supported bridges is studied. The findings are presented as follows:

- In the case of bridges with bearings-set of zero rotational stiffness, the variation of maximum transverse displacement as a function of number of spans is not significant.
- In the case of bridges with sliding-bearings, the maximum transverse displacement of two span bridges is found to be much smaller than that of the ones with larger number of spans. However, the difference between the maximum transverse displacements of bridges with more than two spans is not considerable. Therefore, the transverse seismic capacities of multi-span simply supported bridges are almost identical. In fact, regardless of the bearing stiffness, the transverse seismic capacity of multi-span simply supported bridges is nearly identical to that of a two span simply supported bridge with bearings-set of zero rotational stiffness and identical individual span length.



- Bridges with smaller number of spans are found to be more vulnerable to seismic excitations than those with larger number of spans of identical end-to-end length.

The effect of damage to bearings on the seismic response is investigated. It is found that high intensity earthquakes are required to slide these bridges when the bearings at the abutments are damaged. Therefore, sliding is not a serious problem in multi-span simply supported bridges. However, the most important impact of damage to bearings is to reduce the seismic capacity of two span simply supported bridges that results from large deflections at the column locations.

#### **9.6.2 Response in the Longitudinal Direction**

The followings are observed for the seismic response of multi-span simply supported bridges in the longitudinal direction;

- Columns in multi-span simply supported bridges can sustain displacements as much as twice the expansion joint width. Therefore, the columns in two span simply supported bridges are considered to be safe in the longitudinal direction.
- For simply supported bridges with more than two spans, the safety of the columns in the longitudinal direction can not be assured since the system can potentially have a maximum displacement equal to the sum of the expansion joint widths.
- The peak ground accelerations required for collision increases with span length, but they are all less than 0.1g. Therefore, collision of the decks in the longitudinal direction is inevitable.
- Impacting between the two adjacent sections of a bridge superstructure upon collision causes high shear forces in the bearings and therefore these components

fails before the columns (Zimmerman and Brittain, 1981; Imbsen and Penzien, 1986). Then, the system becomes unstable since the superstructure is disconnected from the columns and the simply supported decks may fall off their support if the seat width is not adequate. Therefore, impacting should certainly be prevented to avoid damage to bearings hence to the structure.

## **CHAPTER 10**

### **FORMULATION OF COMPONENT STRESSES AND RANKING OF SLAB-ON-GIRDER STEEL BRIDGES**

In this chapter, first, analytical expressions to obtain the seismic forces in critical bridge superstructure components, namely bearings and columns, will be derived. As a first step, mass and period of the bridge types considered in this study will be expressed analytically. Using these expressions and properties of the structures, analytical functions for the forces in the bearings and columns will be obtained. Using these functions and the findings from the previous chapters, a methodology will be derived to rank slab-on-girder steel bridges according to their vulnerability to earthquake excitations.

#### **10.1 Formulation of Component Stresses**

##### **10.1.1 Calculation of Bridge Mass**

In the following sub-sections, analytical expressions are derived to be used in the absence of available data, to calculate the mass of existing slab-on-girder steel bridges of the types considered. These empirical expressions are derived using the relationship between mass, span length and width observed for the bridges studied.

#### 10.1.1.1 Single and Multi-Span Simply Supported Bridges

The total mass of single and multi-span simply supported existing slab-on-girder steel bridges as a function of span length is approximated by the following equation:

$$m = b \sum_{i=1}^{n_s} 0.686 L_i + \frac{L_i^2}{190} \quad (10.1)$$

where  $n_s$  is the number of spans and  $b$  and  $L_i$  are width and length of a single span respectively. In Figure 10.1, the ratio of the mass of simply supported bridges considered in this study to the mass obtained from the above equation is illustrated. Since the ratio is close to 1.0 for all span ranges, a good match exists between the two.

#### 10.1.1.2 Continuous Bridges

Assuming that all the spans have identical girder sizes, the total mass of slab-on-girder continuous steel bridges is approximated by the following equation;

$$m = b L_T \left( 0.694 + \frac{L_{av}}{172} \right) \quad (10.2)$$

where  $L_T$  is the total end-to-end length of the bridge and  $L_{av}$  represents the largest of the average lengths of adjacent spans. In Figure 10.1, the ratio of the mass of continuous bridges considered in this study to the mass obtained from the above equation is illustrated. As seen, the ratio is close to 1.0 for all span ranges.

#### 10.1.2 Calculation of Fundamental Periods of the Bridges

In the following sub-sections, analytical expressions are derived for the fundamental periods of slab-on-girder steel bridges of the types considered in this study.

These expressions will later be used in conjunction with a response spectrum to calculate the spectral acceleration attracted by the structure.

### 10.1.2.1 Single Span Simply Supported Bridges

#### 10.1.2.1.1 Transverse Direction

The transverse fundamental mode shape of single span simply supported bridges is represented by the trigonometric function expressed in Eq. 7.9. Using this function, the generalized mass, as expressed in Eq. 7.10, was obtained as half of the total mass of the structure. Including the effect of local translational and rotational springs  $k_{ti}$  and  $k_{ri}$ , located at distances  $x_i$  from the support, the generalized stiffness is represented by the following equation (Clough and Penzien, 1975);

$$k^* = \int_0^L EI \left( \frac{d^2 \Phi(x)}{dx^2} \right)^2 dx + \sum k_{ti} \Phi^2(x_i) + \sum k_{ri} \left( \frac{d\Phi(x_i)}{dx} \right)^2 \quad (10.3)$$

In the case of single span simply supported bridges, the only local spring is the one that represents the rotational stiffness,  $K_{b\theta}$ , of the bearings-set located at the support (i.e. at  $x=0$ ). Accordingly, for an unrestrained rotation at both ends, the third term on the right hand side of the above equation becomes the rotational stiffness of the bearings-set multiplied by the square of the rotation,  $\theta_{s0}$ , at the support which is the first derivative of the shape function. However, this rotation is not equal to the actual rotation in bridges having bearings-set with non-zero rotational stiffness, since the shape function does not include the effect of negative rotation  $\theta_M$  produced by the in-plane support moment. Knowing this, a correction is needed and the actual rotation,  $\theta_s$ , is expressed as;

$$\theta_s = \theta_{s0} - \theta_M \quad (10.4)$$

The negative rotation is obtained by dividing the support moment,  $M_s$ , by the rotational

stiffness of the bridge deck.

$$\theta_M = \frac{M_s}{\frac{3EI_D}{L} + K_{b\theta}} = \frac{\theta_s K_{b\theta}}{\frac{3EI_D}{L} + K_{b\theta}} \quad (10.5)$$

Substituting the above equation in Eq. 10.4, the actual rotation is expressed as follows;

$$\theta_s = \theta_{s0} \left( \frac{1}{1 + \frac{1}{\frac{3EI_D}{L K_{b\theta}} + 1}} \right) \quad (10.6)$$

Rearranging the above equation;

$$\theta_s = \theta_{s0} \left( 1 - \frac{1}{\frac{3EI_D}{L K_{b\theta}} + 2} \right) \quad (10.7)$$

Substituting the shape function and the corrected support rotation in Eq. 10.3, the generalized stiffness is expressed as;

$$k^* = \int_0^L EI_D \left( \frac{\pi^2}{L^2} \sin \frac{\pi x}{L} \right)^2 dx + K_{b\theta} \left( \left( 1 - \frac{1}{\frac{3EI_D}{L K_{b\theta}} + 2} \right) \frac{\pi}{L} \cos \frac{\pi x}{L} \Big|_{x=0} \right)^2 \quad (10.8)$$

or,

$$k^* = \frac{\pi^4 EI_D}{2 L^3} + K_{b\theta} \left( 1 - \frac{1}{\frac{3EI_D}{L K_{b\theta}} + 2} \right)^2 \frac{\pi^2}{L^2} \quad (10.9)$$

The fundamental period is expressed as;

$$T_1 = 2\pi \sqrt{\frac{m^*}{k^*}} \quad (10.10)$$

Substituting Eqs 7.10 and 10.9 into the above equation and simplifying, the fundamental period of a simply supported bridge is obtained as follows;

$$T_1 = \sqrt{\frac{2 m L^2}{\frac{\pi^2 EI_D}{2 L} + K_{bb} \left( 1 - \frac{1}{\frac{3EI_D}{L K_{bb}} + 2} \right)^2}} \quad (10.11)$$

Note that the above equation can be used only if the bearings have infinite transverse stiffness and the rotational stiffness of the bearings-set is not very large relative to the stiffness of the bridge deck. The ratio of the deck stiffness to the rotational stiffness of the bearings-set is expressed as;

$$RS = \frac{3EI_D}{L K_{bb}} \quad (10.12)$$

The fundamental periods of bridges with  $RS$  as small as 1.20 are calculated using both computer analyses and the derived analytical expression, and a very good agreement is found. For the simply supported bridges considered in this study, the ratio of the transverse fundamental periods obtained from computer analyses to the ones obtained from Eq. 10.11 is plotted in Figure 10.2 as a function of span length. As seen in the figure, the ratio is very close to 1.0 for all the span ranges. The small differences are essentially due to the fact that:

- i) Bridges are divided into 10 segments when analyzed by the computer whereas the derived equation assumes continuity;

- ii) Shear deformation of the bridge deck is taken into account in the computer analyses but neglected in the above equations;
- iii) In the derivation of the equation, the support rotation is obtained by correcting the rotation obtained from the first derivative of the mode shape. This factor has the largest impact on the observed differences, and the discrepancies decrease when the rotational stiffness of bearings-set approaches zero.

The same explanation holds for the subsequent comparisons.

For elastomeric bearings, the stiffness of the bearings is very small compared to the stiffness of the deck, and the deformation of the bridge deck is negligible relative to the deformation of the bearings. Consequently, an alternate approach is necessary, in that case, it is more appropriate to idealize the bridge as a SDOF system. The stiffness of this SDOF system is the sum of the transverse stiffness,  $K_{bT}$ , of the bearings-set at each end of the bridge, i.e  $2 \cdot K_{bT}$ , and its mass is the total mass of the bridge deck. Knowing this, the fundamental period is simply expressed as;

$$T_1 = 2\pi \sqrt{\frac{m}{2 K_{bT}}} \quad (10.13)$$

It is noteworthy that, as the stiffness of elastomeric bearings in orthogonal directions are similar, the transverse and longitudinal direction fundamental periods are identical.

#### 10.1.2.1.2 Longitudinal Direction

In the longitudinal direction, the bridge is idealized as a beam connected to a translational spring, where the spring and the beam represent the fixed bearings-set and the bridge deck respectively. Suppose that a unit axial load is applied at the end of this



spring-beam system. In that case, the displacement at the spring location is equal to the deformation of the bearings which is the inverse of the longitudinal stiffness,  $K_{bL}$ , of the fixed-bearings-set, and the displacement of the system at a distance  $x$  from the support is the sum of the bearings displacement and the deck displacement at the specified location. Assuming that the variation of the axial deformation of the deck is linear (Bathe, 1987), the displacement of the system due to a unit axial load applied at the end of the deck is defined as;

$$\Delta_D(x) = \frac{1}{K_{bL}} + \frac{1}{K_{DA}} \frac{x}{L} \quad (10.14)$$

$K_{DA}$  in the above equation is the axial stiffness of the bridge deck defined as;

$$K_{DA} = \frac{EA_D}{L} \quad (10.15)$$

and  $A_D$  is the cross-sectional area of the deck. The above displacement function is normalized such that the displacement at the end of the bridge deck is unity. For this purpose, the function is divided by the longitudinal displacement of the bridge deck at  $x=L$ . This new function, as expressed below, is assumed to be the fundamental mode shape of the structure;

$$\Phi(\tau) = \frac{1}{K_{DA} + K_{bL}} \left( K_{DA} + K_{bL} \frac{x}{L} \right) \quad (10.16)$$

Substituting this shape function in Eq. 7.10 and integrating, the generalized mass of the system is obtained as;

$$m^* = \frac{m \left( (K_{bL} + K_{DA})^3 - K_{DA}^3 \right)}{3 K_{bL} (K_{bL} + K_{DA})^2} \quad (10.17)$$

The generalized axial stiffness of a system is defined by the following equation (Clough

and Penzien, 1975);

$$k^* = \int_0^L EA_D \left( \frac{d\Phi(x)}{dx} \right) dx \quad (10.18)$$

Substituting the mode shape function in the above equation, and using Eq. 10.15, the generalized stiffness is obtained as;

$$k^* = \frac{K_{bL} K_{DA}}{K_{bL} + K_{DA}} \quad (10.19)$$

Substituting  $k^*$  and  $m^*$  into Eq. 10.10, the fundamental period is defined as:

$$T_1 = 2\pi \sqrt{\frac{m \left( (K_{bL} + K_{DA})^3 - K_{DA}^3 \right)}{3 K_{bL}^2 K_{DA} (K_{bL} + K_{DA})}} \quad (10.20)$$

By substituting Eq. 10.15 into the above equation, the fundamental period can be obtained as;

$$T_1 = \sqrt{\frac{4 \pi^2 m L}{3 E A_D} \left[ 1 + \frac{EA_D}{L K_{bL}} \left( 3 - \frac{1}{\frac{EA_D}{L K_{bL}} + 1} \right) \right]} \quad (10.21)$$

For longitudinally rigid bearings (no deformation at the support), the fundamental period is defined as (Clough and Penzien, 1975);

$$T_1 = \sqrt{\frac{16 m L}{EA_D}} \quad (10.22)$$

For the simply supported bridges considered in this study, the ratio of the longitudinal fundamental periods obtained from computer analyses to the ones obtained from Eq. 10.21 is plotted in Figure 10.2 as a function of span length. As seen in the figure, the ratio is very close to 1.0 for all the span ranges.

### 10.1.2.2 Continuous Bridges

#### 10.1.2.2.1 Transverse Direction

Due to their relatively small stiffness, steel columns do not have a significant effect on the deformation of the bridge deck in the transverse direction. Therefore, the fundamental mode shape of continuous bridges is also represented by the trigonometric function expressed in Eq. 7.9. Using this function, the generalized mass, as expressed in Eq. 7.10, was obtained as half of the total mass of the structure, and the generalized stiffness is obtained from Eq. 10.3. The local translational spring in Eq. 10.3 are the ones that represent the transverse stiffnesses of the columns-sets,  $K_{cT_i}$  and the local rotational spring is the one that represent the rotational stiffness,  $K_{b\theta}$ , of the fixed-bearings-set located at the support. The number of local translational springs depends on the number of spans. For example, in 2, 3 and 4 span continuous bridges, the deck is supported respectively by 1, 2 and 3 columns-set, and therefore the same number of local translational springs are present. Accordingly, the generalized stiffness is represented as;

$$k^* = \frac{\pi^4 EI_D}{2 L_T^3} + \sum_{i=1}^{n_c} K_{cT_i} \left( \sin \frac{\pi x_i}{L_T} \right)^2 + K_{b\theta} \left( 1 - \frac{1}{\frac{3EI_D}{L_T K_{b\theta}} + 2} \right)^2 \frac{\pi^2}{L_T^2} \quad (10.23)$$

where  $n_c$  is the number of columns-set. Substituting Eqs. 7.10 and 10.23 into Eq. 10.10 and simplifying, the fundamental period is expressed as;

$$T_1 = \sqrt{\frac{2 m L_T^2}{\frac{\pi^2 EI_D}{2 L_T} + \frac{L_T^2}{\pi^2} \sum_{i=1}^{n_c} K_{cT_i} \left( \sin \frac{\pi x_i}{L_T} \right)^2 + K_{b\theta} \left( 1 - \frac{1}{\frac{3EI_D}{L_T K_{b\theta}} + 2} \right)^2}} \quad (10.24)$$

For example in a 3 span symmetric continuous bridge, there are 2 columns-sets located

at 1/3 and 2/3 of the total end-to-end length. In this case the fundamental period is:

$$T_1 = \sqrt{\frac{2 m L_T^2}{\frac{\pi^2 EI_D}{2 L_T} + 1.5 \frac{L_T^2}{\pi^2} K_{cT} + K_{bb} \left( 1 - \frac{1}{\frac{3EI_D}{L_T K_{bb}} + 2} \right)^2}} \quad (10.25)$$

Practically, the stiffness of the columns are negligible relative to the stiffness of the deck. Therefore, the contribution of the columns' stiffness in the above equations can be ignored. In that case Eq. 10.24 becomes identical to Eq.10.11. For the continuous bridges considered in this study, the ratio of the transverse fundamental periods obtained from computer analyses to the ones obtained from Eq. 10.24 is plotted in Figure 10.2 as a function of span length. As seen in the figure, the ratio is very close to 1.0 for all the span ranges.

#### 10.1.2.2.2 Longitudinal Direction

In the longitudinal direction, the steel columns do not have a significant effect on the deformation of the structure due to their negligibly small stiffness relative to the axial stiffness of the deck. Accordingly, to calculate the fundamental periods of continuous bridges in this direction, the total end-to-end length of the structure should be substituted in Eqs. 10.21 or 10.22 obtained for simply supported bridges.

### 10.1.2.3 Multi-Span Simply Supported Bridges

#### 10.1.2.3.1 Transverse Direction

In Chapter 9, analyses results showed that the transverse fundamental periods of multi-span simply supported bridges are close to those of two span simply supported bridges with bearings-set of zero rotational stiffness and identical individual span length. Accordingly, an analytical expression only for the fundamental periods of two span simply supported bridges with spans  $L_1$  and  $L_2$  will be derived here. Assuming that the decks are rigid, the fundamental mode shape of the structure is represented by the following linear function;

$$\phi(x) = \begin{cases} \frac{x}{L_1} & 0 \leq x \leq L_1 \\ 1 + \frac{L_1}{L_2} - \frac{x}{L_2} & L_1 < x \leq L_1 + L_2 \end{cases} \quad (10.26)$$

The above function is substituted in Eq. 7.10 and the generalized mass is obtained as;

$$m^* = \frac{m}{3} \quad (10.27)$$

In this system, there are one translational and two rotational local springs. The translational spring represents the transverse direction stiffness,  $K_{ct}$ , of the columns-set connected by fixed-bearings to the span with length  $L_2$  and the two rotational springs represent respectively the torsional stiffness,  $K_{ct\theta}$ , of the columns-set and the rotational stiffness of the fixed bearings-set supporting the span with length  $L_1$ . Substituting the

fundamental mode shape function and the stiffnesses of the local springs with their distances to the support in Eq. 10.3, the generalized stiffness is obtained as follows;

$$k^* = K_{cr} + \frac{K_{b\theta}}{L_1^2} + \frac{K_{c\theta}}{L_2^2} \quad (10.28)$$

It is noteworthy that since the deck is assumed rigid, the effect of in-plane support moment in reducing the rotation is not included in the above equation. Substituting Eq. 10.27 and the above equation in Eq. 10.10, the period of the structure is expressed as;

$$T_1 = \sqrt{\frac{4 \pi^2 m}{3 \left( K_{cr} + \frac{K_{b\theta}}{L_1^2} + \frac{K_{c\theta}}{L_2^2} \right)}} \quad (10.29)$$

For the two span simply supported bridges considered in this study, the ratio of the transverse fundamental periods obtained from computer analyses to the ones obtained from Eq. 10.29 is plotted in Figure 10.2 as a function of span length for fixed-bearings-sets with rotational stiffnesses of 16,000,000 and 0.0 kN·m. As seen in the figure, the ratio is very close to 1.0 when the rotational stiffness is 0.0, but it is around 1.15 when the rotational stiffness is 16,000,000 kN·m.

It is noteworthy that, to obtain the transverse fundamental periods of simply supported bridges with more than two spans, two adjacent spans with the largest average length should be selected if the column sizes and heights are identical at each bent. The rotational stiffness of the bearings-set should be neglected and only the total mass of these spans and stiffness of the columns supporting them should be used in the above equation.

However, if the columns' sizes and heights are not identical at each bent, then each pair of adjacent spans should be considered separately and their periods should be calculated. The largest of these is the transverse fundamental period of the whole system.

#### 10.1.2.3.2 Longitudinal Direction

In the longitudinal direction, the fundamental period of the abutment-fixed deck is identical to that of a simply supported bridge of the same span length. Therefore Eqs. 10.21 and 10.22 can be used to calculate its fundamental period. In the case of column-fixed deck, columns are the only elements that resist the inertial forces on the bridge deck. Therefore, it is appropriate to idealize that part of the structure as a SDOF system. Accordingly, the longitudinal direction fundamental period is defined as;

$$T_1 = 2\pi \sqrt{\frac{m_D}{K_{cL}}} \quad (10.30)$$

### 10.1.3 Calculation of Bearing Forces

Bearing forces are functions of geometric and structural properties of the bridge deck and stiffness of the bearings. Considering these parameters, analytical expressions for the forces in the bearings of the bridge types considered in this study will be derived in the following sub-sections.

#### 10.1.3.1 Single Span Simply Supported and Continuous Bridges

##### 10.1.3.1.1 Transverse Direction Excitation

Bearing forces produced by transverse direction seismic excitation are composed of two components. The first component is produced by the reaction force at the support and oriented in the transverse direction. The other component is produced by the in-plane support moment and oriented in the longitudinal direction. The resulting bearing force is the vectorial sum of these two forces.

For single span simply supported bridges, the transverse component of this bearing force,  $b_{ry}$  is expressed as:

$$b_{ry} = \frac{1}{n_b} \left( \frac{H}{2} + \frac{M_s}{L} \right) \quad (10.31)$$

where  $n_b$  is the number of bearings,  $H$  is the total seismic force acting on the structure expressed in Eq. 7.17, and  $M_s$  is the in-plane support moment expressed as follows:

$$M_s = \theta_s K_{b\theta} \quad (10.32)$$

Substituting Eq. 10.7 into the above equation, the in-plane support moment is obtained as;

$$M_s = \theta_{so} \left( 1 - \frac{1}{\frac{3EI}{L K_{b\theta}} + 2} \right) K_{b\theta} \quad (10.33)$$

The magnitude of  $\theta_{so}$  typically varies according to the severity of the earthquake excitation and physical properties of the bridge, i.e. the actual dynamic deformation response of the bridge. To obtain this actual deformation of the structure, the shape function of Eq. 7.9, which is normalized to have the midspan displacement equal to unity, must be multiplied by the midspan displacement,  $\Delta_{ms}$ , expressed as (Clough and Penzien, 1975);

$$\Delta_{ms} = \frac{g}{m^* \omega_1^2} S_a = \frac{g}{m^* \omega_1^2} \beta A_p \quad (10.34)$$

where,  $\omega_1$  is the fundamental circular frequency. Substituting Eqs. 7.10 and 7.16 into the



above equation and rewriting;

$$\Delta_{ms} = \frac{4 \beta A_p}{\pi \omega_1^2} \quad (10.35)$$

Then, the deformed shape of the structure is defined by the following function:

$$u_1 = \frac{4 \beta A_p}{\pi \omega_1^2} \sin \frac{\pi x}{L} \quad (10.36)$$

The derivative of the above function at  $x=0$  gives the support rotation  $\theta_{s0}$  as follows:

$$\theta_{s0} = \frac{4 \beta A_p}{L \omega_1^2} \quad (10.37)$$

Substituting the above expression into Eq. 10.33, the support moment is expressed as:

$$M_s = \frac{4 \beta A_p}{L \omega_1^2} \left( 1 - \frac{1}{\frac{3EI}{L K_{b\theta}} + 2} \right) K_{b\theta} \quad (10.38)$$

Substituting the above equation and Eq.7.17 into Eq. 10.31, the transverse component of the bearing force is expressed as;

$$b_{ry} = \frac{4 \beta A_p}{n_b} \left( \frac{m}{\pi^2} + \frac{K_{b\theta}}{(L \omega_1)^2} \left( 1 - \frac{1}{\frac{3EI}{L K_{b\theta}} + 2} \right) \right) \quad (10.39)$$

The longitudinal component of the bearing force,  $b_{rx}$ , due to seismic excitation in the

transverse direction is given by;

$$b_{xy} = \frac{M_s}{K_{b\theta}} l_b k_{bL} \quad (10.40)$$

where  $l_b$  is the distance of the bearing to the centerline of the bridge deck. Substituting Eq. 10.38 into the above equation,  $b_{xy}$  is expressed as;

$$b_{xy} = \frac{4 \beta A_p}{L \omega_1^2} \left( 1 - \frac{1}{\frac{3EI}{L K_{b\theta}} + 2} \right) l_b k_{bL} \quad (10.41)$$

The bearing force due to seismic excitation in the transverse direction is;

$$B_{ry} = \sqrt{b_{ry}^2 + b_{xy}^2} \quad (10.42)$$

For continuous bridges, as demonstrated in Chapter 8, the stiffness of the columns relative to the stiffness of the deck is very small, therefore they have no significant effect on the forces attracted by the bearings. Accordingly, the equations derived for single span simply supported bridges can be used to obtain the bearing forces in continuous bridges. However, the span length,  $L$ , in these equations should be replaced by the total end-to-end length,  $L_T$ , of the continuous bridge.

For single span simply supported and continuous bridges considered in this study, the ratios of the transverse bearing forces obtained from computer analyses to the ones obtained from the derived equations are plotted in Figure 10.3 as a function of span length. As seen in the figure, the ratios are very close to 1.0 for all the span ranges.

### 10.1.3.1.2 Longitudinal Direction

For single span simply supported bridges, to obtain the bearing force,  $b_{rx}$ , in the longitudinal direction, first the fundamental mode shape function defined in Eq. 10.16 is substituted in Eq. 7.16 to obtain the modal earthquake-excitation factor,  $\xi$  as;

$$\xi = \frac{m \left( K_{DA} + \frac{K_{bL}}{2} \right)}{K_{DA} + K_{bL}} \quad (10.43)$$

Then, substituting the above equation and Eq. 10.17 into Eq. 7.15 and arranging, the total seismic force acting on the structure is expressed as;

$$H = \frac{\frac{K_{bL}}{K_{DA}} + \frac{K_{bL}^2}{2K_{DA}^2}}{\left( \frac{K_{bL}}{K_{DA}} + 1 \right)^3 - 1} 3 m \beta A_p \quad (10.44)$$

Substituting Eq. 10.15 into the above equation and dividing by the number of bearings, the bearing forces due to longitudinal direction seismic excitation is obtained as;

$$b_{rx} = \left[ \frac{3 \left( 2 + \frac{L K_{bL}}{EA_D} \right)^2}{\left( 3 + 2 \frac{L K_{bL}}{EA_D} \right)^2 + 3} \right] \frac{m \beta A_p}{n_b} \quad (10.45)$$

For bridges with rigid bearings, the following shape function given for the axial fundamental mode of a beam element is used (Clough and Penzien, 1975):

$$\Phi(x) = \sin \frac{\pi x}{2L} \quad (10.46)$$

Substituting this shape function into Eqs. 7.10 and 7.16 the generalized mass and earthquake excitation factor are first obtained. Then, the results are substituted in Eq. 7.15 and the total force acting on the structure is determined. This force is found to be identical to the one expressed in Eq. 7.17. Dividing this force by the number of bearings, the bearing forces due to longitudinal direction seismic excitation is obtained as follows:

$$b_{rx} = \frac{8 m \beta A_p}{n_b \pi^2} \quad (10.47)$$

For continuous bridges, the longitudinal stiffness of the columns are negligible compared to the stiffness of the bearings. Therefore they have no significant effect on the forces attracted by the bearings. Accordingly, the equations derived for single span simply supported bridges can be used to obtain the longitudinal bearing forces in continuous bridges. However, the span length  $L$  in these equations should be replaced by the total end-to-end length,  $L_T$ , of the continuous bridge.

For single span simply supported and continuous bridges considered in this study, the ratios of the longitudinal bearing forces obtained from computer analyses to the ones obtained from the derived equations are plotted in Figure 10.3 as a function of span length. As seen in the figure, the ratios are very close to 1.0 for all the span ranges.

### 10.1.3.2 Multi-Span Simply Supported Bridges

Due to non-linearities resulting from the collision of adjacent superstructure components in the longitudinal direction, simple analytical expressions for the bearing forces cannot be obtained. However, it is generally accepted that impact may produce high forces in the bearings and damage them.

### 10.1.4 Calculation of Column moments

In the following sub-sections analytical expressions for the seismic moments in the columns of continuous and multi-span simply supported bridges will be derived.

#### 10.1.4.1 Continuous Bridges

##### 10.1.4.1.1 Transverse Direction - Linear Elastic Response

In Chapter 8 it was demonstrated that the columns do not have a significant effect on the displacement of the bridge deck. Therefore, the deformation of continuous bridges can be assumed to be only a function of the deck stiffness and is expressed by Eq. 10.36. The first order column seismic moment,  $M_{Ey0}$ , in the transverse direction is obtained by multiplying the displacement,  $\Delta_c$ , of the deck at the column location, by the stiffness and height of the column;

$$M_{Ey0} = \Delta_c k_{cT} h_c \quad (10.48)$$

The distance,  $x_c$ , of the columns-set measured from the abutment is substituted in

Eq. 10.36 to obtain  $\Delta_c$  as:

$$\Delta_c = \frac{4 \beta A_p}{\pi \omega_1^2} \sin\left(\frac{\pi x_c}{L_T}\right) \quad (10.49)$$

Substituting the above equation into Eq. 10.48, the first order transverse column seismic moment is expressed as follows;

$$M_{Ey0} = \frac{4 \beta A_p}{\pi \omega_1^2} \sin\left(\frac{\pi x_c}{L_T}\right) k_{cT} h_c \quad (10.50)$$

As previously stated, additional deformation of the structure produced by second order effects is negligibly small due to the high in-plane stiffness of the deck. Therefore, a single iteration cycle is adequate to determine the second order moment. Knowing this and neglecting the effect of seismic axial force (which is small compared to the axial force due to dead load of the structure) the magnified transverse direction seismic moment is obtained as;

$$M_{Ey} = M_{Ey0} + \Delta_c P_D \quad (10.51)$$

Substituting Eqs. 10.49 and 10.50 into the above equation, the magnified transverse column seismic moment (i.e. including second-order effects) is expressed as follows;

$$M_{Ey} = \frac{4 \beta A_p}{\pi \omega_1^2} \sin\left(\frac{\pi x_c}{L_T}\right) (k_{cT} h_c + P_D) \quad (10.52)$$

For the continuous bridges considered in this study, the ratios of the first order transverse column moments obtained from computer analyses to the ones obtained from Eq.10.50 are plotted in Figure 10.4 as a function of span length. As seen in the figure, the ratios are very close to 1.0 for all the span ranges. A similar comparison of second-order moments is not possible.

#### 10.1.4.1.2 Transverse Direction - Non-linear Inelastic Response

If the bearings are damaged, large sliding displacements may be produced in the transverse direction. Should that happen, at the columns' location, the sliding displacement,  $u_s$ , must also be added to the displacement,  $u_{ec}$ , due to the elastic deformation of the structure. Accordingly, the total displacement at the columns location is expressed as:

$$\Delta_c = u_s + u_{ec} \quad (10.53)$$

The magnitude of the elastic deformation of the structure is controlled by the friction resistance at the supports. To obtain the elastic displacement at the columns' location, first,  $S_a$  or  $\beta A_p$  in Eq. 7.17 is expressed as:

$$\beta A_p = \frac{H \pi^2}{8 m} \quad (10.54)$$

The force,  $H$ , acting on the structure is equal to the friction resistance,  $F_s$ , when the structure is sliding. Knowing this and substituting the above equation into Eq. 10.36, the elastic deformation of the structure at the column location is defined as:

$$u_{ec} = \frac{\pi F_s}{2 m \omega_1^2} \sin \frac{\pi x_c}{L} \quad (10.55)$$

Substituting the above equation in Eq. 10.53, the total displacement at the column location

is expressed as:

$$\Delta_c = u_s + \frac{\pi F_s}{2 m \omega_1^2} \sin \frac{\pi x_c}{L} \quad (10.56)$$

The above equation is substituted in Eq. 10.48 to obtain the first order transverse seismic column moment as;

$$M_{Ey0} = \left( u_s + \frac{\pi F_s}{2 m \omega_1^2} \sin \frac{\pi x_c}{L} \right) k_{cT} h_c \quad (10.57)$$

Substituting the above equation and Eq. 10.56 in Eq. 10.51, the magnified transverse seismic column moment is approximately obtained as:

$$M_{Ey} = \left( u_s + \frac{\pi F_s}{2 m \omega_1^2} \sin \frac{\pi x_c}{L} \right) (k_{cT} h_c + P_D) \quad (10.58)$$

All terms in the above equation can be calculated prior to analysis, except  $u_s$  which can be read from figures as will be shown later.

#### 10.1.4.1.3 Longitudinal Direction

In the longitudinal direction, the column moments are negligible due to the high axial stiffness of the structure which prevents large longitudinal displacements at the column location. However, if the bearings are damaged, large sliding displacements may be produced. Since the elastic displacement due to the axial deformation of the structure is negligible, the total displacement at the column location is equal to the sliding displacement of the structure. Neglecting the deformation of the abutments, this sliding



displacement can not be larger than the expansion joint width. Knowing this, the first order longitudinal seismic moment is expressed as:

$$M_{Ex0} = u_s k_{cL} h_c \leq 0.00084 L_T k_{cL} h_c \quad (10.59)$$

where the term  $0.00084L_T$  is the expansion joint width obtained by conservatively assuming a temperature difference of  $70\text{ C}^\circ$ . The second order moment is approximately determined by multiplying the sliding displacement by the axial force in the column due to dead load. Then the magnified longitudinal seismic column moment is obtained by adding this second order moment to the above equation, and is expressed as follows;

$$M_{Ex0} = u_s ( k_{cL} h_c + P_D ) \leq 0.00084 L_T ( k_{cL} h_c + P_D ) \quad (10.60)$$

#### 10.1.4.2 Multi-Span Simply Supported Bridges

##### 10.1.4.2.1 Transverse Direction

In Chapter 9 it was demonstrated that the maximum of the seismic moments in the columns of multi-span simply supported bridges are close to the ones in columns of two span simply supported bridges with bearings-set of zero rotational stiffness and identical individual span length. During the analyses, it was observed that, the displacement at the columns location of two span simply supported bridges can accurately be obtained considering only the contribution of the first mode. Accordingly, the fundamental mode shape expressed in Eq. 10.26 is substituted into Eq. 7.16 to obtain the

earthquake excitation factor as:

$$\xi = \frac{m}{2} \quad (10.61)$$

Then, the above equation and Eq. 10.27 are substituted in Eq. 10.34 to obtain the displacement at the columns locations as:

$$\Delta_c = \frac{3 \beta A_p}{2 \omega_1^2} \quad (10.62)$$

The magnified transverse seismic moment is then obtained by multiplying the displacement at the column location respectively by the column stiffness, column height and the magnification factor expressed by Eq. 9.42.

$$M_{Ey} = \frac{3 \beta A_p}{2 \omega_1^2} \beta_{my} k_{cT} h_c \quad (10.63)$$

To calculate the longitudinal direction seismic moments of the columns due to transverse direction seismic excitation, first the deck rotation is obtained by dividing the transverse displacement at the columns' location by the length of the span where the columns are attached. The magnified longitudinal seismic moment of any column,  $i$ , is then obtained by multiplying this rotation respectively by the distance of the column,  $d_{ci}$ , to the centerline of the bridge, the column longitudinal stiffness and height as well as the magnification factor expressed in Eq. 9.43. Accordingly, the longitudinal column seismic moment is expressed as;

$$M_{Exyi} = \frac{3 \beta A_p}{2 L_2 \omega_1^2} \beta_{mxy} k_{cL} h_c d_{ci} \quad (10.64)$$

To obtain the maximum transverse seismic moments in the columns of simply supported bridges with more than two spans, the properties of the two adjacent spans with the largest average length and the columns supporting them can simply be used in the above equations if the sizes and the heights of the columns at each bent are the same. Otherwise, each pair of adjacent spans should be checked separately to determine the most critical columns. Note that, the rotation of the decks in simply supported bridges with more than two spans may not be as much as those in two span bridges. Therefore, the above equation may give conservative results in some cases. Higher modes are observed to more dominantly affect the response when a very large number of spans are present.

If collision between the decks occurs at the expansion joint due to relative rotation of the adjacent spans, then the above equations may not give correct results. However, as demonstrated earlier, in most of the cases, steel columns are likely to be severely damaged before impact takes place. Therefore the derived equations can generally be used to predict the columns' seismic moments.

#### **10.1.4.2.2 Longitudinal Direction**

In the longitudinal direction, due to non-linearities resulting from the collision of adjacent superstructure components, the response of the bridge is very complex. Assuming that the bearings are not damaged due to the effect of impact, displacements as large as the sum of the expansion joint widths at one or the other side of the column under consideration may possibly be obtained as demonstrated earlier. Based on that

knowledge, the maximum possible longitudinal seismic moment for the  $k^{th}$  column is defined by the following equation:

$$M_{Ex} = \text{Max} \left\{ \sum_{i=1}^k EJW_i, \sum_{i=k+1}^{n_d} EJW_i \right\} (k_{cL} h_c + P_D) \quad (10.65)$$

## 10.2 Seismic Screening of Slab-On-Girder Steel Bridges

It has become apparent in recent years that many existing bridges are inadequate to resist seismic loads. Therefore, to avoid earthquake related failures in the future, an effort must be made to identify seismically deficient bridges. To identify these bridges, the engineer needs to determine the physical state of the bridge based on engineering drawings and field inspection. Additionally, the consequences of failure on the economic, social and administrative aspects should be defined. Having these information, the four steps described below must be followed to determine which bridges must be most urgently retrofitted.

- i) Bridges should be ranked according to their vulnerability to seismic excitations considering the seismicity of the site, type of bridge, capacity of individual components and the impact of each component's failure on the potential total collapse of the structure.
- ii) The level of importance of the bridges to the given locality should be determined considering the type of highway, traffic volume, accessibility of other crossings etc.. Additionally, the cost and applicability of retrofitting should also be considered. For example, bridges with a high ratio of repair or replacement cost to seismic retrofit cost generally receive a high priority for retrofit.

- iii) The selection of bridges to be retrofitted based on some judgemental weighing of the above two criteria.
- iv) Determination and design of the retrofit measures.

To rank the bridges according to their vulnerability, engineers need to determine the response of the bridges when subjected to the probable seismic excitations occurring at the site. Determining the seismic response of each specific bridge by complete structural analysis is a long and tedious process. Therefore a methodology is required to rapidly find the seismically deficient bridges and rank them in terms of their respective vulnerability. While numerous rapid ranking methodologies exist (e.g. Seismic ..., 1983; CALTRANS, 1992; Filiatrault et al., 1993) the bridge vulnerability component of these methodologies is coarse and often limited to simple recognition of undesirable structural features known to have performed inadequately in past earthquakes. Bridges sharing such features would also share the same rating independent of variations in geometry and other structural properties.

To improve on this situation, and considering the first two steps described above, a two variable system is proposed to better define and identify the most important and vulnerable steel bridges. These variables are the importance index,  $I_i$ , and the overall damage index,  $I_d$ , of the bridge. A ranking index,  $I_R$ , is defined to prioritize the bridges based on the interaction between the importance and vulnerability of the structure. These indices are defined in the following sub-sections.

### 10.2.1 Importance Index

The importance index is a factor that varies between 0 and 1, and defines the impact of damage on the social, economic and practical aspects of the problem. The importance of the bridge is evaluated by considering the effects associated with the loss

of the bridge with regard to the highway system and the local community, the ability to provide emergency services, the national security/defense network, and overall post-earthquake recovery in the affected area. The ratio of repair or replacement cost to seismic retrofit cost is also an additional aspect to be considered in determining the importance index.

The procedure followed to classify the bridges according to their importance is well established (Caltrans, 1992) and need not be modified. In summary, bridges with importance index of 0 are those for which all consequences of failure are acceptable, whereas, bridges with importance index of 1 are those whose loss is unacceptable; bridges in the latter case would urgently need to be retrofitted if deficient.

#### 10.2.2 Damage Index

Bridge components that could potentially be damaged during an earthquake should be evaluated to determine their ability to resist earthquakes likely to occur at the site with a pre-selected probability of exceedance, and called the site earthquake thereafter. This is done by calculating a damage index expressing the demand/capacity ratio of these components. Values of component damage index less than one indicate that the corresponding component is unlikely to fail during the site earthquake, whereas values greater than one indicate a possible failure. The overall damage index,  $I_d$ , of the structure is a function of the damage indices of different components and therefore varies depending on the bridge type. For the types of bridges studied, three types of component damage indices are considered. These are seat width index,  $I_{sw}$ , bearing damage index,  $I_{bd}$  and column damage index,  $I_{cd}$ . These indices will be described in detail in the subsequent sections. Component damage indices for the foundations and abutments are not considered since they are beyond the scope of this study.

The seismic demands for the bridge components are determined from the equations derived in this study. Seismic capacities are calculated at their nominal ultimate values without capacity reduction factors. Minimum requirements for certain bridge components are also specified in the previous chapters. In certain cases the demand/capacity ratios will be calculated using these minimum requirements as demands. The overall damage index of the structure will be determined considering the impact of damage to each component on the successive failure of other components and the structure. The procedure to obtain the overall damage index for the types of bridges considered in this study will be described in detail in Section 10.2.2.4.

### **10.2.2.1 Seat Width Index**

#### **10.2.2.1.1 Single Span Simply Supported and Continuous Bridges**

For single span simply supported and continuous bridges, seat width index is defined in both transverse and longitudinal directions. In the transverse direction it is defined as;

$$I_{swT} = \frac{u_s + 50}{SWT} \quad (10.66)$$

where  $u_s$  is the transverse sliding displacement of the structure (mm) obtained from Figure 10.5 using the site peak ground acceleration and  $SWT$  is the seat width (mm) measured in the transverse direction from the edge of the exterior bearing to the edge of the abutment. An additional length of 50 mm is provided in the above equation considering that damage may occur to the unreinforced part of the abutment when the deck slides. It is a judgemental value which may be increased for additional safety or changed to more accurately reflect the detailing particulars of bridges in other parts of the world.

In the longitudinal direction, the movement of the deck is restricted by the abutment walls at both ends. Therefore, neglecting the deformations of the abutments, the sliding displacement of the bridge can not be larger than the expansion joint width. Accordingly, the seat width index in the longitudinal direction is defined by the following equation:

$$I_{swL} = \frac{u_s + 50}{SWL} \leq \frac{0.84 L_T + 50}{SWL} \quad (10.67)$$

where  $SWL$  is the seat width (mm) measured in the longitudinal direction from the centerline of the bearings to the edge of the abutment. The term  $0.84L_T$  in the above equation is the expansion joint width in mm obtained conservatively assuming a temperature difference of  $70^\circ\text{C}$ .

To obtain the sliding displacement, the information in Figures 7.19 and 7.20 is merged and presented in a different form in Figure 10.5. In that figure, the vertical axis is the sliding displacement normalized by peak ground acceleration,  $A_p$ , expressed as a fraction of gravitational acceleration and, the horizontal axis is the fundamental period. However, instead of the previously defined  $\mu_f/A_p$  ratio, a new dimensionless ratio,  $\psi$  is used in this figure. It is defined as the ratio of the sliding resistance,  $F_s$ , of the structure to the effective modal mass,  $M_{eff}$  times the peak ground acceleration;

$$\psi = \frac{F_s}{M_{eff} A_p g} \quad (10.68)$$



where,

$$M_{eff} = \frac{\mathcal{Q}^2}{m^*} \quad (10.69)$$

$$F_s = w_f \mu_f m g \quad (10.70)$$

In the above equation  $w_f$  is the percentage of the weight of the structure transferred to all supports where there is friction resistance. Using Eqs. 10.69 and 10.70, Eq. 10.68 is expressed as:

$$\psi = \frac{w_f \mu_f m m^*}{\mathcal{Q}^2 A_p} \quad (10.71)$$

The generalized mass,  $m^*$  and the earthquake excitation factor,  $\mathcal{Q}$  are obtained from Eqs. 7.10 and 7.16, and substituted in the above equation to obtain  $\psi$  for single span simply supported and continuous bridges as;

$$\psi = \frac{8 w_f \mu_f}{\pi^2 A_p} \quad (10.72)$$

Figure 10.5 is obviously useful only if the peak acceleration at the site is larger than the minimum peak ground acceleration required for sliding. For single span simply supported and continuous bridges, using Eqs 7.17 and 10.70, this minimum required peak ground acceleration for sliding, as a fraction of gravitational acceleration, for both transverse and longitudinal direction of earthquake excitations is expressed as;

$$A_p = \frac{\pi^2}{8\beta} w_f \mu_f \quad (10.73)$$

For single span simply supported bridges the total weight of the structure is transferred to the abutments. Therefore,  $w_f$  is 1.0 for the transverse direction sliding, but it is only

0.5 for the longitudinal direction sliding since the bridge is designed to expand freely at one end in this direction. For continuous bridges, some portion of the weight is transferred to the columns and therefore  $w_f$  is a function of the number of column bents in the structure. However, it can be calculated approximately by considering the tributary weight transferred to the supports where sliding can occur.

The following steps describe how to obtain the seat width index for single span simply supported and continuous bridges:

- 1) Using Eqs. 10.11 and 10.22 for single span simply supported, and Eqs. 10.24 and 10.22 for continuous bridges, calculate the fundamental periods in the transverse and longitudinal directions assuming zero rotational stiffness for the bearings-set.
- 2) Using the transverse and longitudinal direction fundamental periods of the structure, obtain  $\beta$  in both orthogonal directions from a design spectrum or a spectrum developed specifically for the site under consideration. Also obtain the site peak ground acceleration,  $A_{ps}$ , as a fraction of gravitational acceleration from a seismic map or site-specific information.
- 3) Considering the condition of the abutments and type of bearings, specify a friction coefficient for the bearings at the abutments.
- 4) Using Eq. 10.73, obtain  $A_p$  required for sliding in both orthogonal directions. If  $A_p > A_{ps}$  then  $u_s$  is 0, otherwise calculate  $\psi$  using Eq. 10.72 and obtain  $u_s$  from Figure 10.5. Substitute  $u_s$  obtained for transverse and longitudinal directions respectively in Eqs. 10.66 and 10.67 to obtain the seat width indices in both orthogonal directions. The larger of these two is the seat width index,  $I_{sw}$ , of the structure.

#### 10.2.2.1.2 Multi-Span Simply Supported Bridges

For multi-span simply supported bridges only the seat width index in the longitudinal direction is considered since sliding is not critical in the transverse direction. The seat width index is defined by the following equation;

$$I_{sw} = \left( \frac{SW_i}{SWL_i} \right)_{\max} \quad (10.74)$$

In the above equation,  $SW_i$  is the minimum required seat width (mm) defined by Eqs. 9.59 and 9.67 ( or 9.76 in the rare cases where it produces larger results than 9.67) and  $SWL_i$  is the width (mm) of the bearing seat supporting the unrestrained expansion ends of girders and measured from the centerline of the bearing to the support edge at expansion joint  $i$ . The ratio of  $SW_i$  to  $SWL_i$  is calculated for each expansion joint and the maximum of these defines the seat width index for multi-span simply supported bridges.

#### 10.2.2.2 Bearing Damage Index

##### 10.2.2.2.1 Single Span Simply Supported and Continuous Bridges

Bearing damage index for simply supported and continuous bridges is defined as the ratio of the bearing force demand,  $B_r$ , to the bearing capacity,  $B_{rc}$ , and is expressed as follows;

$$I_{bd} = \frac{B_r}{B_{rc}} \quad (10.75)$$

The bearing force demand is defined by the following equation;

$$B_r = \sqrt{(C_1 b_{ry})^2 + (C_1 b_{rx} + C_2 b_{rx})^2} \quad (10.76)$$

In the above equation  $b_{rv}$  and  $b_{rv}$  are respectively the bearing forces in the transverse and longitudinal direction due to seismic excitation in the transverse direction and  $b_{rv}$  is the bearing force in the longitudinal direction due to seismic excitation in the longitudinal direction.  $C_1$  and  $C_2$  are correlation factors to account for the simultaneous occurrence of seismic excitations in both orthogonal directions and the directional uncertainty of the earthquake motions. In this study, to account for bi-directional seismic excitations, the same two load cases proposed by the 1987 edition of Seismic Design and Retrofit Manual For Highway bridges, are considered. In the first load case 100 percent (  $C_1 = 1.0$  ) of the forces from the analysis in the transverse direction is added to 30 percent (  $C_2 = 0.3$  ) of the forces from the analysis in the longitudinal direction. In the second load case 30 percent (  $C_1 = 0.3$  ) of the forces from the analysis in the transverse direction is added to 100 percent (  $C_2 = 1.0$  ) of the forces from the analysis in the longitudinal direction. The largest of the results from these two load combinations is used to determine the bearing damage index.

The capacity of traditional type of bearings is assumed to be governed by the shear capacity of the anchor bolts. Accordingly, the bearing capacity is defined by the following equation;

$$B_{rc} = n_{ab} A_{ab} \tau_y \quad (10.77)$$

where  $\tau_y$ ,  $A_{ab}$ , and  $n_{ab}$  are respectively the shear strength, area and number of anchor bolts. However, other local failure modes should not be overlooked if probable.

The following steps describe how to obtain the bearing damage index for single span simply supported and continuous bridges;

- 1) Calculate the translational and rotational stiffness of the bearings-set using Eqs. 7.1 to 7.8.

- 2) Using Eqs. 10.11 and 10.21 for single span simply supported, and Eqs. 10.24 and 10.21 for continuous bridges, calculate the fundamental periods in the transverse and longitudinal directions.
- 3) Using the transverse and longitudinal direction fundamental periods of the structure, obtain  $\beta$  in both orthogonal directions from a design spectrum or a spectrum developed specifically for the site under consideration. Also obtain the site peak ground acceleration,  $A_{ps}$ , as a fraction of gravitational acceleration from a seismic map or site-specific information.
- 4) Obtain the bearing forces due to seismic excitations in both orthogonal directions using Eqs. 10.39, 10.41 and 10.45. Substitute these forces in Eq. 10.76 to obtain the bearing force demand. Determine the capacity of the bearings using Eq. 10.77. The bearing damage index of the structure is the bearing force demand divided by the capacity.

#### **10.2.2.1.2 Multi-Span Simply Supported Bridges**

In the case of multi-span simply supported bridges, the loss of support due to damage to bearings has been responsible for several bridge failures in the past. Impacting between the two adjacent sections of a bridge superstructure can cause high shear forces and damage in the bearings. Conservatively, the capacity of the bearings is limited by the peak ground acceleration,  $A_{pc}$ , required for collision of the column-fixed decks. This peak ground acceleration is obtained from Eq. 9.54. The bearing damage index for multi-

span simply supported bridges as expressed below is then defined as the ratio of site peak ground acceleration to the peak ground acceleration required for collision:

$$I_{bd} = \frac{A_{ps}}{A_{pc}} \quad (10.78)$$

The following steps describe how to obtain the bearing damage index for multi-span simply supported bridges;

- 1) Choose the column-fixed deck adjacent to the narrowest expansion joint. If the expansion joint widths are identical choose the deck with the longest span and calculate its mass. Then, obtain the axial force due to dead load in the supporting columns using their tributary area.
- 2) Calculate the longitudinal direction stiffness,  $k_{cL}$ , of the columns using Eq. 9.5. The stiffness of old type bearings is usually very high compared to the stiffness of the columns, therefore the stiffness of column-bearing assembly can be calculated neglecting the term for bearing stiffness in Eq. 9.5. Then, obtain the stiffness,  $K_{cL}$ , of the column-set by summing up the column stiffnesses.
- 3) Using Eq. 10.30 calculate the period of the column-fixed deck. Using this period obtain  $\beta$  from a design spectrum or a spectrum developed specifically for the site under consideration. Also obtain the site peak ground acceleration,  $A_{ps}$ , as a fraction of gravitational acceleration from a seismic map or site-specific information.
- 4) Using Eq. 9.54, obtain the minimum required peak ground acceleration for collision to occur. Divide the peak ground acceleration of the site by this acceleration to obtain the bearing damage index for multi-span simply supported bridges.

### 10.2.2.3 Column Damage Index

The shear forces in steel columns are generally negligible compared to their shear capacities. Accordingly, only the effect of flexural forces is considered and the column damage index is obtained using conservatively a linear biaxial moment interaction relationship. In the following equation, the column damage index is defined as the sum of the ratios of seismic moment demands of the columns to their flexural capacities in both orthogonal directions:

$$I_{cd} = \frac{C_1 M_{Ey} + C_2 M_{Eyx}}{M_{ay}} + \frac{C_1 M_{Exy} + C_2 M_{Ex}}{M_{ax}} \quad (10.79)$$

In the above equation,  $M_{ay}$  and  $M_{ax}$  are the transverse and longitudinal flexural capacity of the column obtained from Eqs. 8.2 and 8.6 under axial force due only to the dead load of the structure. For practical purposes, this axial force can be obtained from the tributary area of the column.  $M_{Ey}$  and  $M_{Eyx}$  are the transverse seismic moment demands due to seismic excitation in the transverse and longitudinal directions respectively.  $M_{Eyx}$  has not been used in this study but is included above to make the equation more general.  $M_{Ex}$  and  $M_{Exy}$  are the longitudinal seismic moment demands due to seismic excitation in the longitudinal and transverse directions respectively. The largest of the column damage indices resulting from the two same load cases considered to account for bi-directional earthquake excitation ( e.i.  $C_1 = 1.0, C_2 = 0.3$  and  $C_1 = 0.3, C_2 = 1.0$  ) is selected.

#### 10.2.2.3.1 Continuous Bridges

For continuous bridges, the seismic moments in the columns are determined by the displacement of the deck at the columns' location. In the transverse direction, the columns closer to the midspan have larger displacement hence larger seismic moments. Therefore, the damage index should be calculated for these columns if the bridge has identical column sizes and heights at each bent, otherwise each columns-set should be

checked separately. In the longitudinal direction, the stiffness of the deck is relatively high and therefore the deformations and column seismic moments are negligible. Accordingly, the column damage index is calculated considering only the transverse direction response. However, if the bearings are damaged, the deck may slide and seismic moments may be produced in both transverse and longitudinal directions.

Considering the condition of the bearings (damaged or undamaged), the following steps are listed to describe how to obtain the column damage index for continuous bridges:

A) Bearings are undamaged :

A-1) Obtain the transverse stiffness and flexural capacity of the columns from Eqs. 9.4 and 8.2.

A-2) Knowing the transverse direction fundamental period,  $\beta$  and  $A_{ps}$ , obtain the transverse seismic moment demand,  $M_{Ey}$ , from Eq. 10.52. For the case at hand,  $M_{Exy}$ ,  $M_{Ex}$  and  $M_{Eyx}$  are zero. Using Eq. 10.79 with  $C_1 = 1.0$ ,  $C_2 = 0.3$ , calculate the column damage index.

B) Bearings are damaged:

B-1) Using Eqs. 10.24 and 10.22, calculate the fundamental periods in the transverse and longitudinal directions assuming zero rotational stiffness for the bearings-set. Using these periods, obtain  $\beta$  in both orthogonal directions from a design spectrum or a spectrum developed specifically for the site under consideration. Also obtain the site peak ground acceleration,  $A_{ps}$ , as a fraction of gravitational acceleration from a seismic map or a site-specific information.



- B-2) Considering the condition of the abutments and type of bearings, specify a friction coefficient for the bearings at the abutments.
- B-3) Using Eq. 10.73, obtain  $A_p$  required for sliding in both orthogonal directions. Depending on the magnitude of  $A_p$ , four cases arise:
- (i) If  $A_p > A_{ps}$  in both orthogonal directions then  $u_x$  is 0. Follow steps A-1, A-2 to calculate the column damage index
  - (ii) If  $A_p > A_{ps}$  in the transverse direction and  $A_p < A_{ps}$  in the longitudinal direction, use Eqs. 10.52 and 8.2 to calculate the seismic moment demand and flexural capacity of the column in the transverse direction. For the longitudinal direction calculate  $\psi$  using Eq. 10.72 and obtain  $u_x$  from Figure 10.5. Substitute  $u_x$  in Eq. 10.60 to obtain the longitudinal seismic moment demands. Use Eq. 8.6 to obtain the flexural capacity of the columns in the longitudinal direction. Substitute the obtained seismic moment demands and capacities in both orthogonal directions in Eq. 10.79 to obtain the damage indices for the two load cases. The larger of the results from the two load cases is the column damage index.
  - (iii) If  $A_p < A_{ps}$  in the transverse direction and  $A_p > A_{ps}$  in the longitudinal direction, ignore the response in the longitudinal direction and calculate  $\psi$  using Eq. 10.72 and obtain  $u_x$  from Figure 10.5 for the transverse direction. Substitute  $u_x$  in Eq. 10.58 to obtain the transverse seismic moment demands. Then, obtain the flexural capacity of the columns in the transverse direction from Eq. 8.2. Use Eq. 10.79 with  $C_1 = 1.0$ ,  $C_2 = 0.3$  to obtain the column damage index.

- (iv) If  $A_p < A_{ps}$  in both orthogonal directions, calculate  $\psi$  in both directions using Eq. 10.72. Substitute  $u_x$  obtained for transverse and longitudinal directions respectively in Eqs. 10.58 and 10.60 to obtain the transverse and longitudinal seismic moment demands. Use Eqs. 8.2 and 8.6 to obtain the flexural capacities of the columns in both orthogonal directions. Substitute the obtained seismic moment demands and capacities in both orthogonal directions in Eq. 10.79 to obtain the damage indices for the two load cases. The larger of the results from the two load cases is the column damage index.

