3.6. Dead Load Effects

The dead load effects on the pier cap beam and columns were obtained by first applying the self-weight of the superstructure plus the 1.0 kN/m^2 allowance for construction tolerances and construction methods as uniformly distributed loads on the superstructure [Maday, 2002]. The 1.0 kN/m^2 allowance was turned into a uniformly distributed load by multiplying it by the width of the superstructure, which is 25.1 m. This analysis didn't produce accurate dead load effects on the pier cap beam and the columns, because the rigid link only connected the midpoint of the superstructure to the middle column, and therefore it produced erroneously high axial loads on the middle column and erroneously low axial loads on the leftmost and rightmost columns. So to get more accurate results, the axial load on the rigid link due to the self-weight of the superstructure and the 1.0 kN/m^2 allowance was divided by ten, which was the number of prestressed concrete girders. Then an analysis was performed on the pier structure, in which the pier was subjected to ten point loads on the pier cap beam, each representing a girder that sits on the pier cap beam, plus the self-weight of the pier cap beam and the columns. This is shown in Figure 3.10.

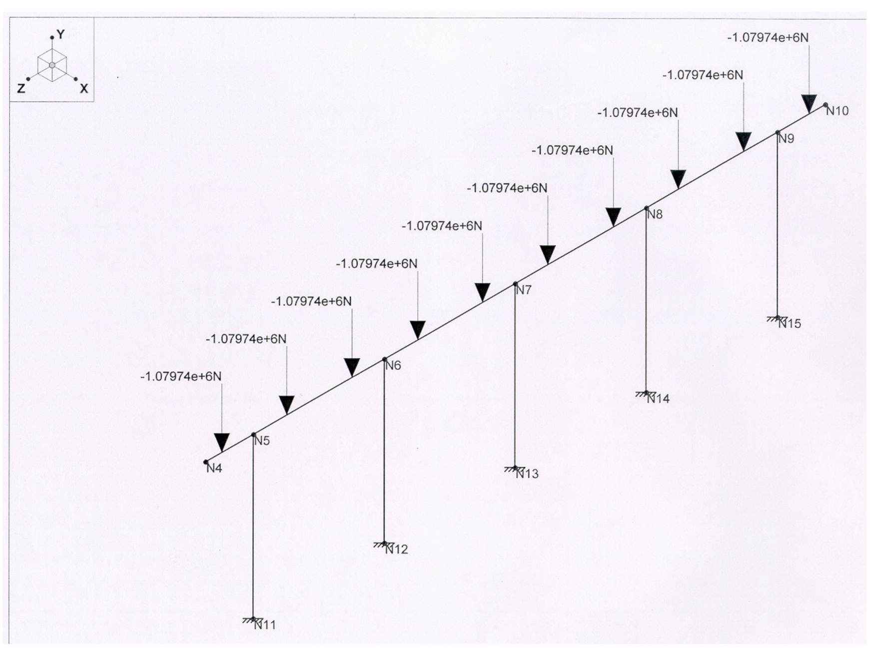

Figure 3.10. The loading on the pier performed to determine more accurate dead load effects on the pier cap beam and columns.

3.7. Live Load Effects

The live load effects were found by adding the maximum effects from the three moving live load cases to the lane load effects. The three moving live load cases and the lane load are shown below in Figure 3.11. The lane load is a 9.3-N/mm distributed load [Barker and Puckett, 1997]. The case that always controlled was the third, which was the two-truck case. Each of these moving live load cases was run along the superstructure, and the largest axial load produced on the rigid link was used to run an analysis on the pier similar to that for the dead loads. The same procedure was also used to determine the lane load effects.

The maximum effects of the three moving load cases, which was always the two-truck case for this bridge, were combined with the lane load effects by using the

