Reliability of Structures – Part 4

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Load Models

- Dead Load
- Live load (buildings and bridges, static and dynamic)
- Environmental loads (wind, snow, earthquake)
- Special loads (collision, fire, scour)
- Load combinations

STRUCTURAL LOAD MODELS

To design a structure, the designer must have an understanding of the types and magnitudes of the loads which are expected to act on the structure during its lifetime

Types of load:

- Loading Type I
- Loading Type II
- Loading Type III

TYPES of STRUCTURAL LOADS

- Loading Type I - data are obtained by load intensity measurements without regard to the frequency of occurrence. The time dependence of the loads is not explicitly considered. Examples of loads in this category are dead load and sustained live load.

- Loading Type II - load data are obtained from measurements at prescribed periodic time intervals. Thus, some time dependence is captured. Examples of loads in this category include severe winds, snow loads, and transient live load.

- Loading Type III - The available data for Type III loads are obtained from infrequent measurements because the data are typically not obtainable at prescribed time intervals. These loads occur during extreme events such as earthquakes and tornadoes.

GENERAL LOAD MODEL

Load effect i denoted by Q_i can be expressed as:

$$Q_i = A_i\, B_i\, C_i$$

where:

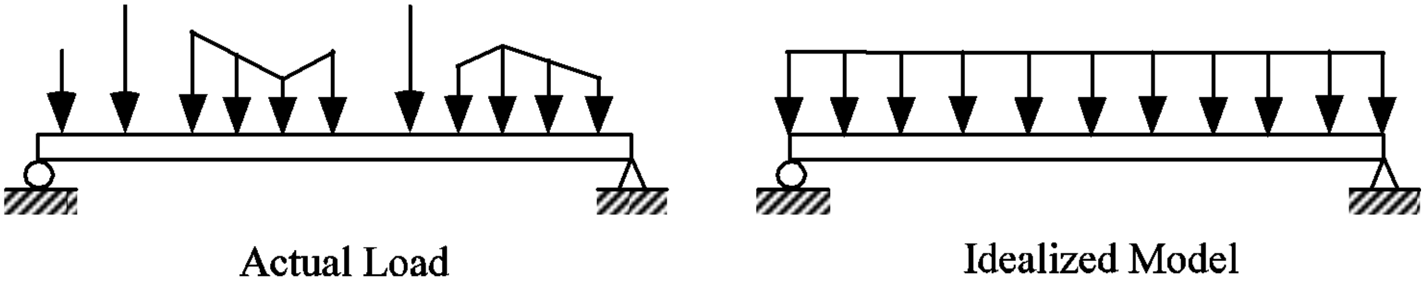
A_i – variable representing the load itself

B_i – variable representing the mode (in which the load effect is assumed to act)

C_i – variable representing variation due to method of analysis, for example a two-dimensional idealization of a three-dimensional structure, fixing of supports, rigidity of connections, continuity, etc.

GENERAL LOAD MODELS

Idealization of loads on a structure



The parameters of load Q:

$$\overline{Q}_i = \overline{A}_i \overline{B}_i \overline{C}_i \qquad V_{Q_i} = \sqrt{V_{A_i}^2 + V_{B_i}^2 + V_{C_i}^2}$$

Usually mean B_i = 1.0. Other parameters may vary.

GENERAL LOAD MODELS

When several loads are acting together, then the total load can be considered as:

$$Q = C\,(A_1\, B_1\, C_1 + A_2\, B_2\, C_2 + \ldots\ldots)$$

C = common factor for all loads (load combination factor)

ASCE STANDARD

ASCE/SEI
7–05

Includes Supplement No. 1 and Errata

Minimum Design Loads for Buildings and Other Structures

This document uses both the International System of Units (SI) and customary units

ASCE

SEI
Structural Engineering Institute
of the American Society of Civil Engineers

DEAD LOAD

The dead load considered in design is usually the gravity load due to the self-weight of the structural and non-structural elements permanently connected to the structure.

In bridge design, components of the total dead load include:

1. weight of factory-made elements (steel, precast concrete members)
2. weight of cast-in-place concrete members.
3. for bridges, a third component of dead load is the weight of the wearing surface (asphalt).

DEAD LOAD

All components of dead load are typically treated as normal random variables. Usually it is assumed that the total dead load, D, remains constant throughout the life of the structure.

Table below shows some representative statistical parameters of dead load:

	$\lambda = \overline{D}/D_n$	$\sqrt{V_B^2 + V_A^2}$	V_D
Buildings	1.00	0.06-0.09	0.08-0.10
Bridges	1.03-1.05	0.04-0.08	0.08-0.10

$$\lambda = \overline{D} / D_n = 1.05 \qquad V_D = 0.10$$

LIVE LOAD IN BUILDINGS

Design (Nominal) Live Load

- Live load represents the weight of people and their possessions, furniture, movable partitions and other portable fixtures and equipment.
- Usually, live load is idealized as a uniformly distributed load. The design live load is specified in psf (pounds per square foot) or kN/m^2.
- The magnitude of live load depends on the type of occupancy. For example, live loads specified by ASCE 7-95, *Minimum Design Loads for Buildings and Other Structures,* range from 10 psf (0.48 kN/m2) for uninhabited attics not used for storage, to 250 psf (11.97 kN/m2) for storage areas above ceilings.
- The value of live load also depends on the expected number of people using the structure and the effects of possible crowding.

Values of live load in different national codes

No	Country	Code	Load [kN/m²]
1	Poland	PN-82/B-02003 Structure load	2.0- 5.0
2	United Kingdom	BS 5400 : Part 2 Specification for loads : 1978	5.0
3	Europe	EUROCODE 1	5.0
4	USA	AASHTO LRFD	3.6 – 4.1
5	Canada	CAN/CSA-56-00 Canadian Highway Bride Design Code	1.6 – 4.0

The heaviest crowds were observed in front of stadium's and similar facilities

Crowd in front of stadium entrance, San Jose, California

Extreme weight of crowd – mosque in Mecca (Saudi Arabia)

During the peak of pilgrimage season, the number of visitors exceed one million

General overview

Ground floor level

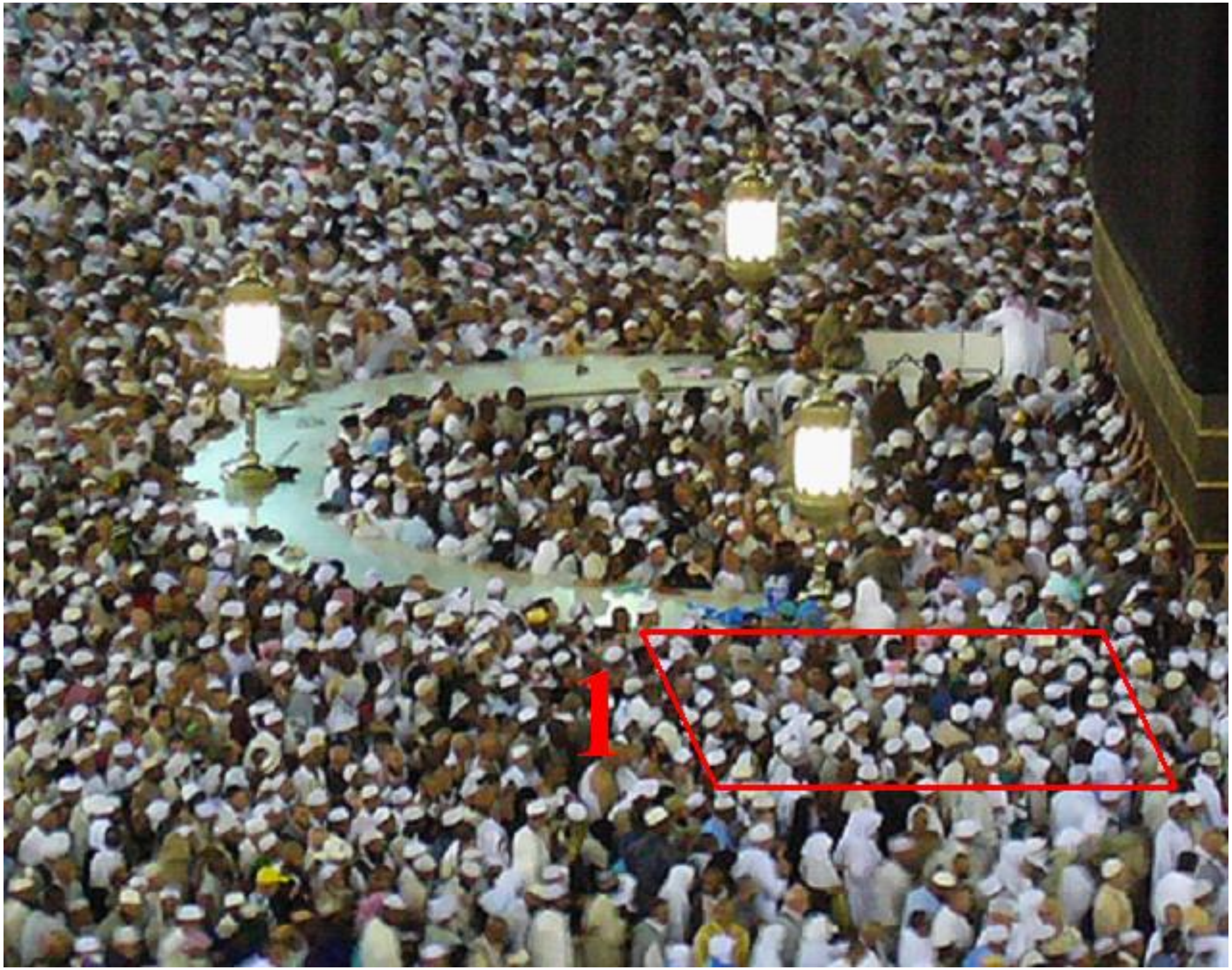
Ground floor level

Heaviest concentration of people

Review and analysis of the photographs requires some references with actual dimensions.

The heaviest crowds were observed in the immediate proximity of the Kaaba and Hateem. Therefore, these two structures served as references to facilitate the head count.

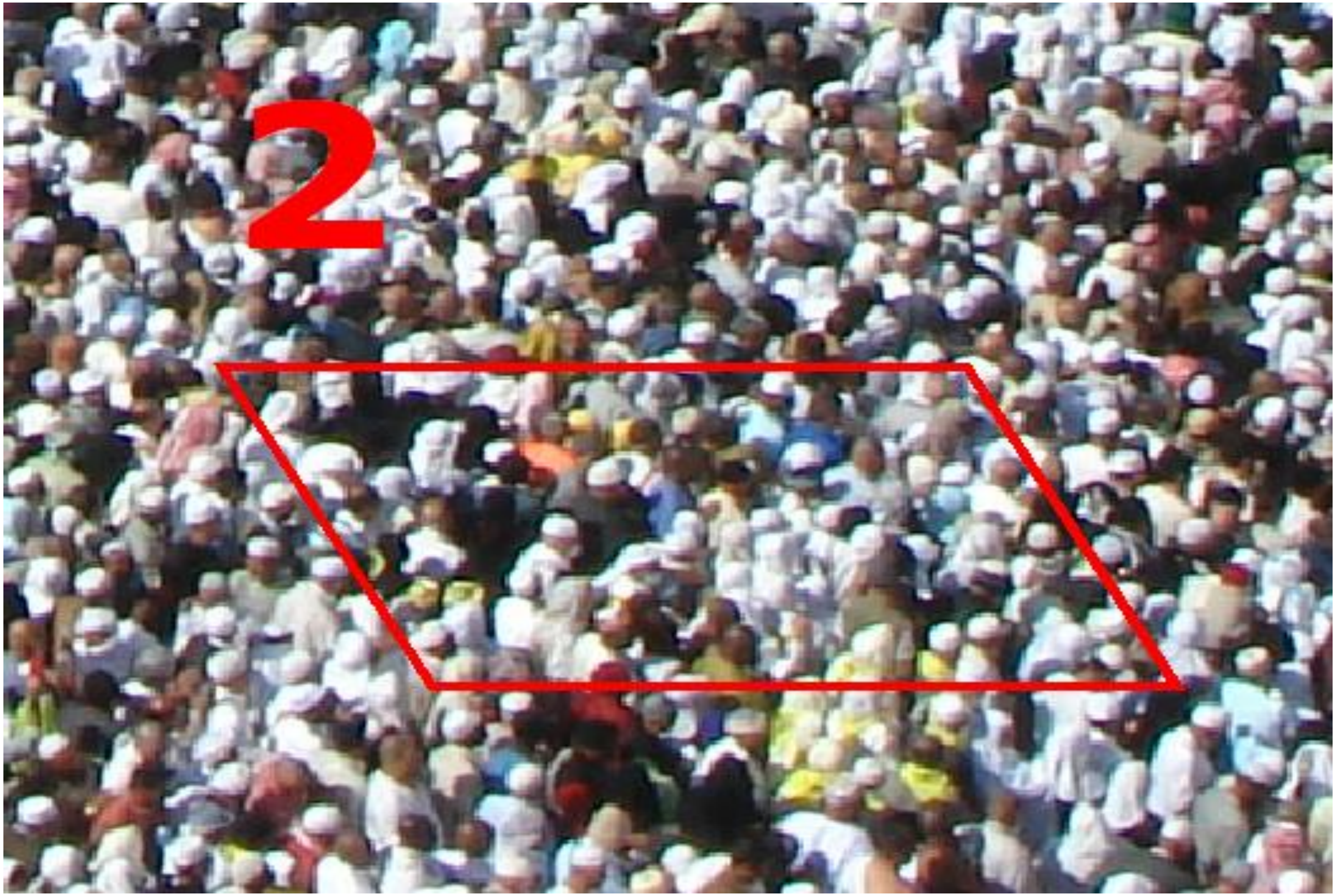
Crowd of pilgrims on the ground floor level



(75 persons @ 75 kg) / 16 m² = 351 kg/m²

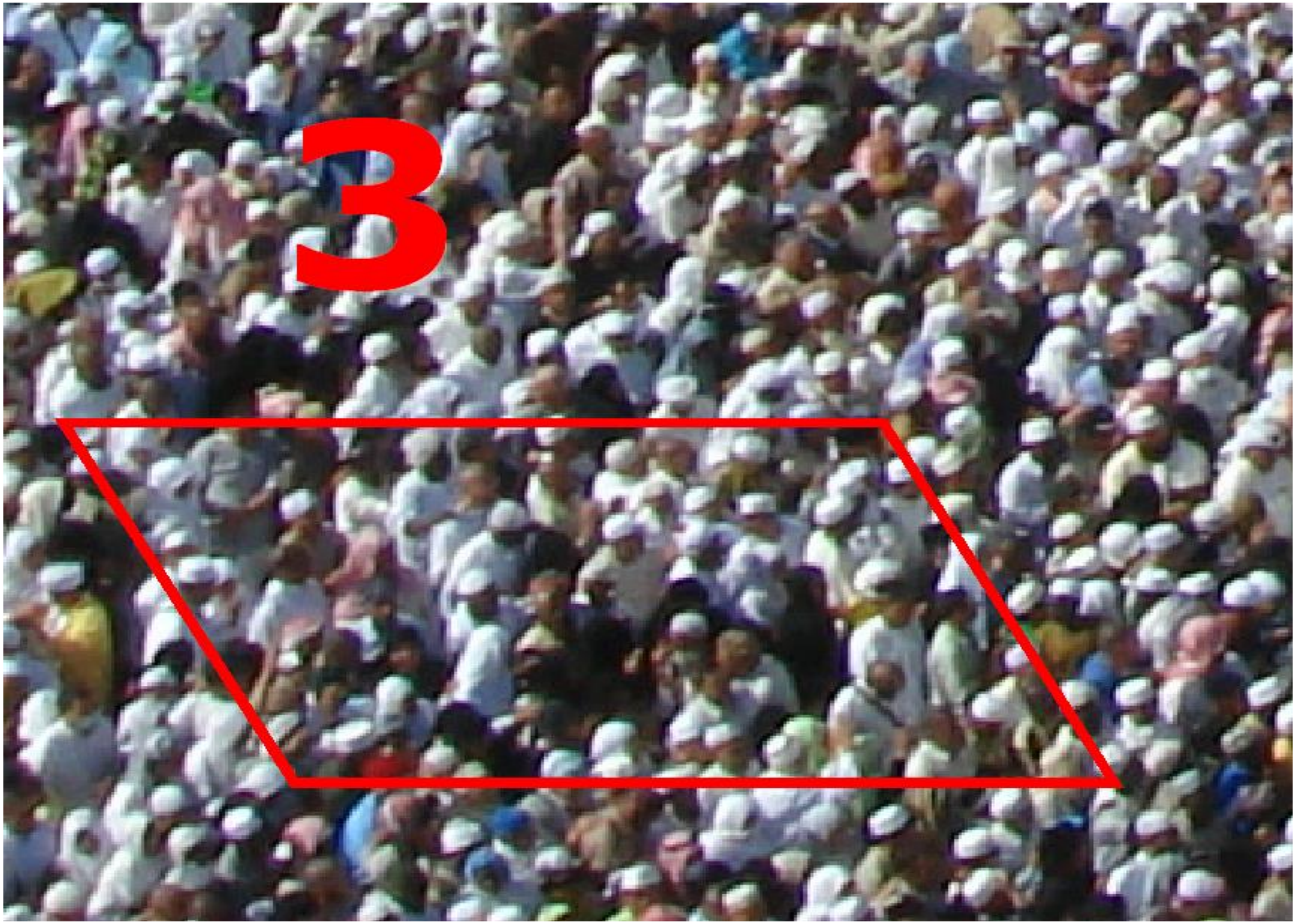
Location of square 2 and square 3

Crowd in square 2



(61 persons @ 75 kg) / 16 m² = 286 kg/m²

Crowd in square 3



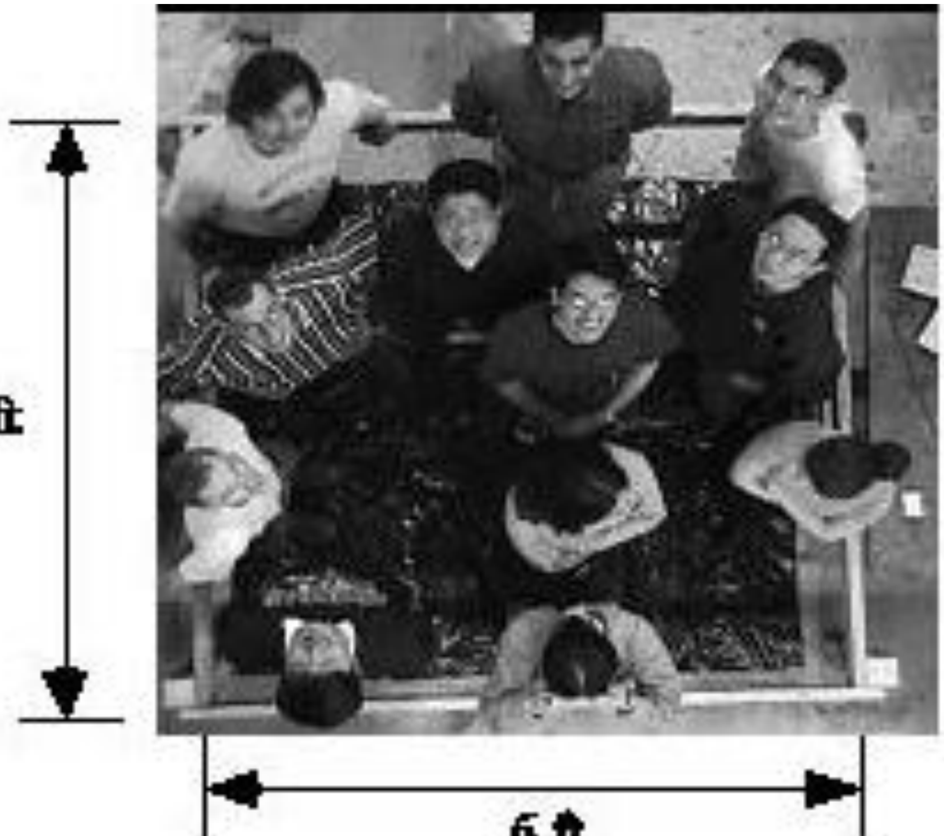
(63 persons @ 75 kg) / 16 m² = 295 kg/m²

Comparison results from Mecca with simulated in the Structures Lab

- In lab each person was weighed, and then they were placed in a square 1.8m x 1.8m.

- The total weight was controlled so that it was exactly equal to: 250 kg/m^2, 500 kg/m^2 and 750 kg/m^2.

LIVE LOAD IN BUILDINGS



50 psf (200 kgf/m²)

100 psf (500 kgf/m²)

LIVE LOAD IN BUILDINGS

150 psf (750 kgf/m^2)

Code specified live load for office space, 250kg/m² = 2,45[kPa]

Code specified live load for lobbies, platforms and corridors, 500 kg/m² = 4,9[kPa]

Code specified live load for stack rooms in libraries (heavy books), $750 kg/m^2 = 7,4$ [kPa]

The crowd shown in last photography is considerably above the upper physical limit. Such a density practically cannot be achieved because:

- There is no room, the space is all filled up, and the bodies were actually overflowing, outside of the boundaries of the marked square.
- People would suffocate because of the squeeze and lack of oxygen
- People cannot move. The crowd in photographs from Mecca is in motion.

Analysis of the results

- The dense crowd of people in the Holy Mosque in Mecca were examined and compared with the experimental results from the lab.

- The heaviest crowd in the mosque compound was observed in the immediate vicinity of Kaaba, and it is estimated at 351 kg/m^2.

- It is recommended to use **500 kg/m^2** as design live load for the Holy Mosque in Mecca. This value of live load is an upper limit, as the actual observed load densities are lower.

LIVE LOAD IN BUILDINGS

- The statistical parameters of live load depend on the area under consideration. The larger the area which contributes to the live load, the smaller the magnitude of the load intensity.
- ASCE 7-95 specifies the reduction factors for live load intensity as a function of the *influence area*. It is important to distinguish between influence area and tributary area. The tributary area is used to calculate the live load (or load effect) in beams and columns. The influence area is used to determine the reduction factors for live load intensity.

LIVE LOAD IN BUILDINGS

Members for which $K_{LL} A_T$ is larger than 400 ft^2 (37 m^2) are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

In SI units

$$L = L_0\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right)$$

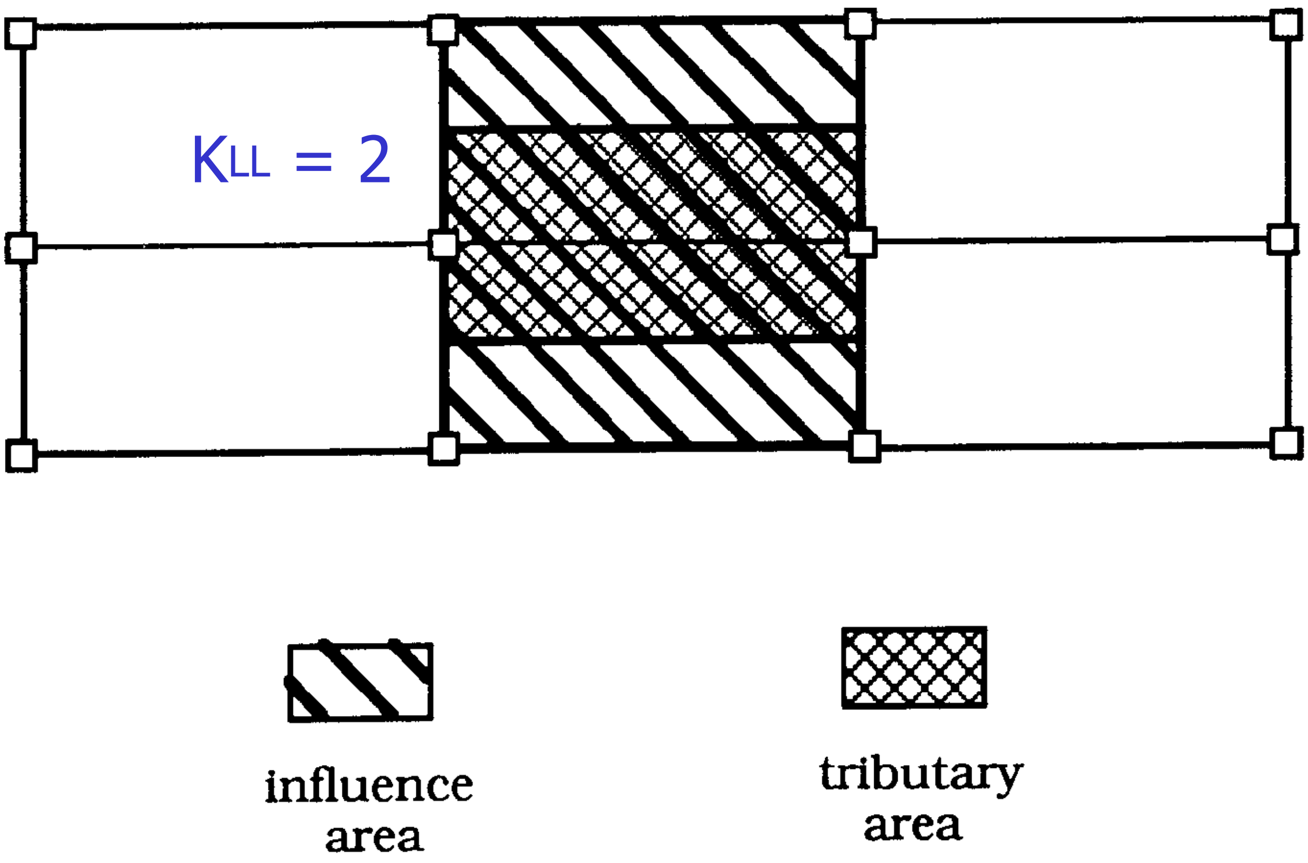
- L – reduced design live load per ft^2 (m^2) of area supported by the member
- L_0 – unreduced design live load per ft^2 (m^2) of area supported by the member
- K_{LL} – live load element factor (K_{LL} = 4 for columns and K_{LL} = 2 for beams)
- A_T – tributary area, ft^2 (m^2)

LIVE LOAD IN BUILDINGS - Exceptions

- L shall not be less than 0.5 L_o for members supporting one floor
- L shall not be less than 0.4 L_o for members supporting two or more floors
- Live loads that exceed 100 lb/ft^2 (4.79 kN/m^2) shall not be reduced
- Live loads shall not be reduced in passenger car garages
- Live loads of 100 lb/ft^2 (4.79 kN/m^2) or less shall not be reduced in public assembly occupancies

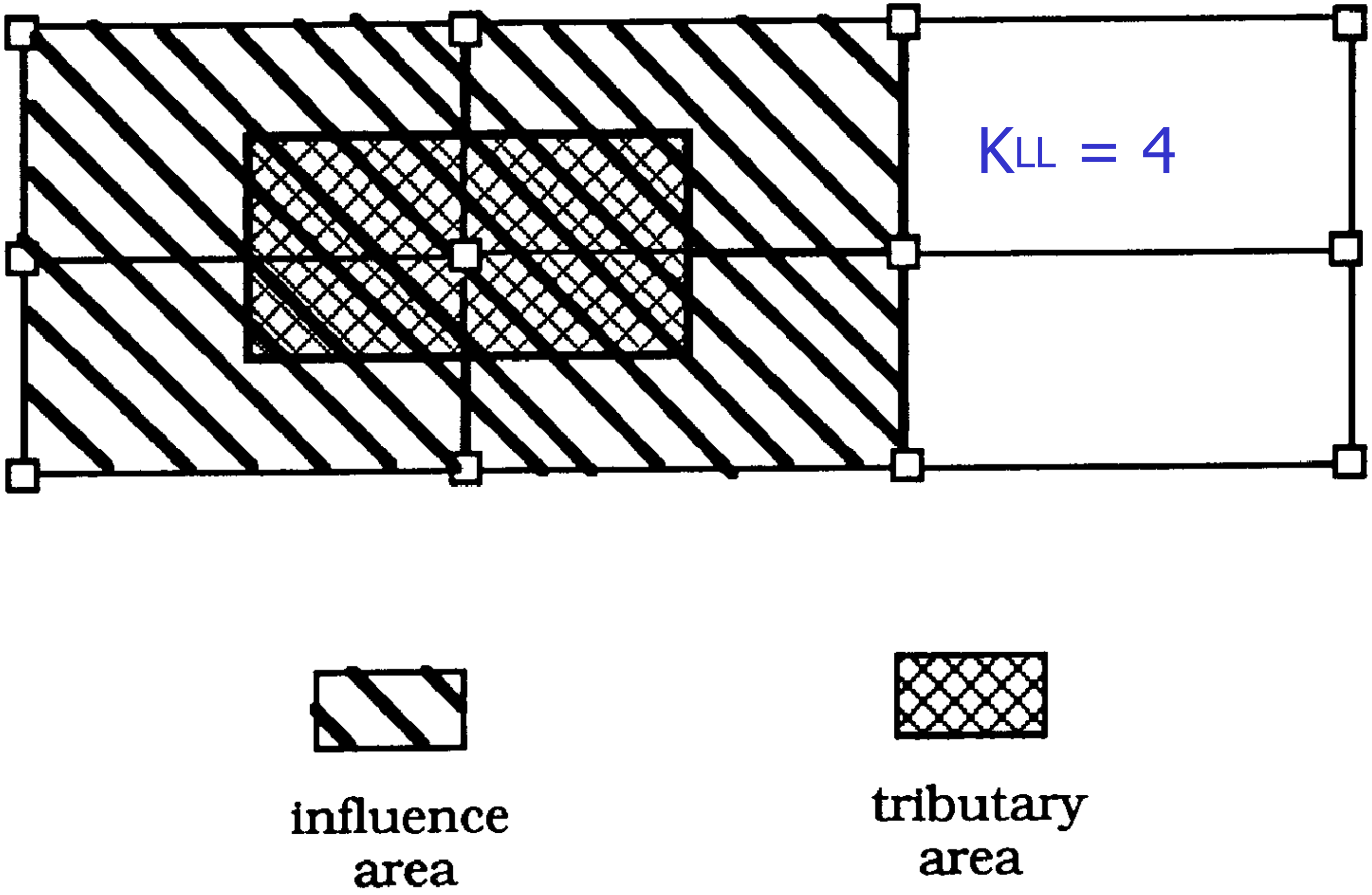
LIVE LOAD IN BUILDINGS

Influence and Tributary areas for beams



LIVE LOAD IN BUILDINGS

Influence and Tributary area for columns



$K_{LL} = 4$

Sustained Live Load

- Sustained live load is the typical weight of people and their possessions, furniture, movable partitions and other portable fixtures and equipment.
- The term “sustained” is used to indicate that the load can be expected to exist as a usual situation (nothing extraordinary). Sustained live load is also called an *arbitrary-point-in-time* live load, L_{apt}. It is the live load which you would most likely find in a typical office, apartment, school, hotel, etc...
- The sustained live load can be model as a gamma distributed random variable.
- The bias factor for L_{apt} offices and influence area ≤ 400 ft^2 (37 m^2), :

$$\lambda = \frac{\overline{L}_{apt}}{L_n} = \frac{12}{50} = 0.24$$

Sustained Live Load

The Table presents some typical values of the bias factors and the coefficients of variation for sustained live load as a function of influence area.

A_I	$\bar{L}_{apt} / L_n$	V_A
400	0.24	0.59 - 0.89
1,000	0.33	0.26 - 0.52
5,000	0.52	0.20 - 0.41
10,000	0.60	0.18 - 0.40

Transient Live Load

- Transient live load is the weight of people and their possessions that might exist during an unusual event such as an emergency, when everybody gathers in one room or when all the furniture is stored in one room.
- Since the load is infrequent and its occurrence is difficult to predict, it is called a transient load.
- Like sustained live load, the transient live load is also a function of the influence area rather than the tributary area.
- Some data on the transient component of live load are:

Influence Area	Coefficient of Variation
200	0.14 - 0.23
1,000	0.13 - 0.18
5,000	0.10 - 0.16
10,000	0.09 - 0.16

Maximum Live Load

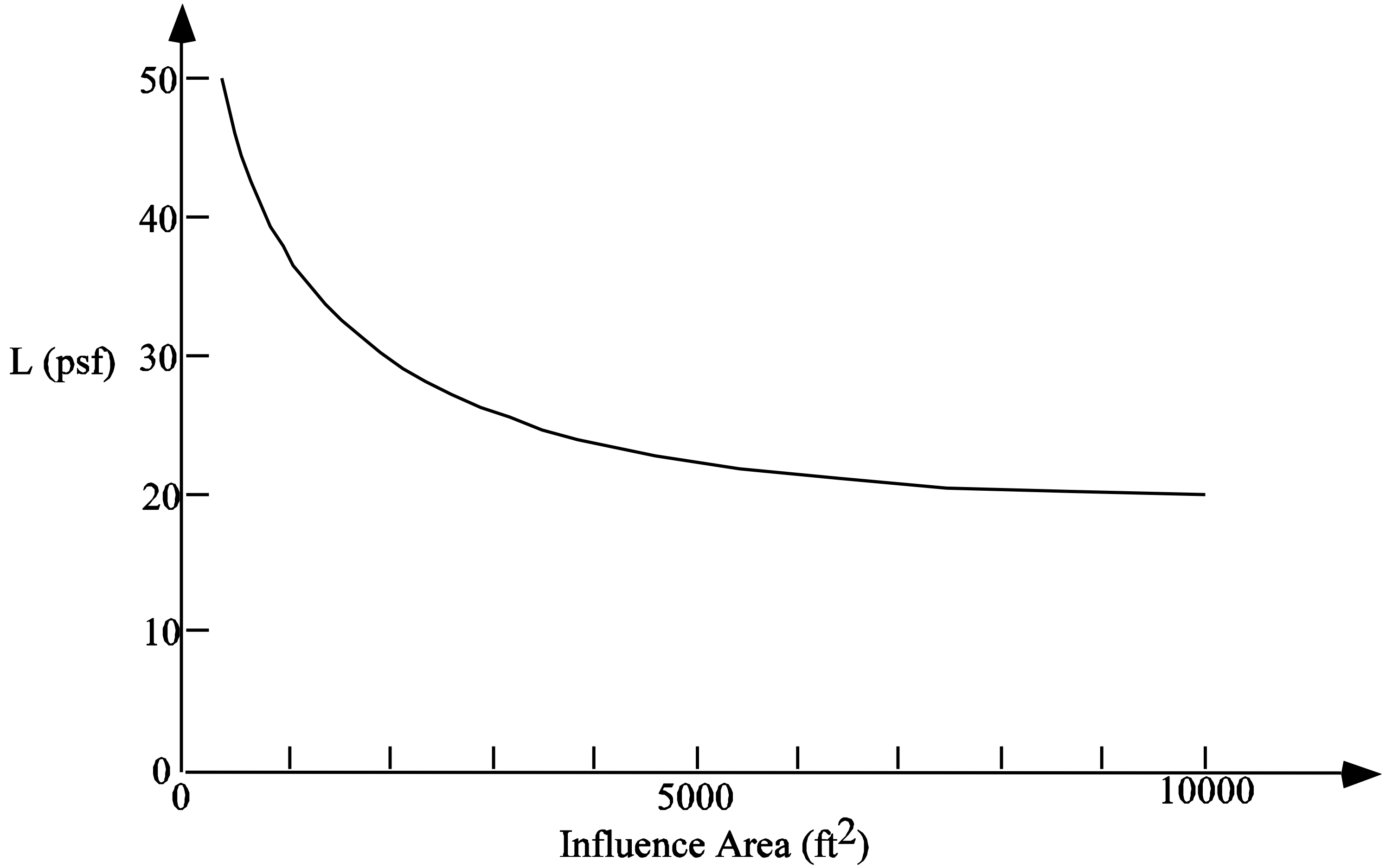
- For design purposes, it is necessary to consider the expected combinations of sustained live load and transient loads that may occur during the building's design lifetime (50-100 years).

- The probabilistic characteristics of the maximum live load depend on the temporal variation of the transient load, the duration of the sustained load (which is related to the frequency of tenant changes or changes in use), the design lifetime, and the statistics of the random variables involved

- The combined maximum live load can be modeled by an extreme type I distribution for the range of probability values usually considered in reliability studies.

Maximum Live Load

Mean Maximum 50 Year Live Load



ENVIRONMENTAL LOADS

The major types of environmental loads are:

- Wind load
- Snow load
- Earthquake load
- Temperature effects

Wind Load

The major parameters related to wind include:

- Wind speed
- Pressure coefficient
- Exposure
- Gust factor
- Dynamic response

Wind effect can be considered as a product of several parameters:

$$W = c \cdot C_P \cdot E_z \cdot G \cdot V^2$$

Where:

c = constant

C_P = pressure coefficient (geometry of structure)

E_z = exposure coefficient (location, urban area, open country)

G = gust factor (turbulence, dynamic interaction of structure and wind)

V = wind speed at height of 10 m

Wind Load

Each of the major parameters related to wind is treated as an independent event.

Then, the CDF for one year is:

$$F_V(v) = P(V \leq v)$$

For two years (occurrence of wind in each year is independent event):

$$F_{V_2}(v) = P(V \leq v \text{ in the first year and } V \leq v \text{ in the second yr}) =$$
$$= P(V \leq v) * P(V \leq v) = [P(V \leq v)]^2 = F_V^2(v)$$

For 50 years:

$$F_{V_{50}}(v) = [F_V(v)]^{50}$$

Wind Load

- Wind has Extreme Type I distribution
- All parameters of wind load are random variables
- The bias factors for wind parameters can be taken equal 1.0
- Coefficients of variation are:

$$V_{C_p} = 0.12, \quad V_{E_Z} = 0.16, \quad V_G = 0.11, \quad V_C = 0.05$$

- Constant c can be treated as deterministic value

Wind Load

Wind load data for some selected sites in United States are presented below:

Site	Annual			50-yr Max.			V_n	W/W_n	
	m	$\overline{V}$	V_V	$\overline{V}_{50}$	$V_{V_{50}}$	c.o.v	A58.1-1972	u	α
Baltimore, MD	29	55.9	0.12	76.9	0.09	0.11	75	0.96	5.48
Detroit, MI	44	48.9	0.14	69.8	0.10	0.12	80	0.51	5.31
St. Louis, MO	19	47.4	0.16	70.0	0.11	0.14	70	0.62	3.18
Austin, TX	35	45.1	0.12	61.9	0.09	0.11	80	0.43	8.03
Tucson, AZ	30	51.4	0.17	77.6	0.11	0.14	70	0.69	2.52
Rochester, NY	37	53.5	0.10	69.3	0.08	0.09	70	0.71	4.83
Sacramento, CA	29	46.0	0.22	77.3	0.13	0.16	65	0.65	1.77

Snow Load

The weight of snow on roofs can be a significant load to consider for structures in mountainous regions and snow belts. For design purposes, the snow load on a roof is often calculated based on information on the ground snow cover.

