where:

OHBDC = factored braking force as specified in the 3rd edition of the *Ontario Highway Bridge Design Code*

LFD = factored braking force as specified in the AASHTO Standard Specifications (Load Factor)

LRFD = factored braking force as originally specified in the early versions of the LRFD Specifications (up to the 2001 Interim edition)

LRFD′ = factored braking force as specified in Article 3.6.4

CHBDC = factored braking force as specified in the *Canadian Highway Bridge Design Code*

The sloping portion of the curves represents the braking force that includes a portion of the lane load. This represents the possibility of having multiple lanes of vehicles contributing to the same braking event on a long bridge. Although the probability of such an event is likely to be small, the inclusion of a portion of the lane load gives such an event consideration for bridges with heavy truck traffic and is consistent with other design codes.

Because the LRFD braking force is significantly higher than that required in the Standard Specifications, this issue becomes important in rehabilitation projects designed under previous versions of the design code. In cases where substructures are found to be inadequate to resist the increased longitudinal forces, consideration should be given to design and detailing strategies which distribute the braking force to additional substructure units during a braking event.

3.6.5—Vehicular Collision Force: *CT*

3.6.5.1—Protection of Structures

Unless the Owner determines that site conditions indicate otherwise, abutments and piers located within a distance of 30.0 ft to the edge of roadway shall be investigated for collision. Collision shall be addressed by either providing structural resistance or by redirecting or absorbing the collision load. The provisions of Article 2.3.2.2.1 shall apply as appropriate.

Where the design choice is to provide structural resistance, the pier or abutment shall be designed for an equivalent static force of 600 kip, which is assumed to act in a direction of zero to 15 degrees with the edge of the pavement in a horizontal plane, at a distance of 5.0 ft above ground.

C3.6.5.1

Where an Owner chooses to make an assessment of site conditions for the purpose of implementing this provision, input from highway or safety engineers and structural engineers should be part of that assessment.

The equivalent static force of 600 kip is based on the information from full-scale crash tests of rigid columns impacted by 80.0-kip tractor trailers at 50 mph. For individual column shafts, the 600-kip load should be considered a point load. Field observations indicate shear failures are the primary mode of failure for individual columns and columns that are 30.0 in. in diameter and smaller are the most vulnerable. For wall piers, the load may be considered to be a point load or may be distributed over and area deemed suitable for the size of the structure and the anticipated impacting vehicle, but not greater than 5.0 ft wide by 2.0 ft high. These dimensions were determined by considering the size of a truck frame.

Requirements for train collision load found in previous editions have been removed. Designers are encouraged to consult the AREMA *Manual for Railway*

Where the design choice is to redirect or absorb the collision load, protection shall consist of one of the following:

- An embankment;
- A structurally independent, crashworthy ground-mounted 54.0-in. high barrier, located within 10.0 ft from the component being protected; or
- A 42.0-in. high barrier located at more than 10.0 ft from the component being protected.

Such barrier shall be structurally and geometrically capable of surviving the crash test for Test Level 5, as specified in Section 13.

Engineering or local railroad company guidelines for train collision requirements.

For the purpose of this Article, a barrier may be considered structurally independent if it does not transmit loads to the bridge.

Full-scale crash tests have shown that some vehicles have a greater tendency to lean over or partially cross over a 42.0-in. high barrier than a 54.0-in. high barrier. This behavior would allow a significant collision of the vehicle with the component being protected if the component is located within a few ft of the barrier. If the component is more than about 10.0 ft behind the barrier, the difference between the two barrier heights is no longer important.

One way to determine whether site conditions qualify for exemption from protection is to evaluate the annual frequency of impact from heavy vehicles. With the approval of the Owner, the annual frequency for a bridge pier to be hit by a heavy vehicle, AF_{HPB}, can be calculated by:

$$AF_{HBP} = 2(\text{ADTT})\,(P_{HBP})365 \qquad \text{(C3.6.5.1-1)}$$

where:

ADTT = the number of trucks per day in one direction

P_{HBP} = the annual probability for a bridge pier to be hit by a heavy vehicle

Table C3.6.1.4.2-1 may be used to determine ADTT from available ADT data.

$P_{HBP} = 3.457 \times 10^{-9}$ for undivided roadways in tangent and horizontally curved sections

1.090×10^{-9} for divided roadways in tangent sections

2.184×10^{-9} for divided roadways in horizontally curved sections

Design for vehicular collision force is not required if AF_{HBP} is less than 0.0001 for critical or essential bridges or 0.001 for typical bridges.

The determination of the annual frequency for a bridge pier to be hit by a heavy vehicle, AF_{HPB}, is derived from limited statistical studies performed by the Texas Transportation Institute. Due to limited data, no distinction has been made between tangent sections and horizontally curved sections for undivided roadways. The target values for AF_{HBP} mirror those for vessel collision force found in Article 3.14.5.

Table C3.6.5.1-1 provides typical resulting values for AF_{HBP}.