#### 10.2.2.3.2 Multi-Span Simply Supported Bridges

When a multi-span simply supported bridge is subjected to seismic excitation in the transverse direction, the exterior columns of any given columns-set suffer more damage due to their higher longitudinal seismic moments resulting from the rigid body rotation of the column-fixed deck. Therefore, the exterior columns are the most vulnerable ones and should be considered in the calculation of column damage index. If the bridge has identical column sizes and heights at each bent, two adjacent spans with the largest average length and the columns supporting them are selected, otherwise the columns at each bent should be considered separately and the damage index is the largest of the ones calculated for each bent. The following steps describe how to obtain the column damage index ;

- 1) Calculate the total mass of the selected two adjacent spans and transverse and longitudinal direction stiffness of the columns supporting them using Eqs. 9.4 and 9.5.
- 2) Obtain the transverse fundamental period of the structure from Eq 10.29. Note that  $K_{ps}$  in Eq. 10.29 is neglected for bridges with number of spans larger than

two. Using this period, obtain  $\beta$  from a design spectrum or a spectrum developed specifically for the site under consideration. Also obtain the site peak ground acceleration,  $A_{pg}$ , as a fraction of gravitational acceleration from a seismic map or a site-specific information.

- 3) Use Eqs. 10.63, 10.64 and 10.65 to calculate the seismic moment demands in both orthogonal directions. Obtain the flexural capacities of the columns in the transverse and longitudinal directions using Eqs. 8.2 and 8.6. Substitute the obtained seismic moment demands and capacities in both orthogonal directions in Eq. 10.79 to obtain the damage indices for the two load cases. The larger of the results from the two load cases is the column damage index.

#### **10.2.2.4 Overall Damage Index of the Structure**

Knowing the seat width, bearing damage and column damage indices, the overall damage index,  $I_d$ , of the structure will be defined in the following sub-sections considering the impact of damage to each component on the successive failure of other components and the structure.

##### **10.2.2.4.1 Single Span Simply Supported Bridges**

Bearings are the most vulnerable superstructure components in the case of single span simply supported bridges. Fortunately, damage to these components does not necessarily result in failure of the structure. However its consequences should be estimated. Accordingly, the overall damage index,  $I_d$  of the structure is defined

considering two possible cases;

- i) If the bearings are not damaged (  $I_{bd} < 1.0$  ), the damage index of the structure is defined as the smaller of the bearing damage or the seat width indices.
- ii) If the bearings are damaged (  $I_{bd} > 1.0$  ) the damage index of the structure is defined only by the seat width index of the structure.

The first of the above two cases is explained by the following example. Consider two bridges, A and B with identical bearing damage indices of 0.7 but seat width indices of 1.4 and 0.5 respectively. In the case of bridge A, since the bearings are not likely to be severed, the structure can not slide. Therefore, the fact that the seat width index is larger than 1.0 does not pose any danger. Accordingly, the bearing damage index is selected as the overall damage index for bridge A. In the case of bridge B, the seat width index is smaller than the bearing damage index. This means that even if the bearings are severed the structure can not fall off its support and get damaged. Consequently, the seat width index is selected as the overall damage index of the structure. If the bearings are severed, the structure, within the assumptions of this study, can only get damaged if it falls off its support. Therefore, the seat width index is always the overall damage index of the structure for this case.

#### **10.2.2.4.2 Continuous Bridges**

Bearings and columns are the most vulnerable superstructure components in the case of continuous bridges. Damage to columns may result in the total failure of structure, but damage to bearings does not have significant consequences unless the structure slides. When sliding occurs, the structure may fall off its support if there is not adequate seat width, or the columns may be damaged due to large displacements at the columns' locations produced by the combined effect of sliding and elastic deformation

of the structure. Accordingly, the overall damage index,  $I_d$  of the structure is defined considering three possible cases:

- i) If  $I_{bd} < 1.0$  and  $I_{bd} < I_{cd}$ , then  $I_d = I_{cd}$
- ii) If  $I_{bd} < 1.0$  and  $I_{bd} > I_{cd}$ , then the consequences of damage to bearings should be investigated. Accordingly, assuming that the bearings are damaged, seat width and column damage indices  $I_{sw}^*$  and  $I_{cd}^*$  are calculated and the larger of these is selected as the temporary damage index,  $I_d^*$ . Then, overall damage index of the structure is determined considering the following three possible outcome:
  - a) If  $I_d^* > I_{bd}$  then  $I_d = I_{bd}$ .
  - b) If  $I_{cd} < I_d^* < I_{bd}$  then  $I_d = I_d^*$ .
  - c) If  $I_d^* < I_{cd}$  then  $I_d = I_{cd}$ .
- iii) If  $I_{bd} > 1.0$ , then  $I_d$  is determined by the larger of  $I_{cd}$  or  $I_{sw}$ .

In the first of the above three cases, since the column damage index is larger than the bearing damage index and the bearings are not damaged, the failure of the structure can only result from damage to columns and therefore the column damage index is the overall damage index of the structure. The second case is explained by the following example. Consider two bridges, A and B with identical bearing and column damage indices of 0.95 and 0.60 respectively. Since the bearing and column damage indices are identical for both bridges, they may be ranked as equally vulnerable if the consequences of damage to bearings are not considered. Now assume that bridge A has a very low friction coefficient at the bearings and therefore it may have large sliding displacements if the bearings are severed, whereas bridge B has a larger friction coefficient at the bearings and therefore the sliding displacements are not as much. Assuming that the bearings are severed, the seat width and column damage indices are calculated as 1.4 and 1.1 for bridge A and 0.4 and 0.8 for bridge B respectively. Accordingly, the temporary

damage index is the larger of the seat width and column damage indices and is 1.4 and 0.8 for bridges A and B respectively. For bridge A, since the temporary damage index is larger than 1.0, the result of damage to bearings is the total failure of the structure. Consequently, the bearings are the fuse elements of the bridge and therefore the bearing damage index is selected as the overall damage index. For bridge B, the consequence of damage to bearings is only to increase the risk of damage to the structure from 0.6 to 0.8. Accordingly, the temporary damage index is selected as the overall damage index of the structure.

It is noteworthy that in some occasions sliding may produce less displacements at the column locations than those produced by the elastic deformation of the structure before the bearings are damaged. This may happen if the ground motion has a high frequency content or high  $A_p/V_p$  ratio. Consequently, the column damage index calculated assuming that the bearings are severed may be smaller than the one calculated assuming that bearings are not severed. Considering this, the part c of the second case is specified for this purpose. In the third of the above three cases, the behavior of the structure after the bearing are severed is considered. In this case, the structure may get damaged either if it falls off its support or if the columns are damaged. Therefore, the overall damage index of the structure is determined by the larger of the seat width or column damage indices.

#### **10.2.2.4.3 Multi-Span Simply Supported Bridges**

In the case of multi-span simply supported bridges, damage to the bearings on the columns may create an unstable structure and result in failure. Inadequate seat width and column capacity are also equally responsible for the failure of the structure. Accordingly, the largest of the seat width, bearing and column damage indices defines the overall damage index of the bridge.

#### 10.2.2.5 Alternative Simplified Approach for Overall Damage Index

An even more rapid and conservative evaluation of damage indices can be used by taking the larger of  $I_{cd}$  or  $I_{sw}$  as overall damage index for any particular bridge assuming that bearings are damaged. If bearings are of a type which is unable to sustain any damage, then the overall damage index of the bridge would be the largest of  $I_{bd}$ ,  $I_{sw}$  and  $I_{cd}$  calculated assuming undamaged bearings.

#### 10.2.3 Ranking Index

The prioritization of seismic retrofitting of bridge structures is based on the calculation of a ranking index defined as the multiplication of the importance and damage indices.

$$I_R = I_i I_d \quad (10.80)$$

According to above expression, bridges with higher ranking index have higher priority for retrofitting. At first glance, this approach appears logical and properly considerate of other non-structural but important societal issues. It is similar in this respect to the other existing methodologies. However, there is a philosophical deficiency in the application of the above equation if left without a cut-off mechanism to alleviate the potentially undue impact of dominating societal aspects in existing bridges with already existing excellent seismic-resistance adequacy. This is explained by an example as follows.

Consider a bridge X in a region of high seismicity. Assume that this bridge has a damage index of 1.8 and an importance index of 0.2. Consider another bridge Y in a region of low seismicity. Assume that it has a damage index of 0.6 and an importance index of 0.8. The ranking index for bridge X is 0.36 and for bridge Y is 0.48. Since the ranking index of bridge Y is larger than that of bridge X, bridge Y is given priority for

retrofitting. However, note that although bridge Y is already very earthquake-resistant since its damage index is far below 1.0, it is given higher priority for retrofitting. In order to correct this inconsistency, and to ensure that the perceived priorities concur with actual needs, it is suggested that ranking index be set to zero when the damage index falls below a certain value, i.e. a cut-off value. Recall that damage indices are calculated using the equations proposed earlier which are functions of peak ground acceleration at the site obtained from a seismic map. The damage indices calculated following the procedures defined in this chapter are reliable within the specified constraints of their application, i.e. if there is no soil-structure interaction, and no risk of abutment or foundation failure. However, there is always a risk that the site peak ground acceleration specified in the seismic maps in probabilistic terms be exceeded. Therefore, the upper bound value below which the damage index is assumed to be zero must be calculated by relating the risk level adopted in current seismic maps and a lower pre-determined acceptable risk for the cut-off level. This is explained as follows.

In Canada, the seismic map is constructed assuming a 10 percent chance of exceedance in 50 years which is equivalent to a return period of 475 years, or a frequency of  $1/475=0.002105$ . In this study, a 5 percent chance of exceedance in 50 years, which is identical to a 975 years return period or a frequency of 0.001026, is considered as a most appropriate lower risk level to define the aforementioned cut-off value. The following procedure is adopted to convert the risk-based decision into practical terms.



The relationship between the magnitude,  $M$ , of Canadian earthquakes and their frequencies of occurrence,  $f_E$ , for an assumed risk, is expressed as follows (Basham et al., 1982);

$$f_E = N_0 e^{-\beta_f M} \left( 1 - e^{-\beta_f (M_x - M)} \right) \quad (10.81)$$

where,  $N_0$ ,  $\beta_f$  and  $M_x$  are the source zone magnitude recurrence parameters listed for various divided seismic zones in Canada. The above equation is inverted to represent the magnitude as a function of frequency of occurrence;

$$M = - \frac{1}{\beta_f} \ln \left( \frac{f_E}{N_0} + e^{-\beta_f M_x} \right) \quad (10.82)$$

To obtain the peak ground accelerations for a given magnitude, the attenuation relations proposed by Hasegawa et al. (1981), for strong seismic ground motion in Canada, are used. The attenuation relations for Eastern and Western Canada are respectively expressed in Eqs 10.83 and 10.84 as;

$$A_p = 0.035 e^{1.3M} R_h^{-1.1} \quad (10.83)$$

$$A_p = 0.102 e^{1.3M} R_h^{-1.5} \quad (10.84)$$

where,  $A_p$  is the peak ground acceleration as a fraction of gravitational acceleration and  $R_h$  is the hypocentral distance (km). Using the above equations, the ratio,  $R_A$ , of the peak ground acceleration,  $A_{p5}$ , for a 5 percent probability of exceedance in 50 years to  $A_{p10}$  for a 10 percent probability of exceedance in 50 years is expressed as follows;

$$R_A = \frac{A_{p5}}{A_{p10}} = e^{1.3 (M_5 - M_{10})} \quad (10.85)$$

where,  $M_5$  and  $M_{10}$  are the earthquake magnitudes obtained from Eq. 10.82 using the frequencies of occurrence corresponding to the probabilities of 5 and 10 percent

exceedance in 50 years respectively. Using Eqs. 10.82 and 10.85, and the data presented by Basham et al. (1982),  $R_A$ 's are obtained for 12 regions; 6 in eastern and 6 in western parts of Canada. These regions are selected such that their calculated magnitude  $M_{10}$  is greater than 5.0. No offshore seismic regions were considered. Then, each  $R_A$  is multiplied by the area of the region and the result is summed up. The sum is divided by the total area to obtain a weighted average of the peak acceleration ratios. This is equivalent to calculating an average seismic risk exposure of the bridges inventory, assuming that the number of bridges in these seismic regions is proportional to their area. Moreover, it also prevents the effect of extreme  $R_A$  values applicable over only very small seismic regions.

The results are illustrated in Tables 10.1 and 10.2 for the eastern and western parts of Canada respectively. The weighted average,  $R_w$ , of the acceleration ratios are obtained as 1.131 and 1.239 for the eastern and western parts of Canada respectively. This shows that when the probability of exceedance is reduced from 10 percent to 5 percent in 50 years, the site peak ground acceleration increases approximately 13 and 25 percent for the eastern and western regions of Canada respectively. Accordingly, if the seismic map of Canada is used to obtain the site peak ground acceleration to calculate the damage index, structures with damage index of 0.88 (i.e.  $1/1.131$ ) and 0.80 (i.e.  $1/1.239$ ) respectively in the eastern and western parts of Canada are considered to have a much lower risk of damage. Conservatively, bridge structures in all over Canada with damage index below 0.8 can be considered to have an acceptable very small risk of damage, equivalent to a 5% probability of exceedance in 50 years. Accordingly, the final ranking index proposed in this study is;

$$I_R = \left\{ \begin{array}{ll} I_i I_d & \text{if } I_d \geq 0.8 \\ 0.0 & \text{if } I_d < 0.8 \end{array} \right\} \quad (10.86)$$

#### **10.2.4 Examples**

Three bridges are considered to illustrate the procedure proposed. MPISD spectrum of Western USA earthquakes is used as the site spectrum in all the examples. The strengths of steel and concrete used in the bridges are respectively 350 and 20 MPa unless otherwise specified.

##### **10.2.4.1 Example 1**

The first example considered is a single span simply supported bridge. The girders and slab of this bridge are assumed to be composite for the response in both orthogonal directions. The properties of the bridge are listed as follows;

Type : Single span simply supported slab-on-girder steel bridge

Span length : 38 m.

Deck width : 7.5 m.

Slab thickness : 180 mm.

Pavement thickness : 70 mm.

Number of lanes : 2

Number of girders : 3

Girder spacing : 2.5 m.

Girder size : WWF 1200x380

Bearing type : Hinged rocker

Number of bolts : 4

Bolt diameter : 32 mm (area = 804 mm<sup>2</sup> )

Bolt spacing (parallel to the longitudinal direction) : 170 mm.

Bolt length : 475 mm.

Bolt shear strength : 230 MPa

Base plate width (parallel to the longitudinal direction) : 250 mm.

Bearing bar type : Four parallel triangular bearing hinges 120 mm apart

Thickness of each bearing hinge : 40 mm.

Height of the bearing hinges : 150 mm.

Base width of the bearing hinges : 200 mm.

Assumed friction coefficient : 0.2

Seat width (transverse) : 80 mm

Seat width (longitudinal) : 100 mm

Site peak ground acceleration = 0.4g (for 10% probability of exceedance in 50 years)

Importance index : 0.6

**(i) Calculation of mass, axial area and moment of inertia of the deck :**

- The mass of the bridge is calculated as;

$$m = 1.05 \times 38 \times ( 3 \times 0.38 + 7.5 \times 2.4 \times ( 0.18 + 0.07 ) ) = 225 \text{ tons}$$

Note that the factor 1.05 is used to increase the mass of the structure for the additional weight of secondary elements such as cross beams, guide rails, etc..

Mass of the bridge is also calculated using Eq. 10.1. It is obtained as 250 tons, 11% larger than the calculated value.

- To calculate the area and transverse direction moment of inertia of the deck, first the modulus of elasticity of steel is divided by that of concrete and a modulus ratio of 9 is obtained. Then, the thickness of the concrete slab is divided by 9 and an equivalent thickness of 20 mm is obtained. Using this thickness and the properties of the girders obtained from the Handbook of Steel Construction (1989), the area and transverse direction moment of inertia of the deck are calculated as;

$$A_D = 20 \times 7500 + 3 \times 48400 = 295200 \text{ mm}^2 = 0.2952 \text{ m}^2$$

$$I_D = \frac{1}{12} \times 20 \times 7500^3 + 2 \times 48400 \times 2500^2 = 1.31 \times 10^{12} \text{ mm}^4 = 1.31 \text{ m}^4$$

(ii) Calculation of translational and rotational stiffnesses of the bearings-set :

- The four triangular bearing hinges are approximated as rectangular ones for the purpose of calculating their stiffness. The height of each rectangular element is taken identical to the height of the triangular bearing hinges but its width is taken as half of their base width. Then, the sum of the moment of inertias of the four bearing hinges is calculated as;

$$I_{bb} = 4 \times \frac{1}{12} \times 40 \times \left( \frac{200}{2} \right)^3 = 13.33 \times 10^6 \text{ mm}^4$$

- Using Eq. 7.1 the stiffness of the triangular bearing hinges are calculated as;

$$k_{bb} = \frac{3 E I_{bb}}{h_{bb}^3} = \frac{3 \times 200 \times 10^3 \times 13.33 \times 10^6}{150^3} = 2.37 \times 10^6 \text{ kN/m}$$

- The distance of each pair of anchor bolts to the edge of the base plate is calculated as;  $(250 - 170)/2 = 40 \text{ mm}$ , and  $170 + (250 - 170)/2 = 210 \text{ mm}$ .

Then, using Eq. 7.3, the sum of the stiffnesses of the anchor bolts is calculated as;

$$\sum_{i=1}^{n_{ab}} k_{ab_i} = 2 \times \left[ \frac{40 \times 200 \times 10^3 \times 804}{150 \times 475} + \frac{210 \times 200 \times 10^3 \times 804}{150 \times 475} \right] = 1.13 \times 10^6 \text{ kN/m}$$

- Substituting the stiffness of the bearing bars and that of the anchor bolts in Eq. 7.4, longitudinal stiffness of each bearing is obtained as 760,000 kN/m. Then this

stiffness is substituted in Eqs. 7.6 and 7.8, to obtain the longitudinal and rotational stiffness of the bearings-set as  $2.28 \times 10^6$  kN/m and  $9.50 \times 10^6$  kN.m respectively.

**(iii) Calculation of bearing damage index :**

- Using Eq. 10.77, the capacity of each bearing is calculated as;

$$B_{rc} = 4 \times 804 \times 230 = 739680 \text{ N} = 740 \text{ kN}$$

- Using Eqs. 10.11 and 10.21, the transverse and longitudinal direction fundamental periods of the bridge are calculated respectively as 0.128 and 0.071 second. The  $\beta$  values corresponding to the transverse and longitudinal direction fundamental periods are obtained from MP1SD spectrum of Western USA as 1.96 and 1.53 respectively.
- Using Eqs 10.39, 10.41 and 10.45, the exterior bearing forces  $b_{ry}$  and  $b_{rx}$  due to transverse direction seismic excitation and the bearing force  $b_{rx}$  due to longitudinal direction seismic excitation are respectively obtained as 253, 468 and 425 kN.
- These bearing forces are substituted in Eq. 10.76 and the bearing force demand considering the two load cases are obtained as;

$$B_r = \sqrt{(1.0 \times 253)^2 + (1.0 \times 468 + 0.3 \times 425)^2} = 647 \text{ kN}$$

$$B_r = \sqrt{(0.3 \times 253)^2 + (0.3 \times 468 + 1.0 \times 425)^2} = 570 \text{ kN}$$

- The largest of these forces and the capacity of the bearing are substituted in Eq. 10.75 to obtain the bearing damage index as:

$$I_{bd} = \frac{647}{740} = 0.87$$

**(iv) Calculation of seat width index :**

- Using Eq. 10.73, the peak ground acceleration required for sliding is obtained respectively in the transverse and longitudinal direction as:

$$A_p = \frac{\pi^2}{8 \times 1.95} \times 1.0 \times 0.2 = 0.13g$$

$$A_p = \frac{\pi^2}{8 \times 1.53} \times 0.5 \times 0.2 = 0.08g$$

These accelerations are less than the peak ground acceleration of the site. Therefore, sliding may occur if the bearings are damaged.

- Using Eq. 10.72 the  $\psi$  values in the transverse and longitudinal directions are respectively obtained as:

$$\psi = \frac{8 \times 1.0 \times 0.2}{\pi^2 \times 0.4} = 0.405$$

$$\psi = \frac{8 \times 0.5 \times 0.2}{\pi^2 \times 0.4} = 0.203$$

- Using the above calculated  $\psi$  values and periods of the structure, the sliding displacements in the transverse and longitudinal directions are obtained from Figure 10.5 as 17 and 35 mm respectively.

- Substituting these displacements and the seat widths in Eqs. 10.66 and 10.67, the seat width indices in the transverse and longitudinal directions are respectively obtained as;

$$I_{swT} = \frac{17 + 50}{80} = 0.84$$

$$I_{swL} = \frac{35 + 50}{100} = 0.85 \leq \frac{0.84 \times 38 + 50}{100} = 0.82$$

the larger result being 0.84, and therefore defined as the seat width index of the bridge.

**(v) Calculation of overall damage and ranking indices of the structure :**

Since the bearings are not severed, the damage index of the structure is selected as the smaller of the bearing damage and seat width indices. Accordingly the damage index of the structure is 0.84. Using Eq. 10.86 the ranking index of the structure is obtained as,  $0.6 \times 0.84 = 0.50$ . It is noteworthy that the simplified alternative approach yields the same result.

#### **10.2.4.2 Example 2**

The second example considered is a four lane continuous bridge with three spans. The girders and slab of this bridge are assumed to be composite for the response in both orthogonal direction. The properties of the bridge are listed as follows;

Type : Continuous slab-on-girder steel bridge

Number of spans : 3

Length of each span : 35, 25, 30 m.



Total end-to-end length : 90 m.  
Deck width : 16 m.  
Slab thickness : 190 mm.  
Pavement thickness : 70 mm.  
Number of lanes : 4  
Number of girders : 6  
Girder spacing : 3.0 m.  
Girder size : WWF 1200x333  
Type of column connection at foundation : hinge  
Column height : 6 m at each bent  
Column Size : W 310x79 (strong axis bending in the longitudinal direction)  
Bearing type : Sliding-bearings  
Number of bolts : 2  
Bolt diameter : 32 mm (area = 804 mm<sup>2</sup> )  
Bolt spacing (parallel to the longitudinal direction) : 150 mm.  
Bolt length : 400 mm.  
Bolt shear strength : 230 MPa  
Base plate width (parallel to the longitudinal direction) : 230 mm.  
Bearing bar type : rectangular bar  
Width of bearing bar (parallel to the longitudinal direction) : 90 mm.  
Depth of bearing bar (parallel to the transverse direction) : 400 mm.  
Height of the bearing bar : 220 mm.  
Assumed friction coefficient : 0.4  
Seat width (transverse) : 85 mm  
Seat width (longitudinal) : 110 mm  
site peak ground acceleration = 0.3g (for 10% probability of exceedance in 50 years)  
Importance index : 0.9

**(i) Calculation of mass, axial area and moment of inertia of the deck :**

- The mass of the bridge is calculated as ;

$$m = 1.05 \times 90 \times ( 6 \times 0.333 + 16 \times 2.4 \times ( 0.19 + 0.07 ) ) = 1130 \text{ tons}$$

Mass of the bridge is also calculated using Eq. 10.1. It is obtained as 1250 tons, 21% larger than the calculated value.

- The thickness of the concrete slab is divided by the modulus ratio and an equivalent thickness of 21.1 mm is obtained. Then, using this thickness and the properties of the girders obtained from the Handbook of Steel Construction (1989), the area and transverse direction moment of inertia of the deck are calculated as;

$$A_D = 21.1 \times 16000 + 6 \times 42400 = 592000 \text{ mm}^2 = 0.592 \text{ m}^2$$

$$I_D = \frac{1}{12} \times 21.1 \times 16000^3 + 2 \times 42400 \times ( 1500^2 + 4500^2 + 7500^2 ) = 13.9 \text{ m}^4$$

**(ii) Calculation of columns' properties :**

- The stiffness of the columns are calculated as 111 and 492 kN/m in the transverse and longitudinal directions respectively. Then, the stiffness of the columns-set in the transverse and longitudinal direction are calculated as  $6 \times 111 = 666$  and  $6 \times 492 = 2952$  kN/m.

- Since the columns' sizes and heights are identical at each bent, the ones closest to the midspan are selected. Using their tributary area, the axial force due to dead load of the structure is calculated as :

$$P_D = 1.05 \times \frac{35 + 25}{2} \times (0.333 + 3 \times 2.4 \times (0.19 + 0.07)) \times 9.81 = 683 \text{ kN}$$

- Using Eqs 8.2 and 8.6, the flexural capacity of the column,  $M_{uy}$ , in the transverse and  $M_{ux}$ , in the longitudinal directions are calculated as, 115 and 295 kN.m respectively.

**(iii) Calculation of translational and rotational stiffnesses of the bearings-set :**

- The moment of inertia of the bearing bar is calculated as;

$$I_{bb} = \frac{1}{12} \times 400 \times 90^3 = 24.3 \times 10^6 \text{ mm}^4$$

- Using Eq. 7.1 the stiffness of the bearing bar is calculated as;

$$k_{bb} = \frac{3 \times 200 \times 10^3 \times 24.3 \times 10^6}{220^3} = 1.37 \times 10^6 \text{ kN/m}$$

- The distance of the anchor bolts to the edge of the base plate is calculated as;  $(230 - 150)/2 = 40 \text{ mm}$ , and  $150 + (230 - 150)/2 = 190 \text{ mm}$ . Then, using Eq. 7.3, the sum of the stiffnesses of the anchor bolts is calculated as;

$$\sum_{i=1}^{n_{ab}} k_{ab_i} = \frac{40 \times 200 \times 10^3 \times 804}{220 \times 400} + \frac{190 \times 200 \times 10^3 \times 804}{220 \times 400} = 0.42 \times 10^6 \text{ kN/m}$$

- Substituting the stiffness of the bearing bars and that of the anchor bolts in Eq. 7.4, longitudinal stiffness of each bearing is obtained as 320,000 kN/m. Then this stiffness is substituted in Eqs. 7.6 and 7.8, to obtain the longitudinal and rotational stiffness of the bearings-set as  $1.92 \times 10^6$  kN/m and  $51 \times 10^6$  kN.m respectively.

**(iv) Calculation of bearing damage index :**

- Using Eq. 10.77, the capacity of each bearing is calculated as;

$$B_{rc} = 2 \times 804 \times 230 = 369840 \text{ N} = 370 \text{ kN}$$

- Using Eqs. 10.24 and 10.21, the transverse and longitudinal direction fundamental periods of the bridge are calculated respectively as 0.300 and 0.165 second. The  $\beta$  values corresponding to the transverse and longitudinal direction fundamental periods are obtained from MP1SD spectrum of western USA as 2.50 and 2.24 respectively.
- Using Eqs 10.39, 10.41 and 10.45, the exterior bearing forces  $b_{ry}$  and  $b_{rx}$  due to transverse direction seismic excitation and the bearing force  $b_{rx}$  due to longitudinal direction seismic excitation are respectively obtained as 563, 1320 and 1068 kN.
- These bearing forces are substituted in Eq. 10.76 and the bearing force demand considering two load cases are obtained as 1589 and 1329 kN.
- The largest of these forces are selected and substituted in Eq. 10.75 together with the bearing capacity and the bearing damage index is obtained as 4.30

**(v) Calculation of seat width index :**

- Since the bearings are damaged, the peak ground acceleration required for sliding should be obtained. First, the fraction of the weight of the structure transferred to the supports is calculated using the tributary area of each end spans. The fraction of the weight at the left and right abutments are respectively  $17.5/90=0.195$  and  $15/90=0.167$ . The proportion of the weight which is useful for resisting the sliding in the transverse direction is  $0.195+0.167=0.36$  and it is 0.195 in the longitudinal direction. Using Eqs. 10.24 and 10.22, the transverse and longitudinal direction fundamental periods of the bridge are re-calculated respectively as 0.340 and 0.118 second, considering that the bearings are damaged. The  $\beta$  values corresponding to the transverse and longitudinal direction fundamental periods are obtained as 2.50 and 1.89 respectively. Using Eq. 10.73, the peak ground acceleration required for sliding is obtained respectively in the transverse and longitudinal direction as;

$$A_p = \frac{\pi^2}{8 \times 2.50} \times 0.36 \times 0.4 = 0.07g$$

$$A_p = \frac{\pi^2}{8 \times 1.89} \times 0.195 \times 0.4 = 0.051g$$

The peak ground accelerations required for sliding in both orthogonal directions are less than the site peak ground acceleration of 0.3g. Therefore sliding may occur when the bearings are damaged.

- Using Eq. 10.72 the  $\psi$  values in the transverse and longitudinal directions are respectively obtained as 0.389 and 0.211.

- Using the above calculated  $\psi$  values and periods of the structure, the sliding displacements in the transverse and longitudinal directions are obtained from Figure 10.5 as 46 and 45 mm respectively.
- Substituting the sliding displacement together with the seat widths, the seat width indices in the transverse and longitudinal directions are respectively obtained as;

$$I_{swT} = \frac{46 + 50}{85} = 1.13$$

$$I_{swL} = \frac{45 + 50}{110} = 0.86 \leq \frac{0.84 \times 90 + 50}{110} = 1.14$$

the larger result being 1.13, and therefore defined as the seat width index of the bridge.

**(vi) Calculation of column damage index :**

- Using Eqs. 10.58 and 10.60 the seismic moment demands are obtained as 70 and 148 kN.m in the transverse and longitudinal directions respectively. These moments and the flexural capacities of the columns are substituted Eq. 10. 79 to obtain the column damage index for two load cases as;

$$I_{cd} = \frac{1.0 \times 70}{115} + \frac{0.3 \times 148}{295} = 0.76$$

$$I_{cd} = \frac{0.3 \times 70}{115} + \frac{1.0 \times 148}{295} = 0.68$$

The column damage index is the larger of the above two value, i.e. 0.76.

**(vii) Calculation of overall damage and ranking indices of the structure :**

The damage index of the structure is selected as the larger of seat width and column damage indices. Accordingly the damage index of the structure is 1.13. Using Eq. 10.86 the ranking index of the structure is obtained as,  $0.9 \times 1.13 = 1.02$ . It is noteworthy that conservatively assuming that the bearings are damaged, the simplified alternative approach yields the same result.

**10 2.4.3 Example 3**

The third example considered is a two lane simply supported bridge with three spans. The properties of the bridge are listed as follows;

Type : Multi-span simply supported slab-on-girder steel bridge

Number of spans : 3

Length of each span : 35, 43, 37 m.

Widths of expansion joints : 0, 25, 35, 30 mm.

Deck width : 7.4 m.

Slab thickness : 175 mm.

Pavement thickness : 65 mm.

Number of lanes : 2

Number of girders : 4

Girder spacing : 2.0 m.

Girder size : WWF 1200x333

Type of column connection at foundation : fixed

Column height : 5.5 m at each bent

Column Size : WWF 400x178 (strong axis bending in the longitudinal direction)

Bearing type on the columns : Sliding-bearings

Bearing bar type : rectangular bar

Width of bearing bar (in the longitudinal dir) : 100 mm.

Depth of bearing bar (in the transverse dir) : 400 mm.

Height of the bearing bars : 200 mm.

Seat width (Longitudinal) : 175 mm at each bent

site peak ground acceleration = 0.3g (for 10% probability of exceedance in 50 years)

Importance index : 0.8

#### **(i) Calculation of mass**

- Spans with the longest average length are selected. These are the ones with 43 and 37 metres length. The total length of this subassembly is  $43+37=80$  m. The mass is calculated as 470 tons.

#### **(ii) Calculation of columns' properties :**

- Including the stiffness of the bearing bar, the stiffness of each column is calculated as 847 and 2472 kN/m respectively in the transverse and longitudinal directions using Eqs. 9.4 and 9.5. The stiffness of the columns-set in the transverse and longitudinal direction are calculated as  $4 \times 847 = 3388$ ,  $4 \times 2472 = 9888$  kN/m. Using Eq. 9.6, the rotational stiffness of the columns-set is calculated as 49440 kN.m.
- Using the tributary area, the axial force in the columns, due to dead load of the structure is calculated as 612 kN.
- Using Eqs 8.2 and 8.6, the flexural capacities of the column,  $M_{ny}$ , in the transverse and  $M_{nx}$ , in the longitudinal directions are calculated as, 615 and 1207 kN.m respectively.



- Using Eq. 9.7, the transverse stiffness,  $K_{TT}$ , of the subassembly is calculated as 3424 kN/m. Then, it is substituted in Eqs. 9.42 and 9.43 and the magnification factors for the columns in the transverse and longitudinal direction are calculated as 1.151 and 1.052 respectively. These factors will be used to magnify the moments due to seismic excitation in the transverse direction.

**(iii) Calculation of bearing damage index**

- Using Eqs. 10.24 and 10.21, the transverse and longitudinal direction fundamental periods of the selected subassembly and the column-fixed deck with the longest span are calculated respectively as 1.26 and 1.00 second. The  $\beta$  values corresponding to the transverse and longitudinal direction fundamental periods are obtained from MPISD spectrum of western USA as 0.99 and 1.25 respectively.
- Using Eq. 9.54, and considering the deck with the longest span, the minimum required peak ground acceleration for collision to occur is obtained as;

$$A_p = \frac{K_{cl} EJW}{\beta m g} \left( 1 - \frac{P}{k_{cl} h_c} \right) = \frac{9888 \times 0.035}{1.25 \times 253 \times 9.81} \times \left( 1 - \frac{612}{2472 \times 5.5} \right) = 0.11g$$

Dividing the peak ground acceleration of the site by this acceleration, the bearing damage index is obtained as 2.73.

**(iv) Calculation of seat width index :**

- The seat width is given as 175 mm at every bent. Therefore, it is only necessary to check the bent which supports the longest spans since it is the most critical one. Using Eqs 9.59 and 9.67, the minimum required seat width is obtained as 183 mm at the expansion joint adjacent to the longest span. Substituting this minimum required seat width and the seat width at the bent in Eq. 10.68, the seat width index is obtained as 1.05

**(v) Calculation of column damage index :**

- Using Eqs. 10.63 and 10.64 the seismic moment demands  $M_{Ey}$  and  $M_{Ex}$  in the exterior column, due to seismic excitation in the transverse direction are respectively obtained as 942 and 204 kN.m in the transverse and longitudinal directions respectively. Using Eq. 10.65, the seismic moment demand,  $M_{Ex}$  due to seismic excitation in the longitudinal direction is obtained as 926 kN.m. These moments and the flexural capacities of the columns are substituted in Eq. 10.79 to obtain the column damage indices for two load cases as;

$$I_{cd} = \frac{1.0 \times 942}{615} + \frac{1.0 \times 204 + 0.3 \times 926}{1207} = 1.93$$

$$I_{cd} = \frac{0.3 \times 942}{615} + \frac{0.3 \times 204 + 1.0 \times 926}{1207} = 1.28$$

The column damage index is the larger of the above two value, i.e. 1.93.

**(vi) Calculation of overall damage and ranking indices of the structure :**

- The damage index of the structure is selected as the larger of bearing damage, seat width and column damage indices. Accordingly the damage index of the structure is 2.73. Using Eq. 10.86 the ranking index of the structure is obtained as,  $0.8 \times 2.73 = 2.18$ . Assuming that the bearings are unable to sustain any damage, the simplified alternative approach yields the same result.

## CHAPTER 11

### SOME RETROFITTING TECHNIQUES FOR SLAB-ON-GIRDER STEEL BRIDGES

In the previous chapters, results showed that continuous bridges longer than 30-40 metres span and multi-span simply supported bridges are vulnerable to seismic excitations. In both types of bridges, the columns are found to suffer damage due to excessive displacements. In the case of multi-span simply supported bridges, large openings in the expansion joints is an additional problem. In this chapter, some retrofitting methods are suggested and their effects on seismic response and capacity are investigated.

#### 11.1 Retrofitting Continuous Bridges

As a retrofitting solution, cross bracings are introduced to the columns-set of the continuous bridges. The braced system is shown in Figure 11.1(a). One end of the bracing is connected to the base of the column at one side of the deck and the other end is connected to the top of the column at the other side of the deck. The other bracing is connected to the columns' ends in the reverse order to obtain a cross bracing. Using the tributary mass on the columns which is half of the total mass of the bridge, and assuming a spectral acceleration of  $3 \cdot A_p$ , where  $A_p$  is chosen as 0.4g, the transverse seismic force acting on the bracing system is calculated as 60 percent of the total weight of the structure, i.e.  $0.6 \cdot W$ . This spectral acceleration of  $3 \cdot A_p$  is the amplitude of the flat part

of Canadian design spectra, and is used knowing that the bracing would reduce and move the period of the structure to that part the spectra. The peak acceleration is chosen as 0.4g considering high intensity earthquakes to be on the safe side. Then, the required areas of the bracing elements are calculated for each bridge and WWT250x111.5, WWT250x153, WWT250x190.5 are selected respectively for 40, 50 and 60 metres span bridges. Response spectrum analyses of these modified bridges are conducted using Eastern Canada design spectrum. It is noteworthy that, in the analyses, instead of two, only one bracing element is considered knowing that bracings can not accommodate significant compression forces. The bracing forces obtained from the response spectrum analyses for a peak ground acceleration of 0.4g, closely matched the initially estimated bracing design forces.

Analyses results showed that, with the introduction of bracing elements, the period of the bridges reduce considerably. The reduction is 46 percent for the shortest and 66 percent for the longest of the bridges considered in the analyses. The reductions in the midspan displacements and the column moments are 64 percent for the shortest and 85 percent for the longest of the bridges considered. The bearing forces are also reduced by a factor of 2 for the shortest and 4 for the longest of the bridges considered. However, the seismic axial forces on the exterior columns are drastically increased. Using these new moments and axial forces acting on the columns, the maximum resistible peak ground accelerations for each bridge are calculated using Eq. 8.5.

Additionally, as seen in Figure 11.1(b), one horizontal bracing connected to the columns' web at mid-height, and a cross bracing are considered together. For this second configuration also, the maximum resistible peak ground accelerations are also calculated for each bridge. The results are plotted in Figure 11.2 as a function of span length. As seen in the figure, using cross bracing in bridges of spans less than 55 metres does not improve the seismic capacity, and the contribution is not considerable for longer span bridges. This is a consequence of the larger seismic forces attracted due to a remarkable decrease in the fundamental period, as well as the reduction in the flexural capacity of

the exterior columns due to the negative impact of high axial loads transferred to the exterior columns by the bracing elements. In this case, using cross bracing along with reinforced or new larger columns would certainly be a more adequate approach to increase the seismic capacity, keeping the beneficial effect of cross bracings in reducing bearing forces.

However, without reinforcing columns, when horizontal and cross bracings are used together, there is a remarkable increase in the seismic capacity of bridges longer than 45 metres. For example, the maximum resistible peak ground acceleration of 0.14g for the 60 metres continuous bridge increased to 0.32g. Moreover, although detrimental to columns, using cross and horizontal bracings together is beneficial in the case of short span bridges due to the resulting drastic reduction in the bearing forces; The reduction in the seismic capacity due to columns is not as important in that case since short span bridges can still accommodate high intensity seismic excitations.

It is noteworthy that characteristics of the seismic excitations may affect the degree of contribution of cross bracings to the seismic capacity of the structures. Consider the different design response spectra representative of earthquakes in Eastern and Western Canada, shown in Figure 6.7. Both have the same amplitude in the short period region, but Western Canada design spectrum has a larger amplitude beyond that region. Therefore, when Western Canada design spectrum is used in the analyses, the reduction in the fundamental period of the structures due to the addition of cross bracings does not produce increases in spectral acceleration as large as when Eastern Canada spectrum is used, and therefore a relatively larger increase in the capacity of the structure may be obtained. For example, assume that a continuous bridge has a transverse fundamental period of 0.45 second. When cross bracing is used, the period drops down to 0.24 second. In the case of Western Canada design spectrum, this produces no change in the spectral acceleration of the structure since the flat part of this spectrum extends to 0.50 second. Whereas, in the case of Eastern Canada design spectrum, the constant pseudo-acceleration plateau extends only to 0.25 second, thus, the spectral acceleration

for this structure increases by almost 50 percent with the reduction in its period. Therefore, the beneficial effects of cross bracing are less compared to the case with Western Canada design spectrum.

## **11.2 Retrofitting Multi-Span Simply Supported Bridges**

In multi-span simply supported bridges, the steel columns are found to suffer damage due to excessive displacements in the transverse direction. In most of the cases, the columns are found to be damaged before the collision of two neighbouring decks. Large openings in the expansion joints due to seismic excitation in both principal directions and the effect of impacting are additional problems.

To prevent excessive displacement of the columns and reduce the openings of the expansion joints due to seismic excitation in the transverse direction, cross bracings again appear to be logical retrofitting solution. The bracing system shown in Figure 11.1a is introduced to 2-lane bridges with two simply supported spans of 20, 30, 40, 50, and 60 metres each and columns of 6 metres height. These bridges are assumed to have bearings-set with zero rotational stiffness. Using the tributary mass on the columns and assuming a spectral acceleration of  $3 \cdot A_p$ , where  $A_p$  is chosen as 0.4g, the seismic force acting on the bracing system is calculated. Similar to the continuous bridges' case, this spectral acceleration of  $3 \cdot A_p$  is used knowing that the bracing would reduce and move the period of the structure to the flat part of the design spectrum. The peak acceleration is chosen as 0.4g to be representative of high intensity earthquakes.

The required size of bracing elements are calculated to be WT100x50, WT155x79, WT180x108, WWT250x138, WWT250x190.5 respectively for 20, 30, 40, 50 and 60 metres span bridges. Response spectrum analyses of these modified bridges are conducted using Eastern Canada design spectrum. The bracing forces obtained from the response spectrum analyses for a peak ground acceleration of 0.4g, closely matched the

initially estimated bracing design forces. Analyses results showed that, with the introduction of bracing elements, the period of the bridges are reduced almost by a factor of 8. The midspan displacements, expansion joint openings and the column moments are reduced by a factor of 24 for the shortest and 40 for the longest of the bridges considered. However, the bearing forces are increased by 63 percent for the shortest and 120 percent for the longest of the bridges considered. The seismic axial forces on the exterior columns are also drastically increased.

The longitudinal direction seismic moments in the columns, due to rotation of the column-fixed deck about a vertical axis perpendicular to it, are negligibly small when cross bracing is used. Therefore, only the effect of transverse direction moments is considered. Knowing that the sway of the bridge is prevented by the bracing system, the extra lateral force due to second order moments in the columns does not cause a significant additional displacement. Thus, one iteration is adequate to calculate the second order moment and therefore, Eq 8.5 which was derived on this basis is used to calculate the maximum resistible peak ground accelerations for the bridges considered. The results are presented in Figure 11.3. In the figure, the maximum resistible peak ground acceleration as a function of span length is plotted for braced and unbraced systems. As seen, using a cross bracing increases the seismic capacity of the structure drastically. It is noteworthy that the maximum resistible peak ground accelerations for the braced structures are obtained assuming that the bracing elements remain within the elastic range. In fact, these elements were observed to start yielding at a peak ground acceleration of 0.4g for the design spectrum considered in the analyses.

In the longitudinal direction, bridges should be retrofitted to prevent excessive openings in the expansion joints and collision of the adjacent superstructure components. Restrainer cables are proven to be effective in preventing such problems (Degenkolb, 1978; Seismic, 1983; Imbsen and Penzien, 1986; Seismic, 1987).

### **11.3 Additional Comments on Retrofitting**

It is noteworthy that using cross bracings improves the seismic performance of both continuous and multi-span simply supported bridges in the transverse direction. However, high uplift forces and pressures on the foundations may simultaneously be produced. Therefore, the capacity of the foundation system in resisting these forces should be checked and, if required, improved. Additionally, when cross bracings are used, the bearing forces on the columns and abutments increases in multi-span simply supported bridges with bearings-set of zero rotational stiffness. Therefore, the capacity of the bearings should also be checked. Note that using bracing in bridges with bearings-set of high rotational stiffness would decrease the bearing forces on the abutments due to reduced support rotations.

The columns in multi-span simply supported bridges are designed considering that the structure can sway. However, the columns in continuous bridges are designed assuming that sway is prevented due to high in-plane stiffness of the deck. Accordingly, the column sizes in multi-span simply supported bridges are larger than those in continuous bridges. Therefore, the negative impact of axial force produced by cross bracings, on the moment capacity of the columns is less in multi-span simply supported bridges. Consequently, when cross bracing is used, a much larger increase in the seismic capacity is obtained for multi-span simply supported bridges than for continuous bridges.

### **11.4 Summary**

- Introduction of a cross bracing between the exterior bridge columns results in a significant reduction in the midspan displacement and the bearing forces, however, a significant increase in the axial load on the exterior columns.



- For multi-span simply supported bridges, using cross bracing alone is beneficial for the ranges of spans considered. For continuous bridges, they improve the capacity for spans longer than 55 metres. However, when used with larger columns, it would certainly increase the capacity for all span ranges, with the added beneficial effect of reducing bearing forces.
- When horizontal and cross bracings are used together, there is a considerable increase in the seismic capacity of continuous bridges longer than 45 metres. However, using cross and horizontal bracings together is beneficial, even in the case of short span continuous bridges, due to the drastic reduction in the bearing forces.

## **CHAPTER 12**

### **SUMMARY AND CONCLUSIONS**

This study is composed of two main parts. In the first part the effect of extreme gravity loads and in the second part the effect of seismic loads on slab-on-girder steel bridges are studied.

#### **12.1 Effect of Extreme Gravity Loads**

Typical heavy permit-truck configurations are chosen to investigate the effect of extreme gravity loads on actual steel bridges. In this respect, fatigue and ultimate strength, two important parameters in bridge design are studied and the adequacy of existing methodologies used to permit extra-heavy trucks on Ontario bridges are evaluated. As a first step, influence line analyses are conducted to find the ranges of span lengths of simply supported and continuous bridges for which heavy permit-trucks could have the most detrimental effects. From this preliminary study, simply-supported single span bridges in the 15-20 metres range and continuous bridges in the 10-25 metres span ranges are found to be mostly affected by heavy permit-truck models 1 to 4. The permit-truck 5 has detrimental effects only on bridges shorter than 10 metres. Although slab-on-girder bridges of such spans are unusual, stringers in steel truss bridges are most often less than 10 metres.

Six existing bridges of spans in the above range are then analyzed as loaded by five models of heavy permit-trucks. Results are compared with those obtained using the simplified method of the OHBDC, as well as more accurate computer analyses conducted for direct application of the OHBD-truck model. For slab-on-girder steel bridges, the permit-trucks are found to produce detrimental effects on the exterior girders of the bridges studied. The most critical effects of permit-trucks are found to occur on bridges having a combination of small curb-width and large overhang. For steel truss bridges, only the stringers are found to be detrimentally affected by heavy permit-trucks. However, the live load factor used in the original design of steel bridges produces a strength reserve which is found adequate to accommodate the more predictable weight of heavy permit-trucks. Therefore, ultimate strength is not a problem for bridges designed in compliance with the 1983 edition of OHBDC. However, this is not true for fatigue-related problems where live load factors during design are actually less than 1.0. In fact, both the heavy permit-trucks and those trucks from normal traffic contribute to these fatigue problems.

Other researchers have recently demonstrated that if any of the stress ranges from a variable amplitude stress spectrum exceeds the CAFL of the S-N curve, even by a small amount or by a small frequency, then the fatigue life is not infinite anymore. They experimentally demonstrated that the CAFL can be replaced conservatively by the straight line extension of the finite life portion of the S-N curve. Unfortunately, in addition to permit-trucks, the load effect of a certain percentage of the truck population in Ontario exceeds that of the OHBD-fatigue truck model. Therefore, the fatigue life of some bridges designed according to 1983 edition of the OHBDC is not infinite, contrary to the design intent. Furthermore, as the fatigue truck model considered in design is not placed on the bridge deck at a position causing the largest live load effect, this adds to the probability that the design stress ranges can be exceeded. Although the stress ranges due to heavy permit-trucks are sometimes found to exceed the design stress ranges by approximately 50%, the disappearance of the CAFL can be equally attributable to overweight conditions in the normal traffic. The comparative effect of heavy permit-

trucks passages in shortening fatigue-life has been calculated by adopting the new modified S-N curve. It has been speculated by others that the magnitude of the maximum stress-range above the CAFL may affect the slope by which the finite life portion of the S-N curve is extended below the CAFL, i.e. more severe exceedances could cause larger slopes. In that perspective, heavy permit-trucks that can cause higher stress-ranges in bridge elements than the OHBD-fatigue-truck may, depending on their frequency of passage, produce more severe fatigue problems, i.e. more severe shortening of the fatigue life. The relationship between the magnitude of the peak stress range and slope of the S-N curve below the CAFL, remains to be experimentally investigated, however, in this study, the effect of a linear relationship between this slope and magnitude, on fatigue life has been analytically addressed.

Some findings on the effect of various variables on the reduction and estimation of fatigue life of bridge components are noteworthy :

- The fatigue life of an over-designed bridge is proportional to the cube of the over-design ratio with respect to SLS-I, provided that slope of the S-N curve below the CAFL is 1:3. The reverse is true for an under-designed bridge with respect to SLS-I, i.e. the exponent becomes one-third.
- As the allowable stress of the SLS-I detail category decreases (allowable stress is the highest for detail a and lowest for detail f), and the yield strength of the steel used in design increases, fatigue damage becomes more likely and critical.
- The variability of the percent reduction in fatigue life due to heavy permit-trucks, when obtained using spectra corresponding to various span length is negligible. Therefore, an average stress-frequency spectrum representing the effect of Ontario truck traffic can be used for all spans ranging up to 100 metres.
- The percentage of reduction in fatigue life does not depend on the detail type.

- The variation of the percent reduction in fatigue life as a function of the number of passages of heavy-permit trucks per month seems to be an almost linearly increasing function of the frequency of passage.
- As the slope below the CAFL of the S-N curve decreases, the percentage of reduction in fatigue life due to heavy permit-trucks linearly increases.
- Fatigue life is inversely proportional to ADTT, and the percentage of reduction in fatigue life due to heavy permit-trucks increases as the ADTT decreases.

A fatigue based methodology has been developed and is proposed to assess the percentage reduction in the fatigue life of steel bridges caused by heavy-permit trucks for various fatigue detail categories. As part of the methodology, an analytical expression is provided to calculate the allowable number of heavy permit trucks per month as a function of a specified percentage of reduction in fatigue life, weight, axle width, axle configuration as well as the speed of the permit truck, and average daily truck traffic. Using this expression and data provided by the MTO, it is found that, for slab-on-girder steel bridges, 15, 30, 45 and 60 heavy trucks per month may be permitted for spans shorter than 35 metres, 35 to 50 metres, 50 to 75 metres, and longer than 75 metres respectively. For steel truss bridges, 8, 12, and 33 heavy trucks per month may be permitted for stringer spans less than 7 metres, 7 to 10 metres and more than 10 metres respectively.

## **12.2 Effect of Seismic Loads - Summary**

In the second part of this research, the effect of seismic loads on three common types of slab-on-girder steel bridges is studied. These are single span simply supported, continuous and multi-span simply supported bridges. The geometric and structural properties of these bridges are varied to cover a broad range of existing steel bridges and

to investigate their effect on the seismic response of the structures. Linear and non-linear dynamic analyses of these bridges are conducted for earthquakes of different characteristic and intensity. The responses of vulnerable superstructure components are determined as a function of selected parameters and earthquake types. Using these results, a methodology is developed for an accurate rapid seismic evaluation and ranking of existing steel bridges.

The response of bridges to earthquake excitations is investigated considering two possible cases. First, the bearings are assumed to survive the earthquake without any damage and the response of other structural components is investigated. Then, the bearings are assumed to be damaged and the impact of this failure on the seismic response of the bridges is studied. The conclusions reached for each of these cases follows:

#### **12.2.1 Bridges with Undamaged Bearings**

First, the effect of various geometric and structural properties of bridges on the bearing forces is investigated. For single span simply supported and continuous bridges, bearing forces due to seismic excitation in both transverse and longitudinal direction are found to be proportional to the mass of the bridge and span length. They become larger as the bearings get stiffer. Their magnitude is independent of the number of lanes when the bridge is subjected to seismic excitation in the longitudinal direction. However, they are highly dependent on the number of bearings and mass of the bridge. Longitudinal bearing forces in wider bridges with less number of bearings are larger. When the bridges are subjected to seismic excitation in the transverse direction, the bearing forces generally decreases as the number of lanes or bridge width increases. Bearings with higher stiffness and closer to the edge of the bridge deck attract larger forces than other bearings. However, elastomeric bearings or bearings with small longitudinal stiffness, attract almost equal forces regardless of the number of lanes or width of the bridge;

Nevertheless, these are negligibly small compared to the forces in other types of bearings.

In the case of multi-span simply supported bridges, bearing stiffness is found to considerably affect the seismic response of bridges having only two spans. The transverse period of these bridges is highly dependent on the stiffness of the bearings used. However, for bridges with more than two spans, the effect of bearing stiffness is localized and vanishes with increasing number of spans. Generally, the conclusions reached for the forces in bearings of single span simply supported and continuous bridges are valid for the forces in bearings of multi-span simply supported bridges if collision of superstructure components does not occur, otherwise, very high forces are exerted on the bearings due to the effect of impact.

For continuous bridges, the effect of various geometric and structural properties on the seismic response is further investigated. First, the effect of number of spans on the seismic response is studied. It is found that, as the number of spans increases, the seismic capacity decreases. This is mainly due to the increased mass and flexibility of the structure which produce higher displacements at the column locations, resulting in failure of the columns due to instability. However, multiple span continuous bridges can generally accommodate peak ground accelerations larger than two span continuous bridges of identical end-to-end length. Bridges with even number of spans are found to be more vulnerable to seismic excitations than those with odd number of spans. Nevertheless, this affect vanishes as the span length and number of spans increases. Next, column size is varied and its effect on seismic response is studied. It is found that column size does not have a significant effect on decreasing the deflection of the bridge deck, however, larger columns are found to increase the seismic capacity. This is due to the increasing axial load capacity rather than the flexural capacity of the column as their size increases. The percentage of shear attracted by columns increases as their size gets larger, however, this is not remarkable since most of the seismically-induced force is resisted by the bearings. Additionally, the effect of steel strength on the seismic response is studied. Results show that bridges designed using lower strength steel are more vulnerable to earthquake

excitations. This is a consequence of the increased dead load of the superstructure which subsequently results in higher axial load on the columns and attraction of bigger moments due to larger column sizes.

The same effect of various geometric and structural properties on the seismic response in the transverse direction is then investigated for multi-span simply supported bridges. First, the effect of bearing stiffness, deck width and span length on the seismic capacity is studied. It is found that, the seismic capacity of two span simply supported bridges increases as the deck gets wider when bearings that develop rotational resistance at the supports are used. This effect becomes more pronounced as the rotational stiffness of the bearings-set increases, however, it vanishes for bridges with larger number of spans. For bridges with bearings-set of zero rotational stiffness, the maximum resistible peak ground acceleration is unaffected by the deck width, but greatly reduced compared to that of bridges with other types of bearings. Increasing span length is found to have a negative impact on the seismic capacity due to the higher moments exerted on the columns. Next, the effect of column size on the seismic capacity of multi-span simply supported bridges is studied. It is found that the increase in column size reduced the midspan deflection and expansion joint openings, and increased the seismic capacity significantly. It is also found that columns closer to the edge of the deck are the most vulnerable ones. The effect of column length on the seismic capacity is also studied. Results showed that, although the seismic capacity is increased slightly with the column length, bridges with longer columns are actually not superior to the ones with shorter columns since they are more susceptible to large openings in the expansion joints. Next the effect of number of spans on the seismic response of multi-span simply supported bridges is studied. It is found that in the case of bridges with bearings-set of zero rotational stiffness, the variation of maximum transverse displacement as a function of number of spans is not significant. For bridges with sliding-bearings, the maximum transverse displacement of two span bridges is much smaller than that of the ones with higher number of spans. However, the difference between the maximum transverse displacements of bridges with number of spans larger than two is not considerable.



Therefore, the transverse seismic capacities of these bridges are almost identical regardless of number of spans. It is also found that bridges with smaller number of spans are more vulnerable to seismic excitations than those with larger number of spans of identical end-to-end length. The condition of the expansion joints also have a considerable impact on the seismic capacity. Oddly enough, bridges with expansion joints rusted or plugged with debris due to weathering conditions, may survive larger earthquakes than otherwise.

The response of multi-span simply supported bridges in the longitudinal direction is studied. It is found that the columns can easily sustain displacements as large as twice the expansion joint width. Therefore, the columns in two span simply supported bridges are considered to be longitudinally safe. However, for simply supported bridges with more than two spans, the safety of columns in the longitudinal direction can not be assured since the system can potentially have a maximum displacement equal to the sum of the expansion joint widths. It is also found that, the peak ground accelerations required for collision in the longitudinal direction increases with span length, but they are all less than  $0.1g$ . Therefore, collision of the decks in the longitudinal direction is inevitable. Unfortunately, impacting between two adjacent sections of a bridge superstructure upon collision causes high shear forces in the bearings and therefore these components may fail before the columns (Zimmerman and Brittain, 1981; Imbsen and Penzien, 1986). Then, the system becomes unstable since the superstructure is disconnected from the columns and the simply supported decks may fall off their support if the seat width is not adequate. Therefore, impacting should certainly be prevented to avoid damage to bearings hence to the structure.

### **12.2.2 Bridges with Damaged Bearings**

First the effect of  $A_p/V_p$  ratio of the earthquakes on the sliding displacement of the bridges after the bearings are severed is investigated. It is found that the distribution of the energy content of an earthquake which is the area under acceleration time history is

an indication of how effective the earthquake is for causing high sliding displacements. Ground motions with high frequency content or high  $A_p/V_p$  ratio result in very low sliding displacements, whereas ground motions with intense long duration acceleration pulses or low  $A_p/V_p$  ratios can cause remarkable sliding displacements. Ground motions with highly irregular acceleration pulses which have intermediate  $A_p/V_p$  ratios produce medium sliding displacements. Based on these findings, single span simply supported and continuous bridges in Eastern Canada, where earthquakes generally have high  $A_p/V_p$  ratio, may have smaller sliding displacements than those in Western Canada where earthquakes generally have low  $A_p/V_p$  ratio.