Snow load can be considered as a product of several parameters:

Where:
p_g = ground snow load (psf or kN/m^2)
C_e = exposure coefficient
C_t = thermal factor
I = importance factor

$$p_f = 0.7 \cdot C_e \cdot C_t \cdot I \cdot p_g$$

For sloping roofs:

$$p_s = C_s \cdot p_f$$

where: C_s = roof slope factor

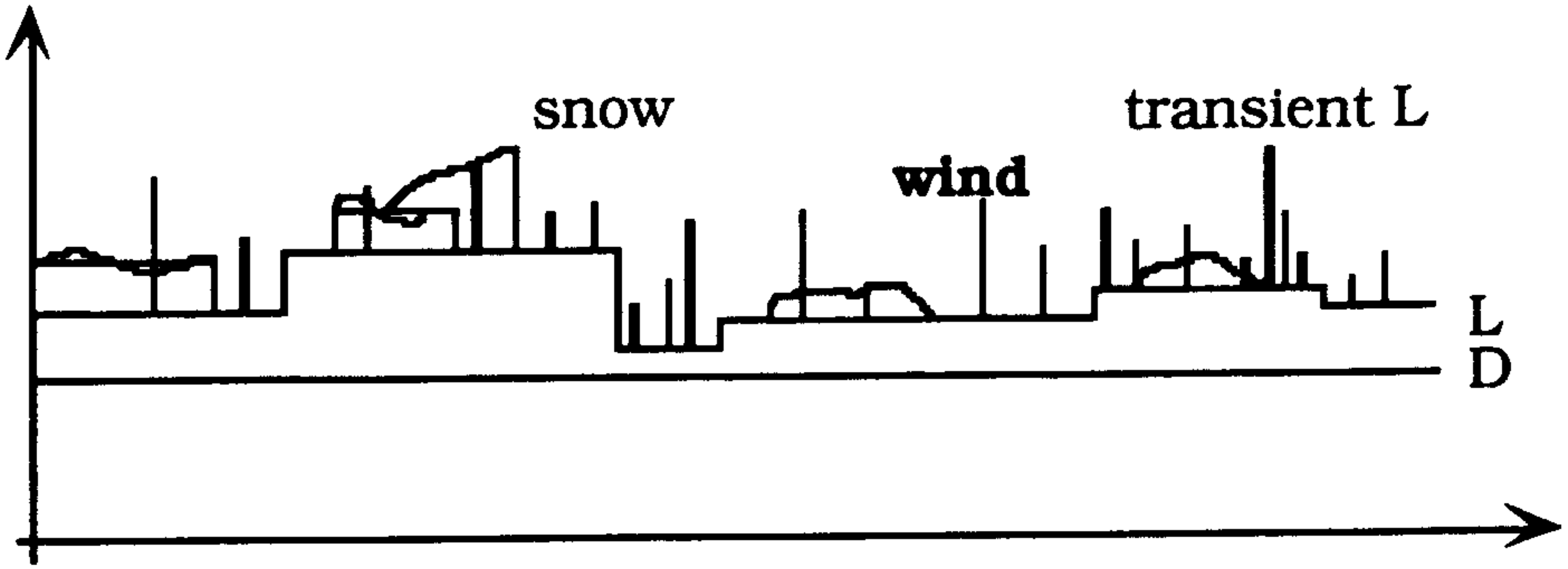
Snow Load

- Snow can be modeled as lognormal or Extreme Type I distribution
- Water-Equivalent Ground Snow load data for some selected sites in United States are presented below:

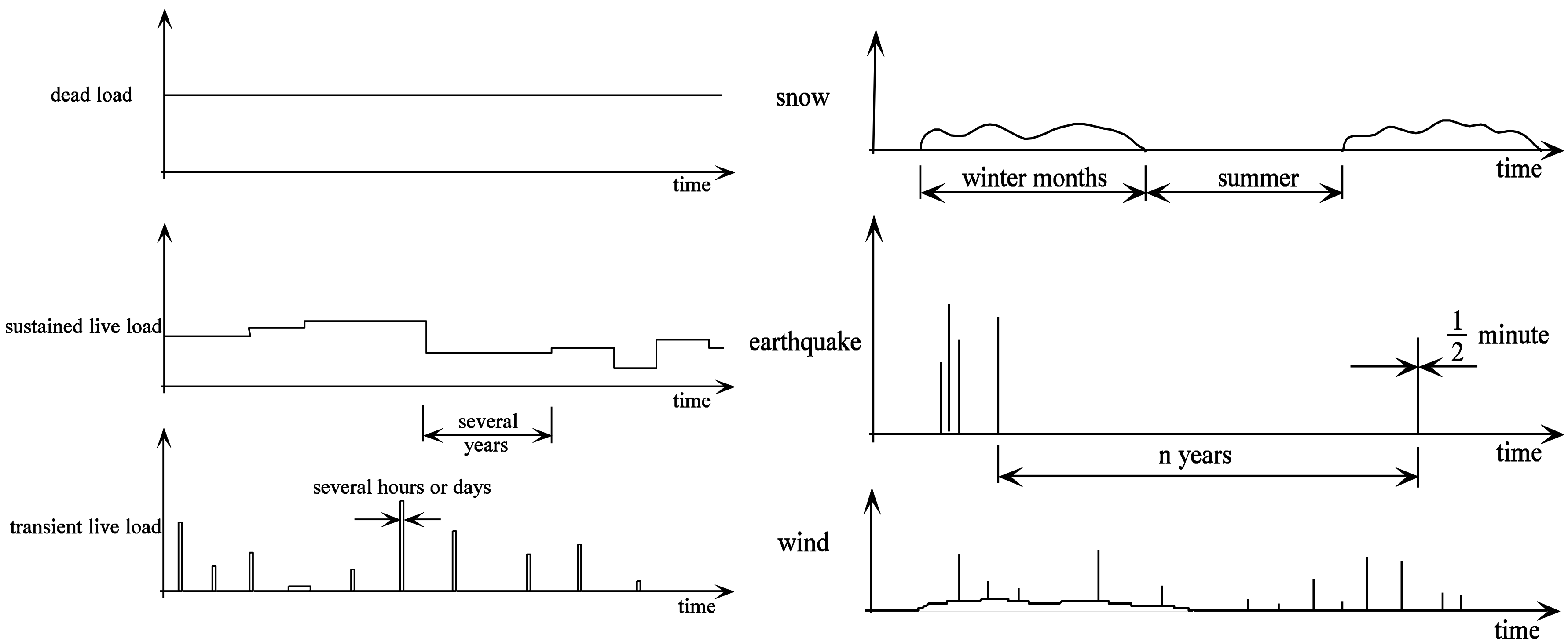
Site	Annual Extreme Ground Load			A58.1-1972	50-yr Maximum Roof Load	
	Years of Record	λ	ζ	q_n	u	α
Green Bay, WI	26	2.01	0.70	28	0.87	5.07
Rochester, NY	26	2.49	0.56	34	0.83	6.16
Boston, MA	25	2.28	0.51	30	0.70	6.63
Detroit, MI	20	1.63	0.58	18	0.69	5.97
Omaha, NB	25	1.60	0.69	25	0.62	5.20
Cleveland, OH	26	1.50	0.58	19	0.60	6.30
Columbia, MO	25	1.21	0.84	20	0.69	4.05
Great Falls, MT	26	1.77	0.49	15	0.80	7.16

LOAD COMBINATIONS

- Total load, Q, is the sum of load components (dead load, live load, snow, wind, earthquake, temperature,…)
- Load components are time-variant and the calculation of CDF is very difficult.

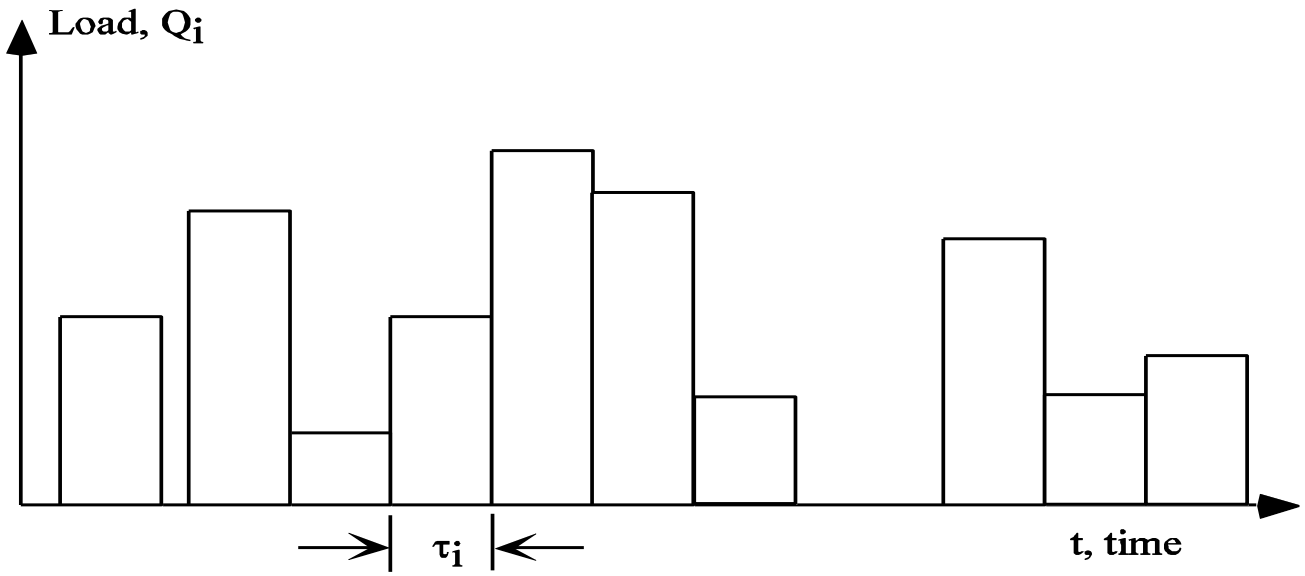


Examples of Time Histories for Various Load Components



Ferry Borges and Castanheta Model for Load Combination

It is assumed that for each load component X_i, there is a basic time interval, τ_i, as shown below:



- Magnitude of Xi can be considered as constant during this period time
- The occurrence or non-occurrence of Xi in each time corresponds to repeated independent trials with probability of occurrence, p

Ferry Borges and Castanheta Model for Load Combination

- For basic time interval, τ,

$$F(x) = \text{Prob}\,(X < x) = p\,F_1(x) + (1 - p)\,1$$

- The CDF for a single time interval is then:

$$F(x) \doteq 1 - p\,[\,1 - F_1(x)\,]$$

- For 2 basic intervals:

$$F(x) = \text{Prob}\,(X < x) = [\,\text{Prob}(X < x \text{ in the first interval}) \text{ times } \text{Prob}(X < x \text{ in the second interval})\,] =$$

- For n basic intervals:

$$F(x) = \{\,1 - p\,[\,1 - F_1(x)\,]\,\}^n$$

Where: n = number of intervals

p = probability of occurrence in each interval

Ferry Borges and Castanheta Model for Load Combination

If $Q = X_1 + X_2$, then the parameters of Q can be determined as follows:

$$F(x_1) = \{ 1 - p_1 [1 - F_1(x_1)] \}^k$$

$$F(x_2) = \{ 1 - p_2 [1 - F_2(x_2)] \}$$

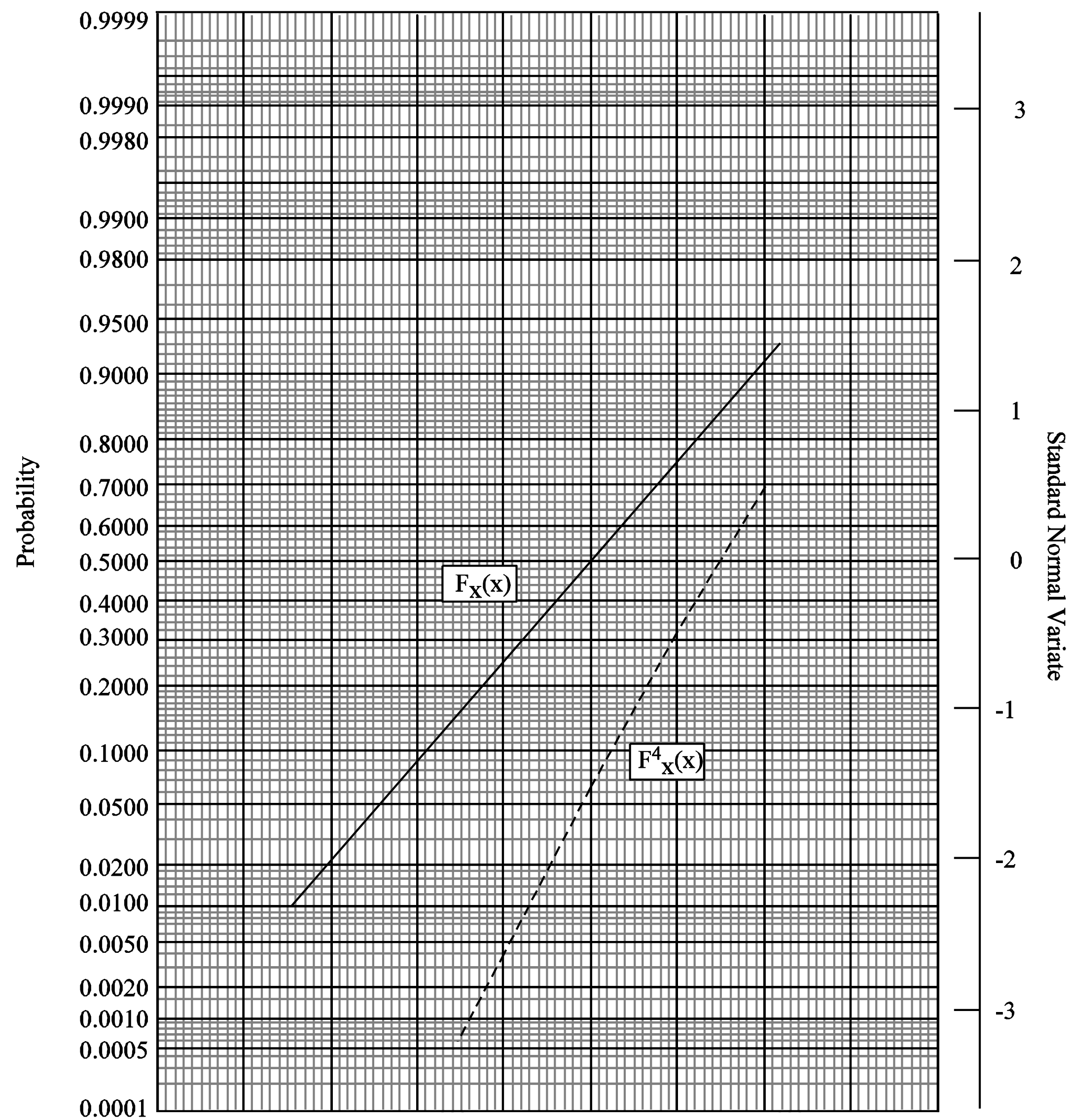
$$\overline{Q} = \overline{X_1} + \overline{X_2} \qquad \sigma_Q = \sqrt{\sigma^2_{X_1} + \sigma^2_{X_2}}$$

Ferry Borges and Castanheta Model for Load Combination

Example

Consider the load component, X, with CDF, $F_X(x)$, corresponding to the basic interval, τ.

If p = 1 and k = 4, then the CDF for the interval 4τ it is $F_X^4(x)$



Turkstra's Rule

This is a practical approach to load modeling

It is assumed that when one load component takes an extreme value then other load components take average value

Let X1, X2, …. , Xn be load components:

$$Q = X_1 + X_2 + \dots + X_n$$

- CDF for the maximum 50 year value:

$$F_{X_{i50}}(x) = P\ (X \le x)\ \text{in 50 years}$$

- CDF for the arbitrary point-in-time:

$$F_{X_i}(x) = P\ (X \le x)\ \text{in any moment}$$

Turkstra's Rule

Turkstra's rule states as follows:

$$\max Q = \max \begin{cases} \max X_1 + \text{ave}\,(X_2 + X_3 + \ldots + X_n) \\ \max X_2 + \text{ave}\,(X_1 + X_3 + \ldots + X_n) \\ \vdots \\ \max X_n + \text{ave}\,(X_1 + X_2 + \ldots + X_{n-1}) \end{cases}$$

where: $\max X_i$ = maximum 50 year load X_i

$\text{ave}\ X_j$ = arbitrary point-in-time load X_i

Then mean and variance (standard deviation) can be calculated as follows:

$$\overline{\max Q} = \max \begin{cases} \overline{\max X_1} + \text{ave}\,(\bar{X}_2 + \bar{X}_3 + \ldots + \bar{X}_n) \\ \vdots \\ \overline{\max X_n} + \text{ave}\,(\bar{X}_1 + \bar{X}_2 + \ldots + \bar{X}_{n-1}) \end{cases}$$

$$\sigma^2_{\max Q} = \sigma^2_{\max X_i} + \sigma^2_{\text{ave} X_i} + \ldots + \sigma^2_{\text{ave} X_n}$$

Turkstra’s Rule

Example

Consider a combination of dead load, live load and wind load. For each load component there are two sets of parameters given: maximum value and average. Calculate the parameters of a combined effect of these components.

- Dead load is normally distributed:

$$\overline{D} = 20, \quad V_D = 10\%$$

- For live load , max L is extreme type I:

$$\overline{\max L} = 30, \quad V_{\max L} = 12\%$$

- For live load , ave L is gamma distributed:

$$\overline{\text{ave } L} = 9, \quad V_{\text{ave } L} = 31\%$$

- For wind load, max W is extreme type I:

$$\overline{\max W} = 24, \quad V_{\max W} = 20\%$$

- For wind load, ave W is lognormal:

$$\overline{\text{ave } W} = 1, \quad V_{\text{ave } W} = 60\%$$

Turkstra's Rule

The total load effect:

$$Q = D + L + W$$

Parameters of max Q:

$$Q_{max} = \max D + \begin{cases} \max L + \text{ave } W \\ \max W + \text{ave } L \end{cases}$$

$$\overline{Q}_{max} = 20 + \max \begin{cases} \overset{\bar{L}_{max}}{30} + \overset{W_{ave}}{1} \\ \boxed{\underset{W_{max}}{24} + \underset{L_{ave}}{9}} \; \text{governs} \end{cases} = 20 + 33 = 53$$

$$\sigma^2_{Q_{max}} = (24 \text{ x } 0.20)^2 + (9 \text{ x } 0.31)^2 + (20 \text{ x } 0.10)^2 = 4.8^2 + 2.79^2 + 2^2 = 5.90^2$$

Live Load for Bridges

- For bridge design, the live load covers a range of forces produced by vehicles moving on the bridge.

- The effect of live load on the bridge depends on many parameters such as:

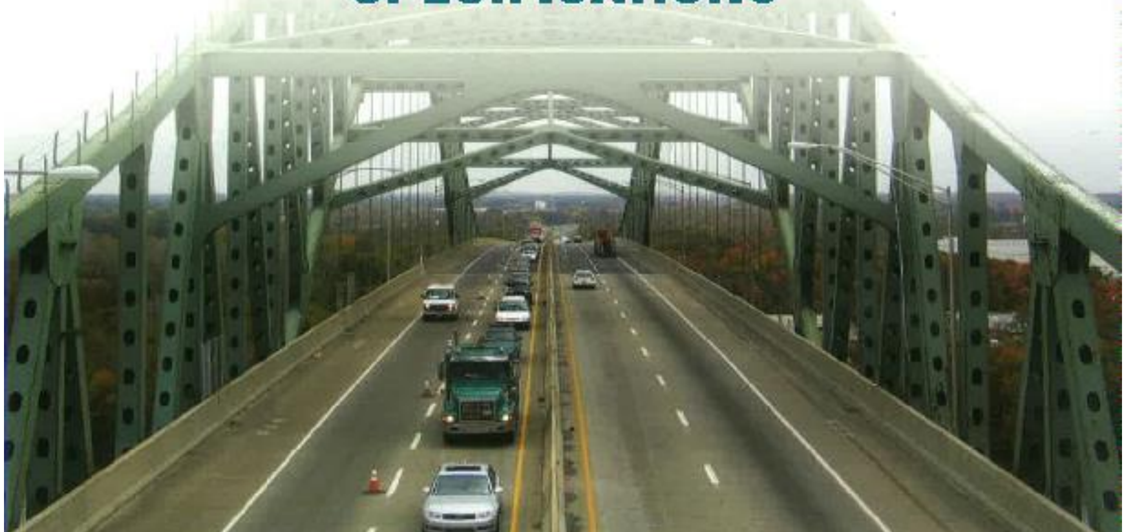
  - span length,
  - truck weight,
  - axle loads, axle configuration,
  - position of the vehicle on the bridge (transverse and longitudinal),
  - number of vehicles on the bridge (multiple presence),
  - girder spacing, and stiffness of structural members (slab and girders).

Live Load for Bridges

- Live load on bridges is characterized not only by the load itself, but also by the distribution of this load to the girders. Therefore, the most important item to be considered is the load spectrum per girder.
- The design live load specified by AASHTO Standard (2002) is shown in Figure (a). For shorter spans, a *military load* is specified in the form of a tandem with two 24 kip (106 kN) axles spaced at 4 ft (1.2 m). The design load specified by AASHTO LRFD (1998) is shown in Figure (c). The *design tandem* in LRFD is based on two 25 kip (110 kN) axles.

AASHTO

LRFD BRIDGE DESIGN SPECIFICATIONS



AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AASHTO
THE VOICE OF TRANSPORTATION

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U.S. Customary Units

Part I: Sections 1—6

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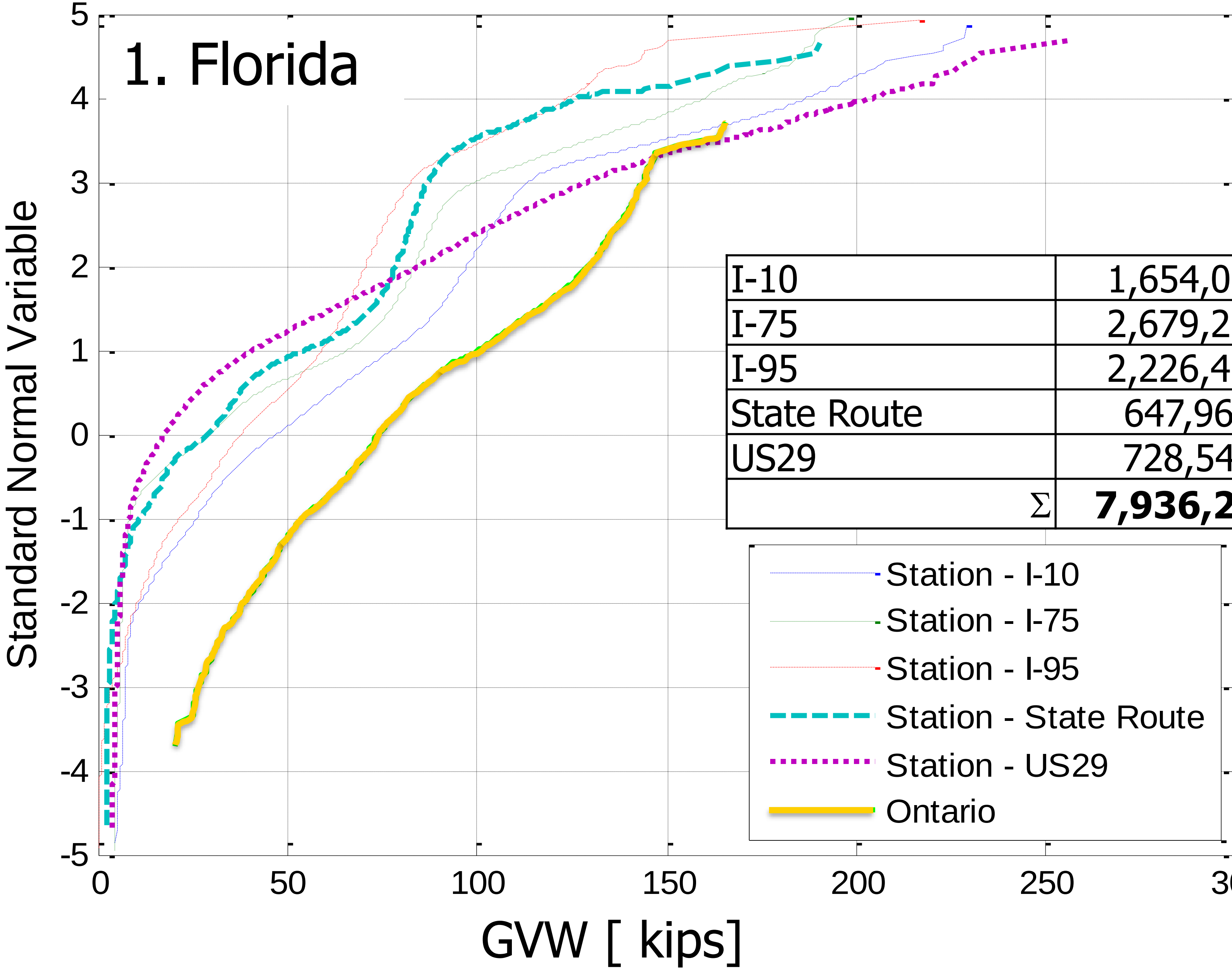
Statistical Data Base

- Load surveys, e.g. weigh-in-motion (WIM) truck measurement
- Load distribution (load effect per component)
- Simulations (e.g. Monte Carlo)
- Finite element analysis
- Boundary conditions (field tests)

Summary of Collected WIM Data

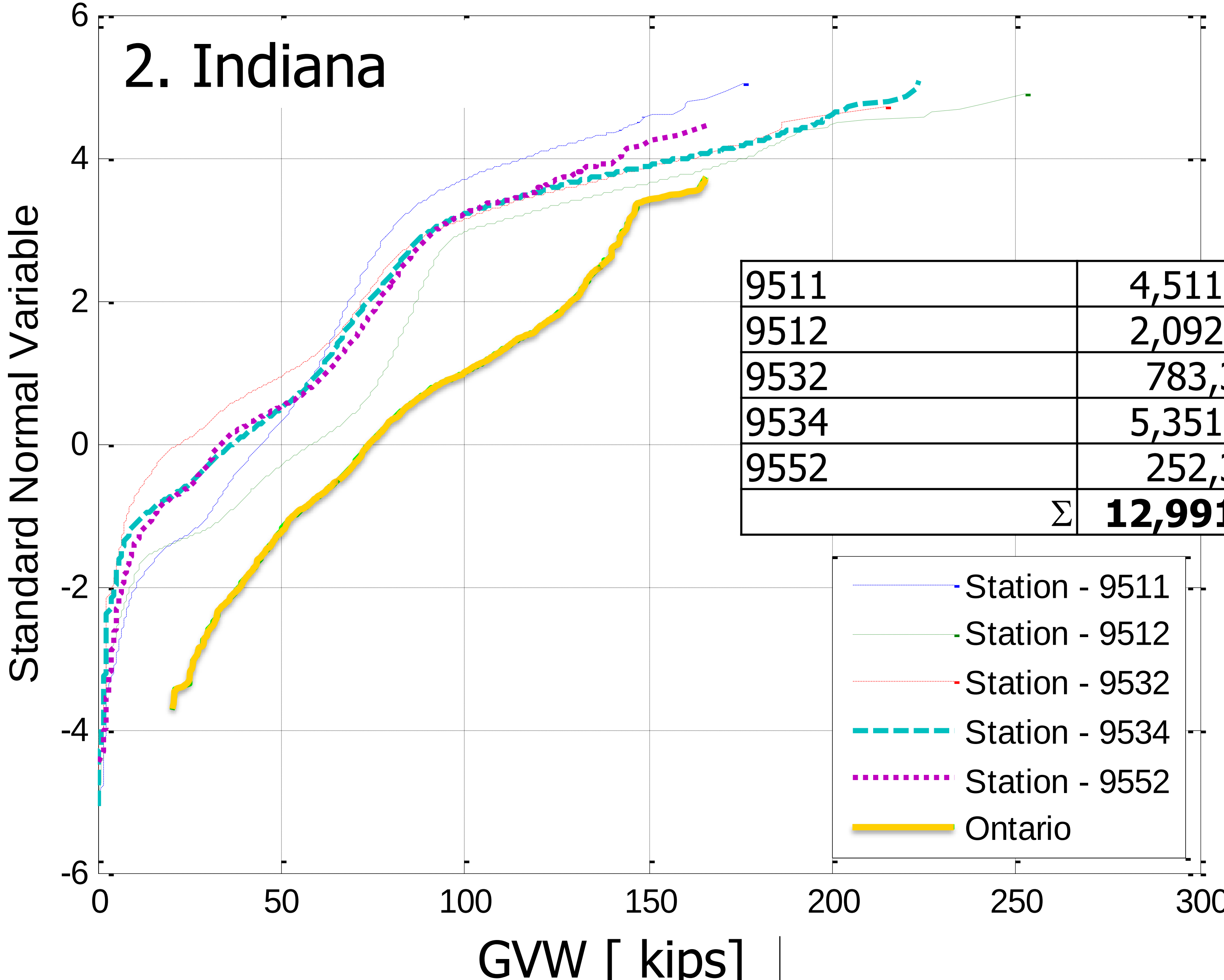
WIM Site Location	Number of trucks
Florida	7,936,283
Indiana	12,991,113
Mississippi	6,709,863
New York	7,791,636
Σ	**35,428,895**

Gross Vehicle Weight



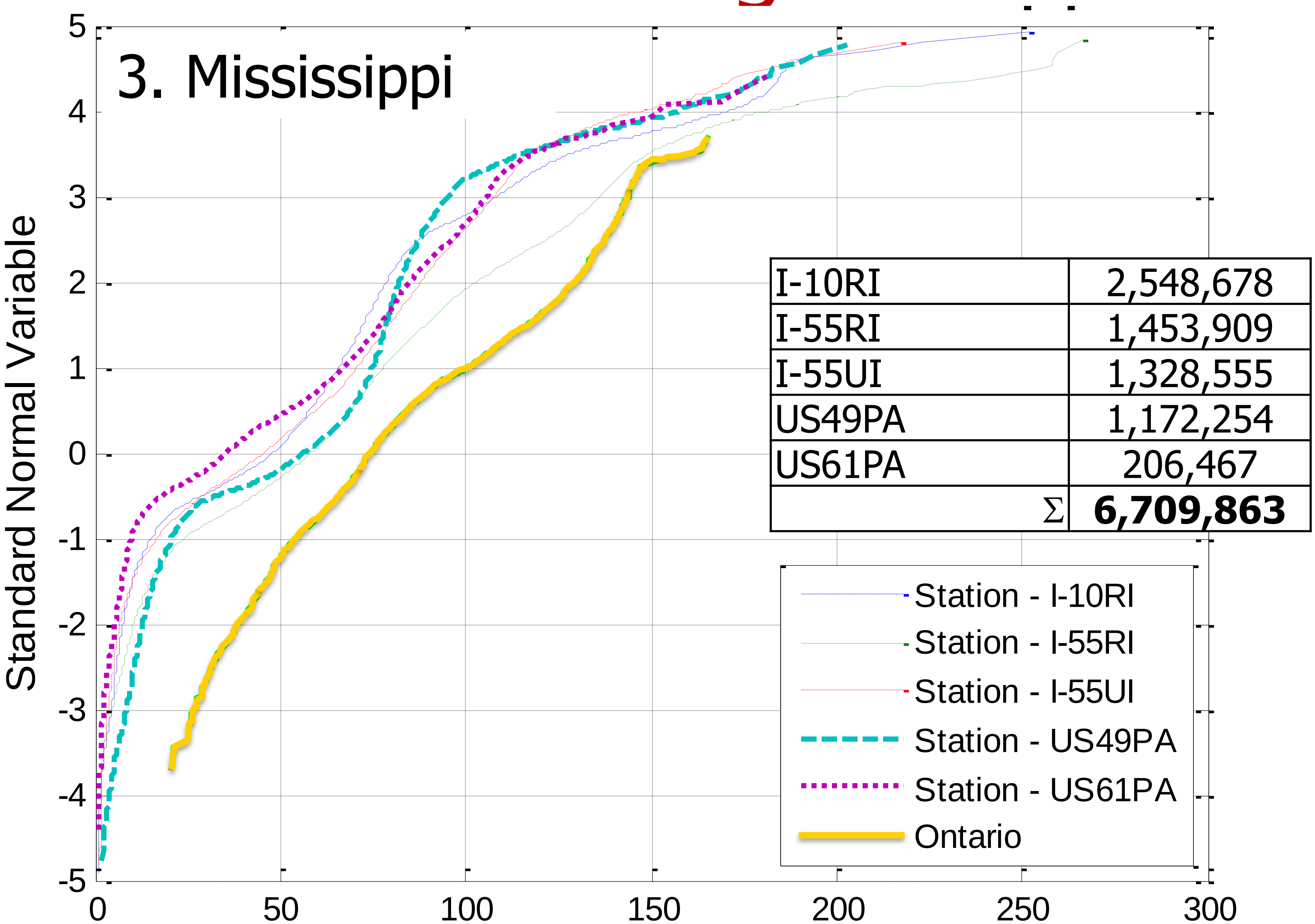
I-10	1,654,006
I-75	2,679,288
I-95	2,226,480
State Route	647,965
US29	728,544
Σ	**7,936,283**

Gross Vehicle Weight



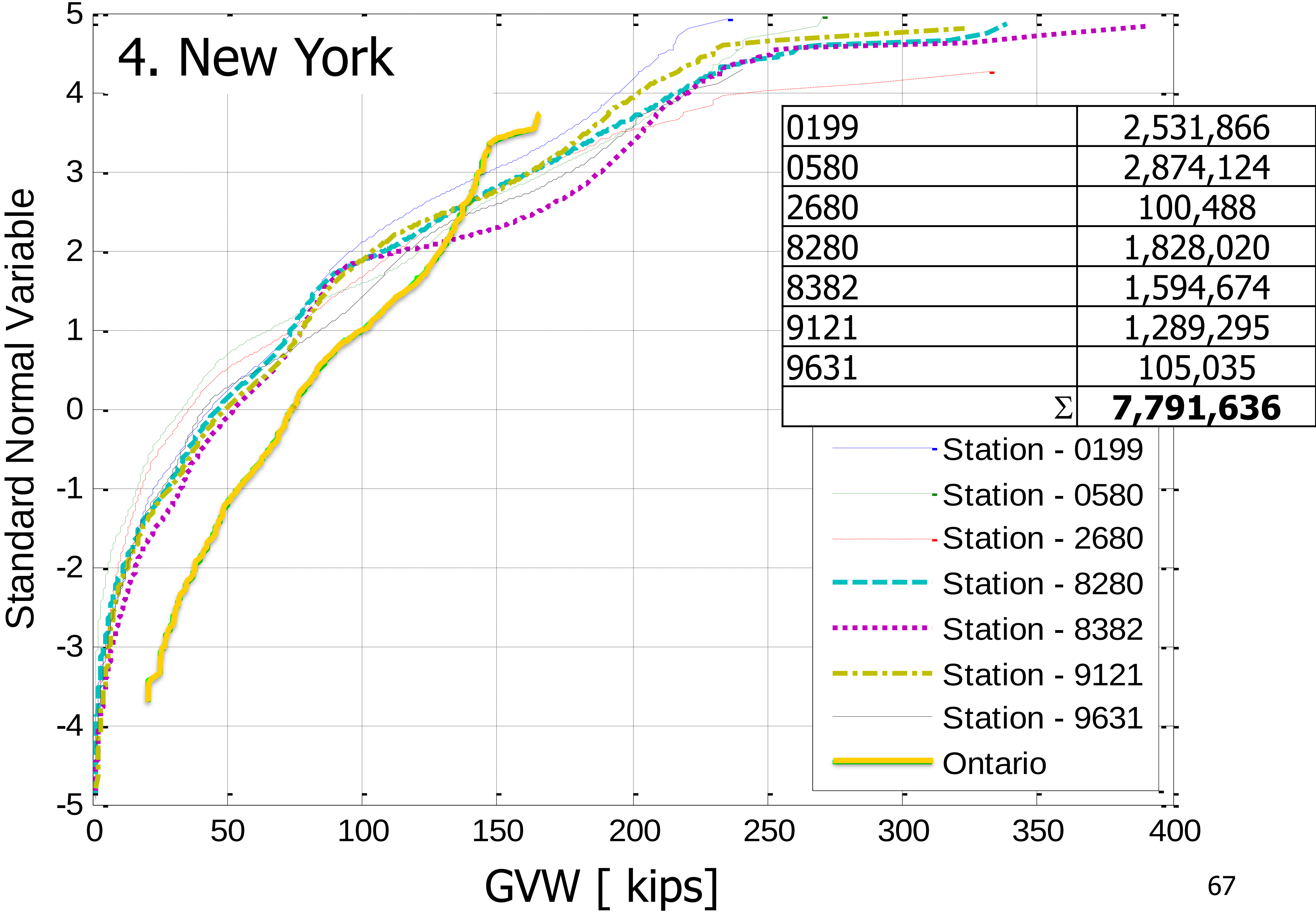
9511	4,511,842
9512	2,092,181
9532	783,352
9534	5,351,423
9552	252,315
Σ	**12,991,113**

Gross Vehicle Weight



I-10RI	2,548,678
I-55RI	1,453,909
I-55UI	1,328,555
US49PA	1,172,254
US61PA	206,467
Σ	**6,709,863**

Gross Vehicle Weight



0199	2,531,866
0580	2,874,124
2680	100,488
8280	1,828,020
8382	1,594,674
9121	1,289,295
9631	105,035
Σ	**7,791,636**

Truck Data Analysis - Development of Numerical Procedure

For each cross-section an influence line was constructed according to:

$$M(z) = P(1-\frac{x}{L})a$$ for x(z) ≥ a

$$M(z) = P(1-\frac{x}{L})a - P(a-x)$$ for x(z) < a

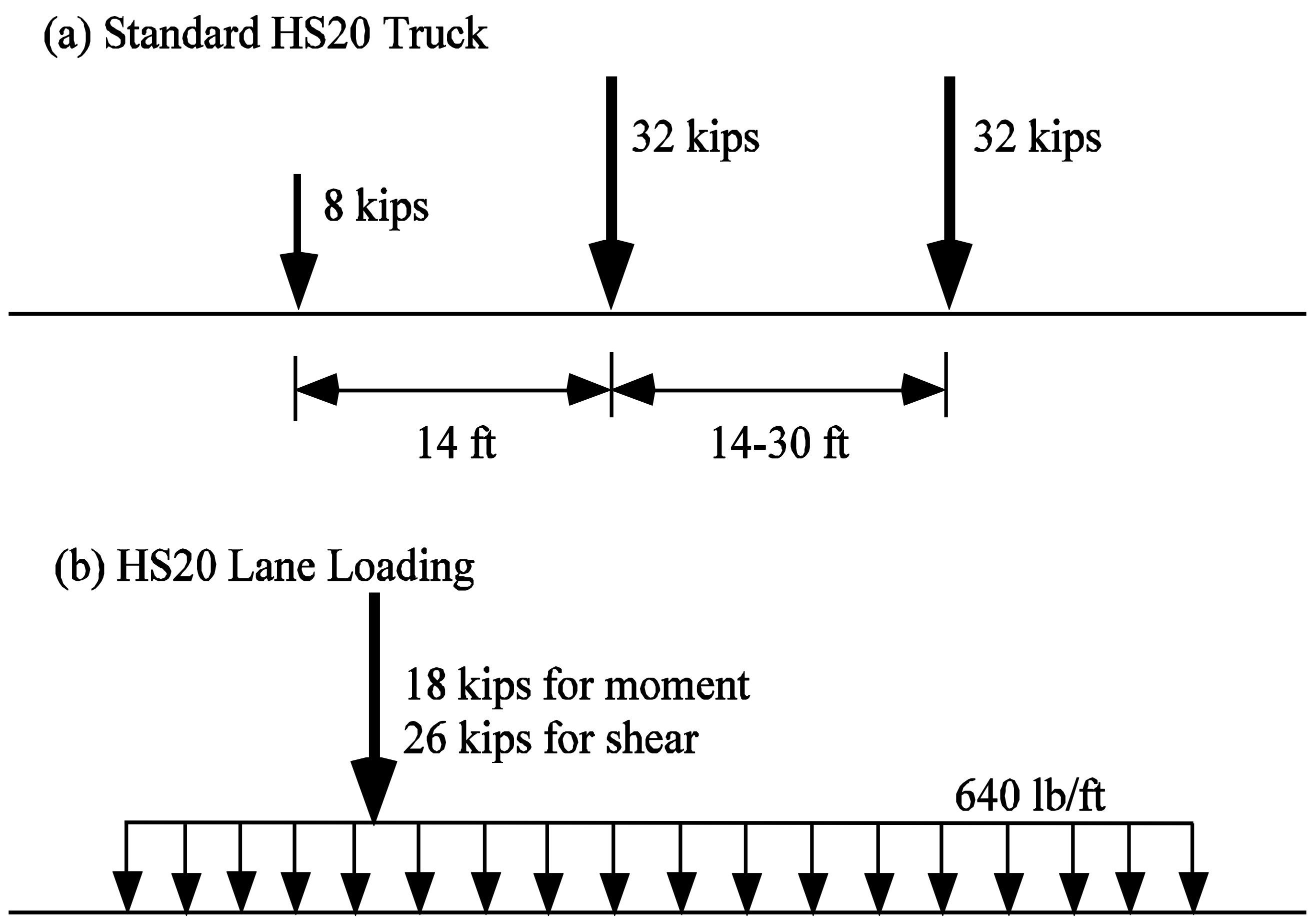
$$M(z) = 0$$ for x(z) ≤ 0

$$M(z) = 0$$ for x(z) ≥ L

X – position of the force
a – location of considered cross-section
L – span length
z – vehicle number

- Method of superposition for the multiple forces

Design Live Load in Bridges (AASHTO Standard Specifications 2002)



Design Live Load in Bridges (AASHTO Standard Specifications 2002)

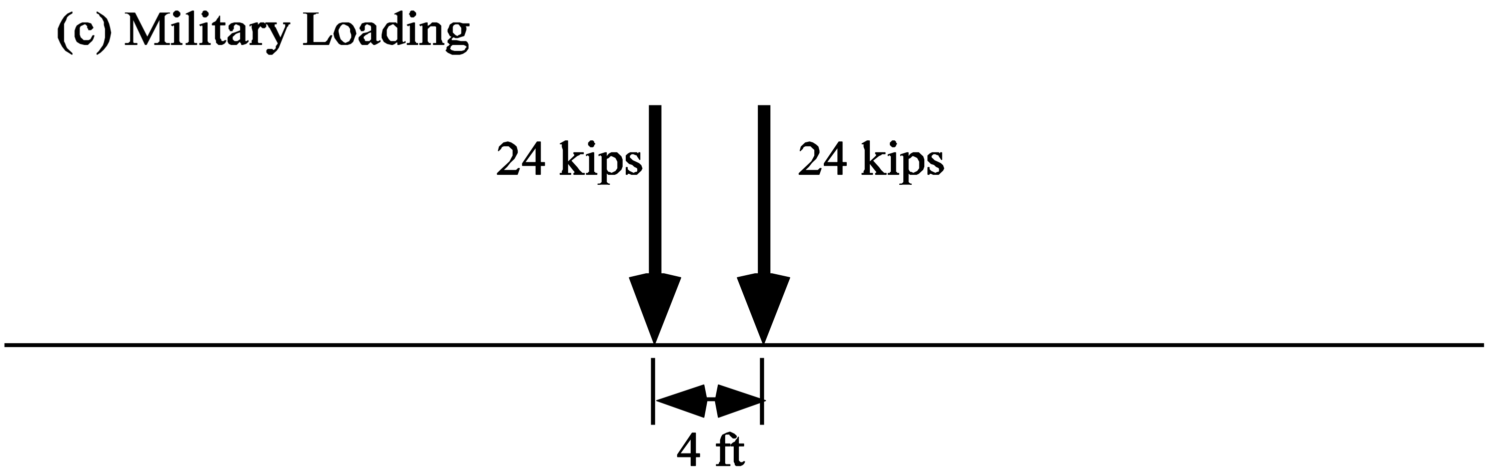
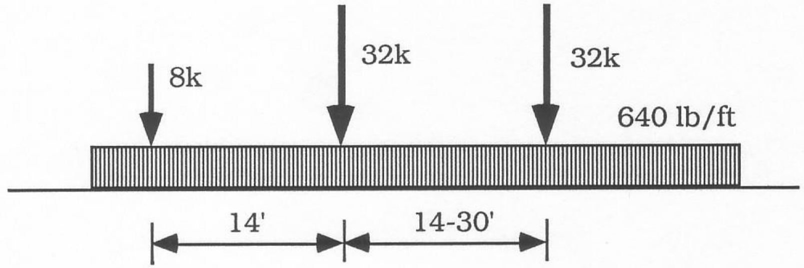
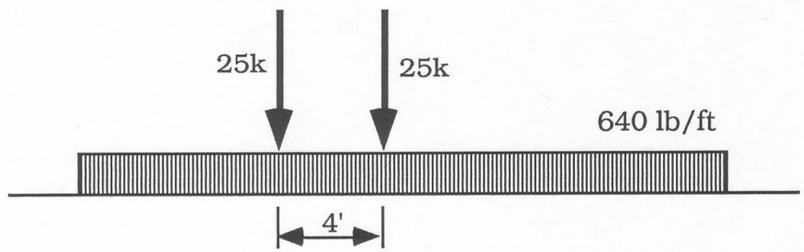


Figure 6-5 HS20 Loading (AASHTO 1996). The code is available only in US units (1 kip = 4.45 kN, 1lb/ft = 15 N/m, 1 ft = 0.3 m).