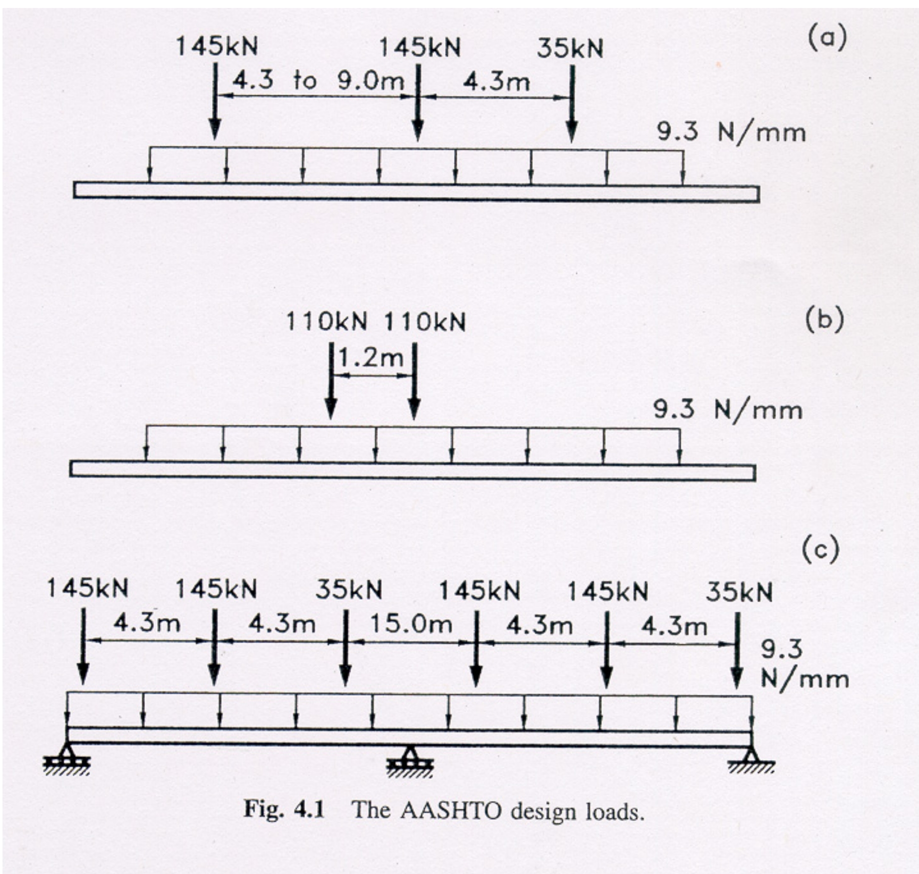

Figure 3.11. The three moving live load cases and the lane load [Barker and Puckett, 1997]. Reprinted by permission of John Wiley & Sons, Inc.

multiple presence factors (m), the dynamic load allowance (IM), and the 0.9 factor, since the controlling case was always the two-truck case [Barker and Puckett, 1997]. The multiple presence factors and the dynamic load allowance are shown below in Table 3.2 and Table 3.3, respectively.

Table 3.2. The Multiple Presence Factors [Barker and Puckett, 1997].]. From *AASHTO LRFD Bridge Design Specifications, 1st Edition*, Copyright 1994, by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission. Reprinted by permission of John Wiley & Sons, Inc.

Number of Design Lanes	Multiple Presence Factors (m)
1	1.20
2	1.00
3	0.85
More than 3	0.65

Table 3.3. The Dynamic Load Allowance [Barker and Puckett, 1997]. From *AASHTO LRFD Bridge Design Specifications, 1st Edition*, Copyright 1994, by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission. Reprinted by permission of John Wiley & Sons, Inc.

Component	IM (%)
Deck Joints-all limit states	75
All other components	
Fatigue fracture limit states	15
All other limit states	33

Since this bridge has three lanes, m = 0.85. And since deck joints and fatigue were not the subject of interest in this analysis, IM = 0.33. Thus the formula to calculate the live load effects of this bridge was

$LL = 0.85\times3\times(1.33\times0.9\times TT + 0.9\times LN)$

TT = two-truck load effects

LN = lane load effects

3.8. Combined Dead and Live Load Effects on the Columns

The dead and live load effects were combined with the load factors from Table 3.5-1 of the new LRFD Guidelines, which was presented as Table 3.4 in this report. Since earthquake loading was a significant part of this study, Extreme Event-I from Table 3.4 of this report was chosen. Suggested values for γ_{EQ} are 0.0, 0.5 and 1.0 [Barker and Puckett, 1997]. $\gamma_{EQ} = 0.5$ was chosen, which means there is no traffic jam on the bridge when the earthquake happens. Thus the combined effects of the dead load and live load are

$P = DL + (0.5 \times LL)$

P = combined dead load and live load effects

DL = dead load effects

LL = live load effects

The complete results of the dead load and live load effects are presented in Appendix IV.

Table 3.4. Load Combinations and Load Factors [MCEER/ATC, 2002].

Table 3.5-1 Load Combinations and Load Factors

Load Combination / Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time: EQ	IC	CT	CV
STRENGTH-I (unless noted)	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV EH, EV, ES, DW DC ONLY	γ_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT-I	1.00	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME EVENT-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

3.9. Determination of the Required Seismic Design and Analysis Procedure (SDAP) and Seismic Design Requirement (SDR)

In order to determine the required Seismic Design and Analysis Procedure (SDAP) and Seismic Design Requirement (SDR) for this bridge, first the following parameters have to be determined:

S_s = 0.2-second period spectral acceleration, obtained from the USGS website zip code lookup for spectral accelerations at the location of the bridge
S_1 = 1-second period spectral acceleration, obtained from the USGS website zip code lookup for spectral accelerations at the location of the bridge
F_a = site coefficients for the short-period range, which are given in Table 3.5
F_v = site coefficients for the long-period range, which are given in Table 3.6

The 0.2-second and 1-second period spectral acceleration maps are based on a probability of exceedance of 2% in 50 years, but all the analyses in this study were performed to investigate if the bridges could endure a maximum considered earthquake, which has a probability of exceedance of 3% in 75 years. Like in Chapter 2, the return periods for both probabilities of exceedance had to be computed to prove that they are approximately equivalent.

For the probability of exceedance of 2% in 50 years, the return period is

$$RP = \frac{-50}{\ln(1-0.02)} = 2475\,years$$

For the probability of exceedance of 3% in 75 years, the return period is

$$RP = \frac{-75}{\ln(1-0.03)} = 2462\,years$$

[Charney, 2001]