The effect of various factors on the sliding displacement of simply supported and continuous bridges is also investigated. It is found that for the same ratio,  $\mu_f/A_p$ , of friction coefficient to peak ground acceleration, the sliding displacement is linearly proportional to the amplitude of the peak ground acceleration. For the ranges of spans considered in this study, generally, the sliding displacement increases with increasing span length and decreasing friction coefficient, but, decreases as the bridge gets wider. For continuous bridges, the magnitude of the friction coefficient at the bearings is found to have considerable impact on the seismic capacity of the structure. As the coefficient of friction increases, the peak ground acceleration required for sliding also increases. Therefore, bridges with higher friction coefficient can sustain bigger earthquakes before sliding. Since the sliding displacement of continuous bridges decreases with increasing friction coefficient, less displacement at the column locations is produced, hence earthquakes with higher intensity are needed to damage the columns. The increase in column stiffness causes only a slight decrease in sliding displacement. The sliding displacement of continuous bridges with larger number of spans is greater than that of bridges with identical end-to-end length but smaller number of spans. This is mainly due to the reduced fraction of gravity load on the abutments which results in less friction resistance.

The effect of damage to bearings on the seismic response of multi-span simply supported bridges is investigated. It is found that high intensity earthquakes are required to slide these bridges when the bearings at the abutments are damaged. Therefore, sliding is not a serious problem in multi-span simply supported bridges. The most important impact of damage to bearings at the abutments is to reduce the seismic capacity of two span simply supported bridges due to large ensuing deflections at the column locations. However, damage to bearings on the columns may result in total collapse of the structure.

Additionally, the effect of horizontal and cross bracings as retrofitting elements for continuous and multi-span simply supported bridges is investigated. It is found that introduction of a cross bracing between the exterior bridge columns results in a significant reduction in the midspan displacement and the bearing forces, however, a significant increase in the axial load on the exterior columns. For multi-span simply supported bridges, using cross bracings alone is beneficial for the ranges of spans considered. For continuous bridges, they improve the capacity for spans longer than 55 metres. However, when used with larger columns, they would certainly increase the capacity of continuous bridges for all span ranges with the beneficial effect of reduction in bearing forces. When horizontal and cross bracings are used together, there is a considerable increase only in the seismic capacity of continuous bridges longer than 45 metres. However, this is beneficial even in the case of continuous bridges with shorter spans due to the drastic reduction in the bearing forces. The effect of pressure and uplift forces exerted by the bracing elements on the foundations should be considered when using cross bracings.

### **12.3 Effect of Seismic Loads - Conclusions**

In this study, threshold of damage to slab-on-girder steel bridges is expressed in simple parameters such as peak ground acceleration, span length, deck width, bearing stiffness etc.. A reliable, accurate, and rapid evaluation and ranking procedure is

developed in a format directly useable by practising engineers wishing to assess the seismic vulnerability of steel bridges.

#### **12.4 Recommendations For Future Research**

The methodology developed in the first part of this study is based on a few assumptions. In particular, the relationship between the slope of the S-N curve below the CAFL and the magnitude of the maximum stress range in a stress range spectrum is not well established. The slope of the S-N curve below the CAFL for a load spectrum represented by a Rayleigh type distribution is already known for detail categories E, F and C, but not for other detail categories. It should be determined experimentally. If found to be different than assumed herein, the effect of heavy trucks on the fatigue life of bridge components should be reassessed, although only minor changes are anticipated.

Assessment of the fatigue life of structural components with other detail categories but using a stress range spectrum representing a loading other than Rayleigh type would also be of interest. Stress range spectra representing the truck traffic of the past are also needed for a more accurate estimation of the remaining fatigue life of existing steel bridges.

In the second part of this research the seismic response of bridges is studied. However, the effect of foundation and abutments on this response is neglected and remains to be investigated. These findings could be implemented into the proposed methodology developed for ranking existing bridges. Also, a review of the literature on the cyclic behavior of steel members revealed that, research has been concentrated solely on sizes and geometry typically found in buildings. Consequently, only a few researchers have addressed the various aspects of weak column strong beam behavior, and further experimental research is needed to determine the cyclic behaviour of columns more representative of those in steel bridges. Additionally, experimental research on the

behaviour of continuous and multi-span simply supported steel bridge models is also desirable to confirm the results obtained from analytical studies.

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**Table 2.1 General properties of the model trucks**

| Truck #<br>(1) | Definition<br>(2) | Number of<br>axles<br>(3) | Axle width<br>(m)<br>(4) | Length<br>(m)<br>(5) | Weight<br>(kN)<br>(6) | Speed Limit<br>(km/h)<br>(7) | Travel Restrictions<br>(8)                                         |
|----------------|-------------------|---------------------------|--------------------------|----------------------|-----------------------|------------------------------|--------------------------------------------------------------------|
| 1              | Mobile crane      | 6                         | 2.5                      | 10.08                | 660                   | 25                           | Only the truck is on the bridge                                    |
| 2              | Mobile crane      | 7                         | 2.5                      | 16.96                | 810                   | 25                           | Only the truck is on the bridge                                    |
| 3              | Float             | 9                         | 2.0                      | 33.09                | 1176                  | 25                           | Only the truck is on the bridge<br>and travels down the centerline |
| 4              | Float             | 10                        | 1.8                      | 51.02                | 1314                  | 10                           | Only the truck is on the bridge<br>and travel in the traffic lane  |
| 5              | Scraper           | 2                         | 1.8                      | 10.97                | 586                   | 10                           | None                                                               |
| 6              | OHBD-truck        | 7                         | 1.8                      | 18.00                | 700                   | None                         | None                                                               |



Table 2.2 Weight distribution and axle spacing of the model trucks

| Axle # | Truck 1          |                  | Truck 2          |                  | Truck 3          |                  | Truck 4          |                  | Truck 5          |                  | OHBD-truck       |                  |
|--------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|
|        | Axle weight (kN) | Axle spacing (m) | Axle weight (kN) | Axle spacing (m) | Axle weight (kN) | Axle spacing (m) | Axle weight (kN) | Axle spacing (m) | Axle weight (kN) | Axle spacing (m) | Axle weight (kN) | Axle spacing (m) |
| (1)    | (2)              | (3)              | (4)              | (5)              | (6)              | (7)              | (8)              | (9)              | (10)             | (11)             | (12)             | (13)             |
| 1      | 112              |                  | 104              |                  | 60               |                  | 68               |                  | 264              |                  | 60               |                  |
| 2      | 112              | 2.87             | 104              | 2.54             | 128              | 4.37             | 146              | 2.87             | 322              | 10.97            | 140              | 3.60             |
| 3      | 100              | 1.70             | 122              | 2.21             | 128              | 1.42             | 146              | 1.82             |                  |                  | 140              | 1.20             |
| 4      | 100              | 2.11             | 122              | 1.78             | 158              | 4.22             | 118              | 6.04             |                  |                  | 200              | 6.00             |
| 5      | 118              | 1.70             | 122              | 1.78             | 158              | 1.37             | 146              | 3.04             |                  |                  | 160              | 7.20             |
| 6      | 118              | 1.70             | 118              | 7.37             | 138              | 15.16            | 146              | 1.82             |                  |                  |                  |                  |
| 7      |                  |                  | 118              | 1.27             | 138              | 1.37             | 136              | 28.96            |                  |                  |                  |                  |
| 8      |                  |                  |                  |                  | 134              | 3.81             | 136              | 1.52             |                  |                  |                  |                  |
| 9      |                  |                  |                  |                  | 134              | 1.37             | 136              | 3.43             |                  |                  |                  |                  |
| 10     |                  |                  |                  |                  |                  |                  | 136              | 1.52             |                  |                  |                  |                  |

Table 3.1 Properties of the modified bridges

| Bridge Name | Design Year | Span |       | Type                         | Width (m) | # of Design Lanes | Design Lane Width | Curb Width (m.) | # of Girders | Girder Spacing | Material Strength (MPa) |       | List of Modifications          |
|-------------|-------------|------|-------|------------------------------|-----------|-------------------|-------------------|-----------------|--------------|----------------|-------------------------|-------|--------------------------------|
|             |             | #    | L (m) |                              |           |                   |                   |                 |              |                | Steel                   | Conc. |                                |
| (1)         | (2)         | (3)  | (4)   | (5)                          | (6)       | (7)               | (8)               | (9)             | (10)         | (11)           | (12)                    | (13)  | (14)                           |
| MCB*-I      | 1958        | 1    | 22    | Slab-on- girder*             | 8.5       | 2                 | 3.75              | 0.45            | 4            | 2.0            | 230                     | 20    | Curb width reduced to 0.50 m.  |
| MCB*-II     | 1958        | 1    | 22    | Slab-on- girder*             | 12.5      | 3                 | 3.83              | 0.45            | 6            | 2.0            | 230                     | 20    | Number of lanes reduced to 2.  |
| SRB*-I      | 1970        | 1    | 24    | Slab-on-girder*<br>15° skew  | 12.4      | 3                 | 3.53              | 0.90            | 6            | 2.0            | 350                     | 27    | Curb width reduced to 0.50 m.  |
| SRB*-II     | 1970        | 1    | 24    | Slab-on- girder*<br>45° skew | 12.4      | 3                 | 3.53              | 0.90            | 6            | 2.0            | 350                     | 27    | Skew angle is increased to 45° |
| SSRB*       | 1960        | 2    | 9     | Slab-on- girder*             | 10.5      | 2                 | 4.25              | 1.00            | 5            | 2.3            | 230                     | 20    | Number of spans reduced to 2   |
| GB*         | 1945        | 1    | 28    | Trough truss                 | 7.70      | 2                 | 3.65              | -               | 5            | 1.8            | 230                     | 20    | skewness is removed            |
|             |             |      |       |                              |           |                   |                   |                 |              |                |                         |       | None                           |

+ All slab-on-girder bridges are composite

\* MCB : Mud Creek Bridge

\* SRB : Saugeen River Bridge

\* SSRB : South Saugeen River Bridge

\* GB : Grigg Bridge

Table 3.2 Analysis results for Serviceability Limit State I

| Bridge Name | Truck # | Maximum<br>Moment Ranges<br>$M_r$ (kN-m) | $\frac{M}{M_{r-OHBD C}}$ | DMF  | DMF $\times M_r$<br>(kN-m) | $\frac{DMF \times M_r}{(DMF \times M_r)_{OHBD C}}$ |
|-------------|---------|------------------------------------------|--------------------------|------|----------------------------|----------------------------------------------------|
| (1)         | (2)     | (3)                                      | (4)                      | (5)  | (6)                        | (7)                                                |
| MCB-I       | 1       | 1454                                     | 1.51                     | 1.2  | 1745                       | 1.30                                               |
|             | 2       | 1289                                     | 1.34                     | 1.2  | 1547                       | 1.15                                               |
|             | 3       | 1255                                     | 1.30                     | 1.2  | 1506                       | 1.12                                               |
|             | 4       | 1405                                     | 1.46                     | 1.12 | 1574                       | 1.17                                               |
|             | 5       | 931                                      | 0.97                     | 1.12 | 1043                       | 0.77                                               |
|             | OHBDT   | 1062                                     | 1.10                     | 1.4  | 1487                       | 1.10                                               |
|             | OHBD C  | 962                                      | 1.00                     | 1.4  | 1347                       | 1.00                                               |
| MCB-II      | 1       | 1418                                     | 1.56                     | 1.2  | 1702                       | 1.34                                               |
|             | 2       | 1362                                     | 1.50                     | 1.2  | 1634                       | 1.28                                               |
|             | 3       | 1224                                     | 1.35                     | 1.2  | 1469                       | 1.15                                               |
|             | 4       | 1368                                     | 1.51                     | 1.12 | 1532                       | 1.20                                               |
|             | 5       | 893                                      | 0.98                     | 1.12 | 1000                       | 0.79                                               |
|             | OHBDT   | 1023                                     | 1.13                     | 1.4  | 1432                       | 1.12                                               |
|             | OHBD C  | 909                                      | 1.00                     | 1.4  | 1273                       | 1.00                                               |
| SRB-I       | 1       | 1535                                     | 1.40                     | 1.2  | 1842                       | 1.20                                               |
|             | 2       | 1534                                     | 1.40                     | 1.2  | 1841                       | 1.20                                               |
|             | 3       | 1382                                     | 1.26                     | 1.2  | 1658                       | 1.08                                               |
|             | 4       | 1474                                     | 1.34                     | 1.12 | 1651                       | 1.08                                               |
|             | 5       | 989                                      | 0.90                     | 1.12 | 1108                       | 0.72                                               |
|             | OHBDT   | 1196                                     | 1.09                     | 1.4  | 1674                       | 1.09                                               |
|             | OHBD C  | 1096                                     | 1.00                     | 1.4  | 1534                       | 1.00                                               |
| SRB-II      | 1       | 1487                                     | 1.36                     | 1.2  | 1784                       | 1.16                                               |
|             | 2       | 1527                                     | 1.39                     | 1.2  | 1832                       | 1.19                                               |
|             | 3       | 1390                                     | 1.27                     | 1.2  | 1668                       | 1.09                                               |
|             | 4       | 1482                                     | 1.35                     | 1.12 | 1660                       | 1.08                                               |
|             | 5       | 970                                      | 0.89                     | 1.12 | 1086                       | 0.71                                               |
|             | OHBDT   | 1210                                     | 1.10                     | 1.4  | 1694                       | 1.10                                               |
|             | OHBD C  | 1096                                     | 1.00                     | 1.4  | 1534                       | 1.00                                               |

| Bridge Name       | Truck # | Maximum<br>Moment Ranges<br>$M_r$ (kN-m) | $\frac{M}{M_{r-OHBDC}}$ | DMF  | DMF $\times M_r$<br>(kN-m) | $\frac{DMF \times M_r}{(DMF \times M_r)_{OHBDC}}$ |
|-------------------|---------|------------------------------------------|-------------------------|------|----------------------------|---------------------------------------------------|
| (1)               | (2)     | (3)                                      | (4)                     | (5)  | (6)                        | (7)                                               |
| SSRB (M')         | 1       | 214                                      | 0.79                    | 1.15 | 246                        | 0.70                                              |
|                   | 2       | 224                                      | 0.83                    | 1.15 | 258                        | 0.74                                              |
|                   | 3       | 282                                      | 1.04                    | 1.15 | 324                        | 0.92                                              |
|                   | 4       | 294                                      | 1.09                    | 1.09 | 320                        | 0.91                                              |
|                   | 5       | 183                                      | 0.68                    | 1.09 | 199                        | 0.57                                              |
|                   | OHBDT   | 250                                      | 0.93                    | 1.3  | 325                        | 0.93                                              |
|                   | OHBDC   | 270                                      | 1.00                    | 1.3  | 351                        | 1.00                                              |
| SSRB (M')         | 1       | 257                                      | 0.74                    | 1.15 | 296                        | 0.66                                              |
|                   | 2       | 253                                      | 0.73                    | 1.15 | 291                        | 0.65                                              |
|                   | 3       | 318                                      | 0.92                    | 1.15 | 366                        | 0.81                                              |
|                   | 4       | 306                                      | 0.88                    | 1.09 | 334                        | 0.74                                              |
|                   | 5       | 306                                      | 0.88                    | 1.09 | 334                        | 0.74                                              |
|                   | OHBDT   | 272                                      | 0.79                    | 1.3  | 354                        | 0.79                                              |
|                   | OHBDC   | 346                                      | 1.00                    | 1.3  | 450                        | 1.00                                              |
| GB<br>(stringers) | 1       | 93                                       | 0.91                    | 1.15 | 107                        | 0.80                                              |
|                   | 2       | 94                                       | 0.92                    | 1.15 | 108                        | 0.81                                              |
|                   | 3       | 112                                      | 1.10                    | 1.15 | 129                        | 0.97                                              |
|                   | 4       | 106                                      | 1.04                    | 1.09 | 116                        | 0.87                                              |
|                   | 5       | 150                                      | 1.47                    | 1.09 | 164                        | 1.23                                              |
|                   | OHBDT   | 86                                       | 0.84                    | 1.3  | 112                        | 0.84                                              |
|                   | OHBDC   | 102                                      | 1.00                    | 1.3  | 133                        | 1.00                                              |

Table 3.3 Analysis results for Mud Creek Bridge I

| Truck #<br>(1)                                           | 1<br>(2) | 2<br>(3) | 3<br>(4) | 4<br>(5) | 5<br>(6) | OHB DT<br>(7) | OHB DC<br>(8) |
|----------------------------------------------------------|----------|----------|----------|----------|----------|---------------|---------------|
| $M_e$ (kN-m)                                             | 1454     | 1289     | 1255     | 1405     | 931      | 1212          | 1002          |
| $M_i$ (kN-m)                                             | 769      | 686      | 728      | 709      | 555      | 936           | 1027          |
| $M_e/(M_e)_{OHBDC}$                                      | 1.45     | 1.29     | 1.25     | 1.40     | 0.93     | 1.21          | 1.00          |
| $M_i/(M_i)_{OHBDC}$                                      | 0.75     | 0.67     | 0.71     | 0.69     | 0.54     | 0.91          | 1.00          |
| DMF                                                      | 1.20     | 1.20     | 1.20     | 1.12     | 1.12     | 1.40          | 1.40          |
| $DMF \times M_e$ (kN-m)                                  | 1745     | 1547     | 1506     | 1574     | 1043     | 1697          | 1403          |
| $DMF \times M_i$ (kN-m)                                  | 923      | 823      | 874      | 794      | 622      | 1310          | 1438          |
| $\frac{DMF \times M_e}{(DMF \times M_e)_{OHBDC}}$        | 1.24     | 1.10     | 1.07     | 1.12     | 0.74     | 1.21          | 1.00          |
| $\frac{DMF \times M_i}{(DMF \times M_i)_{OHBDC}}$        | 0.64     | 0.57     | 0.61     | 0.55     | 0.43     | 0.91          | 1.00          |
| $\alpha_L^R = \frac{\phi M_R - 1.2 M_D}{DMF \times M_e}$ | 1.13     | 1.27     | 1.30     | 1.25     | 1.88     | 1.16          | 1.40          |
| $\alpha_L^R = \frac{\phi M_R - 1.2 M_D}{DMF \times M_i}$ | 2.42     | 2.71     | 2.55     | 2.81     | 3.59     | 1.70          | 1.55          |
| $SF = \frac{M_R}{M_D + DMF \times M_e}$                  | 1.28     | 1.38     | 1.40     | 1.36     | 1.70     | 1.31          | 1.46          |
| $SF = \frac{M_R}{M_D + DMF \times M_i}$                  | 2.01     | 2.13     | 2.07     | 2.17     | 2.41     | 1.66          | 1.57          |

Table 3.4 Analysis results for Mud Creek Bridge II

| Truck #<br>(1)                                           | 1<br>(2) | 2<br>(3) | 3<br>(4) | 4<br>(5) | 5<br>(6) | OHBDT<br>(7) | OHBDC<br>(8) |
|----------------------------------------------------------|----------|----------|----------|----------|----------|--------------|--------------|
| $M_u$ (kN-m)                                             | 1418     | 1362     | 1224     | 1368     | 893      | 1240         | 1016         |
| $M_i$ (kN-m)                                             | 759      | 737      | 722      | 702      | 550      | 935          | 975          |
| $M_u/(M_u)_{OHBDT}$                                      | 1.40     | 1.34     | 1.20     | 1.35     | 0.88     | 1.22         | 1.00         |
| $M_i/(M_i)_{OHBDT}$                                      | 0.78     | 0.76     | 0.74     | 0.72     | 0.56     | 0.96         | 1.00         |
| DMF                                                      | 1.20     | 1.20     | 1.20     | 1.12     | 1.12     | 1.40         | 1.40         |
| $DMF \times M_u$ (kN-m)                                  | 1702     | 1634     | 1469     | 1532     | 1000     | 1736         | 1422         |
| $DMF \times M_i$ (kN-m)                                  | 911      | 884      | 866      | 786      | 616      | 1309         | 1365         |
| $\frac{DMF \times M_u}{(DMF \times M_u)_{OHBDT}}$        | 1.20     | 1.15     | 1.03     | 1.08     | 0.70     | 1.22         | 1.00         |
| $\frac{DMF \times M_i}{(DMF \times M_i)_{OHBDT}}$        | 0.67     | 0.65     | 0.63     | 0.58     | 0.45     | 0.96         | 1.00         |
| $\alpha_L^E = \frac{\phi M_R - 1.2 M_D}{DMF \times M_u}$ | 1.17     | 1.22     | 1.36     | 1.30     | 1.99     | 1.15         | 1.40         |
| $\alpha_L^E = \frac{\phi M_R - 1.2 M_D}{DMF \times M_i}$ | 2.48     | 2.55     | 2.61     | 2.87     | 3.67     | 1.73         | 1.65         |
| $SF = \frac{M_R}{M_D + DMF \times M_u}$                  | 1.31     | 1.35     | 1.43     | 1.40     | 1.74     | 1.30         | 1.46         |
| $SF = \frac{M_R}{M_D + DMF \times M_i}$                  | 2.04     | 2.07     | 2.09     | 2.19     | 2.44     | 1.68         | 1.64         |

Table 3.5 Analysis results for Saugeen River Bridge 1

| Truck #<br>(1)                                           | 1<br>(2) | 2<br>(3) | 3<br>(4) | 4<br>(5) | 5<br>(6) | OHBDT<br>(7) | OHBDC<br>(8) |
|----------------------------------------------------------|----------|----------|----------|----------|----------|--------------|--------------|
| $M_u$ (kN-m)                                             | 1535     | 1534     | 1382     | 1474     | 989      | 1463         | 1206         |
| $M_l$ (kN-m)                                             | 693      | 683      | 662      | 653      | 501      | 904          | 1127         |
| $M_u/(M_u)_{OHBDT}$                                      | 1.27     | 1.27     | 1.15     | 1.22     | 0.82     | 1.21         | 1.00         |
| $M_l/(M_l)_{OHBDT}$                                      | 0.61     | 0.61     | 0.59     | 0.58     | 0.44     | 0.80         | 1.00         |
| DMF                                                      | 1.20     | 1.20     | 1.20     | 1.12     | 1.12     | 1.40         | 1.40         |
| $DMF \times M_u$ (kN-m)                                  | 1842     | 1841     | 1658     | 1651     | 1108     | 2048         | 1688         |
| $DMF \times M_l$ (kN-m)                                  | 832      | 820      | 794      | 731      | 561      | 1267         | 1578         |
| $\frac{DMF \times M_u}{(DMF \times M_u)_{OHBDT}}$        | 1.09     | 1.09     | 0.98     | 0.98     | 0.66     | 1.21         | 1.00         |
| $\frac{DMF \times M_l}{(DMF \times M_l)_{OHBDT}}$        | 0.53     | 0.52     | 0.50     | 0.46     | 0.36     | 0.80         | 1.00         |
| $\alpha_L^E = \frac{\phi M_R - 1.2 M_D}{DMF \times M_u}$ | 2.01     | 2.01     | 2.24     | 2.25     | 3.35     | 1.81         | 2.20         |
| $\alpha_L^E = \frac{\phi M_R - 1.2 M_D}{DMF \times M_l}$ | 5.31     | 5.39     | 5.56     | 6.04     | 7.87     | 3.49         | 2.30         |
| $SF = \frac{M_R}{M_D + DMF \times M_u}$                  | 1.85     | 1.85     | 1.95     | 1.96     | 2.33     | 1.75         | 1.94         |
| $SF = \frac{M_R}{M_D + DMF \times M_l}$                  | 3.34     | 3.36     | 3.41     | 3.52     | 3.87     | 2.74         | 2.43         |

Table 3.6 Analysis results for Saugeen River Bridge II

| Truck #<br>(1)                                           | 1<br>(2) | 2<br>(3) | 3<br>(4) | 4<br>(5) | 5<br>(6) | OHB DT<br>(7) | OHB DC<br>(8) |
|----------------------------------------------------------|----------|----------|----------|----------|----------|---------------|---------------|
| $M_u$ (kN-m)                                             | 1487     | 1527     | 1390     | 1482     | 970      | 1477          | 1206          |
| $M_i$ (kN-m)                                             | 660      | 648      | 656      | 648      | 483      | 891           | 1127          |
| $M_u/(M_u)_{OHBDC}$                                      | 1.23     | 1.27     | 1.15     | 1.23     | 0.80     | 1.22          | 1.00          |
| $M_i/(M_i)_{OHBDC}$                                      | 0.59     | 0.57     | 0.58     | 0.57     | 0.43     | 0.79          | 1.00          |
| DMF                                                      | 1.20     | 1.20     | 1.20     | 1.12     | 1.12     | 1.40          | 1.40          |
| $DMF \times M_u$ (kN-m)                                  | 1784     | 1832     | 1668     | 1660     | 1086     | 2068          | 1688          |
| $DMF \times M_i$ (kN-m)                                  | 792      | 778      | 787      | 726      | 541      | 1247          | 1578          |
| $\frac{DMF \times M_u}{(DMF \times M_u)_{OHBDC}}$        | 1.06     | 1.09     | 0.99     | 0.98     | 0.64     | 1.22          | 1.00          |
| $\frac{DMF \times M_i}{(DMF \times M_i)_{OHBDC}}$        | 0.50     | 0.49     | 0.50     | 0.46     | 0.34     | 0.79          | 1.00          |
| $\alpha_L^E = \frac{\phi M_R - 1.2 M_D}{DMF \times M_u}$ | 2.08     | 2.02     | 2.22     | 2.23     | 3.42     | 1.79          | 2.20          |
| $\alpha_L^E = \frac{\phi M_R - 1.2 M_D}{DMF \times M_i}$ | 5.58     | 5.68     | 5.61     | 6.08     | 8.16     | 3.54          | 2.80          |
| $SF = \frac{M_R}{M_D + DMF \times M_u}$                  | 1.88     | 1.86     | 1.95     | 1.95     | 2.35     | 1.74          | 1.94          |
| $SF = \frac{M_R}{M_D + DMF \times M_i}$                  | 3.41     | 3.43     | 3.42     | 3.53     | 3.91     | 2.77          | 2.43          |



Table 3.7 Analysis results for South Saugeen River Bridge (M')

| Truck #<br>(1)                                         | 1<br>(2) | 2<br>(3) | 3<br>(4) | 4<br>(5) | 5<br>(6) | OHBDT<br>(7) | OHBDT<br>(8) |
|--------------------------------------------------------|----------|----------|----------|----------|----------|--------------|--------------|
| $M_e$ (kN-m)                                           | 155      | 177      | 124      | 220      | 131      | 142          | 242          |
| $M_i$ (kN-m)                                           | 214      | 224      | 282      | 294      | 183      | 310          | 322          |
| $M_e/(M_e)_{OHBDT}$                                    | 0.64     | 0.73     | 0.51     | 0.91     | 0.54     | 0.59         | 1.00         |
| $M_i/(M_i)_{OHBDT}$                                    | 0.66     | 0.70     | 0.88     | 0.91     | 0.57     | 0.96         | 1.00         |
| DMF                                                    | 1.15     | 1.15     | 1.15     | 1.09     | 1.09     | 1.3          | 1.3          |
| $DMF \times M_e$ (kN-m)                                | 178      | 204      | 143      | 240      | 143      | 185          | 315          |
| $DMF \times M_i$ (kN-m)                                | 246      | 258      | 324      | 320      | 199      | 403          | 418          |
| $\frac{DMF \times M_e}{(DMF \times M_e)_{OHBDT}}$      | 0.57     | 0.65     | 0.45     | 0.76     | 0.45     | 0.59         | 1.00         |
| $\frac{DMF \times M_i}{(DMF \times M_i)_{OHBDT}}$      | 0.59     | 0.62     | 0.78     | 0.77     | 0.48     | 0.96         | 1.00         |
| $\alpha_L = \frac{\phi M_R - 1.2 M_D}{DMF \times M_e}$ | 3.06     | 2.67     | 3.81     | 2.27     | 3.81     | 2.95         | 1.73         |
| $\alpha_L = \frac{\phi M_R - 1.2 M_D}{DMF \times M_i}$ | 2.48     | 2.36     | 1.88     | 1.91     | 3.06     | 1.51         | 1.46         |
| $SF = \frac{M_R}{M_D + DMF \times M_e}$                | 2.27     | 2.13     | 2.49     | 1.96     | 2.49     | 2.23         | 1.68         |
| $SF = \frac{M_R}{M_D + DMF \times M_i}$                | 2.19     | 2.13     | 1.84     | 1.85     | 2.48     | 1.58         | 1.54         |

Table 3.8 Analysis results for South Saugeen River Bridge (M')

| Truck #<br>(1)                                         | 1<br>(2) | 2<br>(3) | 3<br>(4) | 4<br>(5) | 5<br>(6) | OHBDT<br>(7) | OHBDC<br>(8) |
|--------------------------------------------------------|----------|----------|----------|----------|----------|--------------|--------------|
| $M_u$ (kN-m)                                           | 197      | 197      | 177      | 225      | 217      | 180          | 250          |
| $M_i$ (kN-m)                                           | 208      | 208      | 260      | 248      | 255      | 311          | 317          |
| $M_u/(M_u)_{OHBDT}$                                    | 0.79     | 0.79     | 0.71     | 0.90     | 0.87     | 0.72         | 1.00         |
| $M_i/(M_i)_{OHBDT}$                                    | 0.66     | 0.66     | 0.82     | 0.78     | 0.80     | 0.98         | 1.00         |
| DMF                                                    | 1.15     | 1.15     | 1.15     | 1.09     | 1.09     | 1.3          | 1.3          |
| $DMF \times M_u$ (kN-m)                                | 227      | 227      | 204      | 245      | 237      | 234          | 325          |
| $DMF \times M_i$ (kN-m)                                | 239      | 239      | 299      | 270      | 278      | 404          | 412          |
| $\frac{DMF \times M_u}{(DMF \times M_u)_{OHBDT}}$      | 0.70     | 0.70     | 0.63     | 0.75     | 0.73     | 0.72         | 1.00         |
| $\frac{DMF \times M_i}{(DMF \times M_i)_{OHBDT}}$      | 0.58     | 0.58     | 0.73     | 0.66     | 0.67     | 0.98         | 1.00         |
| $\alpha_L = \frac{\phi M_R - 1.2 M_D}{DMF \times M_u}$ | 5.96     | 5.96     | 6.64     | 5.52     | 5.71     | 5.78         | 4.16         |
| $\alpha_L = \frac{\phi M_R - 1.2 M_D}{DMF \times M_i}$ | 5.80     | 5.80     | 4.64     | 5.13     | 4.99     | 3.43         | 3.36         |
| $SF = \frac{M_R}{M_D + DMF \times M_u}$                | 4.93     | 4.93     | 5.30     | 4.68     | 4.79     | 4.83         | 3.81         |
| $SF = \frac{M_R}{M_D + DMF \times M_i}$                | 5.16     | 5.16     | 4.35     | 4.71     | 4.60     | 3.40         | 3.35         |

Table 3.9 Analysis results for Grigg Bridge (stringers)

| Truck #<br>(1)                                           | 1<br>(2) | 2<br>(3) | 3<br>(4) | 4<br>(5) | 5<br>(6) | OHBDT<br>(7) | OHBDT<br>(8) |
|----------------------------------------------------------|----------|----------|----------|----------|----------|--------------|--------------|
| $M_e$ (kN-m)                                             | 93       | 94       | 97       | 106      | 150      | 103          | 101          |
| $M_i$ (kN-m)                                             | 66       | 67       | 112      | 78       | 121      | 126          | 118          |
| $M_e/(M_e)_{OHBDT}$                                      | 0.92     | 0.93     | 0.96     | 1.05     | 1.49     | 1.02         | 1.0          |
| $M_i/(M_i)_{OHBDT}$                                      | 0.56     | 0.57     | 0.95     | 0.66     | 1.03     | 1.07         | 1.0          |
| DMF                                                      | 1.15     | 1.15     | 1.15     | 1.09     | 1.09     | 1.30         | 1.30         |
| $DMF \times M_e$ (kN-m)                                  | 107      | 108      | 112      | 116      | 164      | 134          | 131          |
| $DMF \times M_i$ (kN-m)                                  | 76       | 77       | 129      | 85       | 132      | 164          | 153          |
| $\frac{DMF \times M_e}{(DMF \times M_e)_{OHBDT}}$        | 0.82     | 0.82     | 0.85     | 0.89     | 1.25     | 1.02         | 1.00         |
| $\frac{DMF \times M_i}{(DMF \times M_i)_{OHBDT}}$        | 0.50     | 0.50     | 0.84     | 0.56     | 0.86     | 1.07         | 1.00         |
| $\alpha_L^E = \frac{\phi M_R - 1.2 M_D}{DMF \times M_e}$ | 2.18     | 2.16     | 2.08     | 2.01     | 1.42     | 1.74         | 1.78         |
| $\alpha_L^E = \frac{\phi M_R - 1.2 M_D}{DMF \times M_i}$ | 2.94     | 2.90     | 1.73     | 2.63     | 1.69     | 1.36         | 1.46         |
| $SF = \frac{M_R}{M_D + DMF \times M_e}$                  | 2.21     | 2.20     | 2.13     | 2.07     | 1.54     | 1.84         | 1.87         |
| $SF = \frac{M_R}{M_D + DMF \times M_i}$                  | 2.68     | 2.65     | 1.80     | 2.47     | 1.77     | 1.48         | 1.57         |

Table 3.10 Magnified exterior girder moments

| Bridge Name            | Permit Truck # | Influence Line Analysis                  |                                                   | Magnification Factors<br>(4)/(3) | $\frac{M_e}{(M_e)_{OHBDG}}$ |                             |
|------------------------|----------------|------------------------------------------|---------------------------------------------------|----------------------------------|-----------------------------|-----------------------------|
|                        |                | $\frac{M}{M_{OHBDT}}$<br>At Current Span | $(\frac{M}{M_{OHBDT}})_{max}$<br>At Critical Span |                                  | At Current Span             | (5)x(6)<br>At Critical Span |
| (1)                    | (2)            | (3)                                      | (4)                                               | (5)                              | (6)                         | (7)                         |
| MCB-I<br>Span 22.0 m.  | 1              | 1.25                                     | 1.27                                              | 1.02                             | 1.45                        | 1.48                        |
|                        | 2              | 1.20                                     | 1.25                                              | 1.04                             | 1.29                        | 1.34                        |
|                        | 3              | 1.15                                     | 1.22                                              | 1.06                             | 1.25                        | 1.33                        |
|                        | 4              | 1.11                                     | 1.19                                              | 1.07                             | 1.40                        | 1.50                        |
|                        | 5              | 0.86                                     | 1.65                                              | 1.92                             | 0.93                        | 1.79                        |
| MCB-II<br>Span 22.0 m. | 1              | 1.25                                     | 1.27                                              | 1.02                             | 1.40                        | 1.43                        |
|                        | 2              | 1.20                                     | 1.25                                              | 1.04                             | 1.34                        | 1.39                        |
|                        | 3              | 1.15                                     | 1.22                                              | 1.06                             | 1.20                        | 1.27                        |
|                        | 4              | 1.11                                     | 1.19                                              | 1.07                             | 1.35                        | 1.44                        |
|                        | 5              | 0.86                                     | 1.65                                              | 1.92                             | 0.88                        | 1.69                        |
| SRB-I<br>Span 24.0 m.  | 1              | 1.25                                     | 1.27                                              | 1.02                             | 1.27                        | 1.30                        |
|                        | 2              | 1.25                                     | 1.25                                              | 1.00                             | 1.27                        | 1.27                        |
|                        | 3              | 1.15                                     | 1.22                                              | 1.06                             | 1.15                        | 1.22                        |
|                        | 4              | 1.12                                     | 1.19                                              | 1.06                             | 1.22                        | 1.29                        |
|                        | 5              | 0.88                                     | 1.65                                              | 1.88                             | 0.82                        | 1.54                        |

| Bridge Name                  | Permit Truck # | Influence Line Analysis                 |                                                  | Magnification Factors<br>(4)/(3) | $\frac{M_s}{(M_s)_{OHBC}}$ |                             |
|------------------------------|----------------|-----------------------------------------|--------------------------------------------------|----------------------------------|----------------------------|-----------------------------|
|                              |                | $\frac{M}{M_{OHBC}}$<br>At Current Span | $(\frac{M}{M_{OHBC}})_{max}$<br>At Critical Span |                                  | At Current Span            | (5)x(6)<br>At Critical Span |
| (1)                          | (2)            | (3)                                     | (4)                                              | (5)                              | (6)                        | (7)                         |
| SRB-II<br>Span 24.0 m.       | 1              | 1.25                                    | 1.27                                             | 1.02                             | 1.23                       | 1.25                        |
|                              | 2              | 1.25                                    | 1.25                                             | 1.00                             | 1.27                       | 1.27                        |
|                              | 3              | 1.15                                    | 1.22                                             | 1.06                             | 1.15                       | 1.22                        |
|                              | 4              | 1.12                                    | 1.19                                             | 1.06                             | 1.23                       | 1.30                        |
|                              | 5              | 0.88                                    | 1.65                                             | 1.88                             | 0.80                       | 1.50                        |
| SSRB<br>Span 9.0 m.          | 1              | 1.05                                    | 1.15                                             | 1.10                             | 0.64                       | 0.70                        |
|                              | 2              | 1.12                                    | 1.28                                             | 1.14                             | 0.73                       | 0.83                        |
|                              | 3              | 1.12                                    | 1.58                                             | 1.41                             | 0.51                       | 0.72                        |
|                              | 4              | 1.21                                    | 1.31                                             | 1.08                             | 0.91                       | 0.98                        |
|                              | 5              | 0.95                                    | 1.50                                             | 1.58                             | 0.54                       | 0.85                        |
| GB stringers<br>Span 4.68 m. | 1              | 0.90                                    | 1.27                                             | 1.41                             | 0.92                       | 1.30                        |
|                              | 2              | 0.92                                    | 1.25                                             | 1.36                             | 0.93                       | 1.26                        |
|                              | 3              | 1.08                                    | 1.22                                             | 1.13                             | 0.96                       | 1.08                        |
|                              | 4              | 0.92                                    | 1.19                                             | 1.29                             | 1.05                       | 1.35                        |
|                              | 5              | 1.57                                    | 1.65                                             | 1.05                             | 1.49                       | 1.56                        |

**Table 5.1 ODR for various detail categories and design yield limits**

| Detail Category<br>(1) | $S_{all}$ (MPa)<br>(2) | $ODR = \frac{2.63 \sigma_{all}}{\sigma_y}$ |                              |                              |                              |
|------------------------|------------------------|--------------------------------------------|------------------------------|------------------------------|------------------------------|
|                        |                        | $\sigma_y=230$<br>MPa<br>(3)               | $\sigma_y=250$<br>MPa<br>(4) | $\sigma_y=300$<br>MPa<br>(5) | $\sigma_y=350$<br>MPa<br>(6) |
| a                      | 165                    | 1.89                                       | 1.74                         | 1.45                         | 1.24                         |
| b                      | 110                    | 1.26                                       | 1.16                         | 0.97                         | 0.83                         |
| c                      | 70                     | 0.80                                       | 0.74                         | 0.62                         | 0.53                         |
| d                      | 48                     | 0.55                                       | 0.50                         | 0.42                         | 0.36                         |
| e                      | 32                     | 0.37                                       | 0.34                         | 0.28                         | 0.24                         |
| f                      | 18                     | 0.21                                       | 0.19                         | 0.16                         | 0.14                         |

**Table 5.2 Expressions for safe and mean life of various detail categories**

| Detail Category<br>(1) | $N_c$ (life in cycles)<br>(2) | Safe life (Years)<br>(3) | Mean life (Years)<br>(4) |
|------------------------|-------------------------------|--------------------------|--------------------------|
| a                      | 9,223,786                     | $25 \times 10^3 / ADTT$  | $49 \times 10^3 / ADTT$  |
| b                      | 13,835,676                    | $38 \times 10^3 / ADTT$  | $73 \times 10^3 / ADTT$  |
| c                      | 20,753,516                    | $57 \times 10^3 / ADTT$  | $109 \times 10^3 / ADTT$ |
| d                      | 27,671,354                    | $76 \times 10^3 / ADTT$  | $146 \times 10^3 / ADTT$ |
| e                      | 39,201,088                    | $107 \times 10^3 / ADTT$ | $206 \times 10^3 / ADTT$ |
| f                      | 73,790,288                    | $202 \times 10^3 / ADTT$ | $388 \times 10^3 / ADTT$ |

**Table 6.1 Geometric properties of the designed bridges**

| All Bridges |                   |     |                 |     |                          | Simply Sup.<br>Bridges    | Continuous Bridges          |                             | Multi Span Simply<br>Sup. Bridges |                         |
|-------------|-------------------|-----|-----------------|-----|--------------------------|---------------------------|-----------------------------|-----------------------------|-----------------------------------|-------------------------|
| Span<br>(m) | Deck<br>Width (m) |     | # of<br>Girders |     | Girder<br>Spacing<br>(m) | Slab<br>thickness<br>(cm) | Exp. Joint<br>Width<br>(mm) | Exp. Joint<br>Width<br>(mm) | Column<br>Height<br>(m)           | Column<br>Height<br>(m) |
|             |                   |     | 2-L             | 3-L |                          |                           |                             |                             |                                   |                         |
| (1)         | (2)               | (3) | (4)             | (5) | (6)                      | (7)                       | (10)                        | (11)                        | (12)                              | (14)                    |
| 20          | 8                 | 12  | 4               | 6   | 2                        | 20                        | 20                          | 30                          | 6                                 | 6                       |
| 30          | 8                 | 12  | 4               | 6   | 2                        | 20                        | 25                          | 40                          | 6                                 | 6                       |
| 40          | 8                 | 12  | 4               | 6   | 2                        | 20                        | 30                          | 50                          | 6                                 | 6                       |
| 50          | 8                 | 12  | 4               | 6   | 2                        | 20                        | 35                          | 60                          | 6                                 | 6                       |
| 60          | 8                 | 12  | 4               | 6   | 2                        | 20                        | 40                          | 75                          | 6                                 | 6                       |

**Table 6.2 Structural properties of the designed bridges**

| All Bridges |                               |       | Simply Supported Bridges |            |              | Continuous Bridges |            |              | Multi Span Simply Supported Bridges |             |            |            |      |
|-------------|-------------------------------|-------|--------------------------|------------|--------------|--------------------|------------|--------------|-------------------------------------|-------------|------------|------------|------|
| Span<br>(m) | Material<br>Strength<br>(MPa) |       | Girder Sizes             | Mass (ton) | Girder Sizes | Column<br>Sizes    | Mass (ton) | Girder Sizes | Column Sizes                        | Mass (ton)  |            | Mass (ton) |      |
|             | Steel                         | Conc. |                          |            |              |                    |            |              |                                     | 2-L         | 3-L        |            | 2-L  |
| (1)         | (3)                           | (4)   | (5)                      | (6)        | (7)          | (8)                | (9)        | (10)         | (11)                                | (12)        | (13)       | (14)       | (15) |
| 20          | 350                           | 20    | WWF800x184               | 126        | 190          | WWF900x192         | W310x74    | 259          | 389                                 | WWF800x184  | W310x129   | 252        | 380  |
| 30          | 350                           | 20    | WWF1000x262              | 202        | 303          | WWF1000x262        | W310x79    | 406          | 609                                 | WWF1000x262 | W310x143   | 404        | 606  |
| 40          | 350                           | 20    | WWF1200x333              | 286        | 428          | WWF1200x380        | W310x97    | 578          | 867                                 | WWF1200x333 | WWF350x155 | 572        | 856  |
| 50          | 350                           | 20    | WWF1400x405              | 367        | 550          | WWF1600x496        | W310x97    | 769          | 1154                                | WWF1400x405 | WWF400x157 | 734        | 1100 |
| 60          | 350                           | 20    | WWF1600x496              | 465        | 697          | WWF1800x659        | W310x118   | 1001         | 1502                                | WWF1600x496 | WWF400x178 | 930        | 1394 |



**Table 6.3 Properties of the selected earthquakes**

| Site<br>(1)     | Location<br>(2)  | Date<br>(3) | Component<br>(4) | Magnitude<br>(5) | Source<br>Distance (km)<br>(6) | Soil<br>Type<br>(7) | Max.<br>Acc. (xg)<br>(8) |
|-----------------|------------------|-------------|------------------|------------------|--------------------------------|---------------------|--------------------------|
| Imperial Valley | El Centro        | 1940        | E00S             | 6.6              | 8                              | Stiff soil          | 0.348                    |
| Kern County     | Taft             | 1952        | S69E             | 7.6              | 56                             | Rock                | 0.179                    |
| San Fernando    | Pacoima Dam      | 1971        | S16E             | 6.6              | 8                              | Rock                | 1.170                    |
| Parkfield       | Chaloma Shand. 2 | 1966        | N65E             | 5.6              | 0.1                            | Stiff Soil          | 0.480                    |
| Québec          | Chicoutimi-Nord  | 1984        | N18E             | 6.0              | 43                             | Rock                | 0.131                    |
| Québec          | Baie-St-Paul     | 1982        | S59E             | 6.0              | 91                             | Aluvium             | 0.174                    |
| Türkiye         | Erzincan         | 1992        | NS               | 6.9              | 7                              | Aluvium             | 0.500                    |

**Table 10.1 Earthquake magnitudes and ratios of peak ground accelerations for different probabilities of exceedance in Eastern Canada**

| Region                | Area<br>(10 <sup>3</sup> km <sup>2</sup> ) | $\beta_r$ | $N_0$ | $M_z$ | $M_{10}$ | $M_5$ | $R_A = \frac{A_{p5}}{A_{p10}}$ | $R_A \cdot \text{Area}$ |
|-----------------------|--------------------------------------------|-----------|-------|-------|----------|-------|--------------------------------|-------------------------|
| (1)                   | (2)                                        | (3)       | (4)   | (5)   | (6)      | (7)   | (8)                            | (9)                     |
| Attica                | 2.62                                       | 1.32      | 11    | 6.0   | 5.680    | 5.827 | 1.211                          | 3.173                   |
| Charlevoix            | 6.88                                       | 1.66      | 310   | 7.5   | 6.894    | 7.131 | 1.361                          | 9.364                   |
| Lower St. Lawrence    | 24.5                                       | 1.85      | 533   | 6.0   | 5.875    | 5.935 | 1.082                          | 26.51                   |
| Northern Appalachians | 241                                        | 1.87      | 638   | 6.0   | 5.882    | 5.939 | 1.077                          | 259.6                   |
| Western Quebec        | 121                                        | 1.85      | 1030  | 7.0   | 6.665    | 6.810 | 1.210                          | 146.4                   |
| Laurentian Slope      | 15.2                                       | 1.30      | 41    | 7.5   | 7.010    | 7.225 | 1.315                          | 19.99                   |
| Weighted Average      |                                            |           |       |       |          |       |                                | 1.131                   |

**Table 10.1 Earthquake magnitudes and ratios of peak ground accelerations for different probabilities of exceedance in Western Canada**

| Region                  | Area<br>(10 <sup>3</sup> km <sup>2</sup> ) | $\beta_r$ | $N_0$ | $M_x$ | $M_{10}$ | $M_5$ | $R_A = \frac{A_{p5}}{A_{p10}}$ | $R_A \cdot \text{Area}$ |
|-------------------------|--------------------------------------------|-----------|-------|-------|----------|-------|--------------------------------|-------------------------|
| (1)                     | (2)                                        | (3)       | (4)   | (5)   | (6)      | (7)   | (8)                            | (9)                     |
| Cascades                | 145                                        | 1.87      | 1060  | 7.5   | 6.838    | 7.080 | 1.370                          | 3.173                   |
| Coast Mountains         | 139                                        | 1.77      | 272   | 6.5   | 6.178    | 6.320 | 1.203                          | 9.364                   |
| Juan de Fuca - Explorer | 84.8                                       | 1.72      | 7360  | 7.0   | 6.973    | 6.986 | 1.018                          | 26.51                   |
| Southern Saskatchewan   | 40                                         | 2.07      | 188   | 6.0   | 5.358    | 5.587 | 1.346                          | 259.6                   |
| Southeastern B.C.       | 255                                        | 2.28      | 3230  | 6.5   | 6.052    | 6.226 | 1.255                          | 146.4                   |
| Queen Charlotte Fault   | 46.4                                       | 1.50      | 1610  | 8.5   | 8.252    | 8.368 | 1.162                          | 19.99                   |
| Weighted Average        |                                            |           |       |       |          |       |                                | 1.239                   |

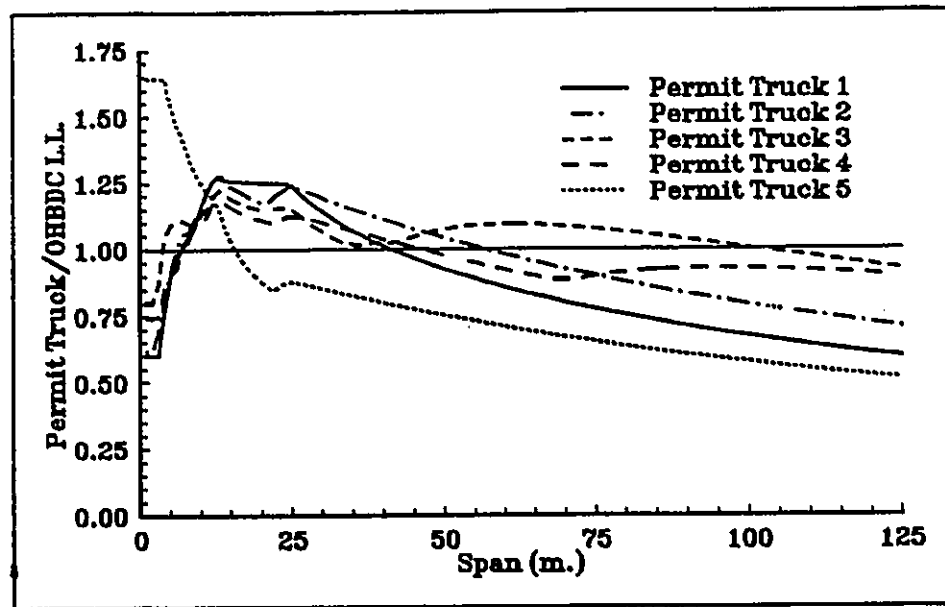


Figure 2.1 Live load ratios for simple span bridges

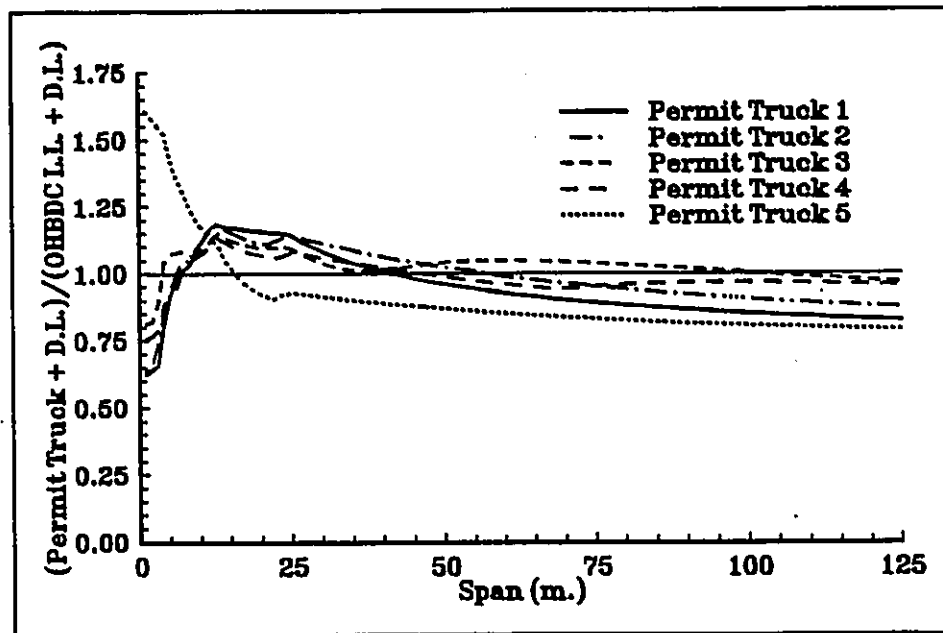


Figure 2.2 Total load ratios for simple span bridges

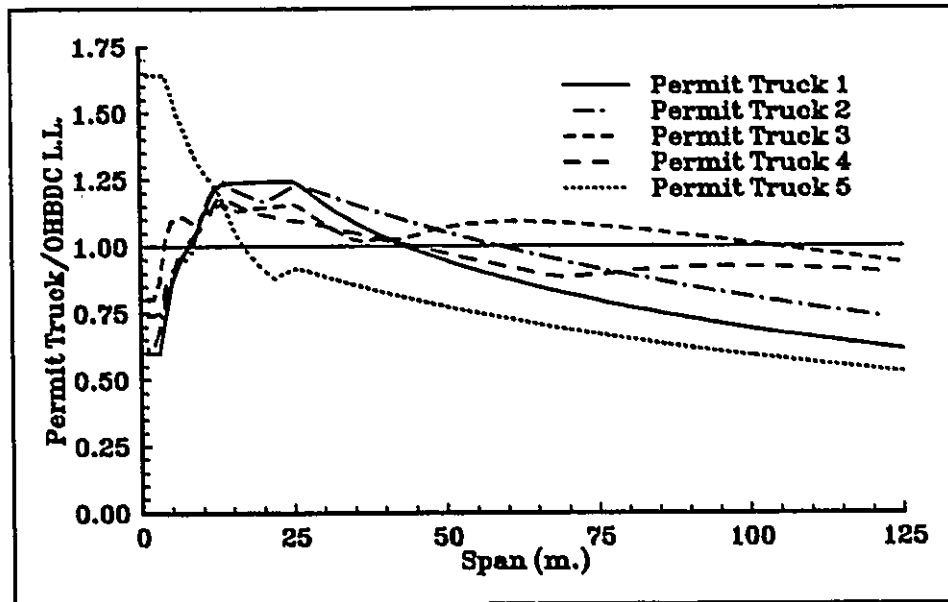


Figure 2.3 Live load ratios for continuous span bridges (midspan moment)

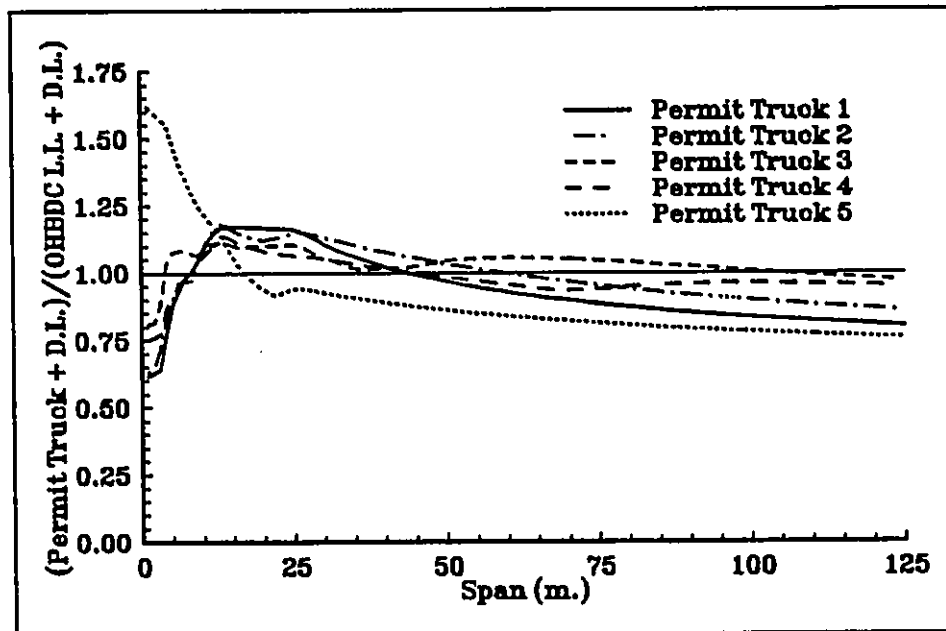


Figure 2.4 Total load ratios for continuous span bridges (midspan moment)

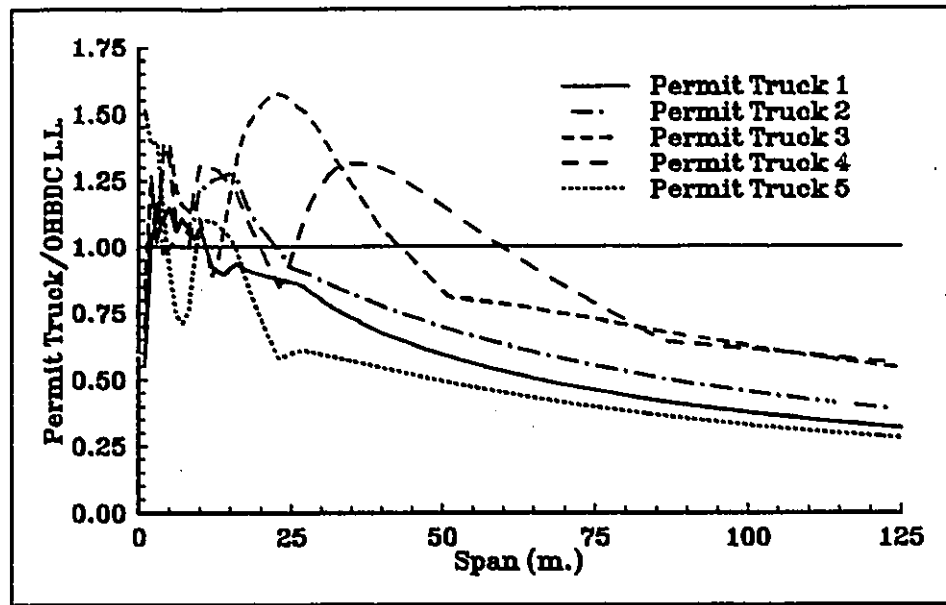


Figure 2.5 Live load ratios for continuous span bridges (support moment)