(a) Truck and Uniform Load



(b) Tandem and Uniform Load



(c) Alternative Load for Negative Moment (reduce to 90%)

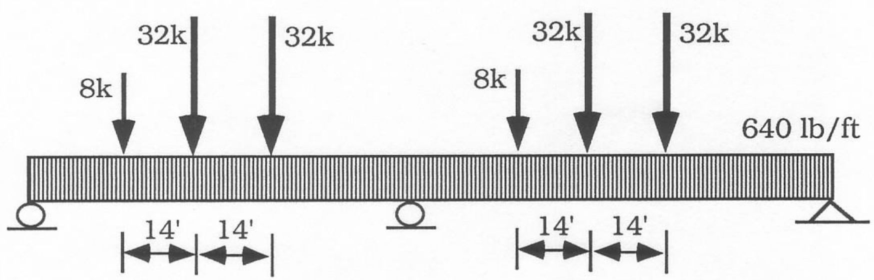


Fig. 5. Design Live Load in LRFD Code.

Development of Numerical Procedure

- For each cross-section the maximum moment and shear was determined
- For each truck maximum value of live load effect was stored

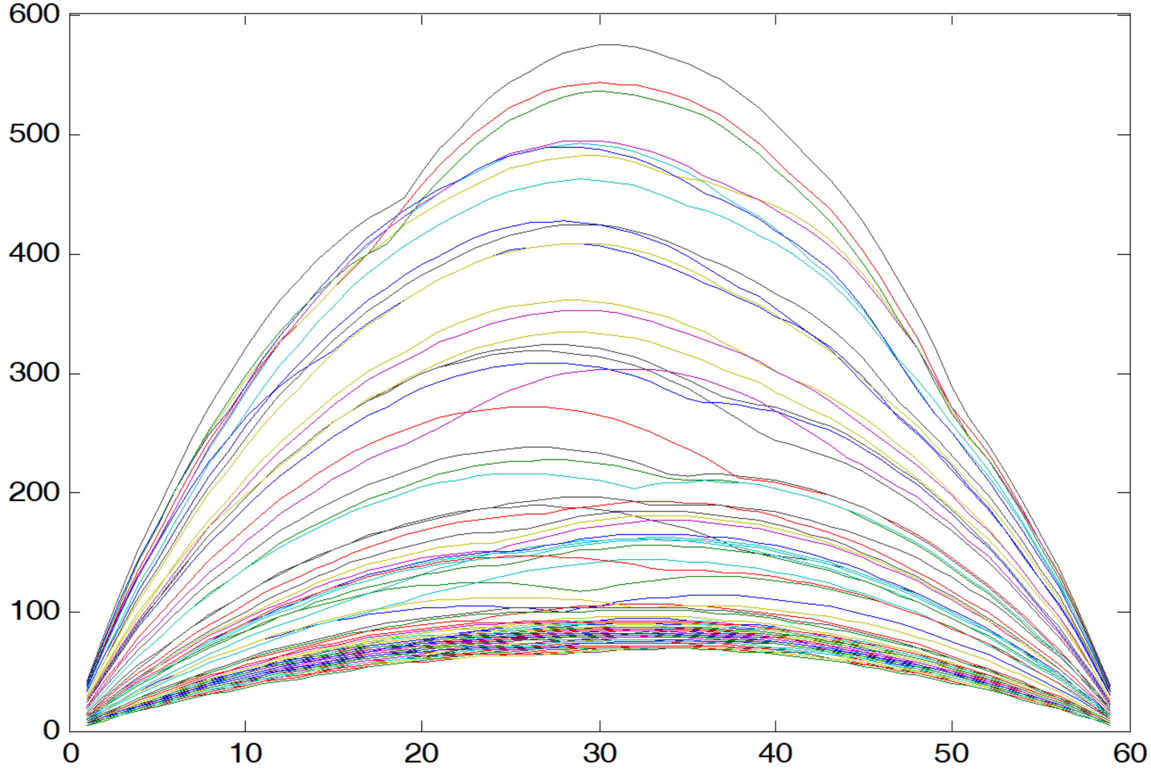
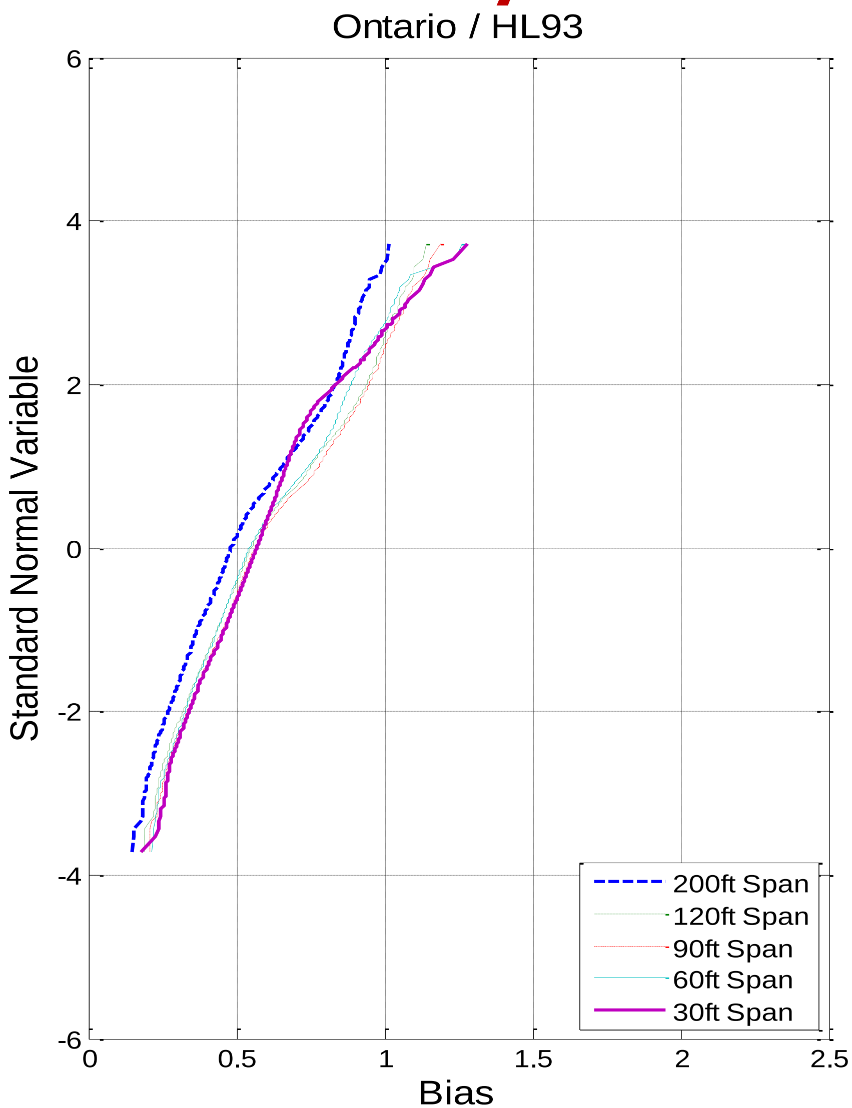


Figure 1 - Bending Moment Envelopes - First 100 Trucks – 60ft Span

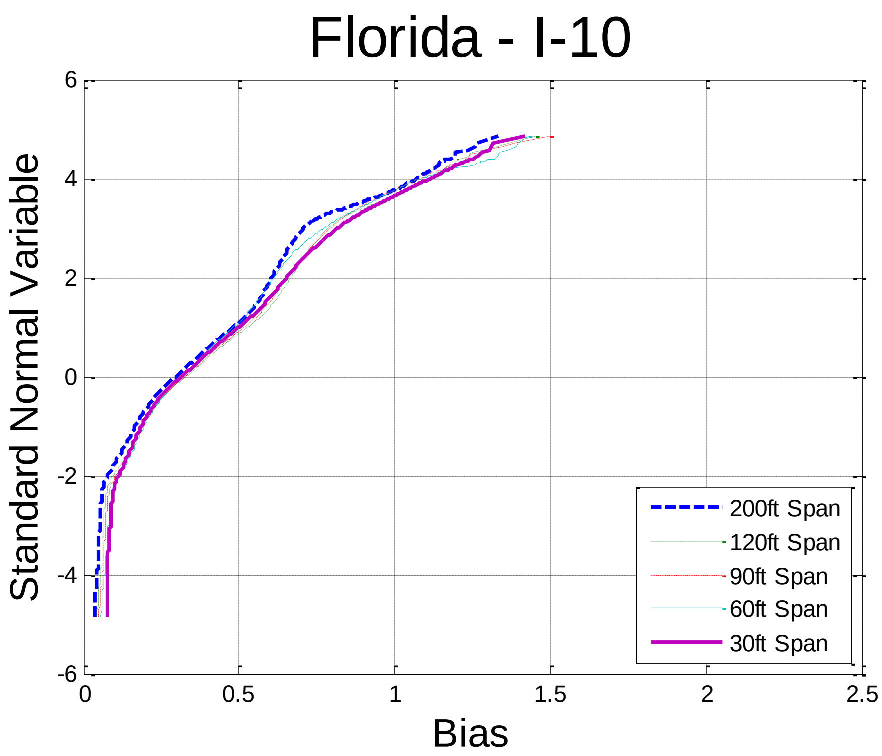
Truck Survey – Ontario 1975



- 9,250 vehicles
- 2 weeks of traffic

Figure 2 – Cumulative Distribution Functions of Ratio of Truck Moment/ HL93Moment - Simple Span Moment – Ontario

Florida – Load Effect – Moments



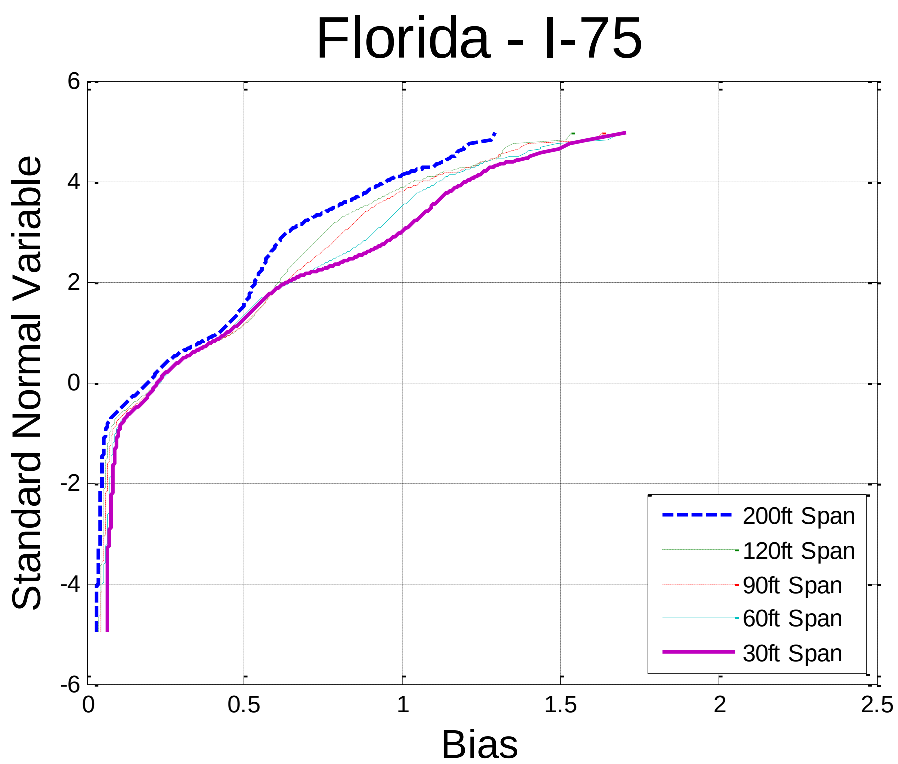
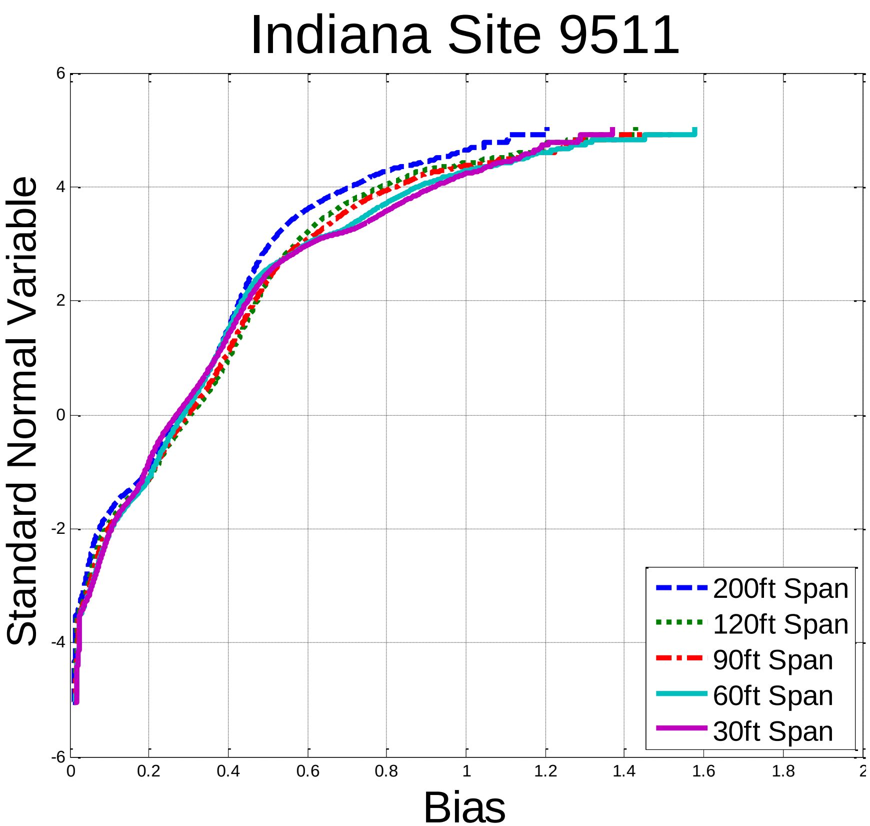


Figure 3 – Cumulative Distribution Functions of Ratio of Truck Moment/ HL93Moment - Simple Span Moment – Florida

Indiana – Load Effect – Moments



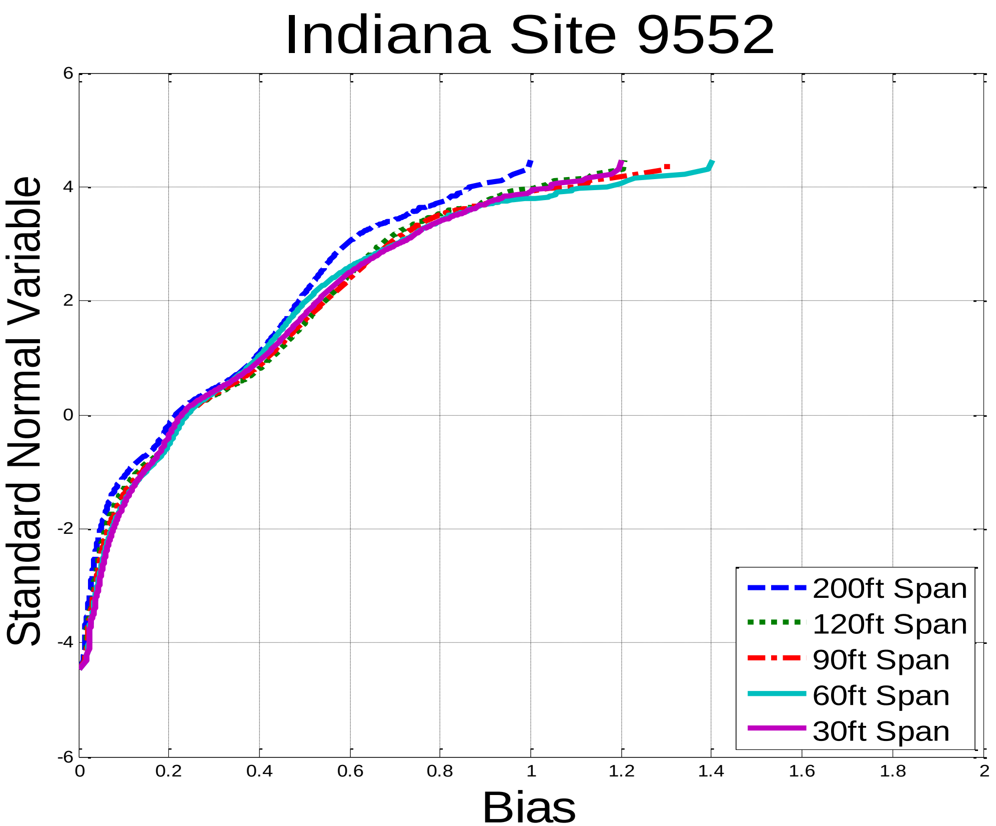


Figure 4 – Cumulative Distribution Functions of Ratio of Truck Moment/ HL93Moment - Simple Span Moment – Indiana

Mississippi – Load Effect – Moments

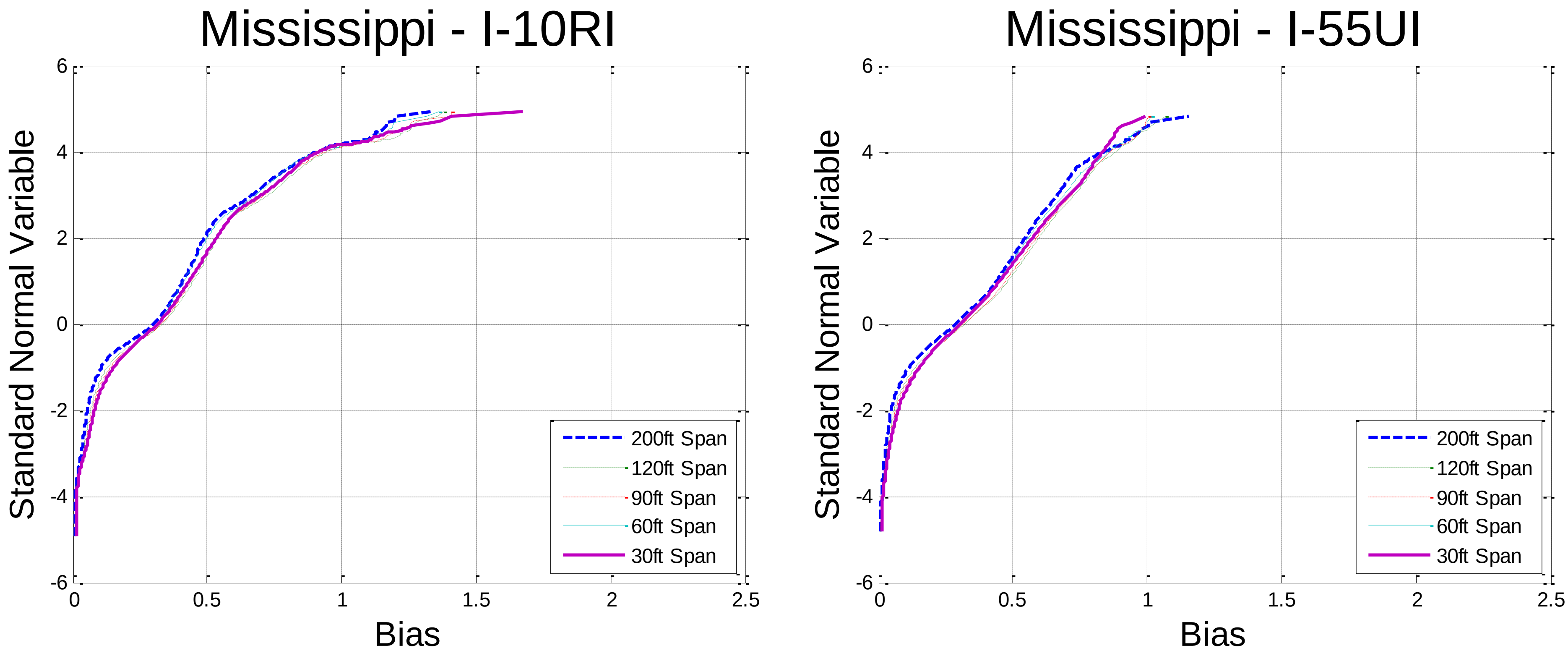


Figure 5 – Cumulative Distribution Functions of Ratio of Truck Moment/ HL93Moment - Simple Span Moment – Mississippi

New York – Load Effect – Moments

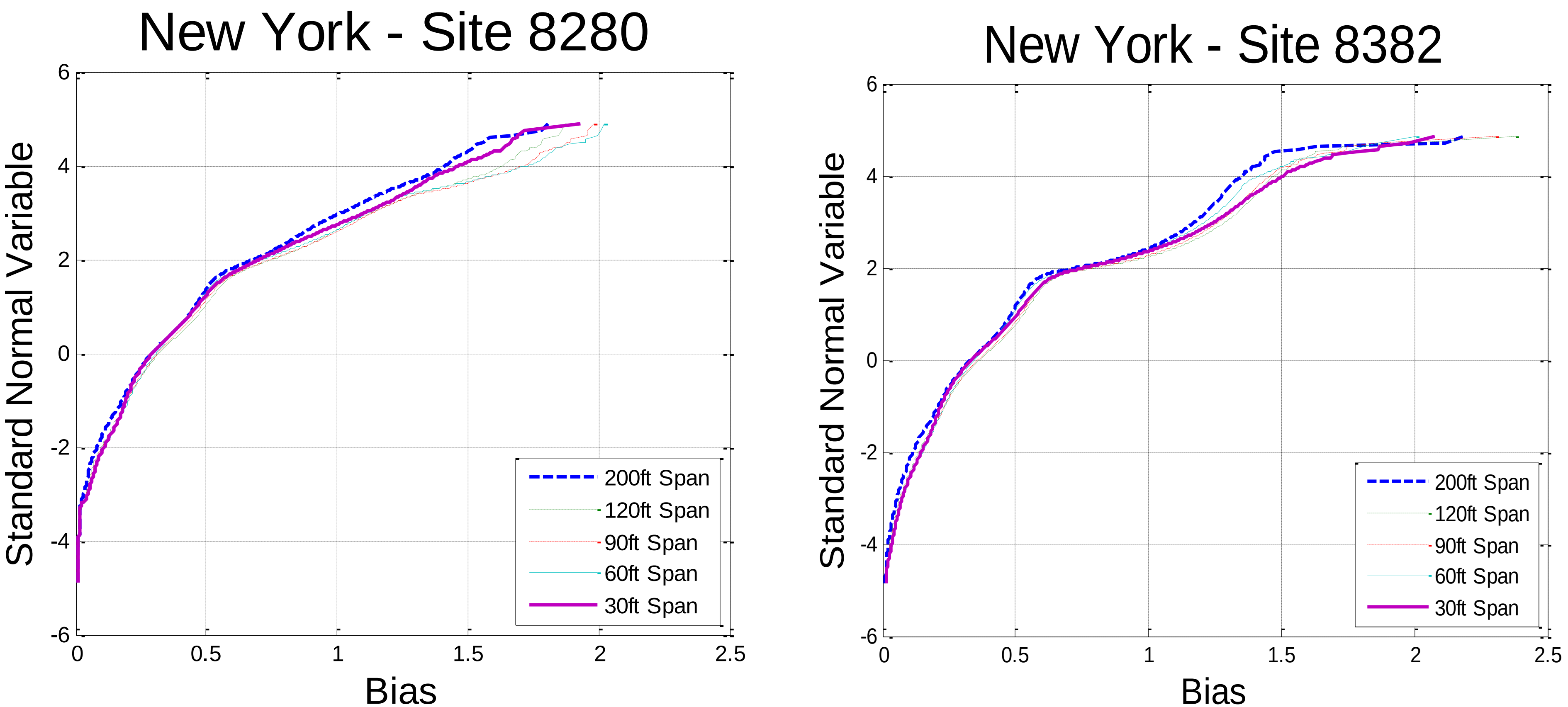
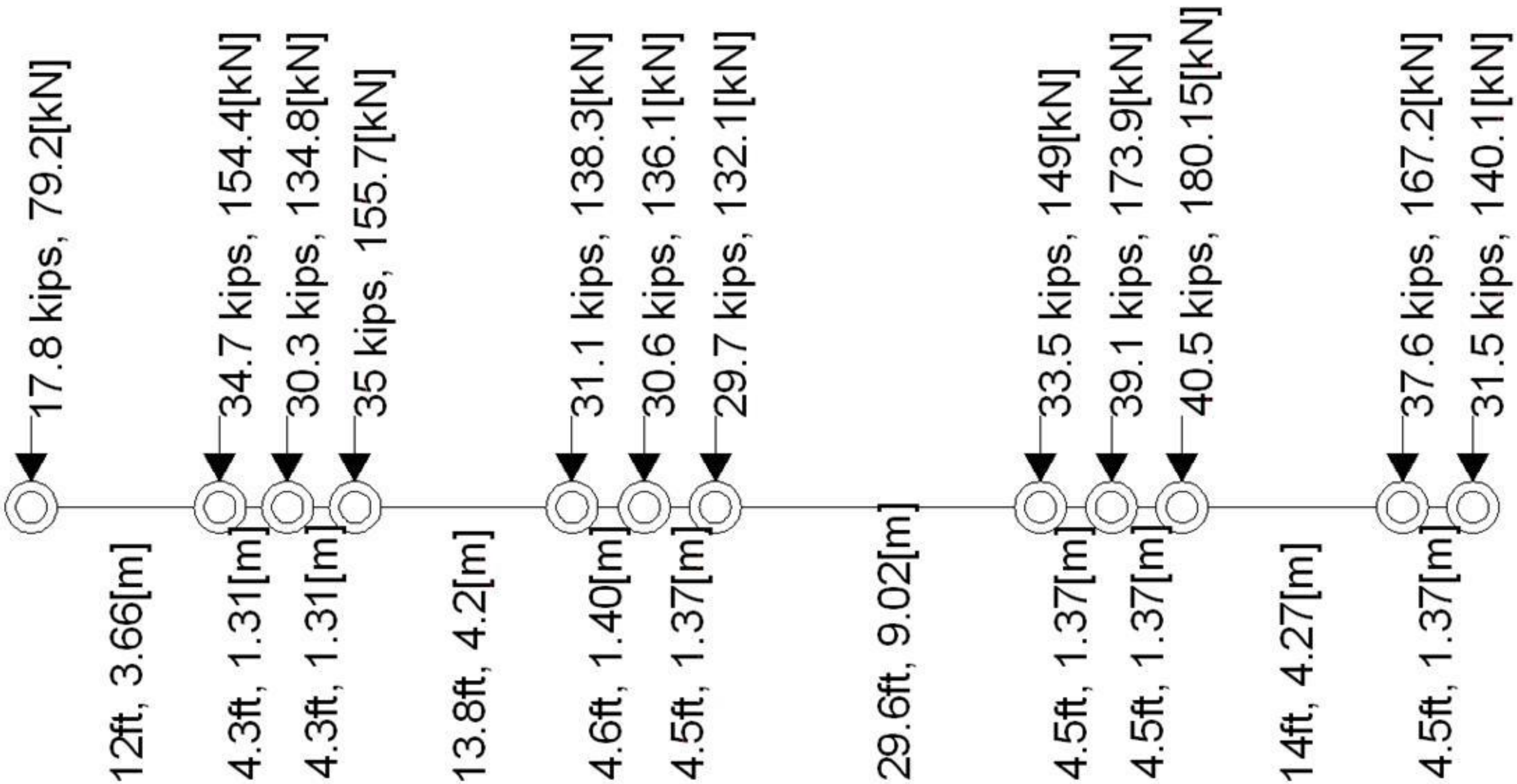
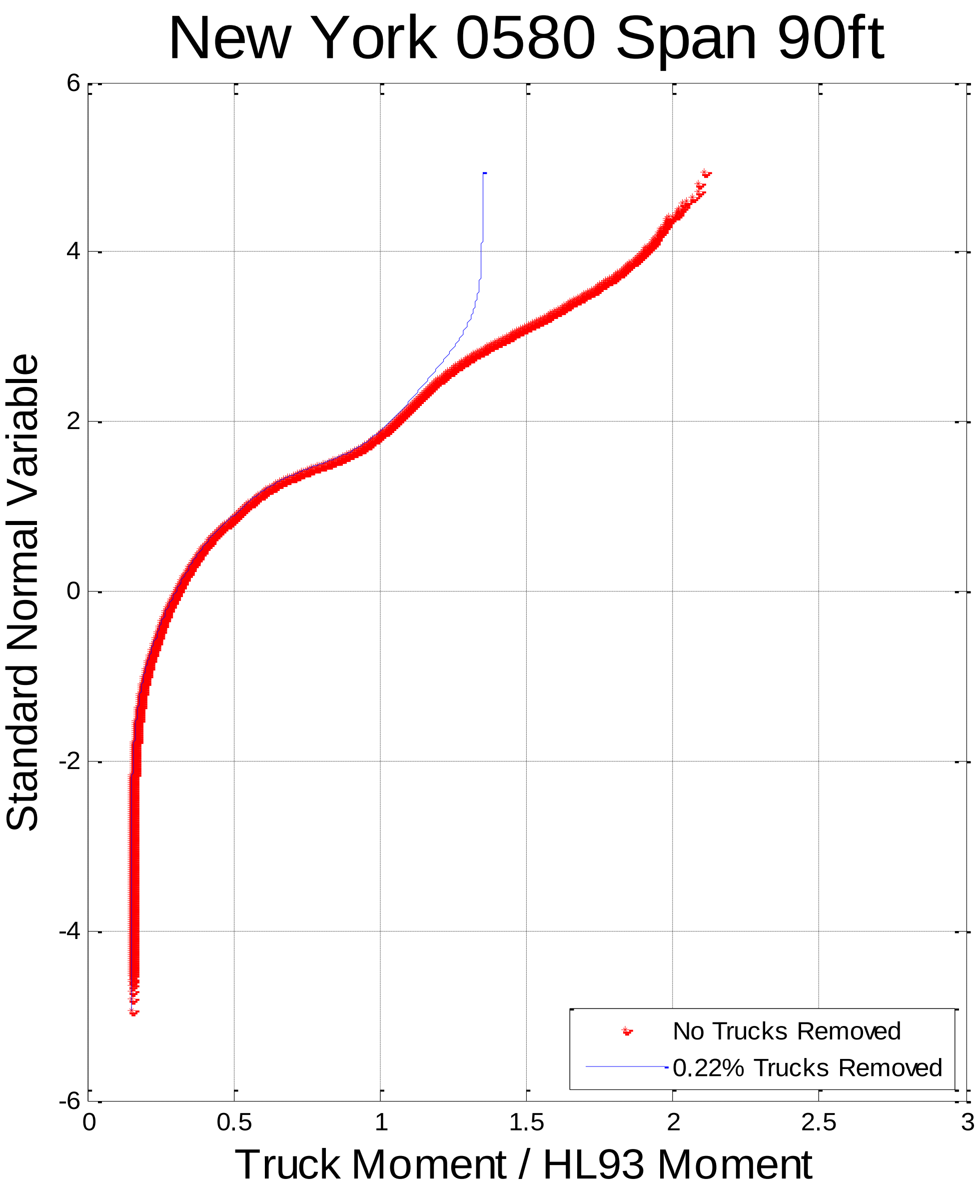


Figure 6 – Cumulative Distribution Functions of Ratio of Truck Moment/ HL93Moment - Simple Span Moment – New York

Configuration of the Heaviest Truck – New York 8382

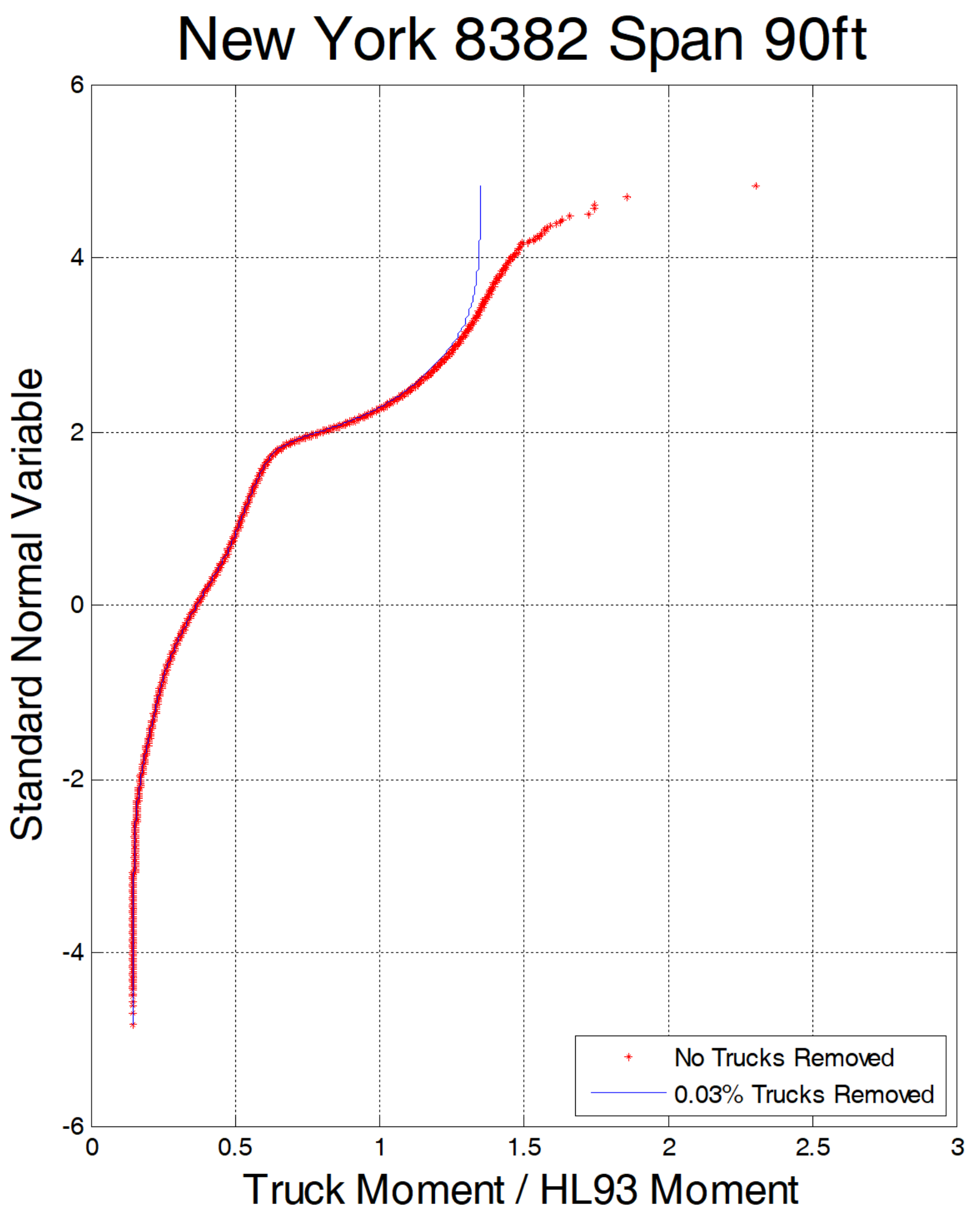


New York Extremely Heavy Trucks



- Number of trucks: 2,474,407
  - Additional filter:
  - $M_{truck}/M_{HL93}>1.35$
    - 5455 trucks removed

New York Extremely Heavy Trucks



- Number of trucks: 1,594,674
  - Additional filter:
  - $M_{truck}/M_{HL93}>1.35$
    - 540 trucks removed

Site Specific Live Load Analysis

- Prediction of maximum 75 year load effect
- The live load data cannot be approximated with any type of distribution
- Distribution-free methods
- Kernel density estimation allows to estimate the PDF for the whole data set

Extreme Value Analysis

- In the sample with a given size n independent observations, maximum values are:

$$M_n = \max(X_1, \ldots, X_n)$$

where: $X_1,\ldots,X_n$ is a sequence of independent random variables having the same distribution function F(x).

Extreme Value Analysis

- Assuming that n is the number of observations and X_1, X_2, X_3,…, X_n are independent, and identically distributed, then:

$$F_{X_1}(x) = F_{X_2}(x) = \ldots = F_{X_n}(x) = F_X(x)$$

- Observing that M_n is less than the particular maximum value m then all the variables (X_1, …, X_n) are less than m.

Extreme Value Analysis

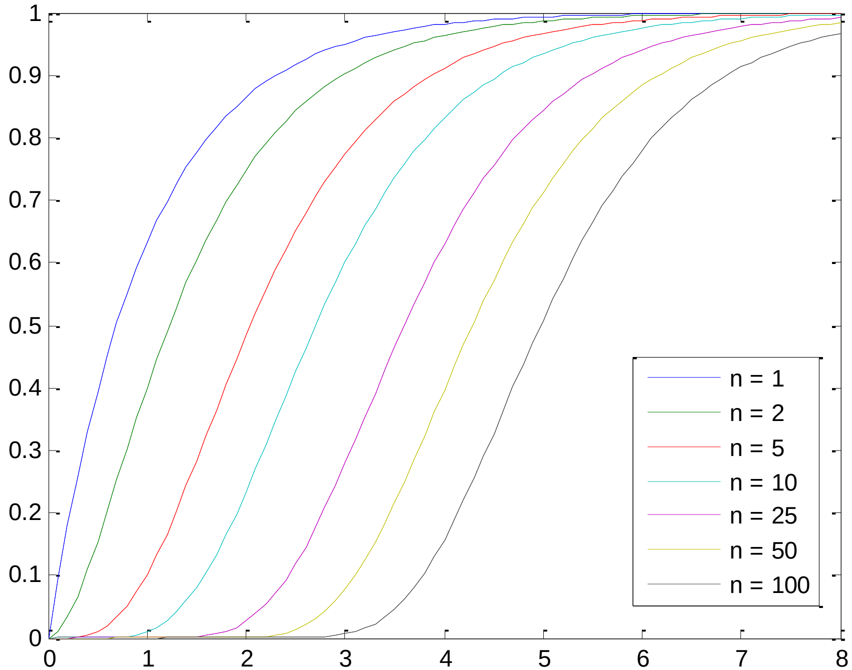
- The cumulative distribution function of X_n can be represented as:

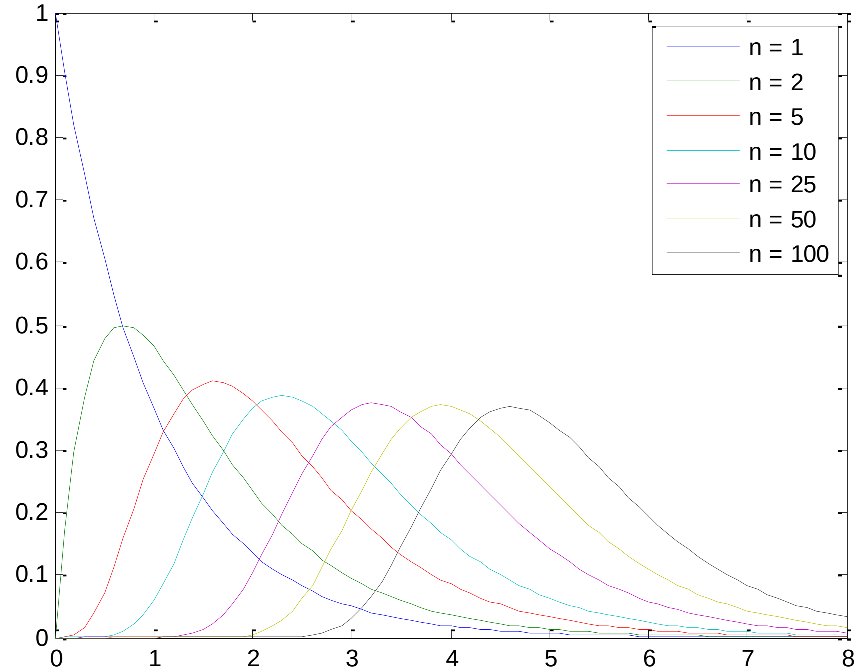
$$F_{M_n}(m) = F_X(m)^n$$

- and the probability density function $f_{M_n}(m)$:

$$f_n(m) = nF(m)^{n-1} f(m)$$

Graphical representation of CDF and PDF – variable X with exponential PDF

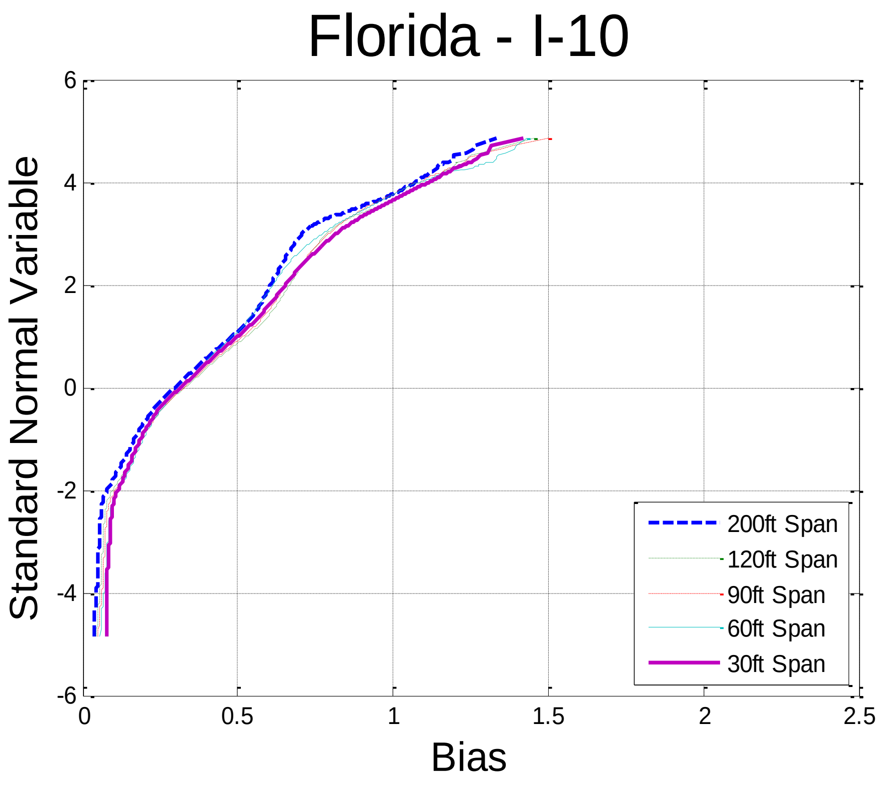




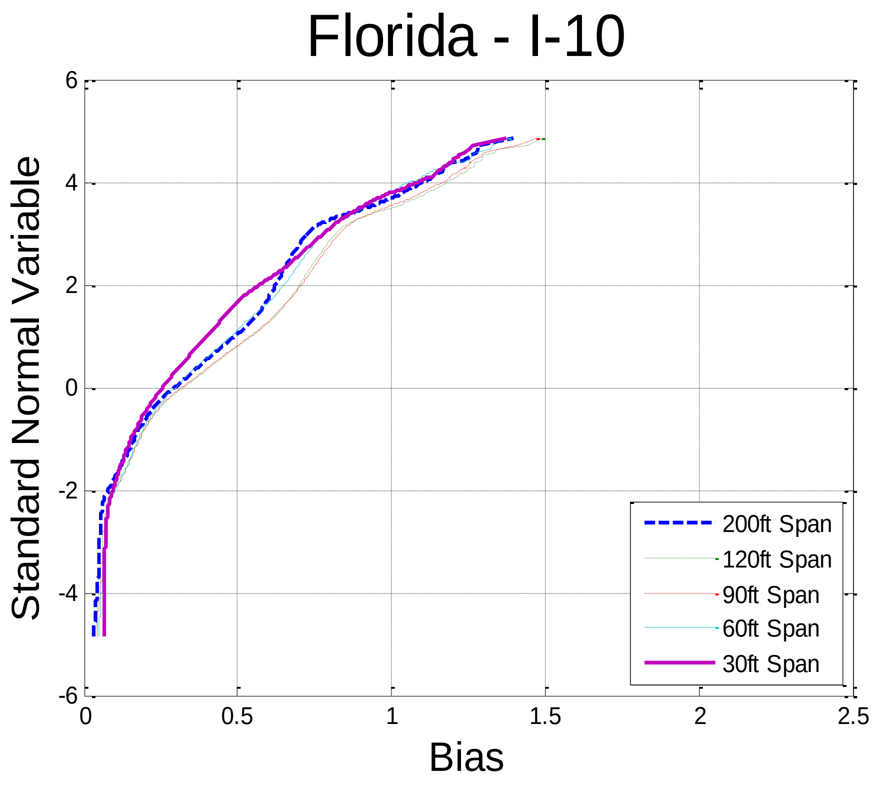
Statistical Models for Live Load - Florida

- Load Effect

Moment



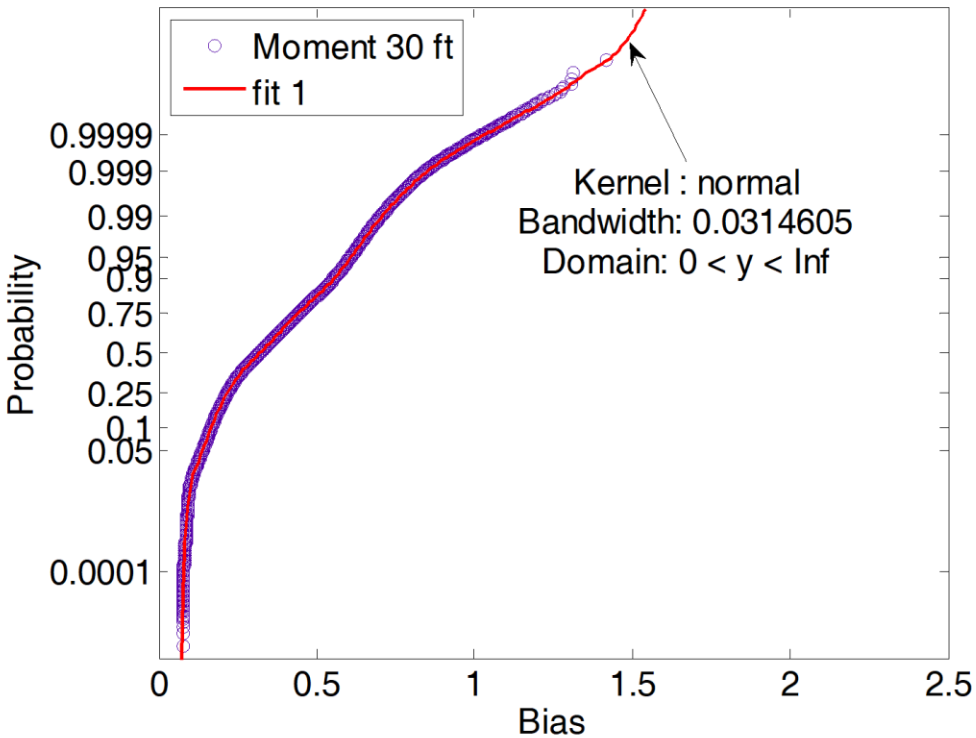
Shear

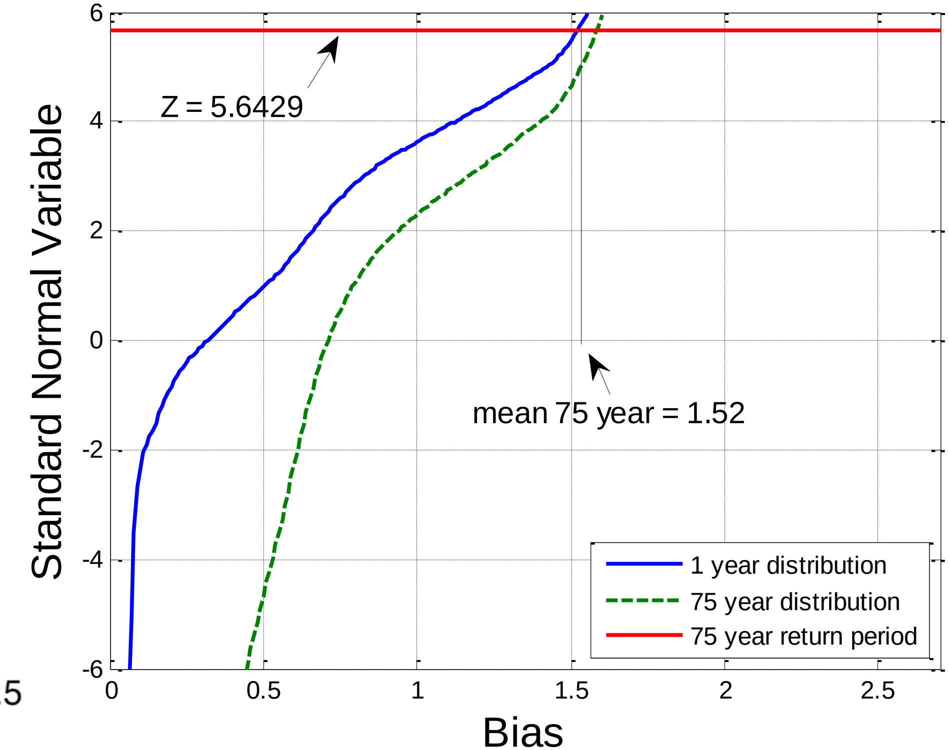


Florida - Number of Trucks with Corresponding Probability and Time Period

Time period	Number of trucks, N	Probability, 1/N	Inverse normal, z
1 month	137,834	7.26E-06	4.34
2 months	275,668	3.63E-06	4.49
6 months	827,003	1.21E-06	4.71
1 year	1,654,006	6.05E-07	4.85
5 years	8,270,030	1.21E-07	5.16
50 years	82,700,300	1.21E-08	5.58
75 years	124,050,450	8.06E-09	5.65

Florida – Extrapolation to 75 Year Return Period





Live Load - Statistical Parameters

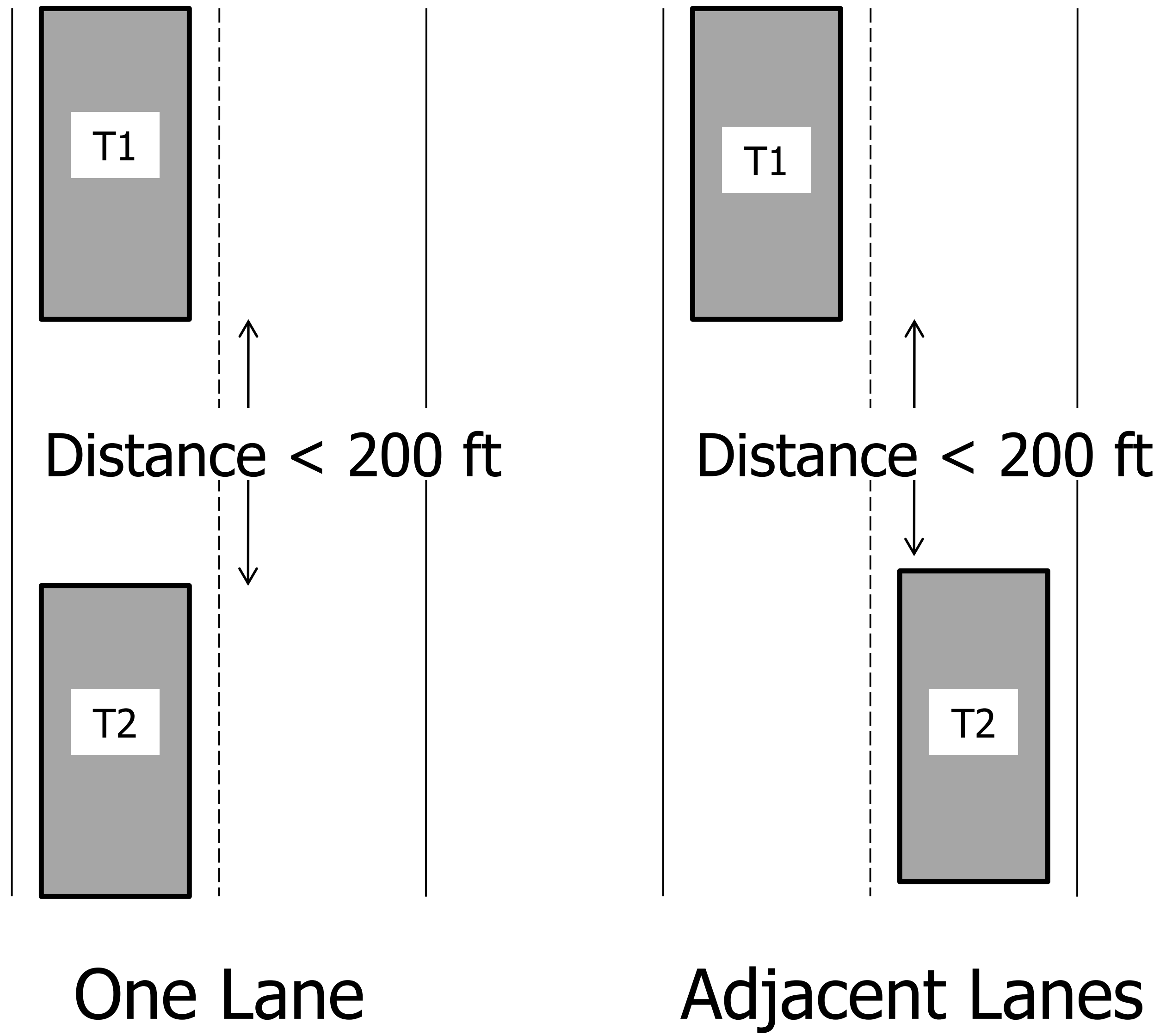
Moment

Span (ft)	1 year	75 years	CoV for 75 year
30	1.42	1.52	0.16
60	1.43	1.53	0.14
90	1.50	1.61	0.16
120	1.46	1.57	0.16
200	1.33	1.43	0.17

Shear

Span (ft)	1 year	75 years	CoV for 75 year
30	1.38	1.47	0.17
60	1.38	1.47	0.15
90	1.47	1.59	0.16
120	1.49	1.61	0.15
200	1.40	1.49	0.18

Simultaneous occurrence of trucks on the bridge and degree of correlation



Coefficient of Correlation

- HL93 load model (NCHRP Report 368, Nowak 1999) was based on the assumption:
  - Multiple Lane - every 500th truck is fully correlated
  - One Lane - every 100th truck is fully correlated

Coefficient of Correlation - Filtering Criteria

- Two trucks have to have the same number of axles

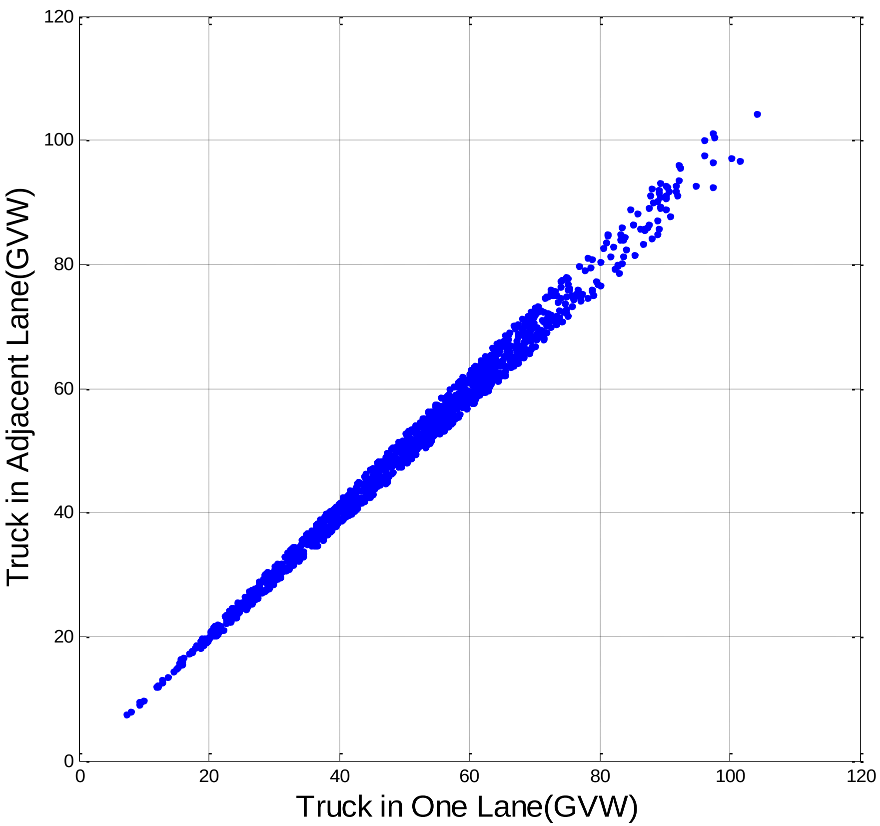
- GVW of the trucks has to be within the +/- 5% limit

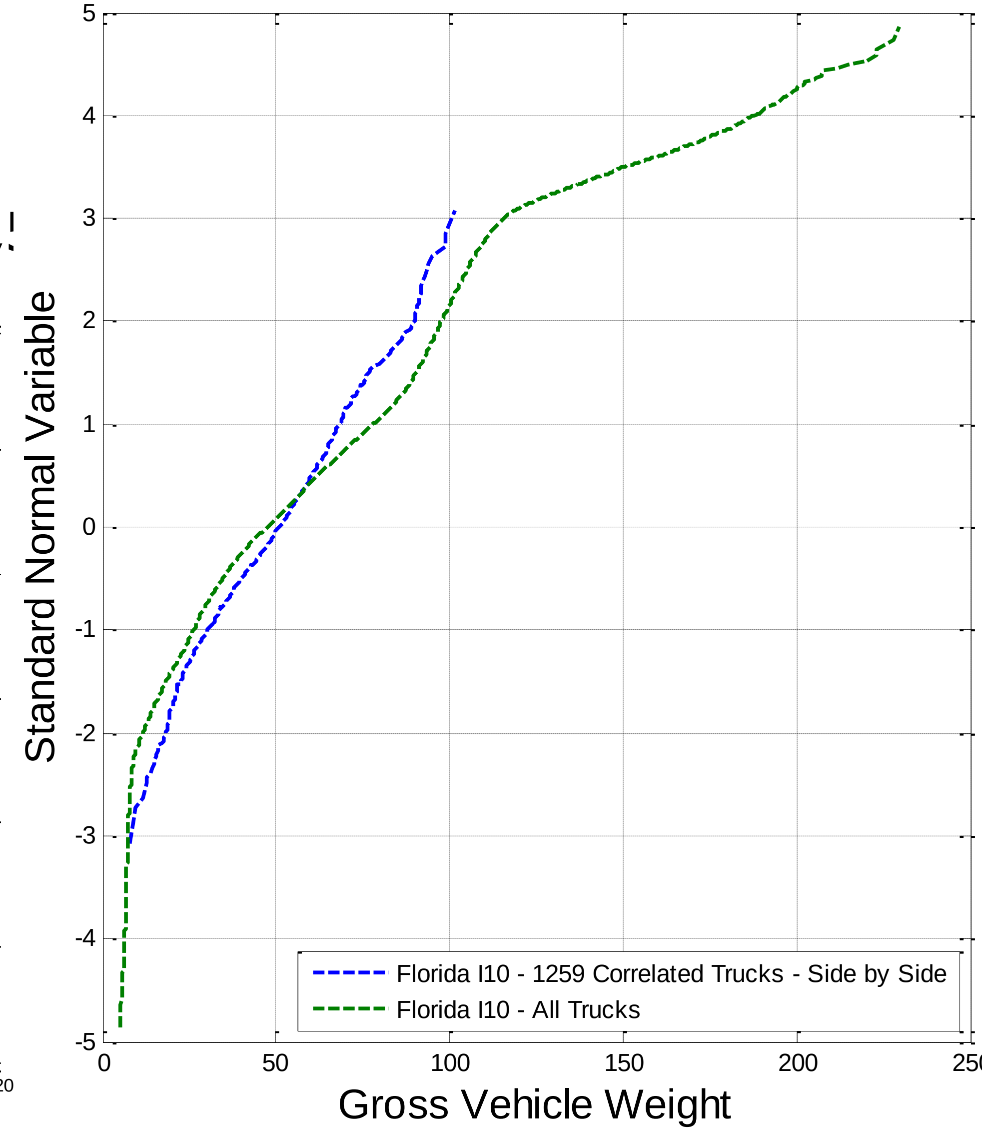
- Spacing between each axle has to be within the +/- 10% limit

Coefficient of Correlation – Florida I-10

1. Adjacent Lanes:

- Number of Trucks : 1,654,004
- Number of Fully Correlated Trucks: 2,!

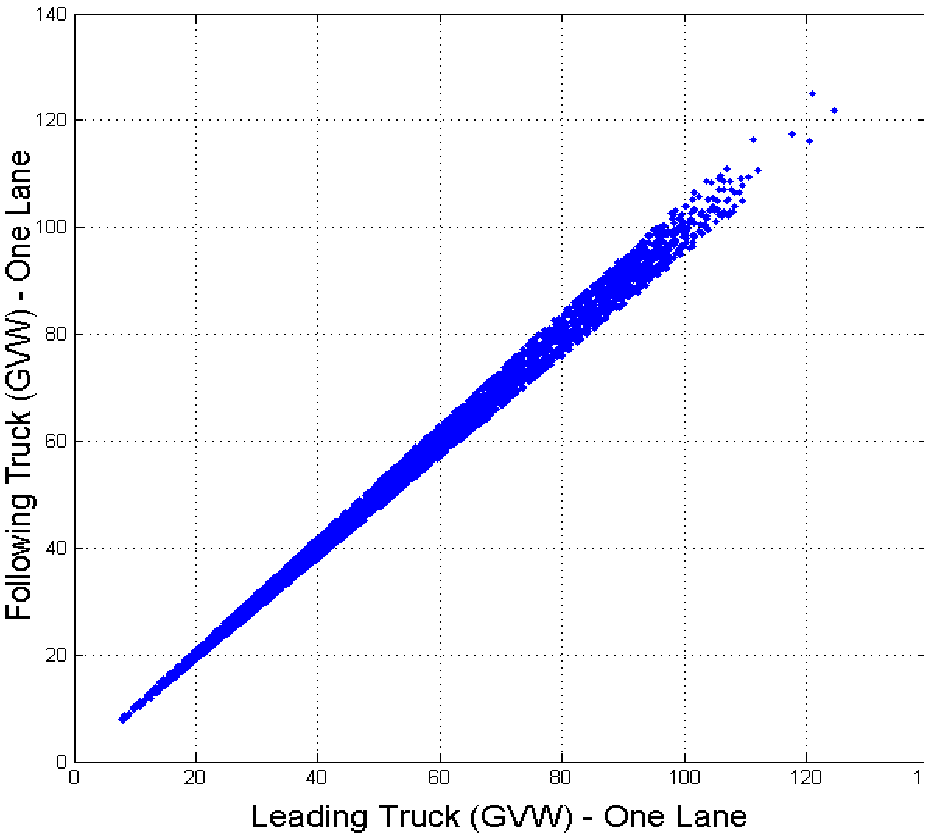


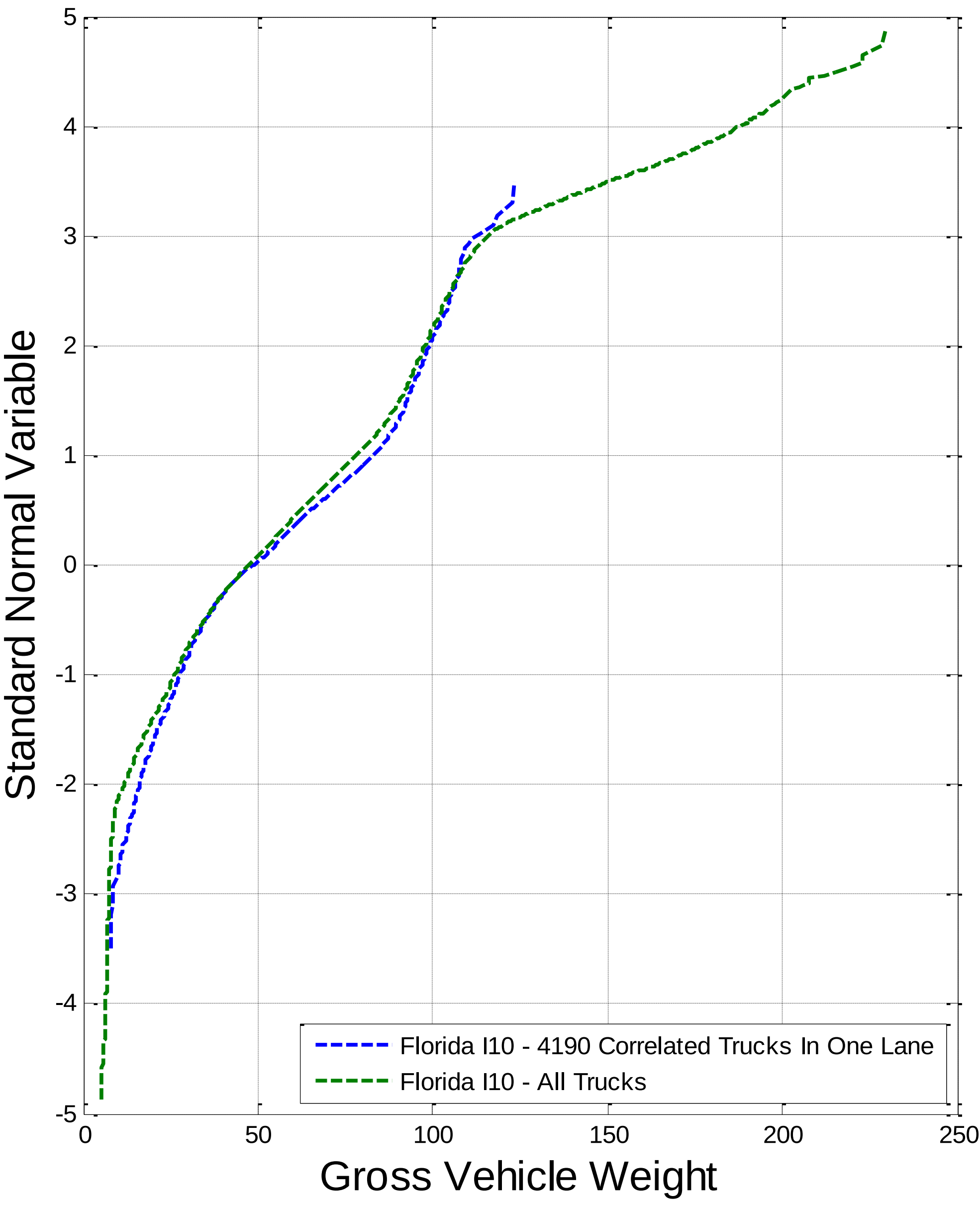


Coefficient of Correlation – Florida I-10

2. One Lane:

- Number of Trucks : 1,654,004
- Number of Fully Correlated Trucks: 8,:





Fully Correlated Trucks

Site	Total Number of Trucks	Adjacent Lane	One Lane
Florida – I10	1,654,004	2,518	8,380
New York - 8382	1,594,674	3,748	9,868

Conclusions

- A correlation analysis performed on the available new WIM data confirmed previous assumption that about every 500th truck is on the bridge simultaneously side-by-side with another fully correlated truck.
- The expected maximum weight of the fully correlated trucks is smaller than the maximum weight of trucks recorded at the same site.

Conclusions

- A load combination with two fully correlated trucks in adjacent lanes does not govern.
- The governing combination is a simultaneous occurrence of the extreme truck and an average truck.
- Filtering of the WIM data is an important issue.
- Quality WIM data from state of New York is needed.

Examples of Bias Factors

Two bridge design codes are considered:

- AASHTO Standard Specifications (1996)
- AASHTO LRFD Code (1998)

For the first one, denoted by HS20, bias factor is non-uniform, so design load in LRFD Code was changed, and the result is much better.

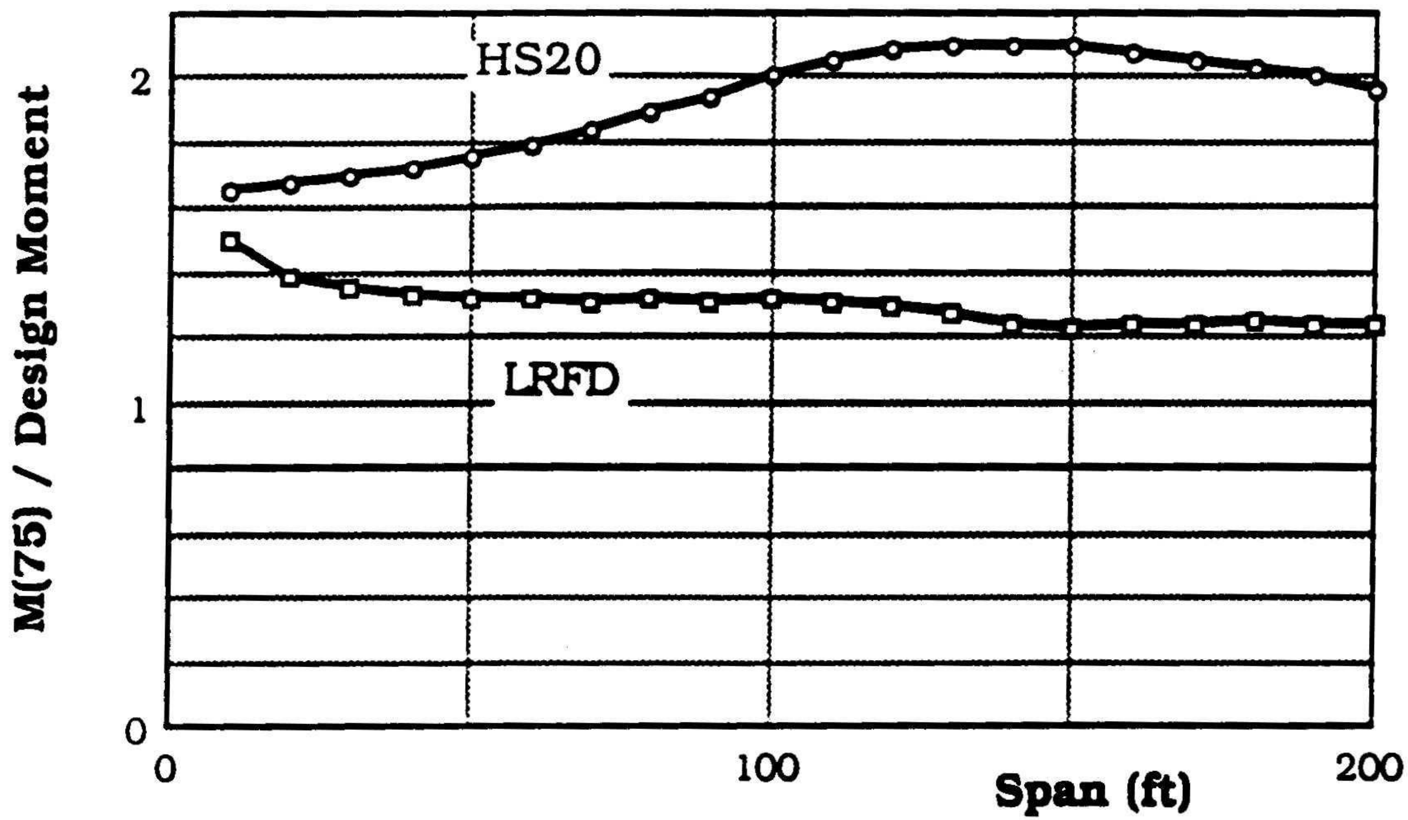


Fig. B-26. Bias Factors for Simple Span Moment; Ratio of M(75)/M(LRFD) and M(75)/M(HS20).

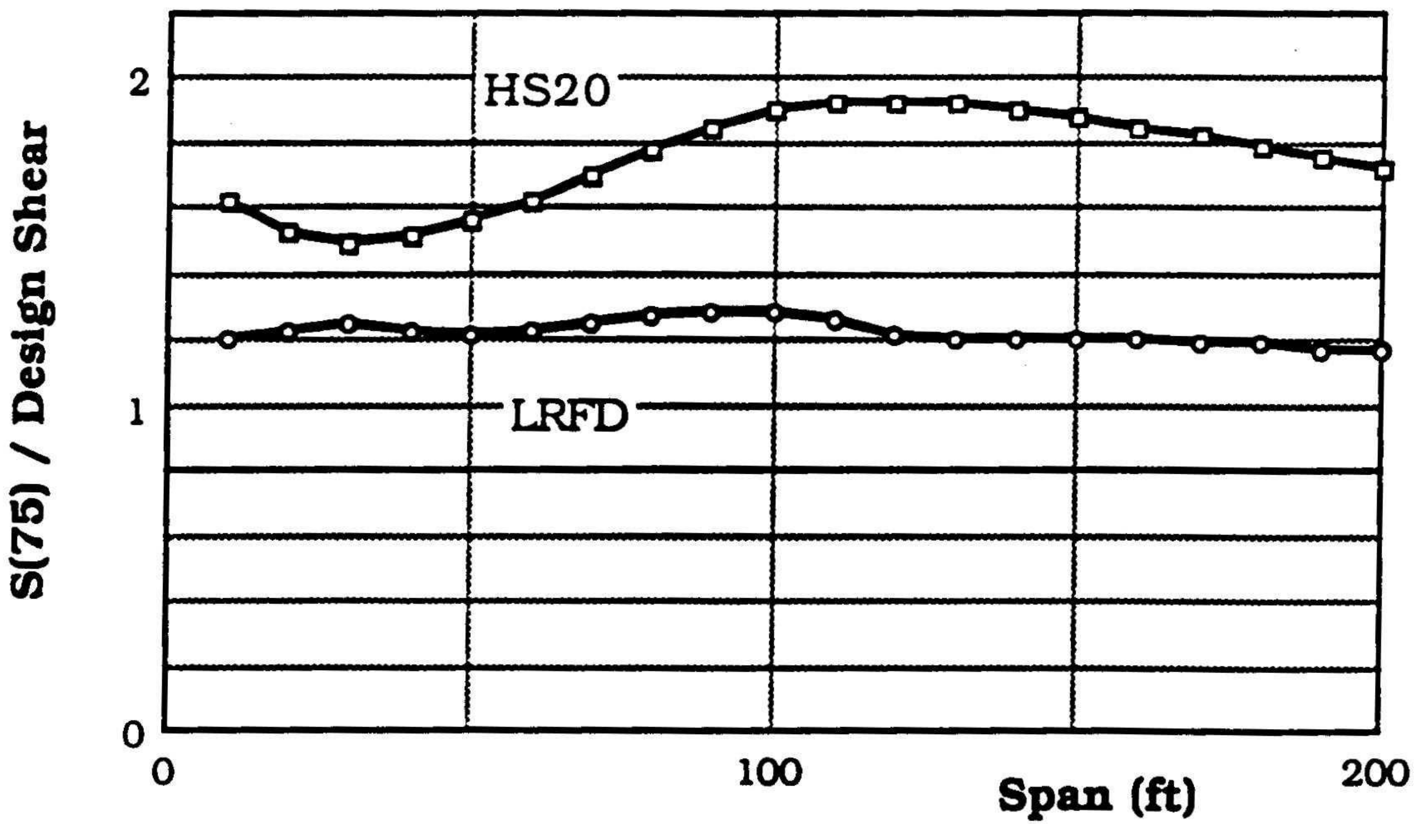


Fig. B-27. Bias Factors for Shear; Ratio of S(75)/S(LRFD) and S(75)/ S(HS20).