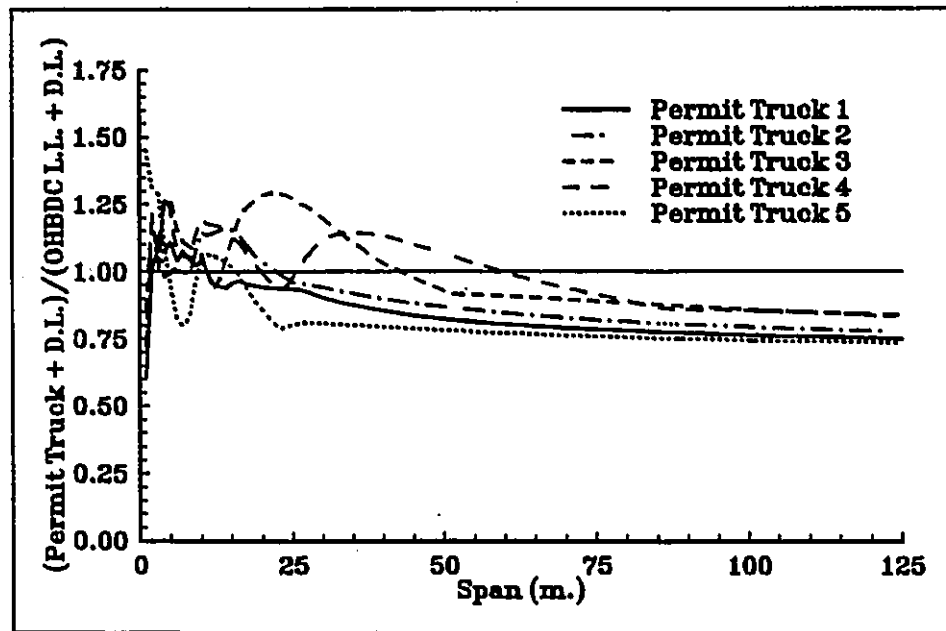


Figure 2.6 Total load ratios for continuous span bridges (support moment)

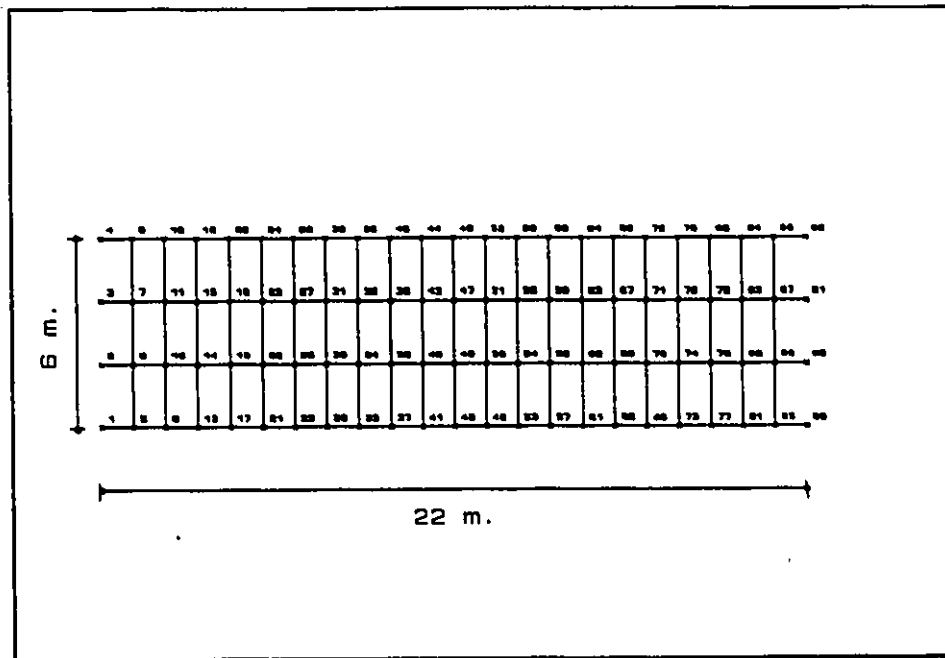


Figure 3.1 Mesh for the grillage analogy model of the MCB-I (Plan View)

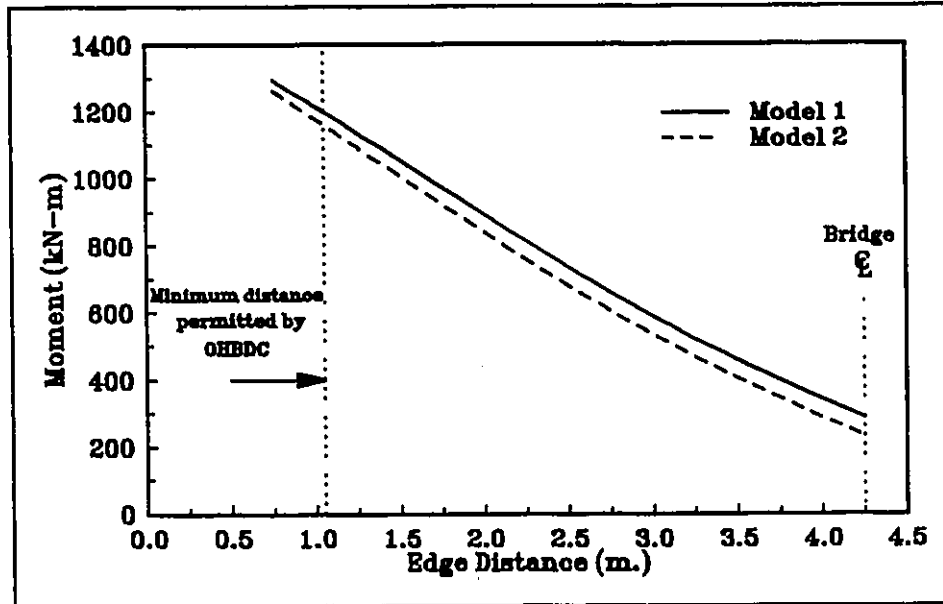


Figure 3.2 Variation of exterior girder moments as a function of edge distance for two different models of MCB-I

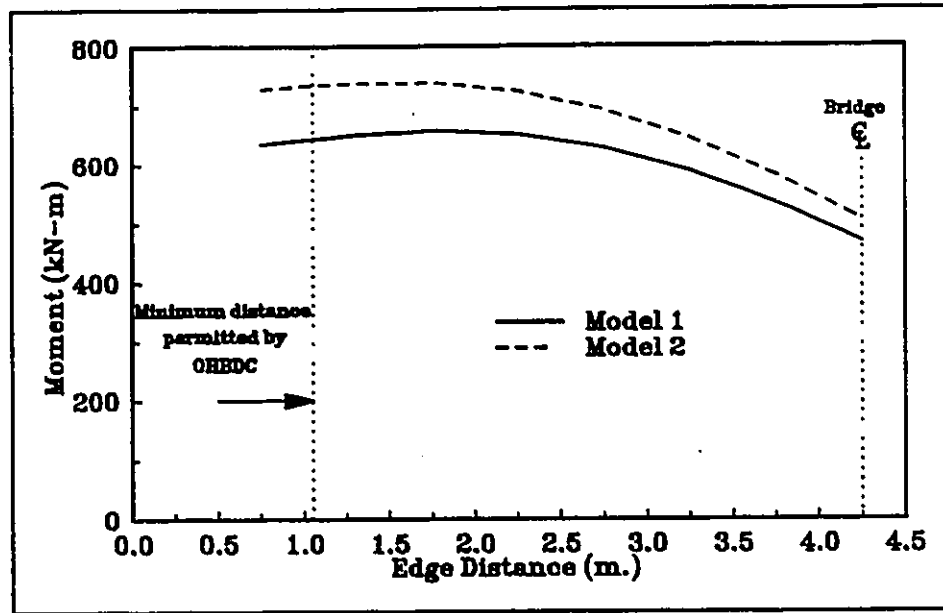


Figure 3.3 Variation of interior girder moment as a function of edge distance for two different models of MCB-I

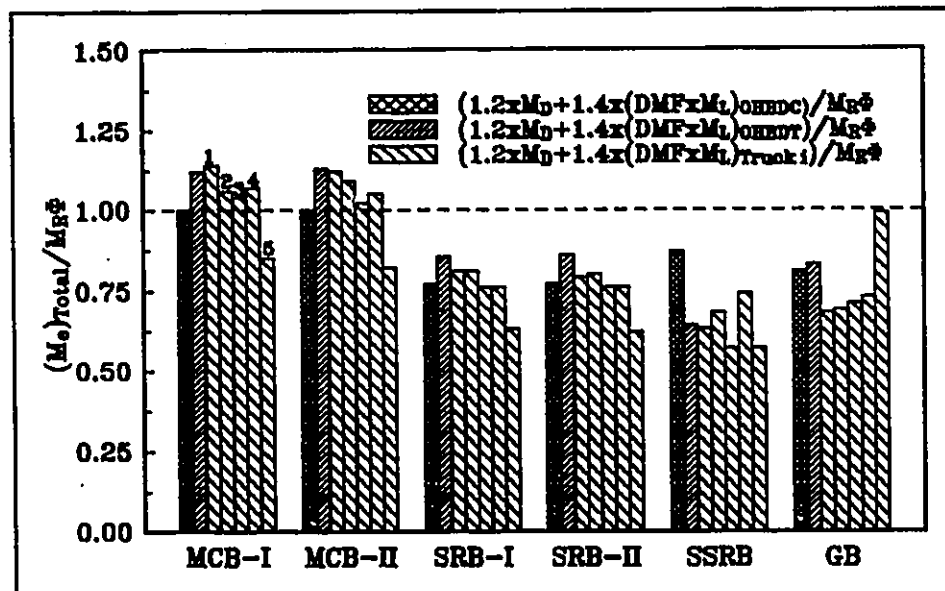


Figure 3.4a Ratio of total applied moment to the factored resistance of the bridges



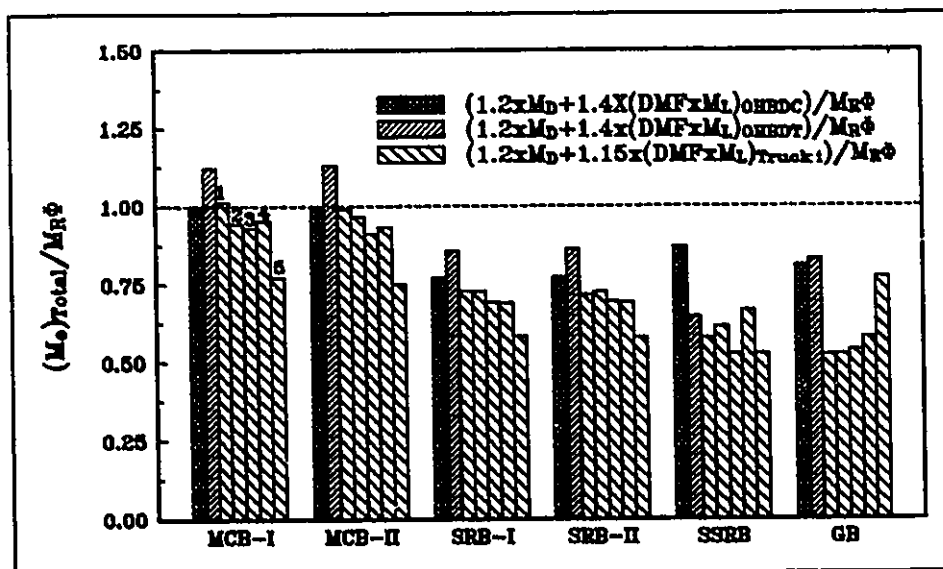


Figure 3.4b Ratio of total applied moment to the factored resistance of the bridges using live load factor of 1.15 (controlled vehicle)

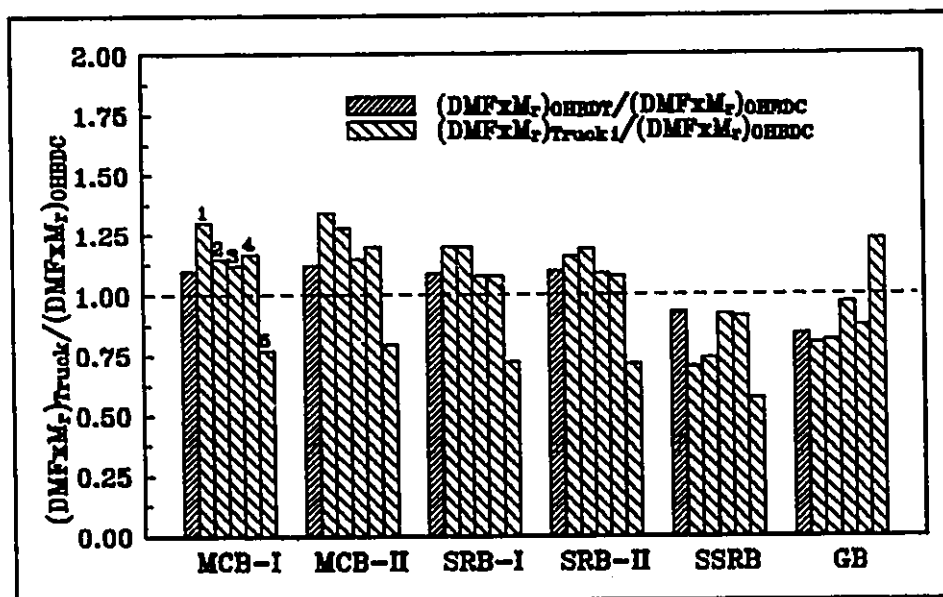


Figure 3.5a Ratio of the moments due to heavy permit-trucks to the moment obtained from OHBC

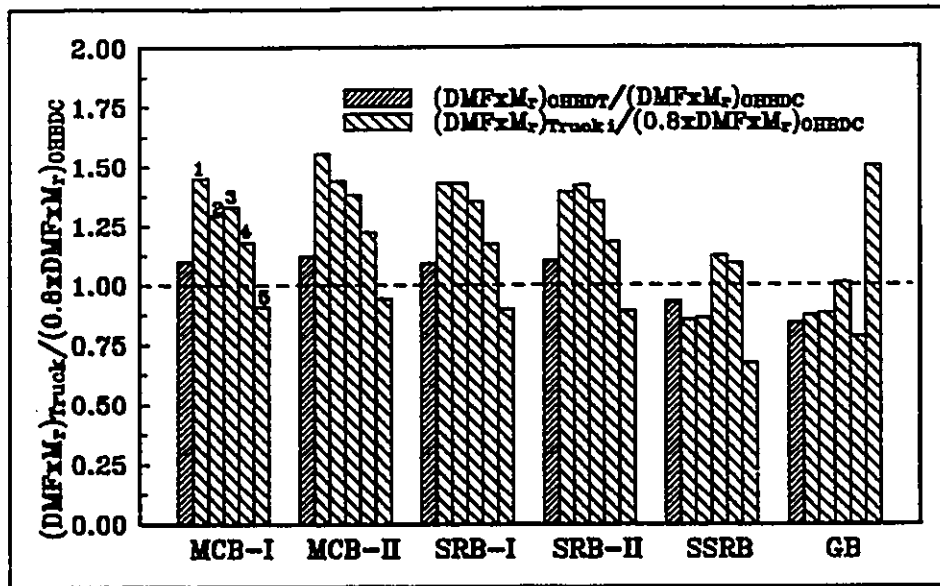


Figure 3.5b Ratio of the moments due to heavy permit-trucks to the moment obtained from OHBDC

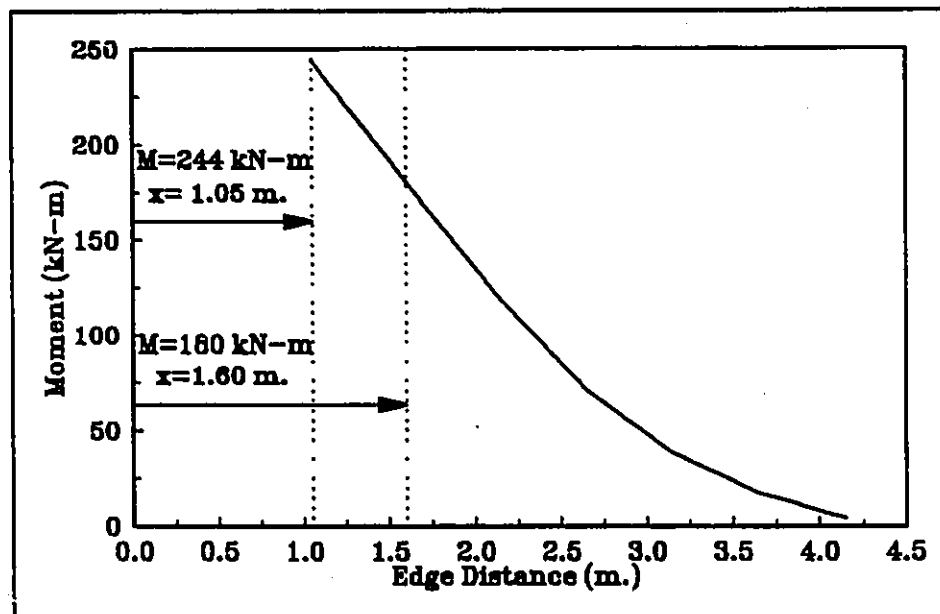


Figure 3.6 Variation of exterior girder positive moment at midspan of SSRB as a function of the lateral position of the first wheel line of the OHBD-truck

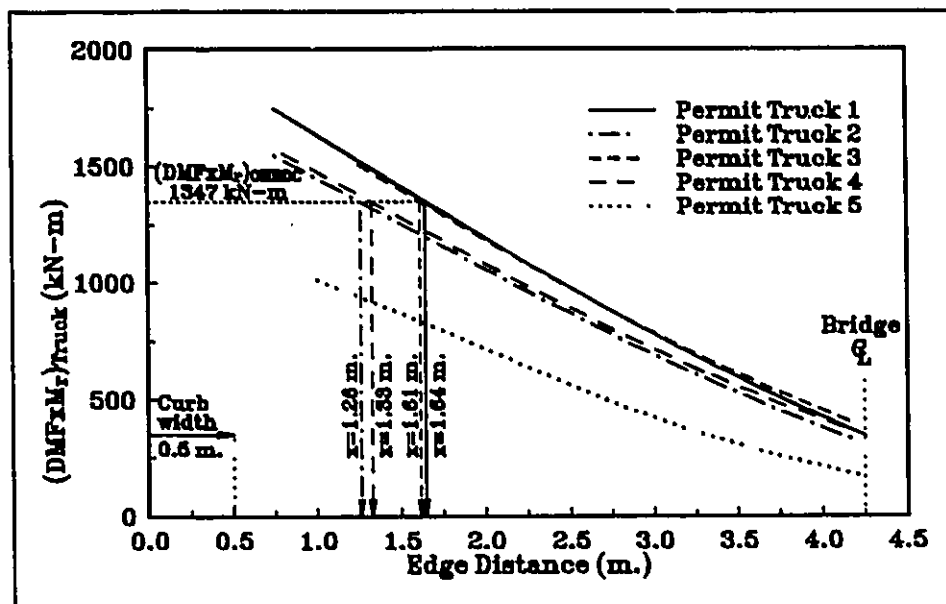


Figure 3.7 Variation of exterior girder moment of MCB-I as a function of the lateral position of the first wheel line of the permit-trucks

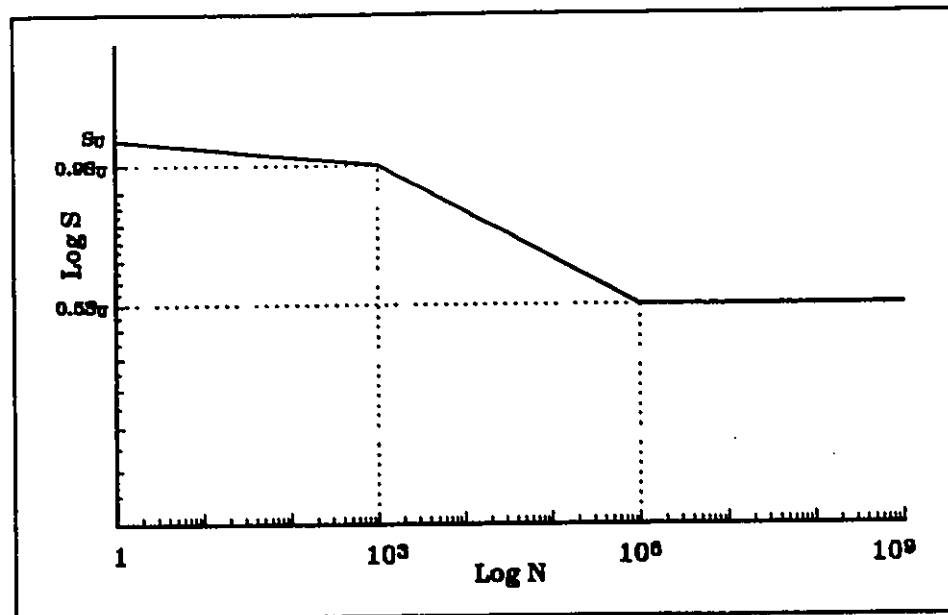


Figure 4.1 Typical S-N curve for steel.

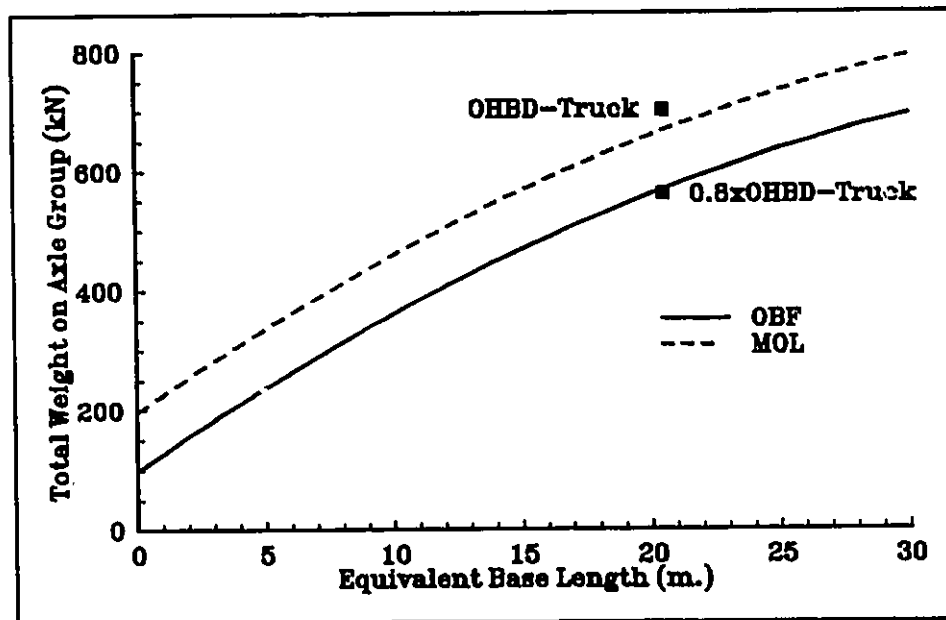


Figure 4.2 Comparison of OHBD-truck with MOL and OBF levels

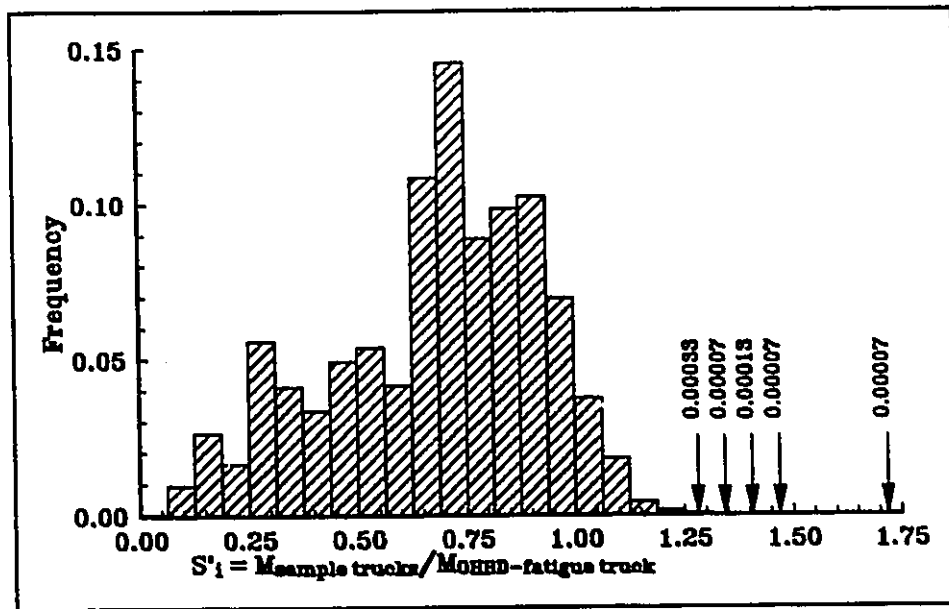


Figure 5.1 Average flexural stress range spectrum of the heavy portion of Ontario's truck traffic

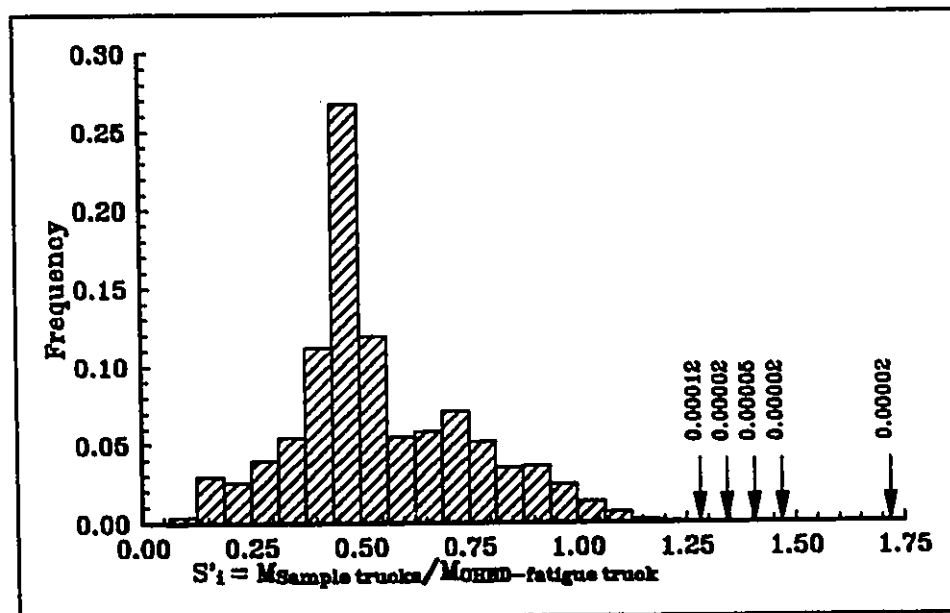


Figure 5.2 Average flexural stress range spectrum of Ontario's truck traffic

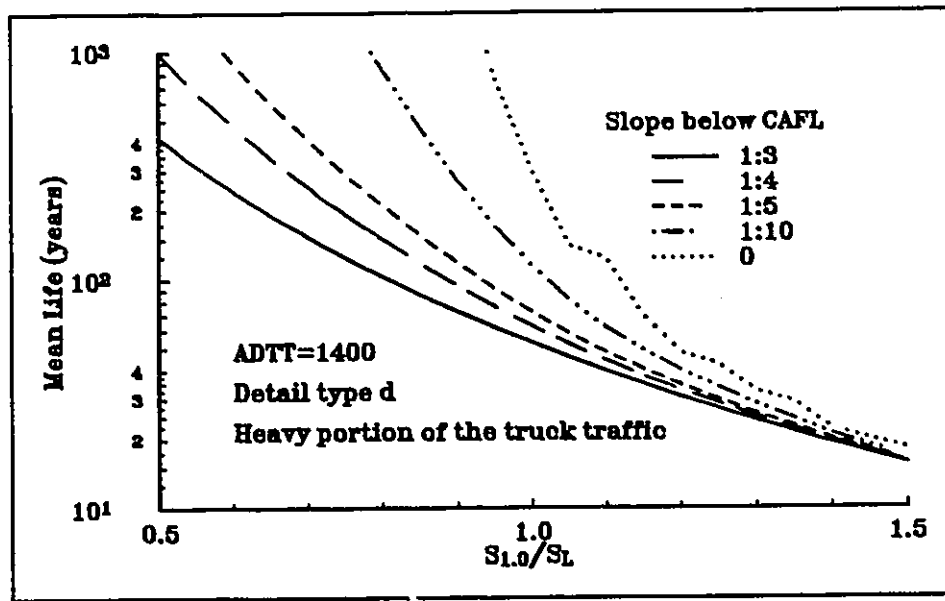


Figure 5.3 Mean fatigue life as a function of spectrum amplitude

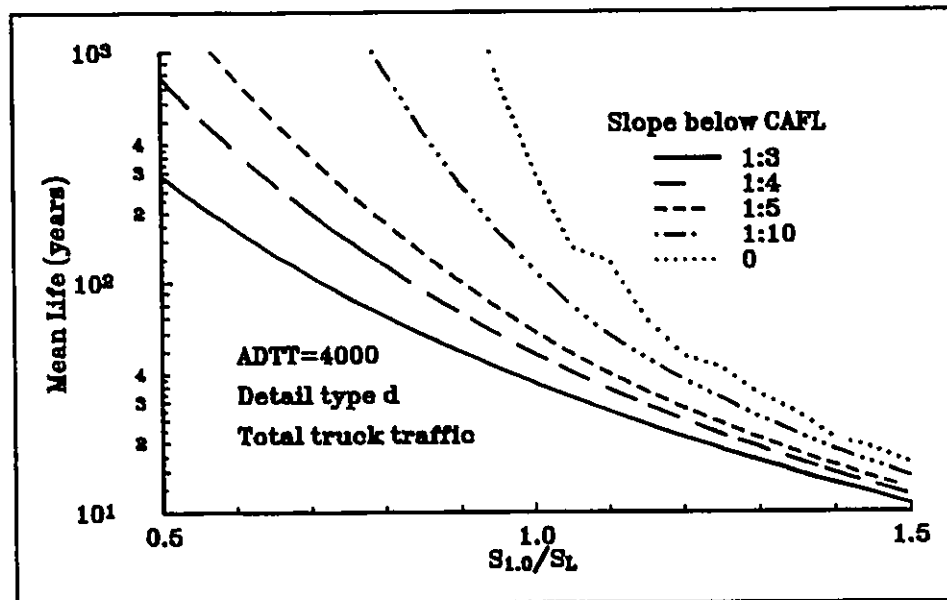


Figure 5.4 Mean fatigue life as a function of spectrum amplitude

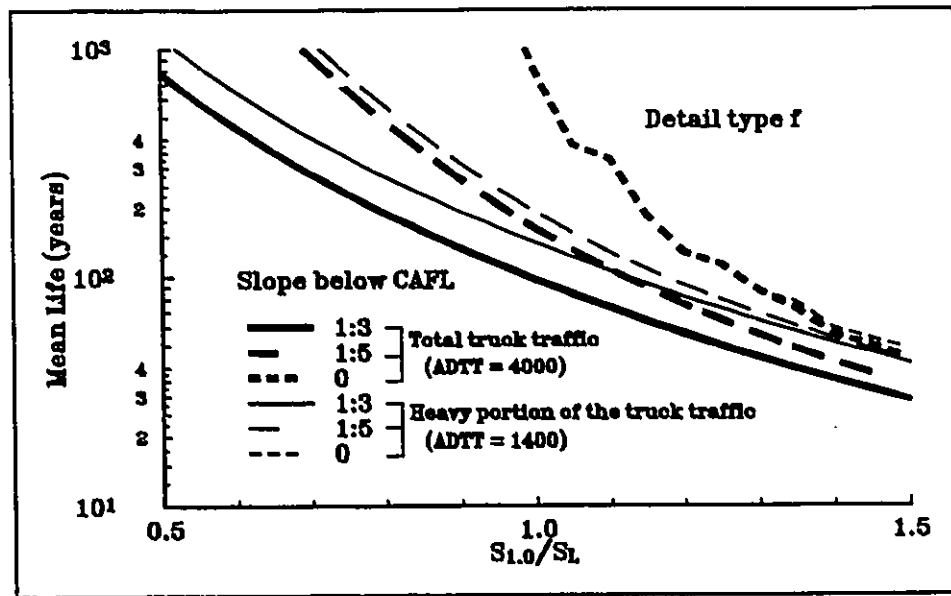


Figure 5.5 Comparison of the spectra for total and heavy portion of the truck traffic

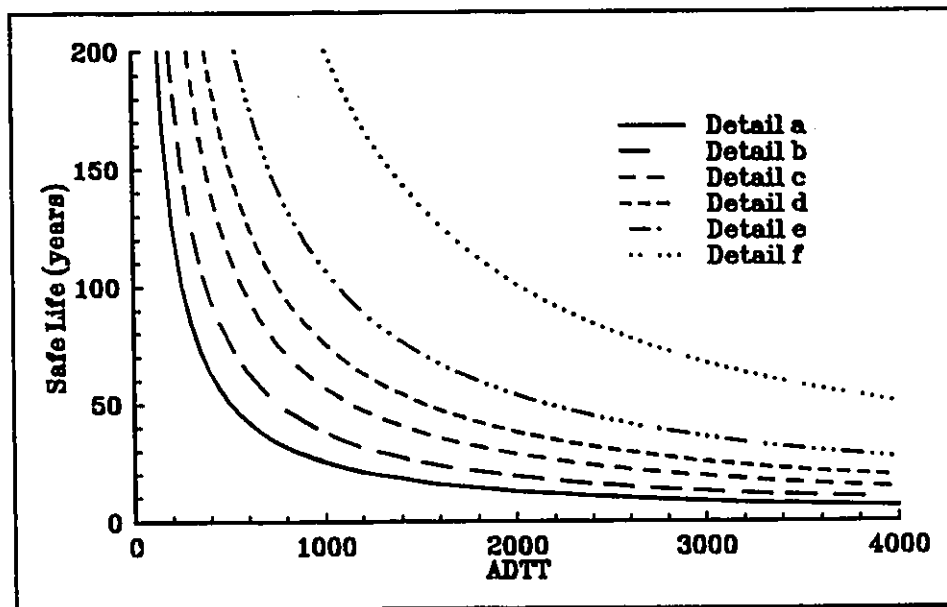


Figure 5.6 Safe fatigue life as a function of ADTT for 1:3 slope

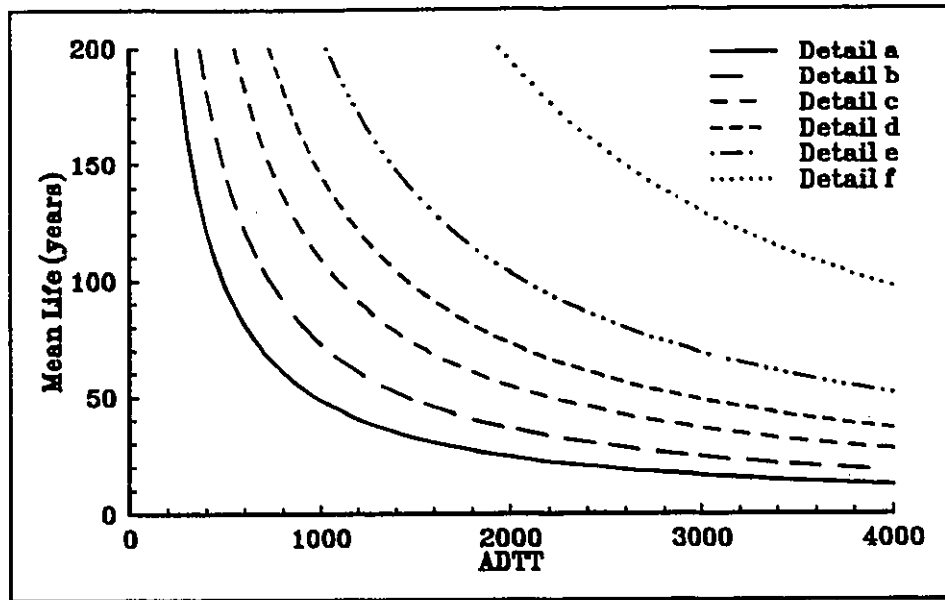


Figure 5.7 Mean fatigue life as a function of ADTT for 1:3 slope

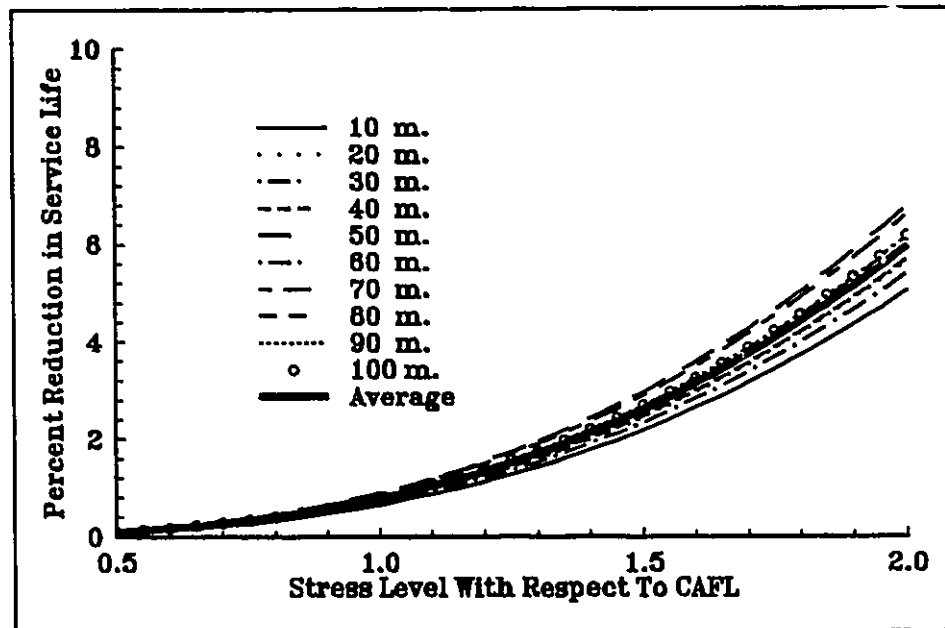


Figure 5.8 Effect of spectrum type on reduction in fatigue life for detail d, 100 permit truck per month and ADTT of 1000



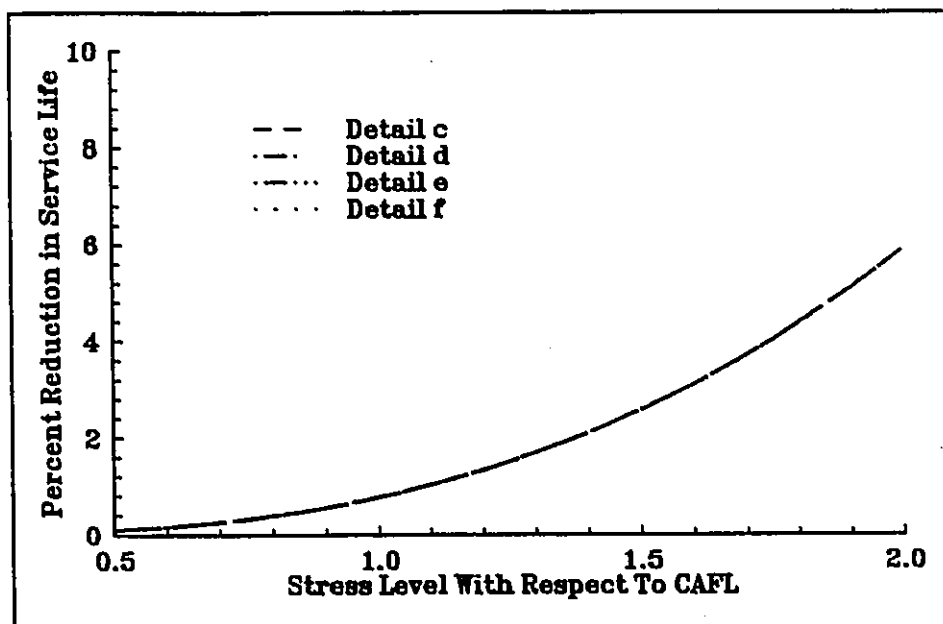


Figure 5.9 Effect of detail type on reduction in fatigue life for 100 permit truck per month and ADTT of 1000

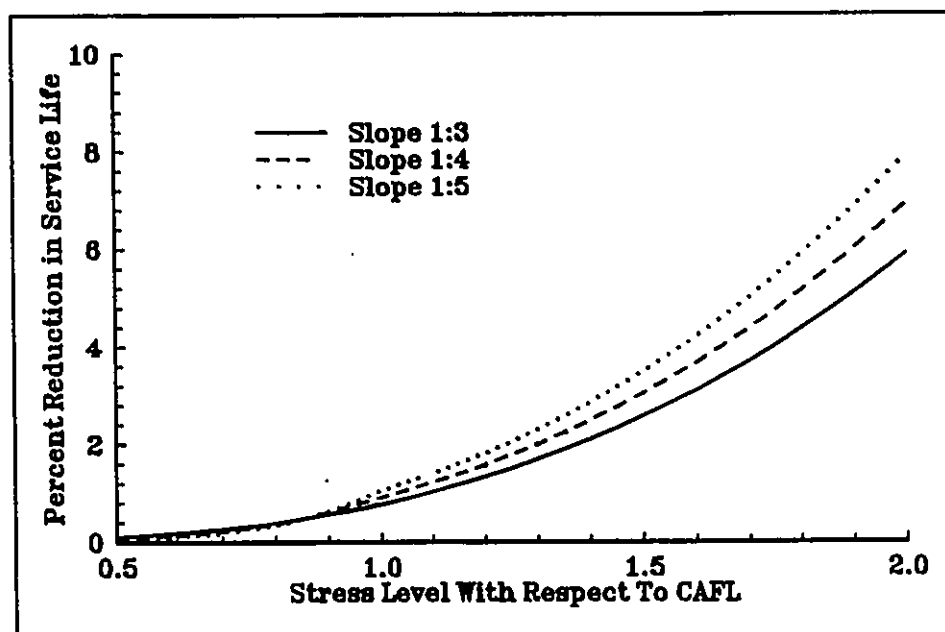


Figure 5.10 Effect of the magnitude of the stress range on reduction in fatigue life for detail d, 100 permit truck per month and ADTT of 1000

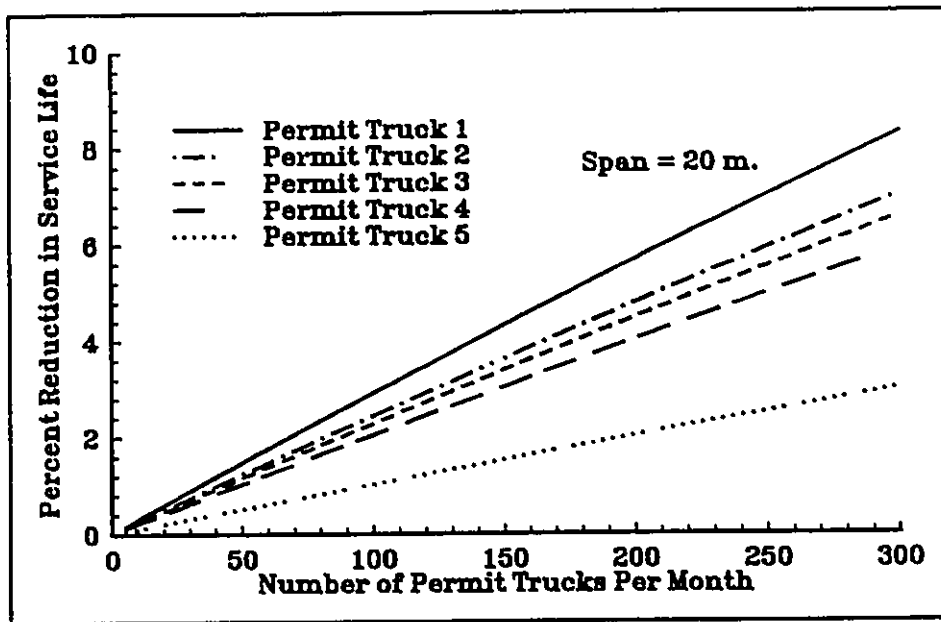


Figure 5.11 Effect of the frequency of heavy permit-trucks on reduction in fatigue life for detail type d and ADTT of 1000

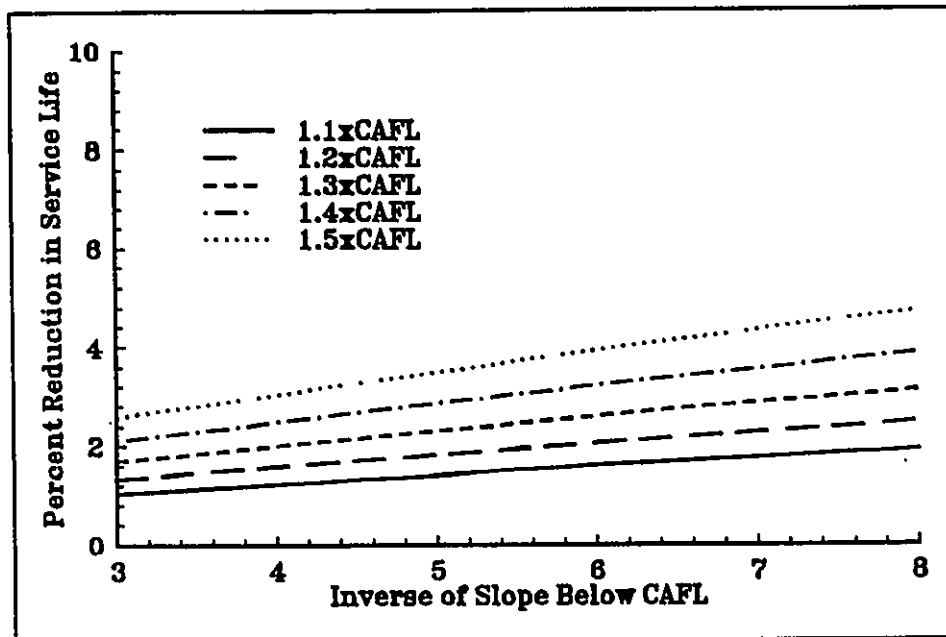


Figure 5.12 Effect of the S-N curve's slope below CAFL on reduction in fatigue life for detail d, 100 permit truck per month and ADTT of 1000

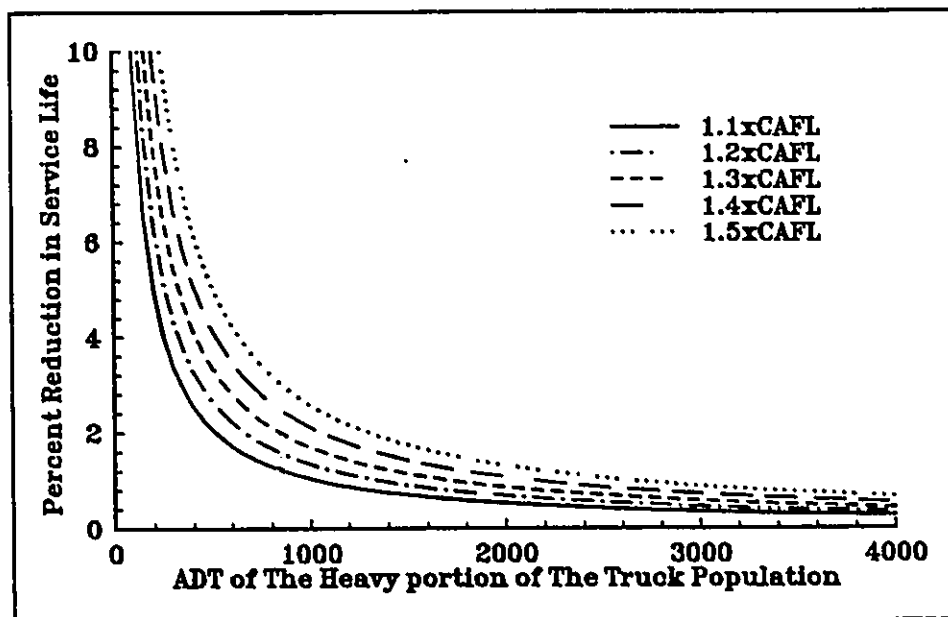


Figure 5.13 Effect of the ADTT on the reduction in fatigue life for detail d and 100 permit truck per month

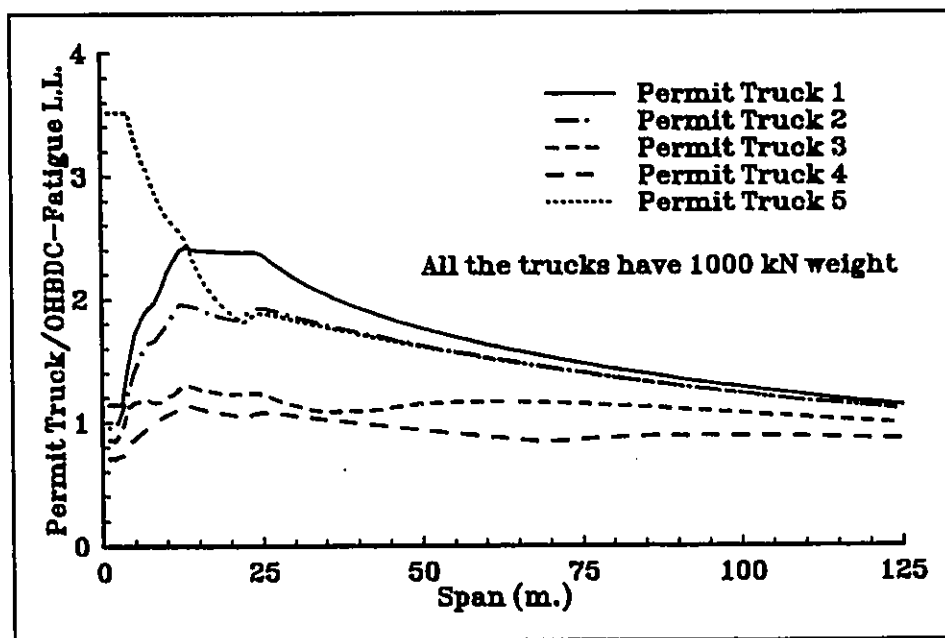


Figure 5.14 LLR for simply supported bridges

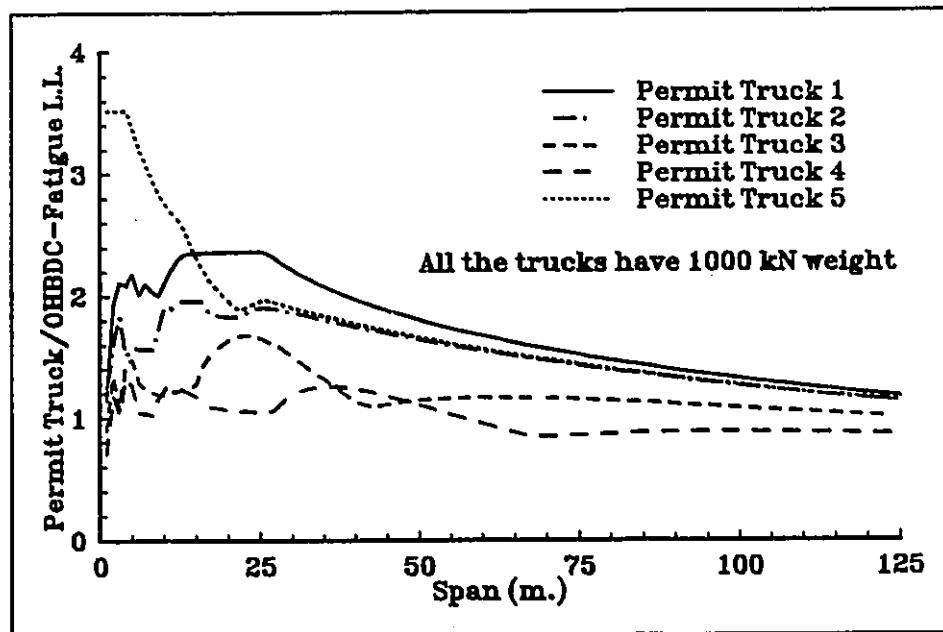


Figure 5.15 LLR for continuous span bridges

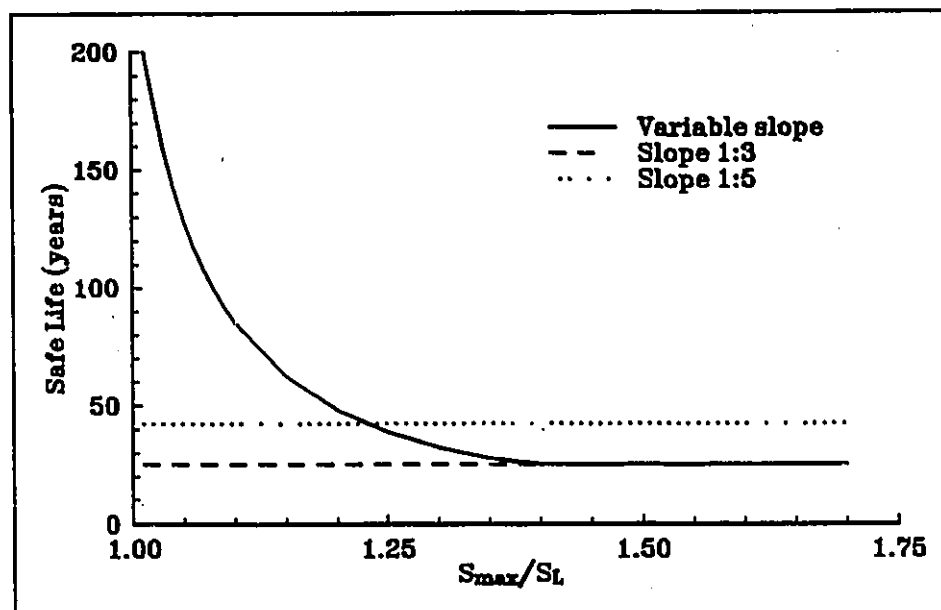


Figure 5.16 Effect of variable slope on fatigue life for detail d and ADTT of 3000

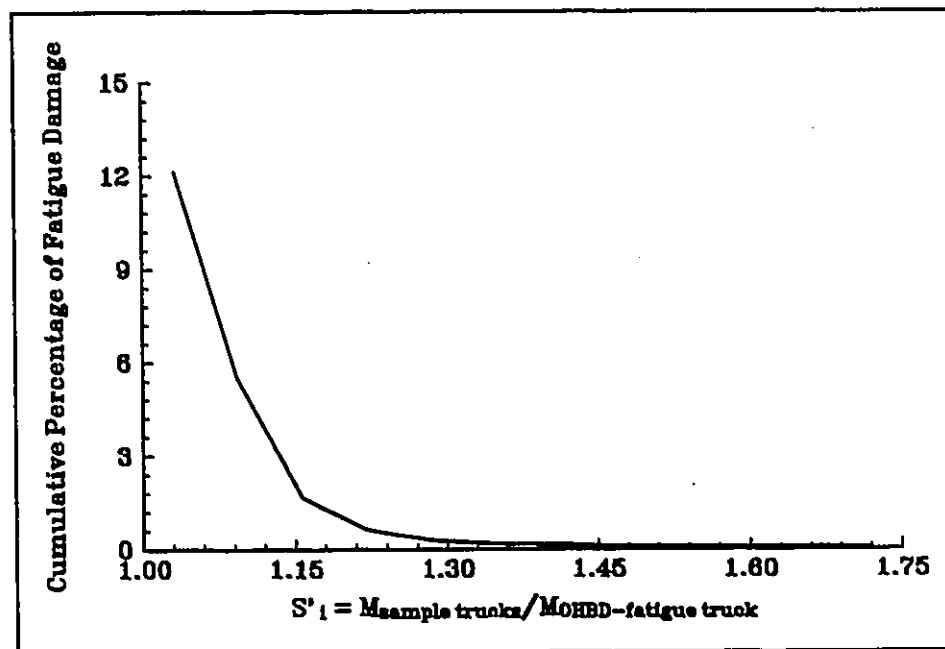
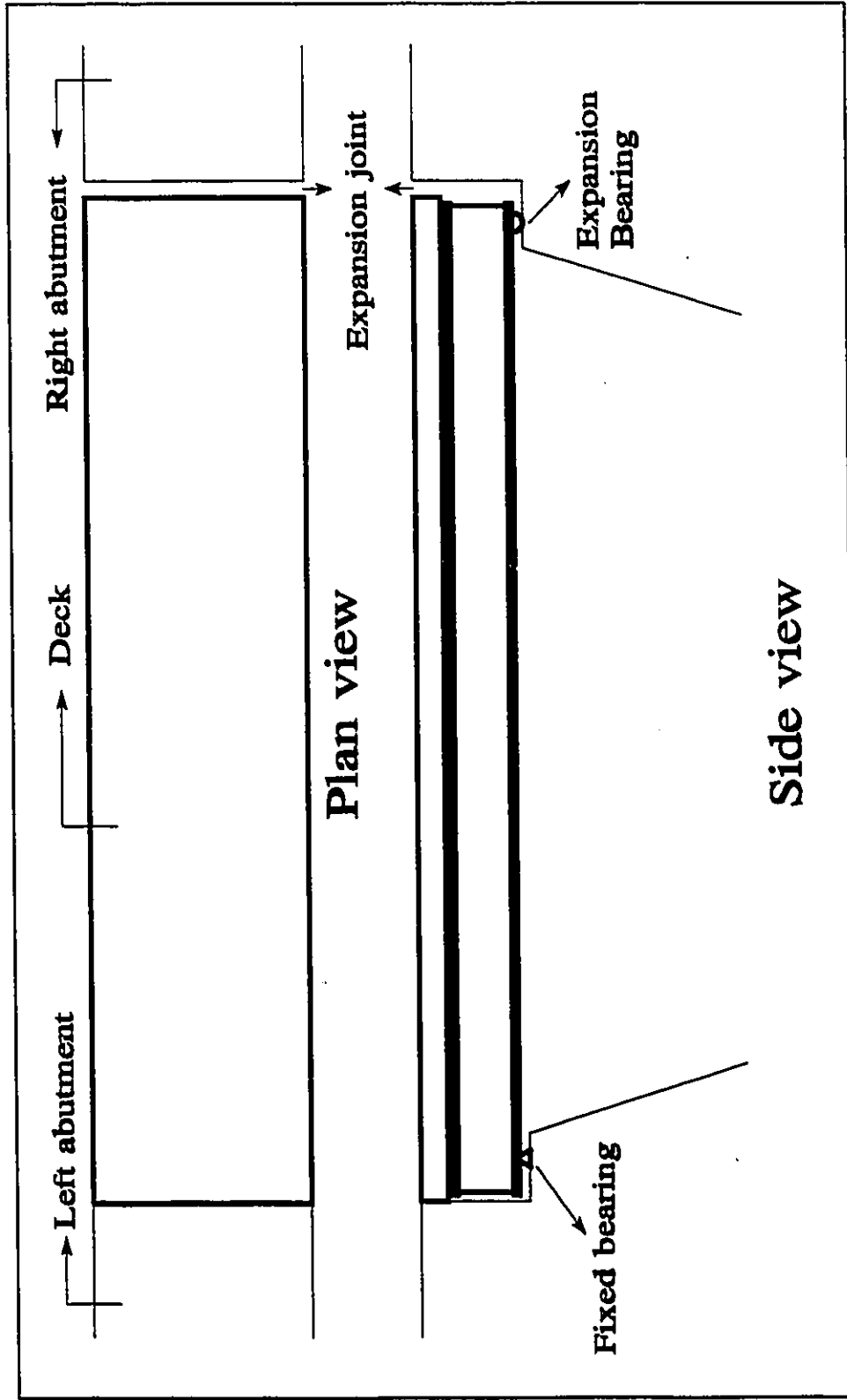