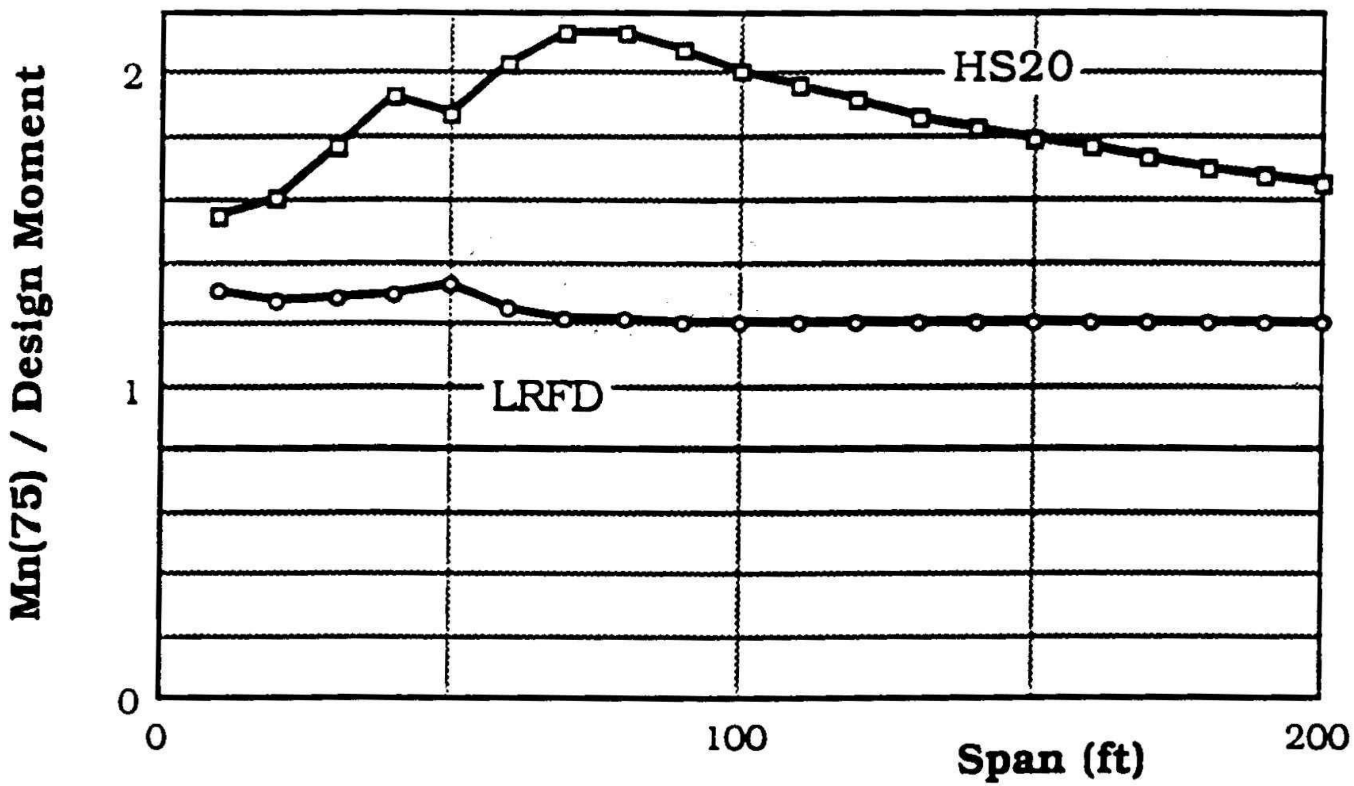
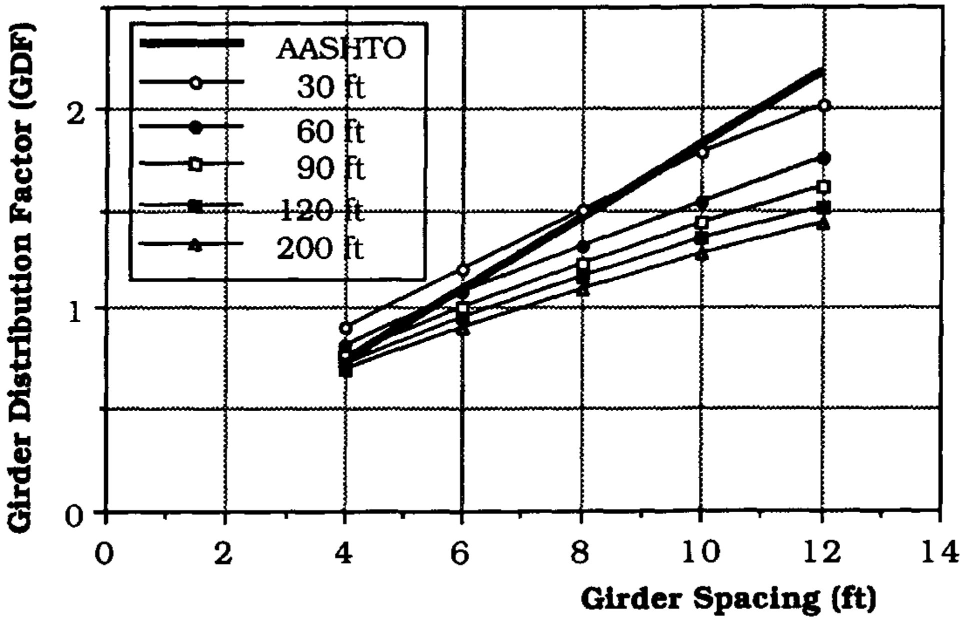


Fig. B-28. Bias Factors for Negative Moment; Ratio of Mn(75)/ Mn(LRFD) and Mn(75)/Mn(HS20).

Girder Distribution Factor

Girder Distribution Factors calculated and specified in AASHTO (1992)



Multilane Live Load for various ADTT

- ADTT (average daily truck traffic) is an important parameter of live load. The live load moments for multilane bridges with various ADTT's are derived by simulations.
- The moment ratios determined by simulations are listed below:

ADTT (in one direction)	Number of lanes 1	2	3	4
100	0.95	0.80	0.55	0.45
1,000	1.00	0.85	0.70	0.50
5,000	1.05	0.90	0.75	0.55

Dynamic Load

- Roughness of the road surface (pavement)
- Bridge as a dynamic system (natural frequency of vibration)
- Dynamic parameters of the vehicle (suspension system, shock absorbers)

Dynamic Load Factor (DLF)

- Static strain or deflection (at crawling speed)
- Maximum strain or deflection (normal speed)
- Dynamic strain or deflection =

  maximum - static
- DLF = dynamic / static

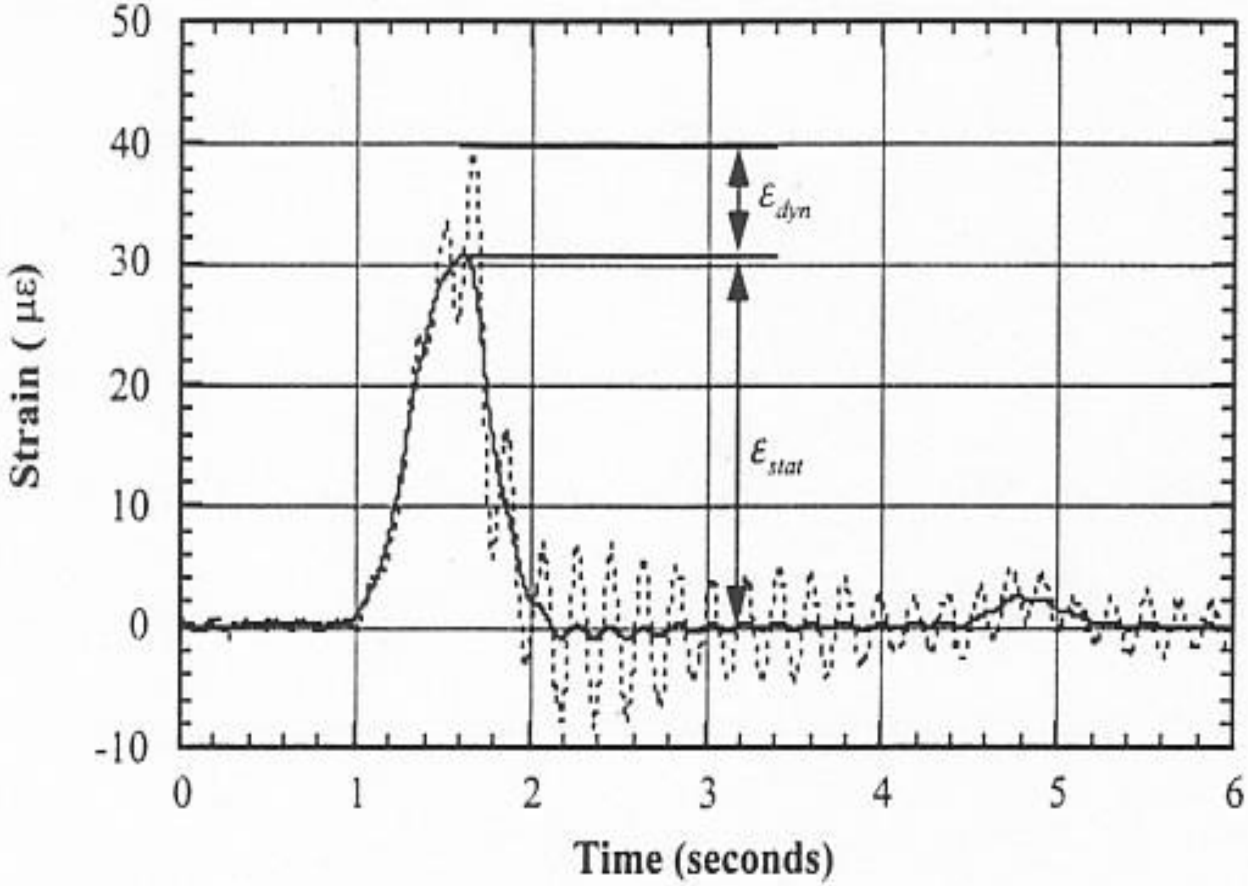


Figure 3.3. Dynamic and Static Strain under a Truck at Highway Speed.

Code Specified Dynamic Load Factor

AASHTO Standard (1996)

$$I = \frac{50}{3.28L + 125} \le 0.3$$

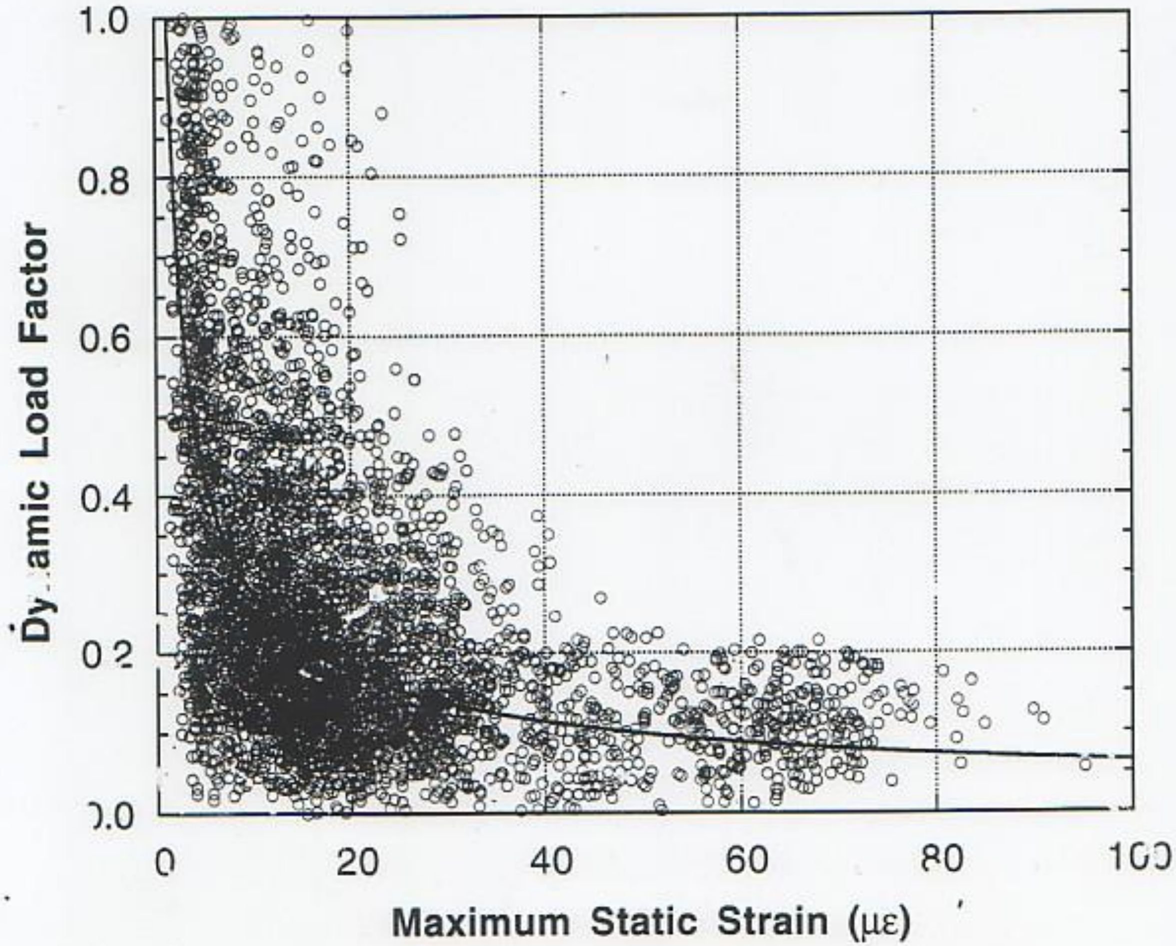
AASHTO LRFD (1998)

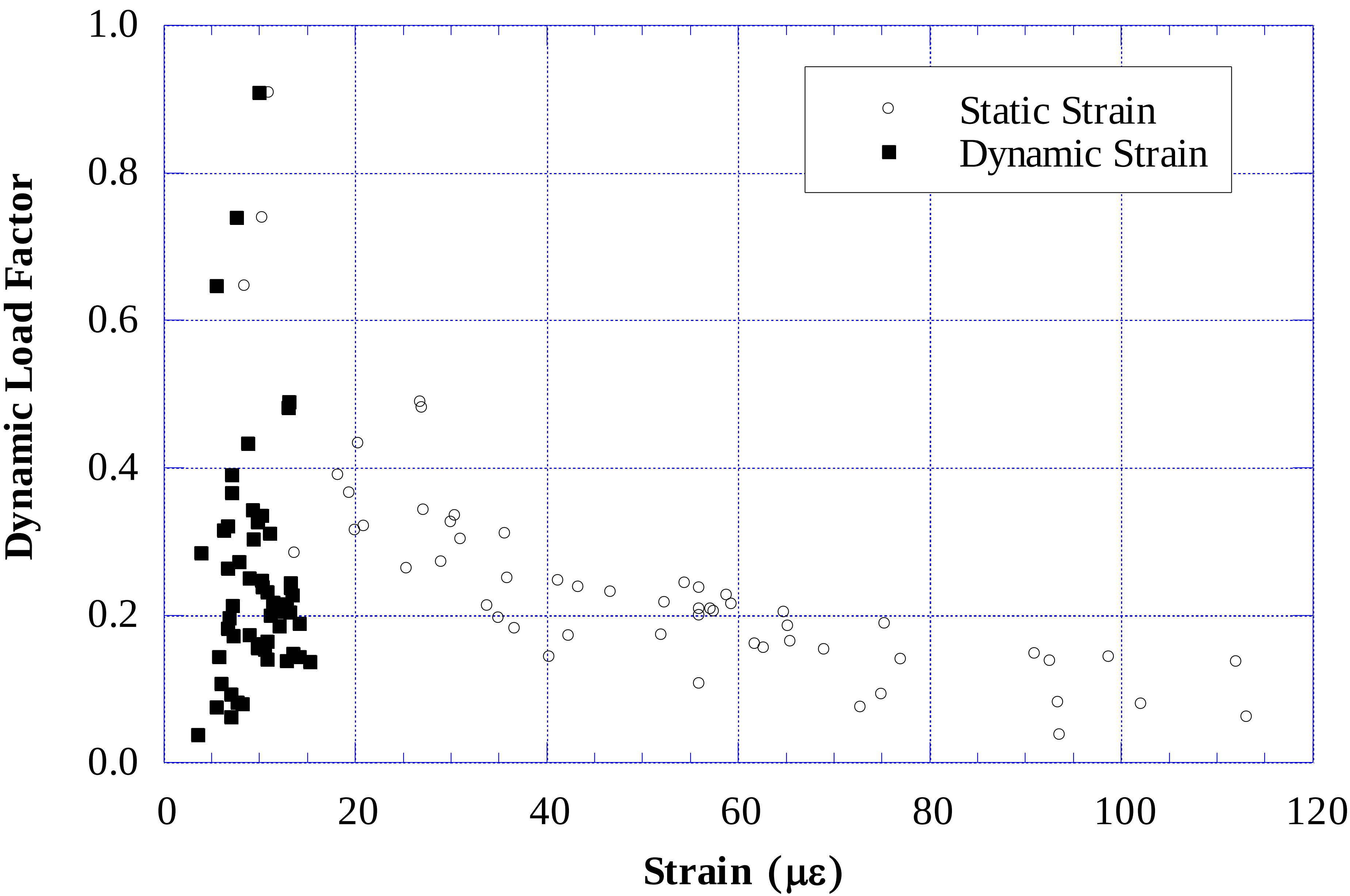
0.33 of truck effect, no dynamic load for the uniform loading

Dynamic Load

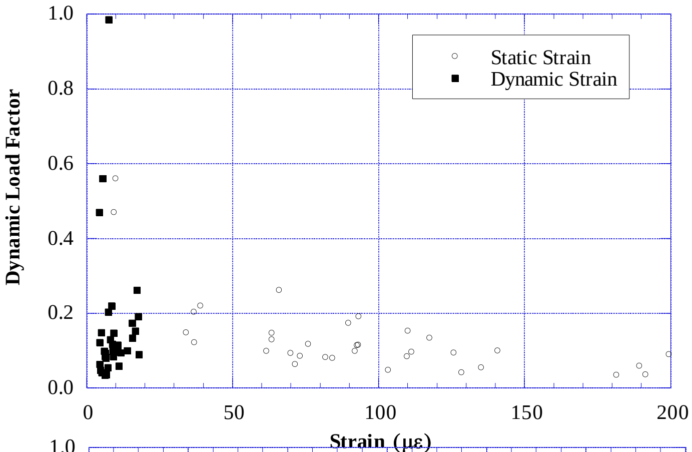
- The dynamic load model is a function of three major parameters
  - Road surface roughness
  - Bridge dynamics (frequency of vibration)
  - Vehicle dynamics (suspension system
- The simulations indicate that DLF values are almost equally dependent on all of three major parameters. The parameters vary from site to site and they are very difficult to predict.
- It was observed that dynamic deflection is almost constant and does not depend on truck weight. Therefore, the dynamic load, as a fraction of live load, decreases for heavier trucks.
- For the maximum 75 years values, corresponding dynamic load does not exceed 17% of live load for a single truck and 10% of live load for two trucks side-by-side. The coefficient of variation is about 0.80.

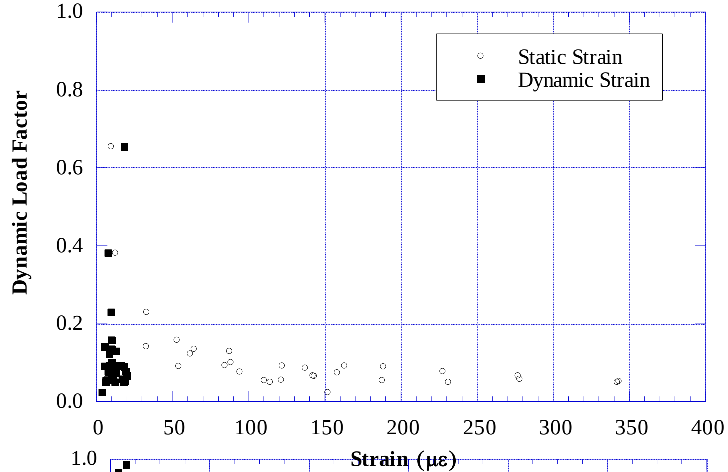
Dynamic Load Factor

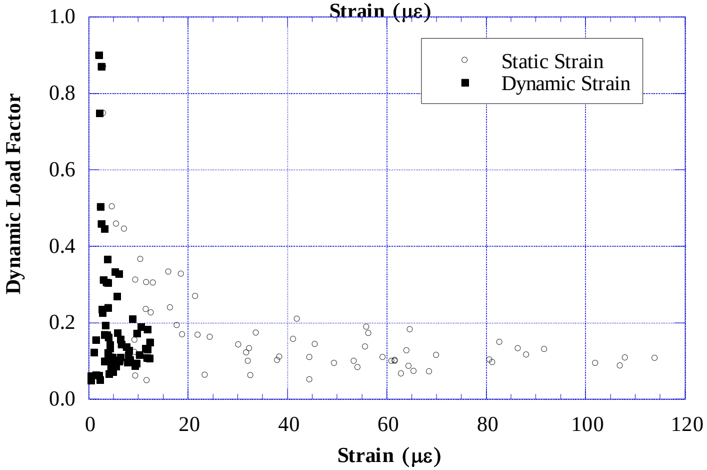


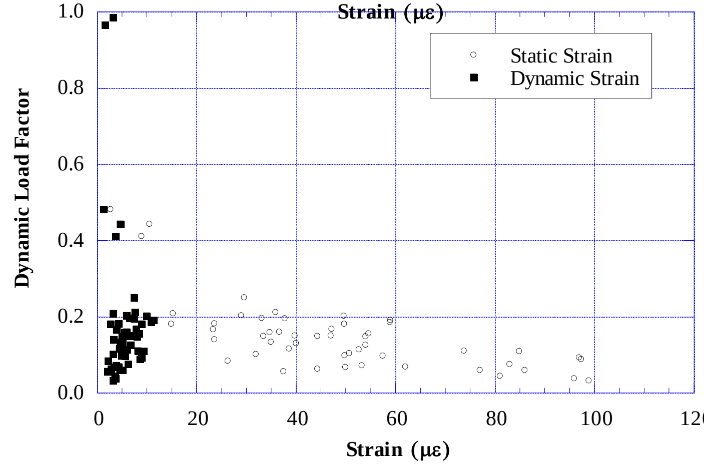


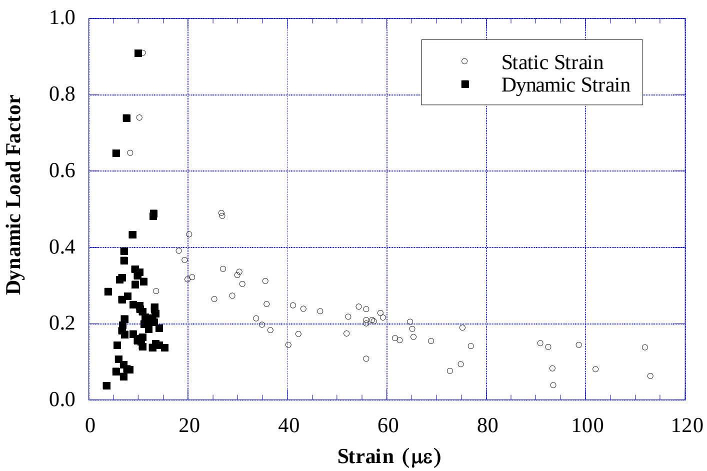
Dynamic Load Factor
1.0
0.8
0.6
0.4
0.2
0.0
0
20
40
60
80
100
120
Strain (με)
Static Strain
Dynamic Strain

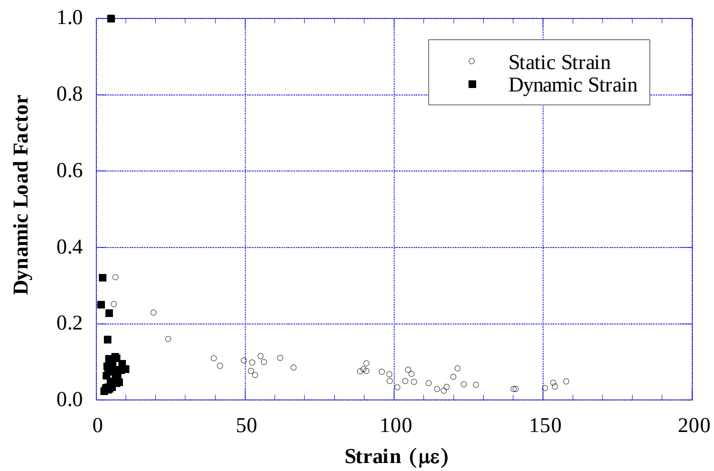

Static Strain
Dynamic Strain
Dynamic Load Factor
Strain (με)
0.0 0.2 0.4 0.6 0.8 1.0
0 50 100 150 200


Static Strain
Dynamic Strain
Dynamic Load Factor
Strain (με)
0.0 0.2 0.4 0.6 0.8 1.0
0 50 100 150 200 250 300 350 400


Static Strain
Dynamic Strain
Dynamic Load Factor
Strain (με)
0.0 0.2 0.4 0.6 0.8 1.0
0 20 40 60 80 100 120


Static Strain
Dynamic Strain
Dynamic Load Factor
Strain (με)
0.0 0.2 0.4 0.6 0.8 1.0
0 20 40 60 80 100 120


Static Strain
Dynamic Strain
Dynamic Load Factor
Strain (με)
0.0 0.2 0.4 0.6 0.8 1.0
0 20 40 60 80 100 120


Static Strain
Dynamic Strain
Dynamic Load Factor
Strain (με)
0.0 0.2 0.4 0.6 0.8 1.0
0 50 100 150 200

Dynamic Load Conclusions

- Dynamic strain and deflection do not depend on truck weight
- Dynamic load factor (DLF) decreases for increased truck weight
- For a single truck DLF < 20%
- For two trucks side-by-side DLF < 10%

Live Load for Long Span Bridges

Outline

- Truck Survey
- Traffic patterns
- Proposed live load model

Statement of the Problem

- Live load in the AASHTO LRFD Code was calibrated for spans up to 200-300 ft. What about longer spans?

- Multiple-presence on long span multilane bridges?

- Dynamic load for long spans?

Live Load for Long Spans

- Data base – WIM surveys, video recordings, observations (e.g. toll booth operators, maintenance staff)
- Dense traffic (moving at regular speed, with headway distance)
- Traffic jams (bumper-to-bumper)

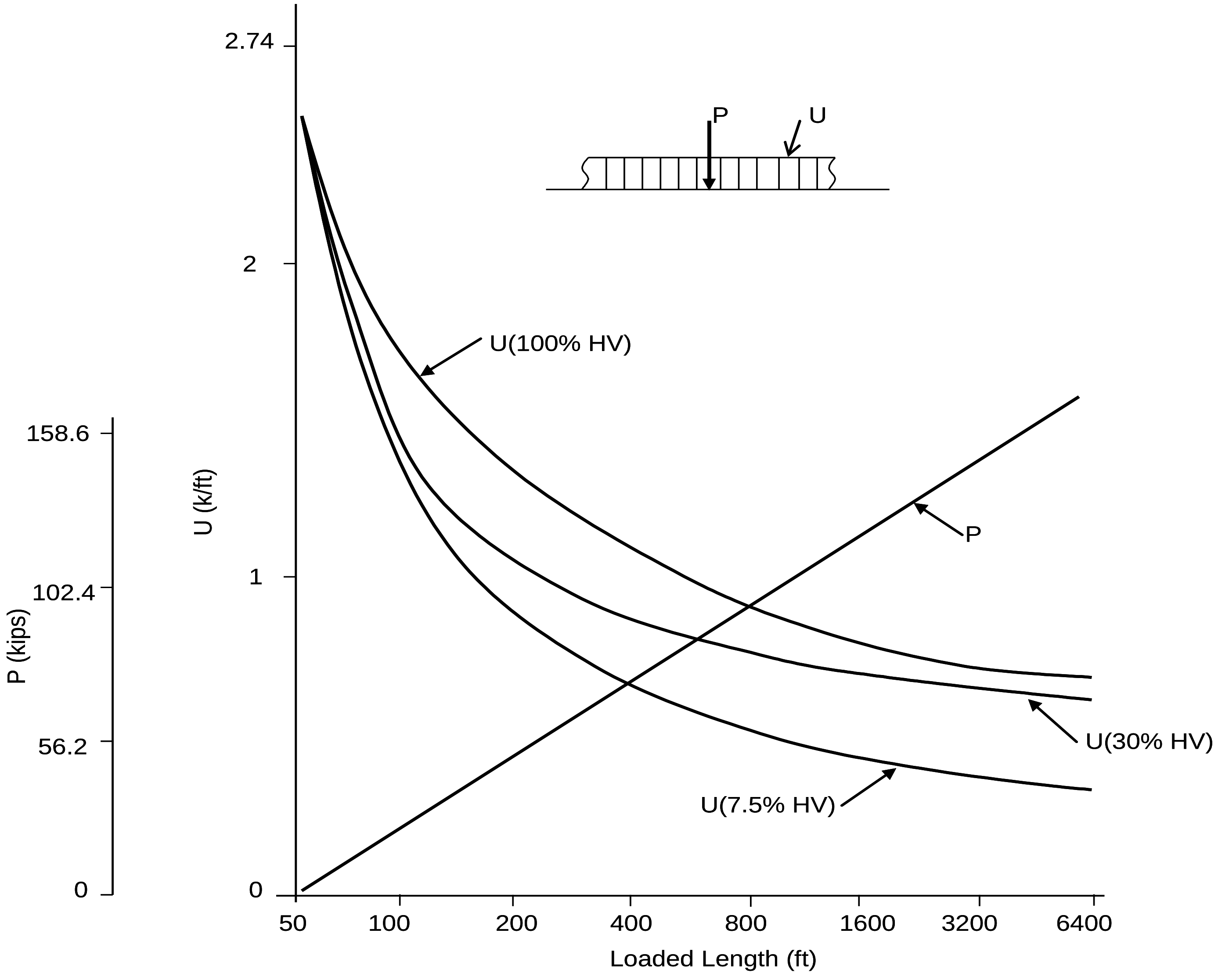
Live Load Parameters

- Weight of trucks
- Traffic volume (ADT, ADTT)
- Multiple presence in lane and in adjacent lanes, traffic patterns
- Correlation between trucks (lack of data)

Live Load per Lane – Long Spans

- Equivalent load per linear foot (kips/ft)
- AASHTO Standard (640 lb/ft plus a concentrated force) (HS-24)
- AASHTO LRFD (640 lb/ft plus a design truck) (HL-93)
- Ontario Highway Bridge Design Code (OHBDC)
- ASCE for 3 levels of heavy truck traffic (7.5%, 30% and 100%)

ASCE Equivalent Live Load for Long Spans (kip/ft)

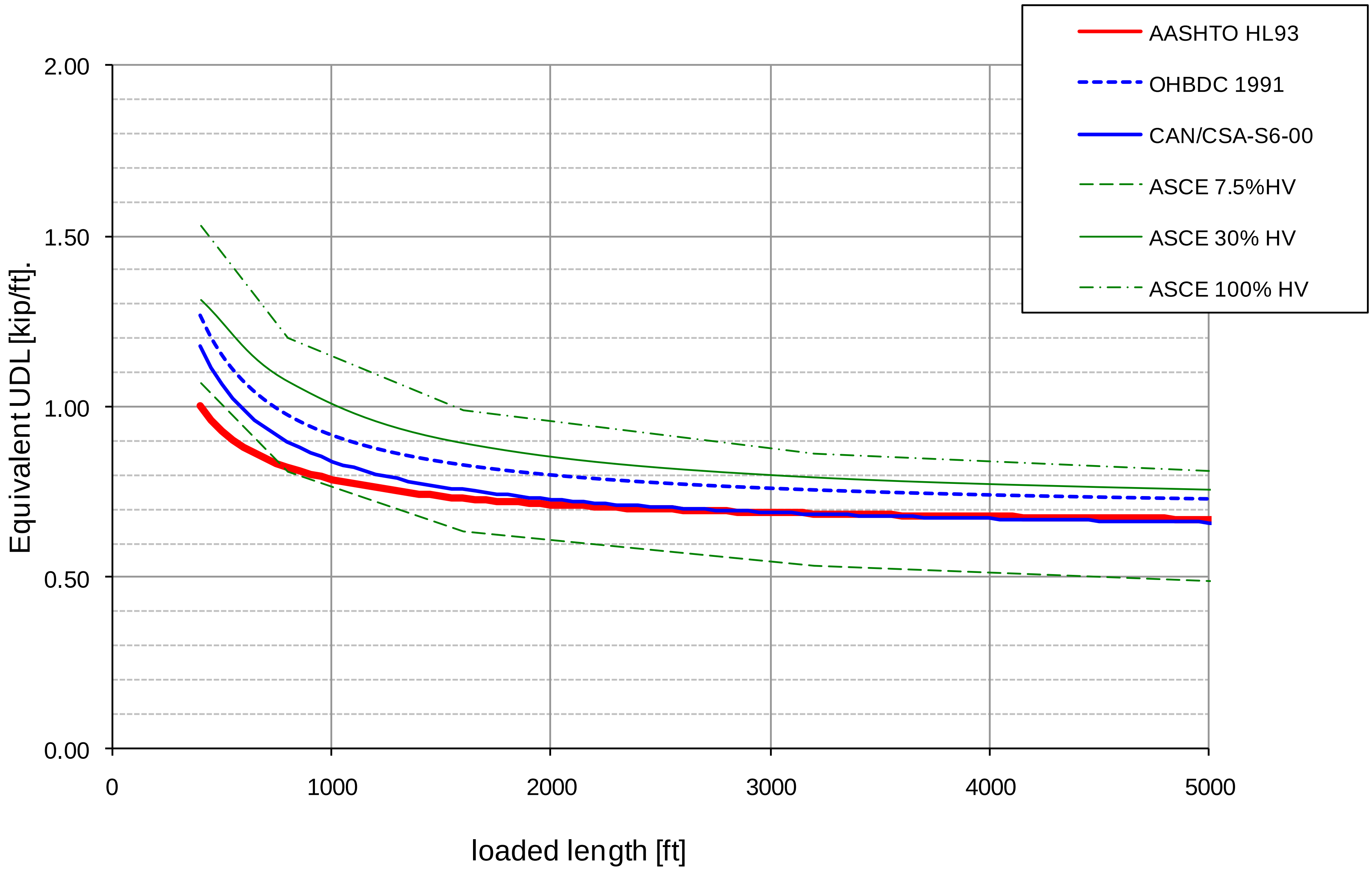


ASCE Equivalent Live Load for Long Spans (kip/ft)

length [ft]	P [kip]	U [kip/ft]		
		7.5%HV	30% HV	100% HV
400	72	0.71	0.95	1.17
800	96	0.57	0.83	0.96
1600	120	0.49	0.74	0.84
3200	144	0.44	0.70	0.77
6400	168	0.40	0.68	0.72

length [ft]	UDL Unfactored [kip/ft]			UDL Factored [kip/ft]		
	7.5%HV	30% HV	100% HV	7.5%HV	30% HV	100% HV
400	1.07	1.31	1.53	1.93	2.36	2.75
800	0.81	1.07	1.20	1.46	1.93	2.16
1600	0.64	0.89	0.99	1.14	1.60	1.78
3200	0.53	0.79	0.86	0.95	1.42	1.55
6400	0.45	0.73	0.77	0.81	1.32	1.39

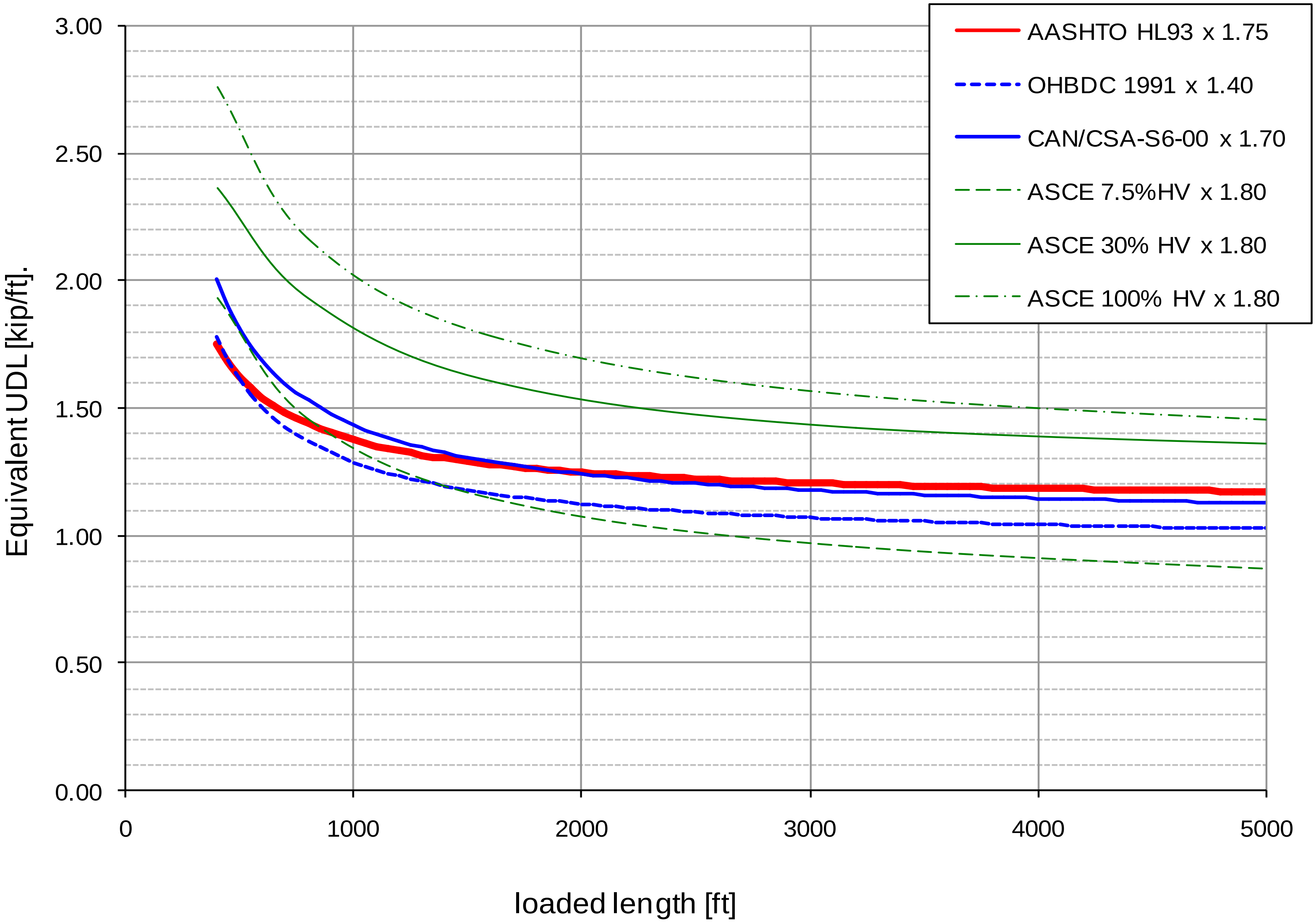
Equivalent Unfactored Live Load (kip/ft)
w/o IM, w/o multilane factors



Equivalent Unfactored Live Load (kip/ft)
w/o IM, w/o multilane factors

length [ft]	OHBDC 1991	CAN/CSA-S6-00	HL-93
500	1.151	1.067	0.928
1000	0.918	0.842	0.784
1500	0.841	0.767	0.736
2000	0.802	0.729	0.712
2500	0.779	0.707	0.698
3000	0.763	0.692	0.688
3500	0.752	0.681	0.681
4000	0.744	0.673	0.676
4500	0.737	0.667	0.672
5000	0.732	0.662	0.669

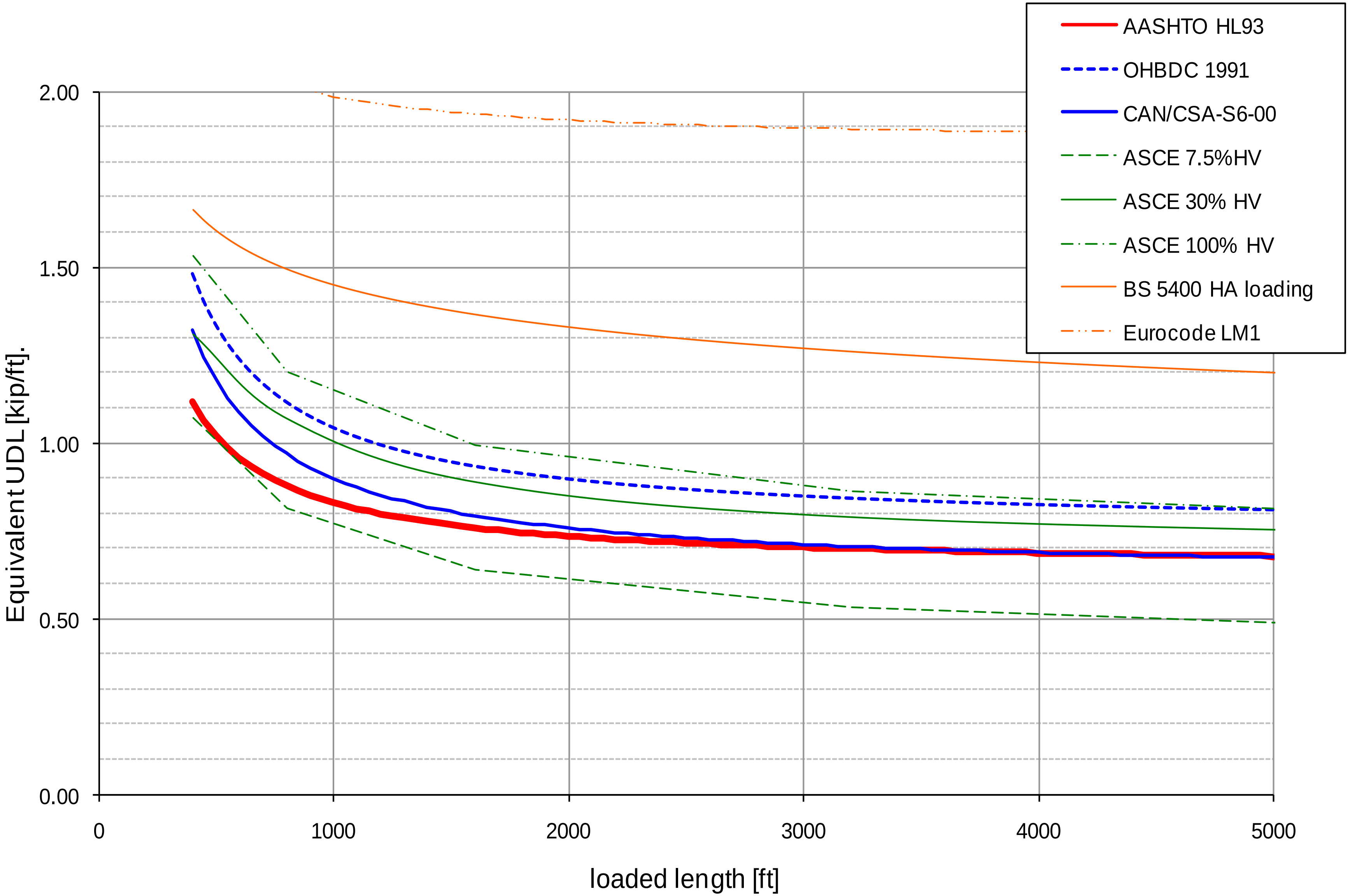
Equivalent Factored Live Load (kip/ft)
w/o IM, w/o multilane factors



Equivalent Factored Live Load (kip/ft) w/o IM, w/o multilane factors

length [ft]	OHBDC 1991	CAN/CSA-S6-00	HL-93
500	1.612	1.813	1.624
1000	1.286	1.431	1.372
1500	1.177	1.303	1.288
2000	1.123	1.240	1.246
2500	1.090	1.202	1.221
3000	1.068	1.176	1.204
3500	1.053	1.158	1.192
4000	1.041	1.144	1.183
4500	1.032	1.134	1.176
5000	1.025	1.125	1.170

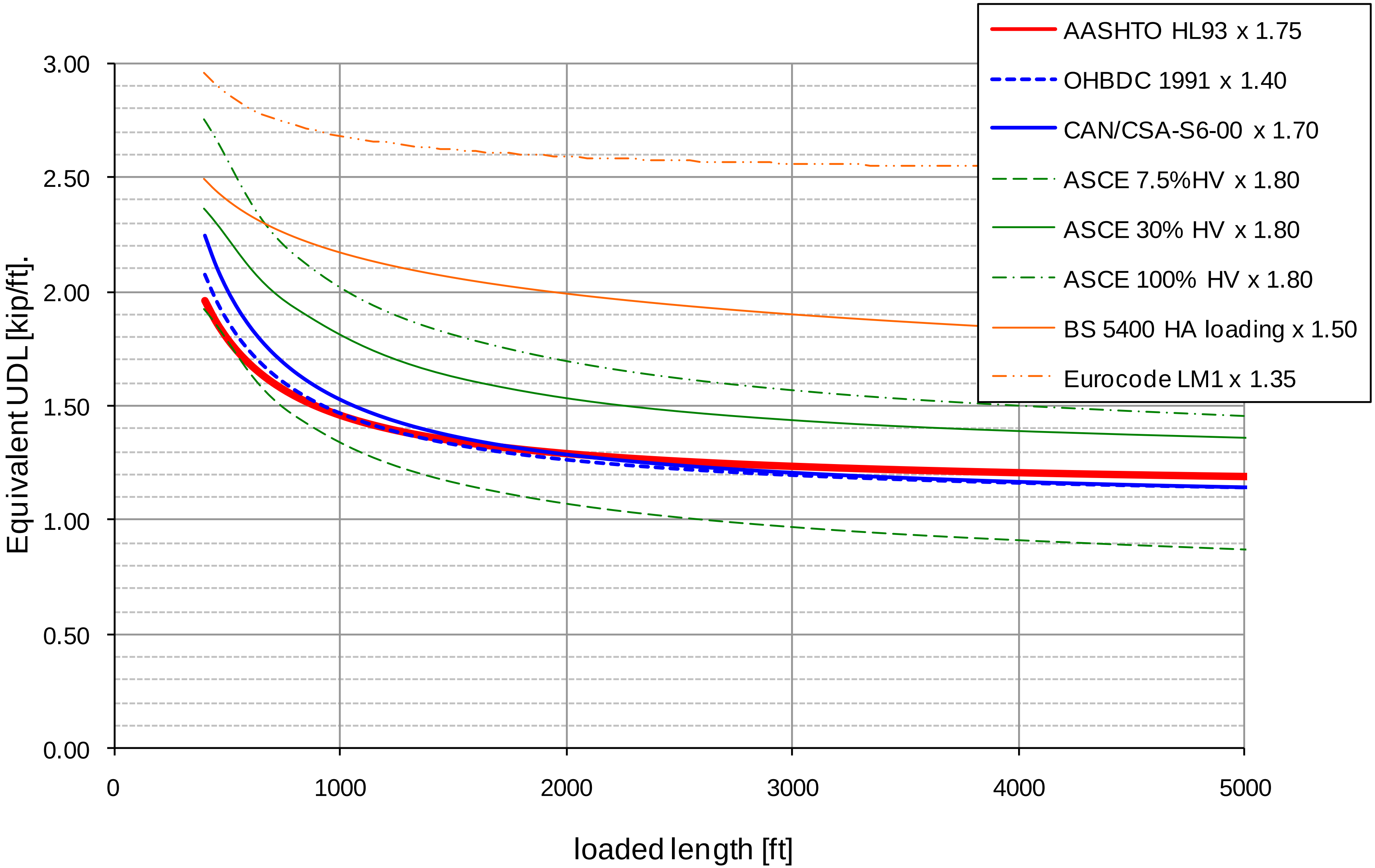
Equivalent Unfactored Live Load (kip/ft) with IM, w/o multilane factors



Equivalent Unfactored Live Load (kip/ft) with IM, w/o multilane factors

length [ft]	OHBDC 1991	CAN/CSA-S6-00	HL-93	BS 5400	Eurocode
500	1.336	1.179	1.023	1.603	2.120
1000	1.045	0.898	0.832	1.449	1.985
1500	0.948	0.804	0.768	1.375	1.941
2000	0.900	0.757	0.736	1.328	1.918
2500	0.870	0.729	0.717	1.294	1.905
3000	0.851	0.711	0.704	1.268	1.896
3500	0.837	0.697	0.695	1.246	1.889
4000	0.827	0.687	0.688	1.228	1.884
4500	0.819	0.679	0.683	1.212	1.881
5000	0.812	0.673	0.678	1.198	1.878

Equivalent Factored Live Load (kip/ft) with IM, w/o multilane factors



Equivalent Factored Live Load (kip/ft) with IM, w/o multilane factors

	OHBDC 1991	CAN/CSA-S6-00	HL-93	BS 5400	Eurocode
500	1.871	2.004	1.790	2.404	2.862
1000	1.463	1.526	1.455	2.173	2.680
1500	1.327	1.367	1.343	2.063	2.620
2000	1.259	1.288	1.288	1.993	2.589
2500	1.219	1.240	1.254	1.942	2.571
3000	1.191	1.208	1.232	1.902	2.559
3500	1.172	1.185	1.216	1.869	2.550
4000	1.157	1.168	1.204	1.842	2.544
4500	1.146	1.155	1.194	1.818	2.539
5000	1.137	1.144	1.187	1.797	2.535

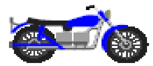
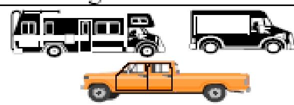
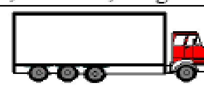
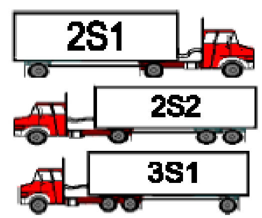
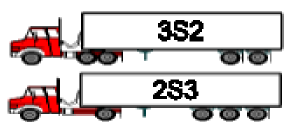
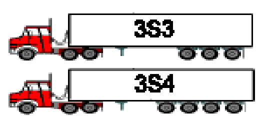
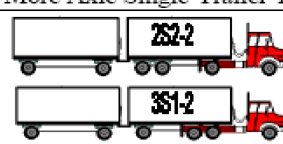
Data Base for Live Load

- Site-specific (ADTT, truck weight, multiple presence, traffic patterns)
- Component-specific (girders, cross frames, hangers, suspension cables, towers)
- WIM data and other site-specific and component-specific data
- ASCE lane loads depending on the percentage of heavy trucks (7.5%, 30% and 100%) (kips/ft)

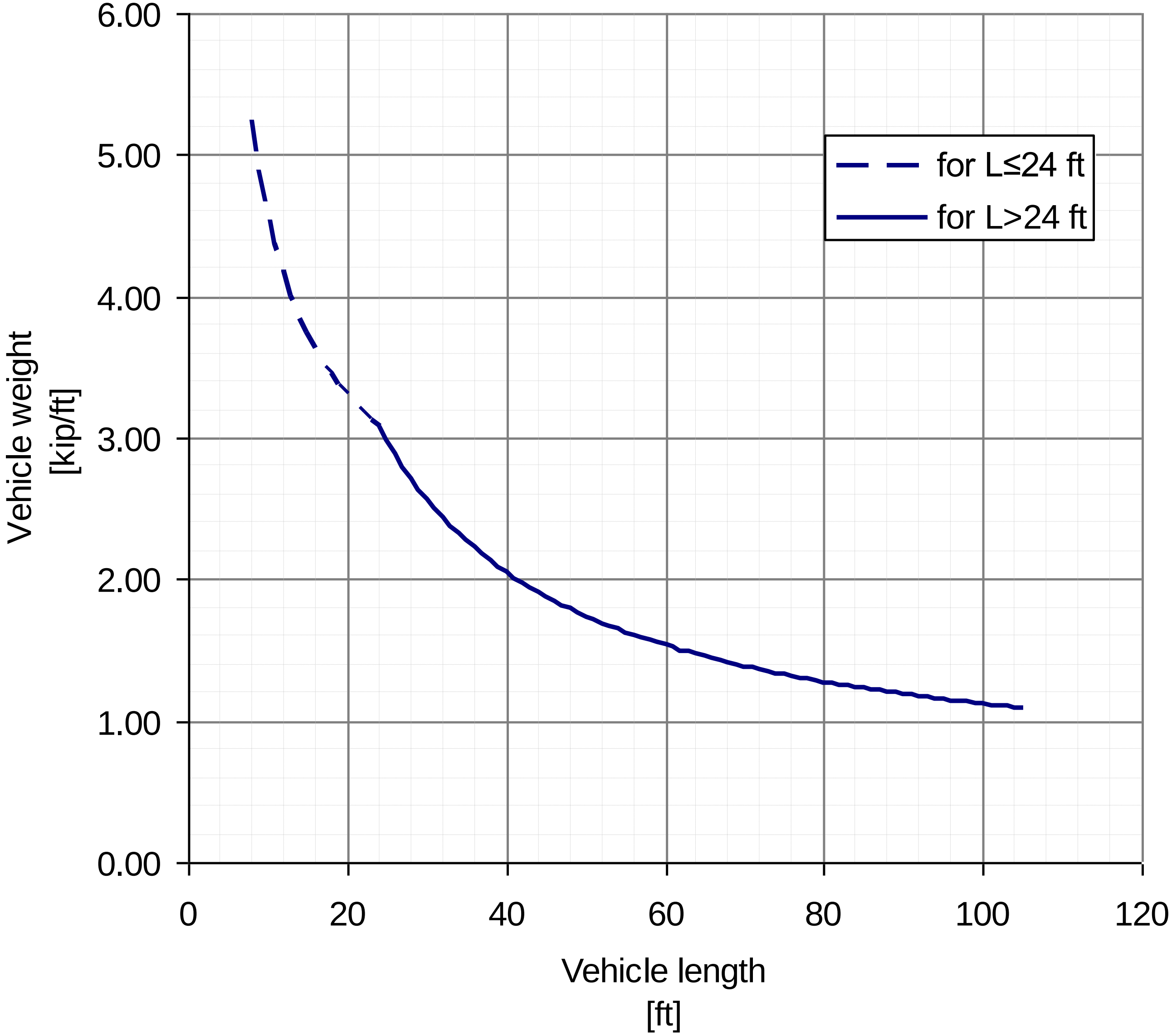
Federal Limits

- 20 000 pounds - maximum gross weight upon any one axle
- 34 000 pounds - maximum gross weight on tandem axles
- 80 000 pounds - maximum gross vehicle weight
- 102 inches - maximum vehicle width
- 48-feet - minimum vehicle length for a semi-trailer in a truck-tractor/semi-trailer combination
- 28 feet - minimum vehicle length

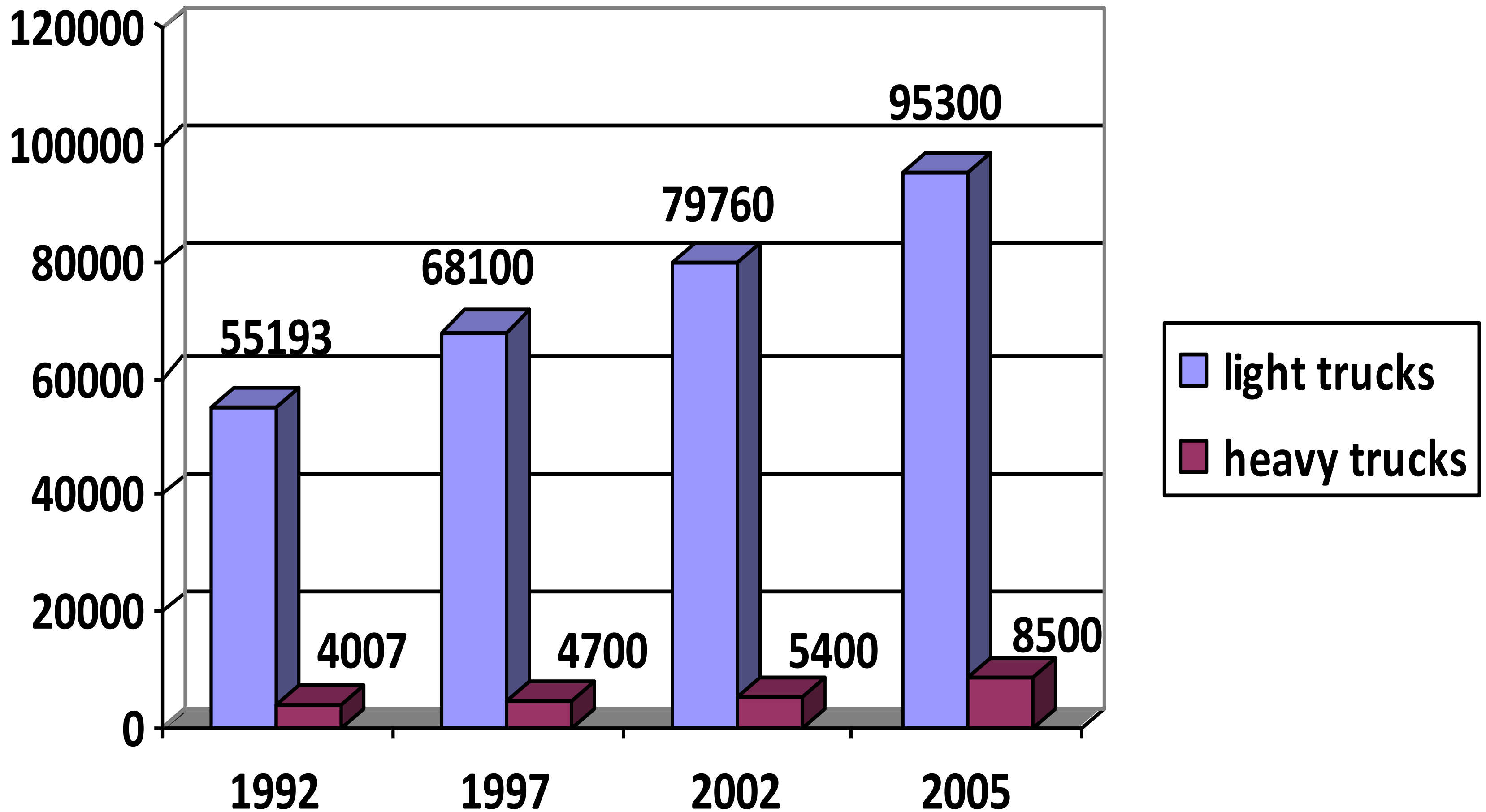
FHWA -13 Categories

 FHWA Class 1 Motorcycles	 FHWA Class 2 Passenger Vehicles
 FHWA Class 3 Other Two-Axle, Four-Tire Single-Unit Vehicles	 FHWA Class 4 Buses
 FHWA Class 5 Two-Axle, Six-Tire, Single-Unit Trucks	 FHWA Class 6 Three-Axle Single-Unit Trucks
 FHWA Class 7 Four or More Axle Single-Unit Trucks	 FHWA Class 8 Four or Fewer Axle Single-Trailer Trucks
 FHWA Class 9 Five-Axle Single-Trailer Trucks	
 FHWA Class 10 Six or More Axle Single-Trailer Trucks	 FHWA Class 11 Five or Fewer Axle Multi-trailer Trucks
 FHWA Class 12 Six-Axle Multi-trailer Trucks	 FHWA Class 13 Seven or More Axle Multi-trailer Trucks

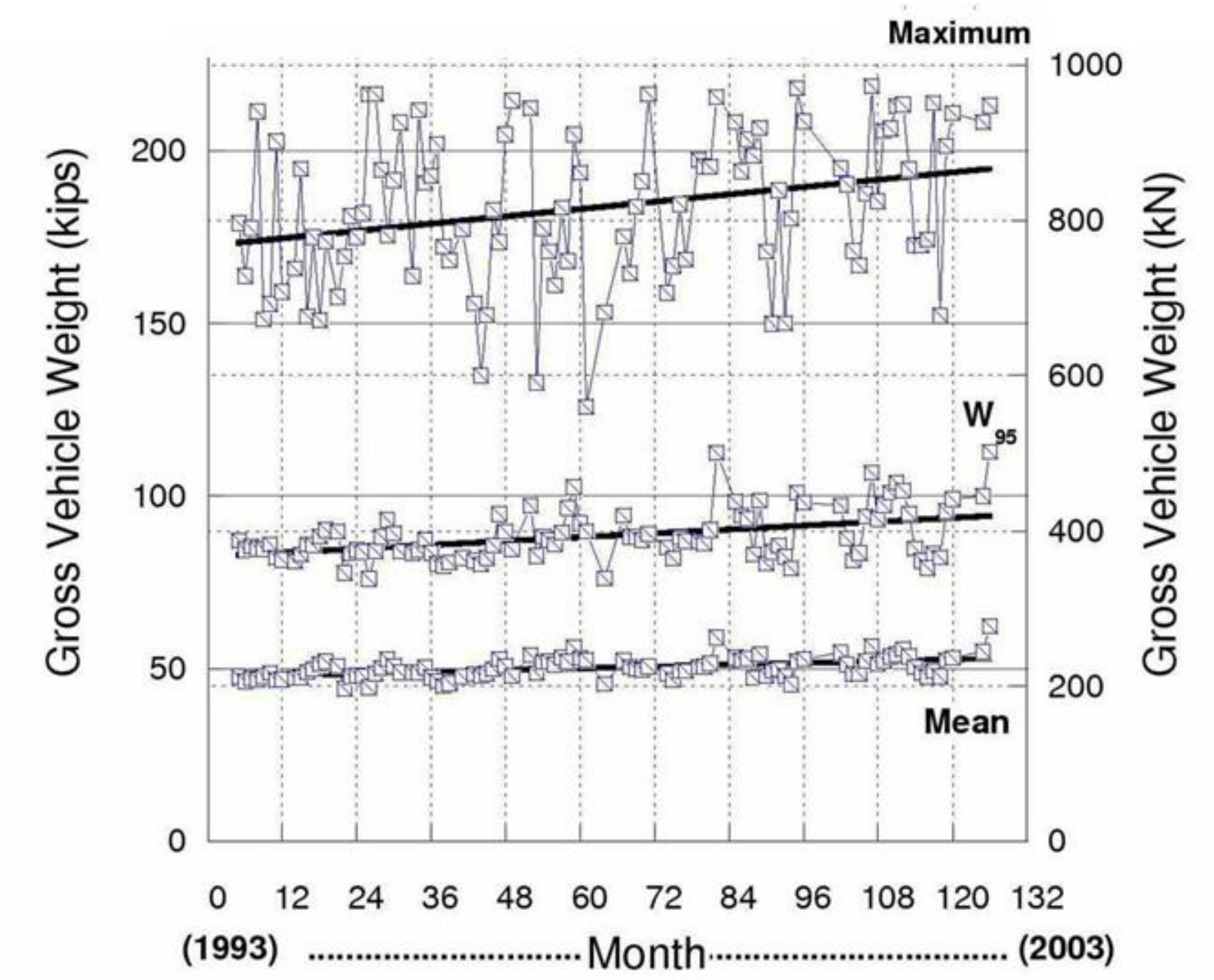
New Bridge Formula



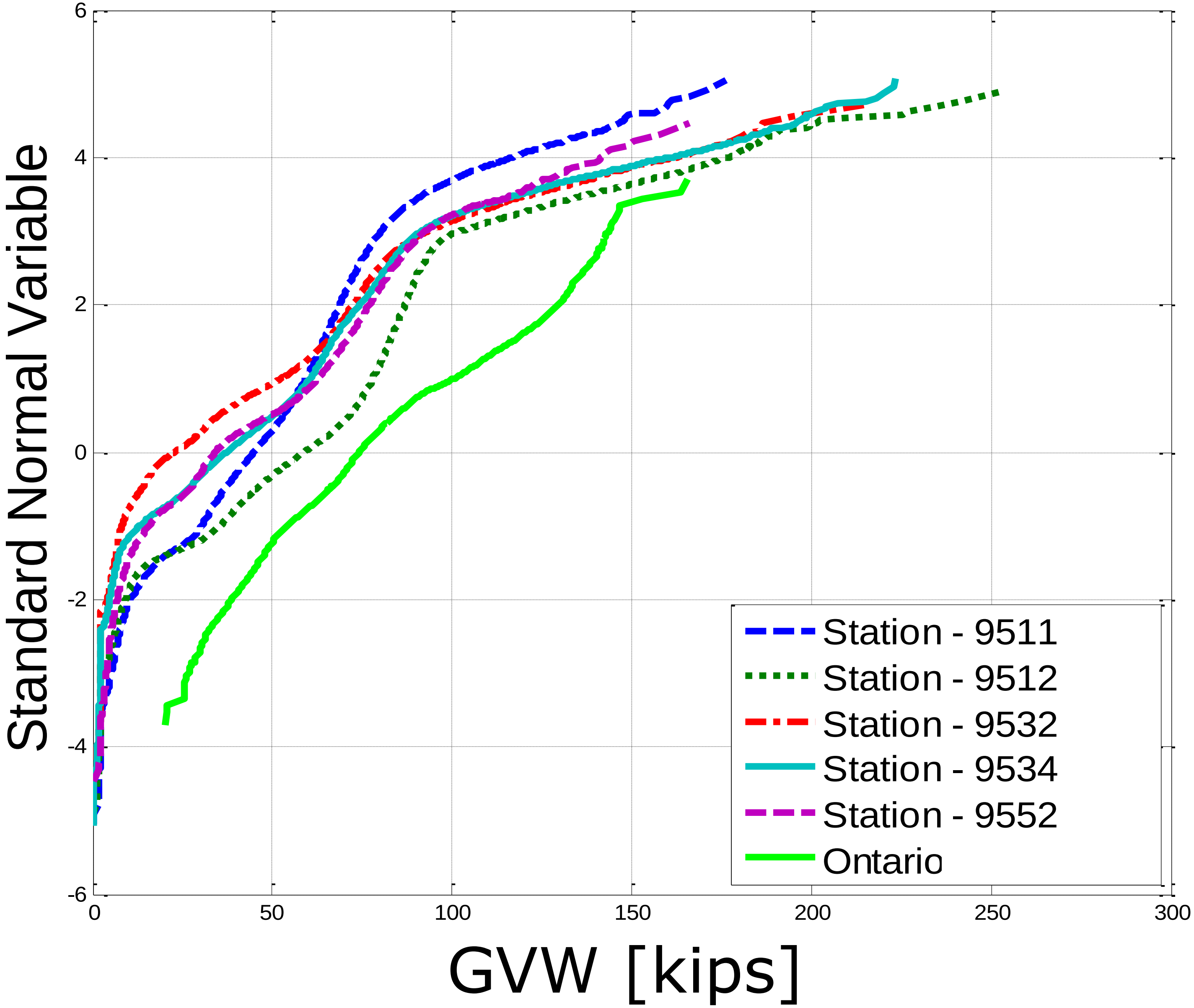
Changes in Number of Trucks (in thousands)



Changes in Statistics (Nassif and Gindy)



CDF's of GVW
WIM Data – Indiana (2006)



Multiple Presence (Sivakumar)

Headway	side-by-side truck traffic			following truck traffic		
[ft]	light	average	heavy	light	average	heavy
H ≤ 20	0.19	0.40	0.61	0.00	0.00	0.00
20 ≤ H ≤ 40	0.14	0.43	0.66	0.00	0.00	0.00
40 ≤ H ≤ 60	0.21	0.41	0.68	0.01	0.00	0.00
60 ≤ H ≤ 80	0.26	0.35	0.62	0.01	0.00	0.00
80 ≤ H ≤ 100	0.20	0.53	0.76	0.06	0.04	0.03
100 ≤ H ≤ 120	0.21	0.41	0.81	0.12	0.15	0.16
120 ≤ H ≤ 140	0.24	0.34	0.66	0.21	0.33	0.45
140 ≤ H ≤ 160	0.17	0.30	0.61	0.36	0.57	0.73
160 ≤ H ≤ 180	0.18	0.29	0.56	0.48	0.67	0.91
180 ≤ H ≤ 200	0.19	0.26	0.52	0.46	0.75	0.98
200 ≤ H ≤ 220	0.10	0.24	0.48	0.51	0.68	0.94
220 ≤ H ≤ 240	0.14	0.24	0.45	0.48	0.67	0.91
240 ≤ H ≤ 260	0.12	0.22	0.43	0.42	0.65	0.87
260 ≤ H ≤ 280	0.14	0.21	0.41	0.41	0.60	0.85
280 ≤ H ≤ 300	0.11	0.20	0.40	0.39	0.59	0.80

Multiple Presence (Sivakumar)

Number of Lanes Simultaneously Loaded in One Direction	6/14/2005		6/15/2005		6/16/2005	
	Northbound	Southbound	Northbound	Southbound	Northbound	Southbound
3 Lanes Loaded Simultaneously						
Moderate Truck Loads	15	2	4	3	2	7
Heavy Truck Loads	5	0	0	0	1	1
Left & Cent Lanes Loaded Simult						
Moderate Truck Loads	26	0	5	61	10	28
Heavy Truck Loads	2	0	6	28	1	0
Cent & Right Lanes Loaded Simult						
Moderate Truck Loads	255	155	215	89	221	182
Heavy Truck Loads	211	79	105	34	113	65
Left & Right Lanes Loaded Simult						
Moderate Truck Loads	3	1	2	0	1	3
Heavy Truck Loads	0	0	0	0	0	0

Two scenarios:

- Random traffic, moving with highway speed
- Traffic jam, crawling speed (governs)

- Dense traffic jam situations (7 videos)
- Some of them being a result of traffic accidents
- Various localizations
- Different time and day of the week
- Total recording time over 2 hours
- We observed traffic patterns, multiple presence of trucks moving at crawling speed
- We can assume that critical loading case is caused by traffic moving at crawling speed, with trucks and occasional cars

- Even in a very dense traffic jam it is common to observe cars or pick-ups among heavy vehicles

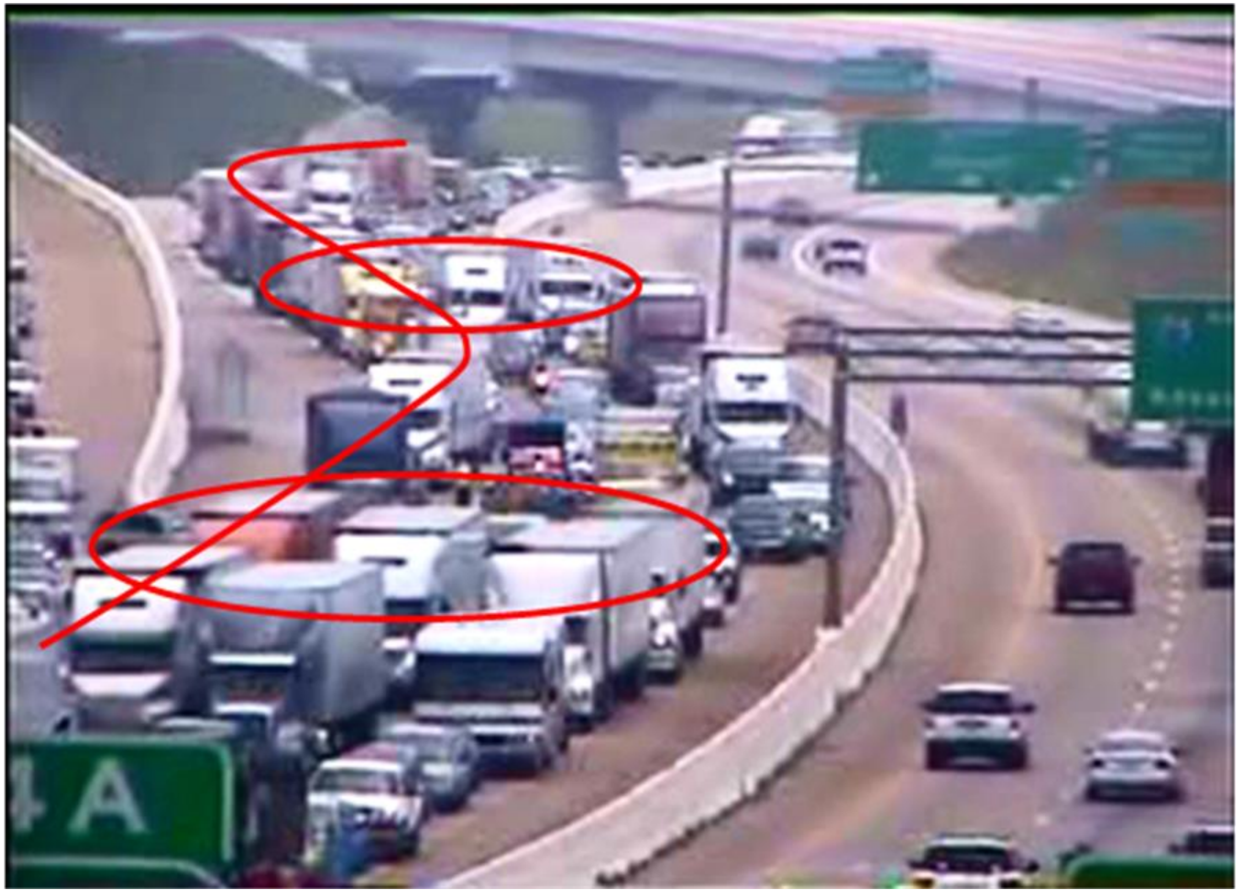
Video 10, time: 00:00:58

- Multiple-presence of trucks occupying four lanes

Video 1, time: 00:05:28

Video Recordings of Traffic Jam Situations FHWA Data

- Multiple-presence of trucks occupying three lanes
- One lane is almost exclusively occupied by trucks



Video 1, time: 00:18:36

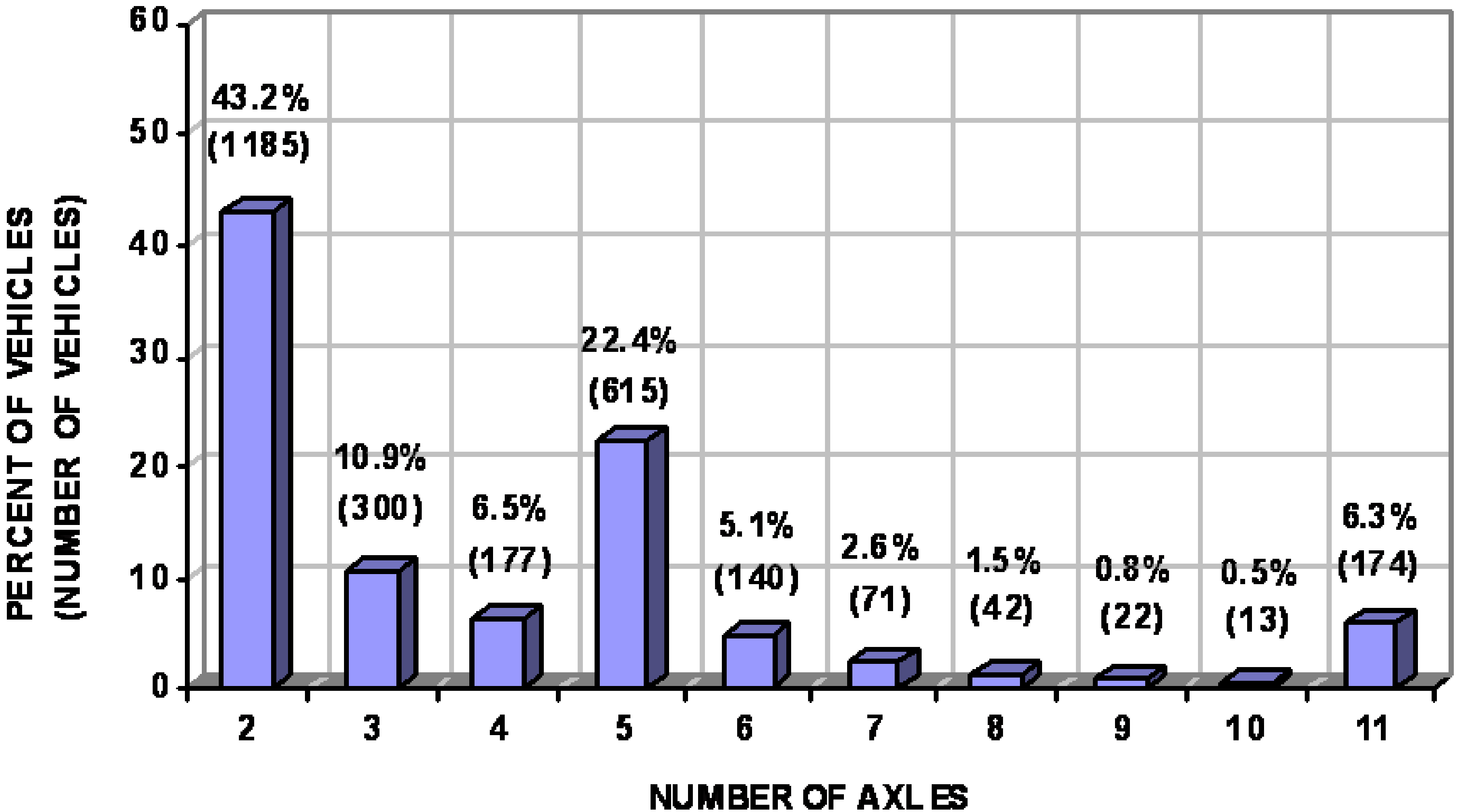
- Site selection is important, traffic pattern close to exit or entrance can be different, with more passenger cars