Figure 5.17 Cumulative fatigue damage as a function of stress range ratio



**Figure 6.1** Typical single span simply supported bridge

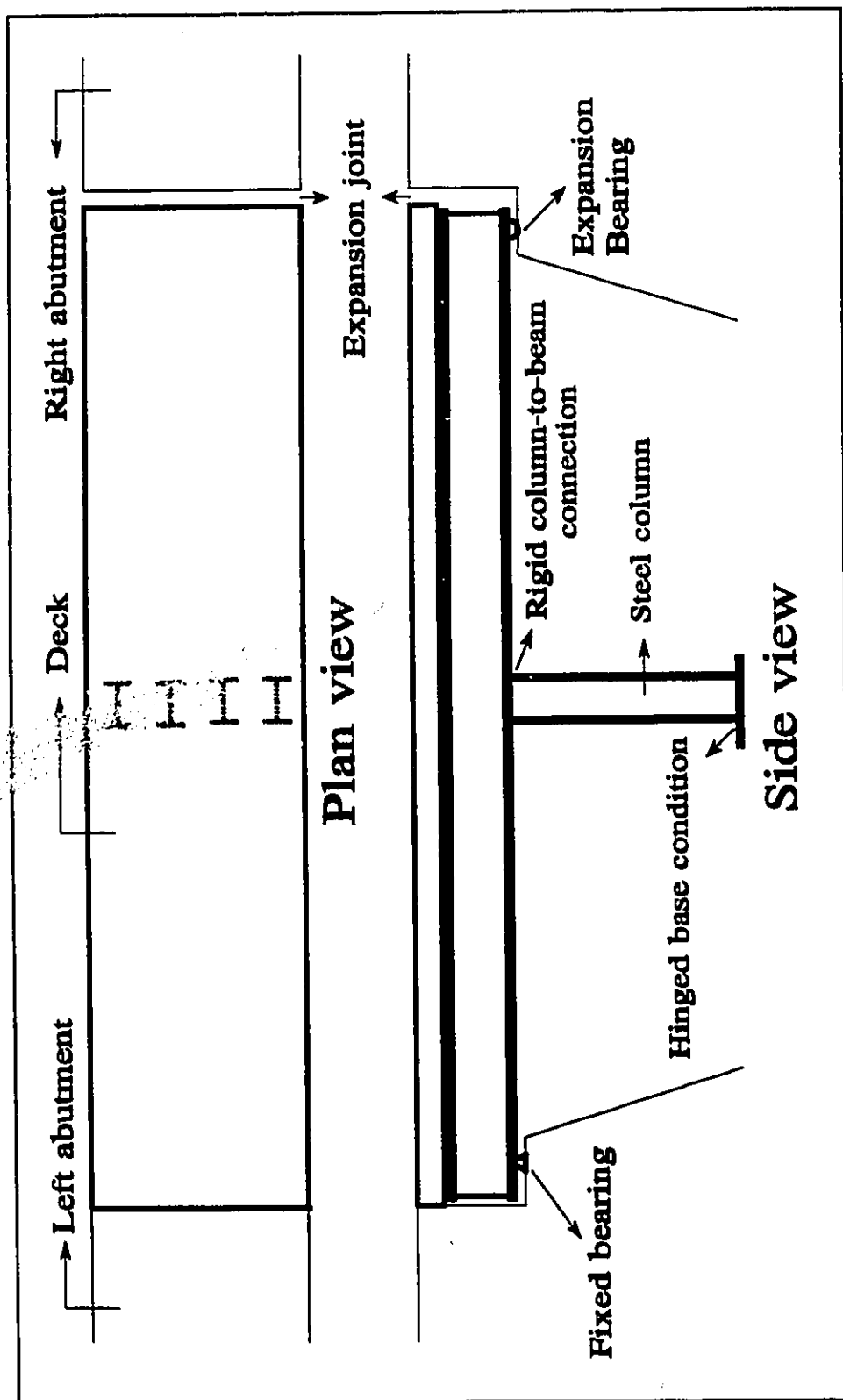


Figure 6.2 Typical two span continuous bridge

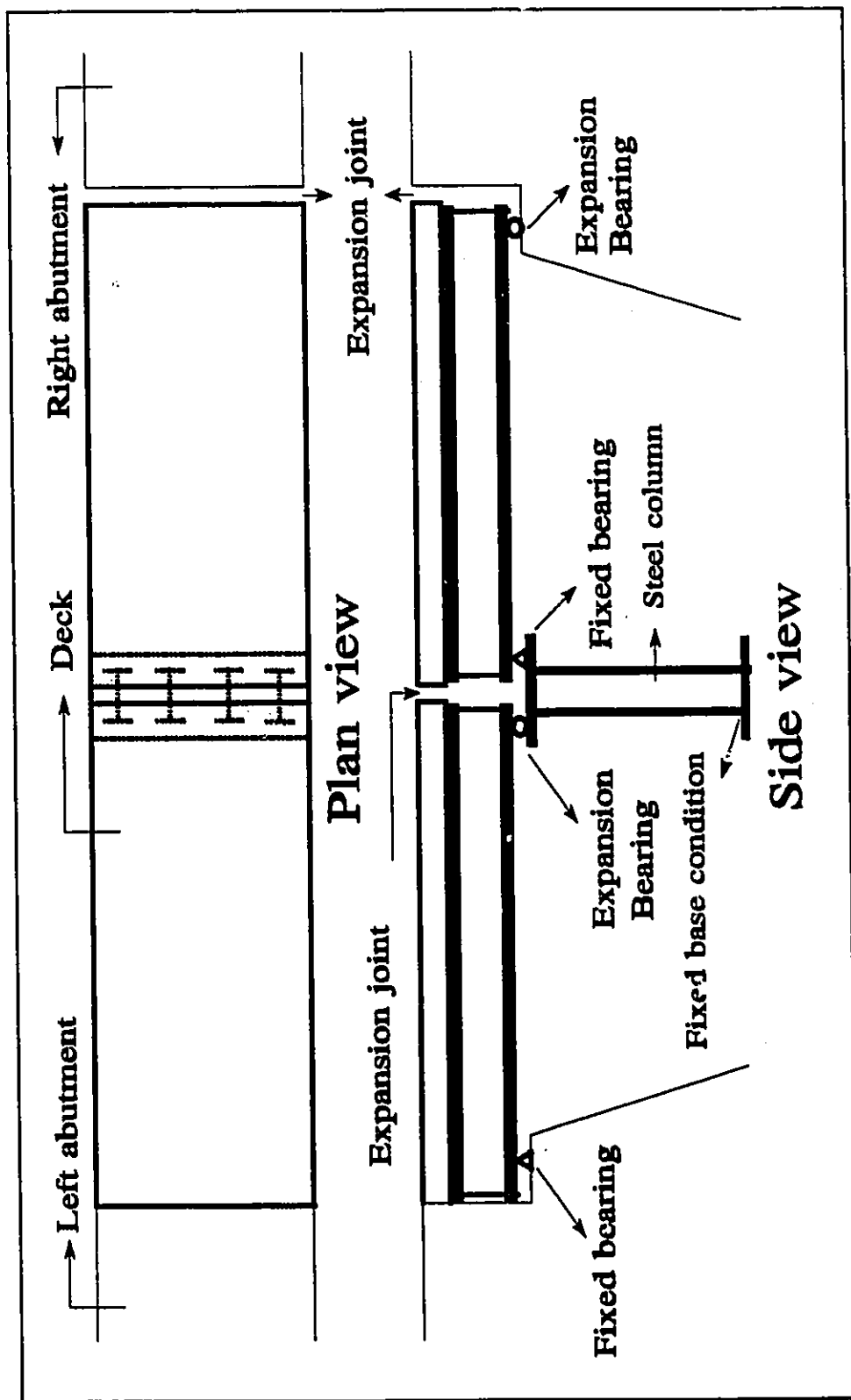


Figure 6.3 Typical two span simply supported bridge



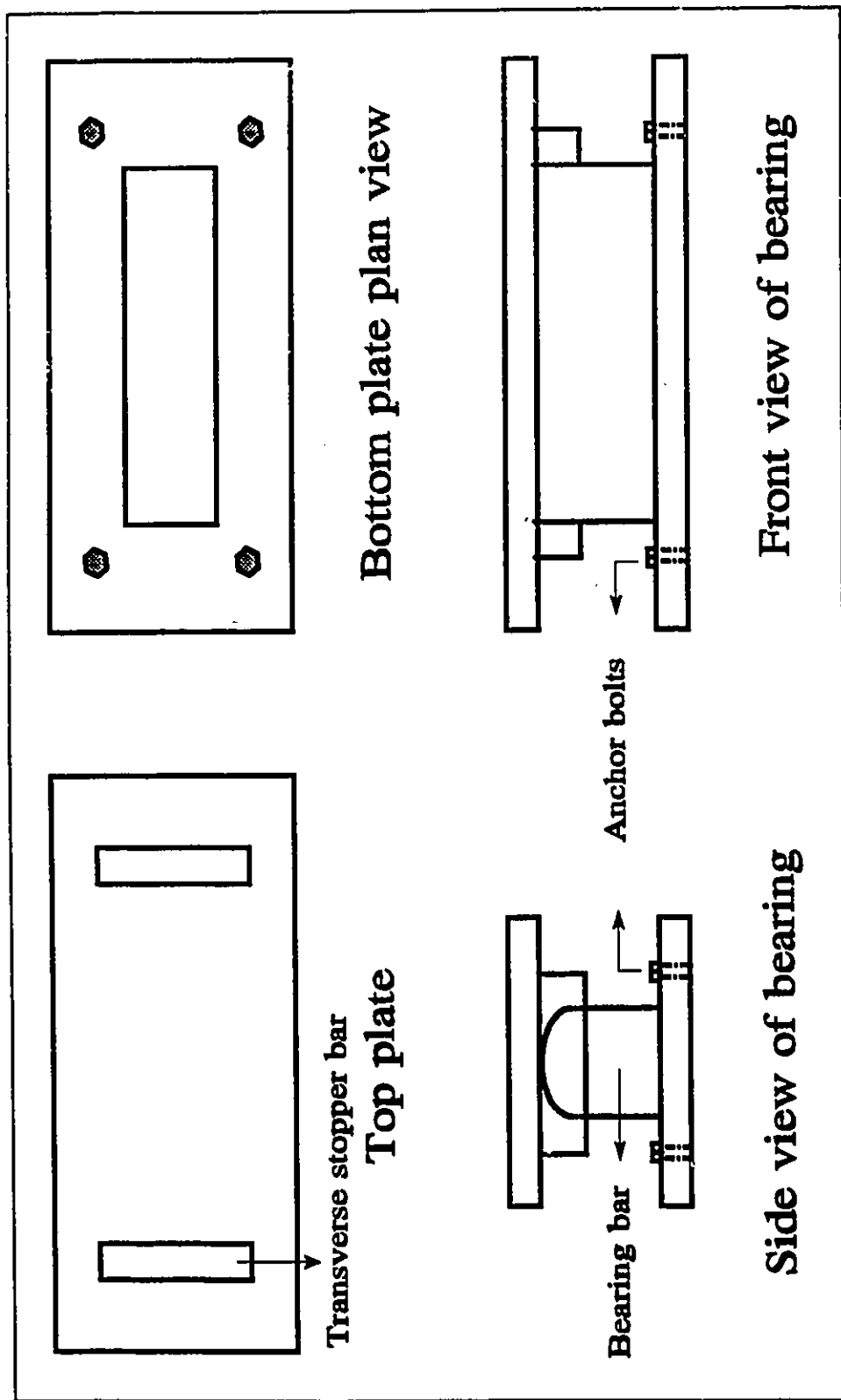


Figure 6.4 Typical expansion sliding bearing

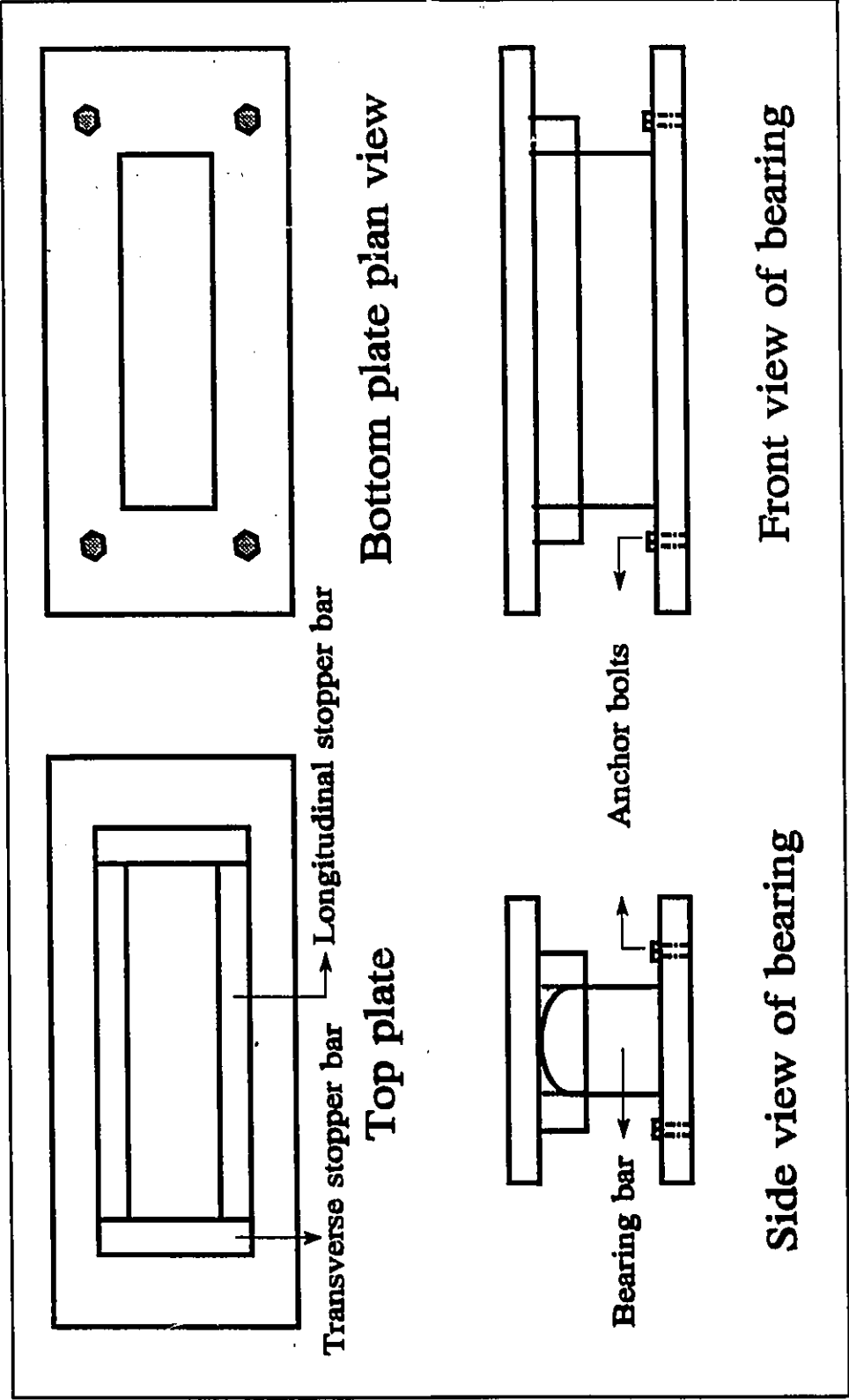


Figure 6.5 Typical fixed sliding bearing

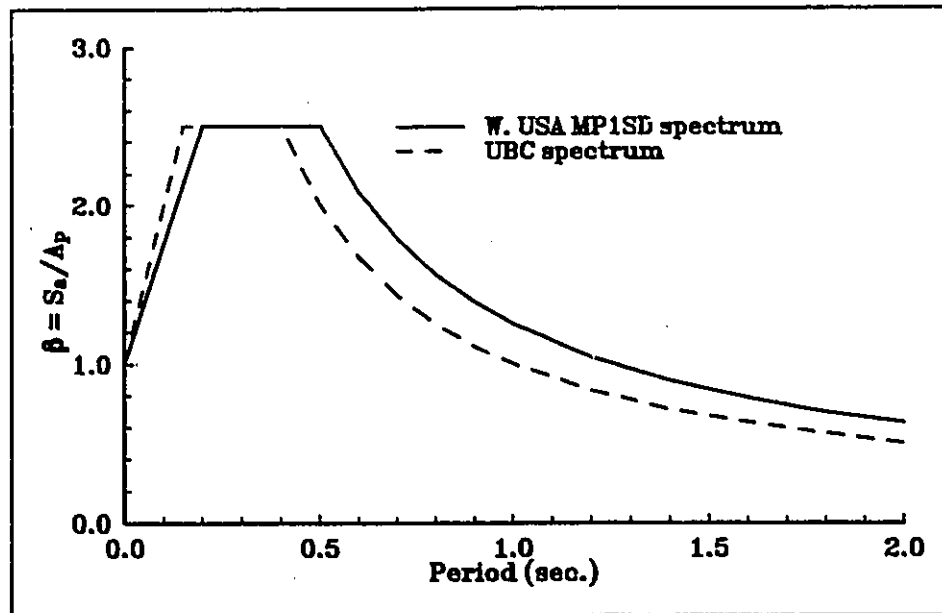


Figure 6.6 Comparison of UBC spectrum and MP1SD spectrum of W. USA earthquakes for rock or stiff soil and 5% damping

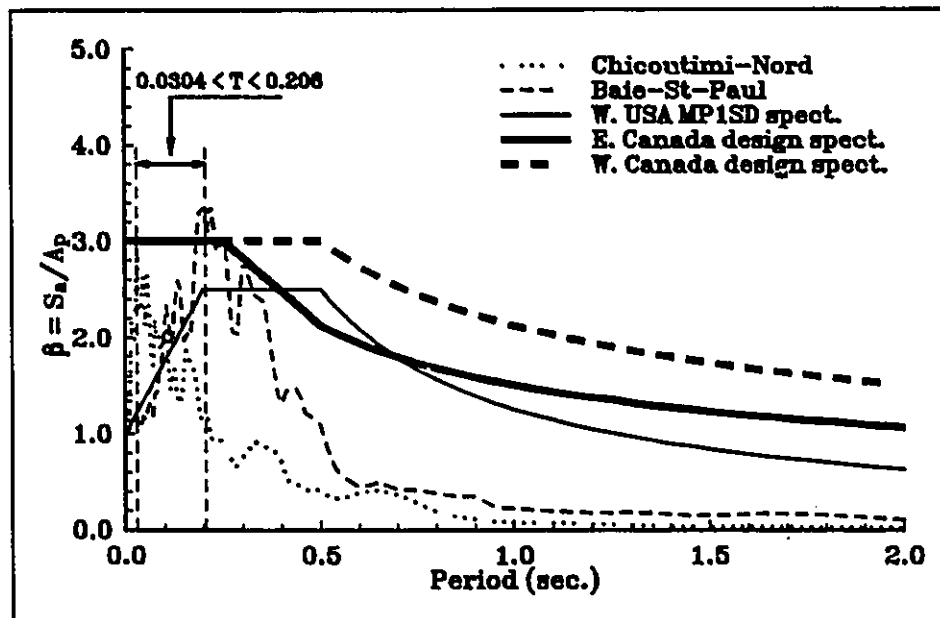
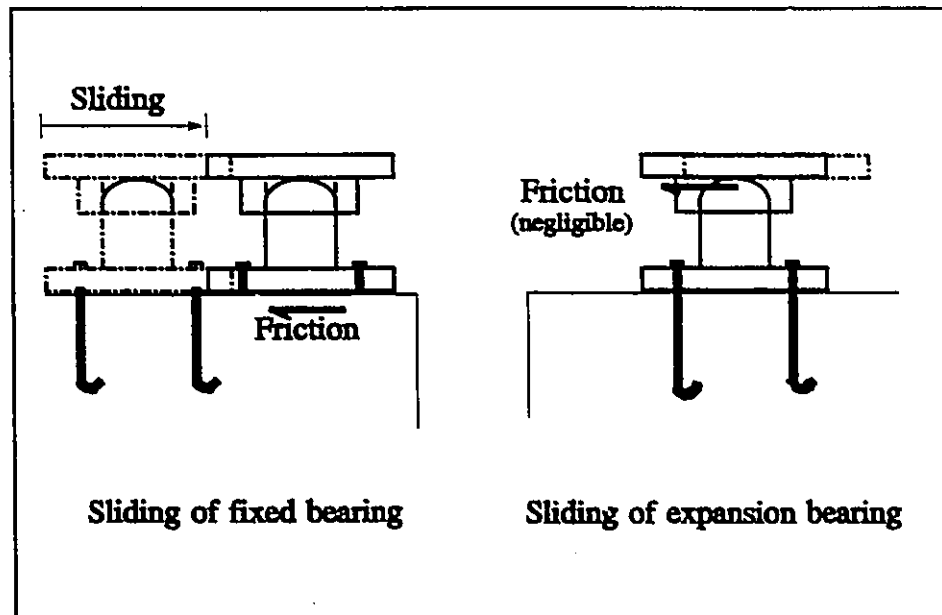
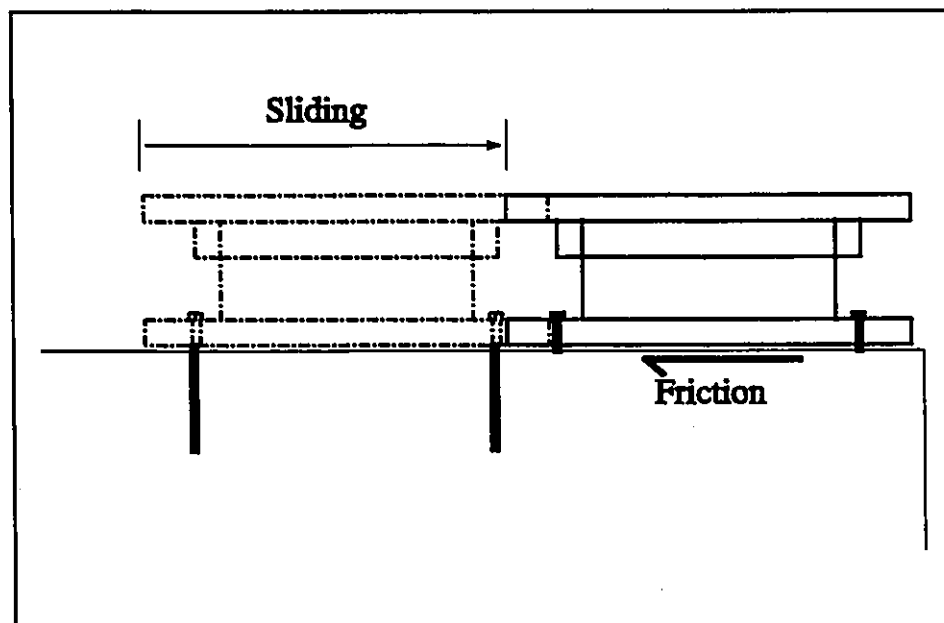


Figure 6.7 Elastic response spectra of Eastern Canada and Western Canada and United States earthquakes for 5% damping



**Figure 7.1** Sliding of bearings in longitudinal direction



**Figure 7.2** Sliding of bearings in transverse direction

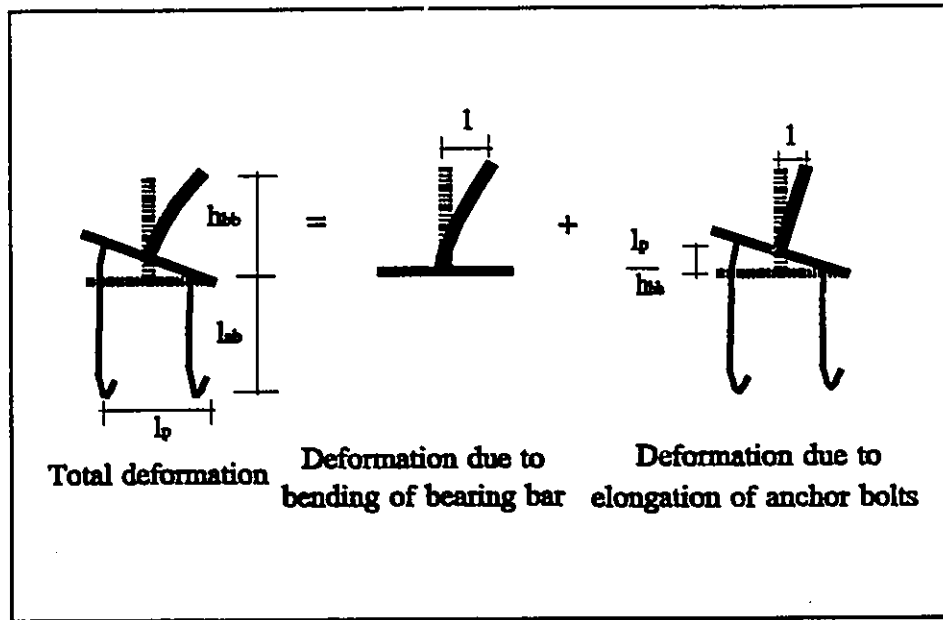


Figure 7.3 Deformation of sliding-bearings

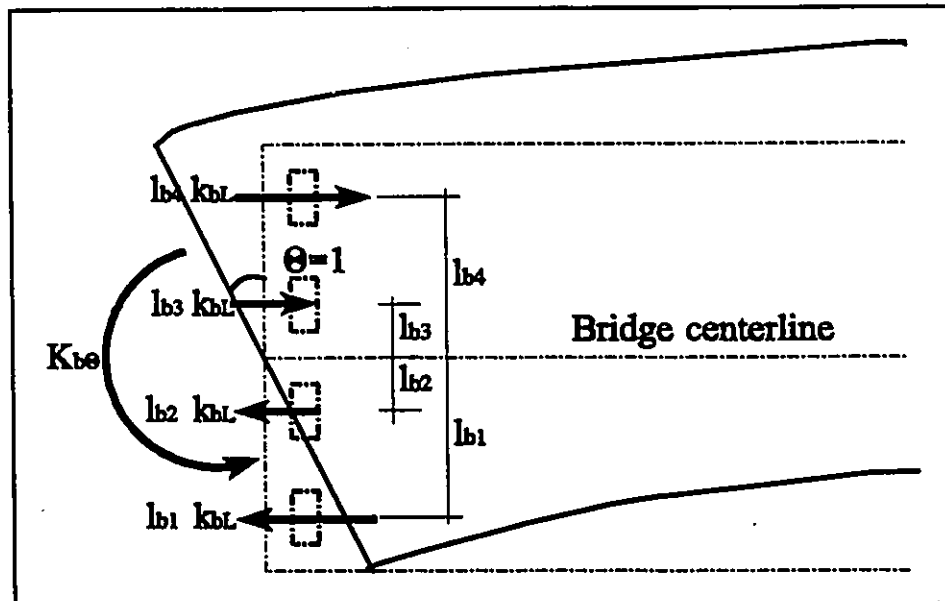
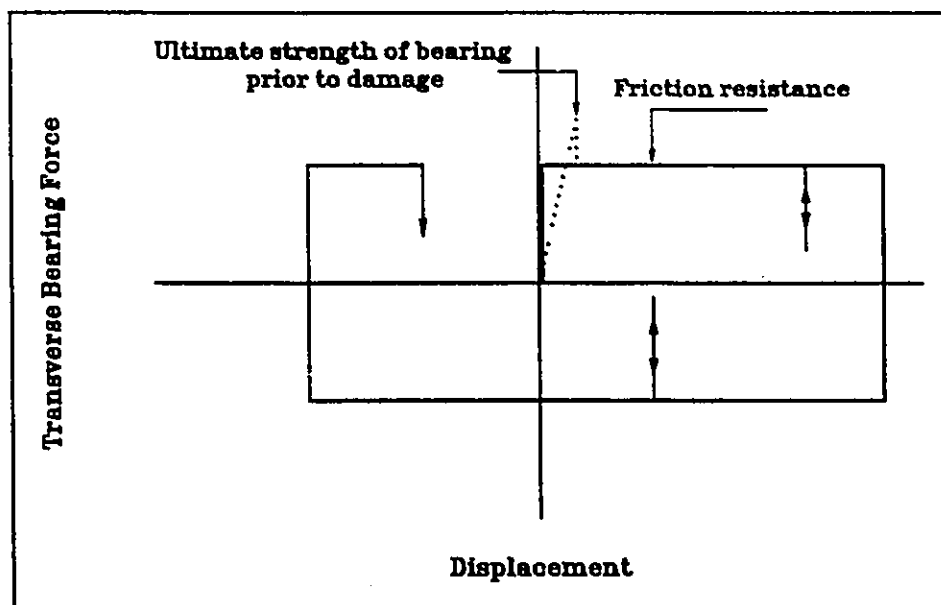
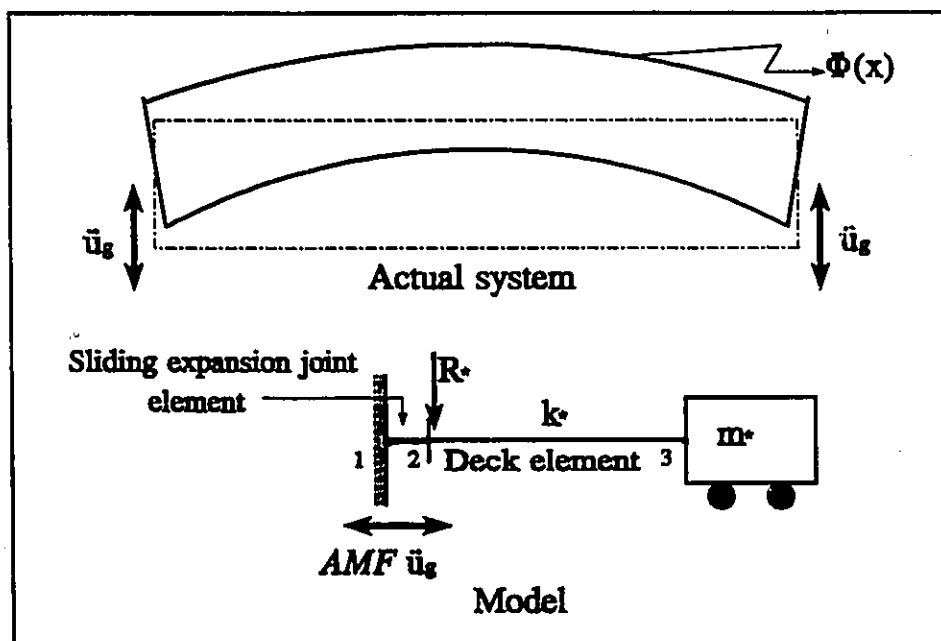


Figure 7.4 Rotational stiffness of bearings-set at the support



**Figure 7.5** Bearing force vs. sliding displacement hysteresis models including and neglecting the ultimate bearing strength before damage occurs.



**Figure 7.6** Actual system and simplified equivalent non-linear inelastic model

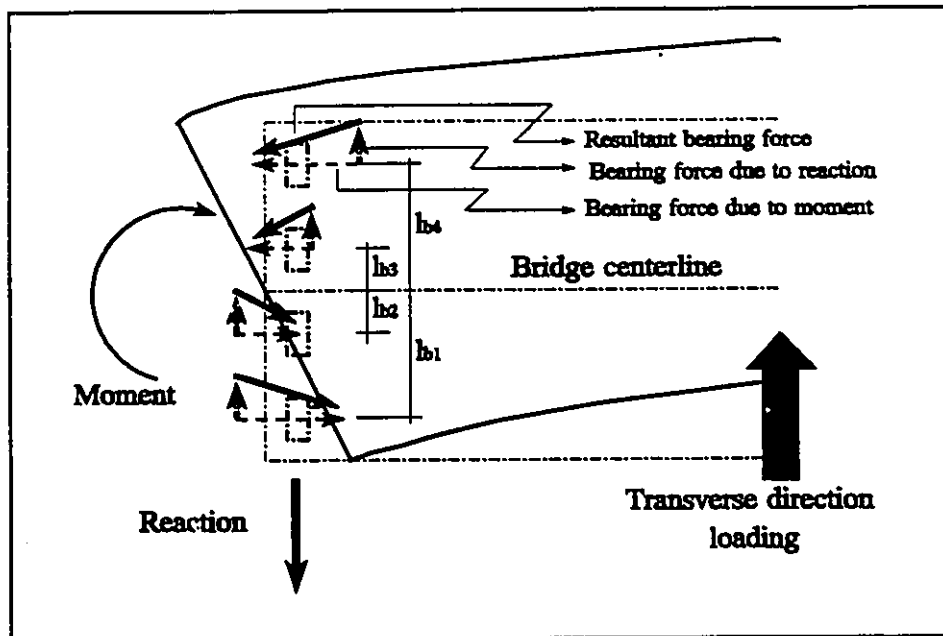


Figure 7.7 Bearing forces due to loading in transverse direction

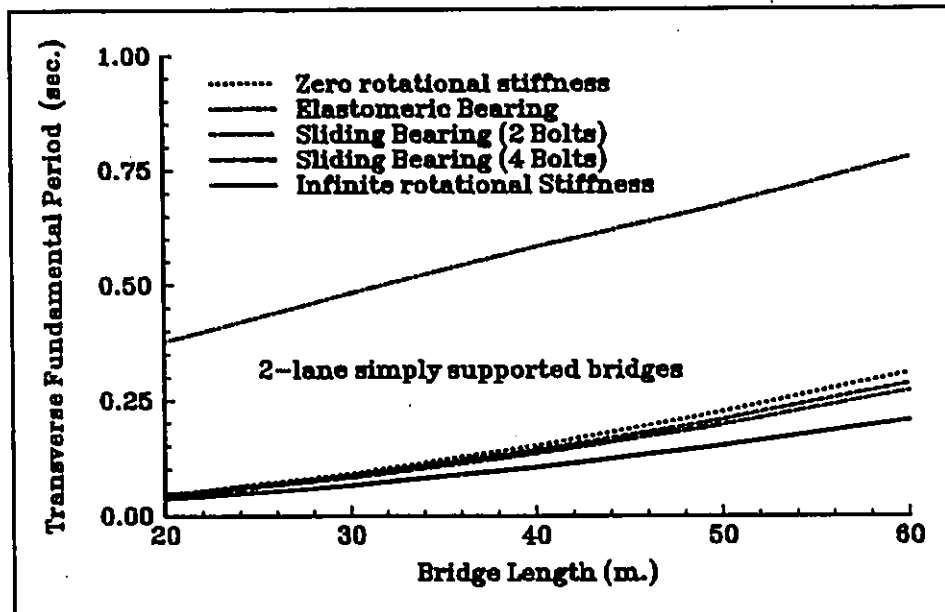


Figure 7.8 Transverse direction fundamental periods of 2-lane simply supported bridges as a function of span length for various bearing types

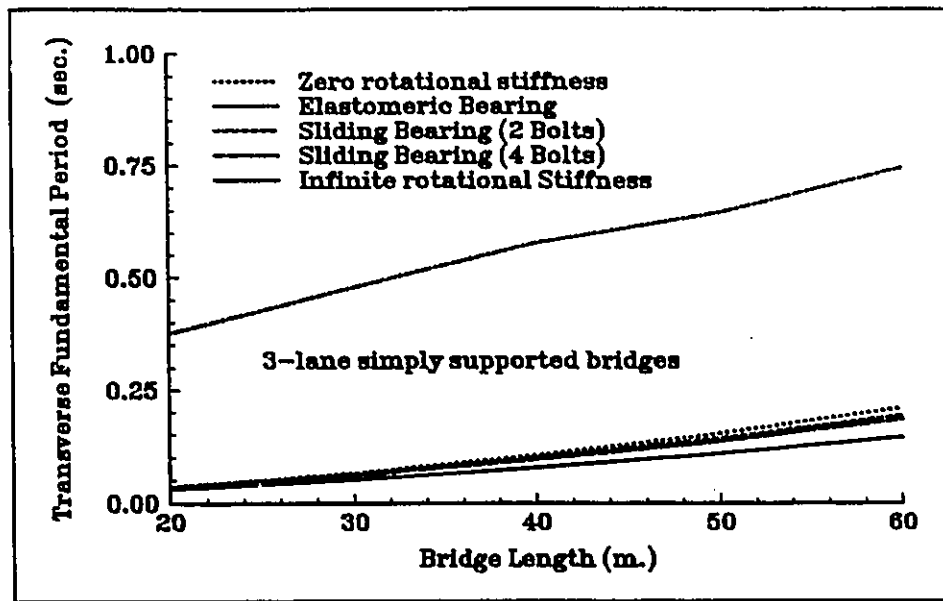


Figure 7.9 Transverse direction fundamental periods of 3-lane simply supported bridges as a function of span length for various bearing types

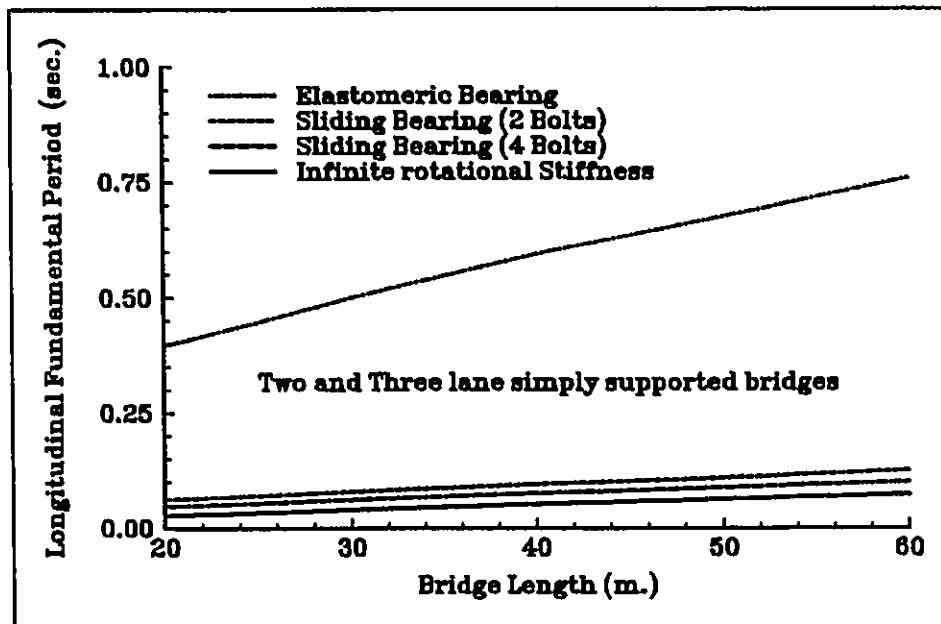


Figure 7.10 Longitudinal direction fundamental periods of 2 and 3-lane simply supported bridges as a function of span length for various bearing types



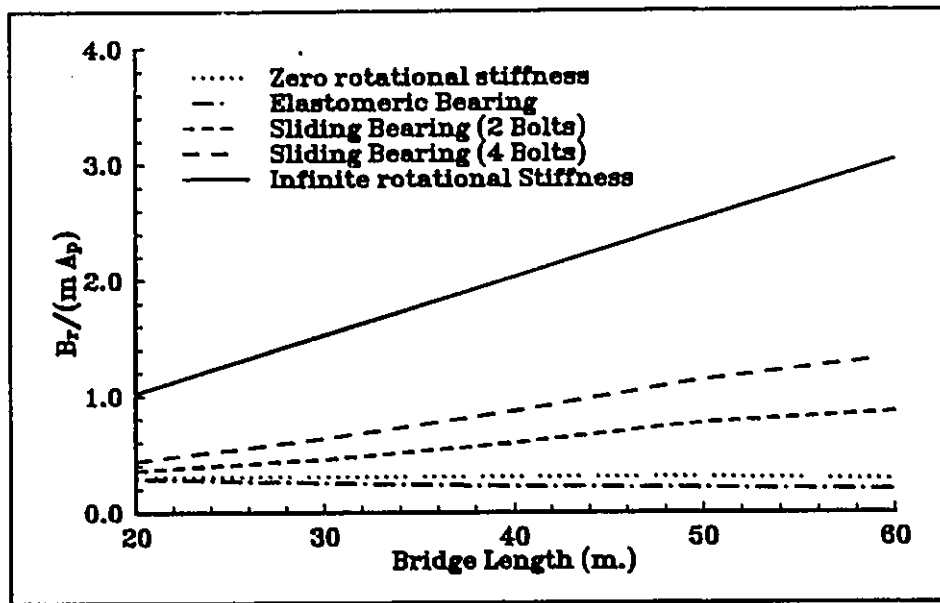


Figure 7.11 Effect of bearing's stiffness on the transverse bearing force for 2-lane simply supported bridges (Eastern Canada design spectrum)

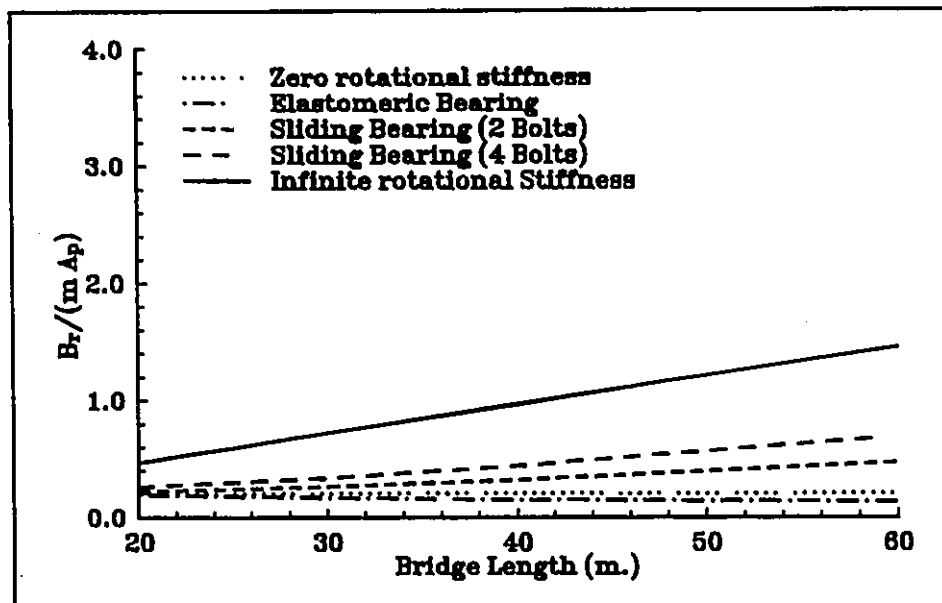


Figure 7.12 Effect of bearing's stiffness on the transverse bearing force for 3-lane simply supported bridges (Eastern Canada design spectrum)

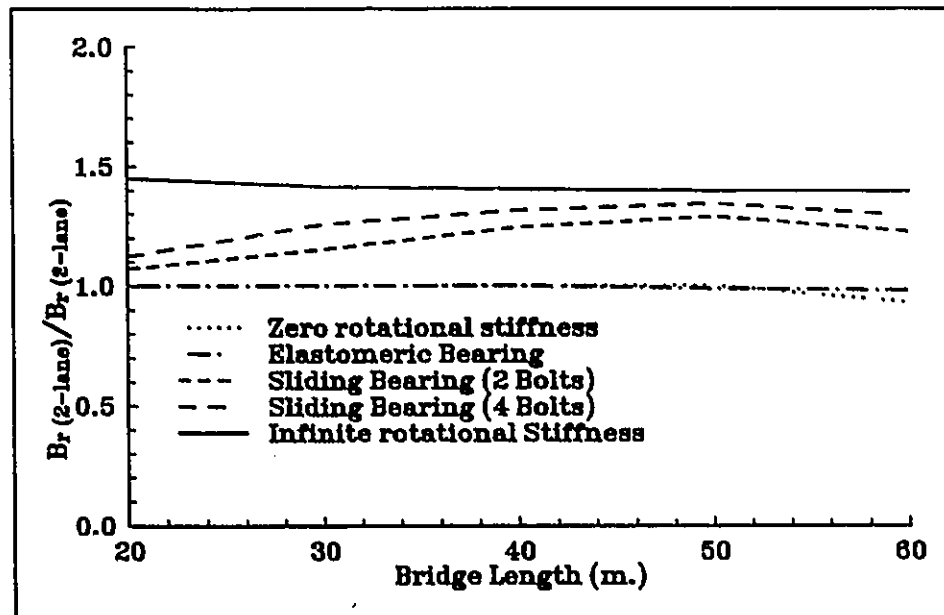


Figure 7.13 Ratio of transverse bearing forces of 2-lane and 3-lane bridges for different end fixity conditions as a function of span length

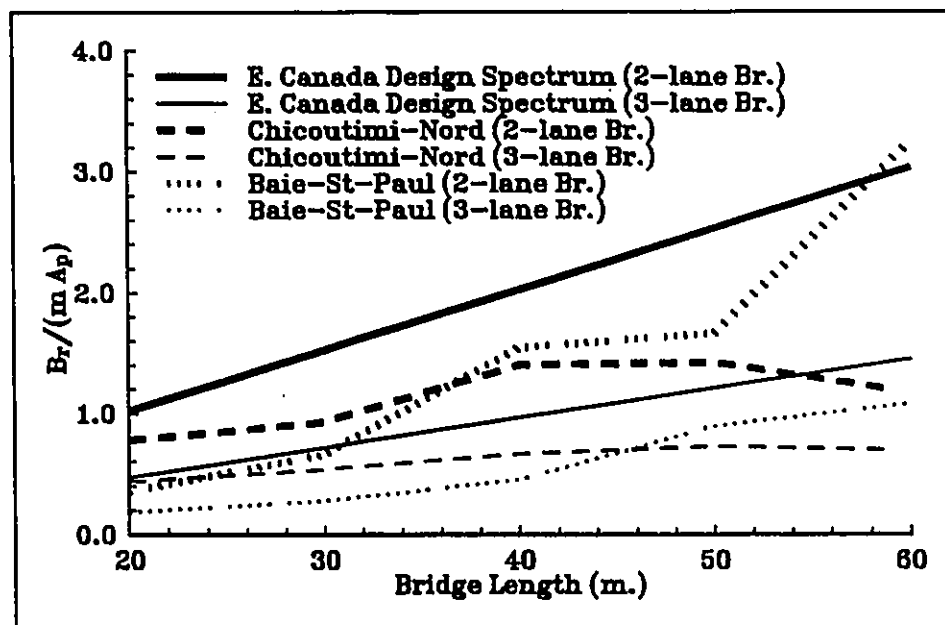


Figure 7.14 Comparison of bearings' transverse forces due to design spectrum and selected Eastern Canada earthquakes' spectra for simply supported bridges (infinite rotational stiffness)

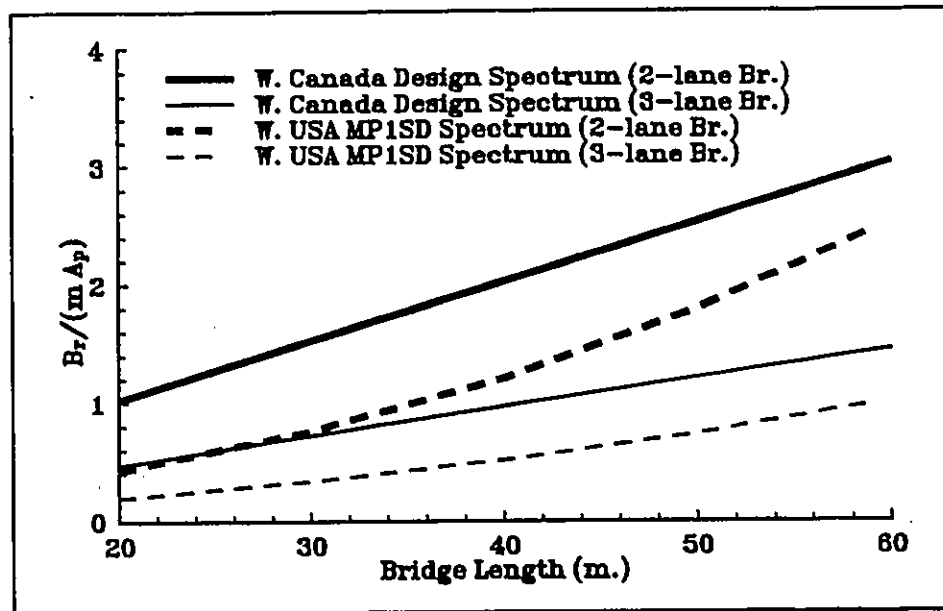


Figure 7.15 Comparison of bearings' transverse forces due to design spectrum and MP1SD spectrum of Western USA earthquakes for simply supported bridges (infinite rotational stiffness)

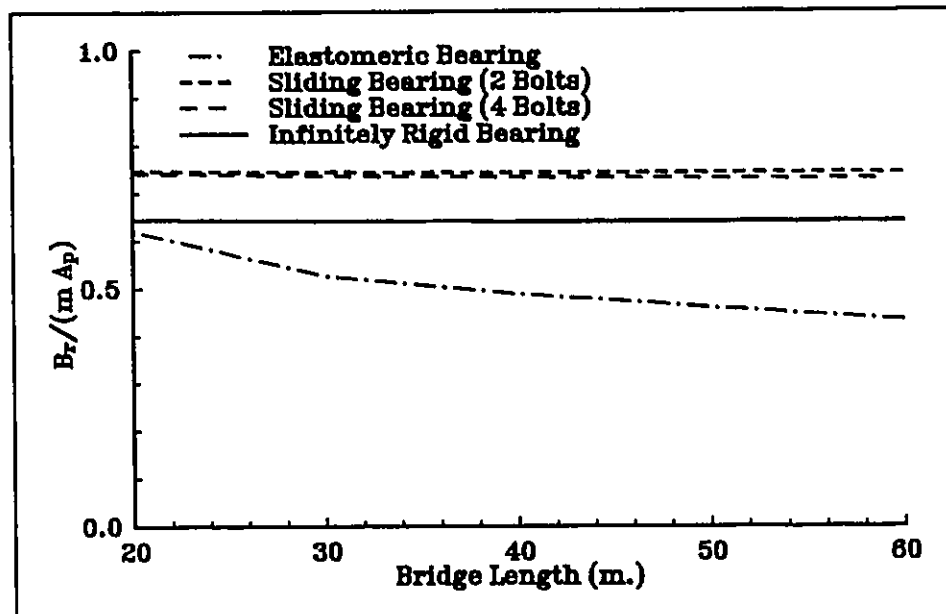


Figure 7.16 Effect of bearing's stiffness on the longitudinal bearing force for 2-lane simply supported bridges (Eastern Canada design spectrum)

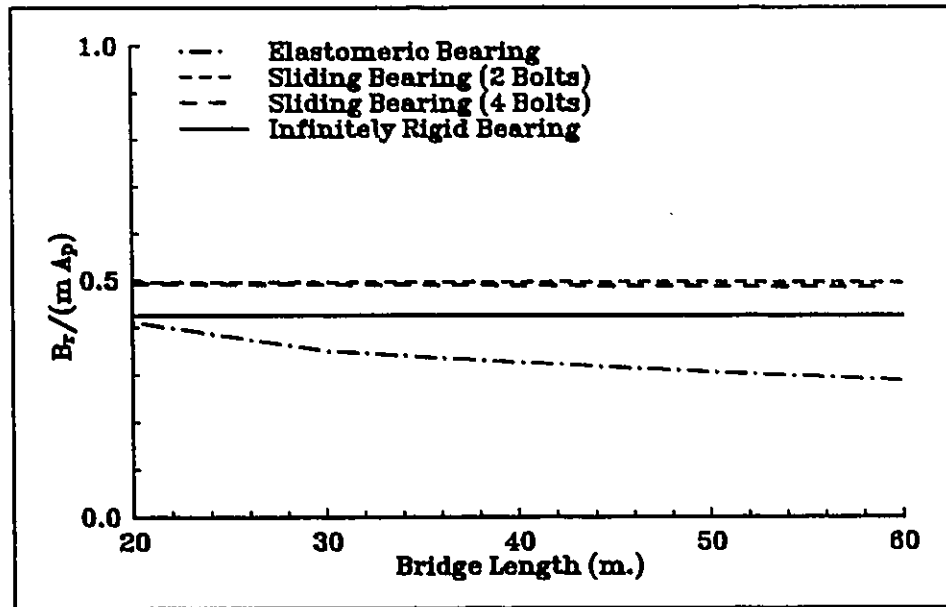


Figure 7.17 Effect of bearing's stiffness on the longitudinal bearing force for 3-lane simply supported bridges (Eastern Canada design spectrum)

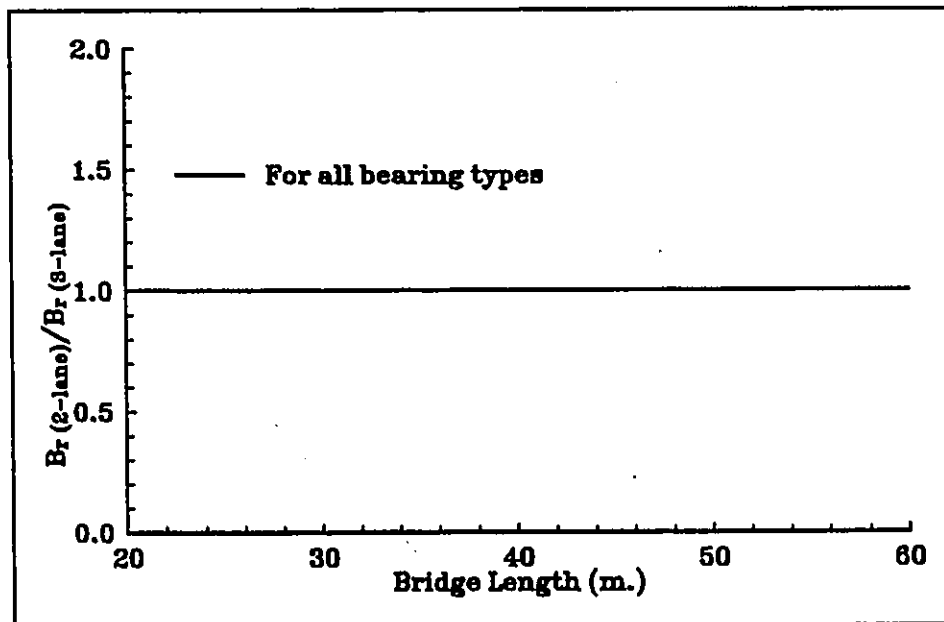


Figure 7.18 Ratio of longitudinal bearing forces of 2-lane and 3-lane bridges as a function of span length for different bearing types