. Video 2, time: 00:00:15

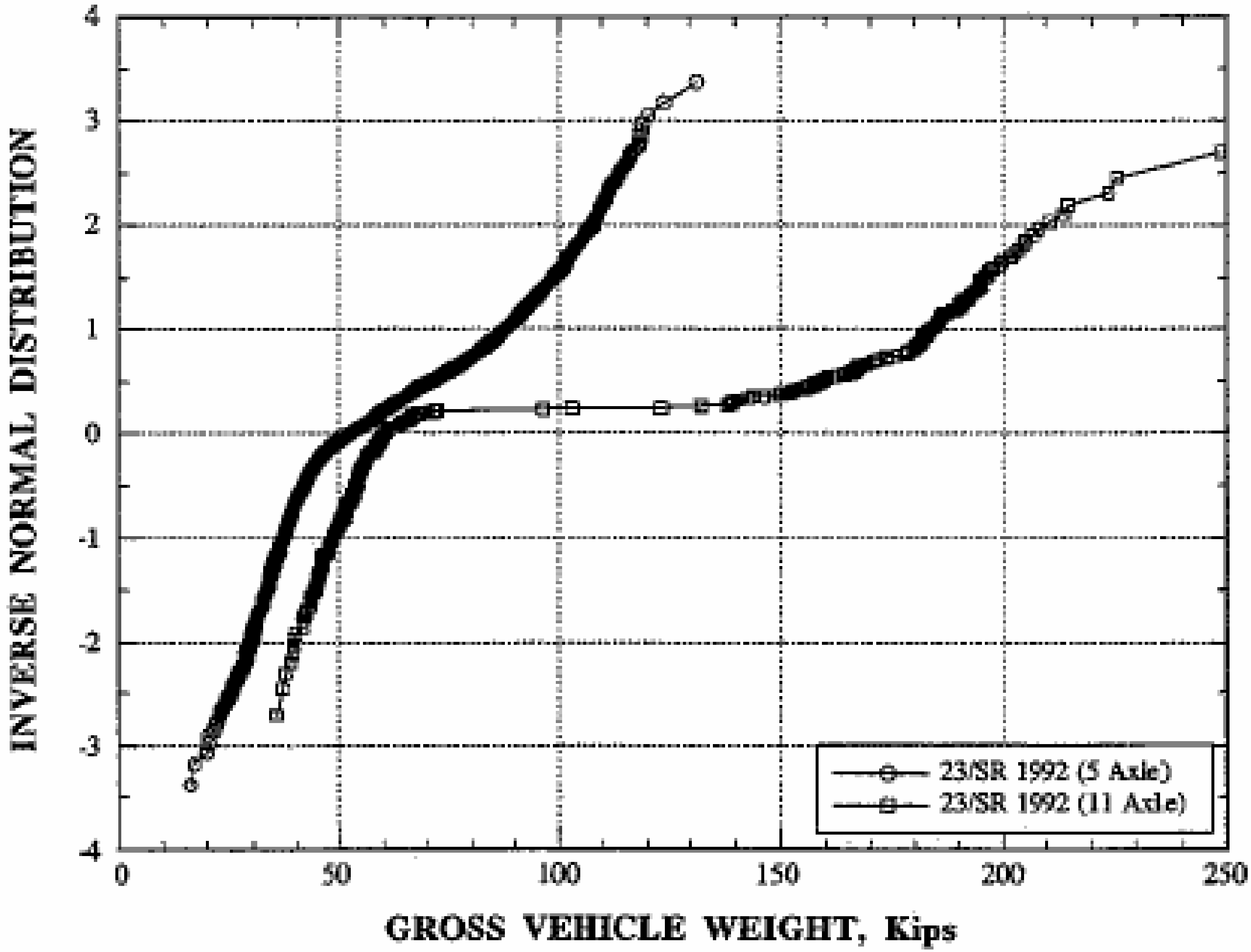
- Initial Study:
  (a) Based on average trucks
  (b) Based on legal trucks

- Detailed Study Based on truck WIM Data (NCHRP 12-76)

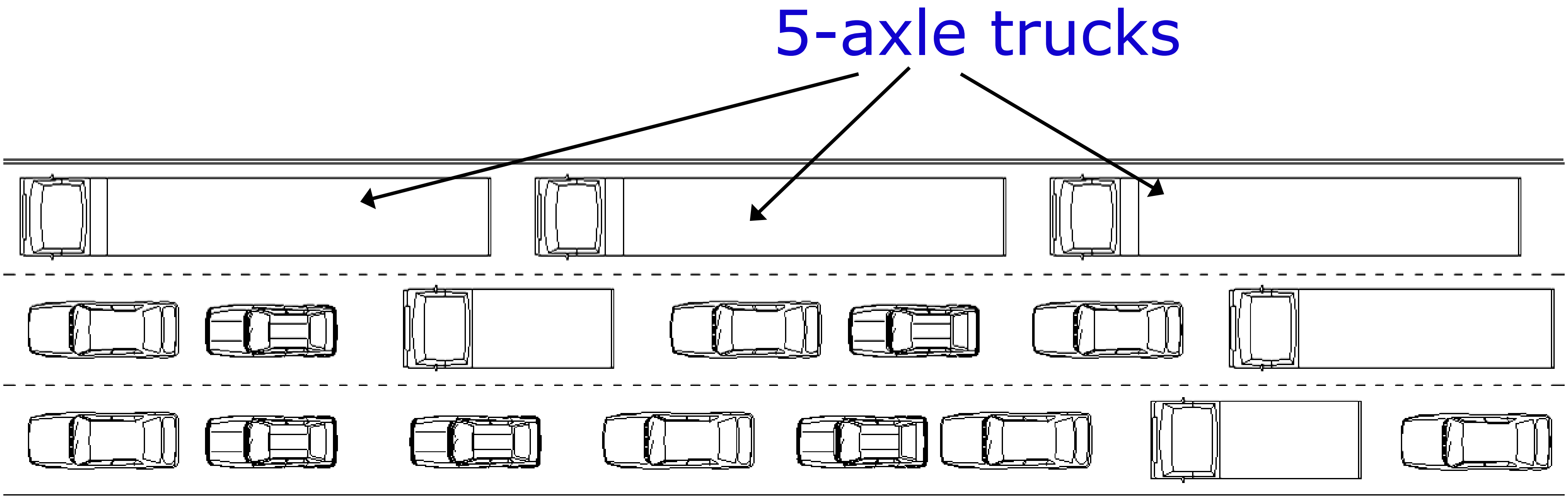
Truck Statistics (Michigan)



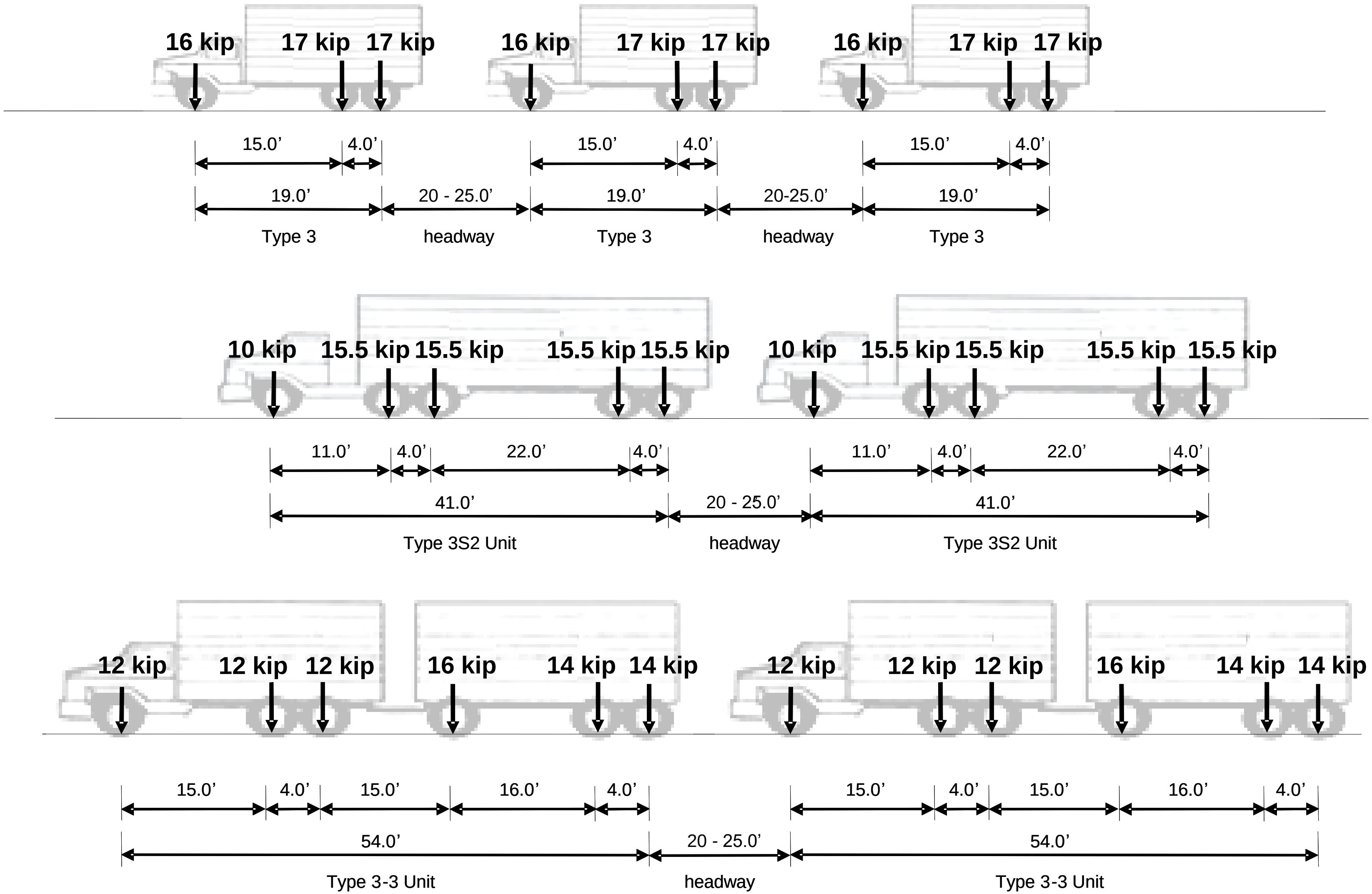
Truck load WIM data (MI)



Load Pattern



Legal Load Types



- Most common trucks are 5-axle vehicles
- Average length 45 ft
- Average weight 53 kips
- Headway distance is 10-15 ft, therefore spacing between last axle of one truck and first axle of the following truck is 20-25 ft
- Load is 53 kips / 70 ft = 0.76 k/ft for 15 ft
- Load is 53 kips / 65 ft = 0.82 k/ft for 10 ft

- Type 3-3 Unit
- Gross Vehicle Weight 80 kips
- Total length 54 ft
- Headway distance 10-15 ft, therefore spacing between last axle of one truck and first axle of the following truck is 20-25 ft)
- Load 80 kips / (54 + 25) ft = 1.08 k/ft for 15 ft
- Load 80 kips / (54 + 20) ft = 1.01 k/ft for 10 ft

This is conservative, therefore, 75% is used

0.75 (1.08) = 0.81 k/ft

0.75 (1.01) = 0.76 k/ft

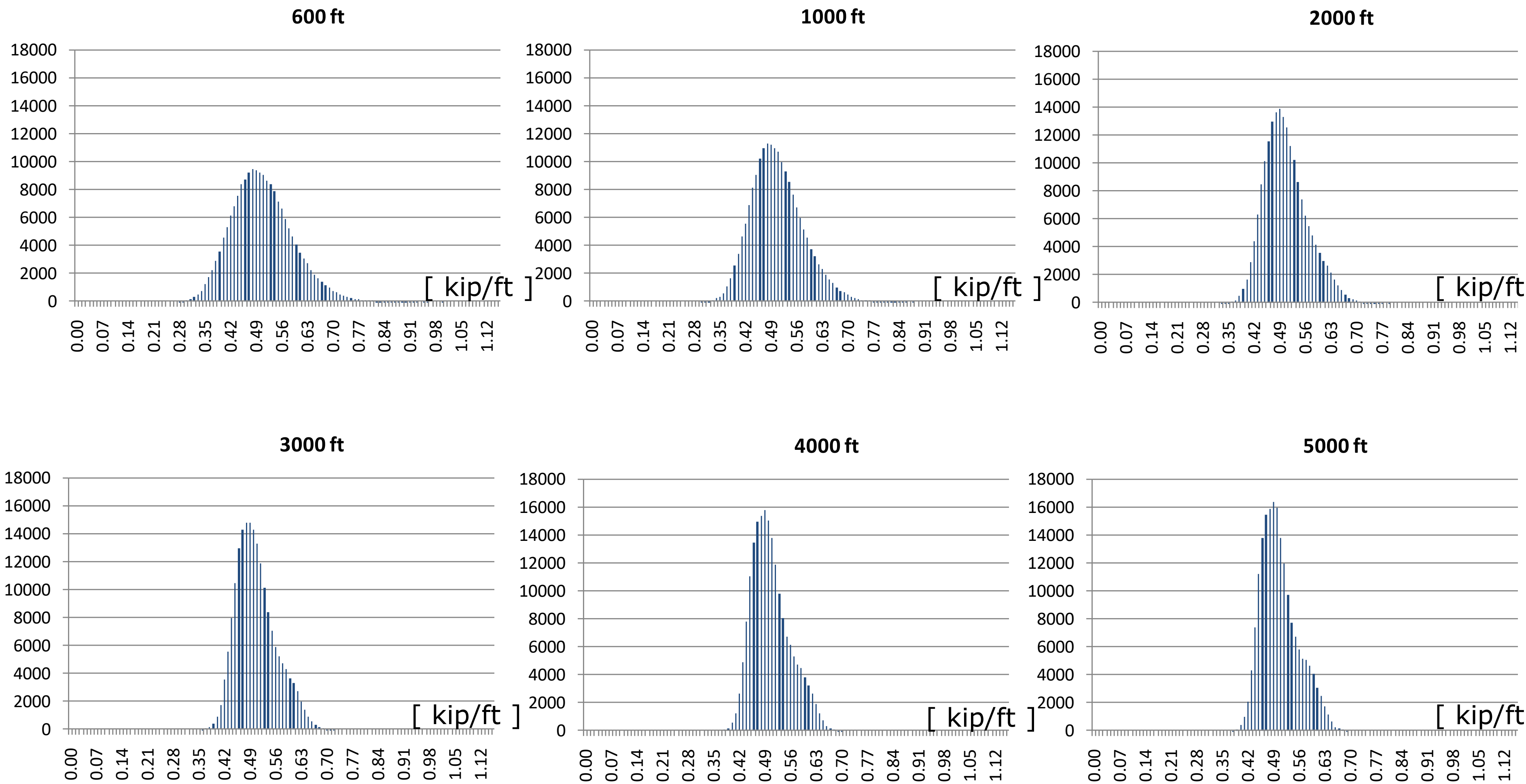
Simulation of Traffic Jam Situation Using WIM Data

- WIM Data from NCHRP 12-76
- Various sites: California (6), Florida (5), Indiana (6), Mississippi (5), New York (7), Oklahoma (16)
- Span lengths: 600ft, 1000ft, 2000ft, 3000ft, 4000ft, 5000ft
- Trucks are in actual order (as recorded in the WIM surveys)
- Headway distance is about 15 ft (spacing between last axle of one truck and first axle of the following truck is 25 ft)
- Only the most loaded lane is considered
- Light vehicles are using faster lanes, therefore, vehicles of 1-3 FHWA category are omitted
- UDL is calculated as a moving average (k/ft) for variety of span lengths

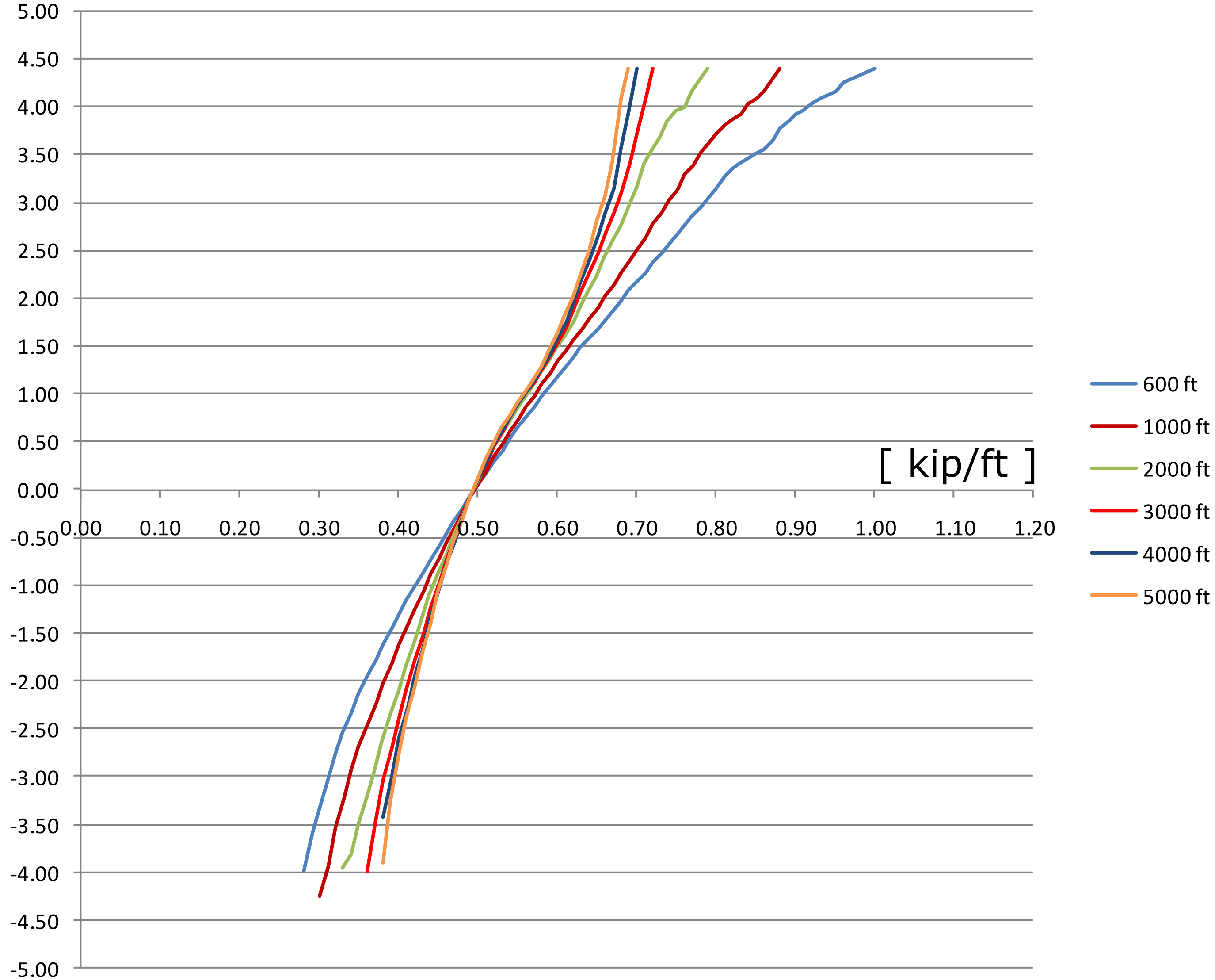
Simulation of Traffic Jam Situation Using WIM Data

- Starting with the first truck, all consecutive trucks were added with a fixed headway distance between them, until the total length exceeded the span length.
- Then, the total load of all trucks was calculated and divided by the span length to obtain the first value of the average uniformly distributed load.
- Next, the first truck was deleted, and one or more trucks were added so that the total length of trucks covers the full span length, and the new value of the average uniformly distributed load was calculated.
- This way, the uniformly distributed load was derived as a moving average for span lengths of 600ft, 1000ft, 2000ft, 3000ft, 4000ft, and 5000ft.

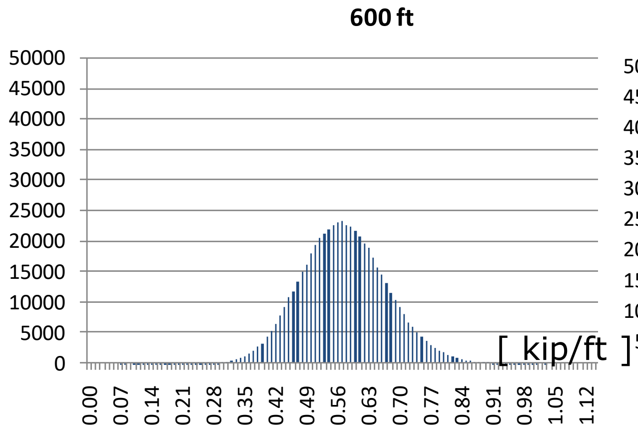
Histogram of lane load for different span lengths Florida 9936, 1st lane

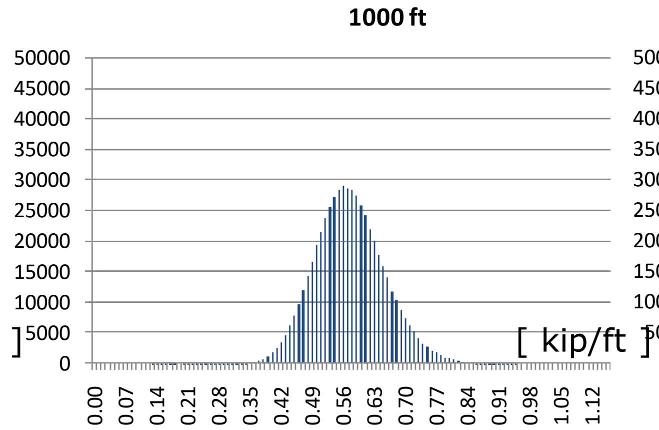


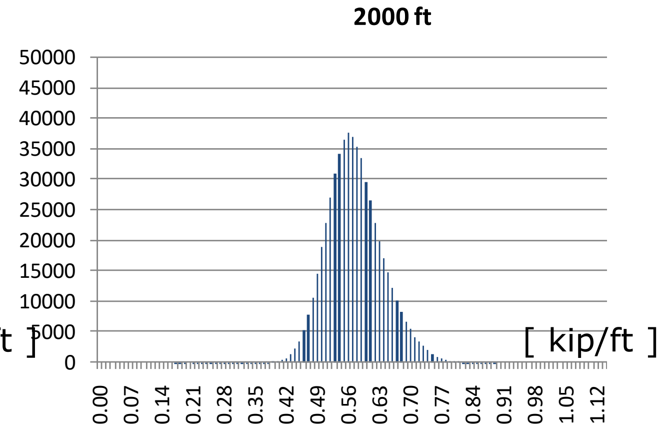
CDF's of UDL for different span lengths (kip/ft)
Florida 9936, 1st lane

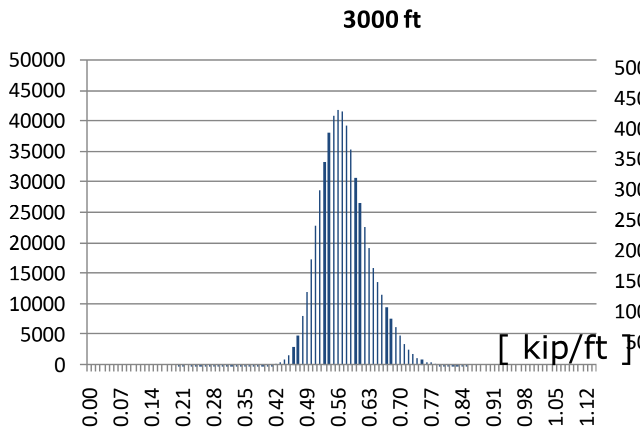


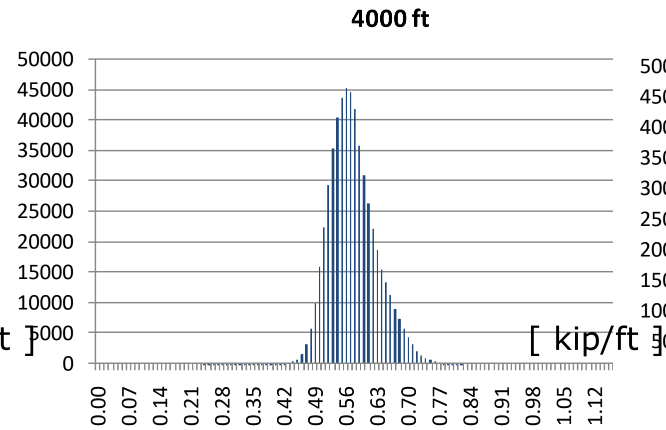
Histogram of lane load for different span lengths Oregon I-5 Woodburn, 1st lane

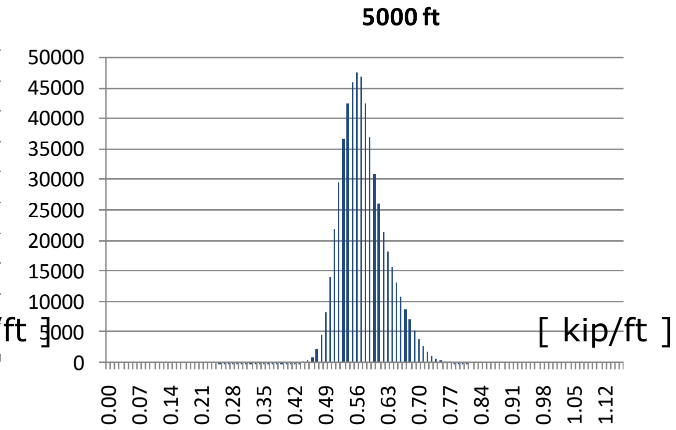




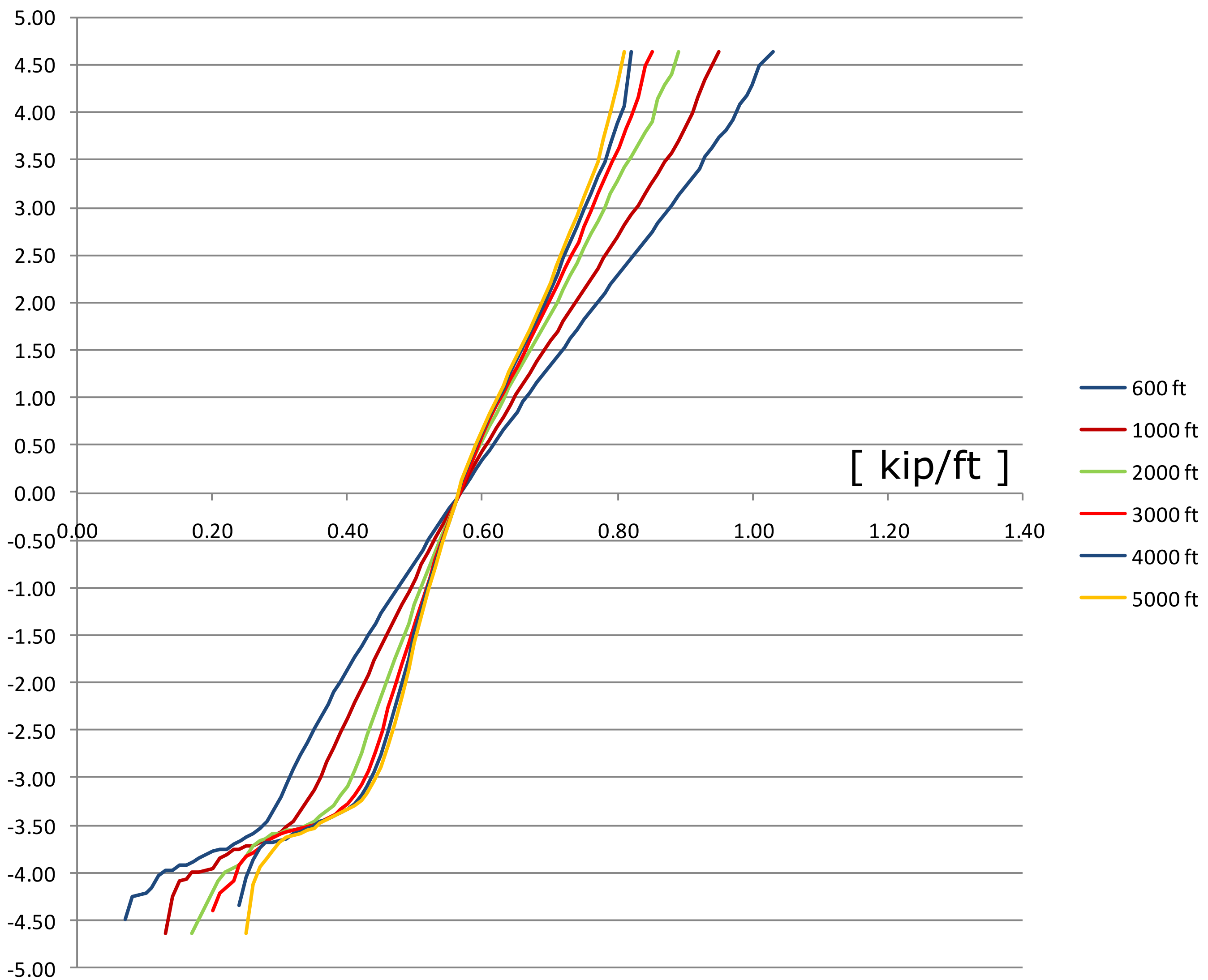




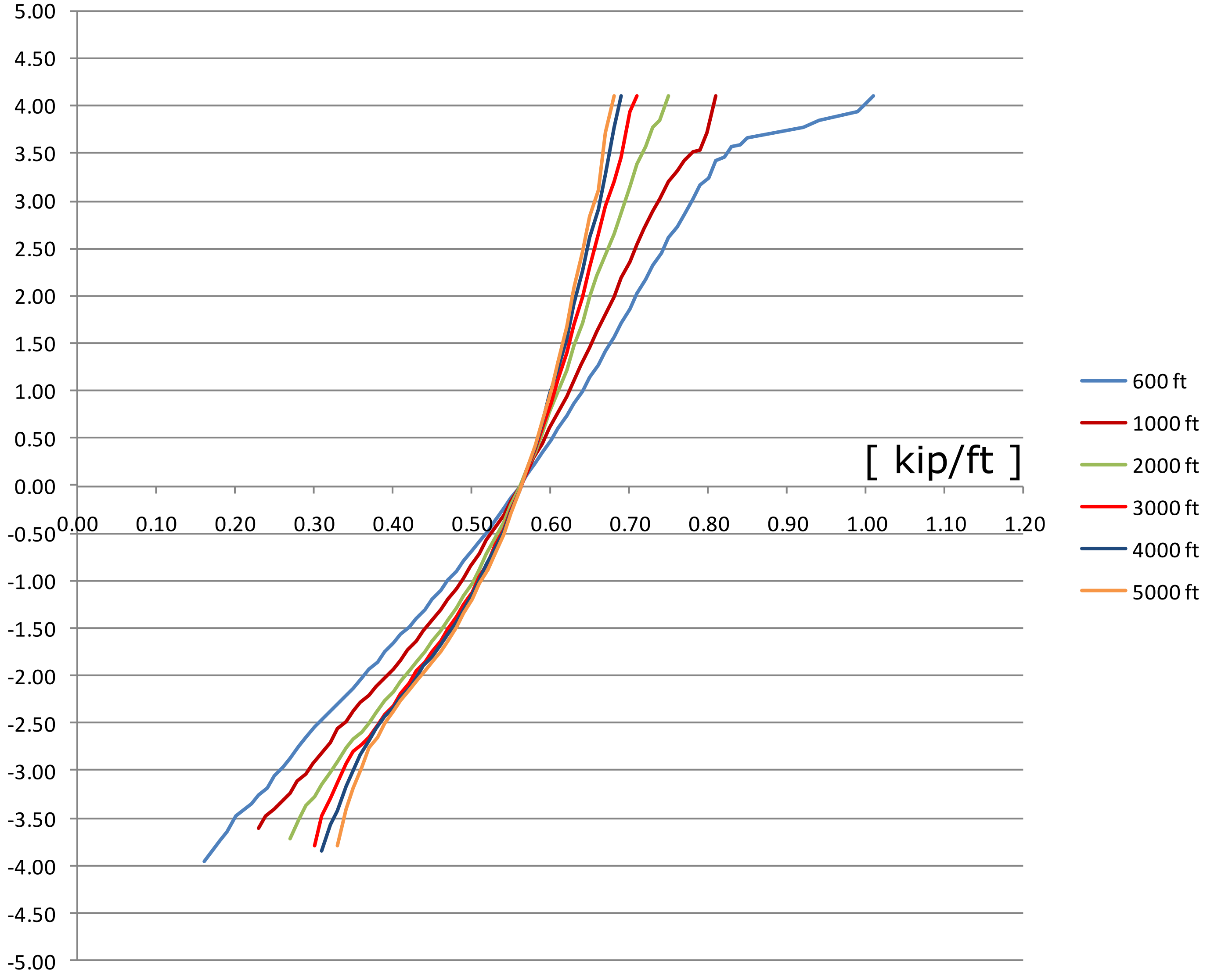




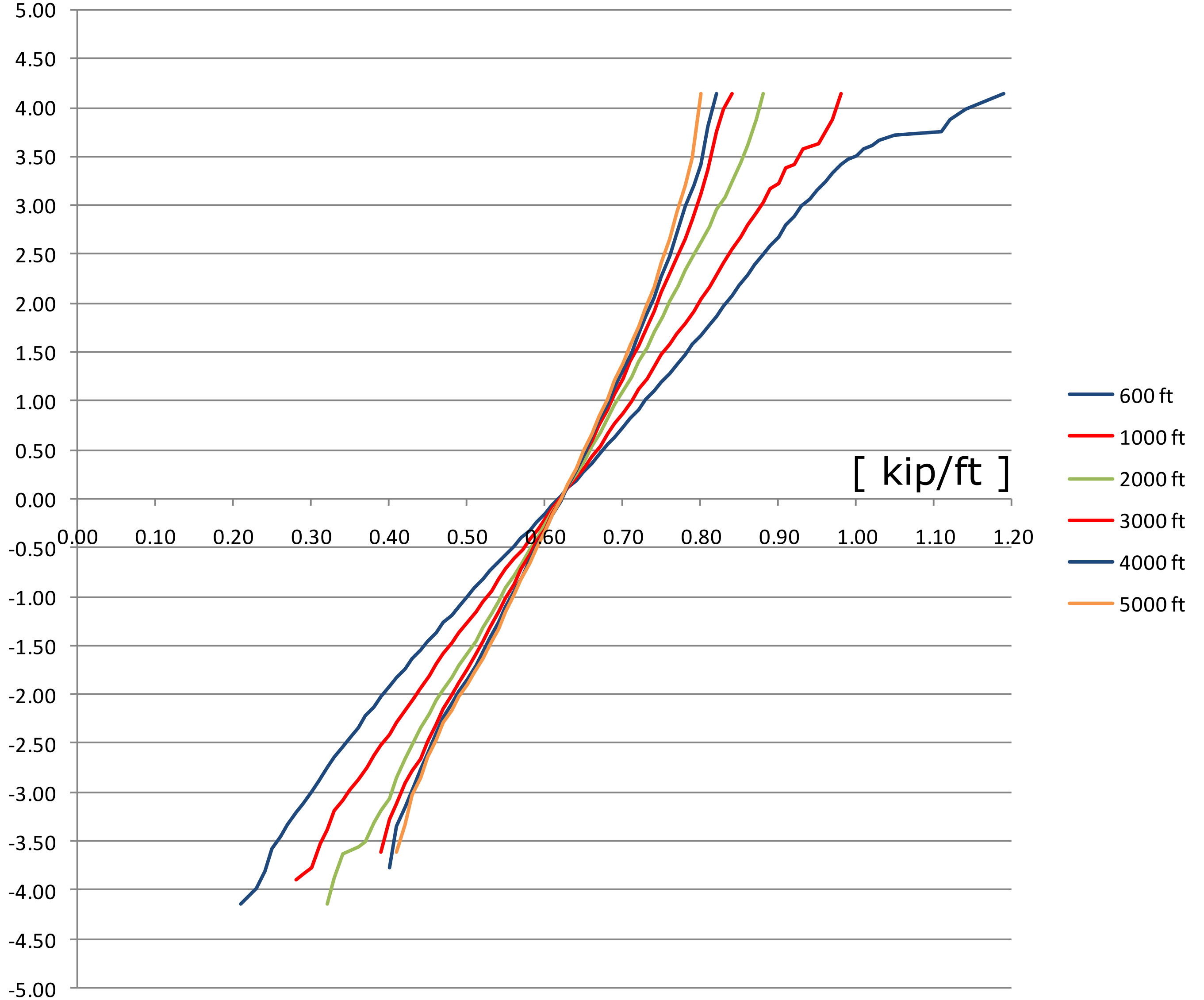
CDF's of UDL for different span lengths (kip/ft)
Oregon I-5 Woodburn, 1st lane