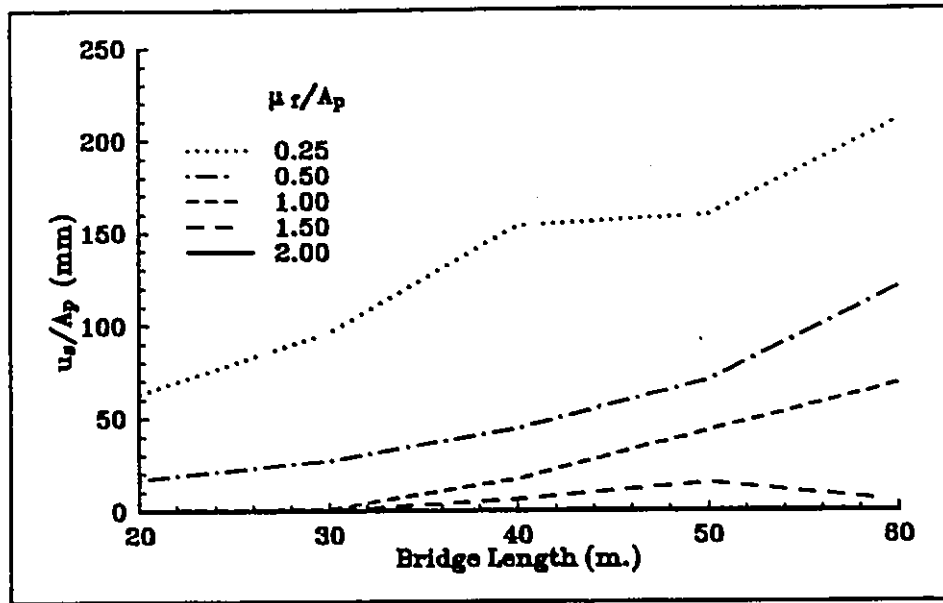


Figure 7.19 Transverse sliding displacement per peak ground acceleration (%g) of 2-lane bridges for various friction coefficient to peak ground acceleration ratios (Average of W. USA earthquakes)

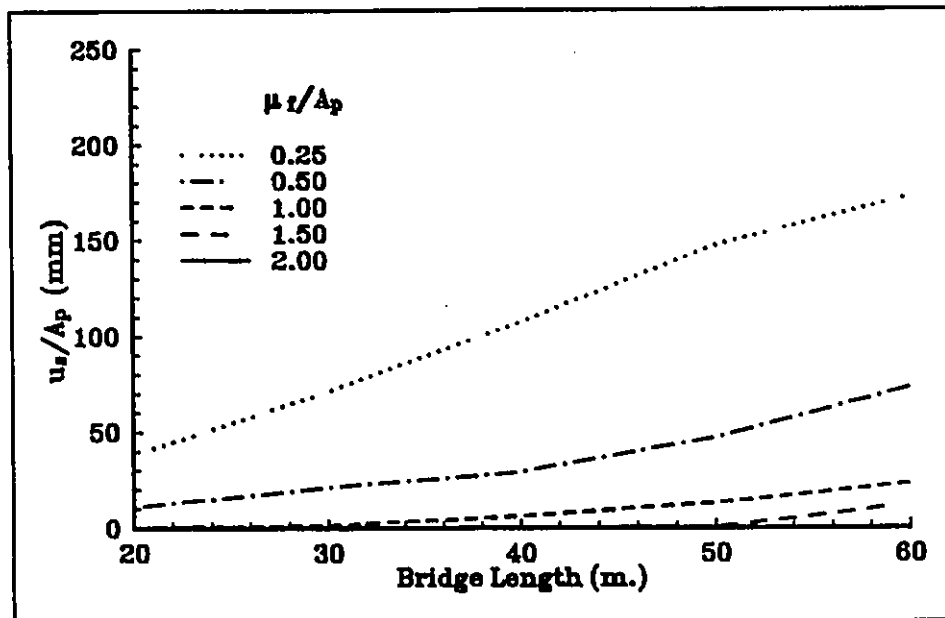


Figure 7.20 Transverse sliding displacement per peak ground acceleration (%g) of 3-lane bridges for various friction coefficient to peak ground acceleration ratios (Average of W. USA earthquakes)

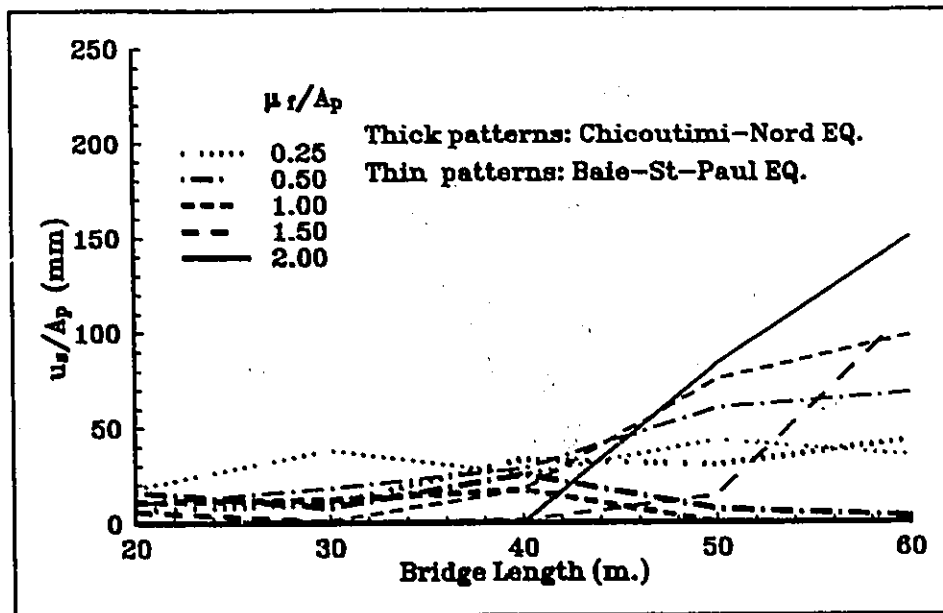


Figure 7.21 Transverse sliding displacement per peak ground acceleration (%g) of 2-lane bridges for various friction coefficient to peak ground acceleration ratios (E. Canada earthquakes)

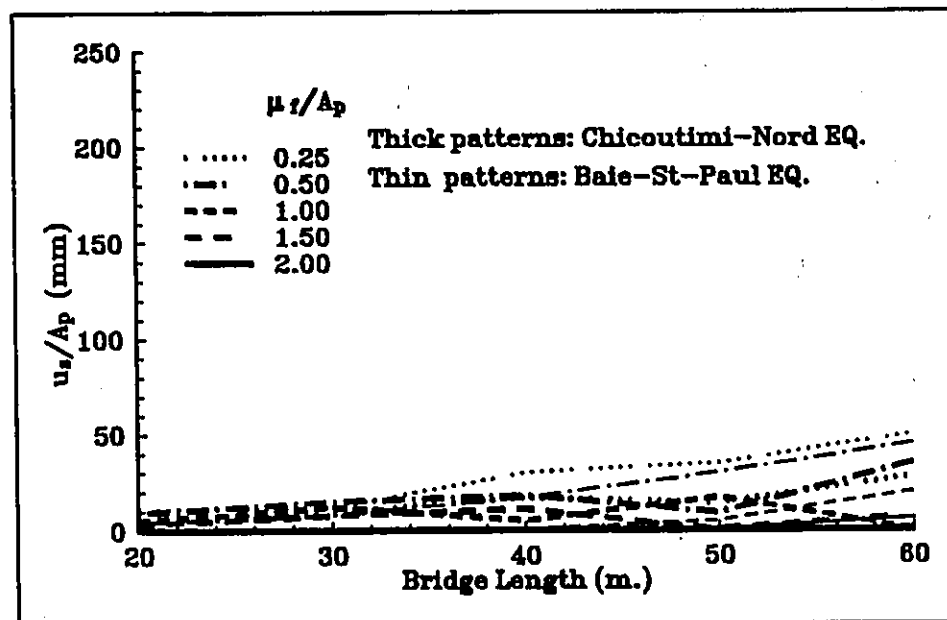
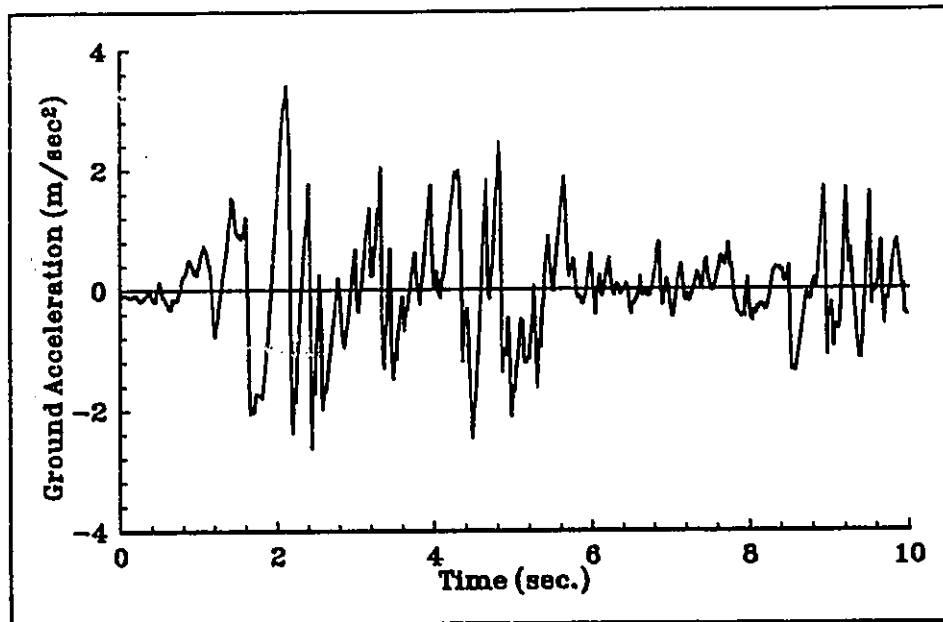
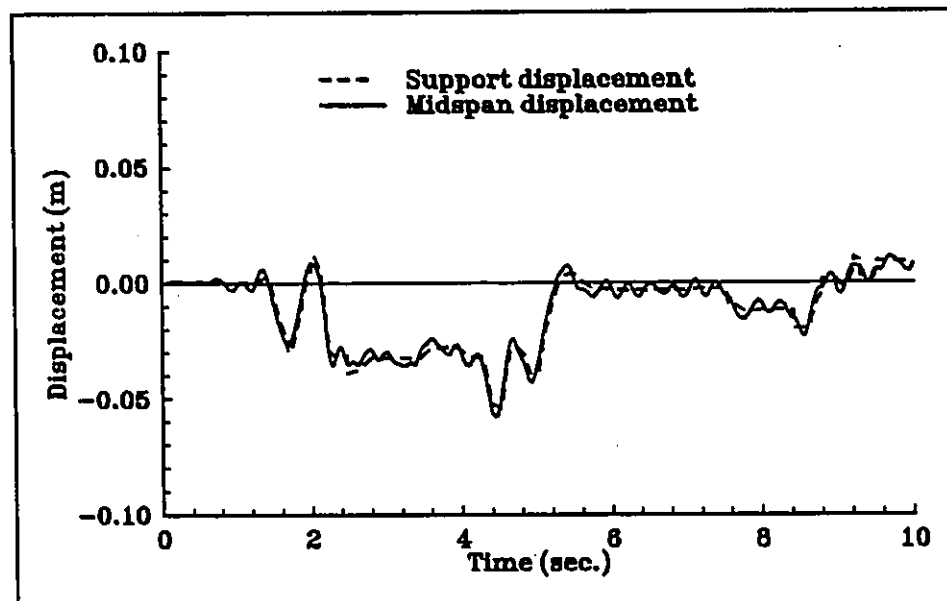


Figure 7.22 Transverse sliding displacement per peak ground acceleration (%g) of 3-lane bridges for various friction coefficient to peak ground acceleration ratios (E. Canada earthquakes)



**Figure 7.23** Ground acceleration time history of El Centro S00E earthquake, first 10 seconds



**Figure 7.24** Support sliding time history of 2-lane 60 m. simply supported bridge (El Centro S00E component  $A_p = 0.4g$ )

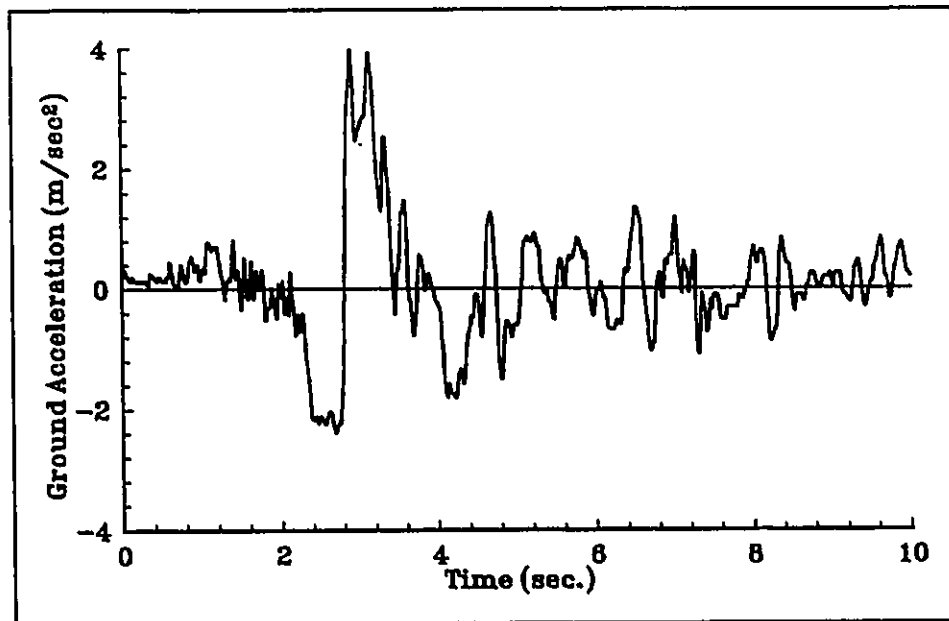


Figure 7.25 Ground acceleration time history of Erzincan, Türkiye, earthquake, first 10 seconds

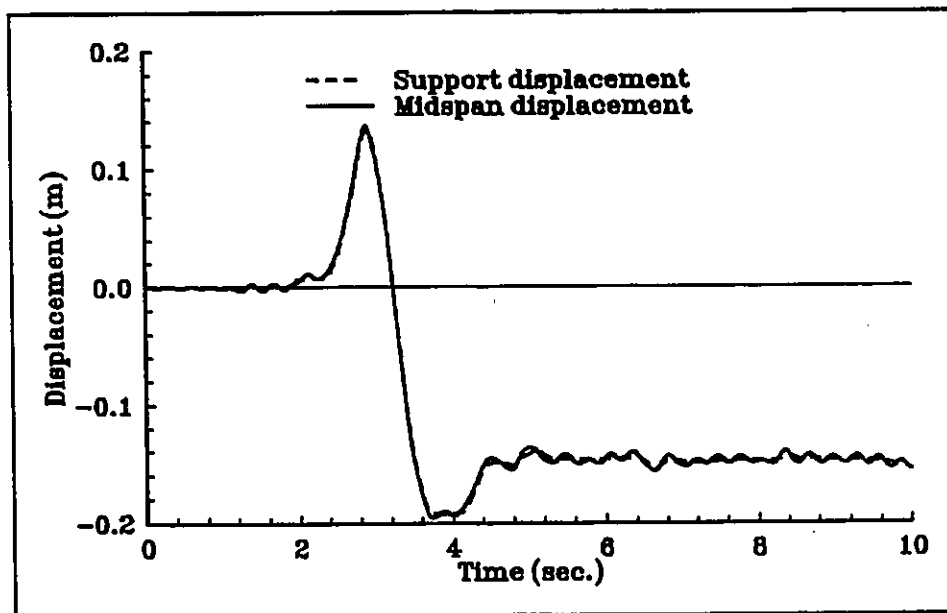
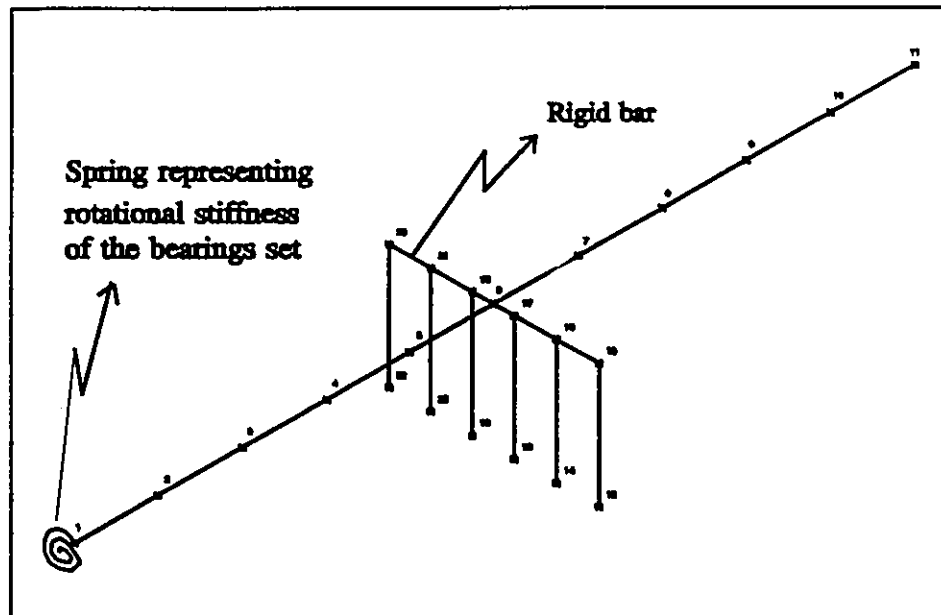
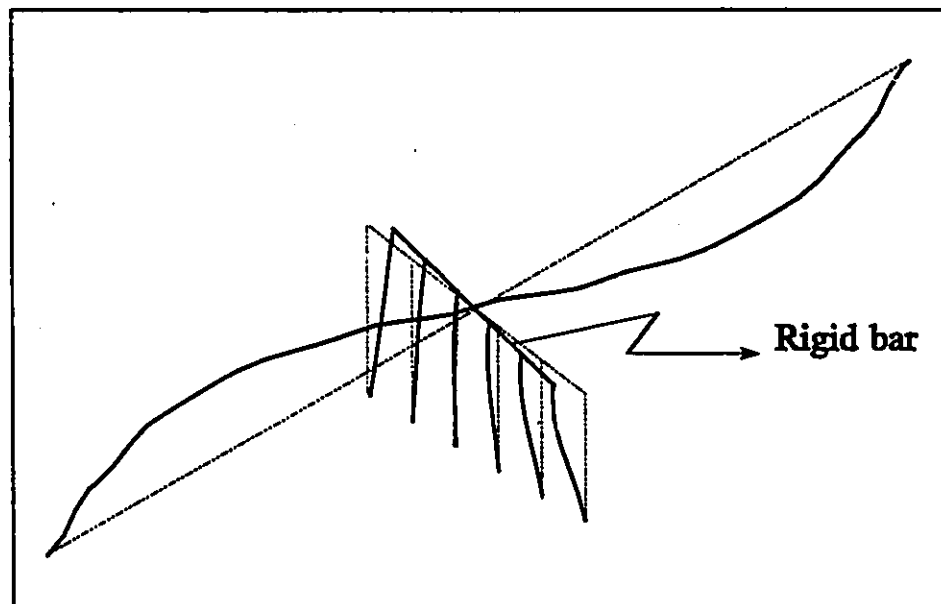


Figure 7.26 Support sliding time history of 2-lane 60 m. simply supported bridge (Erzincan Earthquake, NS component  $A_p = 0.4g$ )





**Figure 8.1** Elastic model of continuous bridges



**Figure 8.2** Function of the rigid bar, as illustrated using the second free vibration mode shape

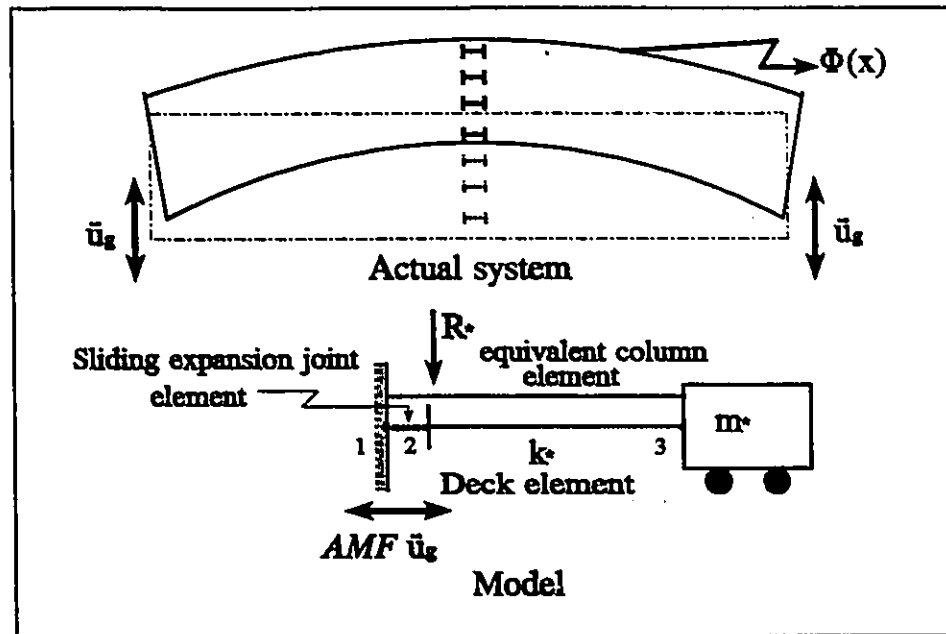


Figure 8.3 Actual system and non-linear model for two span continuous bridge

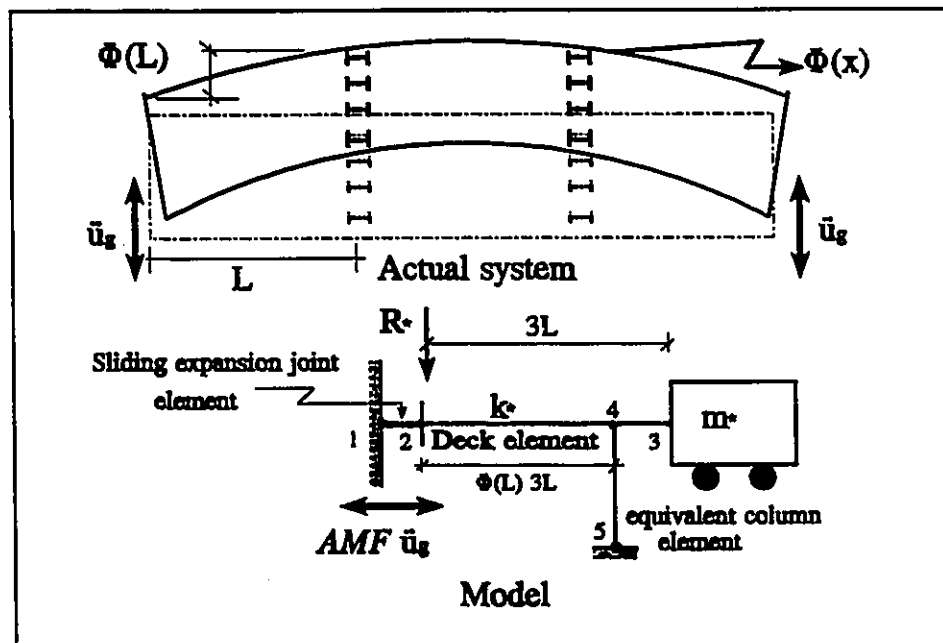


Figure 8.4 Actual system and non-linear model for three span continuous bridge

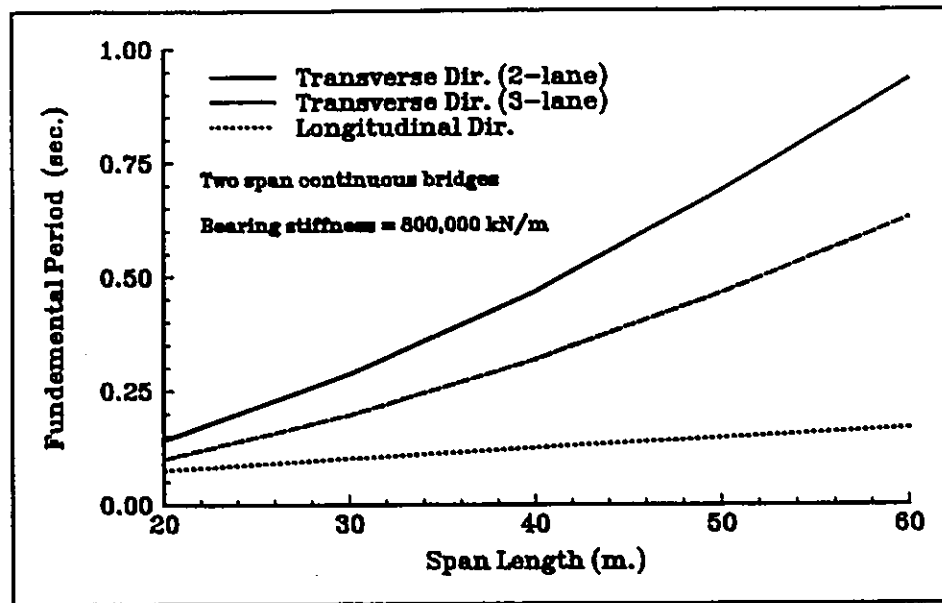


Figure 8.5 Transverse and longitudinal direction fundamental periods of continuous bridges as a function of span length

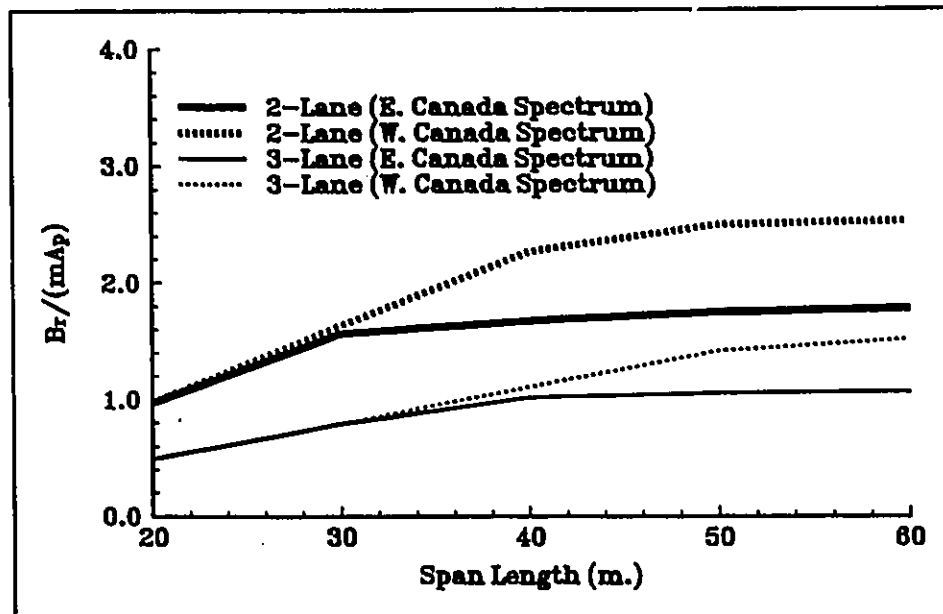


Figure 8.6 Transverse bearing force coefficient as a function of span length.

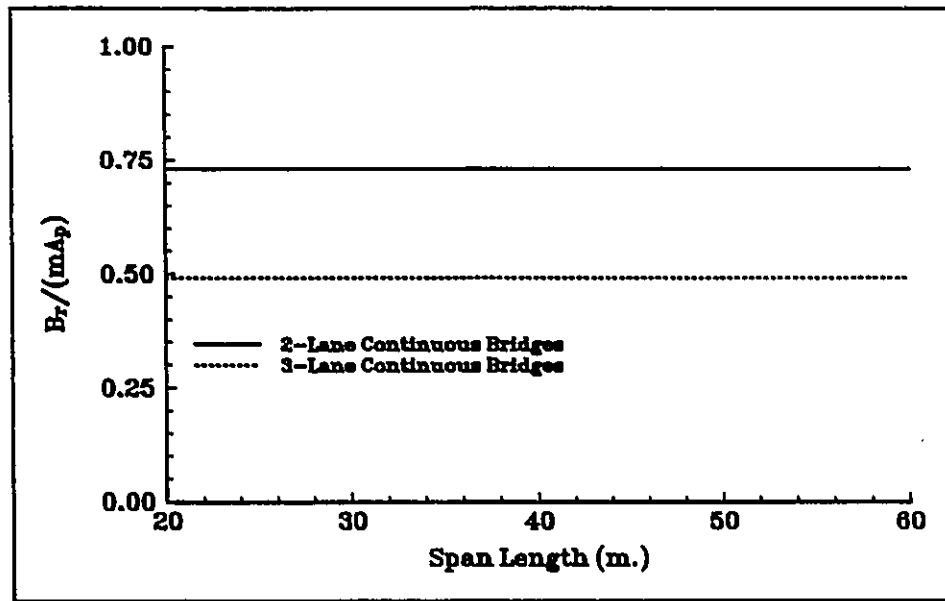


Figure 8.7 Longitudinal bearing force coefficient as a function of span length

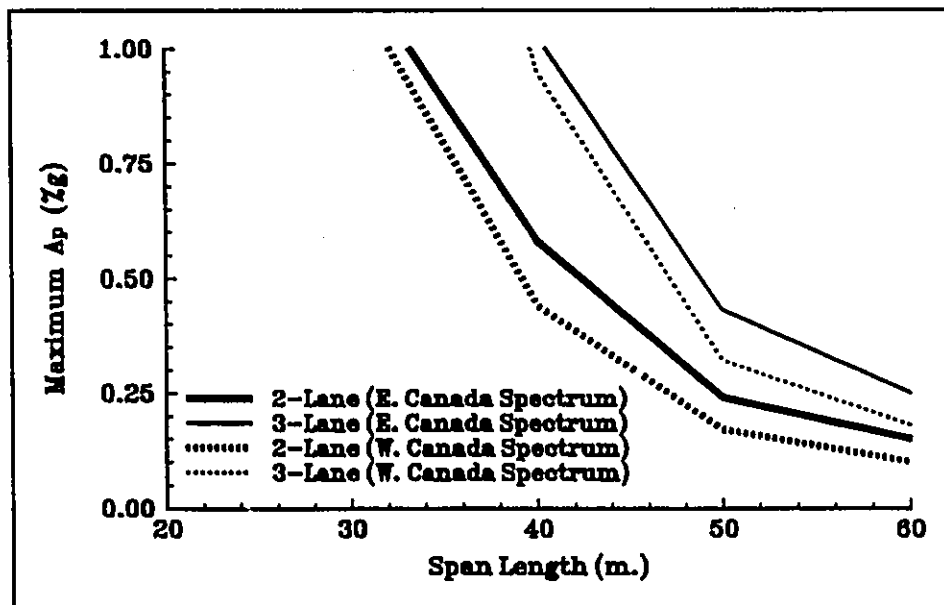


Figure 8.8 Maximum resistible peak ground acceleration as a function of span length (Linear elastic analysis, undamaged bearings)

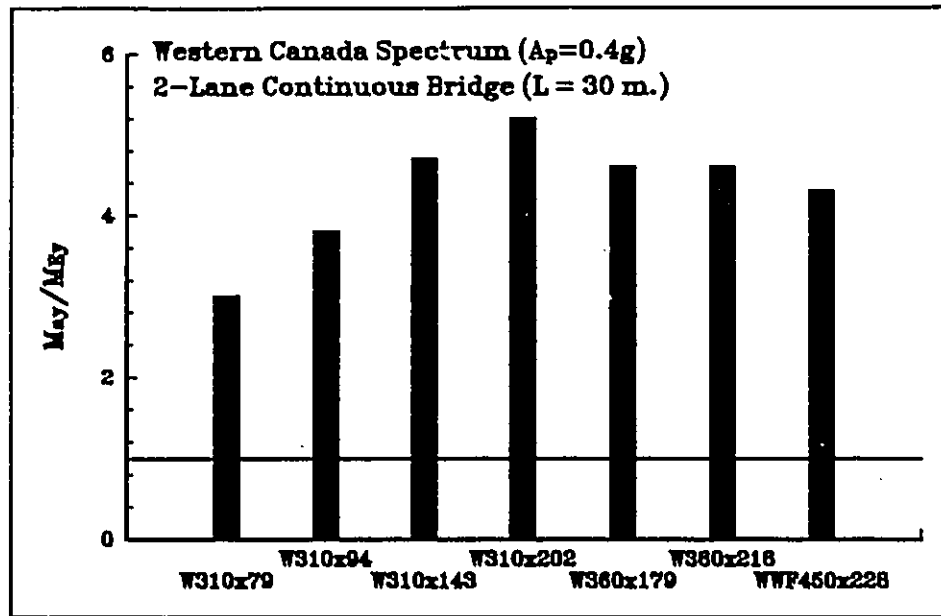


Figure 8.9 Effect of column size on the seismic capacity (Linear elastic analysis, undamaged bearings)

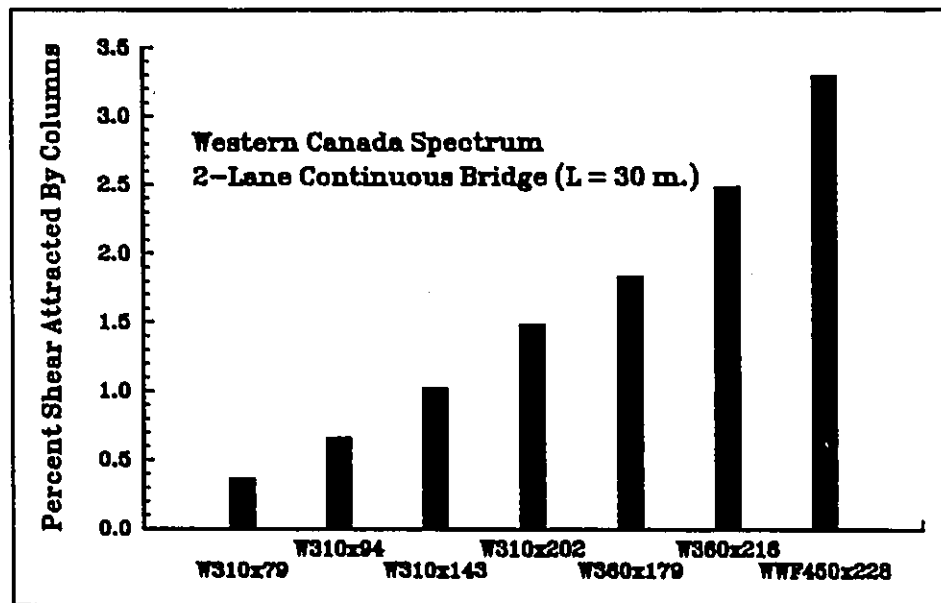


Figure 8.10 Effect of column size on the percentage of total force attracted by columns (Linear elastic analysis, undamaged bearings)

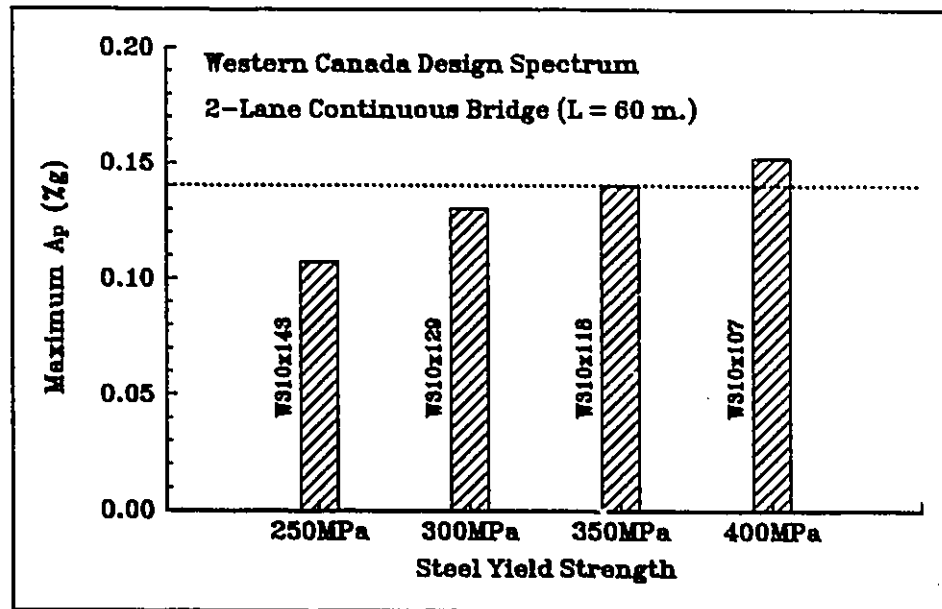


Figure 8.11 Effect of yield strength of steel on the seismic capacity (Linear elastic analysis, undamaged bearings)

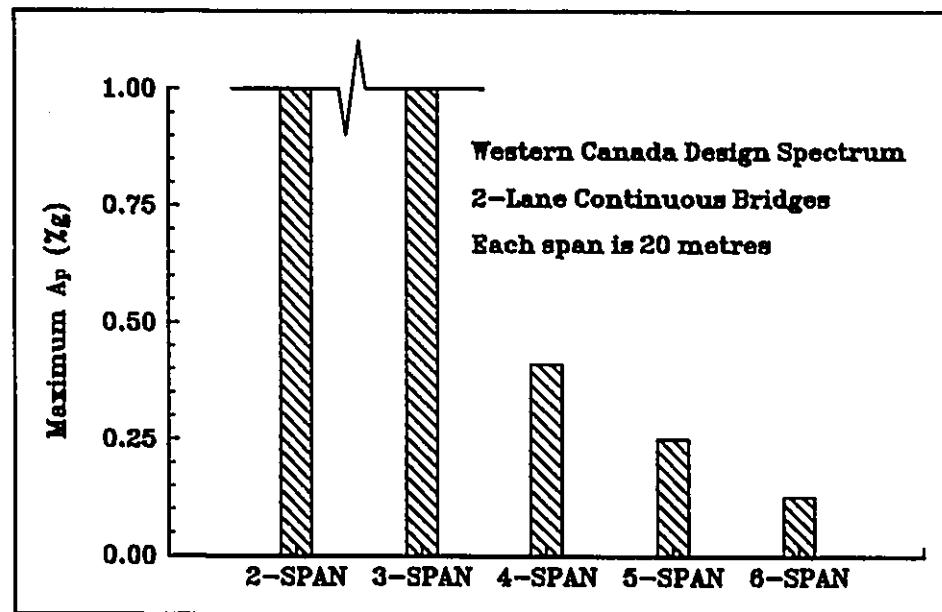


Figure 8.12 Effect of number of spans on the seismic capacity (Linear elastic analysis, undamaged bearings)

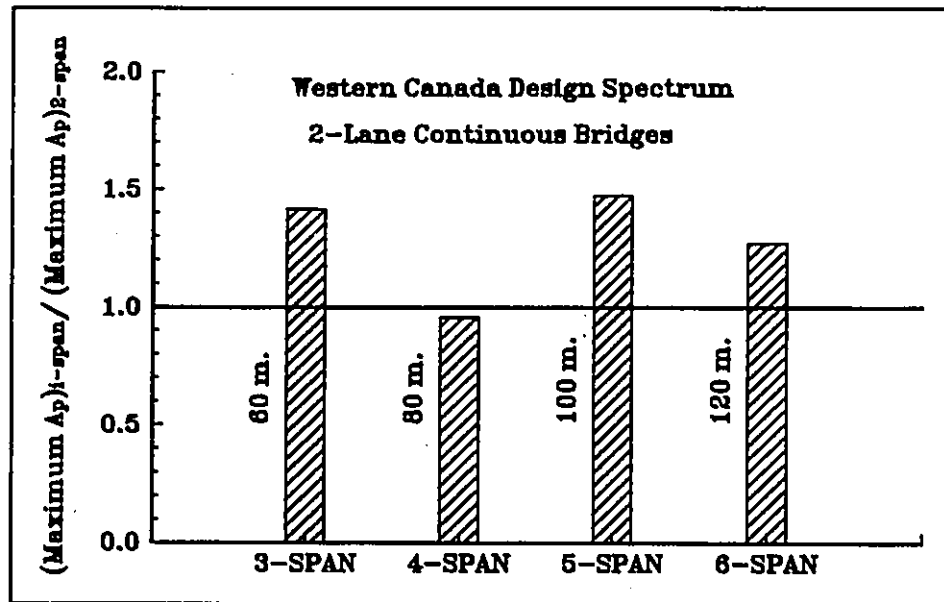


Figure 8.13 Ratio of maximum resistible peak ground acceleration for multiple span bridges to that for two span bridges of equal total length. (Linear elastic analysis, undamaged bearings)

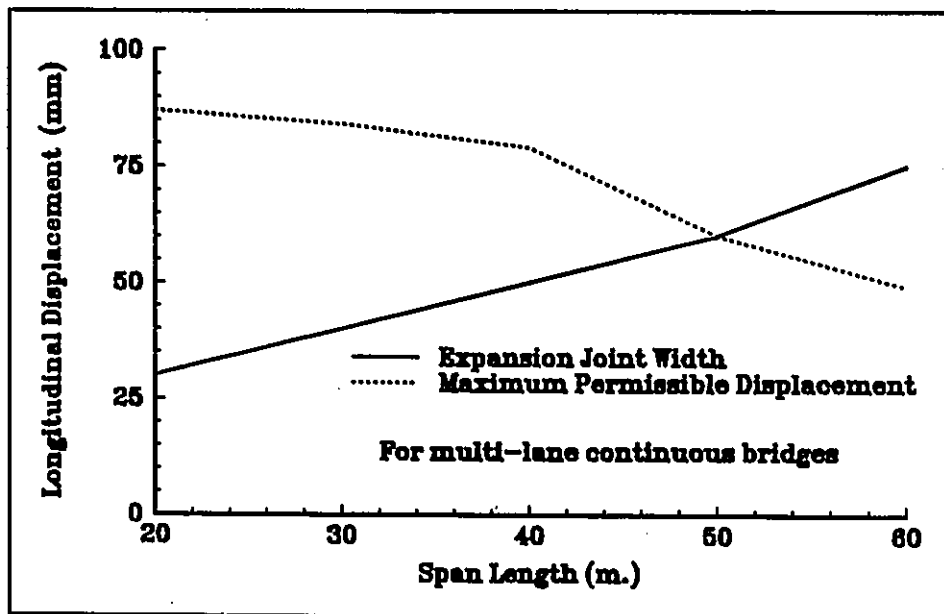


Figure 8.14 Maximum permissible first order longitudinal displacement prior to column instability as a function of span length (Linear elastic analysis, damaged bearings)

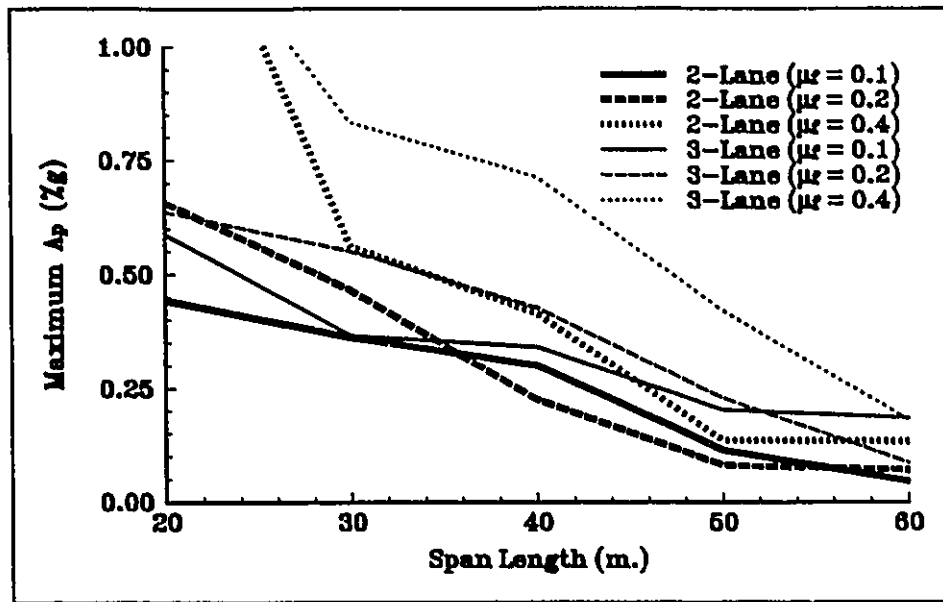


Figure 8.15 Maximum resistible peak ground acceleration as a function of span length, Average of Western USA earthquakes (Non-linear inelastic analysis, damaged bearings)

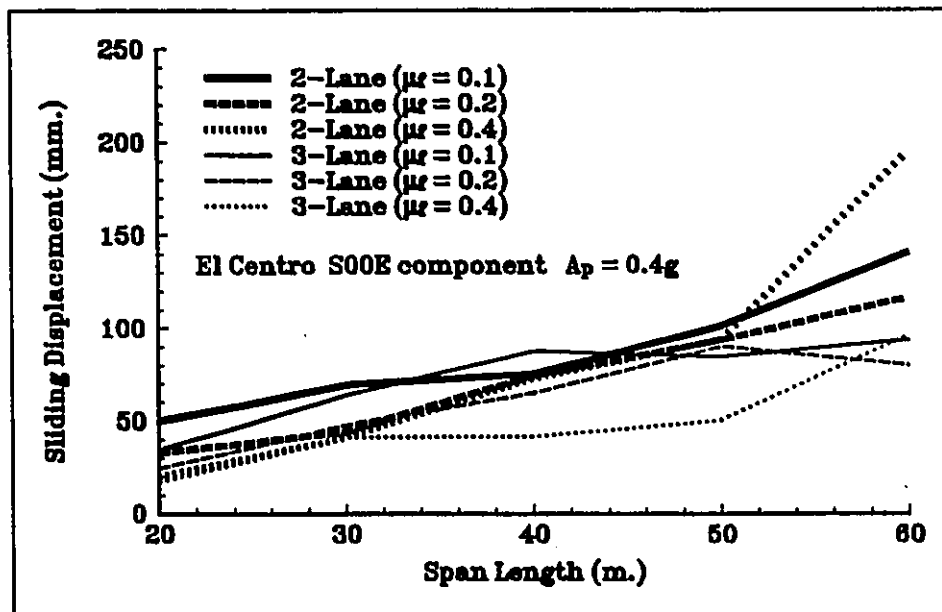


Figure 8.16 Sliding displacement at the support of 2 and 3-lane bridges (Non-linear inelastic analysis, damaged bearings)



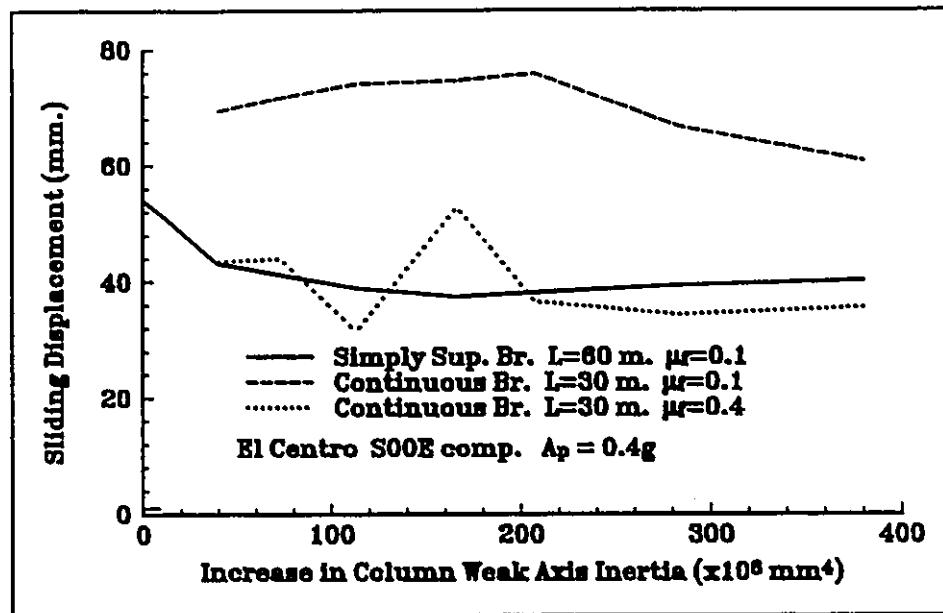


Figure 8.17 Effect of increased column stiffness on the sliding displacement,  $I_{\text{Deck cont.}}=1439 \times 10^9$  mm<sup>4</sup>,  $I_{\text{Deck simp.}}=2214 \times 10^9$  mm<sup>4</sup>

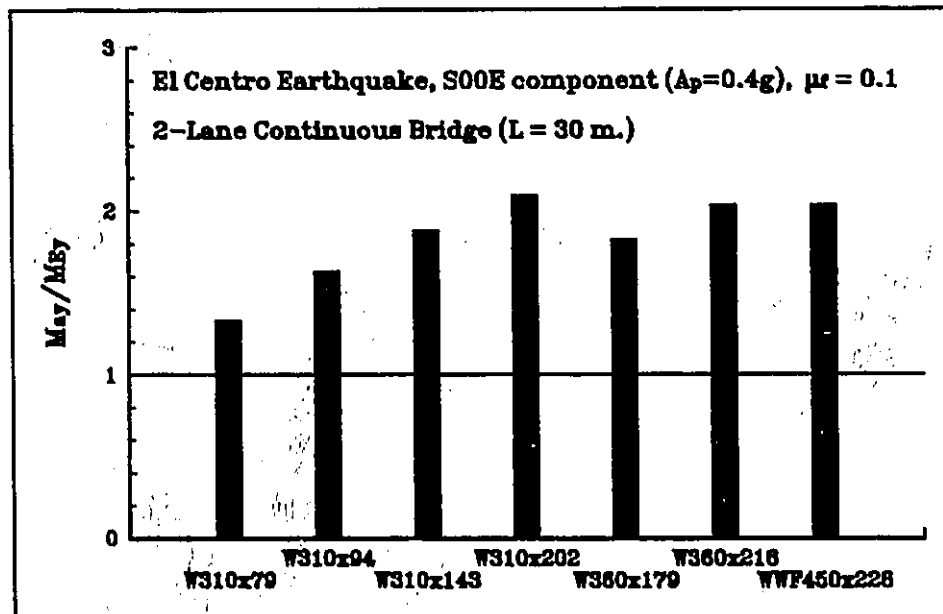
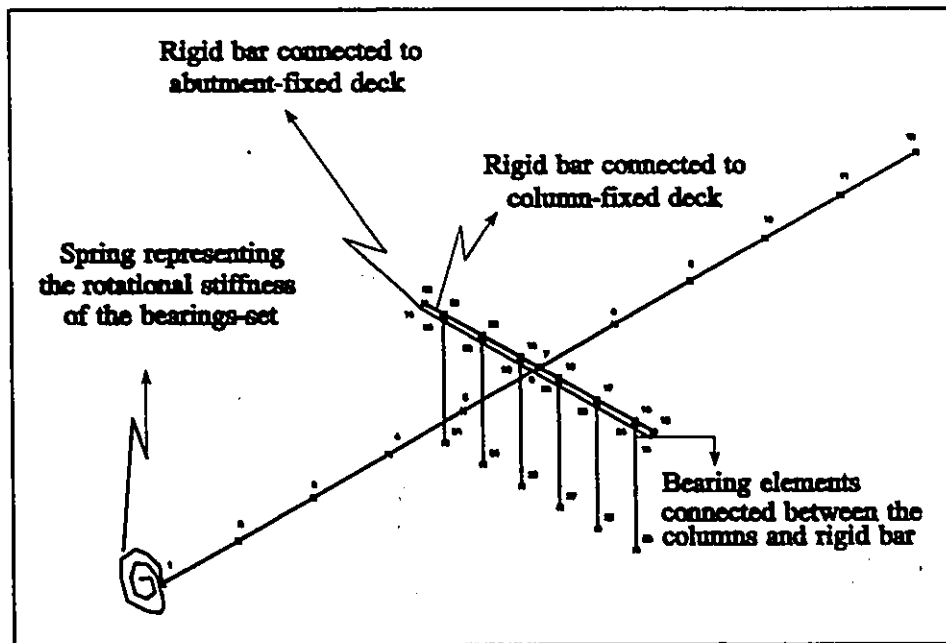
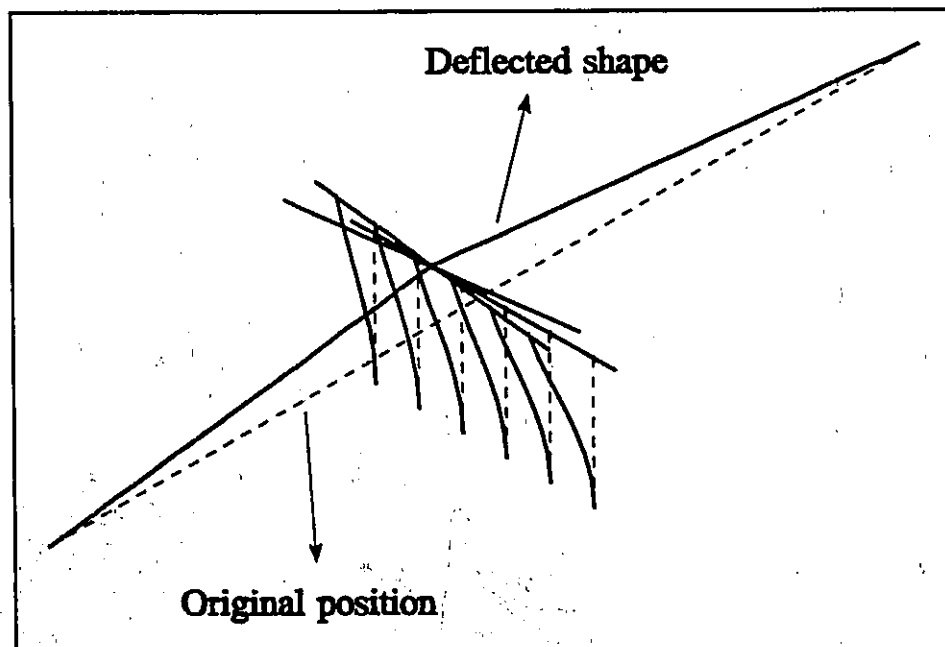


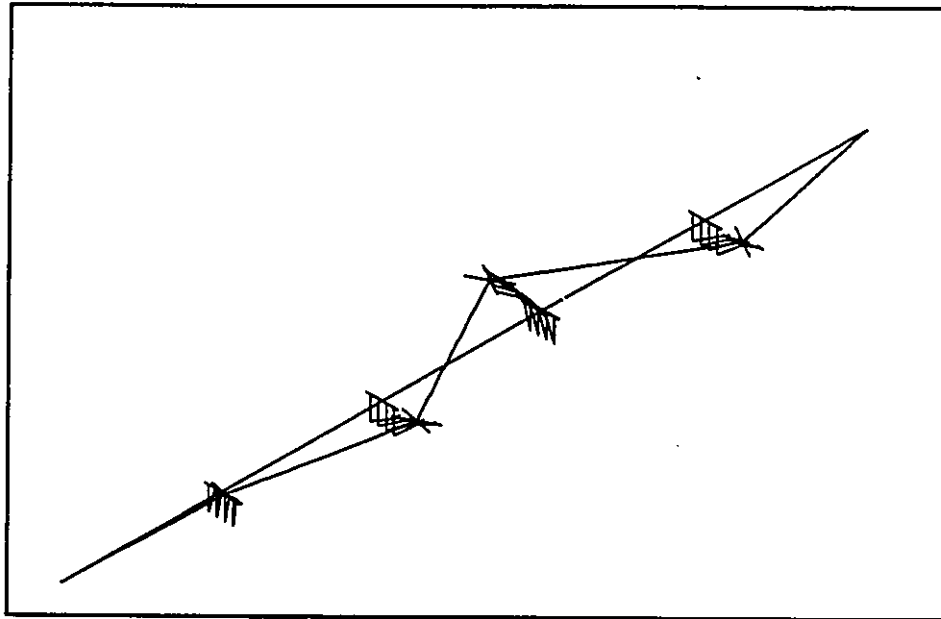
Figure 8.18 Effect of column size on the seismic capacity (Non-linear inelastic analysis, damaged bearings)



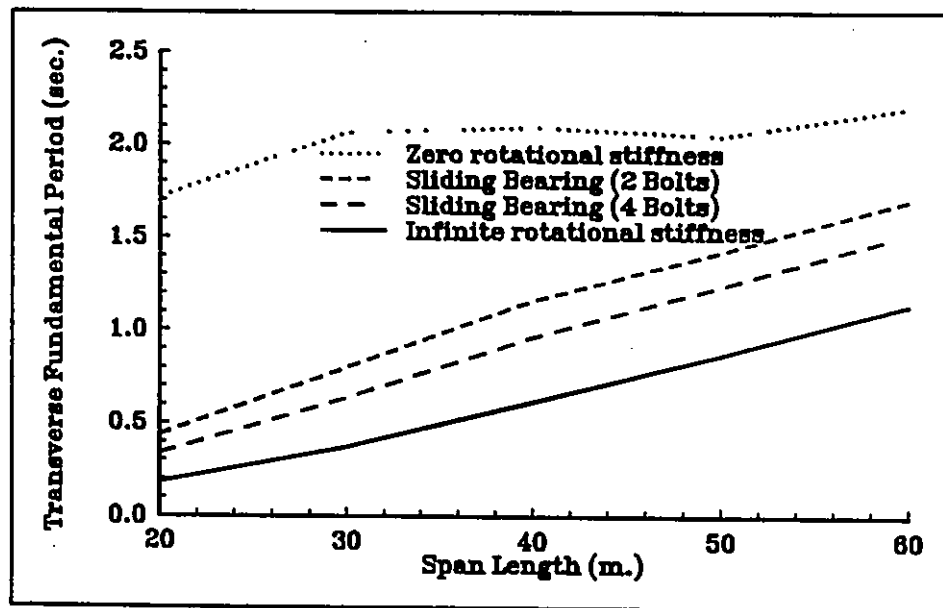
**Figure 9.1** Linear elastic model of 2-span simply supported bridges



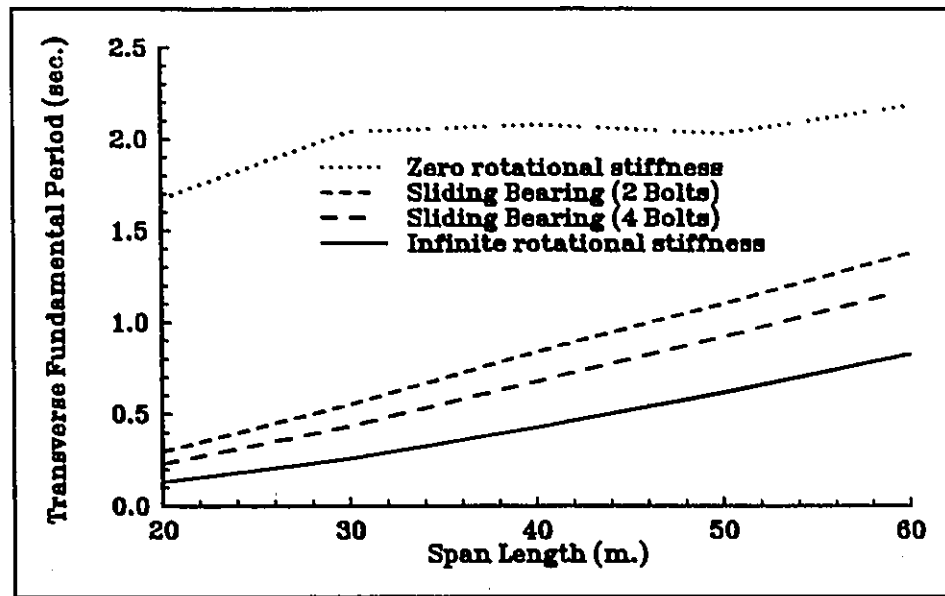
**Figure 9.2** Function of the rigid bar in multi-span simply supported bridges, as illustrated using the first vibration mode shape



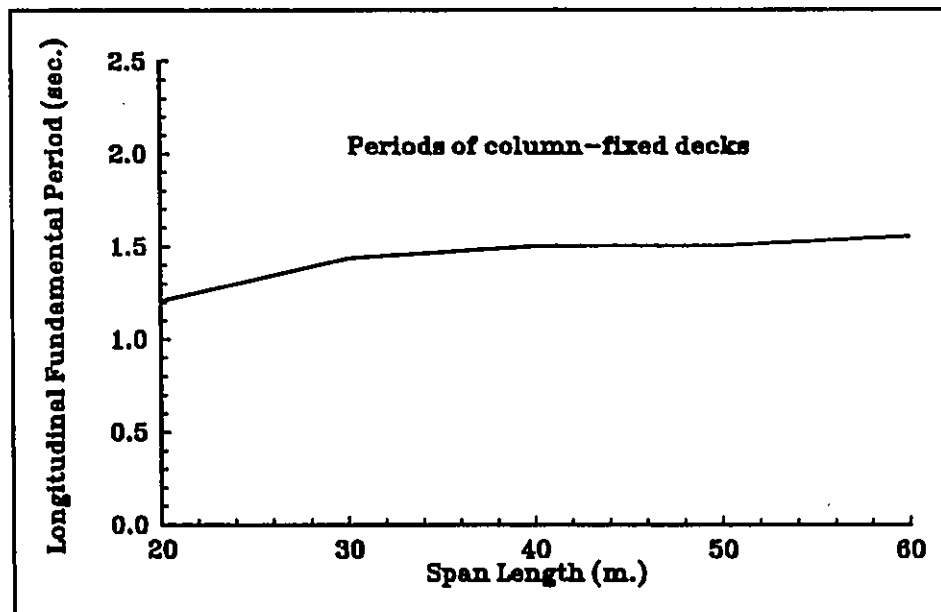
**Figure 9.3** Third vibration mode shape of a 5 span simply supported bridge with a rotational spring on the left support



**Figure 9.4** Transverse direction fundamental periods of 2-lane simply supported bridges with two spans as a function of span length for various bearing types



**Figure 9.5** Transverse direction fundamental periods of 3-lane simply supported bridges with two spans as a function of span length for various bearing types



**Figure 9.6** Longitudinal direction fundamental periods of column-fixed decks of multi-span simply supported bridges as a function of span length

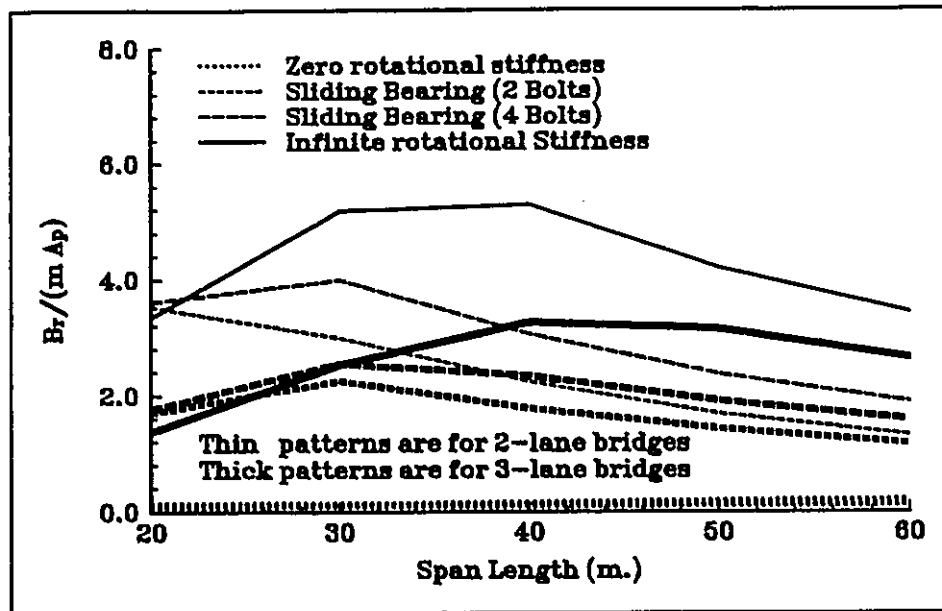


Figure 9.7 TBFC for 2 span simply supported bridges (MP1SD spectrum of Western USA earthquakes)

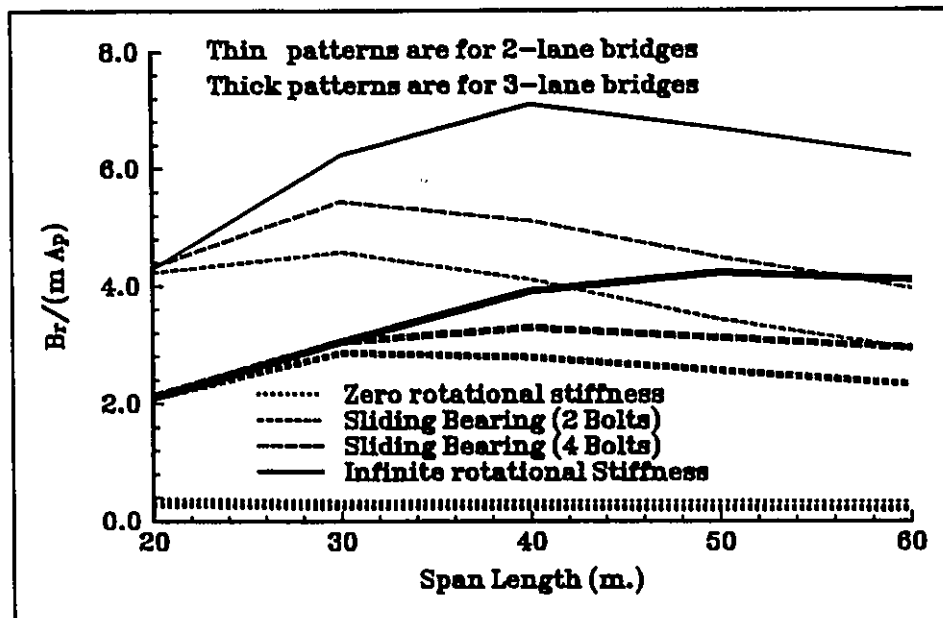


Figure 9.8 TBFC for 2 span simply supported bridges (Western Canada design spectrum)

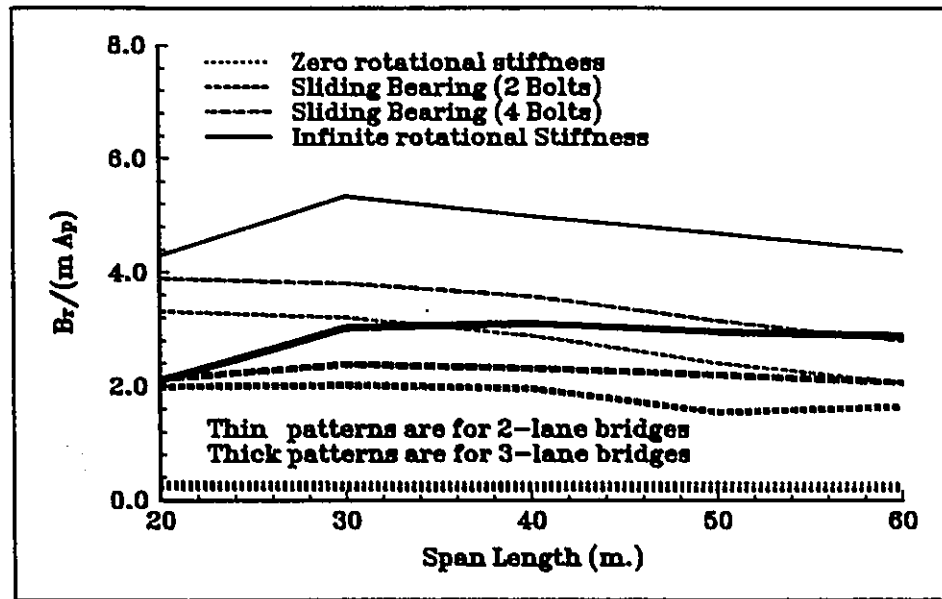


Figure 9.9 TBFC for 2 span simply supported bridges (Eastern Canada design spectrum)

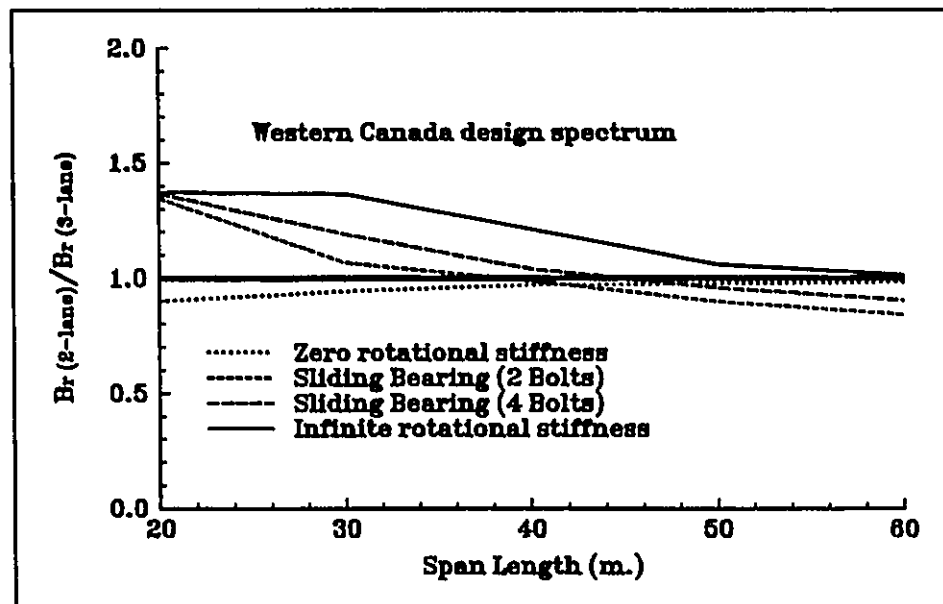


Figure 9.10 Ratio of transverse bearing forces of 2-lane and 3-lane bridges for different bearings as a function of span length (Western Canada design spectrum)

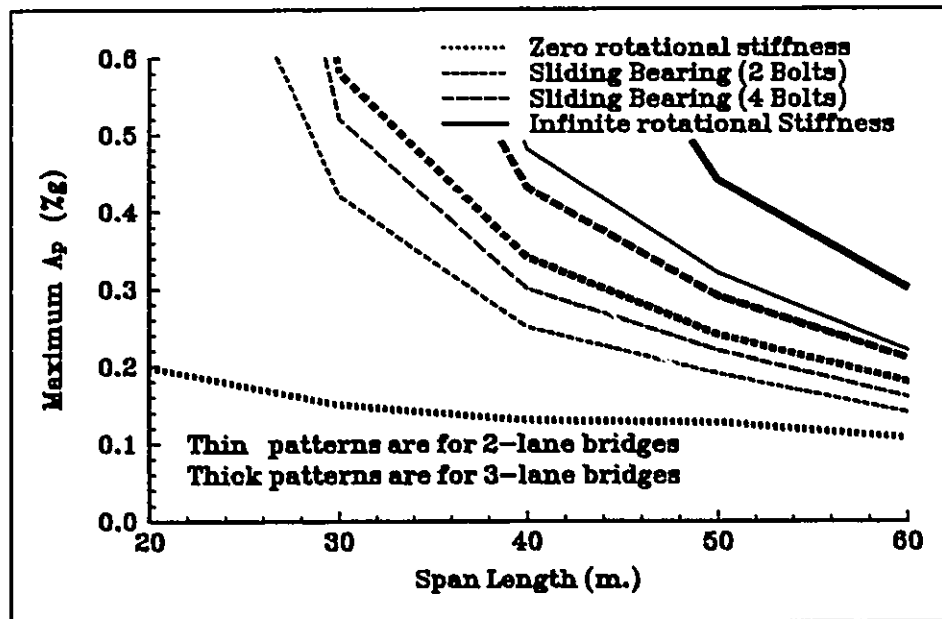


Figure 9.11 Maximum resistible peak ground accceleration as a function of span length (Western USA MP1SD spectrum)

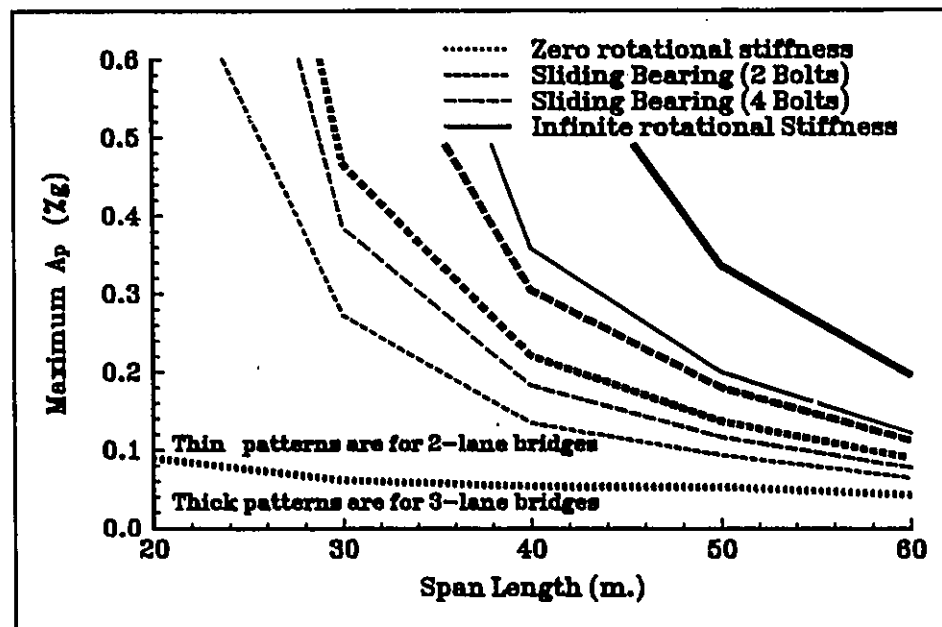


Figure 9.12 Maximum resistible peak ground accceleration as a function of span length (Western Canada design spectrum)

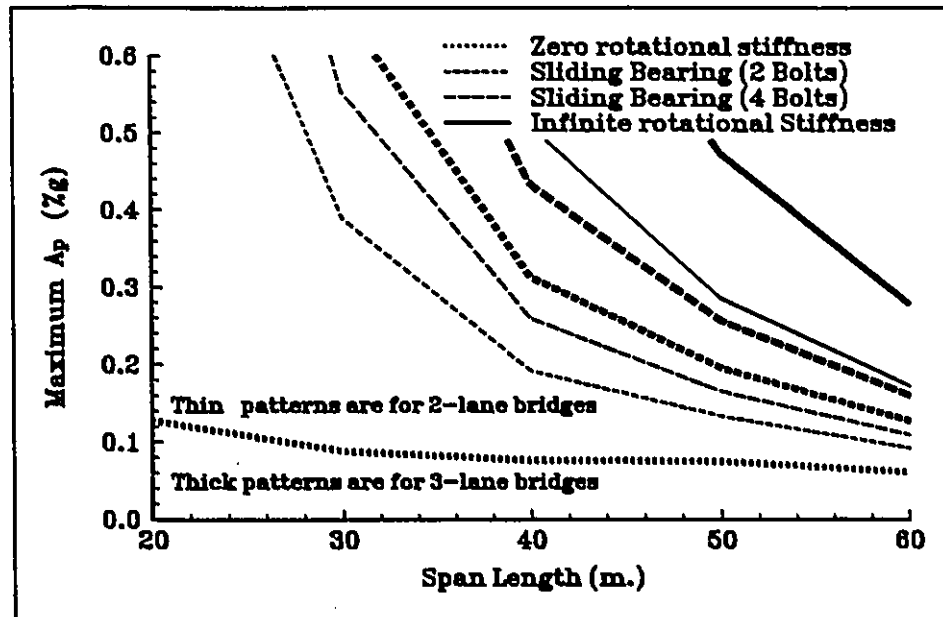


Figure 9.13 Maximum resistible peak ground acceleration as a function of span length (Eastern Canada design spectrum)

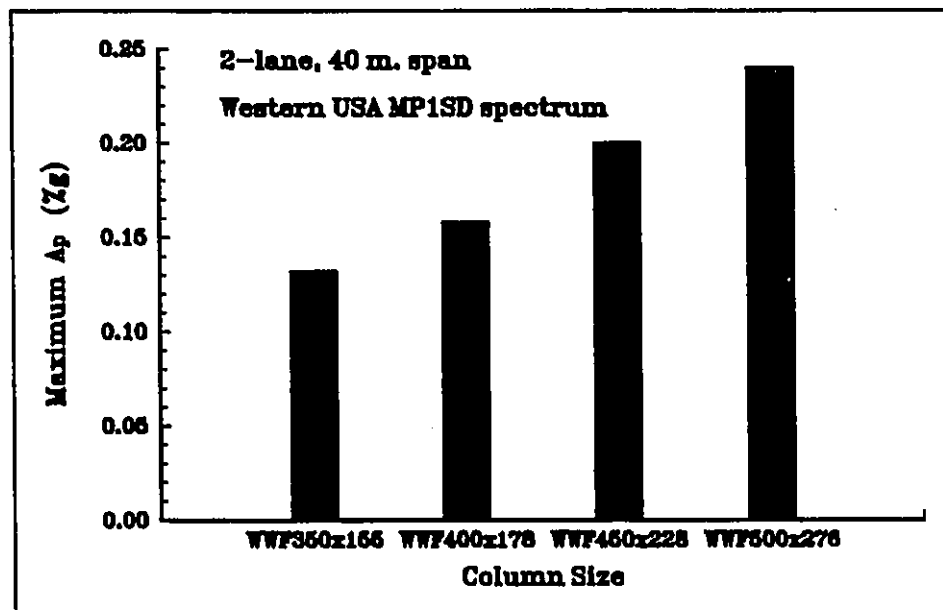
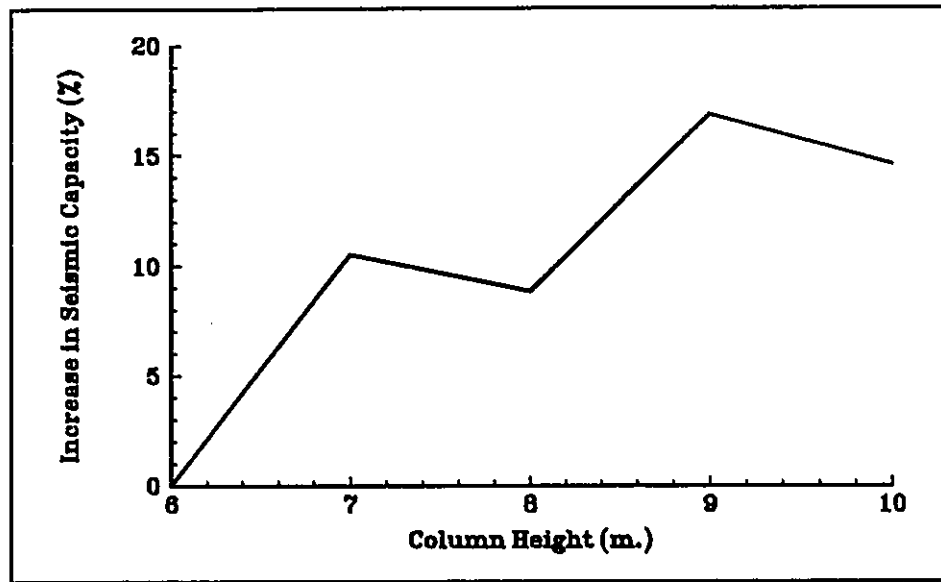
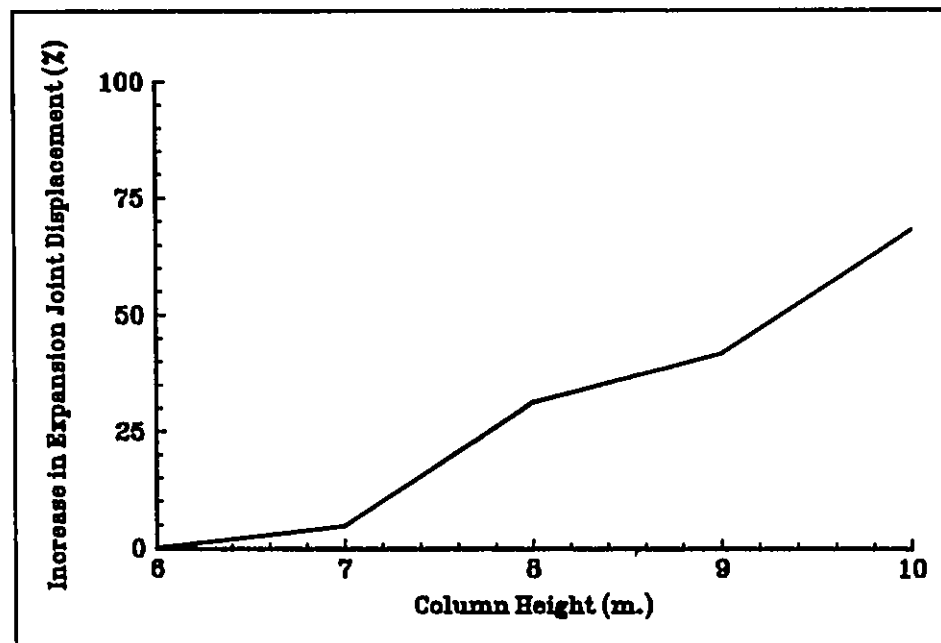


Figure 9.14 Effect of column size on the seismic capacity (zero rotational stiffness is assumed at the abutments, Western USA MP1SD spectrum is used)





**Figure 9.15** Percentage of increase in seismic capacity of bridges with different column length relative to that of the bridge with 6 m. column (Eastern Canada design spectrum)



**Figure 9.16** Percentage of increase in expansion joint opening relative to that of the bridge with 6 m. column.

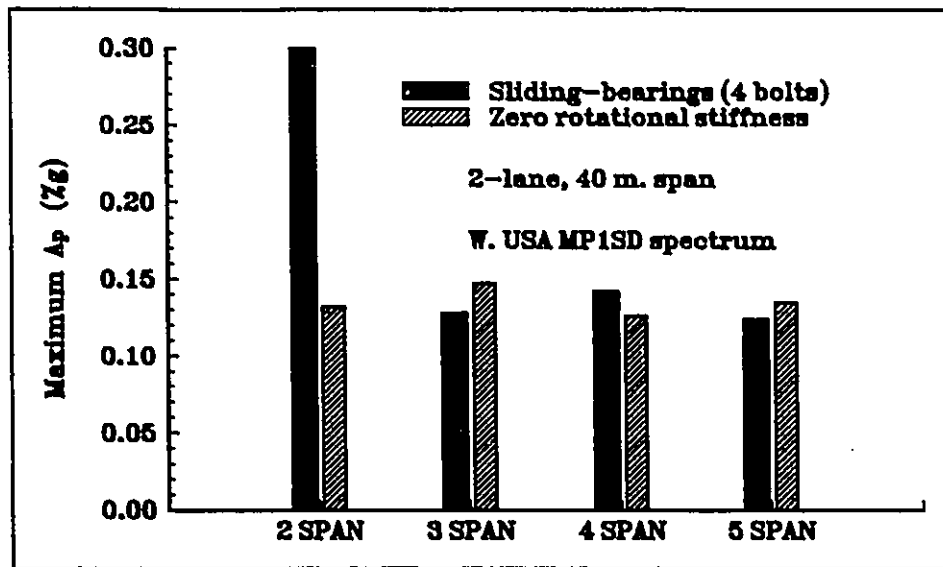


Figure 9.17 Effect of number of spans on the seismic capacity (Western USA MP1SD spectrum)

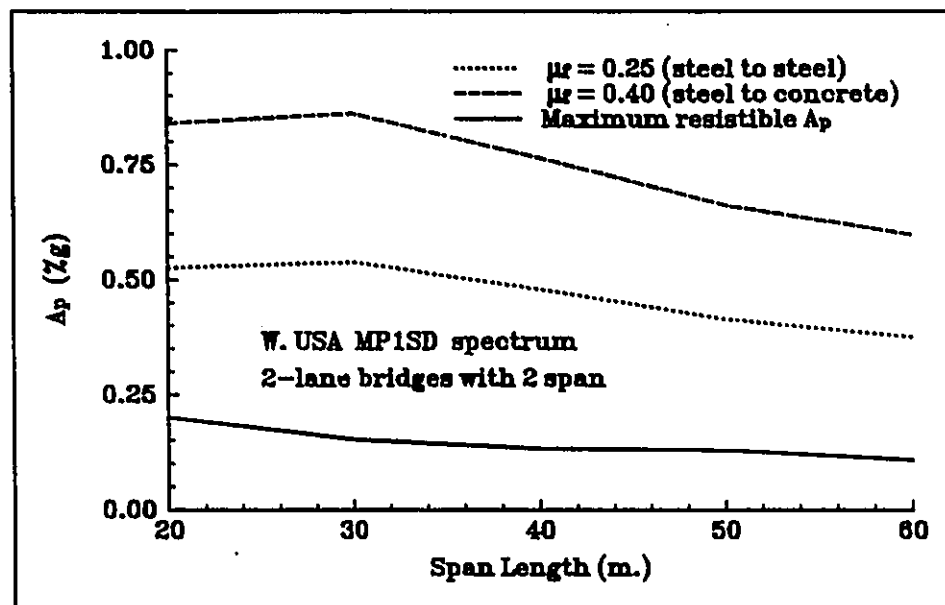


Figure 9.18 Comparison of peak ground acceleration required to initiate sliding and to produce columns instability (Western USA MP1SD spectrum)

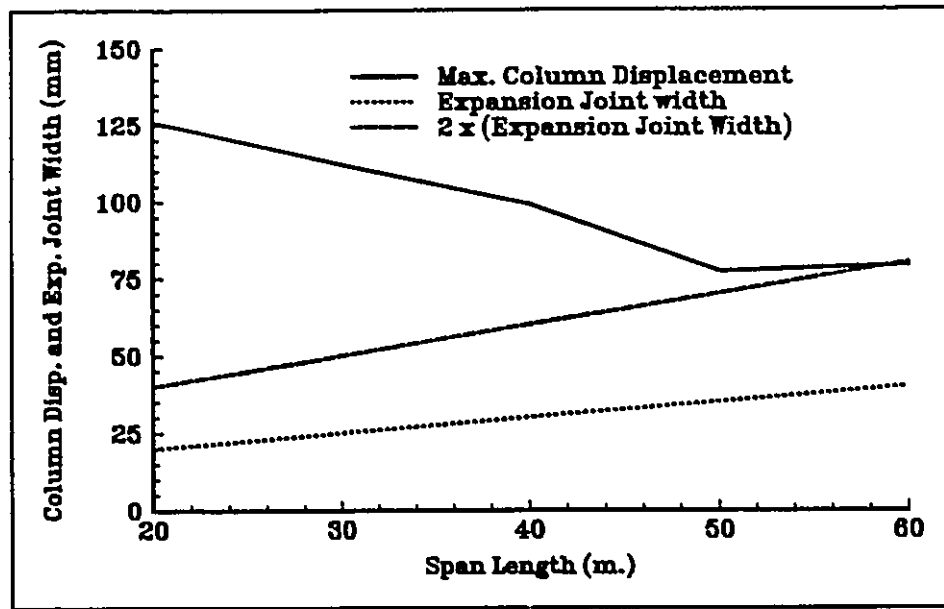


Figure 9.19 Comparison of maximum deck displacement prior to columns' instability failure with expansion joint width

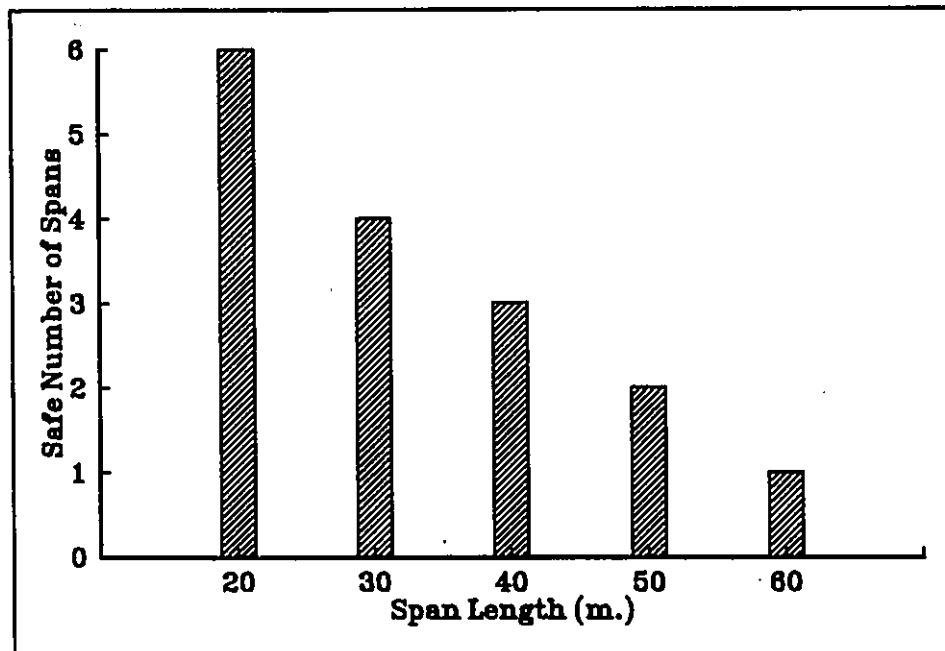
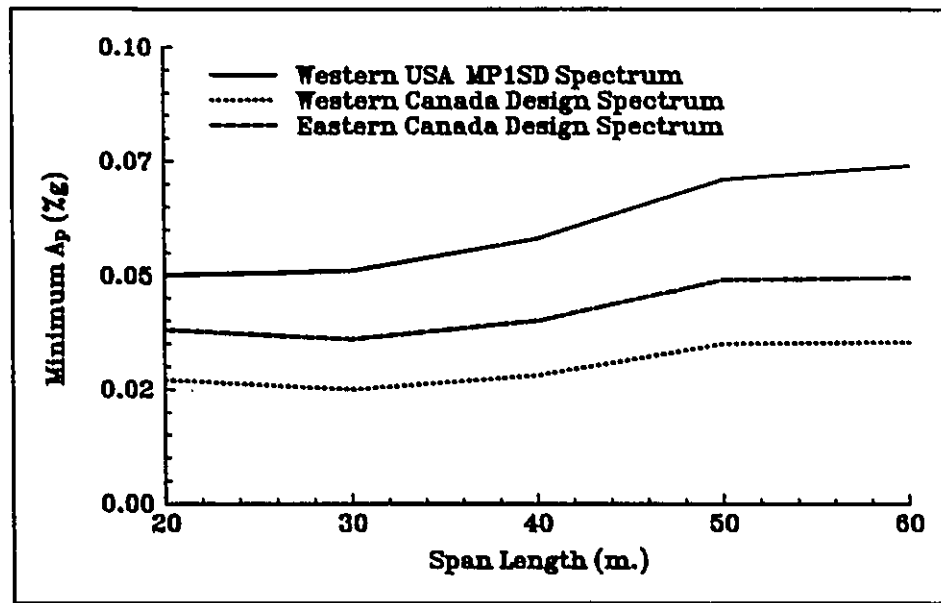
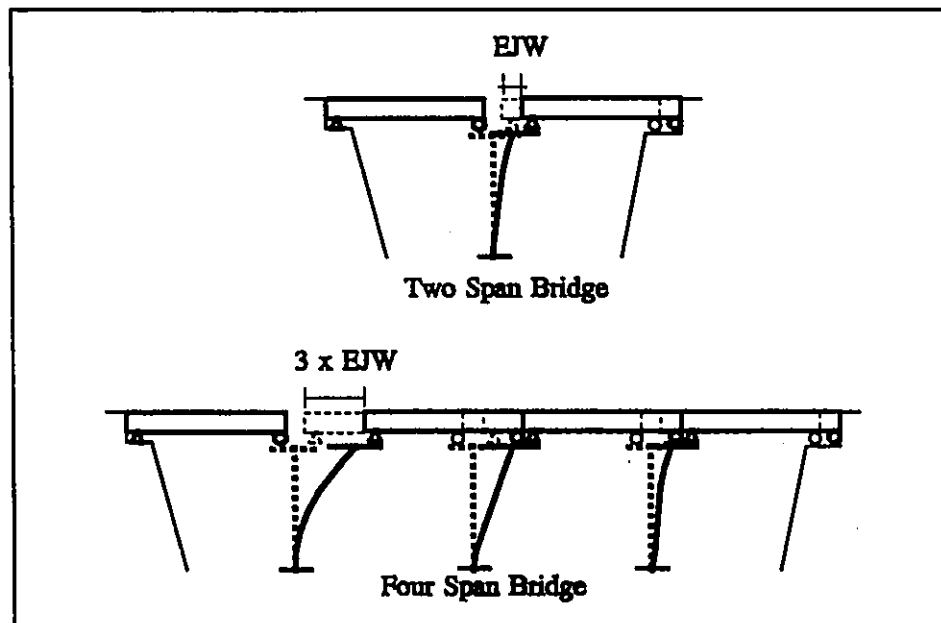


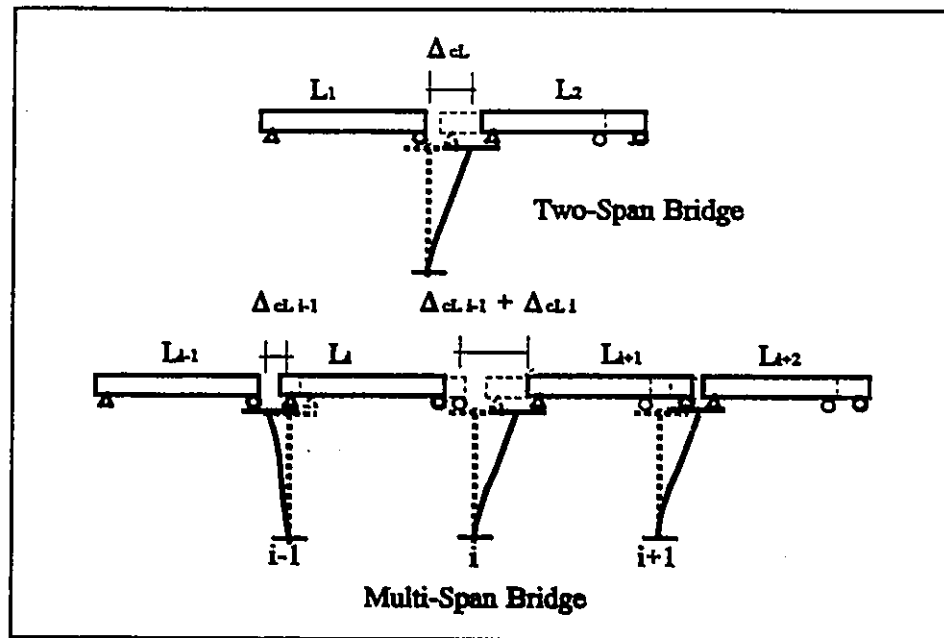
Figure 9.20 Safe number of spans considering failure of columns due to longitudinal direction seismic excitation



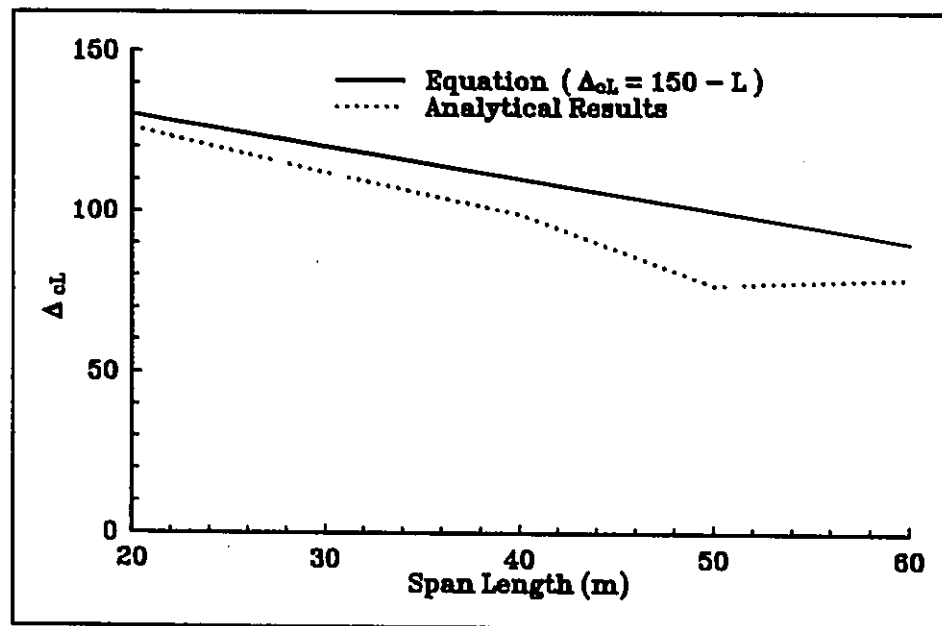
**Figure 9.21** Minimum peak ground acceleration needed to produce collision



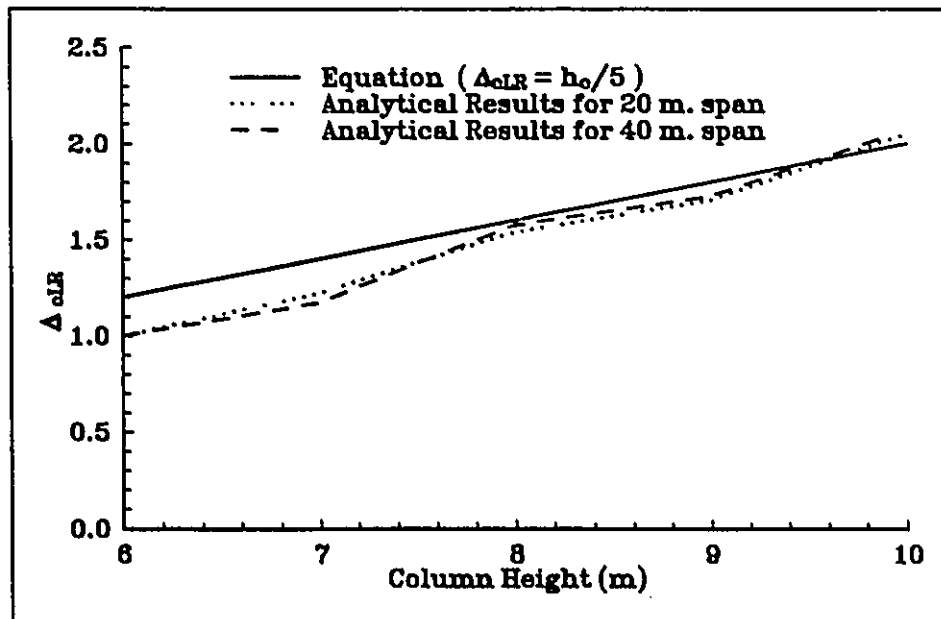
**Figure 9.22** Maximum possible expansion joint opening due to longitudinal direction displacement of the structure



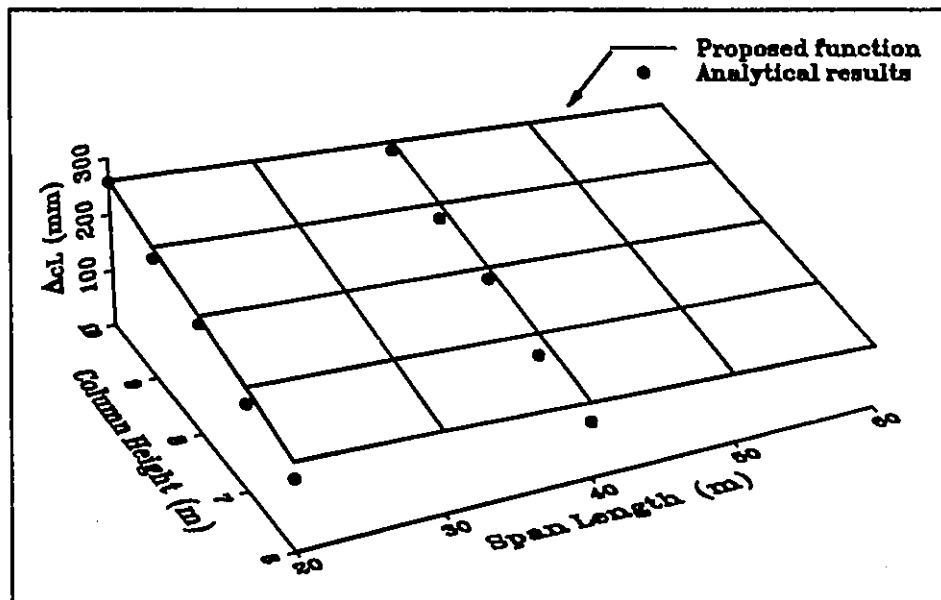
**Figure 9.23** Maximum expansion joint opening before the columns fail due to seismic excitation in the longitudinal direction.



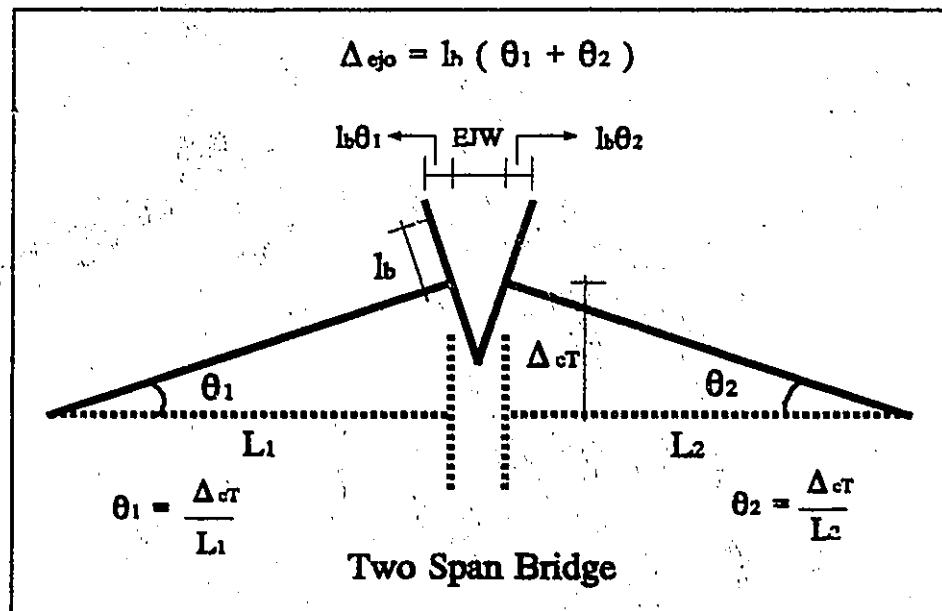
**Figure 9.24** Comparison of exact analytical solutions for the maximum longitudinal displacement of columns before failure with the proposed approximate equation



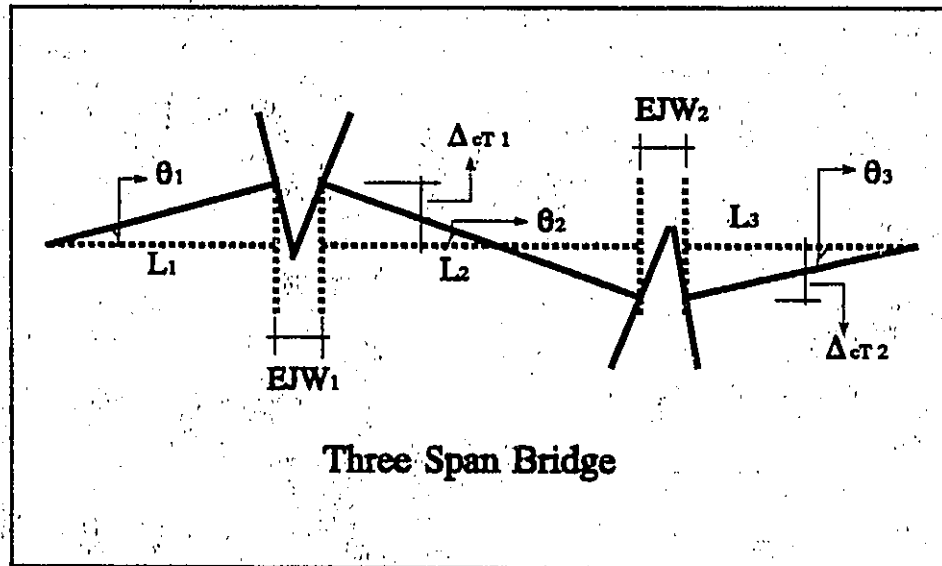
**Figure 9.25** Comparison of analytical exact results with the approximate proposed equation for the ratio of maximum longitudinal displacement of columns to that of 6 metres column.



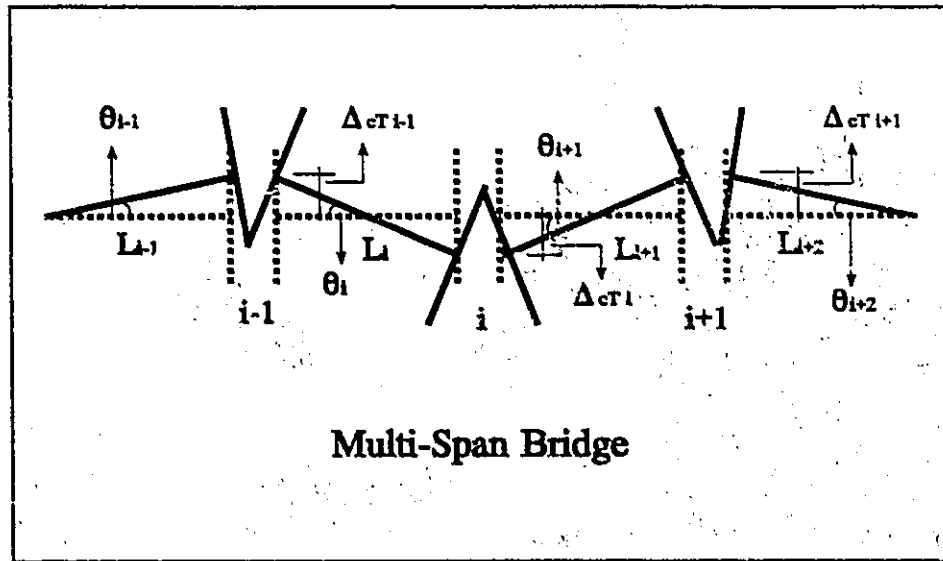
**Figure 9.26** Comparison of exact analytical results with the proposed function for maximum longitudinal displacement of columns before failure as a function of span length and column height



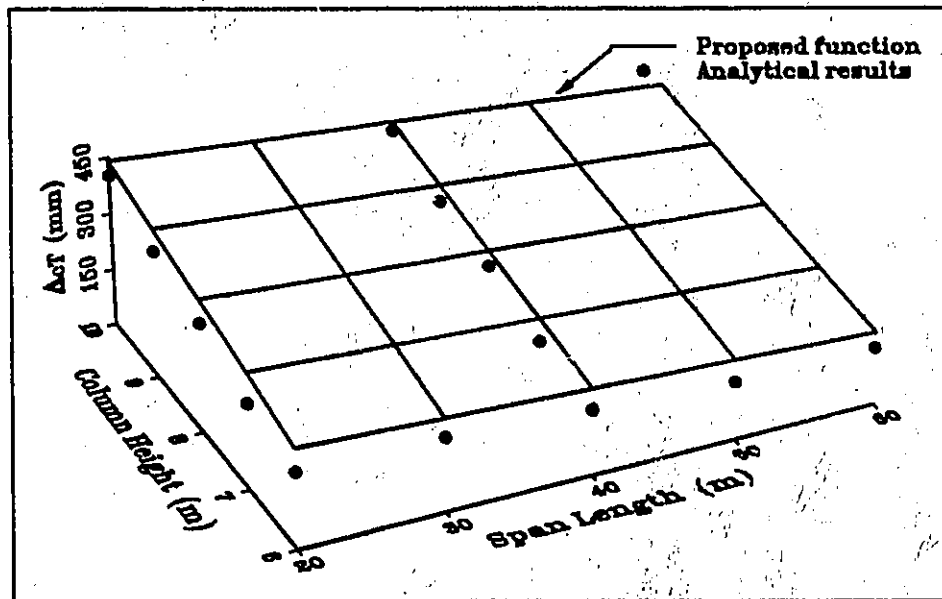
**Figure 9.27** Maximum expansion joint opening of a two span simply supported bridge before the columns fail due to seismic excitation in the transverse direction



**Figure 9.28** Maximum expansion joint opening of a three span simply supported bridge before the columns fail due to seismic excitation in the transverse direction



**Figure 9.29** Maximum expansion joint opening of a multi-span simply supported bridge before the columns fail due seismic loading in the transverse direction



**Figure 9.30** Comparison of exact analytical results with the proposed function for maximum transverse displacement of columns before failure as a function of span length and column height



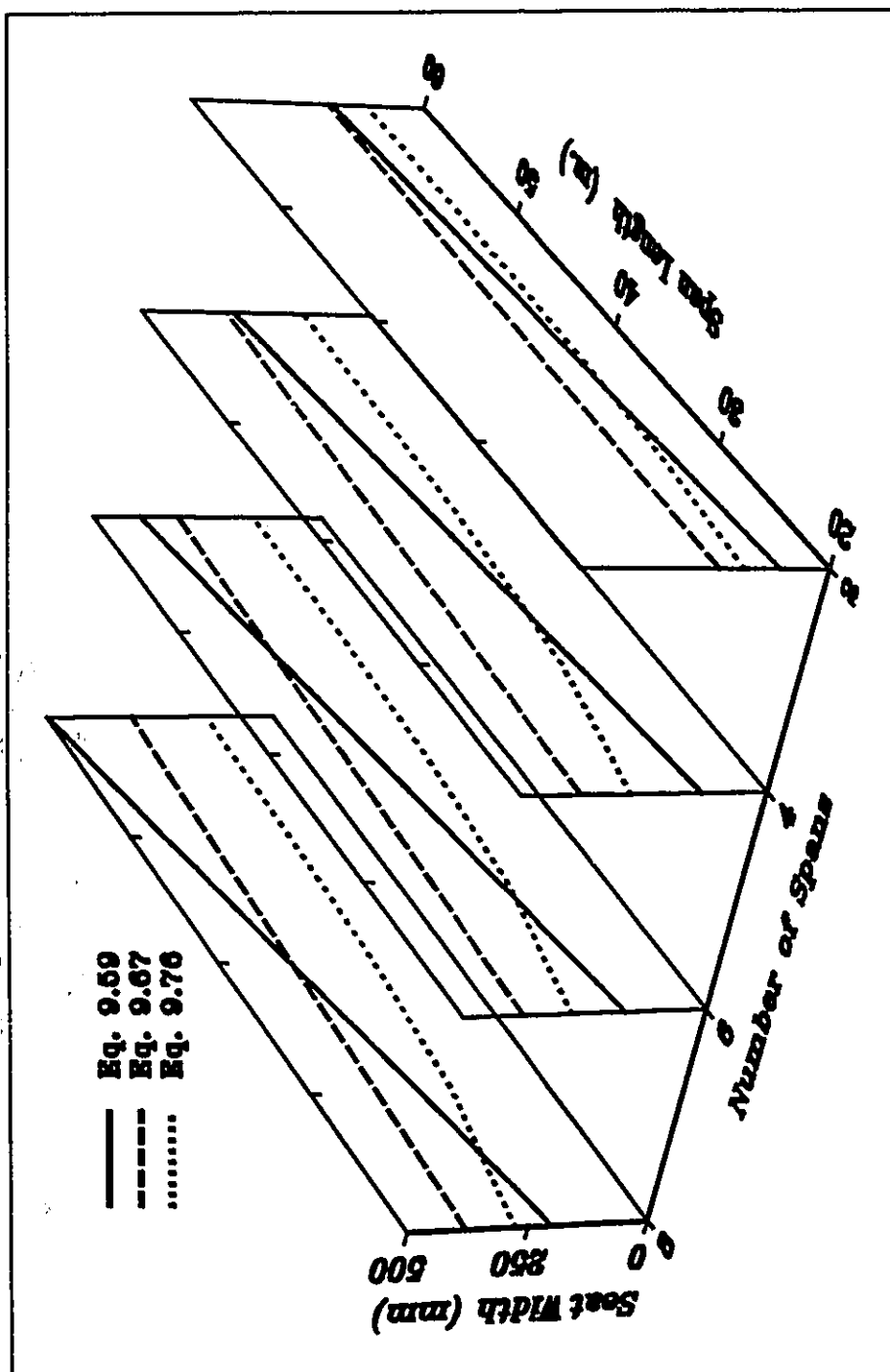


Figure 9.31 Comparison of proposed equations

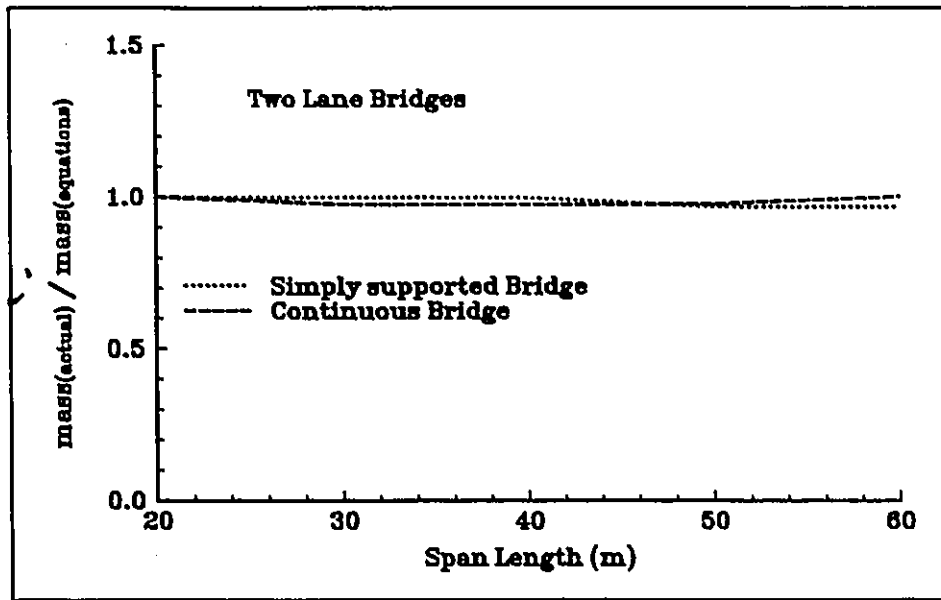


Figure 10.1 Ratio of the actual mass of bridges to those obtained from the proposed empirical equations, for 2 lane bridges