CDF's of UDL for different span lengths (kip/ft)
Oregon – OR 58 Lowell, 1st lane

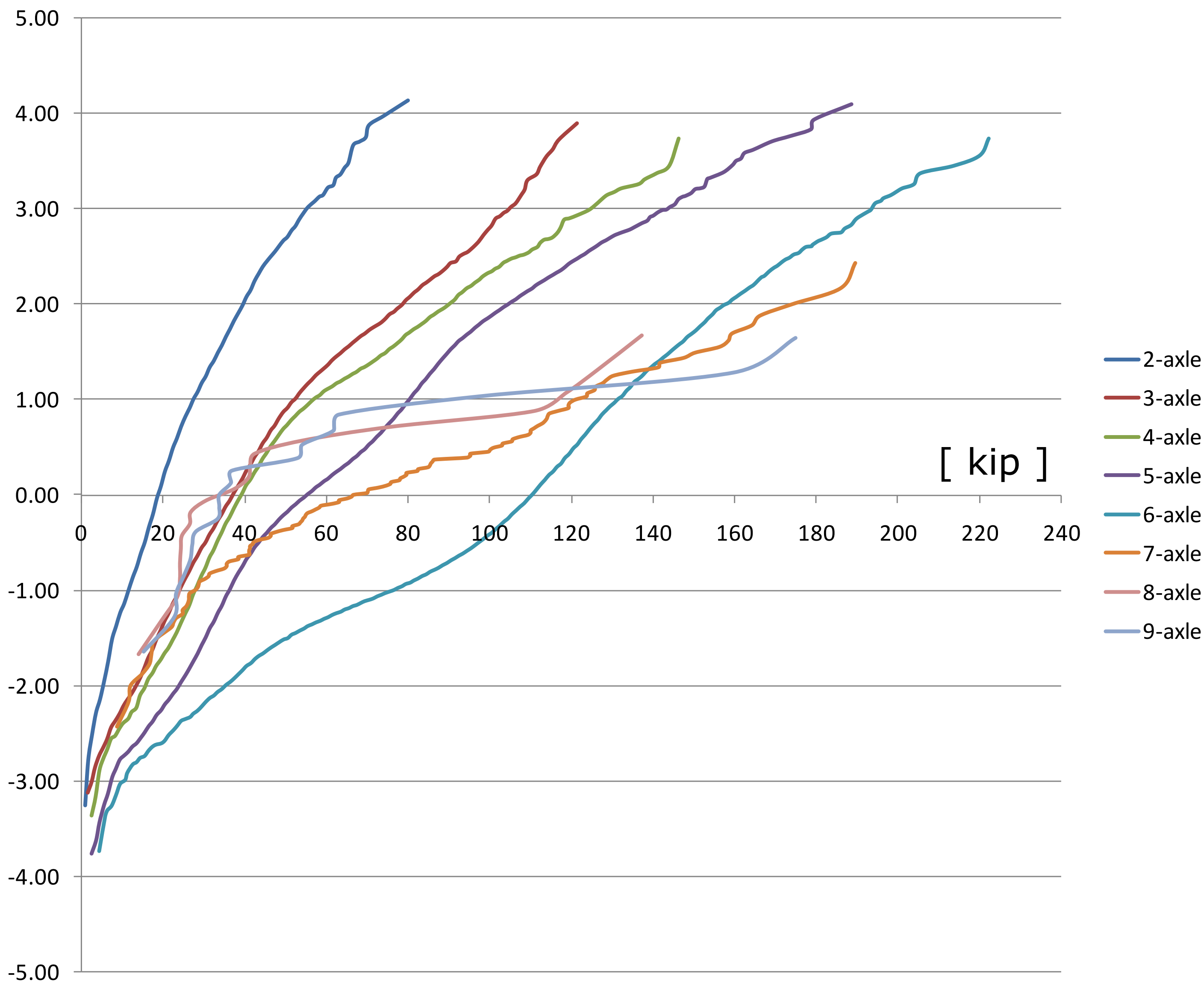


CDF's of UDL for different span lengths (kip/ft)
Oregon – US 97 Bend, 1st lane

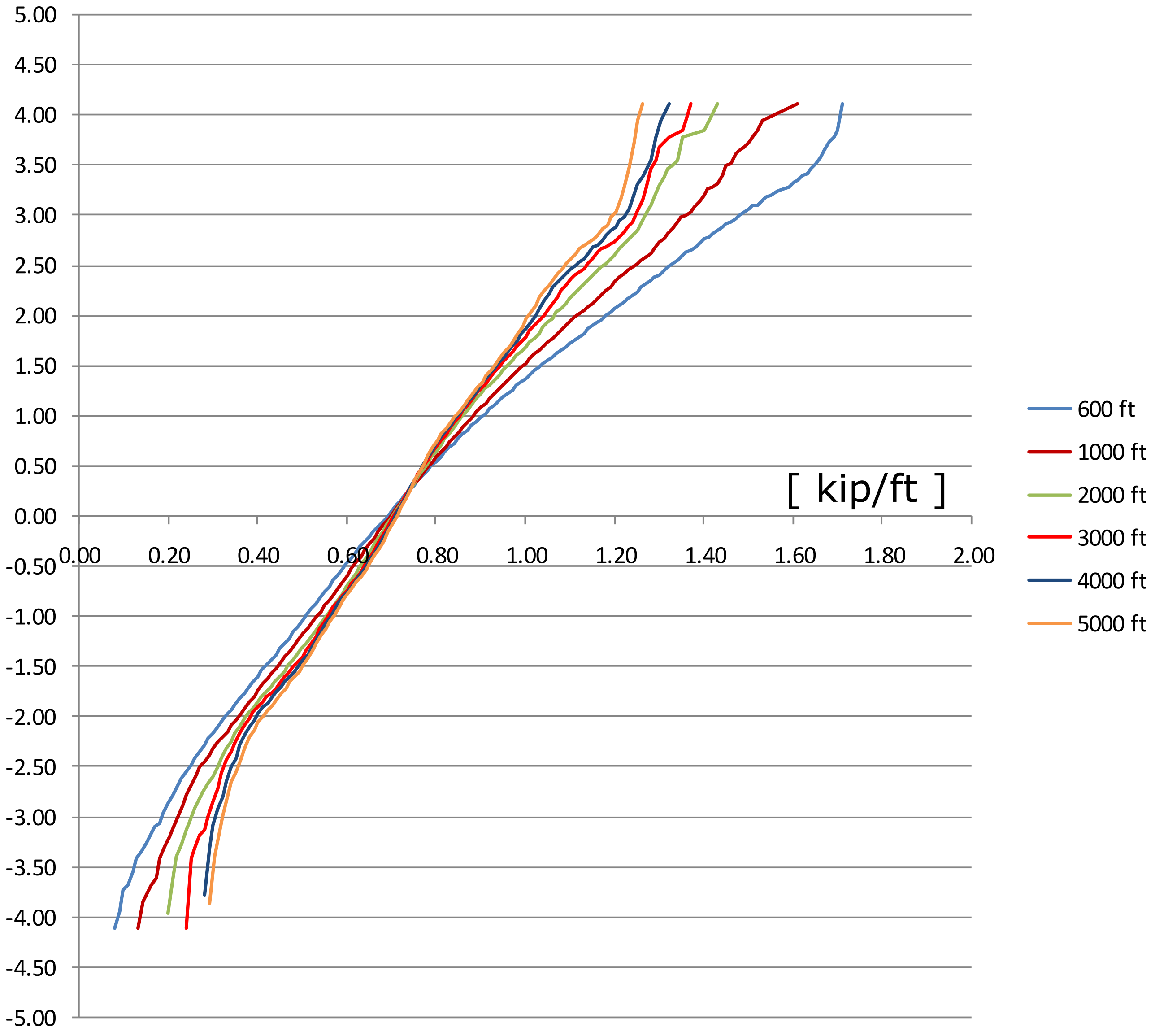


- On some bridges, 10-20% exceed GVW (permit trucks?)
- High ADTT > 3000 per lane
- The heaviest vehicles (6 axle trucks) > 110 kips.
- NYSDOT routine permit trucks that are legal up to 120 kips
- Often overloaded > 150 kips, occasionally above 200 kips (construction debris, gravel and garbage haulers).
- Special design live load should be proposed for those bridges.
- Throggs Neck Bridge NYC (I-495)

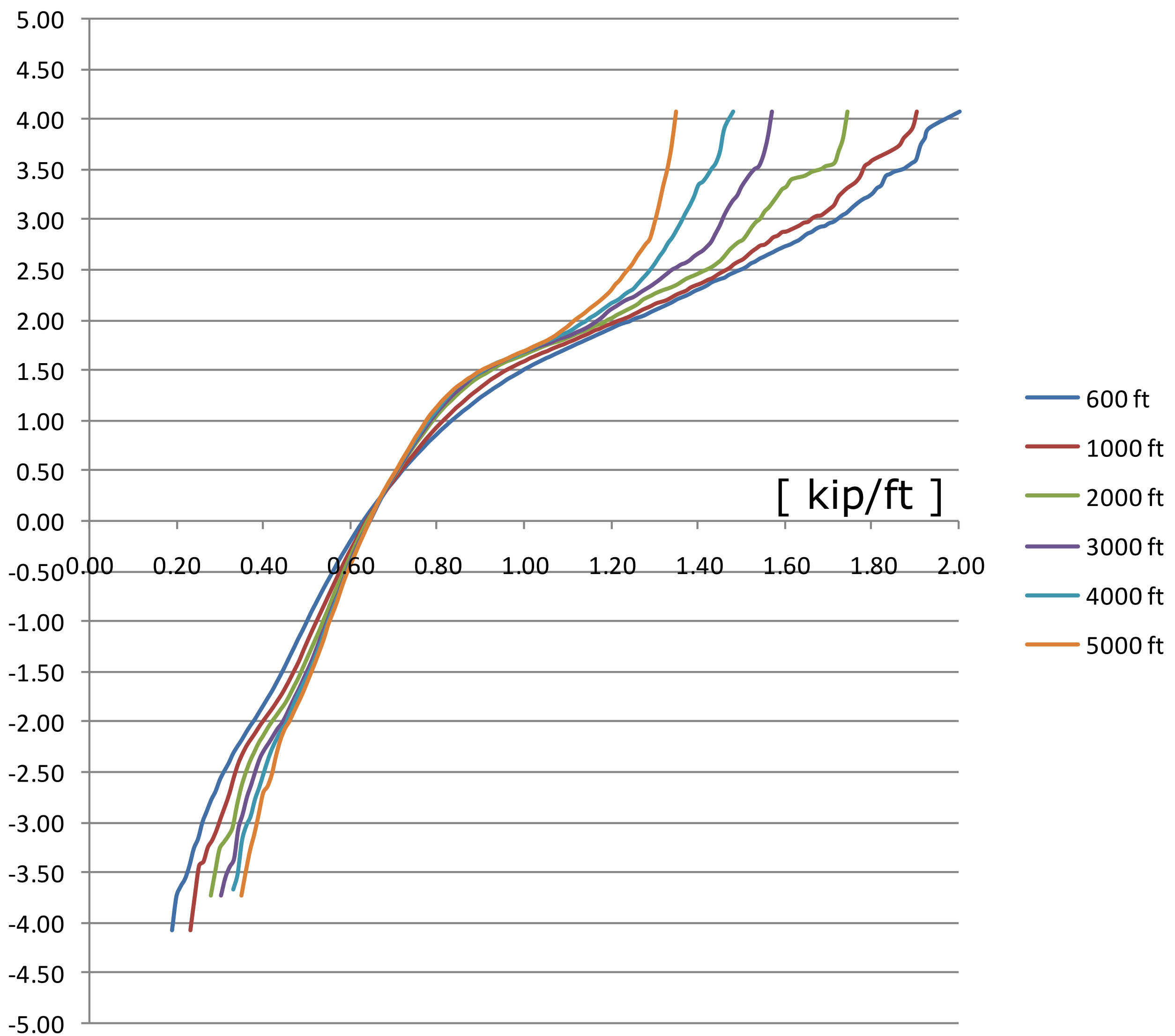
CDF's of GVW
New York I-496 EB, 1st lane



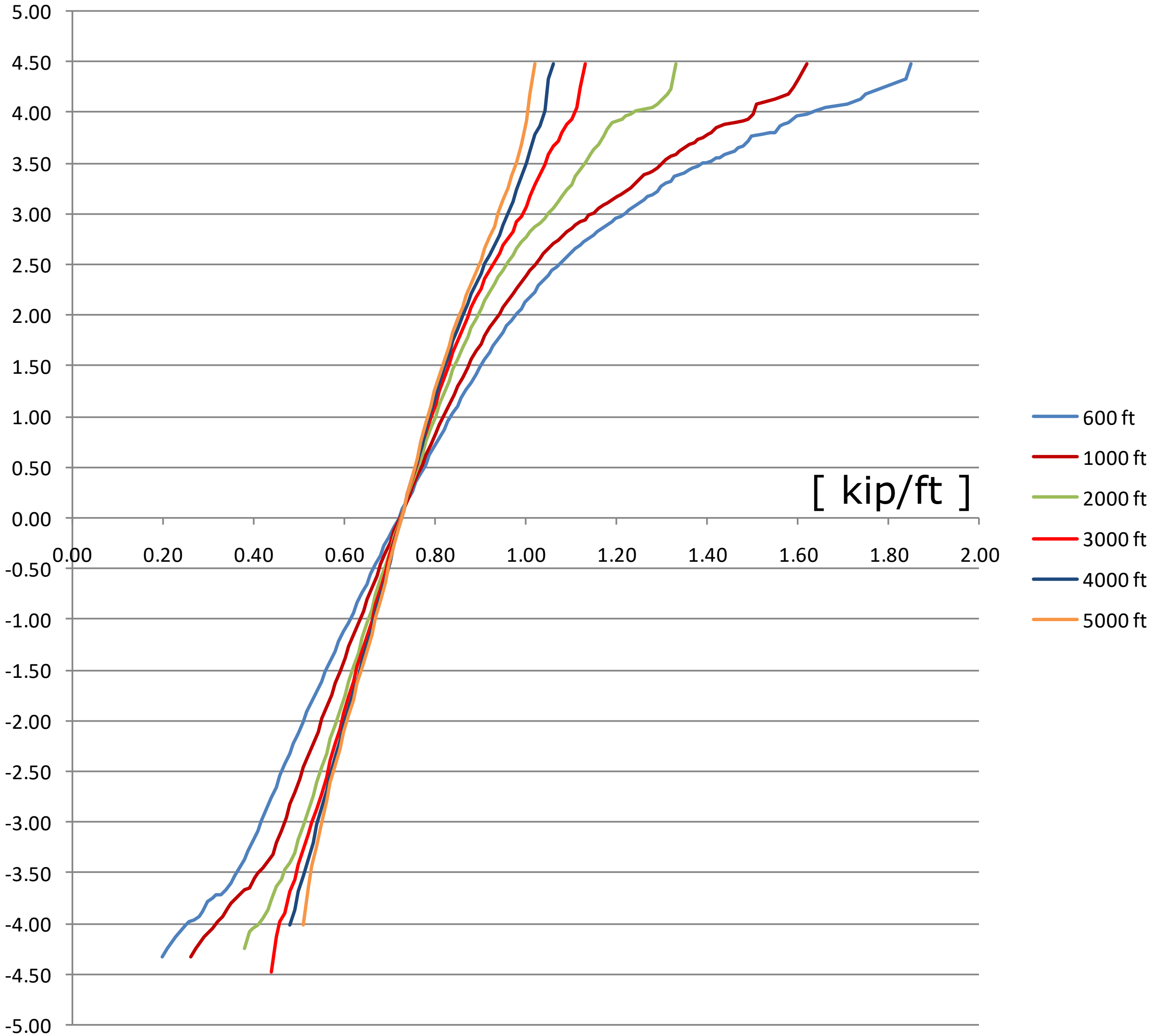
CDF's of UDL for different span lengths (kip/ft)
New York I-495 EB, 1st lane

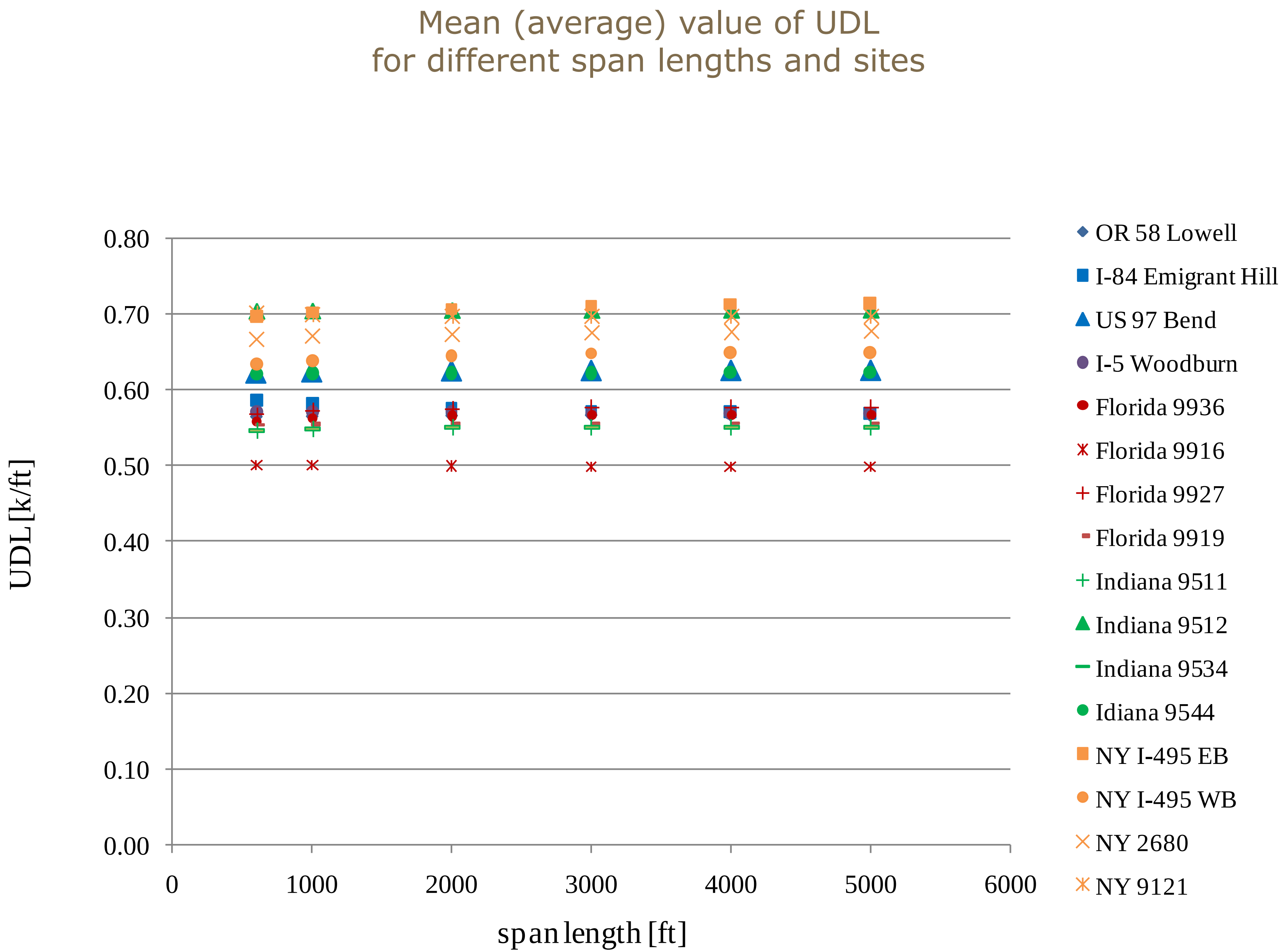


CDF's of UDL for different span lengths (kip/ft)
New York I-495 WB, 1st lane

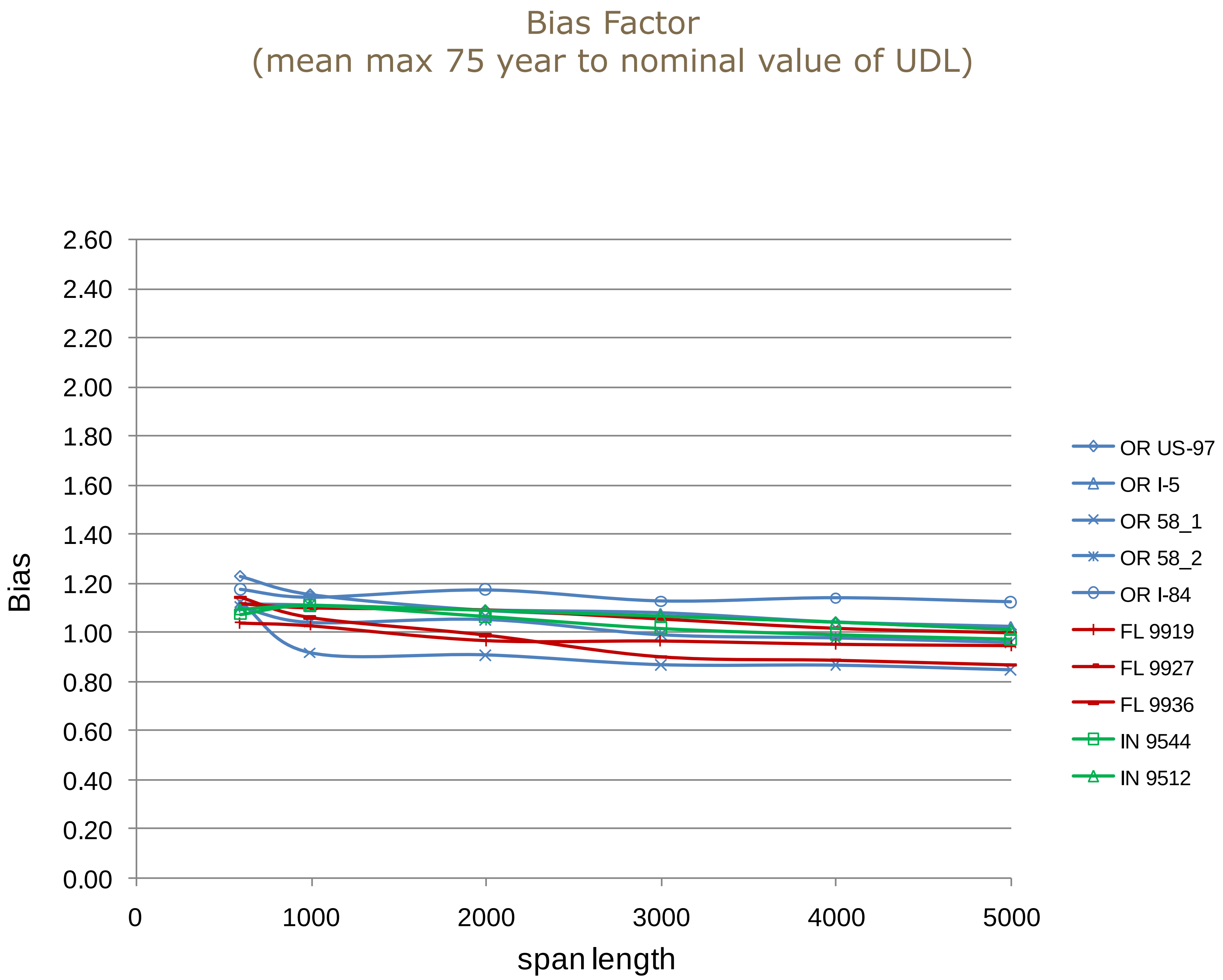


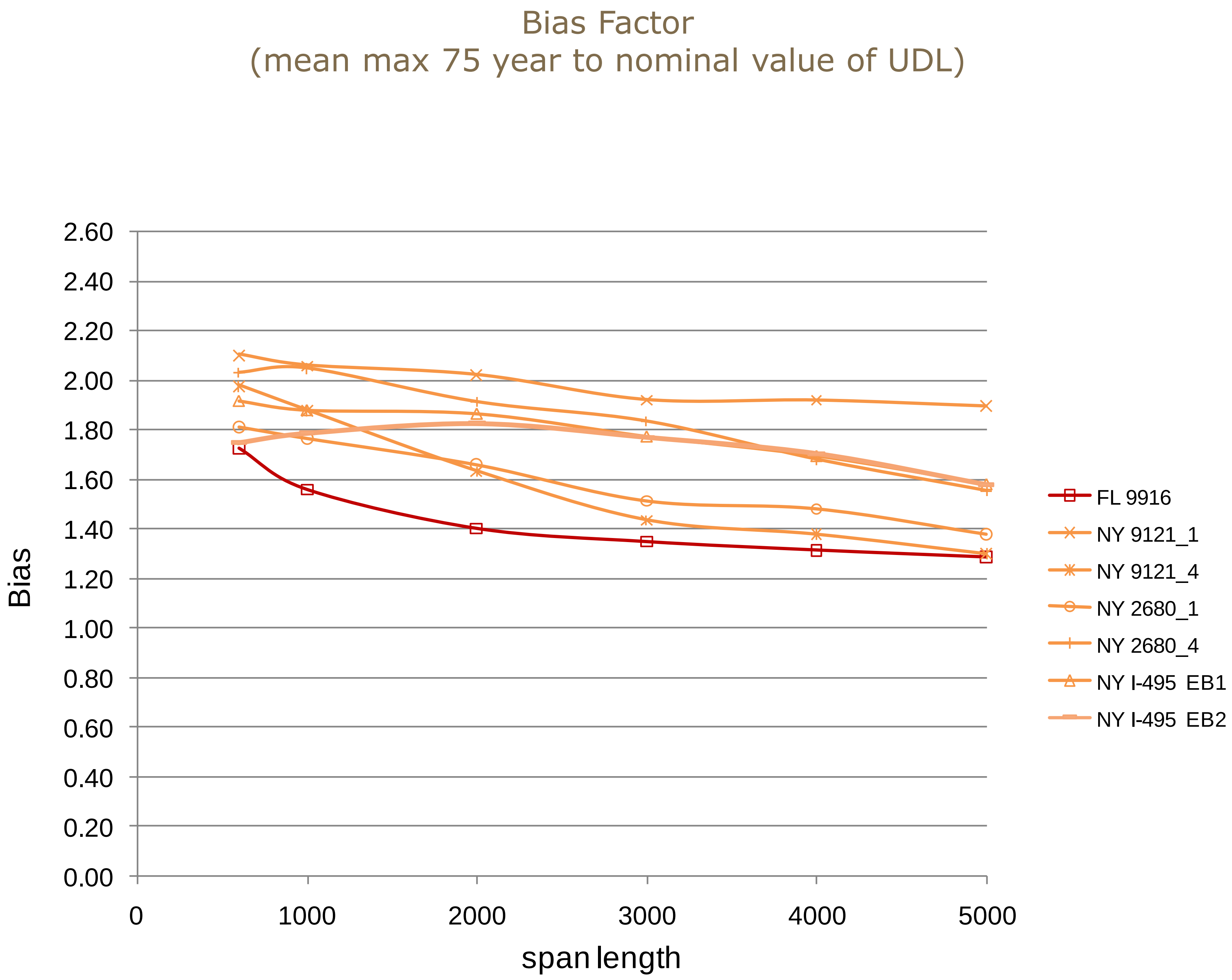
CDF's of UDL for different span lengths (kip/ft)
New York 9121, 4th lane





Mean (average) value of UDL
for different span lengths and sites
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
UDL [k/ft]
0
1000
2000
3000
4000
5000
6000
span length [ft]
OR 58 Lowell
I-84 Emigrant Hill
US 97 Bend
I-5 Woodburn
Florida 9936
Florida 9916
Florida 9927
Florida 9919
Indiana 9511
Indiana 9512
Indiana 9534
Idiana 9544
NY I-495 EB
NY I-495 WB
NY 2680
NY 9121


Bias Factor
(mean max 75 year to nominal value of UDL)
Bias
2.60
2.40
2.20
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0
1000
2000
3000
4000
5000
span length
OR US-97
OR I-5
OR 58_1
OR 58_2
OR I-84
FL 9919
FL 9927
FL 9936
IN 9544
IN 9512

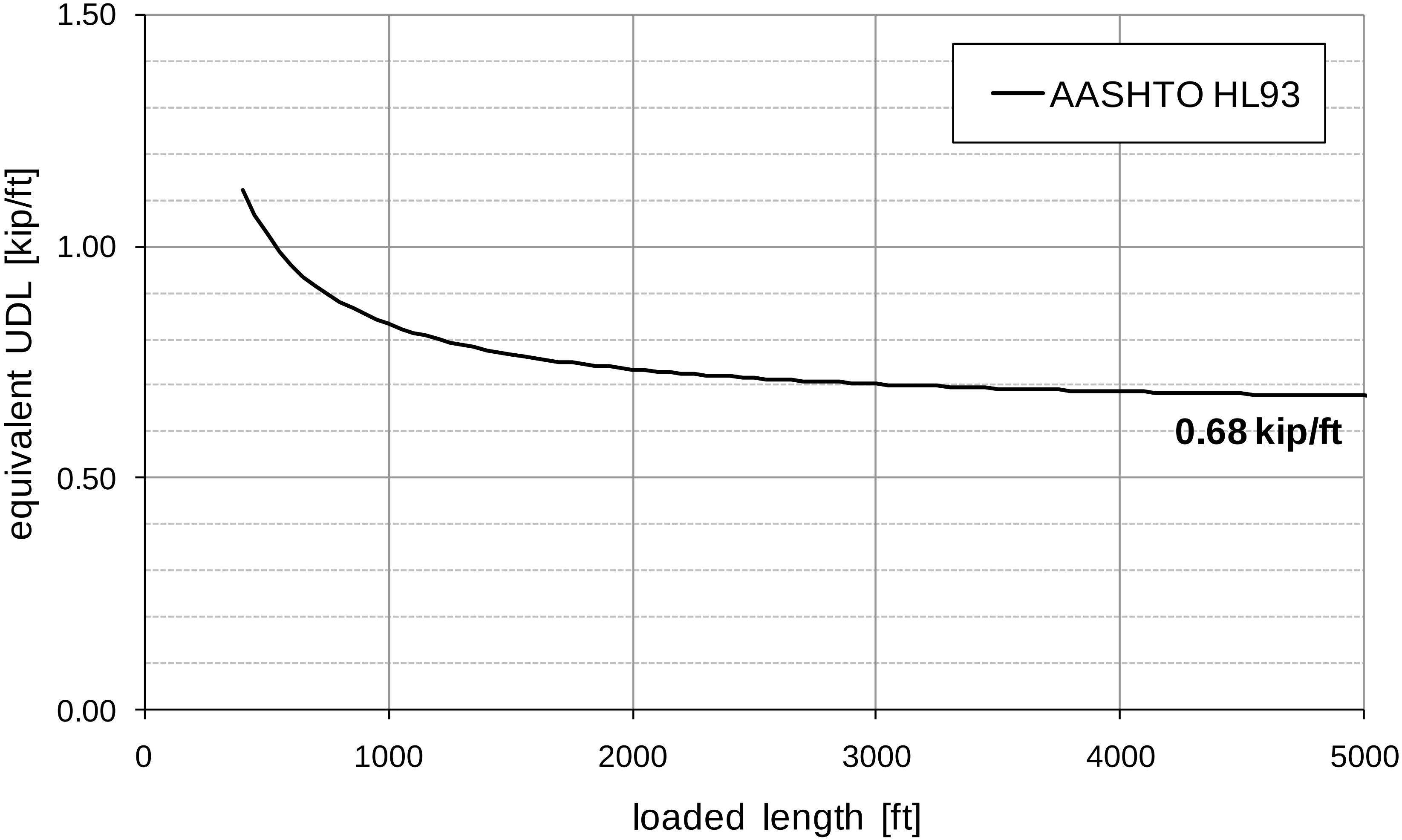


Bias Factor
(mean max 75 year to nominal value of UDL)
Bias
2.60
2.40
2.20
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0
1000
2000
3000
4000
5000
span length
FL 9916
NY 9121_1
NY 9121_4
NY 2680_1
NY 2680_4
NY I-495 EB1
NY I-495 EB2

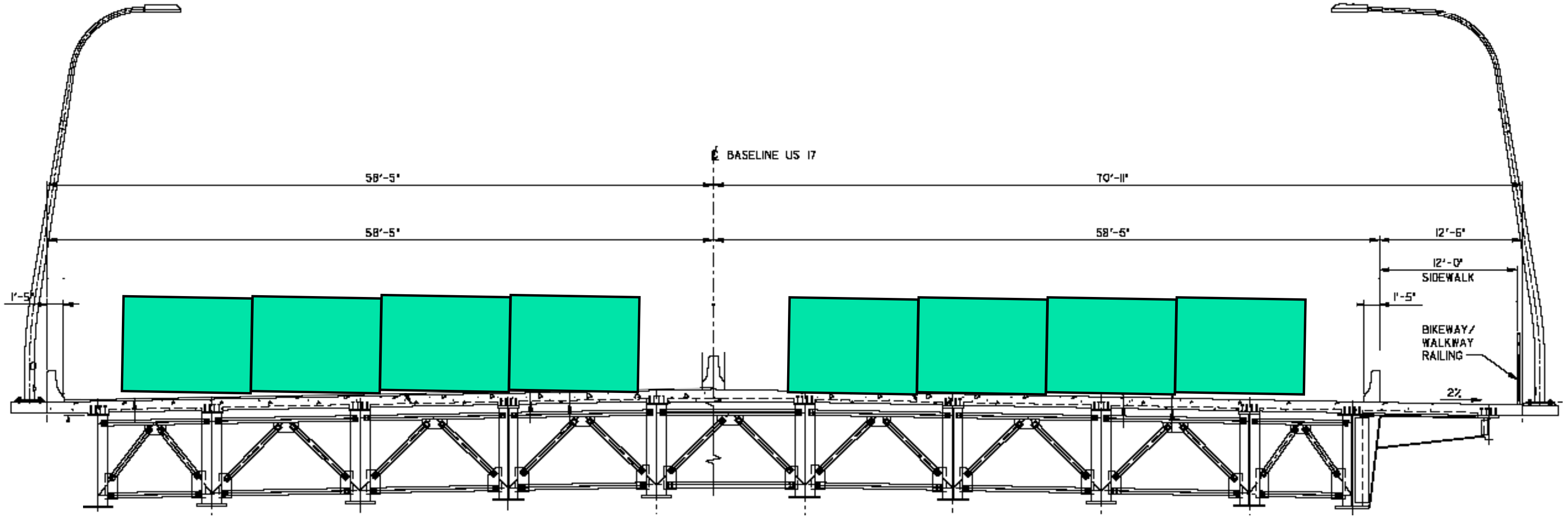
- For intermediate and long span bridges, with span longer than 600 ft
- For longer spans, the uniformly distributed load decreases and is closer to the mean value.
- This observation confirms that for a long loaded span, a single overloaded truck does not have any significant impact.
- It was noticed that the mean (average) value oscillated between 0.50 and 0.70 k/ft.

- Bias factor (the ratio of mean to nominal) was calculated for the heaviest 75-year combination of vehicles, for various WIM sites and a span length of 600 to 5000 ft.
- For most of the sites the bias factor < 1.25 which is similar to short and medium spans, as shown in the NCHRP Report 368 (1999).
- Therefore, it is recommended to use HL-93 for long spans.

Proposed Live Load for Long Span Bridges
with IM, w/o multilane factors

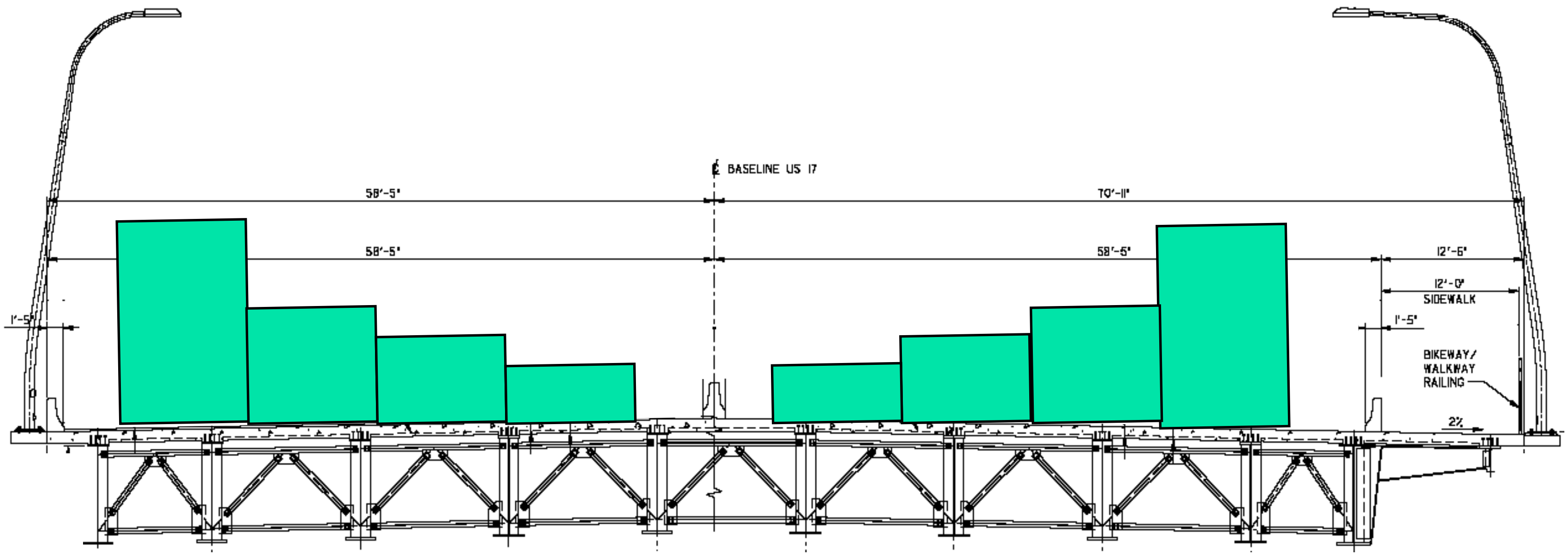


Multi-lane Load
AASHTO LRFD (2007)

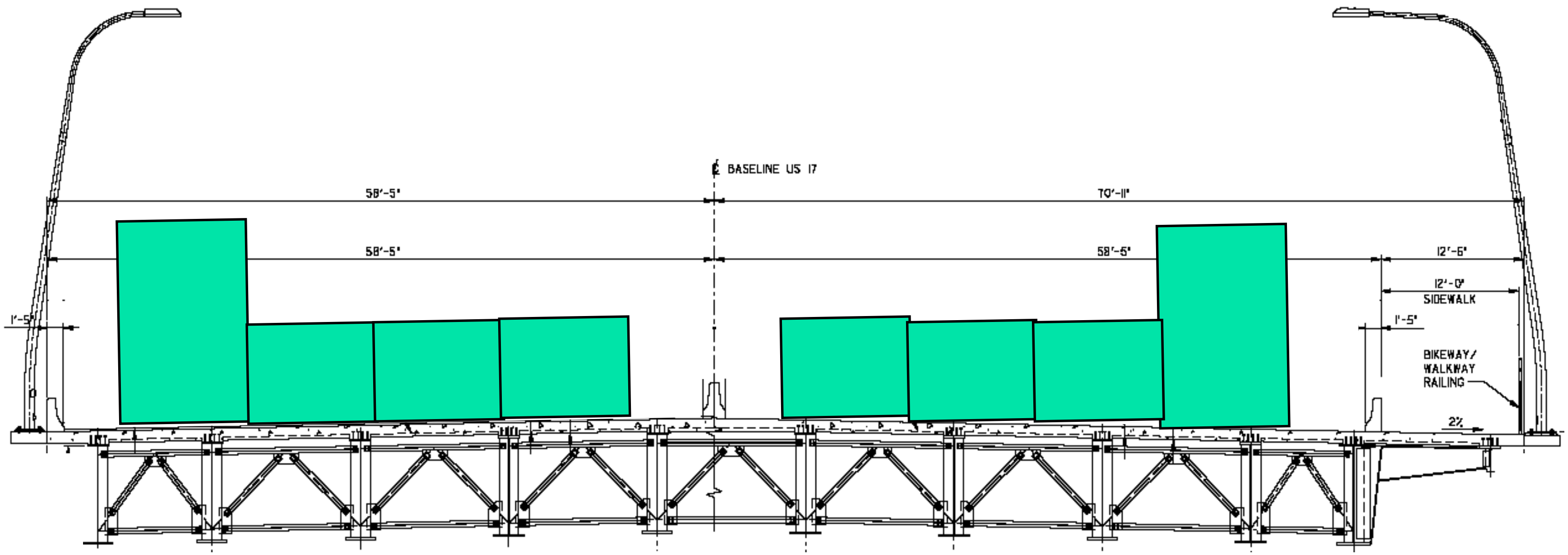


Multilane reduction factors are applied to all lanes

Multi-lane Load
Actual Observation



Multi-lane Load
Possible solution



Multilane Factors

- For bridges with multilanes in two directions, it is proposed to use the multilane factors separately for each direction.
- For multilane bridges, it is unlikely to have all lanes fully loaded simultaneously.
- It was observed on video recordings of traffic jam situations that, in most cases, trucks tended to use only one lane, with other lanes being either empty or loaded with passenger cars.
- However, in some other situations, it was observed that two or even three adjacent lanes could be occupied by trucks.
- The development of a statistical data base will require more field measurements. In the meantime, it is recommended to use the multilane factors as specified in AASHTO LRFD (2007), which is conservative.

Multilane Factors
Summary

Code	Number of Lanes					
	1	2	3	4	5	6 or more
AASHTO LRFD (2007)	1.20	1.00	0.85	0.65	0.65	0.65
OHBDC (1983, 1991)	1.00	0.90	0.80	0.70	0.60	0.55
CAN/CSA-S6-00 (2000)	1.00	0.90	0.80	0.70	0.60	0.55
ASCE (1981)	1.00	0.70	0.40	0.40	0.40	0.40

Dynamic Load due to Truck Loads for Long Span Bridges

- For short spans dynamic load factor is < 30%
- For long spans, live load is the result of multiple presence, many trucks
- Live load is small portion of total load, dynamic due to live load is even smaller
- For long span bridges vibration due to "wheel hop" on approach slab decreases
- Vibrations induced by multiple vehicles can balance each other, resulting in a smaller dynamic effect
- Critical load scenario is traffic jam, at crawling speed – no dynamic load

Dynamic Load for Short and Medium Spans

- AASHTO Standard
  - function of span, max 30% of live load
- AASHTO LRFD
  - 33% of truck load, 0% for uniform load
- Ontario OHBDC
  - 25% of live load
- Canadian CHBDC
  - 25% of live load

- Field measurements for short/medium spans indicate: IM<20% for a single truck and IM<10% for two trucks

- It is proposed to apply the dynamic load factor of 1.33 to design truck only (as specified in AASHTO LRFD 2007)

- It is recommended to use HL-93 for intermediate and long span bridges with spans longer then 600 ft.
- It is proposed to use the multilane factors as specified in AASHTO LRFD (2007), which is conservative.
- For bridges with multilane in two directions, it is proposed to use the multilane factors separately for each direction.
- It is proposed to apply the dynamic load factor of 1.33 applied to the design truck only (as specified in AASHTO LRFD 2007)