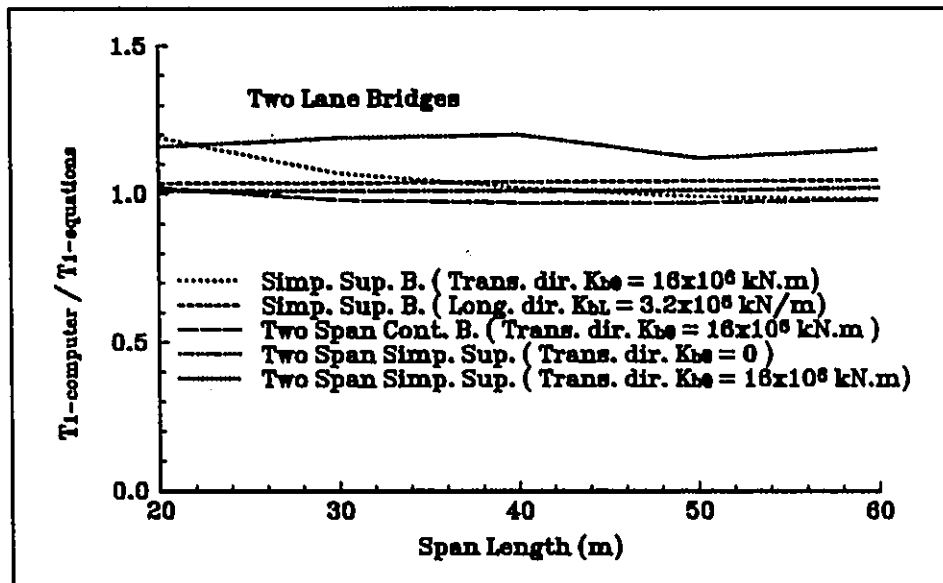
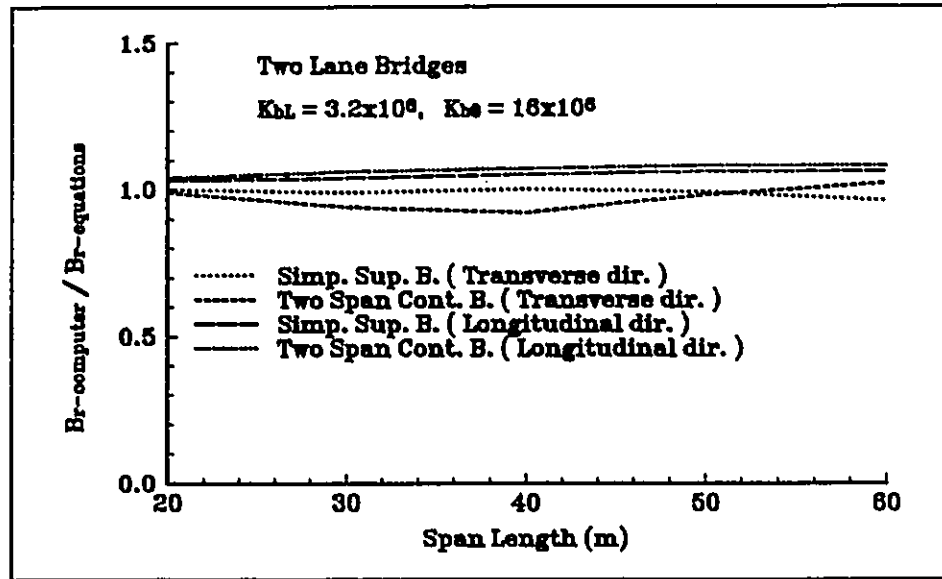
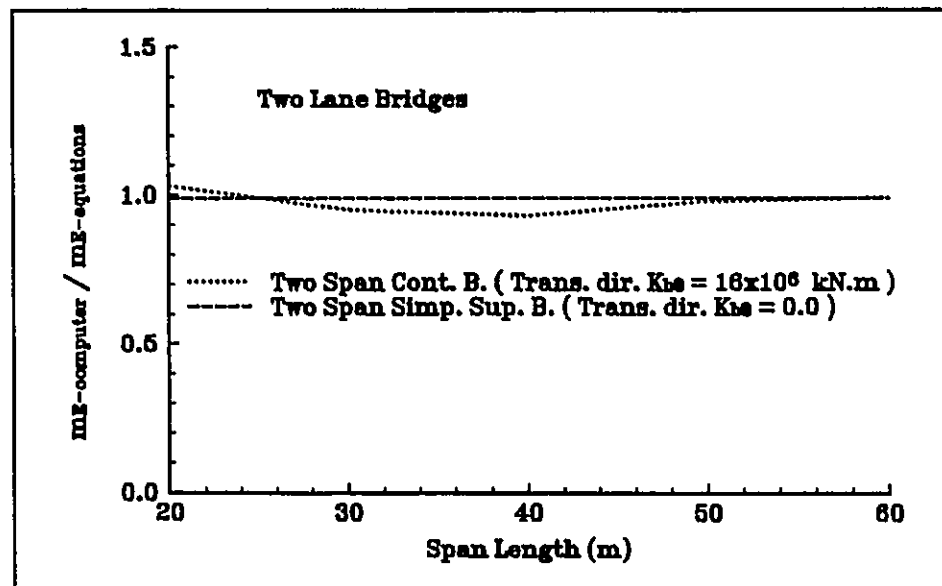


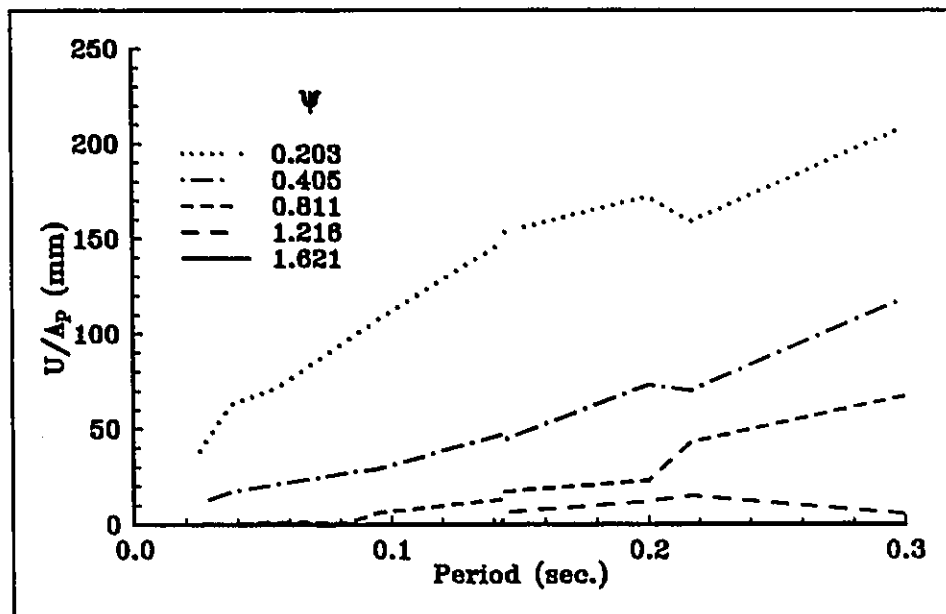
Figure 10.2 Ratio of the fundamental periods obtained from computer analyses to those obtained from the proposed equations, for 2 lane bridges



**Figure 10.3** Ratio of bearing forces obtained from computer analyses to those obtained from proposed equations, two lane bridges,  $K_{bL}=3.2 \times 10^6$ ,  $K_{be}=16 \times 10^6$



**Figure 10.4** Ratio of first order transverse seismic moments obtained from computer analyses to the ones obtained from the proposed equations for two lane bridges



**Figure 10.5** Sliding displacement per peak ground acceleration (%g) as a function of period for various  $\psi$  values (mean of Western USA earthquakes)

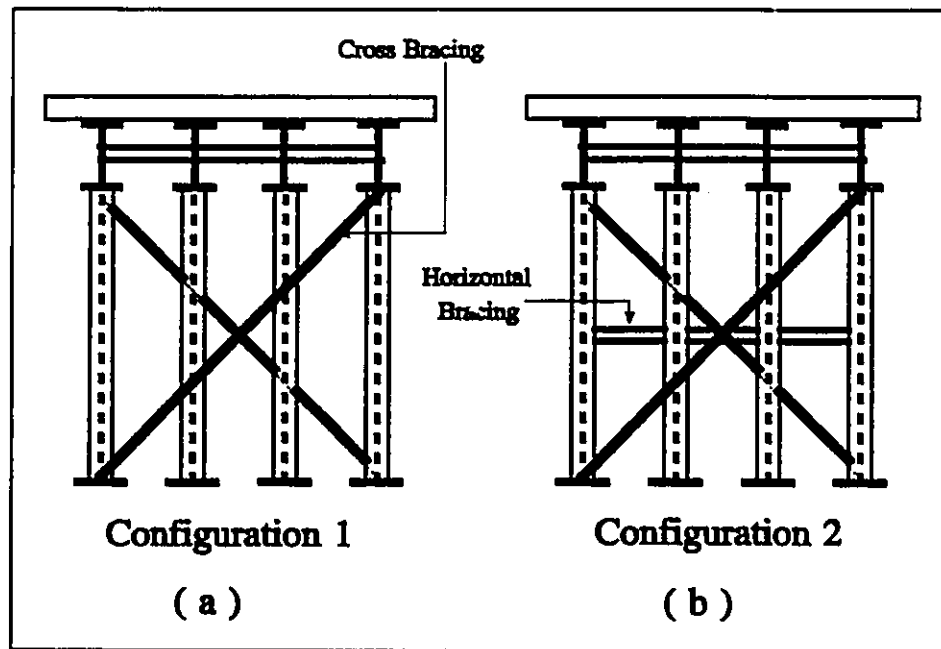


Figure 11.1 Cross bracing configurations

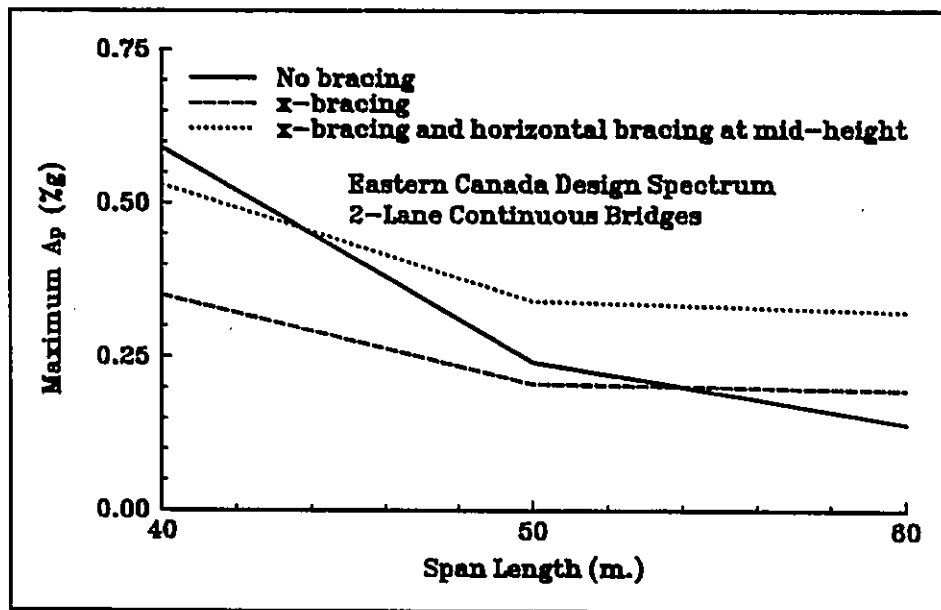
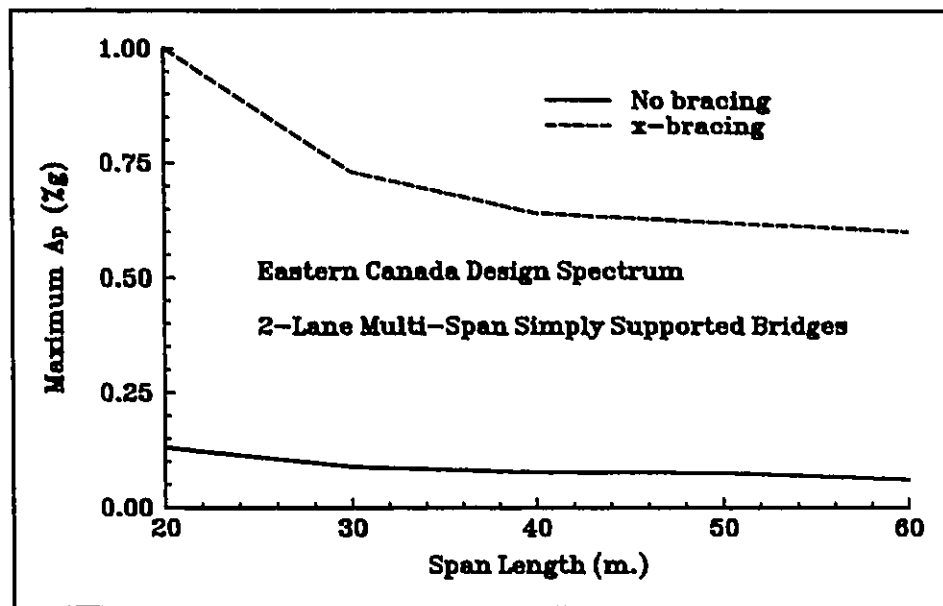


Figure 11.2 Effect of cross bracing on the seismic capacity of continuous bridges, (two lane bridges, Eastern Canada design spectrum)



**Figure 11.3** Effect of cross bracing on the seismic capacity of multi-span simply supported bridges (two lane bridges, Eastern Canada design spectrum)

## **APPENDIX A**

### **Computer Programs Developed to Investigate the Effect of Extreme Gravity Loads on Slab-On-Girder Steel Bridges**

```

. . . . .
. * THIS PROGRAM CALCULATES THE MAXIMUM MOMENT RATIO OF *
. * SPECIFIED HEAVY TRUCKS TO OHBD TRUCK AT THE MIDDLE *
. * OF SIMPLY SUPPORTED BRIDGES FOR DIFFERENT SPAN LENGTHS *
. . . . .

```

```

INPUT * PLEASE ENTER INPUT FILE NAME *;IFNS
INPUT * PLEASE ENTER OUTPUT FILE NAME *;OFNS

```

```

OFN1$ = OFNS + "1.DAT"
OFN2$ = OFNS + "2.DAT"

```

```

OPEN "I", #1, IFNS
OPEN "O", #2, OFNS
OPEN "O", #3, OFN1$
OPEN "O", #4, OFN2$

```

```

' .... data reading

```

```

INPUT #1, NAO1

```

```

DIM WOT(NAO1), ASOT(NAO1), TADOT(NAO1)

```

```

FOR I = 1 TO NAO1
  INPUT #1, WOT(I)
NEXT I

```

```

FOR I = 1 TO NAO1-1
  INPUT #1, ASOT(I)
NEXT I

```

```

INPUT #1, NAB1

```

```

DIM WBT(NAB1), ASBT(NAB1), TADBT(NAB1)

```

```

FOR I = 1 TO NAB1
  INPUT #1, WBT(I)
NEXT I

```

```

FOR I = 1 TO NAB1-1
  INPUT #1, ASBT(I)
NEXT I

```

```

INPUT #1, UDDL, UDLL

```

```

' ..... end of data reading

```



```
CALL TOTDIST(NAOT,ASOT(), TADOT())
CALL TOTDIST(NABT,ASBT(), TADBT())
```

```
PRINT #2," |-----|-----|-----|-----|-----|-----|-----|
-|-----|*
PRINT #2," | L | MOM. OHBD | MOM. OHBD | MAX OF | MOM | MOM | BT/OHBD
| (BT+DL)/(OHBD+DL) |*
PRINT #2," | TRUCK | LANE LOAD | OHBD | BIG TRUCK | DEAD LOAD |
| |*
PRINT #2," | (m) | (t.m) | (t.m) | (t.m) | (t.m) | (t.m) |
| |*
PRINT #2," |-----|-----|-----|-----|-----|-----|-----|
-|-----|*

```

```
FOR L = 1 TO 125
```

```
CALL MOMENT(NAOT,WOT(),TADOT(),L,MOMOT)
CALL MOMENT(NABT,WBT(),TADBT(),L,MAXBT)
```

```
MDL=UDDL*L*L/8
MLL=UDLL*L*L/8
MOMOLL = 0.7*MOMOT + MLL
IF MOMOLL > MOMOT THEN MAXOT = MOMOLL ELSE MAXOT = MOMOT
```

```
RATIO1 = MAXBT/MAXOT
RATIO2 = (MAXBT + MDL)/(MAXOT + MDL)
```

```
PRINT #2,USING " | ### |#####.## |#####.## | #####.## |#####.## |#####.##
| ##.### | ##.###
!";L;MOMOT;MOMOLL;MAXOT;MAXBT;MDL;RATIO1;RATIO2
PRINT #3,USING " ###.## ##.### ";L;RATIO1
PRINT #4,USING" ###.## ##.### ";L;RATIO2
```

```
NEXT L
```

```

* .....
* SUBPROGRAMS
* .....

```

```
****
**** subprogram to calculate distance of each axle from last axle ****
****
```

```
SUB TOTDIST(NA, ASS(1), TAD(1)) STATIC
```

```
TAD(NA) = 0.0
```

```

      FOR I = 1 TO NA-1
        FOR J = I TO NA - 1
          TAD(I) = TAD(I) + ASS(J)
        NEXT J
      NEXT I

```

```

END SUB

```

```

****
*** subprogram to calculate maximum moment due to truck loading ***
****

```

```

SUB MOMENT(NA,W(1),TAD(1),L,MAXMOM) STATIC

```

```

  MAXMOM = -100000.0

```

```

  FOR I = 1 TO NA
    MOM = W(I)*0.25*L

```

```

    IF I <> 1 THEN
      FOR J = 1 TO I-1
        DIF = ABS(TAD(J) - TAD(I))
        IF DIF < (0.5*L) THEN
          MOM = MOM + 0.25*L*(1 - 2*DIF/L)*W(J)
        END IF
      NEXT J
    END IF

```

```

    IF I <> NA THEN
      FOR J = I+1 TO NA
        DIF = ABS(TAD(I) - TAD(J))
        IF DIF < (0.5*L) THEN
          MOM = MOM + 0.25*L*(1 - 2*DIF/L)*W(J)
        END IF
      NEXT J
    END IF
    IF MAXMOM < MOM THEN MAXMOM = MOM:KK=I

```

```

  NEXT I

```

```

END SUB

```

```

. . . . .
. .   THIS PROGRAM CALCULATES THE MAXIMUM MOMENT RATIO OF   .
. .   SPECIFIED HEAVY TRUCKS TO OHBD TRUCK AT THE SUPPORT   .
. .   OF 2-SPAN CONTINUOUS BRIDGES OF DIFFERENT SPAN LENGTHS .
. . . . .

```

```

INPUT * PLEASE ENTER INPUT FILE NAME *:IFNS
INPUT * PLEASE ENTER OUTPUT FILE NAME *:OFNS

```

```

OFN1$ = OFNS + '1.DAT'
OFN2$ = OFNS + '2.DAT'

```

```

OPEN 'I', #1, IFNS
OPEN 'O', #2, OFNS
OPEN 'O', #3, OFN1$
OPEN 'O', #4, OFN2$

```

```

' .... data reading

```

```

INPUT #1, NAO1

```

```

DIM WOTF(NAO1), WOTB(NAO1), ASOTF(NAO1), ASOTB(NAO1), TADOTF(NAO1), TADOTB(NAO1)

```

```

FOR I = 1 TO NAO1
  INPUT #1, WOTF(I)
  WOTB(NAO1+1-I) = WOTF(I)
NEXT I

```

```

FOR I = 1 TO NAO1-1
  INPUT #1, ASOTF(I)
  ASOTB(NAO1-I) = ASOTF(I)
NEXT I

```

```

INPUT #1, NAB1

```

```

DIM WBTf(NAB1), WBTB(NAB1), ASBTf(NAB1), ASBTB(NAB1), TADBTF(NAB1), TADBTB(NAB1)

```

```

FOR I = 1 TO NAB1
  INPUT #1, WBTf(I)
  WBTB(NAB1+1-I) = WBTf(I)
NEXT I

```

```

FOR I = 1 TO NAB1-1
  INPUT #1, ASBTf(I)
  ASBTB(NAB1-I) = ASBTf(I)
NEXT I

```

```

INPUT #1, UDOL, UDLL

```

'..... end of data reading

```
CALL TOTDIST(NAOT,ASOTF(), TADOTF())
CALL TOTDIST(NAOT,ASOTB(), TADOTB())
CALL TOTDIST(NABT,ASBTF(), TADBTF())
CALL TOTDIST(NABT,ASBTB(), TADBTB())
```

```
PRINT #2,* |-----|-----|-----|-----|-----|-----|-----|
          |-----|*
PRINT #2,* | L | MOM. OHBD | MOM. OHBD | MAX OF | MOM | MOM | BT/OHBD
          | (BT+DL)/(OHBD+DL) |*
PRINT #2,* |   | TRUCK | LANE LOAD | OHBD | BIG TRUCK | DEAD
          | LOAD |   |   |   |*
PRINT #2,* | (m) | (t.m) | (t.m) | (t.m) | (t.m) | (t.m) |
          |   |   |   |*
PRINT #2,* |-----|-----|-----|-----|-----|-----|-----|
          |-----|*

```

FOR L = 1 TO 125

```
  FLAGS = "ONT"
  CALL MOMENT(NAOT,WOTF(),TADOTF(),L,FLAGS,MOMOTF)
  CALL MOMENT(NAOT,WOTB(),TADOTB(),L,FLAGS,MOMOTB)
  IF MOMOTF > MOMOTB THEN MOMOT = MOMOTF ELSE MOMOT = MOMOTB

  FLAGS = "BT"
  CALL MOMENT(NABT,WBTF(),TADBTF(),L,FLAGS,MOMBTF)
  CALL MOMENT(NABT,WBTF(),TADBTF(),L,FLAGS,MOMBTF)
  IF MOMBTF > MOMBTF THEN MAXBT = MOMBTF ELSE MAXBT = MOMBTF
```

```
MDL=0.125*UDDL*L*L
MLL=0.125*UDLL*L*L
MOMOLL = 0.7*MOMOT + MLL
IF MOMOLL > MOMOT THEN MAXOT = MOMOLL ELSE MAXOT = MOMOT
```

```
RATIO1 = MAXBT/MAXOT
RATIO2 = (MAXBT + MDL)/(MAXOT + MDL)
```

```
PRINT #2,USING * | ### |#####.## |#####.## |#####.## |#####.## |#####.##
                  | ##.### | ##.###
                  |*;L;MOMOT;MOMOLL;MAXOT;MAXBT;MDL;RATIO1;RATIO2
PRINT #3,USING * ###.## **.* **.* *;L;RATIO1
PRINT #4,USING* ###.## **.* **.* *;L;RATIO2
```

NEXT L

END

```

. ....
. SUBPROGRAMS
. ....

```

```

****
**** subprogram to calculate distance of each axle from last axle ***
****

```

```

SUB TOTDIST(NA, ASS(1), TAD(1)) STATIC

```

```

    TAD(NA) = 0.0

```

```

    FOR I = 1 TO NA - 1

```

```

        FOR J = I TO NA - 1

```

```

            TAD(I) = TAD(I) + ASS(J)

```

```

        NEXT J

```

```

    NEXT I

```

```

END SUB

```

```

****
**** subprogram to calculate maximum moment due to truck loading ***
****

```

```

SUB MOMENT(NA, W(1), TAD(1), L, FLAGS, MAXMOM) STATIC

```

```

    MAXMOM = -100000.0

```

```

    FOR I = 1 TO NA

```

```

        MOM = W(I)*0.096225044*L

```

```

        IF I <> 1 THEN

```

```

            FOR J = 1 TO I-1

```

```

                DIF = ABS(TAD(J) - TAD(I))

```

```

                IF DIF < (0.57735*L) THEN

```

```

                    X = 1.42265*L + DIF

```

```

                    MOM = MOM + (0.5*X - L + 0.25*(2*L - X)*(3 - ((2*L -
                    X)/L)^2))*W(J)

```

```

                END IF

```

```

            NEXT J

```

```

        END IF

```

```

        IF I <> NA THEN

```

```

            FOR J = I+1 TO NA

```

```

                DIF = ABS(TAD(I) - TAD(J))

```

```

                IF DIF < (1.42265*L) THEN

```

```

      X = 1.42265*L - DIF
      IF DIF < (0.42265*L) THEN
        MOM = MOM + (0.5*X - L + 0.25*(2*L - X)*3 - ((2*L -
          X)/L)^2))*W(J)
      ELSE
        MOM = MOM + 0.25*(X - X^3/L^2)*W(J)
      END IF
    END IF
  NEXT J
END IF

IF MAXMOM < MOM THEN MAXMOM = MOM

NEXT I

END SUB

```

```

. . . . .
. * THIS PROGRAM CALCULATES THE MAXIMUM MOMENT RATIO OF *
. * SPECIFIED HEAVY TRUCKS TO OHBD TRUCK AT THE MID-SPAN *
. * OF 2-SPAN CONTINUOUS BRIDGES OF DIFFERENT SPAN LENGTHS *
. . . . .

```

```

INPUT * PLEASE ENTER INPUT FILE NAME *;IFNS
INPUT * PLEASE ENTER OUTPUT FILE NAME *;OFNS

```

```

OFN1$ = OFNS + '1.DAT'
OFN2$ = OFNS + '2.DAT'

```

```

OPEN 'I', #1, IFNS
OPEN 'O', #2, OFNS
OPEN 'O', #3, OFN1$
OPEN 'O', #4, OFN2$

```

```

' .... data reading

```

```

INPUT #1, NAO1

```

```

DIM WOTF(NAO1), WOTB(NAO1), ASOTF(NAO1), ASOTB(NAO1), TADOTF(NAO1), TADOTB(NAO1)

```

```

FOR I = 1 TO NAO1
  INPUT #1, WOTF(I)
  WOTB(NAO1+1-I) = WOTF(I)
NEXT I

```

```

FOR I = 1 TO NAOI-1
  INPUT #1, ASOTF(I)
  ASOTB(NAOI-I) = ASOTF(I)
NEXT I

INPUT #1, NABT

DIM WBTF(NABT), WBTB(NABT), ASBTF(NABT), ASBTB(NABT), TADBTF(NABT), TADBTB(NABT)

FOR I = 1 TO NABT
  INPUT #1, WBTF(I)
  WBTB(NABT+1-I) = WBTF(I)
NEXT I

FOR I = 1 TO NABT-1
  INPUT #1, ASBTF(I)
  ASBTB(NABT-I) = ASBTF(I)
NEXT I

INPUT #1, UDDL, UDLL

'..... end of data reading

CALL TOTDIST(NAOI,ASOTF(), TADOTF())
CALL TOTDIST(NAOI,ASOTB(), TADOTB())
CALL TOTDIST(NABT,ASBTF(), TADBTF())
C/LL TOTDIST(NABT,ASBTB(), TADBTB())

PRINT #2," |-----|-----|-----|-----|-----|-----|-----|
-|-----|*"
PRINT #2," | L | MOM. OHBD | MOM. OHBD | MAX OF | MOM | MOM | BT/OHBD
| (BT+DL)/(OHBD+DL) |*"
PRINT #2," | | TRUCK | LANE LOAD | OHBD | BIG TRUCK | DEAD LOAD |
| | |*"
PRINT #2," | (m) | (t.m) | (t.m) | (t.m) | (t.m) | (t.m) |
| | |*"
PRINT #2," |-----|-----|-----|-----|-----|-----|-----|
-|-----|*"

FOR L = 1 TO 125

  FLAGS = "ONT"
  CALL MOMENT(NAOI,WOTF(),TADOTF(),L,FLAGS,MOMOTF)
  CALL MOMENT(NAOI,WOTB(),TADOTB(),L,FLAGS,MOMOTB)
  IF MOMOTF > MOMOTB THEN MOMOT = MOMOTF ELSE MOMOT = MOMOTB

  FLAGS = "BT"

```

```

CALL MOMENT(NABT,WBTF(),TADBTf(),L,FLAGS,MOMBTF)
CALL MOMENT(NABT,WBTB(),TADBTB(),L,FLAGS,MOMBTB)
IF MOMBTF > MOMBTB THEN MAXBT = MOMBTF ELSE MAXBT = MOMBTB

MDL=0.0703125*UDDL*L*L
MLL=0.095703125*UDLL*L*L
MOMOLL = 0.7*MOMOT + MLL
IF MOMOLL > MOMOT THEN MAXOT = MOMOLL ELSE MAXOT = MOMOT

RATIO1 = MAXBT/MAXOT
RATIO2 = (MAXBT + MDL)/(MAXOT + MDL)

PRINT #2,USING " | ### |#####.## |#####.## | #####.## |#####.## |#####.##
               |  ##.### |   ##.###
               |*:L;MOMOT;MOMOLL;MAXOT;MAXBT;MDL;RATIO1;RATIO2
PRINT #3,USING " ###.##   ##.### *:L;RATIO1
PRINT #4,USING" ###.##   ##.### *:L;RATIO2

NEXT L

END

. ....
. *      SUBPROGRAMS      *
. ....

....
.... subprogram to calculate distance of each axle from last axle ***
....

SUB TOTDIST(NA, ASS(1), TAD(1)) STATIC

TAD(NA) = 0.0

FOR I = 1 TO NA - 1
  FOR J = I TO NA - 1
    TAD(I) = TAD(I) + ASS(J)
  NEXT J
NEXT I

END SUB

....
.... subprogram to calculate maximum moment due to truck loading ***
....

SUB MOMENT(NA,W(1),TAD(1),L,FLAGS,MAXMOM) STATIC

MAXMOM = -100000.0

```



```

FOR I = 1 TO NA
  MOM = W(I)*0.203125*L

  IF I <> 1 THEN
    FOR J = 1 TO I-1
      DIF = ABS(TAD(J) - TAD(I))
      IF DIF < (0.5*L) THEN
        X = 1.5*L + DIF
        MOM = MOM + (1.5*L - 0.75*X - 0.125*(2*L - X)*(3 - ((2*L -
          X)/L)^2))*W(J)
      END IF
    NEXT J
  END IF

  IF I <> NA THEN
    FOR J = I+1 TO NA
      DIF = ABS(TAD(I) - TAD(J))
      IF DIF < (0.5*L) THEN
        X = 1.5*L - DIF
        MOM = MOM + (0.25*X - 0.125*(2*L - X)*(3 - ((2*L -
          X)/L)^2))*W(J)
      END IF

      IF FLAGS = "BT" THEN
        IF (DIF > (0.5*L) AND DIF < (1.5*L)) THEN
          X = 1.5*L - DIF
          MOM = MOM + 0.125*(X^3/L^2 - X)*W(J)
        END IF
      END IF
    NEXT J
  END IF

  IF MAXMOM < MOM THEN MAXMOM = MOM
NEXT I

END SUB

```

.....

```

SUB MOMENTOT(NA,W(1),TAD(1),L,MAXMOM) STATIC

```

```

  MAXMOM = -100000.0

```

```

  FOR I = 1 TO NA
    MOM = W(I)*0.203125*(1.5*L)

    IF I <> 1 THEN
      FOR J = 1 TO I-1
        DIF = ABS(TAD(J) - TAD(I))
        IF DIF < (0.5*L) THEN
          X = 1.5*L + DIF
          MOM = MOM + (1.5*L - 0.75*X - 0.125*(2*L - X)*(3 - ((2*L -

```

```

                                X)/L)^2))*W(J)
                                END IF
                                NEXT J
                                END IF

                                IF I <> NA THEN
                                FOR J = I+1 TO NA
                                DIF = ABS(TAD(I) - TAD(J))
                                IF DIF <= (0.5*L) THEN
                                X = 1.5*L - DIF
                                MOM = MOM +(0.25*X - 0.125*(2*L -X)*(3 - ((2*L-X)/L)^2))*W(J)
                                END IF
                                NEXT J
                                END IF

                                IF MAXMOM < MOM THEN MAXMOM = MOM
                                NEXT I

                                END SUB

```

```

. . . . .
.   THIS PROGRAM CALCULATES FATIGUE LIFE AND PERCENT   .
.   REDUCTION IN FATIGUE LIFE OF BRIDGE COMPONENTS AS A FUNCTION .
.   OF VARIOUS PARAMETERS FOR A SPECIFIED STRESS RANGE SPECTRUM .
. . . . .

```

```

CLEAR
CLS
PRINT "PARAMETERS TO BE INVESTIGATED"
PRINT
PRINT "1) PERCENT LIFE REDUCTION-STRESS FREQUENCY GRAPH FOR VARIOUS STRESS LEVELS"
PRINT "2) PERCENT LIFE REDUCTION-STRESS LEVEL GRAPH FOR VARIOUS DETAIL TYPES"
PRINT "   OR VARIOUS SLOPES BELOW CAFL"
PRINT "3) PERCENT LIFE REDUCTION-SLOPE OF THE S-N CURVE BELOW CAFL GRAPH"
PRINT "   FOR VARIOUS STRESS LEVELS"
PRINT "4) BRIDGE LIFE (IN YEARS)-ADTT GRAPH FOR VARIOUS DETAIL TYPES"
PRINT "5) PERCENT REDUCTION IN BRIDGE LIFE-ADTT GRAPH FOR VARIOUS STRESS LEVELS"
PRINT "6) PERCENT LIFE REDUCTION-STRESS LEVEL GRAPH FOR VARIOUS SPECTRA TYPES"
PRINT "7) ALLOWABLE NUMBER OF TRUCKS PER MONTH - SPAN LENGTH GRAPH"
PRINT "8) SERVICE LIFE - STRESS GRAPH FOR DIFFERENT DETAIL TYPES (OR SLOPES)"
PRINT "9) SERVICE LIFE - Smax/SL GRAPH FOR VARIABLE SLOPES"
PRINT
INPUT "PLEASE ENTER NUMBER "; NUMBER
CLS
INPUT "PLEASE ENTER THE DATA FILENAME "; DFNS
INPUT "PLEASE ENTER THE SPECTRA FILENAME "; SFNS
INPUT "PLEASE ENTER THE OUTPUT FILENAME "; OFNS

```

```

OPEN 'I', #1, DFNS
OPEN 'I', #2, SFNS
OPEN 'O', #3, OFNS

```

```

DIM f(50), SIGMA(50), N(50)

```

```

***** PERCENT LIFE REDUCTION-STRESS FREQUENCY GRAPH *****
      FOR VARIOUS STRESS LEVELS

```

```

IF NUMBER = 1 THEN

```

```

    LINE INPUT #1, AS
    INPUT #1, npms, npmE, npmINC
    LINE INPUT #1, AS
    INPUT #1, NL, SL, Y

```

```

    LINE INPUT #1, AS
    INPUT #1, NTRUCK

```

```

    DIM Shpt(NTRUCK), Nhpt(NTRUCK)

```

```

    LINE INPUT #1, AS
    FOR I = 1 TO NTRUCK
        INPUT #1, Shpt(I)
    NEXT I

```

```

    CALL READFRQ(NFRQ, f(), SIGMA())

```

```

***** calculations *****

```

```

    CALL NORMALIZE(NFRQ, f())

```

```

    FOR I = 1 TO NTRUCK
        CALL SN(NL, SL, Y, Shpt(I), Nhpt(I))
    NEXT I

```

```

    FOR I = 1 TO NFRQ
        SIGMA(I) = SIGMA(I) * SL
        CALL SN(NL, SL, Y, SIGMA(I), N(I))
    NEXT I

```

```

    CALL NCYCLE(f(), N(), NFRQ, SUMFON, NC)
    METHOD = 2
    ADTT = 1000

```

```

FOR npm = npms TO npme STEP npminc

    PRINT #3, USING "*****"; npm;

    FOR I = 1 TO NTRUCK
        CALL NREDUCED(NC, SUMFON, Nhpt(I), fhpt, npm, ADTT, METHOD, NR, PR)
        PRINT #3, USING ", ##.***"; PR;
    NEXT I
    PRINT #3,
NEXT npm

GOTO CQUIT

END IF

```

```

***** PERCENT LIFE REDUCTION-STRESS LEVEL GRAPH FOR VARIOUS DETAIL TYPES ****
OR VARIOUS SLOPES BELOW CAFL

```

```

IF NUMBER = 2 THEN

```

```

    ***** DATA INPUT

```

```

        LINE INPUT #1, AS
        INPUT #1, SMS, SME, SMINC

```

```

        LINE INPUT #1, AS
        INPUT #1, NDETAIL

```

```

        DIM NL(NDETAIL), SL(NDETAIL), Y(NDETAIL), SUMFON(NDETAIL), NC(NDETAIL)

```

```

        LINE INPUT #1, AS
        FOR I = 1 TO NDETAIL
            INPUT #1, NL(I), SL(I), Y(I)
        NEXT I

```

```

        CALL READFRQ(NFRQ, f(), SIGMA())

```

```

    ***** calculations *****

```

```

        CALL NORMALIZE(NFRQ, f())

```

```

        FOR I = 1 TO NDETAIL

```

```

            FOR J = 1 TO NFRQ
                SIGMOD = SIGMA(J) * SL(I)

```

```

        CALL SN(NL(I), SL(I), Y(I), SIGMOD, N(J))
    NEXT J

    CALL NCYCLE(f(), N(), NFRQ, SUMFON(I), NC(I))

NEXT I

METHOD = 2
ADTT = 1000
npm = 100

FOR SM = SMS TO SME STEP SMINC

    PRINT #3, USING "###.###"; SM;

    FOR I = 1 TO NDETAIL

        STRESS = SM * SL(I)
        CALL SN(NL(I), SL(I), Y(I), STRESS, NSTRESS)
        CALL NREDUCED(NC(I), SUMFON(I), NSTRESS, fhpt, npm, ADTT, METHOD, NR, PR)
        PRINT #3, USING ", ##.###"; PR;

    NEXT I

    PRINT #3,

NEXT SM

GOTO CQUIT

END IF

'**** PERCENT LIFE REDUCTION-SLOPE OF THE S-N CURVE BELOW CAFL GRAPH ****
'
'          FOR VARIOUS STRESS LEVELS
IF NUMBER = 3 THEN

    LINE INPUT #1, AS
    INPUT #1, SLOPES, SLOPEE, SLOPEINC

    LINE INPUT #1, AS
    INPUT #1, NL, SL

    LINE INPUT #1, AS
    INPUT #1, NSTR

    DIM STR(NSTR)

```

```

LINE INPUT #1, AS
  FOR I = 1 TO NSTR
    INPUT #1, STR(I)
    STR(I) = SL * STR(I)
  NEXT I

CALL READFRQ(NFRQ, f(), SIGMA())

***** calculations *****

CALL NORMALIZE(NFRQ, f())

FOR I = 1 TO NFRQ
  SIGMA(I) = SIGMA(I) * SL
NEXT I

METHOD = 2
ADTT = 1000
npm = 100

FOR SLOPE = SLOPES TO SLOPEE STEP SLOPEINC
  PRINT #3, USING "###.###"; SLOPE;

  FOR J = 1 TO NFRQ
    CALL SN(NL, SL, SLOPE, SIGMA(J), N(J))
  NEXT J

  CALL NCYCLE(f(), N(), NFRQ, SUMFON, NC)

  FOR I = 1 TO NSTR
    CALL SN(NL, SL, SLOPE, STR(I), NSTRESS)
    CALL NREDUCED(NC, SUMFON, NSTRESS, fhpt, npm, ADTT, METHOD, NR, PR)
    PRINT #3, USING ", ###.###"; PR;
  NEXT I

  PRINT #3,

NEXT SLOPE

GOTO CQUIT

END IF

***** SERVICE LIFE - ADTT GRAPH FOR DIFFERENT DETAIL TYPES

```

```

IF NUMBER = 4 THEN

    LINE INPUT #1, AS
    INPUT #1, ADTTS, ADTTE, ADTTINC

    LINE INPUT #1, AS
    INPUT #1, COEF

    LINE INPUT #1, AS
    INPUT #1, NDETAIL

    DIM NL(NDETAIL), SL(NDETAIL), Y(NDETAIL), NC(NDETAIL)

    LINE INPUT #1, AS
    FOR I = 1 TO NDETAIL
        INPUT #1, NL(I), SL(I), Y(I)
    NEXT I

    CALL READFRQ(NFRQ, f(), SIGMA())

    ***** calculations *****

    CALL NORMALIZE(NFRQ, f())

    FOR I = 1 TO NDETAIL

        FOR J = 1 TO NFRQ
            SIGMOD = SIGMA(J) * SL(I)
            SLML = COEF*SL(I)
            CALL SN(NL(I), SLML, Y(I), SIGMOD, N(J))
        NEXT J

        CALL NCYCLE(f(), N(), NFRQ, SUMFON, NC(I))

    NEXT I

    FOR ADTT=ADTTS TO ADTTE STEP ADTTINC

        PRINT #3, USING "*****.***";ADTT;

        FOR I=1 TO NDETAIL
            NYEAR=NC(I)/(365.25*ADTT)
            PRINT #3, USING ", ****.***";NYEAR;
        NEXT I

        PRINT #3,

```

```

NEXT ADTT

GOTO CQUIT

END IF

'***** REDUCTION IN SERVICE LIFE - ADTT GRAPH FOR DIFFERENT STRESS LEVELS

IF NUMBER = 5 THEN

    LINE INPUT #1, AS
    INPUT #1, ADTTS, ADTTE, ADTTINC

    LINE INPUT #1, AS
    INPUT #1, COEF

    LINE INPUT #1, AS
    INPUT #1, NL, SL, Y

    LINE INPUT #1, AS
    INPUT #1, NSTR

    DIM STR(NSTR)

    LINE INPUT #1, AS
    FOR I = 1 TO NSTR
        INPUT #1, STR(I)
        STR(I) = SL * STR(I)
    NEXT I

    CALL READFRQ(NFRQ, f(), SIGMA())

    ' ***** calculations *****

    CALL NORMALIZE(NFRQ, f())

    FOR J = 1 TO NFRQ
        SIGMOD = SIGMA(J) * SL
        SLML = COEF*SL
        CALL SN(NL, SLML, Y, SIGMOD, N(J))
    NEXT J

    CALL NCYCLE(f(), N(), NFRQ, SUMFON, NC)

    METHOD = 2
    npm = 100

```



```

FOR ADTT=ADTT5 TO ADTTE STEP ADTTINC

  PRINT #3, USING '#####.###';ADTT;

  FOR I = 1 TO NSTR

    CALL SN(NL, SL, Y, STR(I), NSTRESS)
    CALL NREDUCED(NC, SUMFON, NSTRESS, fhpt, npm, ADTT, METHOD, NR, PR)
    PRINT #3, USING ', ###.##'; PR;

  NEXT I

  PRINT #3,

NEXT ADTT

GOTO CQUIT

END IF

' ***** PERCENT LIFE REDUCTION-STRESS LEVEL GRAPH FOR VARIOUS SPECTRA *****

IF NUMBER = 6 THEN

' ***** DATA INPUT

  LINE INPUT #1, AS
  INPUT #1, SMS, SME, SMINC

  LINE INPUT #1, AS
  INPUT #1, NL, SL, Y

  NFRQ=28
  NSPECTRA=11

  DIM SUMFON(NSPECTRA), NC(NSPECTRA)

  FOR I= 1 TO NSPECTRA

    LINE INPUT #2, AS
    LINE INPUT #2, AS
    LINE INPUT #2, AS

    FOR J = 1 TO NFRQ
      INPUT #2, SIGMA(J), f(J)
    NEXT J

```

```

      CALL NORMALIZE(NFRQ, f())

      FOR j = 1 TO NFRQ
        SIGMOD = SIGMA(J) * SL
        CALL SN(NL, SL, Y, SIGMOD, N(J))
      NEXT j

      CALL NCYCLE(f(), N(), NFRQ, SUMFON(I), NC(I))

    NEXT I

    METHOD = 2
    ADTT = 1000
    npm = 100

    FOR SM = SMS TO SME STEP SMINC

      PRINT #3, USING "###.###"; SM;

      STRESS = SM * SL
      CALL SN(NL, SL, Y, STRESS, NSTRESS)
      SUM=0

      FOR I = 1 TO NSPECTRA

        CALL NREDUCED(NC(I), SUMFON(I), NSTRESS, fhpt, npm, ADTT, METHOD, NR, PR)
        PRINT #3, USING ", ###.###"; PR;
        SUM = SUM + PR
      NEXT I

      AVERAGE=SUM/NSPECTRA
      PRINT #3, USING ", ###.###";AVERAGE

    NEXT SM

    GOTO CQUIT

  END IF

'**** SERVICE LIFE - STRESS GRAPH FOR DIFFERENT DETAIL TYPES (OR SLOPES)

  IF NUMBER = 9 THEN

    LINE INPUT #1, AC
    INPUT #1, RATIOS, RATIOE, RATIOINC

```

```

LINE INPUT #1, AS
INPUT #1, NDETAIL

DIM NL(NDETAIL), SL(NDETAIL), Y(NDETAIL), NC(NDETAIL)

LINE INPUT #1, AS
FOR I = 1 TO NDETAIL
  INPUT #1, NL(I), SL(I), Y(I)
NEXT I
LINE INPUT #1, AS
INPUT #1, ADTT

CALL READFRQ(NFRQ, f(), SIGMA())

. ***** calculations *****

CALL NORMALIZE(NFRQ, f())

FOR RATIO = RATIOS TO RATIOE STEP RATIOINC
  PRINT #3, USING "##.###";RATIO;

  FOR I = 1 TO NDETAIL
    FOR J = 1 TO NFRQ
      SIGMOD = SIGMA(J) * SL(I) * RATIO
      CALL SN(NL(I), SL(I), Y(I), SIGMOD, N(J))
    NEXT J

    CALL NCYCLE(f(), N(), NFRQ, SUMFON, NC)

    NYEAR=1.92*NC/(365.25*ADTT)
    PRINT #3, USING " , #####";NYEAR;

  NEXT I
  PRINT #3,

  NEXT RATIO
END IF

***** SERVICE LIFE - Smax/SL GRAPH FOR VARIABLE SLOPES

IF NUMBER = 9 THEN

  LINE INPUT #1, AC

```

```
INPUT #1, RATIOS, RATIOE, RATIOINC
```

```
LINE INPUT #1, AS  
INPUT #1, NL, SL, Y
```

```
LINE INPUT #1, AS  
INPUT #1, ADTT
```

```
CALL READFRQ(NFRQ, f(), SIGMA())
```

```
***** calculations *****
```

```
CALL NORMALIZE(NFRQ, f())
```

```
FOR RATIO = RATIOS TO RATIOE STEP RATIOINC
```

```
PRINT #3, USING "###.###";RATIO;
```

```
FOR J = 1 TO NFRQ
```

```
Y = 1.14/(RATIO-1)  
IF RATIO > 1.38 THEN Y=3  
IF RATIO < 1.0 THEN Y=10000  
RRR = SIGMA(J)*SL  
SIGMOD = RRR
```

```
IF SIGMA(J) > 1.0 THEN
```

```
SIGMOD = RRR*RATIO/1.71875
```

```
IF SIGMOD < RRR THEN
```

```
SIGMOD=RRR
```

```
END IF
```

```
END IF
```

```
CALL SN(NL, SL, Y, SIGMOD, N(J))
```

```
NEXT J
```

```
CALL NCYCLE(f(), N(), NFRQ, SUMFUN, NC)
```

```
NYEAR=NC/(365.25*ADTT)
```

```
PRINT #3, USING ", #####";NYEAR;
```

```

        PRINT #3,

NEXT RATIO

GOTO CQUIT

END IF

CQUIT:

END

.....
..... SUB PROGRAMS .....
.....

SUB READFRQ(NFRQ, f(), SIGMA()) STATIC

***** READS THE SPECTRA FILE *****

CLS
LOCATE 10,10: PRINT "READ FREQUENCIES ****"
LOCATE 10,10: PRINT "READ FREQUENCIES"

LINE INPUT #2, AS
INPUT #2, NFRQ

LINE INPUT #2, AS
FOR I = 1 TO NFRQ
    INPUT #2, SIGMA(I), f(I)
NEXT I

END SUB

SUB NORMALIZE (NFRQ, f()) STATIC

***** NORMALIZES THE FREQUENCIES *****

LOCATE 11,10: PRINT "NORMALIZE THE FREQUENCIES ****"
LOCATE 11,10: PRINT "NORMALIZE THE FREQUENCIES"

SUMFRQ = 0

```

```

FOR I = 1 TO NFRQ
    SUMFRQ = SUMFRQ + f(I)
NEXT I

FOR I = 1 TO NFRQ
    f(I) = f(I) / SUMFRQ
NEXT I

END SUB

SUB SN (NL, SL, Y, STRESS, N) STATIC

***** EQUATION OF THE S-N CURVE *****

LOCATE 12,10 : PRINT "USE S-N CURVE TO OBTAIN THE NUMBER OF CYCLES 'N' ****
LOCATE 12,10 : PRINT "USE S-N CURVE TO OBTAIN THE NUMBER OF CYCLES 'N' *
IF STRESS > SL THEN
    N = NL * (SL / STRESS) ^ 3
ELSE
    IF 'STRESS' >= (SL * (NL / 1.0E+25)^(1/Y)) THEN
        N = NL * (SL / STRESS) ^ Y
    ELSE
        N=1.0E+25
    END IF
END IF

END SUB

SUB NCYCLE (f(), N(), NFRQ, SUMFON, NC) STATIC

***** CALCULATES THE FATIGUE LIFE IN CYCLES *****

LOCATE 13,10: PRINT "CALCULATE THE FATIGUE LIFE ****
LOCATE 13,10: PRINT "CALCULATE THE FATIGUE LIFE"

SUMFON = 0

FOR I = 1 TO NFRQ
    SUMFON = SUMFON + f(I) / N(I)
NEXT I

NC = INT(1 / SUMFON)
END SUB

```

SUB NREDUCED (NC, SUMFON, Nhpt, fhpt, npm, ADTT, METHOD, NR, PR) STATIC

\*\*\*\*\* CALCULATES THE PERCENT REDUCTION IN SERVICE LIFE \*\*\*\*\*

LOCATE 14,10: PRINT \*CALCULATE THE PERCENT REDUCTION IN SERVICE LIFE \*\*\*\*

LOCATE 14,10: PRINT \*CALCULATE THE PERCENT REDUCTION IN SERVICE LIFE \*

IF METHOD = 1 THEN

NR = 1 / ((SUMFON + fhpt / Nhpt) / (1 + fhpt))

ELSE

NR = Nhpt / (Nhpt \* SUMFON \* (1 - npm / (ADTT \* 30)) + npm / (ADTT \* 30))

Nrm = NR / (ADTT \* 30.4375)

fhpt = npm \* Nrm / (NR - npm \* Nrm)

END IF

PR = 100 \* (NC - NR \* (1 - fhpt / (1 + fhpt))) / NC

END SUB

## **APPENDIX B**

### **CALTRANS**



# MULTI-ATTRIBUTE DECISION PROCEDURE

## HAZARD CRITERIA

### Hazard Attributes

### Hazard Weights

|                        |     |
|------------------------|-----|
| SOIL CONDITIONS        | 33% |
| PEAK ROCK ACCELERATION | 38% |
| SEISMIC DURATION       | 29% |

## IMPACT CRITERIA

### Impact Attributes

### Impact Weights

|                                        |     |
|----------------------------------------|-----|
| ADT ON STRUCTURE                       | 28% |
| ADT UNDER/OVER STRUCTURE               | 12% |
| DETOUR LENGTH                          | 14% |
| LEASED AIR SPACE (RESIDENTIAL, OFFICE) | 15% |
| LEASED AIR SPACE (PARKING, STORAGE)    | 07% |
| RTE TYPE ON BRIDGE                     | 07% |
| CRITICAL UTILITY                       | 10% |
| FACILITY CROSSED                       | 07% |

## VULNERABILITY CRITERIA

### Vulnerability Attributes

### Vulnerability Weights

|                            |       |
|----------------------------|-------|
| YEAR DESIGNED (CONST.)     | 25%   |
| HINGES (DROP TYPE FAILURE) | 16.5% |
| OUTRIGGERS, SHARED COL     | 22%   |
| BENT REDUNDANCY            | 16.5% |
| SKEW                       | 12%   |
| ABUTMENT TYPE              | 08%   |

# GLOBAL UTILITY FUNCTIONS

## UTILITY CRITERIA

(0.25->low;0.5->moderate;0.75->active;1->high)

### HAZARD CRITERIA (see maps)

SOIL CONDITIONS

0.33\*(1->if in high risk zone; else 0)

PEAK ROCK ACCELERATION

0.38\*(linear, normalized to 0.7g)

SEISMIC DURATION

0.29\*(0.5->short;0.75->intermediate;1->long)

### IMPACT CRITERIA

ADT ON STRUCTURE

0.28\*(parab., based on max. ADT of 200000)

ADT UNDER/OVER STRUCTURE

0.12\*(see ADT above)

DETOUR LENGTH

0.14\*(linear,normalized to 100 miles)

LEASED AIR SPACE  
(RESIDENTIAL, OFFICE)

0.15\*(1->if present; else 0)

LEASED AIR SPACE  
(PARKING, STORAGE)

0.07\*(1->if present;else0)

RTE TYPE ON BRIDGE

0.07\*(1->interstate;0.8->US or ST rte or stream;  
0.7->RR;0.5->fed funded Co rte or city str;  
0.2->nonfed funded Co rte or city str;  
0->fed land, ST land, other)

CRITICAL UTILITY

0.10\*(1->if present;else 0)

FACILITY CROSSED

0.07\*(see RTE. TYPE ON BRIDGE)

### VULNERABILITY CRITERIA

YEAR DESIGNED (CONST.)

0.25\*(0.5->yr<1946; 1->1946<=yr<=1971;  
0.25->1972<=yr<=1979; 0->yr>1979)

HINGES (DROP TYPE FAILURE)

0.165\*(0->no hinge;0.5->1 hinge;  
1->2 or more hinges)

OUTRIGGERS, SHARED COL., etc.

0.22\*(1 if present;else 0)

BENT REDUNDANCY

0.165\*(0->no col.;0.25->pier walls;  
0.5 multi-col. bents;1->single col. bents)

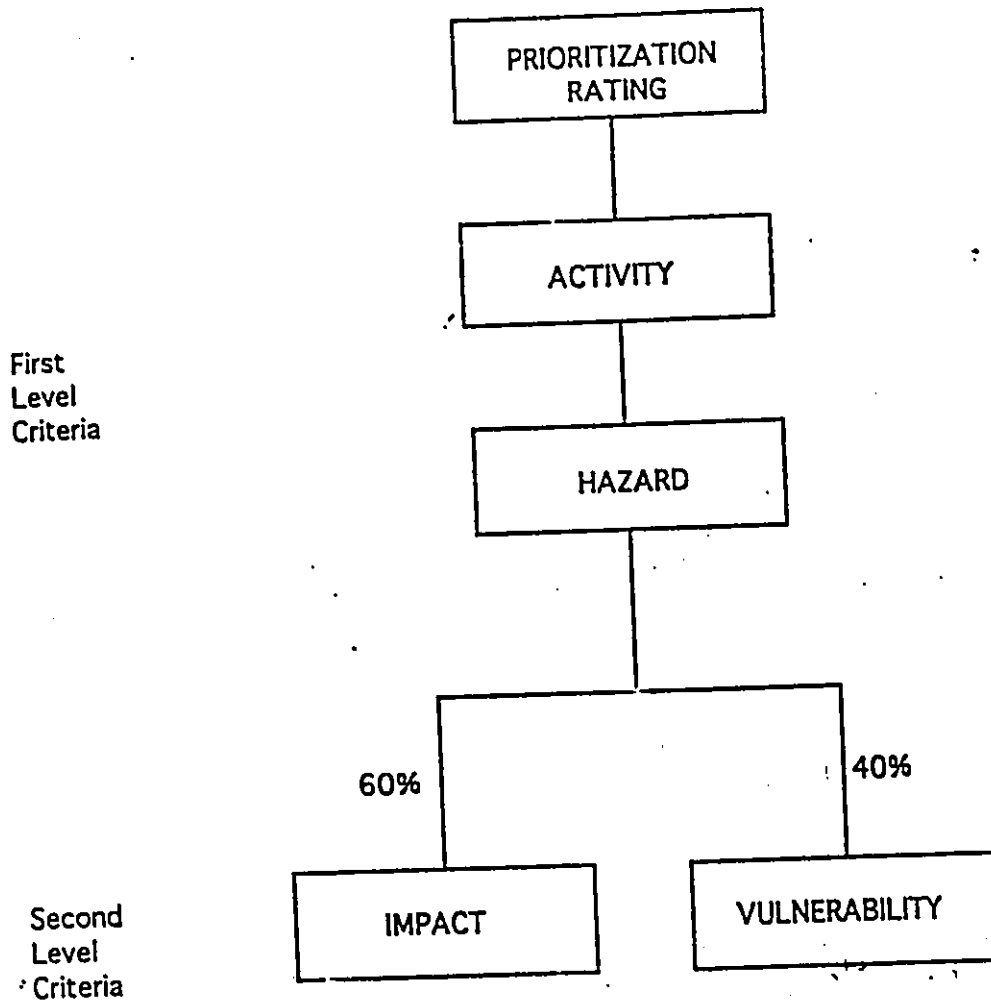
SKEW

0.12\*(linear,normalized to 90)

ABUTMENT TYPE

0.08\*(0->monolithic;1->nonmonolithic)

# MULTI-ATTRIBUTE DECISION PROCEDURE



$$\text{Prioritization Rating} = (\text{Activity})(\text{Hazard}) [ (0.60)(\text{Impact}) + (0.40)(\text{Vulnerability}) ]$$

Where,

$$\text{Activity} = (\text{Global Utility Function Value})$$

$$\text{Hazard} = \sum (\text{Attribute Weight})(\text{Global Utility Function Value})$$

$$\text{Impact} = \sum (\text{Attribute Weight})(\text{Global Utility Function Value})$$

$$\text{Vulnerability} = \sum (\text{Attribute Weight})(\text{Global Utility Function Value})$$